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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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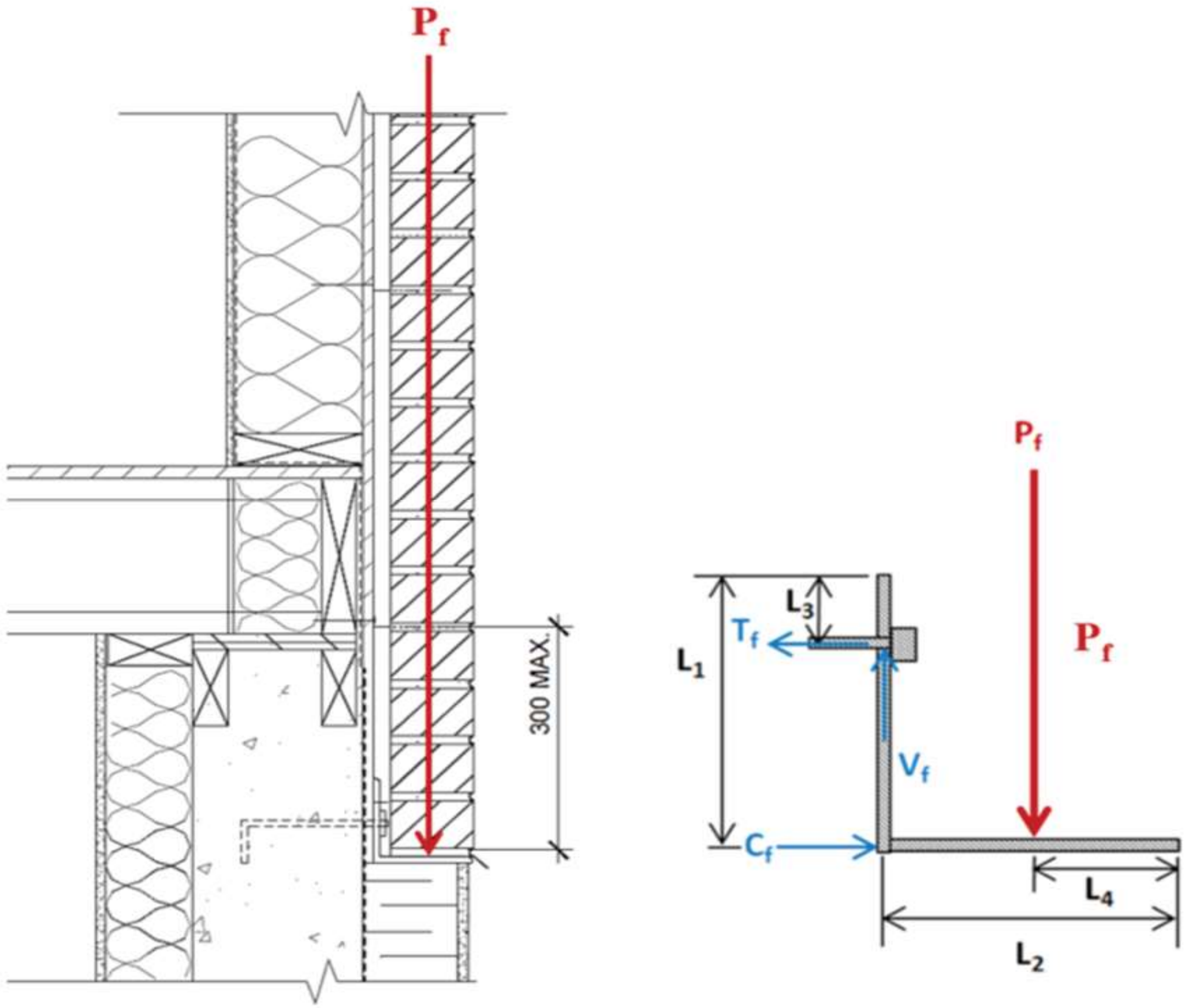



L1 = Vertical Leg length (m)
L2 = Horizontal Leg length (m)
L3 = Spacing of bolt hole (typically 25 mm (1”) from top of leg vertical leg
L4 = Centroid of brick veneer (typically 45 mm for metric modular 90 mm brick)
L5 = Eccentricity of the veneer load = (L2 – L4)

## Bolt Forces (From Statics)

$$T_f = \frac{P_f \cdot (L_2 - L_4)}{(L_1 - L_3)}$$ => Tensile loads on the bolts per metre length of wall.

$$C_f = \frac{P_f \cdot (L_2 - L_4)}{(L_1 - L_3)}$$ => Compression load on the concrete wall or wood rim board per metre length of wall.

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

$M_f = P_f \cdot (L_2 - L_4)$ => Maximum moment on the angle iron per metre length of the wall.

