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SHELF-ANGLE & BRICK LEDGE DESIGN FOR BRICK VENEER ON MID-RISE WOOD-FRAME BUILDINGS

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Abstract

Masonry veneer is an excellent addition to any wood-frame buildings, especially in mid-rise wood-frame buildings, where current building codes require a non-combustible cladding, like masonry.

Shelf angle design for masonry veneer is an important consideration when supporting full bed masonry veneer. Shelf angles are typically used to support masonry at floor level. Shelf angles can also be used at the foundation level especially when cavity insulation is desired to provide continuous insulation between above grade and below grade walls. As an alternative to shelf angles, brick ledges supporting up to 11 m of brick veneer can be used and may reduce the stud size for the load-bearing exterior wood stud walls by transferring the dead load from the first 3 storeys of brick directly to the foundation.

This technical aid focuses on the design of the support of brick veneer without cavity insulation for a mid rise wood-frame building where 30 feet (9.14 m) of masonry veneer is supported on a shelf angle or brick ledge at grade and a shelf angle at floor level supporting up to 3.05 m (10 feet) of masonry is designed for subsequent floors.

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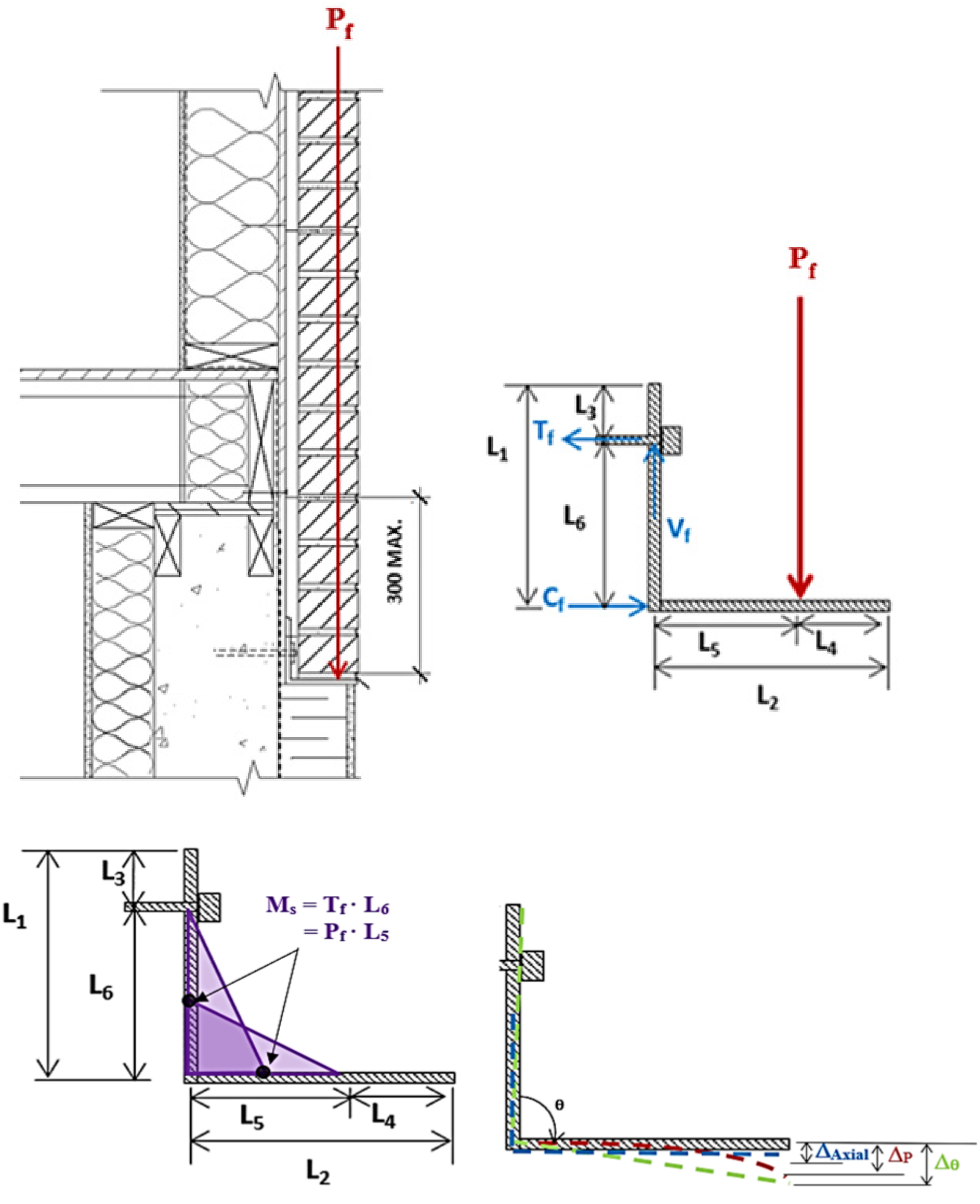
Figure 1: Shelf Angle Design - Foundation



L1 = Vertical Leg length (m)
L2 = Horizontal Leg length (m)
L3 = spacing of bolt hole (typically 25 mm (1”) from top of leg vertical leg
L4 = centroid of brick veneer (typically 45 mm for metric modular 90 mm brick)
L5 = eccentricity of the veneer load = Airspace + brick veneer thickness /2 (25 + 45 mm)
L6 = L1 - L3

Bolt Forces (From Statics)

$T_f = \frac{P_f \cdot L_5}{L_6}$ => Tensile loads on the bolts per metre length of the wall.

$C_f = \frac{P_f \cdot L_5}{L_6}$ => Compression load on the concrete wall or wood rim board per metre length of the wall.

$V_f = P_f$ => Shear load on the bolts per meter length of wall and max shear on the shelf angle.

$M_f = P_f \cdot L_5$ => Maximum moment on the shelf angle per metre length of the wall.

Shelf Angle Design - At Foundation

Assumptions:

- 90 mm thick brick veneer
- L102x102x13 (L4" x 4" x ½") vertical shelf angle anchored into the concrete foundation wall
- 16 mm (5/8") Hilti Kwik Bolt 3 expansion anchor bolts at 406 mm (16") o.c.
- Anchor bolts into a 20 MPa concrete foundation wall
- Veneer supported to max 9.144 m (30')

Design Forces:

$$P_s = 1.0 \cdot (DL_{brick} + DL_{shelfangle}) = 1.0 \cdot (20.1\ kN/m^3 \cdot 0.090\ m \cdot 9.144\ m + 0.25\ kN/m) = \mathbf{16.7\ kN/m}$$
$$P_f = 1.4 \cdot (DL_{brick} + DL_{shelfangle}) = 1.4 \cdot (20.1\ kN/m^3 \cdot 0.090\ m \cdot 9.144\ m + 0.25\ kN/m) = \mathbf{23.4\ kN/m}$$

$$T_f = \frac{23.4\ kN/m \cdot (25.4\ mm + 45\ mm)}{(101.6\ mm - 25.4\ mm)} = \mathbf{21.7\ kN/m}$$

$$C_f = T_f = \mathbf{21.7\ kN/m}$$

$$M_f = P_f \cdot L_5 = 23.4\ kN/m \cdot 0.0704\ m = \mathbf{1.65\ kN\text{-}m\ /m}$$
$$V_f = P_f = \mathbf{23.4\ kN/m}$$

$$M_s = P_s \cdot L_5 = 16.7\ kN/m \cdot 0.0704\ m = \mathbf{1.18\ kN\text{-}m\ /m}$$

i) Shelf Angle Design for Bending, Shear, and Deflection

$L_1 = 101.6$ mm $\quad L_2 = 101.6$ mm $\quad L_3 = 25.4$ mm
$L_4 = 45$ mm $\quad L_5 = 70.4$ mm $\quad L_6 = 114.3$ mm

