Design example: Wood diaphragm on reinforced CMU shearwalls

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PROJECT DESCRIPTION

This building is a school gymnasium located in Surrey, British Columbia. The plan dimensions are 20m x 30m, with a total building height of 7m. The walls are 190 mm reinforced CMU, and the roof diaphragm consists of plywood sheathing and SPF framing members. The roof plan is shown in Figure 1.

The site is Seismic Class 'C'. Wind, snow and seismic data specific to the project location are taken from the latest version of the National Building Code (2010).

Roof dead load is assumed to be 0.9 kPa and the wall weight is 2.89 kPa. The weight of non-structural items including mechanical equipment has not been included in this example for simplicity.

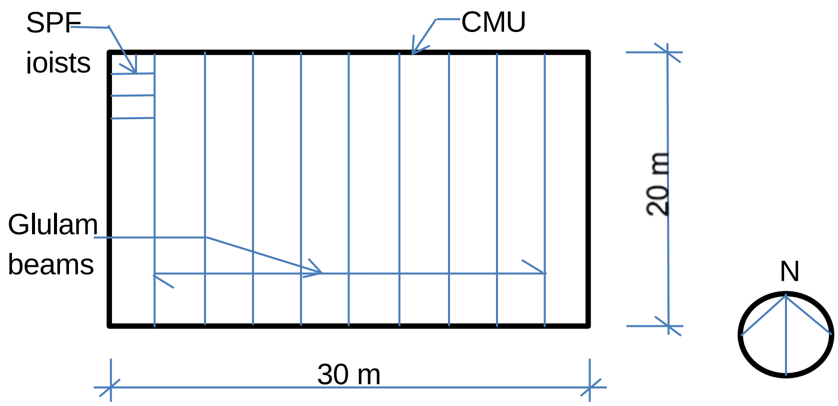


Figure 1 Roof Plan

Snow Load:

[NBCC 4.1.6.2.1)]

$S = I_s [S_s(C_bC_wC_sC_a) + S_r]$

$S_S = 2.4$ kPa, $S_r = 0.3$ kPa

$I_S = 0.9$ (SLS), $I_S = 1.15$ (ULS)

S = 1.15 x (2.4 x 0.8 x 1.0 x 1.0 x 1.0 + 0.3)
= 2.55 kPa (for strength calculation)

S = 0.9 x (2.4 x 0.8 x 1.0 x 1.0 x 1.0 + 0.3)
= 2.00 kPa (for serviceability calculation)

Seismic data and site condition:

$S_a(0.2) = 1.0$, $S_a(0.5) = 0.69$, $S_a(1.0) = 0.33$, $S_a(2.0) = 0.17$

$I_E = 1.3$ (ULS) for school

For Site Class C: $F_a = 1.0$, $F_v = 1.0$

Wind Load:

$q_{1/50} = 0.44$ kPa

Seismic forces calculation - North-South direction

Roof

w_R = (0.25 x 2.55) + 0.9 = 1.54 kPa

Total roof weight = 1.54 x 20 x 30 = 924 kN

Walls

w_w = 2.89 kPa

Assume half height of wall contributes: 0.5 x 7 x 2.89 x 2(30 + 20) = 1011 kN

Total weight = 924 + 1011 = 1935 kN

[NBCC 4.1.8.11 3) c] $T_a = 0.05 \times 7.0^{3/4} = 0.2s$

[NBCC 4.1.8.4 7)] $S(T_a) = F_a S_a(0.2)$ for $T \leq 0.2s$

$S(T_a) = 1.0 \times 1.0 = 1.0$ for $T \leq 0.2s$

$S(0.2) = F_a S_a(0.2) = 1.0$

$S(4.0) = F_v S_a(2.0)/2 = 0.085$

[NBCC 4.1.8.9] Masonry shearwalls, conventional construction → $R_d = 1.5$, $R_o = 1.5$

[NBCC 4.1.8.11.5] $M_v = 1.0$

[NBCC 4.1.8.11 2)] The minimum lateral earthquake force, $V_{N\text{-}S}$:

$$V_{N-S} = \frac{S(T_a)M_v I_E W}{R_d R_o} = \frac{1.0 \times 1.0 \times 1.3 \times W}{1.5 \times 1.5} = 0.578W$$

[NBCC 4.1.8.11 2) a] $V_{N\text{-}S}$ shall not be less than:

$$V_{N-S} = \frac{S(4.0)M_v I_E W}{R_d R_o} = \frac{0.085 \times 1.0 \times 1.3 \times W}{1.5 \times 1.5} = 0.049W$$

[NBCC 4.1.8.11 2) c] $V_{N\text{-}S}$ need not be greater than:

$$V_{N-S} = \frac{\frac{2}{3}S(0.2)I_E W}{R_d R_o} = \frac{\frac{2}{3} \times 1.0 \times 1.3 \times W}{1.5 \times 1.5} = 0.385W \qquad \text{(Governs)}$$

Therefore

$$V_{N-S} = 0.385W$$

$V_{N\text{-}S}$ = 0.385 x 1935 = 745 kN

Seismic forces calculation - East-West direction

As the SFRS is the same in the orthogonal direction, the derivation and forces are identical

$V_{E\text{-}W}$ = 0.385 x 1935 = 745 kN

Diaphragm design to CSA-O86-09

The diaphragm may be designed either to yield or not to yield. The forces for both cases will be determined, with the lower value chosen to allow the more economical choice to be made.