## Shelf Angle Design - At Foundation

**Assumptions:**

- 90 mm thick brick veneer
- L152 x 102 x 9.5 (L6" x 4" x 3/8") angle iron anchored into the concrete
- 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 9.14 m (30')

**Design Forces:**

$P_f = 1.4 \cdot (DL_{brick} + DL_{shelfangle}) = 1.4 \cdot (19.6\ kN/m^3 \cdot 0.090\ m \cdot 9.144\ m + 0.19\ kN/m) = \mathbf{22.8\ kN/m}$

$$T_f = \frac{22.8\ kN/m \cdot (101.6\ mm - 45mm)}{(152.4\ mm - 38.1\ mm)} = \mathbf{11.3\ kN/m}$$

$C_f = T_f = \mathbf{11.3\ kN/m}$
$V_f = P_f = \mathbf{22.8\ kN/m}$
$M_f = P_f \cdot (L_2 - L_4) = 22.8\ kN/m \cdot (0.1016\ m - 0.045\ m) = \mathbf{1.3\ kN\text{-}m\ /m}$

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

$L_1 = 152.4$ mm
$L_2 = 101.6$ mm
$t = 9.525$ mm
$b = 1000$ mm per metre of veneer
$E_s = 200{,}000$ MPa
$I_x = b \cdot t^3/12 = 1000\ mm \cdot (9.525\ mm)^3/12 = 72{,}013\ mm^4$
$Sx = b \cdot t^2/6 = 1000\ mm \cdot (9.525\ mm)^2/6 = 15{,}121\ mm^3$
$F_y = 300$ MPa
$F_s = 0.67\ F_y$

$M_r = \phi \cdot F_y \cdot S_x \times 10^{-6} = 0.90 \cdot 300MPa \cdot 15{,}121\ mm3 \times 10^{-6} = \mathbf{4.1\ kN\text{-}m/m}$
**> 1.3 kN-m/m** **OK**

$V_r = 0.5 \cdot \phi \cdot F_s\ b \cdot t \times 10^{-3} = 0.5 \cdot 0.90 \cdot 0.67 \cdot 300\ MPa \cdot 1000\ mm \cdot 9.525\ mm \times 10^{-3} = \mathbf{862\ kN/m}$
**> 23.7 kN/m** **OK**

$\Delta = (P_s \cdot L_5^2 / 6\ E_s\ I_x) \cdot 3(L_2 - L_4)$
$= (16{,}317N \cdot (56.6\ mm)^2 / (6 \cdot 200{,}000\ N/mm^2 \cdot 72{,}013\ mm^4) \cdot (3 \cdot (101.6mm - 45mm)) = \mathbf{0.10\ mm}$
$< L_2 / 480 = 101.6\ mm / 480 = 0.21\ mm$ **OK**

## ii) Design of Concrete Anchor Bolts to Resist Shear and Withdrawal

**Post Installed Anchor Bolt Resistance**

| | | | |
|---|---|---|---|
| **Bolt Type** | $\phi$5/8" x 3-¼ " Hilti Kwik Bolt 3 Embed 2-3/4" (70mm) | | |

**20 MPa concrete from Hilti**

| | | | |
|---|---|---|---|
| $(T_r)_{bolt}$ | **13.4** | kN/bolt | |
| $(V_r)_{bolt}$ | **17.4** | kN/bolt | |
| bolt spacing | **406** | mm o.c. | |
| $n_{bolts}$ | **2.46** | | bolts per metre |
| $T_r$ | **33.0** | kN/m | **OK** |
| $V_r$ | **42.8** | kN/m | **OK** |
| $(T_f/T_r)^{5/3}+(V_f/V_r)^{5/3} \leq 1$ | **0.43** | | **OK** |

## iii) 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 | | |
| $\beta_1 = 0.97 - 0.0025 \cdot f'c = 0.85 - 0.0025 \cdot (20) =$ | 0.92 | | |
| $b$ | 1000 | mm | |
| $c$ | 57.2 | mm | |
| $a = \beta_1 c =$ | 52.6 | mm | |
| $C_r = \alpha_1 \phi_c f'_c b a$ | **561** | **kN/m** | **OK** |

## Shelf Angle Design - At Floor Level

### Assumptions:

- 90 mm thick brick veneer
- L203x102x6.4 (8"x4"x1/4") angle iron anchored into the wood rim board
- Veneer supported at each floor - max 3.048 m (10')