$t = 12.7$ mm
$b = 1000$ mm per metre of veneer
$E_s = 200{,}000$ MPa
$I_x = b \cdot t^3/12 = 1000\ mm \cdot (12.7\ mm)^3/12 = 170{,}699\ mm^4$
$Sx = b \cdot t^2/6 = 1000\ mm \cdot (12.7\ mm)^2/6 = 26{,}882\ mm^3$
$A = b \cdot t = 1000\ mm \cdot 12.7\ mm = 12{,}700\ mm^2$
$F_y = 345$ MPa

$F_s = 0.67\ F_y = 231$ MPa

$M_r = \phi \cdot F_y \cdot S_x \times 10^{-6} = 0.90 \cdot 345\ \text{MPa} \cdot 26{,}882\ \text{mm3} \times 10^{-6} = \mathbf{8.35\ kN\text{-}m/m}$
> 1.68 kN-m/m => **OK**

$V_r = 0.5 \cdot \phi \cdot F_s\ b \cdot t \times 10^{-3} = 0.5 \cdot 0.90 \cdot 231\ \text{MPa} \cdot 1000\ \text{mm} \cdot 12.7\ \text{mm} \times 10^{-3} = \mathbf{1{,}321\ kN/m}$
> 23.9 kN/m => **OK**

$$\Delta = \Delta_P + \Delta_\theta + \Delta_{Axial}$$
$$= (P_s \cdot L_5^2 / 6\ E_s\ I_x) \cdot (3L_2 - L_5) + (M_s \cdot L_6 / 3\ E_s \cdot I_x) \cdot L_2 + (P_s \cdot L_1 / A \cdot E_s)$$
$$= (16{,}800\ \text{N} \cdot (70.4\ \text{mm})^2 / (6 \cdot 200{,}000\ \text{N/mm}^2 \cdot 170{,}699\ \text{mm}^4) \cdot (3 \cdot 101.6\ \text{mm} - 70.4\ \text{mm})$$
$$+ (1.18 \times 10^6\ \text{N-mm} \cdot 114.3\ \text{mm} / 3 \cdot 200{,}000\ \text{N/mm}^2 \cdot 170{,}699\ \text{mm}^4) \cdot 101.6\ \text{mm}$$
$$+ (16{,}800\text{N} \cdot 152.4\ \text{mm}) / (12{,}700\ \text{mm}^2 \cdot 200{,}000\ \text{N/mm}^2)$$
$$= \mathbf{0.195\ mm}$$
$$< L_2 / 480 = 101.6\ \text{mm} / 480 = 0.212\ \text{mm} \quad \Rightarrow \textbf{OK}$$

ii) Design of Concrete Foundation to Resist Compression

$f'_c =$	20	MPa	
$\phi_c =$	0.65		
$\alpha_1 = 0.85 - 0.0015 \cdot f'c = 0.85 - 0.0015 \cdot (20)\ =$	0.82		
$\beta_1 = 0.97 - 0.0025 \cdot f'c = 0.97 - 0.0025 \cdot (20)\ =$	0.92		
$b =$	1000	mm	
$c =$	55.2	mm	
$a = \beta_1 c =$	50.8	mm	
$C_r = \alpha_1\ \phi_c\ f'_c\ b\ a =$	**542**	**kN/m**	**OK**

iii) Design of Post Installed Concrete Anchor Bolts to Resist Shear & Withdrawal

Post Installed Anchor Bolt Resistance

Bolt Type **ϕ5/8" x 4- ½" Hilti Kwik Bolt 3 Embed 4" (102 mm)**

20 MPa concrete from Hilti

$(T_r)_{bolt}$	**13.4**	kN/bolt	
$(V_r)_{bolt}$	**17.4**	kN/bolt	
bolt spacing	**406.4**	mm o.c.	
n_{bolts}	**2.46**		bolts per metre
T_r	**33.0**	kN/m	**OK**
V_r	**42.8**	kN/m	**OK**
$(T_f/T_r)^{5/3}+(V_f/V_r)^{5/3} \leq 1$	**0.90**		**OK**

Shelf Angle Design - At Floor Level

Assumptions:

- 90 mm thick brick veneer.
- Veneer supported at each floor - max 3.658 m (12').

THROUGH BOLT

• 2-ply S-P-F 2x12 rim board c/w ½" dia. (13 mm dia) A307 through bolts at 16" (406 mm) o.c.
= 2.46 bolts/m

• L102x102x9.5 (L4"x4"x3/8") shelf angle anchored into the wood rim board

LAG SCREW

• 3-ply SPF 2x12 rim board c/w 5/8" dia. (16 mm dia.) lag screws at 203 mm (8") o.c.
=> 4.92 lag screws/m

Minimum length of penetration into rim board for lag screws is 3-3/4" as per
CSA-O86-2014[1] Clause 12.6.3.3

• L102x102x9.5 (102"x102"x3/8") shelf angle anchored into the wood rim board

Typical Design - Through Bolt with 2-ply SPF 2x12 Wood Rim Board

Design Forces (From Figure 2):

$$P_s = 1.0 \cdot (DL_{brick} + DL_{shelfangle}) = 1.0 \cdot (20.1\ \text{kN/m}^3 \cdot 0.090\ \text{m} \cdot 3.658\ \text{m} + 0.15\ \text{kN/m}) = \mathbf{6.77\ kN/m}$$

$$P_f = 1.4 \cdot (DL_{brick} + DL_{shelfangle}) = 1.4 \cdot (20.1\ \text{kN/m}^3 \cdot 0.090\ \text{m} \cdot 3.658\ \text{m} + 0.15\ \text{kN/m}) = \mathbf{9.47\ kN/m}$$

$$T_f = \frac{9.47\ \text{kN/m} \cdot (25.4 + 45\ \text{mm})}{(101.6\ \text{mm} - 25.4\ \text{mm})} = \mathbf{8.75\ kN/m}$$