N-S direction

Diaphragm designed not to yield – CSA O86 Clause 9.8.5.2.2, see also Table 8.8 of Wood Design Manual

The seismic force on the diaphragm is calculated using the following formula:

$F_{D,\,roof} = F_{roof}/w_{roof} \times w_{D,roof}$,

Where,

F_{roof} = seismic force at roof level

= $V_{N\text{-}S}$ for single-storey building

W_{roof} = the weight tributary to roof level

$W_{D,roof}$ = the weight tributary to the diaphragm at roof level

= roof dead load + the weight of half height of perpendicular walls + 25% of snow load

= 924 + 0.5 x 7 x 2.89 x 2 x 30 = 1531 kN

Therefore,

$F_{D,\,roof}$ = 745 kN/ 1935 kN x 1531 kN = 589 kN = F_i

This equation is valid for a single-storey case. For a multi-storey building, the base shear needs to be vertically distributed to different levels in accordance with NBCC, and then these forces are used to evaluate the seismic design force on the diaphragms at different levels. Special attention should be paid where the diaphragm is required to transfer design seismic force from the vertical resisting elements above the diaphragm to other vertical resisting elements below the diaphragm due to offsets in the placement of the elements or to changes in relative lateral stiffness in the vertical elements. For a multi-storey building with diaphragms designed not to yield, the minimum requirement specified in NBCC 4.1.8.15 1) shall also be checked, i.e., the diaphragms shall be designed for a minimum force corresponding to the design-based shear divided by N for the diaphragm at level x, where N is the total number of storeys above exterior grade.

[CSA O86 9.8.5.2.2] Design diaphragm for force V_{Di}: $V_{Di} = Y_i \times F_i$

The resistance of 200 CMU wall reinforced with 15M @ 800 o.c. vertical is:

V_r = 48 kN/m (Sliding shear governs)

And,

Y_i = Overstrength coefficient at roof level for the vertical SFRS

= wall resistance / load on wall

= 48 x 20 / (745/2 + 5% x 745) = 2.34 (Considering 5% offset to take into account the accidental torsion for flexible diaphragm)

Therefore,

V_{Di} = 2.34 x 589 kN = 1378 kN

But V_{Di} need not exceed F_i calculated using R_dR_o =1.3, i.e.

$$V_{Di,\,max} = \frac{F_i(R_d R_o)}{1.3} = (1.5 \times 1.5 / 1.3) \times 589 = 1019 \text{ kN}$$

Therefore, use V_D = 1019 kN

This value can be compared to a diaphragm designed to yield, with the lower value chosen.

Diaphragm designed to yield – CSA O86 Clause 9.8.5.2.1, see also Table 8.8 of Wood Design Manual

$V_{Di} = F_i$, and shall not be less than the force determined using R_dR_o = 2.0.

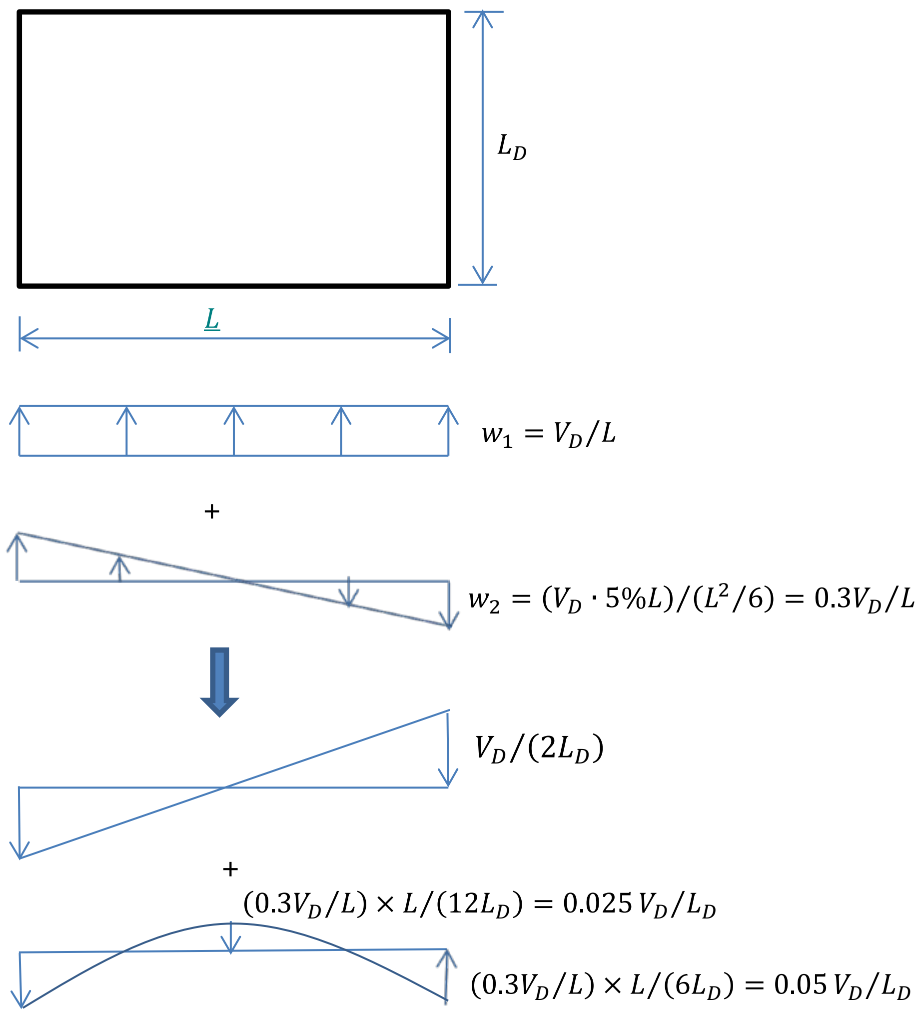
Considering R_d R_o = 2.25 is used in F_i calculation (actual SFRS value for reinforced CMU shear walls), the minimum force to be used in design shall be based on R_d R_o = 2.0.