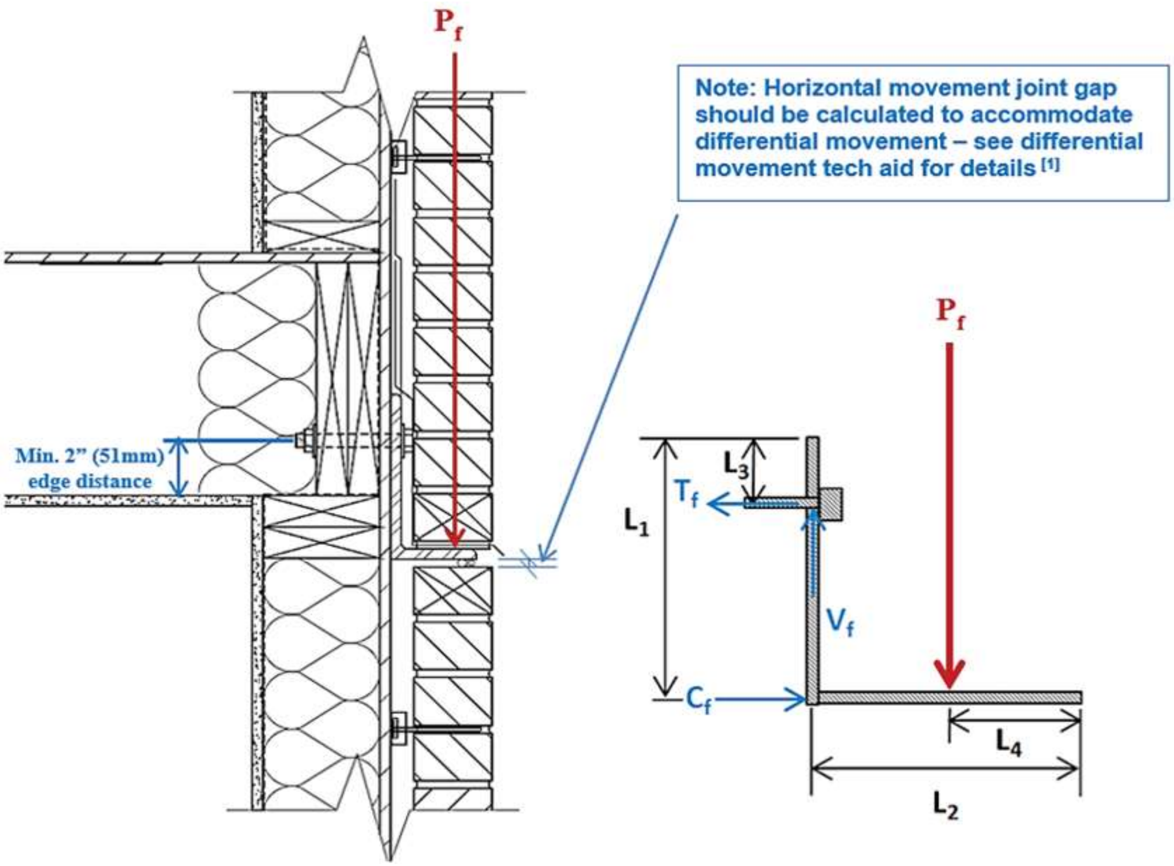
### Through Bolt (RECOMMENDED)

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

### Lag Screw (ALTERNATE SOLUTION)

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 3-1/8" as per CSA-O86-2014 Clause 12.6.3.3

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




**Design Forces:**

$$P_f = 1.4 \cdot 19.6\ \text{kN/m}^3 \cdot 0.090\ \text{m} \cdot 3.048\ \text{m} = \mathbf{7.5\ kN/m}$$

$$T_f = \frac{7.5\text{kN/m} \cdot (101.6\ \text{mm} - 45\text{mm})}{(203.2\ \text{mm} - 25.4\ \text{mm})} = \mathbf{2.4\ kN/m}$$

$$C_f = T_f = \mathbf{2.4\ kN/m}$$

$$V_f = P_f = \mathbf{7.5\ kN/m}$$

$$M_f = P_f \cdot (L_2 - L_4) = 7.5\ \text{kN/m} \cdot (0.1016\ m - 0.045\ m) = \mathbf{0.42\ kN\text{-}m\ /m}$$

### i) Steel Shelf Angled Design for Bending, Shear, and Deflection

$L_2 = 101.6$ mm

$L_5 = 56.6$ mm

$t = 6.35$ mm

$b = 1000$ mm per metre of veneer

$E_s = 200{,}000$ MPa

$I_x = b \cdot t^3/12 = (1000\text{mm})(6.35\text{mm})^3/12 = \mathbf{21{,}337\ mm^4}$

$Sx = b \cdot t^2/6 = (1000\text{mm})(6.35\text{mm})^2/6 = \mathbf{6720\ mm^3}$

$M_r = \phi \cdot F_y \cdot S_x \text{ x } 10^{-6} = 0.90 \cdot 345 \text{ MPa} \cdot 6720\text{mm}^4 \text{ x } 10^{-6} =$ **2.1 kN-m /m > $M_f$ = 0.42 kNm/m OK**

$V_r = 0.5 \cdot \phi F_u \, b \cdot t \text{ x } 10^{-3} = 0.5 \cdot 0.90 \cdot 0.67 \cdot 345 \text{ MPa} \cdot 1000 \text{ mm} \cdot 6.35 \text{ mm x } 10^{-3} =$ **660kN/m > Vf=7.5 kN/m OK**

$\Delta = (P_s \cdot L_5^2 / 6\, E_s\, I_x) \cdot (3(L_2 - L_5)$
$= (5400\text{N} \cdot (56.6)^2 / (6 \cdot 200{,}000 \text{ N/mm}^2 \cdot 21{,}337 \text{ mm}^4) \cdot (3 \cdot (101.6\text{mm} - 56.6\text{mm})) =$ **0.09 mm**
$< L_2 / 480 = 101.6 \text{ mm} / 480 = 0.21 \text{ mm}$ **OK**