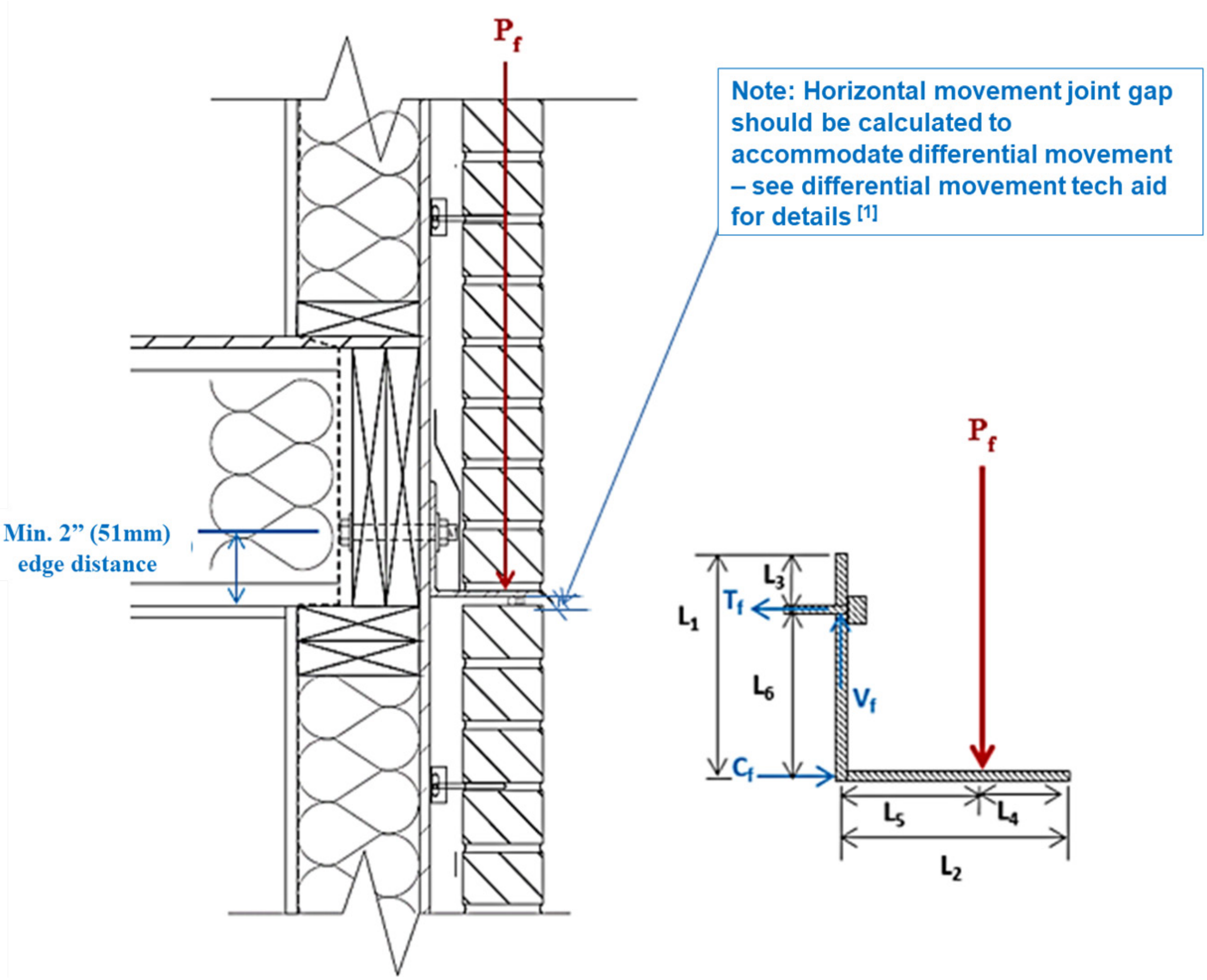
$$C_f = T_f = \mathbf{8.75\ kN/m}$$

$$M_f = P_f \cdot L_5 = 9.47\ \text{kN/m} \cdot (0.0704\ m) = \mathbf{0.667\ kN\text{-}m\ /m}$$

$$V_f = P_f = \mathbf{9.47\ kN/m}$$

$$M_s = P_s \cdot L_5 = 6.77\ \text{kN/m} \cdot (0.0704\ m) = \mathbf{0.476\ kN\text{-}m\ /m}$$

Figure 2: Shelf Angle Design Floor Level – 2-Ply SPF 2x12 Wood Rim Board



I) Steel Shelf Angle Design for Bending, Shear, and Deflection

$L_1 = 101.6$ mm	$L_2 = 101.6$ mm	$L_3 = 25.4$ mm
$L_4 = 45.0$ mm	$L_5 = 70.4$ mm	$L_6 = 76.2$ mm

$t = 9.525$ mm
$b = 1000$ mm per metre of veneer
$E_s = 200{,}000$ MPa
$I_x = b \cdot t^3/12 = (1000 \text{ mm}) \cdot (9.525 \text{ mm})^3/12 = \mathbf{72{,}013\ mm^4}$
$Sx = b \cdot t^2/6 = (1000 \text{ mm}) \cdot (9.525 \text{ mm})^2/6 = \mathbf{15{,}121\ mm^3}$
$A = b \cdot t = (1000 \text{ mm}) \cdot (9.525 \text{ mm})^2/6 = \mathbf{9{,}525\ mm^2}$

$M_r = \phi \cdot F_y \cdot S_x \text{ x } 10^{-6} = 0.90 \cdot 345 \text{ MPa} \cdot 15{,}121 \text{ mm}^4 \text{ x } 10^{-6} = \mathbf{4.7\ kN\text{-}m\ /m}$
$> M_f = 0.47$ kNm/m => **OK**

$V_r = 0.5 \cdot \phi \cdot F_u\ b \cdot t \text{ x } 10^{-3} = 0.5 \cdot 0.90 \cdot 0.67 \cdot 345 \text{ MPa} \cdot 1000 \text{ mm} \cdot 9.525 \text{ mm x } 10^{-3} = \mathbf{991\ kN/m}$
$> V_f = 9.47$ kN/m => **OK**

$\Delta = \Delta_P + \Delta_\theta + \Delta_{Axial}$

$= (P_s \cdot L_5^2 / 6\, E_s\, I_x) \cdot (3L_2 - L_5) + (M_s \cdot L_6 / 3\, E_s \cdot I_x) \cdot L_2 + (P_s \cdot L_1 / A \cdot E_s)$

$= (6{,}770\ \text{N} \cdot (70.4\ \text{mm})^2 / (6 \cdot 200{,}000\ \text{N/mm}^2 \cdot 72{,}013\ \text{mm}^4) \cdot (3 \cdot 101.6\ \text{mm} - 70.4\ \text{mm})$

$+ (0.476 \times 10^6\ \text{N-mm} \cdot 76.2\ \text{mm}) / (3 \cdot 200{,}000\ \text{N/mm}^2 \cdot 72{,}013\ \text{mm}^4) \cdot 101.6\ \text{mm}$

$+ (6{,}770\ \text{N} \cdot (101.6\ \text{mm}) / (9{,}525\ \text{mm}^2 \cdot 200{,}000\ \text{N/mm}^2)$

$= \mathbf{0.177\ mm}$

$< L_2 / 480 = 101.6\ \text{mm} / 480 = 0.212\ \text{mm}$ => **OK**

II) Rim Board Connection and Bearing / Rotation Design

From Load analysis the design loads on the fasteners are:

Tf = tension force on through bolts per metre length of veneer = **8.75 kN/m**

Vf = shear force on through bolts per metre length of veneer = **9.47 kN/m**

i) Steel Through-Bolt Design to Resist Tension and Shear

$T_r = 0.75 \cdot \phi_b \cdot n_b \cdot A_b \cdot F_u \times 10^{-3} = 0.75 \cdot 0.67 \cdot 2.46 \cdot 414\ \text{MPa} \cdot [\pi (12.7)^2 / 4\ ]\text{mm}^2 \times 10^{-3} = \mathbf{64.8\ kN/m}$