$$V_D = \frac{F_i(R_d R_o)}{2.0} = 589 \times 2.25/2.0 = 663 \text{ kN}$$

Note: For flexible diaphragms, a parabolic seismic force distribution along the length of the diaphragm may be assumed in accordance with ASCE 41-06, which results in the same maximum shear at the edge of the diaphragm as yielded by uniform seismic force distribution; however the shear does not vary linearly.

As the force for the diaphragm designed to yield is lower, design for this value.

Calculate the maximum shear in the diaphragm as follows, taking into account 5% offset for accidental torsion.



Therefore, the maximum unit shear in the diaphragm is calculated as follows:

$$\mathrm{v_{f,max}} = (0.5 + 0.05)\,\mathrm{V_D/L_D} = 0.55 \times \frac{663}{20} = 18.2\ \mathrm{kN/m}$$

E-W direction

As diaphragm is designed to yield before the supporting SFRS, based on the N-S direction calculations,

$V_{Di} = F_i$, and shall not be less than the force determined using $R_dR_o = 2.0$.

$F_{D,\,roof} = F_{roof}/W_{roof} \times W_{D,\,roof}$ = 745 kN/ 1935 kN x 1329 kN = 512 kN = F_i

where $W_{D,\,roof}$ is based on the roof dead load and 25% of snow load and the weight of half height of the perpendicular walls.

Considering $R_d\,R_o = 2.25$ is used in F_i calculation, the minimum force to be used in design shall be based on $R_d\,R_o = 2.0$. Therefore the seismic design force on the diaphragm:

$$V_D = \frac{F_i(R_dR_o)}{2.0} = 512 \times 2.25/2.0 = 576 \text{ kN}$$

Therefore, the maximum unit shear in the diaphragm is calculated as follows:

$$v_{f,max} = (0.5 + 0.05)\, V_D/L_D = 0.55 \times \frac{576}{30} = 10.6 \text{ kN/m}$$

Design for the critical case in both directions and the load in N-S direction governs.

Use 18.5 mm Douglas fir plywood with 3" (3.66 mm diameter) nails spaced @ 64 mm o.c. at the blocked diaphragm boundaries and at continuous panel edges and 100 mm o.c. at other panel edges, 89 mm thick framing members minimum. Two lines of fasteners are required. The factored shear resistance (chosen from Wood Design Manual Diaphragm Selection Table) is:

v_r = 18.8 kN/m > 18.2 kN/m

The design of the diaphragm is summarised in Figure 2.

