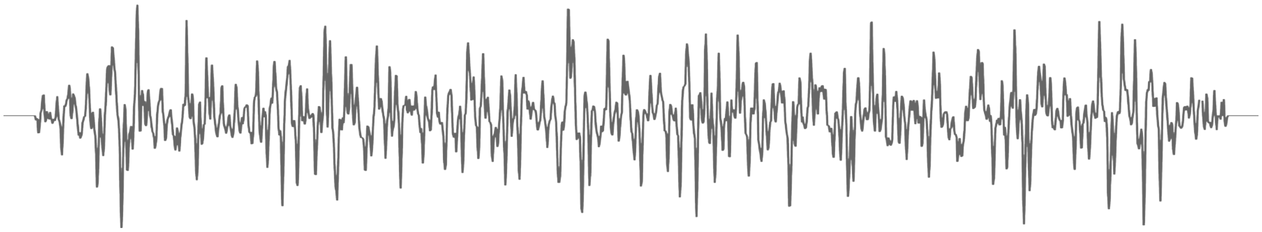
SEISMIC DESIGN GUIDE FOR MASONRY BUILDINGS

Second Edition



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Canadian Concrete Masonry Producers Association
Region 6 of the National Concrete Masonry Association
CCMPA
Building Strength

2018

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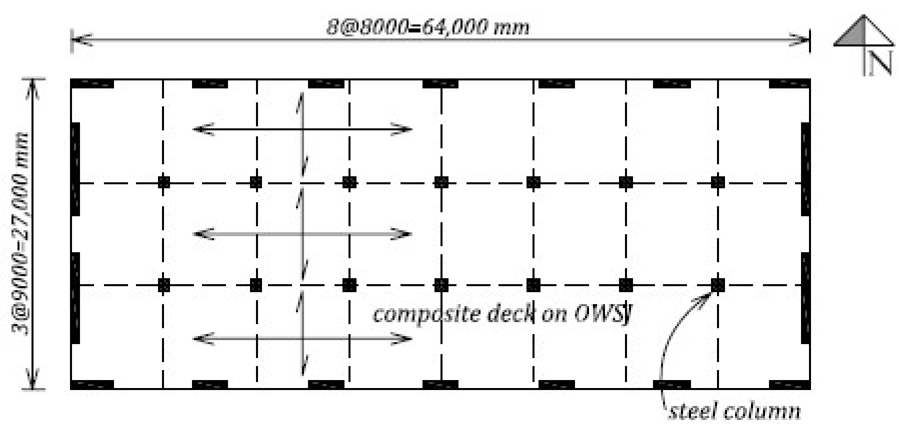
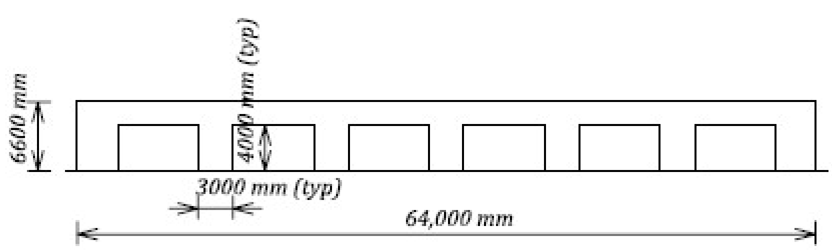
<u>EXAMPLE 1</u>: Seismic load calculation for a low-rise masonry building to NBC 2015

Consider a single-storey warehouse building located in Niagara Falls, Ontario. The building plan dimensions are 64 m length by 27 m width, as shown on the figure below. The roof structure consists of steel beams, open web steel joists, and a composite steel and concrete deck with 70 mm concrete topping. The roof is supported by 190 mm reinforced block masonry walls at the perimeter and interior steel columns. The roof elevation is 6.6 m above the foundation. The soil at the building site is classed as a Site Class D per NBC 2015.

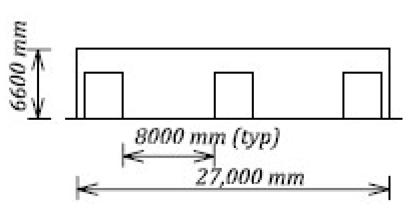
Calculate the seismic base shear force for this building to NBC 2015 seismic requirements (considering the masonry walls to be detailed as “conventional construction”). Next, determine the seismic shear forces in the walls, including the effect of accidental torsional eccentricity. Assume that the roof acts like a rigid diaphragm.



Plan



North and South Elevations



East and West Elevations

<u>SOLUTION:</u>

1. Calculate the seismic weight W (NBC 2015 Cl.4.1.8.2)

a) Roof loads:

- Snow load (Niagara Falls, ON) W_s = 0.25*(1.8*0.8+0.4)= 0.46 kPa

(25% of the total snow load is used for the seismic weight)

- Roof self-weight (including beams, trusses, steel deck, roofing, insulation, and 65 mm concrete topping) W_D = 3.30 kPa

Total roof seismic weight W_{roof} = (0.46kPa+3.30kPa)(64.0m*27.0m)= 6497 kN

b) Wall weight:

Assume solid grouted walls w = 4.0 kN/m^2

(this is a conservative assumption and could be changed later if it is determined that partially grouted walls would be adequate)

The usual assumption is that the weight of all the walls above wall midheight is part of the seismic weight (mass) that responds to the ground motion and contributes to the total base shear.