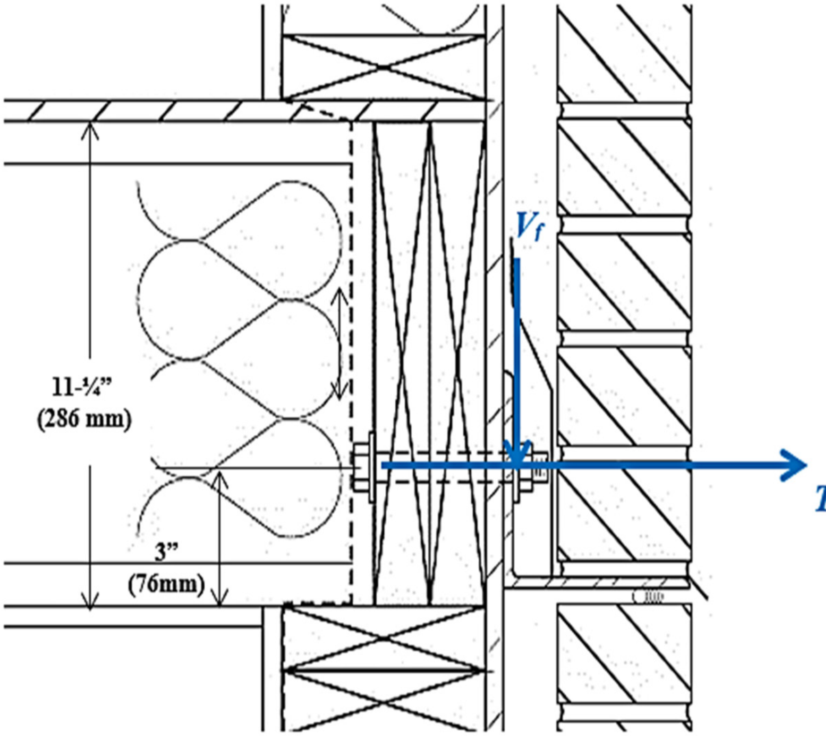
> 6.0 kN/m **OK**

$V_r = 0.60 \cdot \phi_b \cdot m \cdot n_b \cdot A_b \cdot F_u \times 10^{-3} = 0.6 \cdot 0.67 \cdot 1 \cdot 2.46 \cdot 414\ \text{MPa} \cdot [\pi (12.7)^2 / 4\ ]\text{mm}^2 \times 10^{-3} = \mathbf{51.9\ kN/m}$

> 8.1 kN/m **OK**

$(T_f / T_r)^2 + (V_f / V_r)^2 = (6.0 / 64.8)^2 + (8.1 / 51.9)^2 = \mathbf{0.05 \le 1}$ **=> OK**

Figure 3: Shelf Angle Design at Floor Level – SPF Wood Rim Board with Through Bolt



ii) Bolted connection design to resist shear force, Vf = 9.47 kN/m from Figure 3 above

The bolted connection design for shear shall resist all possible yielding and brittle failure modes.

The yielding resistance $\mathbf{N_r = \phi_y n_u n_s n_F}$ (CSA O86 [1] - 14 12.4.4.3.2)

$\phi_y = 0.8$

n_u = unit lateral yielding resistance governed by failure mode (b)

$\Rightarrow n_u = f_2 \cdot d_f \cdot t_2 \times 10^{-3}$

f_2=5.24 MPa (K_D=0.65, K_{SF}=1.0, K_T=1.0),

d_f=12.7 mm

t_2 = 38 + 38 = 76 mm (ignore the contribution of wood sheathing)

$\Rightarrow n_u = 5.24 \text{ MPa} \cdot 12.7 \text{ mm} \cdot 76 \text{ mm} \times 10^{-3} = 5.07 \text{ kN}$

$n_s = 1.0$

n_F = 2.46 bolts per metre

$N_r = 0.8 \cdot 5.07 \text{ kN} \cdot 1.0 \cdot 2.46/\text{m} = \mathbf{10.0} \text{ kN/m} > 9.47 \text{ kN/m}$ => **OK**

The brittle failure mode is perpendicular-to-grain splitting.

The splitting resistance QSrT = QSr1 + QSr2 (ignore the contribution of the wood sheathing)

The perpendicular-to-grain splitting resistance of the rim board

$$\mathbf{QS_{r1} = QS_{r2} = \phi_w QS(K_D K_{SF} K_T) n_F}$$

$$QS = 14 \cdot t \cdot (d_e/(1-d_e/d))^{0.5} = 14 \cdot 38 \cdot (102/(1-102/286))^{0.5} \cdot 10^{-3} = 6.70 \text{ kN}$$

$$QS_{rT} = 2 \cdot 0.7 \cdot 6.70 \text{ kN} \cdot (0.65 \cdot 1.0 \cdot 1.0) \cdot 2.46/\text{m} = \mathbf{15.0} \text{ kN/m} > 9.47 \text{ kN/m} \Rightarrow \textbf{OK}$$

iii) SPF Rim Board Design to Resist Shear

The factored shear resistance of the SPF rim boards can be obtained using CSA-O86-2014 Section 6.5.5.2

$V_r = \phi \cdot f_v \cdot (K_D K_H K_{Sv} K_T) \cdot 2/3 \cdot A_n \cdot K_{Zv} \cdot n_F \times 10^{-3}$

$\phi = 0.90$

fv = 1.5 MPa (CSA-O86 [1]-14 Table 6.3.1A)

$K_D = 0.65$

$K_H = K_T = K_{Sv} = 1.0$

$A_n = 2 \cdot 38 \text{ mm} \cdot 102 \text{ mm} = 7772 \text{ mm}^2$ (ignore contribution from the sheathing)

K_{Zv} = 1.5 (CSA-O86-14 [1] Table 6.4.5)

n_F = 2.46 bolts per metre

$V_r = 0.9 \cdot 1.5 \text{ MPa} \cdot (0.65 \cdot 1.0 \cdot 1.0 \cdot 1.0) \cdot 2/3 \cdot 7752 \text{ mm}^2 \cdot 1.5 \cdot 2.46/\text{m} \times 10^{-3} = \mathbf{16.8 \text{ kN/m}} > 9.47 \text{ kN/m}$

OK

iv) SPF Rim Board Design to Resist Bearing of Washer

44.5 mm (1-3/4") diameter washer against rim board – bearing resistance of rim board using 2 ply 2x12 SPF rim board can be obtained using CSA-O86 [1] -2014 Section 6.5.7.2 which states

$Q_r = \phi f_{cp} \cdot (K_D \cdot K_{Scp} \cdot K_T) \cdot A_b \cdot K_B \cdot K_{Zcp} \cdot n_F \times 10^{-3}$

$\phi = 0.80$

$n_F = 2.46$ bolts per metre

$f_{cp} = 5.3$ MPa (for SPF)

$K_D = 0.65$ (long-term loading)

$K_{Scp} = 1.0$

$K_T = 1.0$

$A_b = \pi\,[(44.5\text{ mm})^2 - (14.7\text{ mm})^2] / 4 = 1382\text{ mm}^2$

$K_B = 1.0$

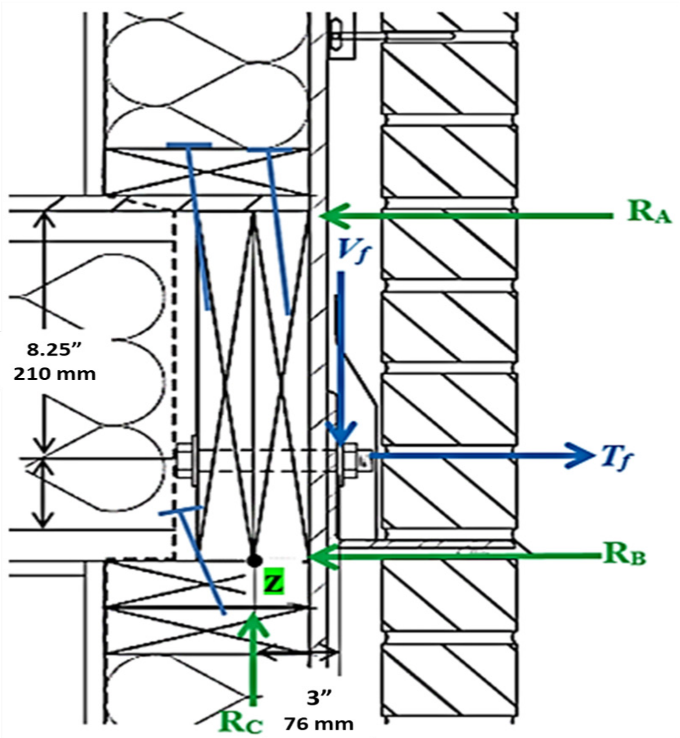
$K_{Zcp} = 1.0$

$(Qr)_{washer} = 0.80 \cdot (5.3\text{ MPa} \cdot 0.65 \cdot 1.0 \cdot 1.0) \cdot 1382\text{ mm}^2 \cdot 1.0 \cdot 1.0 \cdot 2.46 \times 10^{-3} = \mathbf{9.37\ kN/m}$