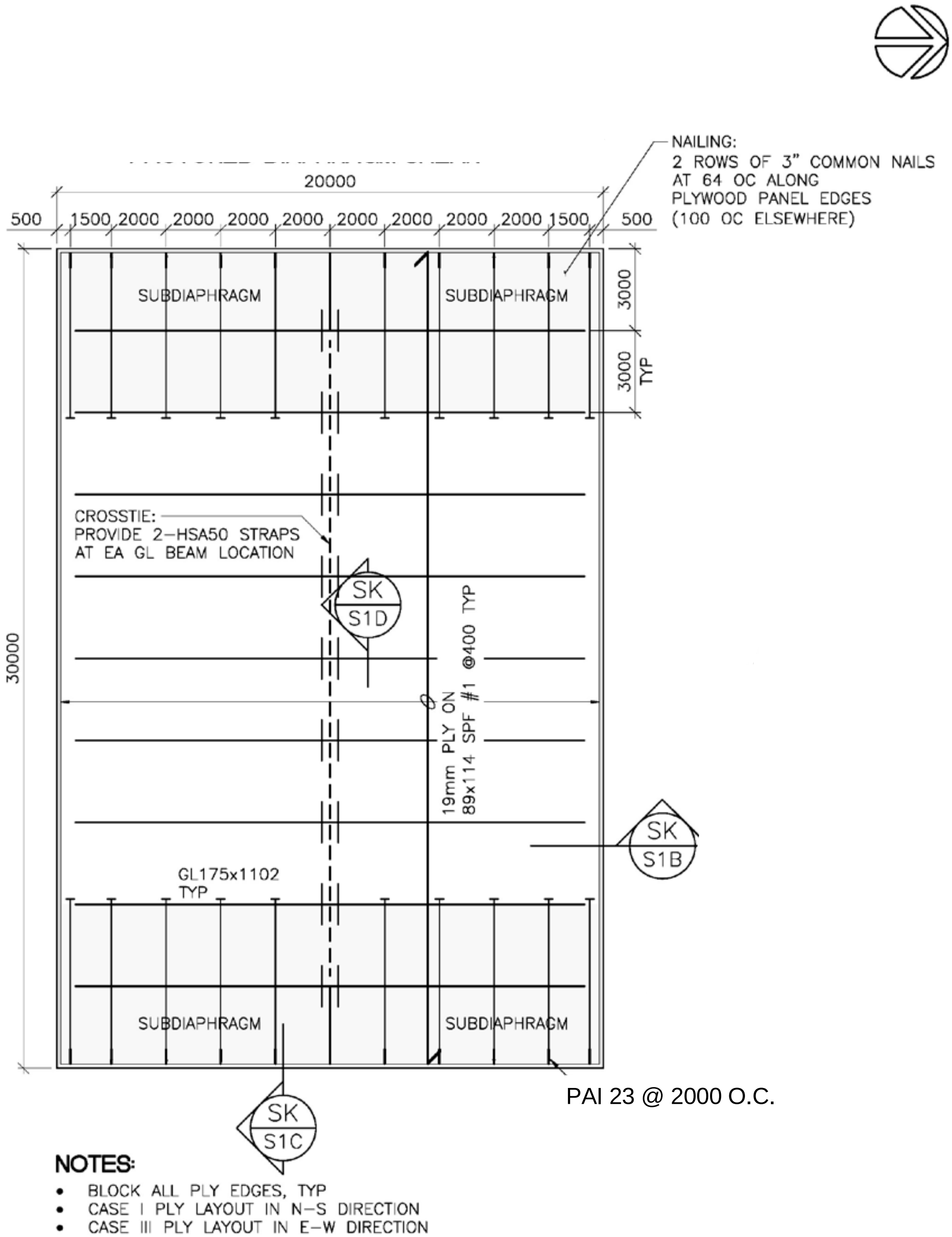


Figure 2 Plan showing diaphragm shear and design

As the shear forces carried by the diaphragm are lower towards the centre, reduced nailing patterns may be used, with the required spacing calculated based on the lower shear for different nailing zones. Similarly, a reduced ply thickness or framing member width can be obtained; in practice, however, this is not often done, as the saving in material is offset by increased time to complete more complex instructions.

Chord Member

Tension member assumed to be bond beam in CMU wall i.e. the steel reinforcement.

N-S direction

The maximum chord force is still at the mid-span, with 5% offset taken into account, and is calculated as follows:

$$\text{chord force} = \frac{M_{max}}{h} = \frac{w_1 L^2/8}{L_D} = \frac{V_D L}{8L_D} = \frac{663 \times 30}{8 \times 20} = 124\ \text{kN}$$

In accordance with Clause 9.8.6 of CSA O86, the diaphragm chords shall be designed for seismic loads that are at least 20% greater than the seismic design load on the diaphragm, and therefore multiply chord force by 1.2 :

124 kN x 1.2 = 149 kN

Try 4 - 15M,

$T_r = 0.85\ A_s\ F_y = 0.85 \times 800 \times 400 = 272\ \text{kN} > 149\ \text{kN}$

E-W direction

The maximum chord force is still at the mid-span, with 5% offset taken into account, and is calculated as follows:

$$\text{chord force} = \frac{M_{max}}{h} = \frac{w_1 L^2/8}{L_D} = \frac{V_D L}{8L_D} = \frac{576 \times 20}{8 \times 30} = 48\ \text{kN}$$

Multiply chord force by 1.2 (per CSA O86 Clause 9.8.6)

48 kN x 1.2 = 58 kN

Try 2 - 15M,

$T_r = 0.85\ A_s\ F_y = 0.85 \times 400 \times 400 = 136\ \text{kN} > 58\ \text{kN}$

The bond beam used as compression chord member shall be checked in accordance with CSA Standard S304.1, Design of Masonry Structures.