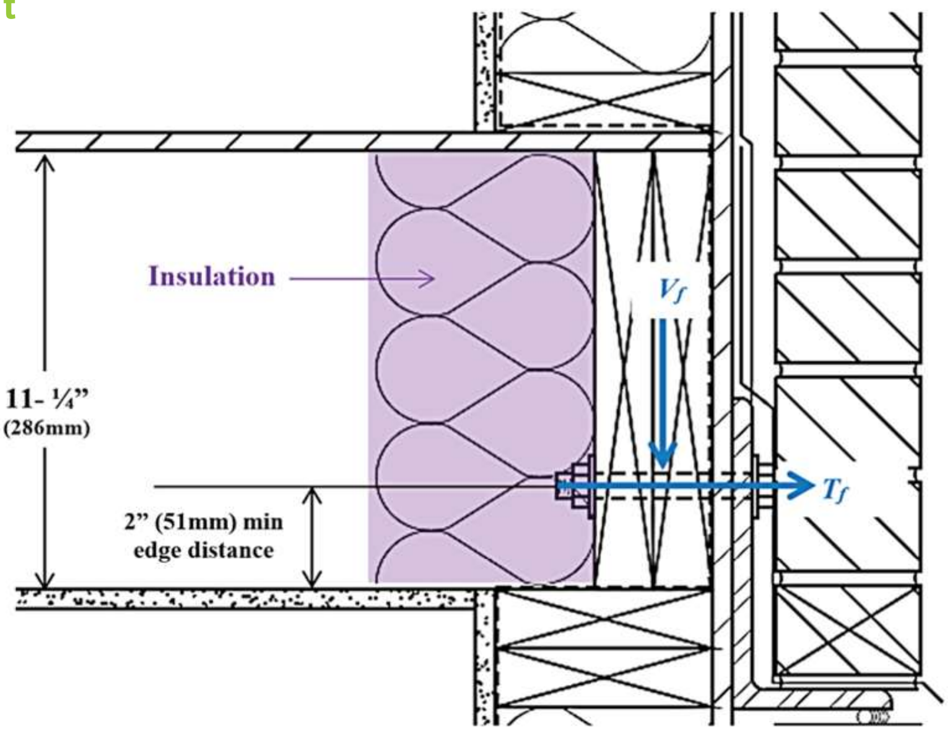
**Recommended Design – Through Bolt With 2-Ply SPF 2x12 Wood Rim Board**

From load analysis the design loads on the fasteners are:

$T_f$ = Tension force on through bolts per metre length of the veneer = **2.4 kN/m**

$V_f$ = Shear force on through bolts per metre length of the veneer = **7.5 kN/m**

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




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

$T_r = 0.75 \cdot \phi_b \, n_b \cdot A_b \cdot F_u \text{ x } 10^{-3} = 0.75 \cdot 0.67 \cdot 2.46 \cdot 414 \text{ MPa} \cdot [\pi (12.7)^2 / 4\,] \text{mm}^2 \text{ x } 10^{-3} =$ **64.8 kN /m**
> 2.4 kN/m **OK**

$V_r = 0.60 \cdot \phi_b \cdot m \cdot n_b \cdot A_b \cdot F_u \text{ x } 10^{-3} = 0.6 \cdot 0.67 \cdot 1 \cdot 2.46 \cdot 414 \text{ MPa} \cdot [\pi (12.7)^2 / 4\,] \text{mm}^2 \text{x } 10^{-3} =$ **51.9 kN/m**
>7.5 kN/m **OK**

$(T_f / T_r)^2 + (V_f / V_r)^2 = (2.4 / 64.8)^2 + (7.5 / 51.9)^2 =$ **0.015 ≤ 1** **OK**

## ii) Bolted Connection Design to Resist Shear Force, Vf = 7.5 kN/m From Figure 3 Above

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

The yielding resistance $N_r = \phi_y n_u n_s n_F$ (CSA O86-14 12.4.4.3.2)

$\phi_y = 0.8$

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

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

Note: SPF rim board should be checked for shear resistance of the member according to CSA-O86 – 6.5.5.2

$f_2 = 5.24$ 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)

$\Rightarrow n_u = 5.24 \text{ MPa} \cdot 12.7 \text{ mm} \cdot 76 \text{ mm} \times 10^{-3} = 5.06 \text{ kN}$

$n_s = 1.0$

$n_F = 2.46$ bolts per metre

$N_r = 0.8 \cdot 5.06 \text{ kN} \cdot 1.0 \cdot 2.46/\text{m} = \mathbf{9.96} \text{ kN/m} > 7.5 \text{ kN/m}$ **OK**

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

The splitting resistance $QS_{rT} = QS_{r1} + QS_{r2}$ (ignore the contribution of the wood sheathing)

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

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

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

$QS_{rT} = 2 \cdot 0.7 \cdot 6.70 \text{ kN} \cdot (0.65 \cdot 1.0 \cdot 1.0) \cdot 2.46/\text{m} = \mathbf{15.0} \text{ kN/m} > 7.5 \text{ kN/m}$ **OK**

38 mm (1-½") diameter washer against rim board – bearing resistance of rim board using 2 ply 2x12 SPF rim board can be obtained using CSA-O86-2014 Section 6.5.7.2 which states:

### 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)

$K_D = 0.65$

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

$A_n = 2 \cdot 38\ mm \cdot 102\ mm = 7752\ mm^2$ (ignore contribution from the sheathing)

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

$n_F = 2.46$ bolts per metre

$V_r = 0.9 \cdot 1.5\ MPa \cdot (0.65 \cdot 1.0 \cdot 1.0 \cdot 1.0) \cdot 2/3 \cdot 7752\ mm^2 \cdot 1.5 \cdot 2.46/m \times 10^{-3} = 16.7\ kN/m > 7.5\ kN/m$