> 8.75 kN/m => **OK**

v) SPF Rim Board Connection Design to Resist Applied Forces

Figure 4: 2-Ply SPF Rim Board Forces



From Statics the Load on the Nails

$+\circlearrowleft \Sigma M_Z = 0 \Rightarrow R_A (0.286\text{ m}) - (8.75\text{ kN/m} \cdot 0.0762\text{ m}) - 9.47(0.0603\text{ m}) = 0$

$\Rightarrow \mathbf{R_A = 4.33\ kN/m}$

$+\leftarrow \Sigma F_x = 0 \Rightarrow R_A + R_B - 8.75\text{ kN/m} = 0$

$\Rightarrow R_B = 8.75\text{ kN/m} - 4.91\text{ kN/m}$

$\Rightarrow \mathbf{R_B = 4.42\ kN/m}$

$+\uparrow \Sigma F_x = 0 \Rightarrow R_C - V_f = 0$

$\Rightarrow \mathbf{R_c = 9.47\ kN/m}$

=> N_f (lateral load on top nails) = R_A = ***4.33*** *kN/m*

=> N_f (lateral load on bottom nails) = R_B = ***4.42*** *kN/m*

vi) N_r (lateral resistance of common nails) = $(\phi\, n_u) \cdot n_F \cdot n_S \cdot K_D \cdot K_{SF} \cdot K_T \cdot J_E \cdot J_A \cdot J_B \cdot J_D$

(From the Canadian Wood Council's – Wood Design Manual 2017[3] – Nail Selection Tables)

For the connection resisting shear force RB, try 3.5" long (4.12 mm in diameter) common wire nails where toenailing starts at approximately 1/3 the nail length from the end of the piece and at an angle of 30 degrees. Try three nails at every 8'' o.c.

The basic factored lateral resistance can be found from the Nail Selection Tables for a 38 mm thick SPF side member (assume the top plate of the wall assembly is constructed with SPF material). Since the penetration length into the main member is 2/3 of the nail length (greater than 33 mm),

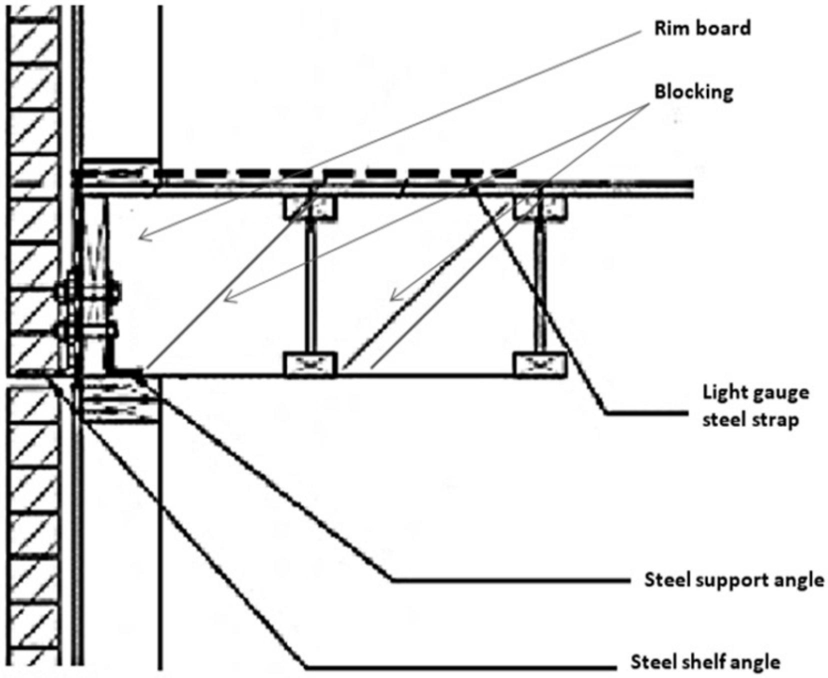
=> $\phi\, n_u$ = 0.877 kN per nail

$K_D \cdot K_{SF} \cdot K_T = 0.65 \cdot 1.0 \cdot 1.0 = 0.65$
$J_E \cdot J_A \cdot J_B \cdot J_D = 1.0 \cdot 0.83 \cdot 1.0 \cdot 1.0 = 0.83$
n_F = 3 nails every 8" o.c. = (1000 mm / 203.2 mm) = **14.8** nails per metre
$n_S = 1.0$

N_r = (0.877 kN)· 14.8 · 1.0 · 0.65 · 0.83 = **6.99 kN/m** > 4.91 kN/m **OK**

An alternate method to resist rim board forces can be found in Figure 5 below:

Figure 5: Light Gauge Steel Strap and Angle Detail to Resist Rim Board Forces



ALTERNATE DESIGN – 5/8" (16 mm) diam. Lag screws spaced at 203 mm (8") with 3-ply SPF 2x12 wood Rim board and L102x102x9.5 shelf angle.

Design Forces (From Figure 6):

$$P_s = \mathbf{6.77\ kN/m}$$
$$P_f = \mathbf{9.47\ kN/m}$$

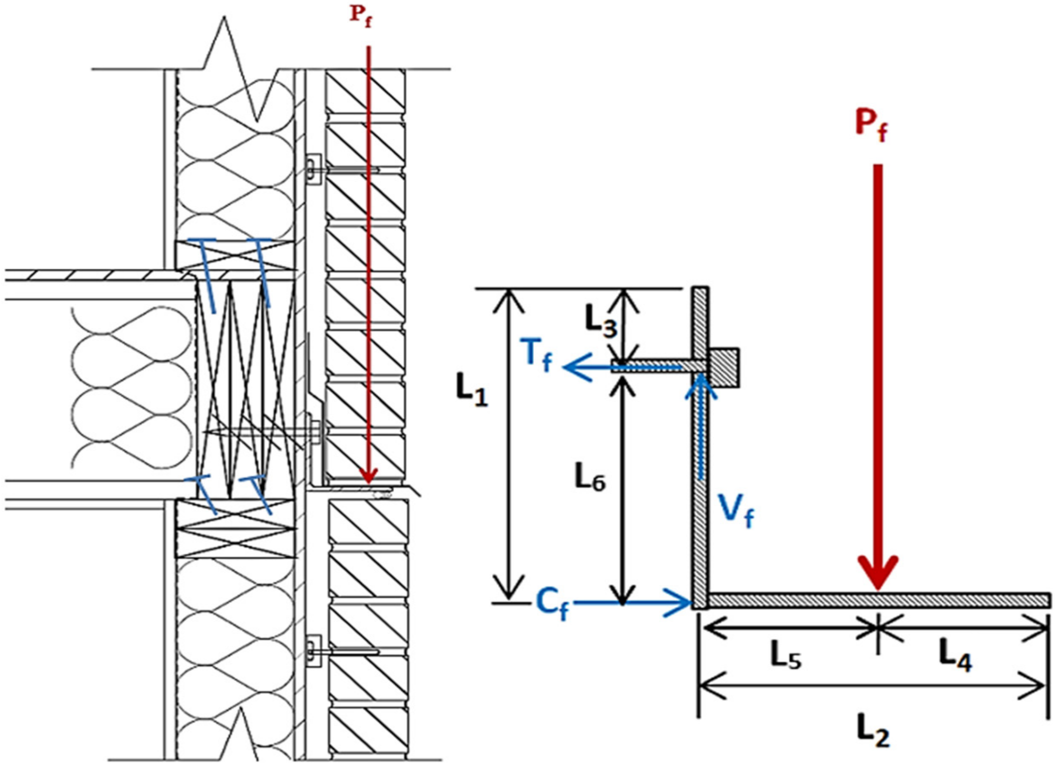
$$T_f = \frac{9.47\text{ kN/m} \cdot (70.4\text{ mm})}{(76.2\text{ mm})} = \mathbf{8.75\ kN/m} \qquad \Rightarrow \mathbf{1.78\ kN\ /\ per\ Lag\ Screw}$$