Tributary wall surface area:

- North face elevation = 0.5*7*3.0m*6.6m + (64m-7*3m)*(6.6m-4.0m)= 181.1 m^2
- South face elevation (same as north face elevation) = 181.1 m^2
- East face elevation = 0.5*2*8.0m*6.6m + (27m-2*8m)*(6.6m-4.0m) = 81.4 m^2
- West face elevation (same as east face elevation) = 81.4 m^2

Total tributary wall area $Area$ = 525.0 m^2

Total wall seismic weight $W_{wall} = w * Area$ = 4.0*525.0= 2100 kN

The total seismic weight is equal to the sum of roof weight and the wall weight, that is,

$$W = W_{roof} + W_{wall} = 6497+2100= 8597 \text{ kN} \approx 8600 \text{ kN}$$

2. Determine the seismic hazard for the site (see Section 1.4).

- Location: Niagara Falls, ON (see NBC 2015 Appendix C)

  $S_a(0.2)$ = 0.321

  $S_a(0.5)$ = 0.157

  $S_a(1.0)$ = 0.072

  $S_a(2.0)$ = 0.032

  $S_a(5.0)$ = 0.0076

  PGA_{ref} = 0.207

- Foundation factor – Site Class D and PGA_{ref} = 0.207 (see Tables 1-3 to 1-7)

  $F(0.2)$ = 1.09

  $F(0.5)$ = 1.30

  $F(1.0)$ = 1.39

  $F(2.0)$ = 1.44

  $F(5.0)$ = 1.48

- Site design spectrum $S(T)$ (see Section 1.4)

For T =0.2 sec: $S(0.2) = F(0.2) \bullet S_a(0.2)$ = 1.09*0.321=0.35 $\boxed{S(0.2)=0.35}$

or $S(0.5) = F(0.5) \bullet S_a(0.5)$ =1.3*0.157=0.20 (larger value governs)

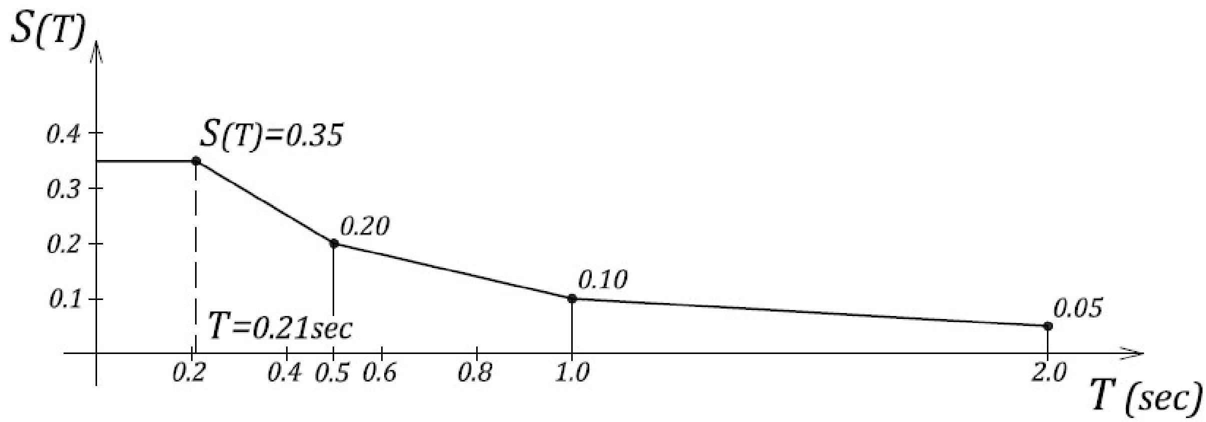
For T =0.5 sec: $S(0.5) = F(0.5) \bullet S_a(0.5)$ =1.3*0.157=0.20 $\boxed{S(0.5)=0.20}$

For T =1.0 sec $S(1.0) = F(1.0) \bullet S_a(1.0)$ =1.39*0.072=0.10 $\boxed{S(1.0)=0.10}$

For T =2.0 sec $S(2.0) = F(2.0) \bullet S_a(2.0)$ =1.44*0.032=0.046 $\boxed{S(2.0)=0.05}$

For T =5.0 sec $S(5.0) = F(5.0) \bullet S_a(5.0)$ =1.48*0.0076=0.011 $\boxed{S(5.0)=0.01}$

The site design spectrum $S(T)$ is shown below.