**OK**

### iv) SPF Rim Board Design to Resist Bearing of Washer

38 mm (1-½”) diameter washer against rim board – bearing resistance of rim board using 2 ply 2x12 SPF rim board can be obtained using CSA-O86-2014 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)

$K_{Scp} = 1.0$

$K_T = 1.0$

$A_b = \pi [(38\ mm)^2 - (14.7\ mm)^2]/4 = 964\ mm^2$

$K_B = 1.0$

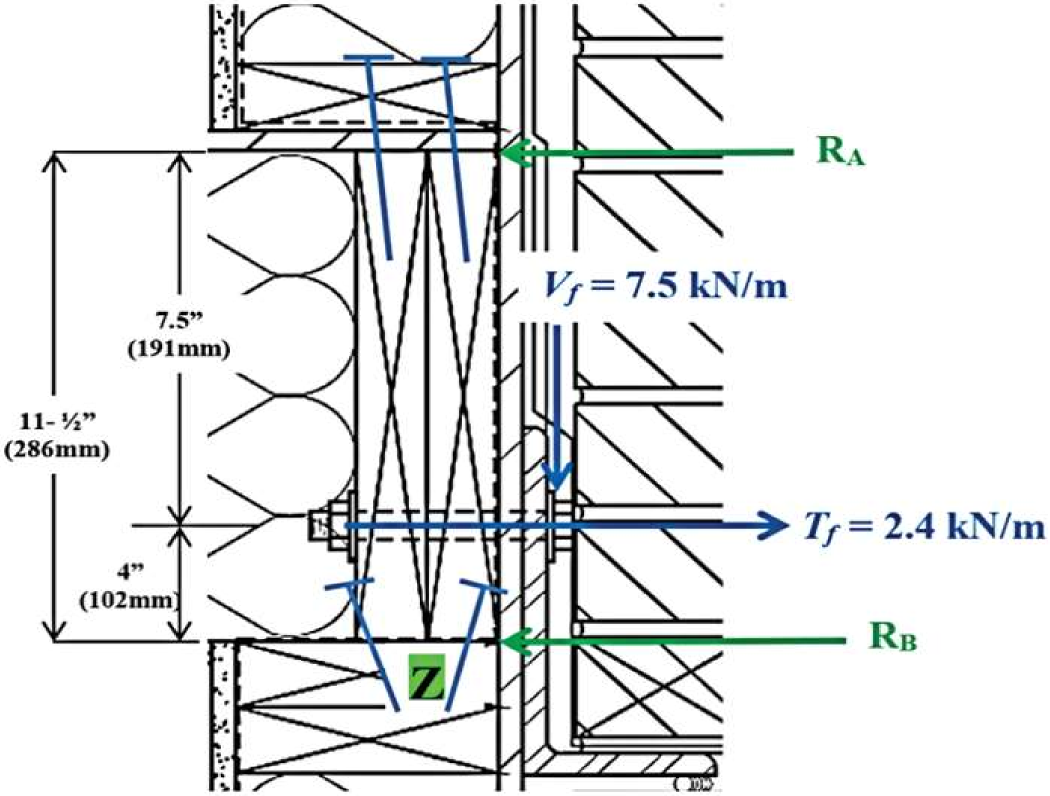
$K_{Zcp} = 1.0$

$(Qr)_{washer} = 0.80 \cdot (5.3\ MPa \cdot 0.65 \cdot 1.0 \cdot 1.0) \cdot 964\ mm^2 \cdot 1.0 \cdot 1.0 \cdot 2.46 \times 10^{-3} = \mathbf{6.5\ kN/m}$

$> 2.4\ kN/m$

## 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

$$\circlearrowleft\!+ \ \Sigma M_Z = 0 \Rightarrow R_A(0.286\text{ m}) - (2.4\text{ kN/m} \cdot 0.102\text{ m}) = 0$$
$$\Rightarrow \mathbf{R_A = 0.856\ kN/m}$$

$$\leftarrow \Sigma F_x = 0 \Rightarrow R_A + R_B - 2.4\text{ kN/m} = 0$$
$$\Rightarrow R_B = 2.4\text{ kN/m} - 0.856\text{ kN/m}$$
$$= \mathbf{1.54\ kN/m}$$

=> $N_f$ (lateral load on top nails) = $R_A$ = *0.86 kN/m*

=> $N_f$ (lateral load on bottom nails) = $R_B$ = *1.54 kN/m*

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

### *From the Canadian Wood Council's - Wood Design Manual 2017 - Nail Selection Tables*

For the connection resisting shear force $R_B$, 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).