$$V_f = P_f = \mathbf{9.47\ kN/m} \qquad \Rightarrow \mathbf{1.92\ kN\ /\ per\ Lag\ Screw}$$

$$C_f = T_f = \mathbf{8.75\ kN/m}$$

$$M_f = P_f \cdot L_5 = 9.47\text{ kN/m} \cdot 0.0704\ m = \mathbf{0.667\ kN\text{-}m\ /m}$$
$$M_s = P_s \cdot L_5 = 6.77\text{ kN/m} \cdot 0.0704\ m = \mathbf{0.476\ kN\text{-}m\ /m}$$

Figure 6: Shelf Angle Design at Floor Level – 3ply SPF 2x12 Rim Board with Lag Screws



$L_1 = 101.6$ mm $L_2 = 101.6$ mm $L_3 = 25.4$ mm
$L_4 = 45.0$ mm $L_5 = 70.4$ mm $L_6 = 76.2$ mm

$t = 9.525$ mm
$b = 1000$ mm per metre of veneer
$E_s = 200{,}000$ MPa
$I_x = b \cdot t^3/12 = (1000\text{ mm}) \cdot (6.35\text{ mm})^3/12 = \mathbf{72{,}013\ mm^4}$
$Sx = b \cdot t^2/6 = (1000\text{ mm}) \cdot (6.35\text{ mm})^2/6 = \mathbf{15{,}121\ mm^3}$

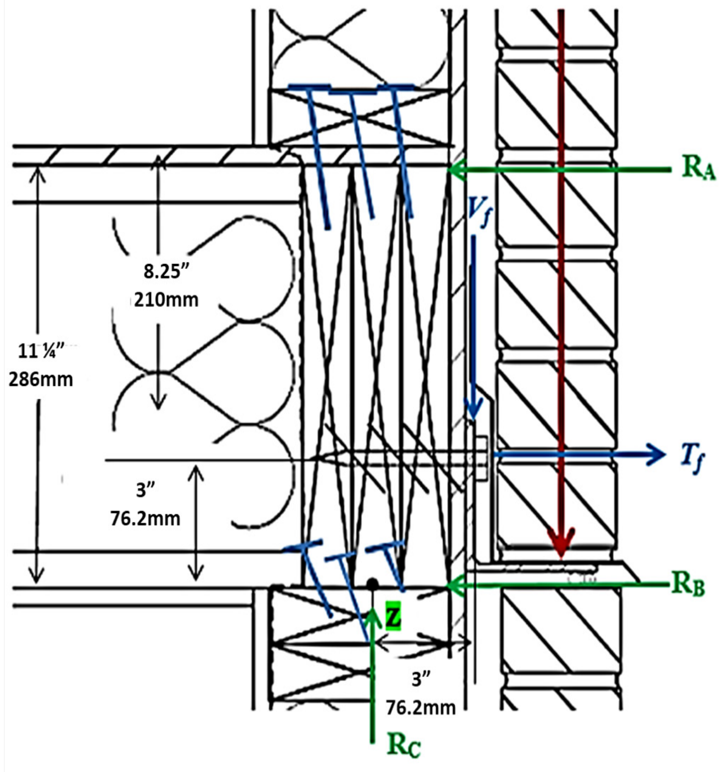
$M_r = \phi \cdot F_y \cdot S_x \times 10^{-6} = 0.90 \cdot 345\text{ MPa} \cdot 15{,}121\text{ mm}^3 \times 10^{-6} = \mathbf{4.7\ kN\text{-}m\ /m}$
$> M_f = 0.46$ kNm/m => **OK**

$V_r = 0.5 \cdot \phi \cdot F_u\, b \cdot t \times 10^{-3} = 0.5 \cdot 0.90 \cdot 0.67 \cdot 345\text{ MPa} \cdot 1000\text{ mm} \cdot 9.525\text{ mm} \times 10^{-3} = \mathbf{991\ kN/m}$
$> V_f = 8.1$ kN/m => **OK**

$\Delta = \Delta_P + \Delta_\theta + \Delta_{Axial}$
$= (P_s \cdot L_5^2 / 6\, E_s\, I_x) \cdot (3L_2 - L_5) + (M_s \cdot L_6 / 3\, E_s \cdot I_x) \cdot L_2 + (P_s \cdot L_1 / A \cdot E_s)$
$= (6{,}770\text{ N} \cdot (56.6\text{ mm})^2 / (6 \cdot 200{,}000\text{ N/mm}^2 \cdot 72{,}013\text{ mm}^4) \cdot (3 \cdot 101.6\text{ mm} - 70.4\text{ mm})$
$+ (0.476 \times 10^6\text{ N-mm} \cdot 165.1\text{ mm}) / 3 \cdot 200{,}000\text{ N/mm}^2 \cdot 72{,}013\text{ mm}^4) \cdot 101.6\text{ mm}$
$+ (6{,}770\text{ N} \cdot (101.6\text{ mm}) / (9{,}525\text{ mm}^2 \cdot 200{,}000\text{ N/mm}^2)$
$= \mathbf{0.177\ mm}$
$< L_2 / 480 = 101.6\text{ mm} / 480 = 0.212\text{ mm}$ => **OK**

vi) SPF rim board connection design to resist applied forces

Figure 7: 2-Ply SPF Rim Board Forces



From Statics the Loads on the Nails (Figure 7)

$$+\circlearrowleft \Sigma \mathbf{M_Z} = 0 \Rightarrow R_A(0.286\text{ m}) - (8.75\text{ kN/m} \cdot 0.0762\text{ m}) - 9.47(0.0762\text{ m}) = 0$$
$$\Rightarrow \mathbf{R_A = 4.86\ kN/m}$$
$$+\leftarrow \Sigma \mathbf{F_x} = 0 \Rightarrow R_A + R_B - 8.75\text{ kN/m} = 0$$
$$\Rightarrow R_B = 8.75\text{ kN/m} - 4.91\text{ kN/m}$$
$$\mathbf{\Rightarrow R_B = 3.89\ kN/m}$$
$$+\uparrow \Sigma \mathbf{F_x} = 0 \Rightarrow R_C - V_f = 0$$
$$\Rightarrow \mathbf{R_c = 9.47\ kN/m}$$

$\Rightarrow N_f$ (lateral load on top nails) = $\mathbf{R_A} = \boldsymbol{4.86\ kN/m}$
$\Rightarrow N_f$ (lateral load on bottom nails) = $\mathbf{R_B} = \boldsymbol{3.89\ kN/m}$

vi) N_r (lateral resistance of common nails) = $(\phi\ n_u) \cdot n_F \cdot n_S \cdot K_D \cdot K_{SF} \cdot K_T \cdot J_E \cdot J_A \cdot J_B \cdot J_D$

(From the Canadian Wood Council’s – Wood Design Manual 2017[3] – Nail Selection Tables)

For the connection resisting shear force RB, try 3.5” long (4.12 mm in diameter) common wire nails where toe nailing start at approximately 1/3 the nail length from the end of the piece and at an angle of 30 degree. Try two nails at every 16’’ o.c. (i.e., same location as the bolts).