Connection to Shear Wall

N-S direction

In accordance with Clause 9.8.6 of CSA O86, the load-transfer elements shall also be designed for seismic loads that are at least 20% greater than the seismic design load on the diaphragm, and therefore

$v_f \times 1.2 = 18.2 \times 1.2 = 21.8$ kN/m

Try 13 mm Ø A307 steel grade anchor bolts @ 300 mm o.c. Minimum edge distance is 19 mm, and the minimum embedment length in CMU wall is 100 mm. The ledger is S-P-F 89 x 114. The factored resistance of the connection is:

$V_r = 7.75/ 0.3 = 25.8$ kN/m > 21.8 kN/m

E-W direction

[CSA O86 9.8.6] $v_f \times 1.2 = 10.6 \times 1.2 = 12.7$ kN/m

Try 13 mm Ø A307 steel grade anchor bolts @ 600 mm o.c. Minimum edge distance is 19 mm, and the minimum embedment length in CMU wall is 100 mm. The ledger is S-P-F 89 x 114. The factored resistance of the connection is:

$V_r = 7.75 / 0.6 = 12.9$ kN/m > 12.7 kN/m

Subdiaphragm & components

A common failure observed in past earthquake events is the separation of walls from the roof diaphragm, particularly for high mass walls such as concrete or masonry. To address this problem, the US codes have required continuous cross ties from one wall to the other parallel wall. The subdiaphragm is used to reduce the number of fasteners needed to achieve a continuous cross tie connection between parallel walls by concentrating the uniform anchorage force into main beams (APA Report: Diaphragms and Shear Walls 2001). Although not required in the Canadian Building Code and Standards, for heavy walls in particular, it is recommended to complete a sub-diaphragm check to ensure the local wall anchorage forces can be safely transferred through the connections and members to the main diaphragm.

Wall anchorage forces

In accordance with Clause 4.1.8.18 of NBCC 2010, the attached components need to be designed and detailed so that they retain their integrity and do not become detached from the structure when subjected to forces due to earthquake ground motion, and the component design force, V_p, is calculated in the following:

$V_p = 0.3\, F_a\, S_a(0.2)\, I_E\, S_p\, W_p$

$S_p = C_p\, A_r\, A_x / R_p$

where:

$C_p = 1.0$, $A_r = 1.0$, $R_p = 2.5$

$A_x = 1 + 2\ h_x/h_n = 1 + 2x7/7 = 3.0$

$S_p = 1.0 \times 1.0 \times 3.0/2.5 = 1.2$

$V_p = 0.3 \times 1.0 \times 1.0 \times 1.3 \times 1.2 \times W_P = 0.468W_P$

In accordance with Clause 9.8.6 of CSA O86, connections that are transferring shear forces between the segments of the vertical SFRS and the diaphragm shall be designed for seismic loads that are at least 20% greater than the shear force that is being transferred, and therefore:

$v_P = 1.2 \times 2.89 \times 7/2 \times 0.468 = 5.7$ kN/m

In ASCE 7 (Minimum Design Loads for Buildings and Other Structures), Clause 12.11.2.2.1 requires that in high seismic zones diaphragms shall be provided with continuous ties or struts between diaphragm chords to distribute wall anchorage forces into the diaphragms. Diaphragm connections shall be positive, mechanical, or welded. Added chords are permitted to be used to form sub-diaphragms to transmit the anchorage forces to the main continuous cross-ties. The maximum length-to-width ratio of the structural sub-diaphragm shall be 2.5 to 1. Connections and anchorages capable of resisting the prescribed forces shall be provided between the diaphragm and the attached components. Connections shall extend into the diaphragm a sufficient distance to develop the force transferred into the diaphragm. In particular in wood diaphragms the continuous ties shall be in addition to the diaphragm sheathing.

In the North-South direction, the wall anchorage forces can be transferred into the diaphragm through the glulam beams that span the full depth of the diaphragm (20m) and are closely spaced. Every glulam beam must carry a tension force of 5.7 x 3 = 17.1 kN/tie. A proprietary purlin anchor PAI 23 (Tr = 17.22 kN) is used to tie the wall to the sheathing, as shown in Figure 3. Note that because the anchor spacing exceeds 4-ft (1,219 mm) the CMU wall should be designed for bending in accordance with Clause 12.11.2.1 of ASCE 7, but it is not addressed in this example.