$$\Rightarrow \phi\ n_u = 0.877 \text{ kN per nail}$$

$K_D \cdot K_{SF} \cdot K_T = 0.65 \cdot 1.0 \cdot 1.0 = 0.65$

$J_E \cdot J_A \cdot J_B \cdot J_D = 1.0 \cdot 0.83 \cdot 1.0 \cdot 1.0 = 0.83$

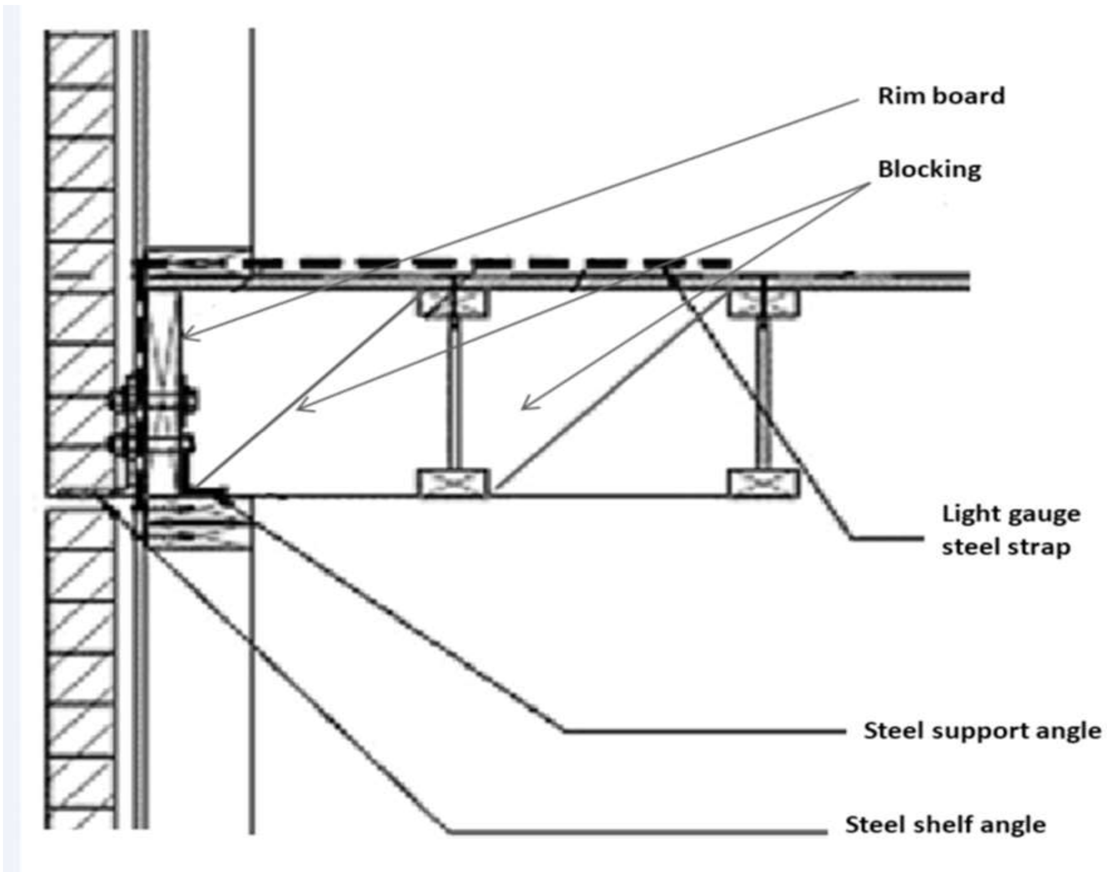
$n_F$ = 2 nails every 16" o.c. = (1000 mm / 406.4mm) = **4.92** nails per metre

$n_S = 1.0$

$$N_r = (0.877 \text{ kN}) \cdot 4.92 \cdot 1.0 \cdot 0.65 \cdot 0.83 = \mathbf{2.33\ kN/m} > 1.56 \text{ kN/m} \quad \textbf{OK}$$