The basic factored lateral resistance $\phi\, n_u$ can be found from the Nail Selection Tables for a 38 mm thick SPF side member (assume the top plate of the wall assembly is constructed with SPF material). Since the penetration length into the main member is 2/3 of the nail length (greater than 33 mm),

=> $\phi\, n_u$ = 0.877 kN per nail

$K_D \cdot K_{SF} \cdot K_T = 0.65 \cdot 1.0 \cdot 1.0 = 0.65$
$J_E \cdot J_A \cdot J_B \cdot J_D = 1.0 \cdot 0.83 \cdot 1.0 \cdot 1.0 = 0.83$
n_F = 3 nails every 8” o.c. = (1000 mm / 203.2 mm) = **14.8** nails per metre
$n_S = 1.0$

N_r = (0.877 kN)· 14.8 · 1.0 · 0.65 · 0.83 = **6.99 kN/m**
> 4.86 kN/m **OK**

From load analysis the design loads on the fasteners are:

Prf = withdrawal force on lag screws per metre length of veneer = Tf = **8.75 kN/m**

Qf = shear force on lag screws per metre length of veneer = Vf =**9.47 kN/m**

•3-ply SPF 2x12 rim board c/w 5/8” dia. (ϕ 16mm) **lag screws** at 203mm (8”) o.c.

•Minimum penetration for lag screws = 5d = 3-1/8” (79 mm)

nF = 1000mm / 203.2 mm = 4.92 lag screws per metre

From the Canadian Wood Council’s - Wood Design Manual 2017[3] - Lag screw resistance (Table 7.14 and Table 7.15 and Lag Screw Selection Tables):

For a 5/8” dia. (16 mm dia.) 4-1/4” long Lag screw at 203mm o.c. – withdrawal resistance for SPF Rim board.

$P_{rw} = P'_{rw} \cdot L_t \cdot n_F \cdot K_T \cdot K_D \cdot K_{SF} \cdot J_E$

n_F = 4.92 screws per metre
P'_{rw} = 0.074 kN/mm (Table 7.14)
L_T = lesser of L/2 + 12.7 –E or 152 mm – E (Table 7.15)
E = 9.5 mm (Lag screw tip length)
=> L_T = 76.2 mm
K_D = 0.65
K_{SF} =1.0
K_T = 1.0
J_E = 1.0

P_{rw} = 0.074 kN/mm· 76.2 mm · 4.92/m · 1.0· 0.65· 1.0 1.0 = **18.0 kN/m** > 8.75 kN/m

For a ϕ5/8” (ϕ16 mm) Lag screw – shear resistance for SPF Rim board with 6.35 mm thick steel plate

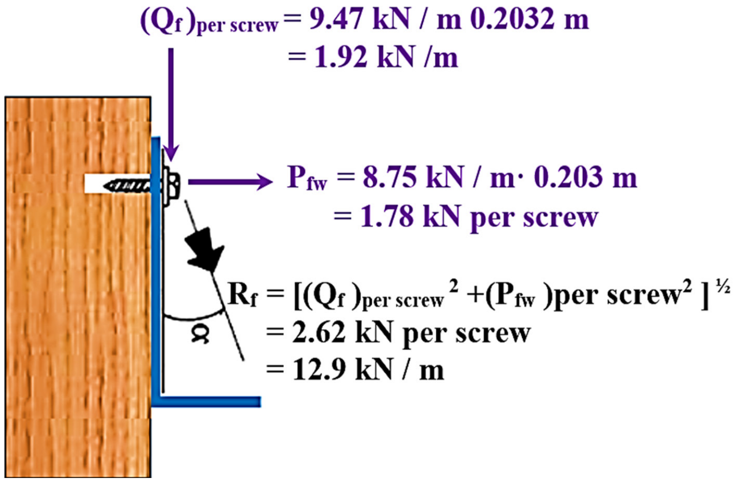
$Q_r = Q_r' \cdot n_{Fe} \cdot n_R \cdot K_D \cdot K_{SF} \cdot K_T$

$Q_r' = 3.67$ kN (from Lag Screw Selection Table for single shear, 4 mm steel side plate)
n_{FE} = 4.92 screws per metre
n_R = 1 row of fasteners
$K_D = 0.65$
$K_{SF} = 1.0$
$K_T = 1.0$

$$Q_r = 3.67 \text{ kN} \cdot 4.92/\text{m} \cdot 1 \cdot 0.65 \cdot 1.0 \cdot 1.0 = \mathbf{11.7\ kN/m} > 9.47 \text{ kN/m}$$

For a 5/8" dia. (16 mm dia.) lag screw, the combined shear and withdraw resistance can be estimated using section 12.4 in the NDS-2018 National Design Specification (NDS) for Wood Construction which states:

$$Z_a' = \frac{(W' \cdot p)\, Z'}{(W' \cdot p)\cos^2 a + Z' \sin^2 a}$$



Where:

Z_α' = adjusted resistance for combined lateral and withdrawal (lbs.) *per single fastener*
$(W' \cdot p)$ = withdrawal resistance *for a single fastener* (lbs.) = P_{rw} =(13.2 kN / m / 4.92) = **3.66 kN**
Z'= lateral resistance *for a single fastener* (lbs.) = Q_r = (11.5 kN / m /4.92) = **2.39kN**
α = degree between force in grain = tan^{-1} (1.78 kN / 1.92 kN) = **42.7°**

$$Z_\alpha' = (3.66 \text{ kN}) \cdot (2.39 \text{ kN}) / [(3.66 \text{ kN}) \cdot \cos^2(42.7°) + 2.39 \text{ kN} \cdot \sin^2(42.7°)]$$

$$\Rightarrow Z_\alpha' = \mathbf{2.84\ kN\ per\ screw}$$

$$= \mathbf{2.84\ kN \cdot 4.92 = 14.0\ kN / m}$$

$$> R_f = 2.62 \text{ kN per screw} = 12.9 \text{ kN/m} \qquad \Rightarrow \mathbf{OK}$$

vii) SPF rim board design to resist bearing of angle iron

$N_r = \phi f_{cp} \cdot (K_D \cdot A_b \cdot K_B \cdot K_{Zcp} \cdot n_F \times 10^{-3}$
$\phi = 0.80$
f_{cp} = 5.3 MPa (for SPF)
K_D = 0.65 (long-term loading)
A_b = b t = 1000 mm 50.8 mm = 50,800 mm^2
K_{Zcp} = 1.0