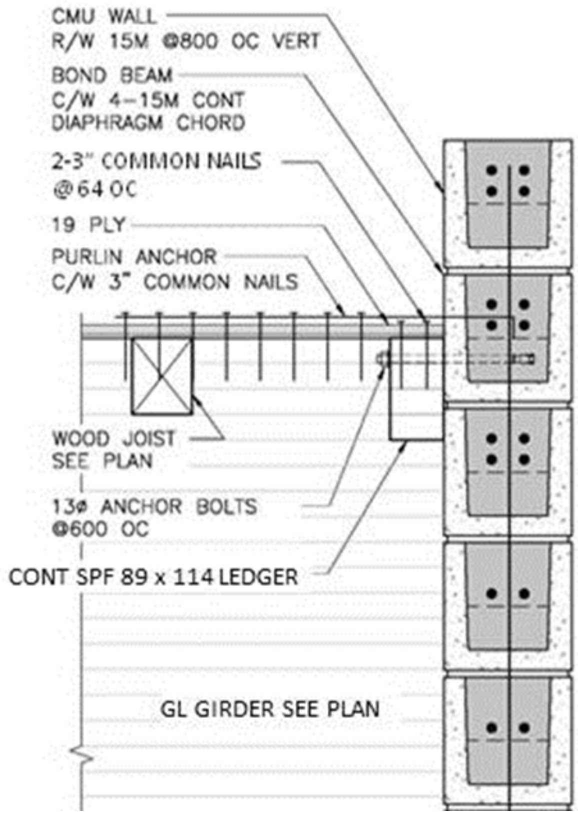


Figure 3 North wall anchorage details

In the E-W direction, continuous cross-ties across the building width (30 m) between diaphragm chords are not present. Two solutions are considered:

1. Connect all or some of the joists across the glulam beams from East to West for the full width of the building.
2. Design a sub-diaphragm to transfer the force to a number of main cross-ties.

By inspection, solution 1 will require 11 connections per joist, and will be labour intensive. Designing a subdiaphragm will likely present a more cost-effective solution.

Each subdiaphragm must meet all of the diaphragm requirements. Based on the maximum length-to-width ratio of 2.5:1 for subdiaphragm, a sub-diaphragm for the entire depth of the building could be selected, with a chord at 8 m minimum from the CMU wall; however, as there is no existing chord at 8 m, two subdiaphragms of 6 m x 10 m will be used, and the length-to-width ratio is 1.7. This uses the existing glulam beam at 6 m as one of the chords of the subdiaphragm, with the bond beam in the CMU wall as the other.

A continuous cross-tie at mid-depth from East to West will be required based on this solution, as outlined below.

Wall anchorage check

Assuming the reinforced CMU wall will span between points of support at 2 m o.c., and since the anchor spacing exceeds 4 ft (1,219 mm) the CMU wall needs to be designed to resist bending between anchors. Every fifth SPF No.1 grade 89 x 114 joist must carry a tension force of 5.7 kN x 2 m = 11.4 kN/tie. One PAI 23 purlin anchor (T_r = 17.22 kN) per tension joist is adequate to transfer this load into the joist. By inspection, the joist itself has adequate capacity in tension; however, the effect of combined tension and bending (due to gravity loads) should be verified. The ASCE requirement to provide continuous crossties between diaphragm chords applies to subdiaphragms as well. As the subdiaphragm extends across two bays, the tension force must be transferred across the glulam beam to the second joist – a MST 37 tie (T_r = 17.62 kN) is adequate. See Figure 4. The required anchorage force of 11.4 kN must be transferred into roof sheathing over the depth of the subdiaphragm. This requires shear transfer capacity of 11.4/6 =1.9 kN/m along the length of subdiaphragm. The maximum nail spacing based on the main diaphragm design is 100 mm o.c. and there are two lines of fasteners, therefore the shear transfer of 95 N/nail is required. This requirement is satisfied by inspection.