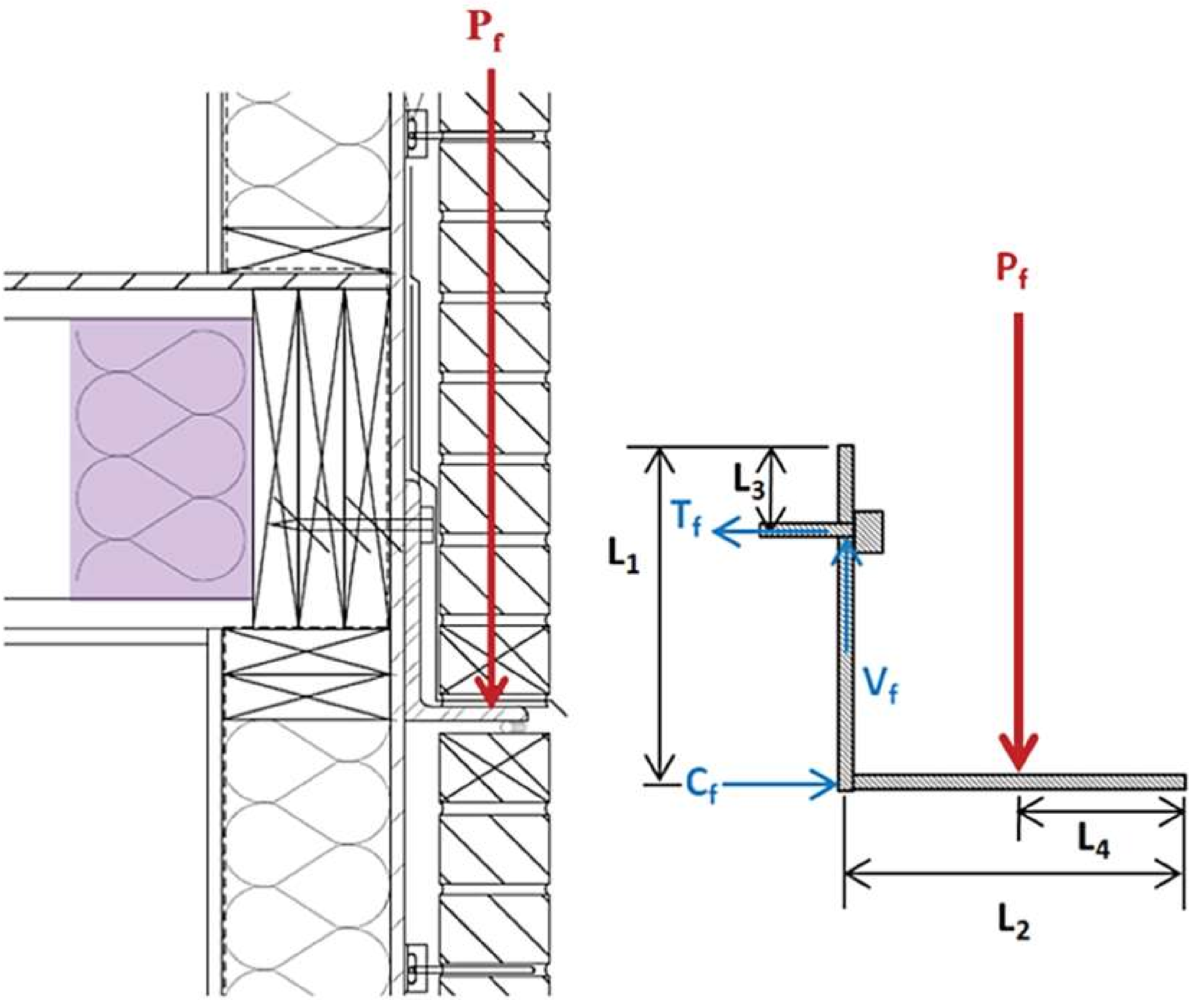
An alternate method to resist rim board forces can be found in Figure 5 below:

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



**<u>ALTERNATE DESIGN</u> – Lag Screws with 3-Ply SPF 2x12 Wood Rim Board**

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




From load analysis the design loads on the fasteners are:

Prf = Withdrawal force on lag screws per metre length of veneer = T$f$ = **2.4 kN/m**

Q$f$ = Shear force on lag screws per metre length of veneer = V$f$ =**7.5 kN/m**

- 3-ply SPF 2x12 rim board c/w 5/8" dia. (16mm dia.) 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 - Lag Screw Resistance (Table 7.14 & Table 7.15 and Lag Screw Selection Tables):***

For a 5/8" dia. (16 mm dia.) 4-1/4" long Lag screw at 203 mm 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)

=> $L_T$ = 57.2 mm

$K_D$ = 0.65

$K_{SF}$ =1.0

$K_T$ = 1.0

$J_E$ = 1.0

$P_{rw}$ = 0.074 kN/mm· 57.2mm · 4.92/m · 1.0· 0.65· 1.0 1.0 = **13.5 kN/m** > 2.4 kN/m

**For a 5/8" dia. (16 mm dia.) 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

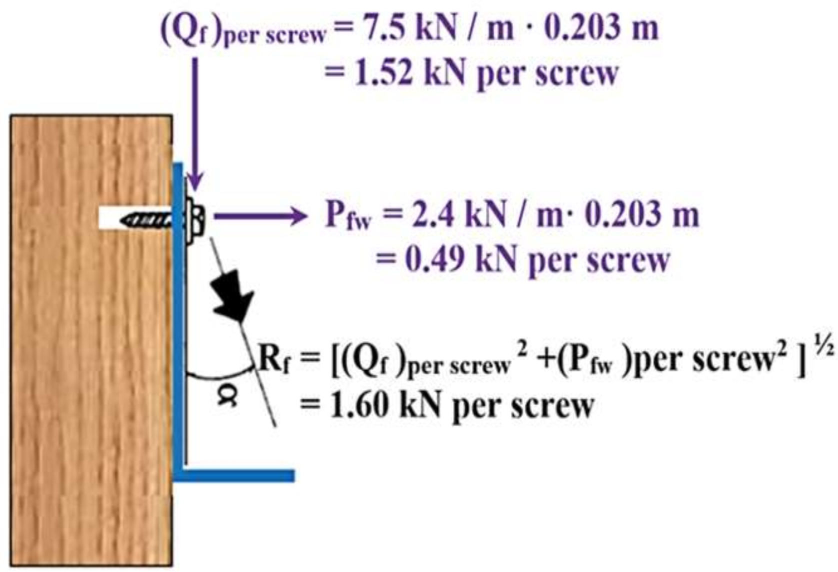
$K_{SF}$ = 1.0

$K_T$ = 1.0

$Q_r$ = 3.67 kN ·4.92/m · 1 ·0.65 ·1.0 ·1.0 = **11.7 kN/m** > 7.5 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:

$$Z_{\alpha}' = \frac{(W'p)Z'}{(W'p)\cos^2\alpha + Z'\sin^2\alpha}$$




$Z_\alpha'$ = 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) = **2.68 kN**