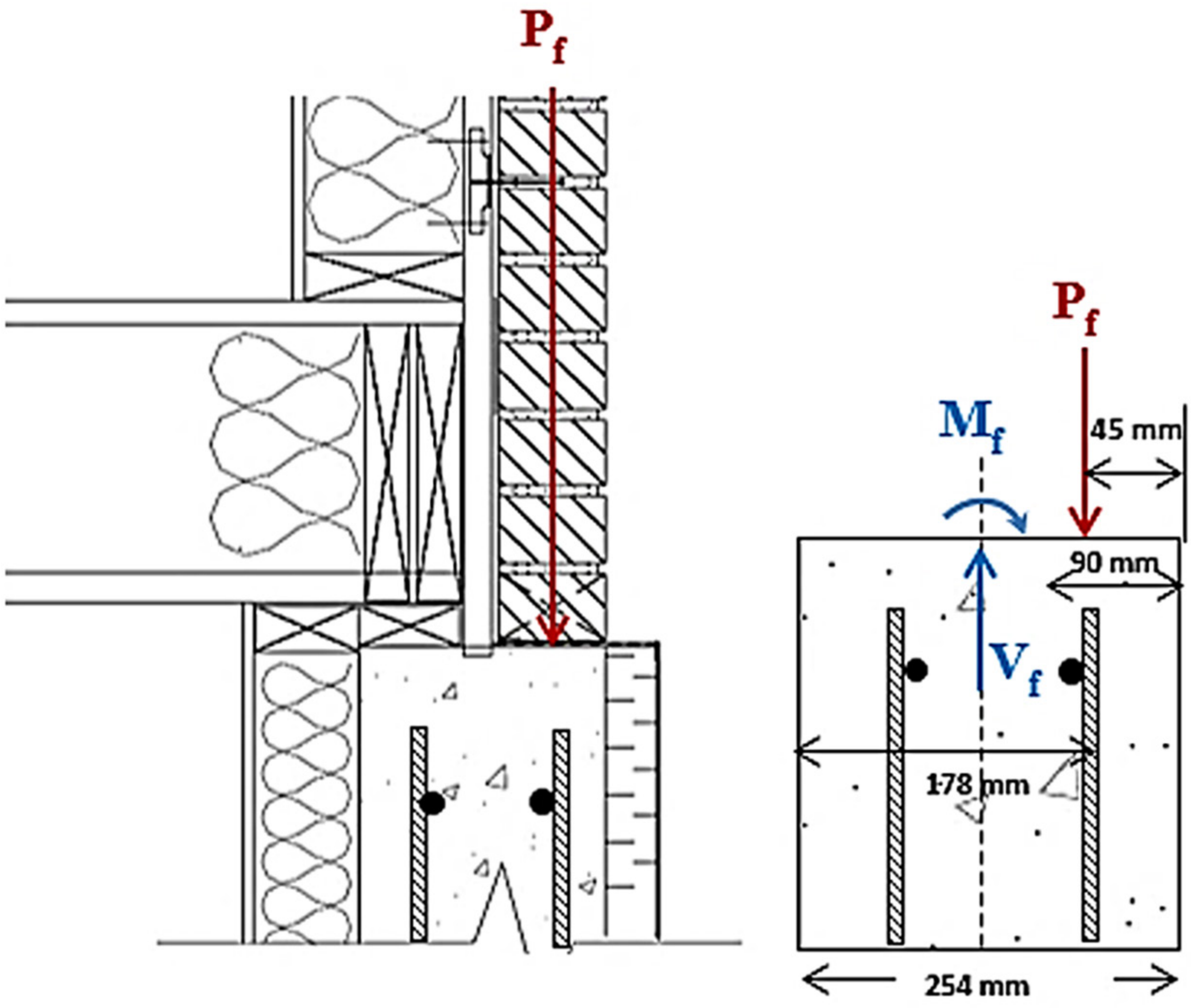
$$(N_r)_{angle\ heal} = 0.80 \cdot (5.3 \text{ MPa} \cdot 0.65 \cdot 1.0 \cdot 1.0) \cdot 50{,}800 \text{ mm}^2 \cdot 1.0 \cdot 1.0 \cdot \times 10^{-3} = \mathbf{140\ kN/m}$$
$$> 8.75 \text{ kN/m} \Rightarrow \mathbf{OK}$$

Brick Veneer Support on a Concrete Foundation Brick Ledge

Assumptions:

- 90 mm thick brick veneer
- 254 mm (10”) Reinforced Concrete Wall
- 20MPa Concrete
- 15M bars at 24” o.c (609.6 mm) horizontally and 24” o.c (609.6 mm) vertically
- Veneer supported to max 9.144 m (30’)

Figure 8: Brick Ledge Design for Supporting Masonry Veneer



Design Forces:

$P_f = 23.2$ kN/m
$M_f = 1.90$ kN-m/m
$V_f = 23.2$ kN/m

Reinforced Concrete parameters from CSA-A23.3- 2014 – Design of Concrete Structures [4]:

$$\phi_c = 0.65$$
$$\phi_s = 0.85$$
$$\lambda = 1.00$$
$$f'_c = 20 \text{ MPa}$$
$$\alpha_1 = 0.85 - 0.0015 \cdot f'c = 0.85 - 0.0015 \cdot (20) = 0.82$$
$$\beta_1 = 0.97 - 0.0025 \cdot f'c = 0.85 - 0.0025 \cdot (20) = 0.92$$
$$f_y = 400 \text{ MPa}$$
$$A_s = (1000 \text{ mm} / 609.6\text{mm}) \cdot 200 \text{ mm}^2 = 328 \text{ mm}^2$$
$$d = 254\text{mm} - 76.2 \text{ mm} = 178\text{mm}$$
$$b = 1000 \text{ mm}$$

Solve for the moment resistance at the factored axial load:

$$P_{r@Pf} \Rightarrow P_r = C_r - T_r = \mathbf{23{,}200\ N/m} \Rightarrow \alpha_1 \cdot \phi_c \cdot f'_c \cdot b\,(\beta_1 \cdot c) - \phi_s \cdot f_y \cdot A_s = 23{,}200 \text{ N/m}$$
$$\Rightarrow 0.82 \cdot 0.65 \cdot 20 \text{ MPa} \cdot 1000\text{mm}\,(0.92 \cdot c) - 0.85 \cdot 400 \text{ MPa} \cdot 328 \text{ mm}^2 = 23{,}200 \text{ N}$$
$$\Rightarrow c = 13.1 \text{ mm}$$
$$\Rightarrow a = \beta_1 \cdot c = 0.92 \cdot 13.8 = 12.0 \text{ mm}$$

$$M_{r@Pf} = \phi_s \cdot f_y \cdot A_s \cdot [d - a/2] \times 10^{-6} = 0.85 \cdot 400 \text{ MPa} \cdot 328 \text{ mm}^2 \cdot [178 - 12.0/2] \times 10^{-6} = \mathbf{18.0\ kN\text{-}m/m}$$
$$\mathbf{> 1.90\ kN\text{-}m/m \quad OK}$$

$$V_r = V_c = \phi_c \cdot \lambda \cdot b \cdot \sqrt{f'_c} \cdot b_w \cdot d_v \times 10^{-3} = 0.65 \cdot 1.0 \cdot 0.18 \cdot \sqrt{20 \text{ MPa}} \cdot 102\text{mm} \cdot (0.8 \cdot 1000\text{mm}) = \mathbf{53.3\ kN/m}$$
$$\mathbf{> 23.2kN/m \quad OK}$$

References

[1] Canadian Standards Association, CSA O86- Engineering design in wood, including Update 1 (May 2016) and Update 2 (June 2017)

[2] Canadian Standards Association, CSA A371- Masonry Construction for Buildings, Canadian Standards Association, Mississauga, Ontario, Canada, 2014

[3] Canadian Wood Council – Wood Design Manual 2017, Canadian Wood Council, Ottawa, Ontario, Canada, 2017

[4] Canadian Standards Association, CSA A23.3 – Design of Concrete Structures, Canadian Standards Association, Mississauga, Ontario, Canada, 2014

[4] Canadian Standards Association, CSA S16 – Design of Steel Structures, Canadian Standards Association, Mississauga, Ontario, Canada, 2014