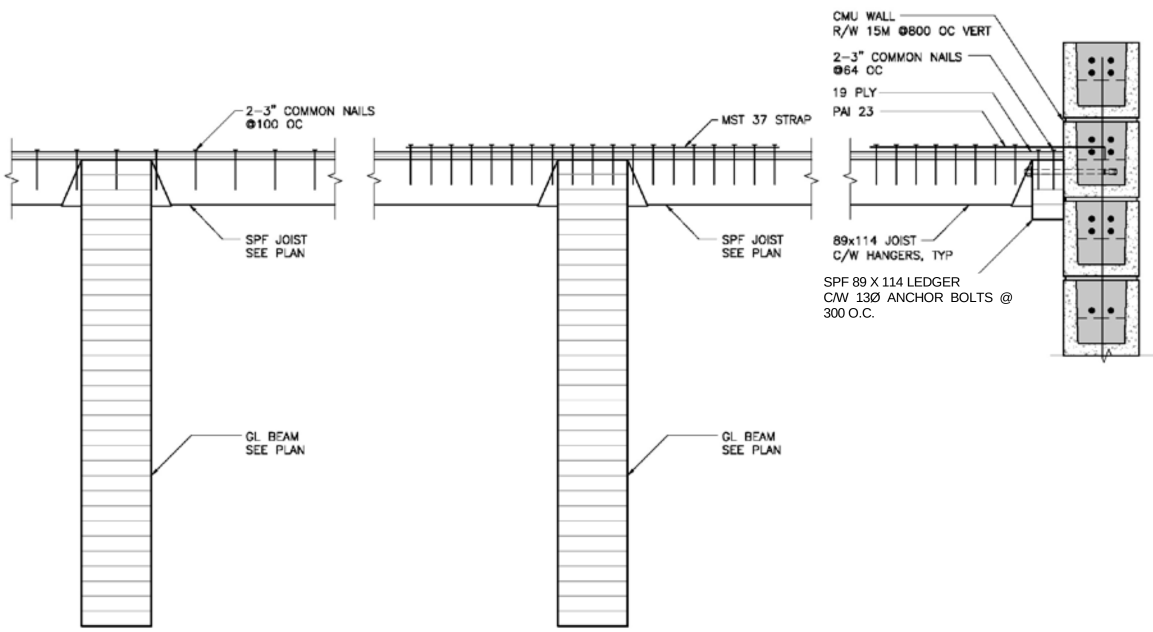


Figure 4 Subdiaphragm and connections

Subdiaphragm shear check

Each subdiaphragm must be checked for subdiaphragm shear – (5.7 x 10)/ (2 x 6) = 4.75 kN/m. Note that the nail spacing at subdiaphragm boundaries is 64 mm or 100 mm o.c. based on the main diaphragm design. By inspection, the factored shear resistance of subdiaphragm is sufficient. Therefore, the main diaphragm sheathing will perform adequately as subdiaphragm sheathing and no further checks of the subdiaphragm sheathing are required.

Subdiaphragm chord check

Subdiaphragm chord force is given by $(5.7 \times 10^2)/(8 \times 6) = 11.9$ kN. By inspection, the glulam beam and bond beam have adequate capacity for this tension/compression force; however, these glulam members should be checked for combined bending and axial load (from roof gravity and subdiaphragm chord loads, respectively). Similarly, as these members extend across the full subdiaphragm width (10 m), there are no splices. Were this not the case, a splice would be required.

Crosstie check

The crosstie at its connection to the CMU wall carries the same load as for other ties, i.e. 11.4 kN/tie, but increases over the depth of the subdiaphragm to a tension force of $(5.7 \times 10 / 2) \times 2 = 57$ kN. This force must be transferred across the building width, requiring 9 across-beam connections and 2 wall-to-tie connections. As mentioned above, the end connection for the crosstie is as for other ties and a PAI 23 purlin anchor will be sufficient.

For the internal connections, to transfer the tension load between joists across the glulam beam, 2 HSA50 strap connections (capacity = 2 x 28.5 = 57 kN) per connection, with 1 strap each side of the tension tie member, will be sufficient – see Figure 5. The SPF 89 x 114 joist similarly has adequate tension capacity for this force; however, the force should be checked in combination with vertical loading, per NBCC 2010.

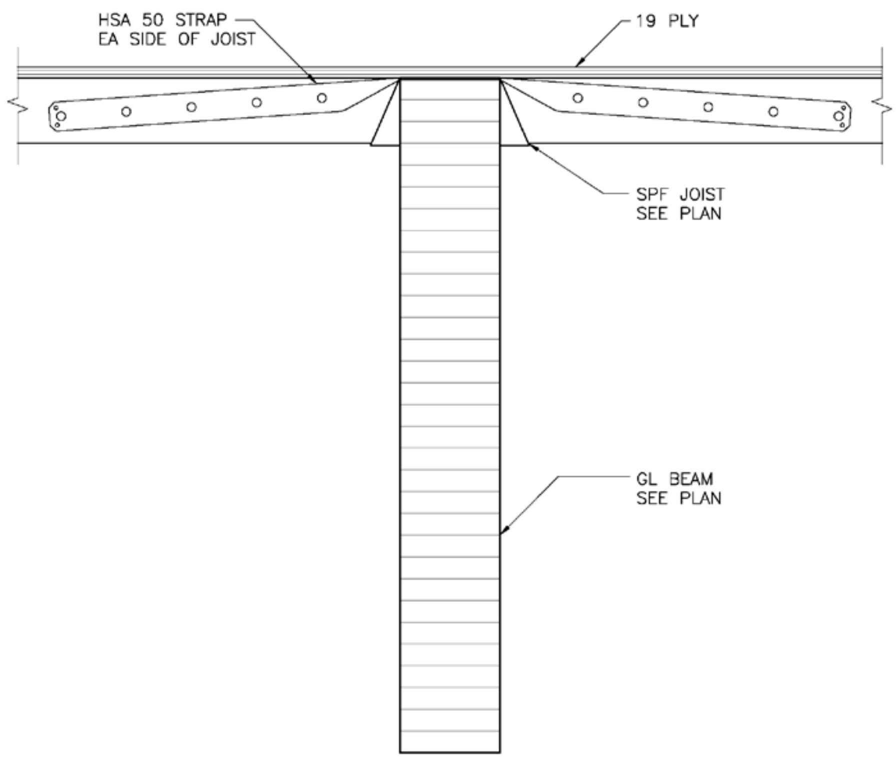


Figure 5 Crosstie connection detail

Reference:

ASCE 7-10. Minimum Design Loads for Buildings and Other Structures. American Society of Civil Engineers. Reston, Virginia, U.S.A.

Breyer, D.E.; Fridley, K.J.; Pollock, D.G.; and Cobeen, K.E. (2006) Design of wood structures – ASD, Sixth Edition. McGraw-Hill, New York, NY, U.S.A.