$Z'$= lateral resistance *for a single fastener* (lbs.) = $Q_r$ = (11.5 kN / m /4.92) = **2.34 kN**

$\alpha$ = degree between force in grain = $tan^{-1}$ (0.49 kN / 1.52 kN) =**17.8°**

$Z_\alpha' = (2.68\ kN)\cdot(2.34\ kN) / [(2.68\ kN)\cdot\cos^2(17.8°) + 2.34\ kN\cdot\sin^2(17.8°)]$

=> $Z_\alpha'$ = **2.37 kN**

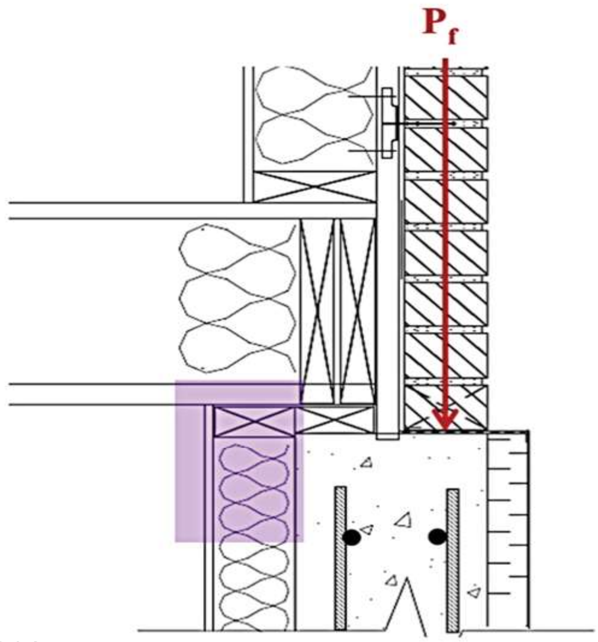
> $R_f$ = 1.60 kN **OK**

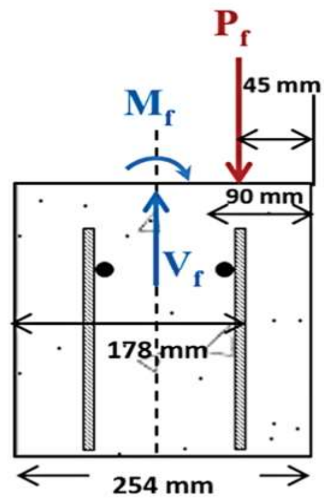
## Brick Veneer Support On a Concrete Foundation Brick Ledge

**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 7: Brick Ledge Design for Supporting Masonry Veneer




**Design Forces:**

Pf = 23.7 kN/m
Mf = 4.95 kN-m/m
Vf = 23.7 kN/m

**Reinforced Concrete Parameters:**

$\phi_c = 0.65$
$\phi_s = 0.85$
$\lambda = 1.00$
$f'_c = 20$ MPa
$\alpha_1 = 0.85 - 0.0015 \cdot f'c = 0.85 - 0.0015 \cdot (20) = 0.82$
$\beta_1 = 0.97 - 0.0025 \cdot f'c = 0.85 - 0.0025 \cdot (20) = 0.92$
$f_y = 400$ MPa
$A_s = (1000 \text{ mm} / 609.6\text{mm}) \cdot 200 \text{ mm}^2 = 328 \text{ mm}^2$
$d = 254\text{mm} – 76\text{mm} = 178\text{mm}$
$b = 1000$ mm

**Solve for the moment resistance at the factored axial load:**

$$P_{r@Pf} \Rightarrow P_r = C_r - T_r = \mathbf{23{,}700\ N/m} \Rightarrow \alpha_1 \cdot \phi_c \cdot f'_c \cdot b\,(\beta_1 \cdot c) - \phi_s \cdot f_y \cdot A_s = 23{,}700 \text{ N/m}$$
$$\Rightarrow 0.82 \cdot 0.65 \cdot 20 \text{ MPa} \cdot 1000\text{mm}\,(0.92 \cdot c) - 0.85 \cdot 400 \text{ MPa} \cdot 328 \text{ mm}^2 = 23{,}700 \text{ N}$$
$$\Rightarrow c = 13.8 \text{ mm}$$

$$M_{r@Pf} = \phi_s \cdot f_y \cdot A_s \cdot [d - a/2] \times 10^{-6} = 0.85 \cdot 400 \text{ MPa} \cdot 328 \text{ mm}^2 \cdot [178 - 13.8/2] \times 10^{-6} = \mathbf{19.1\ kN\text{-}m/m}$$
$$\mathbf{> 4.95\ kN\text{-}m/m \quad OK}$$

$$V_r = V_c = \phi_c \cdot \lambda \cdot b \cdot \sqrt{f'_c} \cdot b_w \cdot d_v \times 10^{-3} = 0.65 \cdot 1.0 \cdot 0.18 \cdot \sqrt{20 \text{ MPa}} \cdot 102\text{mm} \cdot (0.8 \cdot 1000\text{mm}) = \mathbf{53.3\ kN/m}$$
$$\mathbf{> 23.7kN/m \quad OK}$$

## 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, June 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, June 2014