Lateral load connections for low-slope roof diaphragms. APA Data File. APA, Tacoma, WA, U.S.A.

Diaphragms and Shearwalls. APA Report - L350. APA, Tacoma, WA, U.S.A.

APPENDIX

Deflection calculation

In accordance with CSA O86 Clause 9.7.2 the lateral deflection at mid-span of a simply supported blocked wood diaphragm may be taken as follows:

$$\Delta_d = \frac{5vL^3}{96EAL_D} + \frac{vL}{4B_v} + 0.000614Le_n + \frac{\sum(\Delta_c x)}{2L_D}$$

This equation was developed based on top plate being tension and compression chords, and is valid only for blocked diaphragms without openings, assuming the lateral load is uniformly distributed. The first term of the equation has been modified in this instance, as the chords of the diaphragm are not 'equal' - that is, the compression chord is concrete masonry unit, while the tension chord is steel reinforcement. Without accidental torsion taken into account, the following relationship holds:

$$w_1 = 2vL_D/L$$

The first term becomes:

$$\Delta_1 = \frac{5w_1L^4}{384EI} = \frac{5vL^3L_D}{192(EI)_{eff}}$$

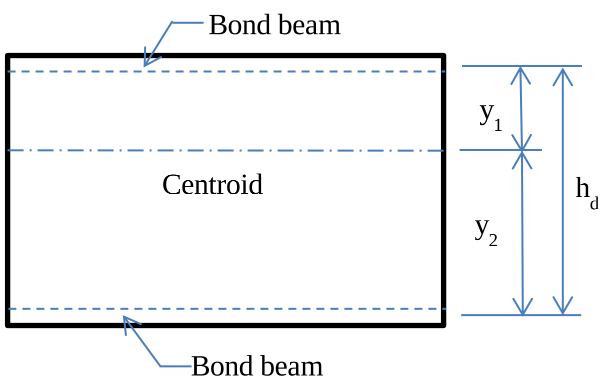
Where $(EI)_{eff}$ is the effective stiffness of this cross section.

N-S Direction

Assume one block height (200 mm) of bond beam contributes:

f_m' = 10 Mpa, from Table 4 of S304.1; E_m = 850 x 10 = 8 500 MPa; E_s = 200 000 MPa

A_{block} = 200 x 190 = 38 000 mm^2; A_{steel} = 800 mm^2



$$n = \frac{E_s}{E_m} = \frac{200{,}000}{8{,}500} = 23.53$$

$$h_d = L_D - 190 = 20{,}000 - 190 = 19{,}810 \text{ mm}$$

$$y_1 = \frac{nA_{steel} \cdot h_d}{nA_{steel} + A_{block}} = \frac{23.53 \times 800 \times 19{,}810}{23.53 \times 800 + 38{,}000} = 6{,}562 \text{ mm}$$

$$y_2 = h_d - y_1 = 19{,}810 - 6{,}562 = 13{,}248 \text{ mm}$$

$$(EI)_{eff} = E_m A_{block} \cdot y_1^2 + E_s A_{steel} \cdot y_2^2 = 4.20 \times 10^{16} \text{ N} \cdot \text{mm}^2$$

$$v = \frac{V_D}{2L_D} = 0.5 \times \frac{663}{20} = 16.6 \text{ kN/m}$$

$$\Delta_{d,N-S} = \frac{5 \times 16.6 \times 30000^3 \times 20000}{192 \times 4.20 \times 10^{16}} + \frac{16.6 \times 30000}{4 \times 9800} + 0.000614 \times 30000 \times 1.13 + 0$$

$$= 39 \text{ mm}$$

Where e_n = 1.13 mm for 3" common nail, based on load per nail = 16.6 kN/m x 0.064 m = 1062 N [CSA O86 Table A.9.7]

E-W Direction

Similarly,

$$h_d = 30{,}000 - 190 = 29{,}810 \text{ mm}$$
$$y_1 = 9{,}875 \text{ mm}$$
$$y_2 = 19{,}935 \text{ mm}$$
$$(EI)_{eff} = 9.51 \times 10^{16} \text{ N} \cdot \text{mm}^2$$

$$v = \frac{V_D}{2L_D} = 0.5 \times \frac{576}{30} = 9.6 \text{ kN/m}$$

$$\Delta_{d,E-W} = \frac{5 \times 9.6 \times 20000^3 \times 30000}{192 \times 9.51 \times 10^{16}} + \frac{9.6 \times 20000}{4 \times 9800} + 0.000614 \times 20000 \times 0.344 + 0$$
$$= 9.8 \text{ mm}$$

Where e_n = 0.344 mm for 3" common nail, based on load per nail = 9.6 kN/m x 0.064 m = 614 N [CSA O86 Table A. 9.7]

The deflection of the diaphragm shall not exceed the permissible deflection of the perpendicular CMU walls in N-S and E-W direction respectively**.**

Design example: Wood diaphragm on reinforced CMU shearwalls