- Building period (T) calculation (see Section 1.6 and NBC 2015 Cl.4.1.8.11(3).c) for wall structures)

h_n = 6.6 m building height

$T = 0.05(h_n)^{3/4}$ = 0.21 sec

Then interpolate between $S(0.2)$ and $S(0.5)$ to determine the design spectral acceleration:

$S(T)$= $S(0.21)$= 0.35

3. Compute the seismic base shear (see Section 1.6)

The base shear is given by the expression (NBC 2015 Cl.4.1.8.11)

$$V = \frac{S(T)M_v I_E}{R_d R_o} W$$

where

I_E = 1.0 (building importance factor, equal to 1.0 for normal importance, 1.3 for high importance, and 1.5 for post-disaster buildings)

M_v = 1.0 (higher mode factor, equal to 1.0 for $T \leq 1.0$ sec, that is, most low-rise masonry buildings)

Building SFRS description: masonry structure – conventional construction (see Table 1-13 or NBC 2015 Table 4.1.8.9), hence R_d = 1.5 and R_o = 1.5

The design base shear V is given by:

$$V = \frac{S(T) M_v I_E}{R_d R_o} W = \frac{0.35*1.0*1.0}{1.5*1.5} W = 0.16W$$

but should not be less than

$$V_{\min} = \frac{S(4.0) M_v I_E W}{R_d R_o} = \frac{0.023*1.0*1.0}{1.5*1.5} W = 0.001W$$

Note that $S(4.0)$ value (0.023) was obtained by interpolation from the site design spectrum chart $S(T)$.

The design base shear V need not be taken more than greater of the following two values:

$$V_{\max} = \left(\frac{2S(0.2)}{3}\right)\left(\frac{I_E W}{R_d R_o}\right) = \left(\frac{2*0.35}{3}\right)\left(\frac{1.0}{1.5*1.5}\right) W = 0.10W$$, provided $R_d \geq 1.5$.

And

$$V_{\max} = S(0.5)\left(\frac{I_E W}{R_d R_o}\right) = 0.20\left(\frac{1.0}{1.5*1.5}\right) W = 0.09W$$

The upper limit on the design seismic base shear governs and therefore

$V = 0.10W = 0.10*8600 = 860$ kN

Note that the upper limit on the base shear is often going to govern for low-rise masonry structures which have low fundamental periods. The lower bound value would generally only apply to very tall buildings.

4. Determine if the equivalent static procedure can be used (see Section 1.6 and NBC 2015 Cl. 4.1.8.7).

According to the NBC 2015, the dynamic method is the default method of determining member forces and deflections, but the equivalent static method can be used if the structure meets <u>any</u> of the following criteria:

(a) is located in a region of low seismic activity where the seismic hazard index $I_E F_a S_a(0.2) < 0.35$.

In this case, the seismic hazard index is $I_E F_a S_a(0.2)$=1.0*1.09*0.321=0.35 since $F_a = F(0.2) = 1.09$.

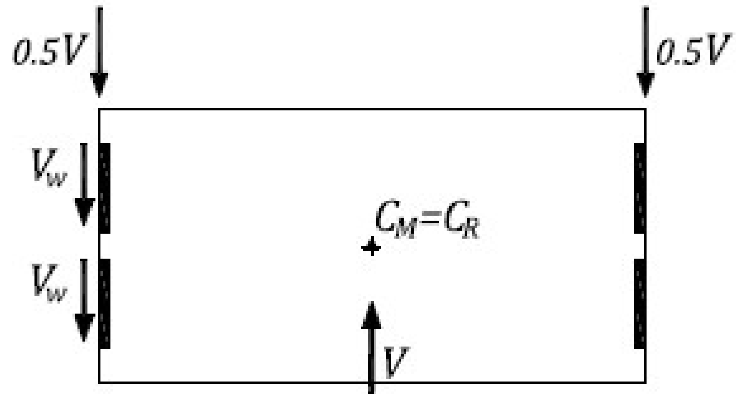
(b) is a regular structure less than 60 m in height with period T ***< 2 seconds in either direction.***

This building is clearly less than 60 m in height and the period T < 2 sec (as discussed above). A structure is considered to be regular if it has none of the irregularities discussed in Table 1-16 of Section 1.12.1. A single storey structure by definition will not have any irregularities of Type 1 to 6. It does not have a Type 8 irregularity (non-orthogonal system) but could have a Type 7 irregularity (torsional sensitivity), and so this criterion may or may not be satisfied, depending on the torsional sensitivity.

(c) has any type of irregularity, other than Type 7 and Type 9, and is less than 20 m in height with period T ***< 0.5 seconds in either direction.***

This structure satisfies the height and period criteria.

Since the criterion c) has been satisfied, the design can proceed by using the equivalent static analysis procedure. It will be shown later that, even when using a conservative assumption, the torsional sensitivity parameter B=1.2<1.7. Thus criterion b) would also be satisfied. For



structures with the lateral resisting elements distributed around the perimeter walls the B value will almost always be less than 1.7.

5. Distribute the base shear force to the individual walls.
In this example, the structure is symmetric in each direction and so the centre of mass, C_M, and the centre of resistance, C_R, coincide at the geometric centre of the structure. One might argue that in this simple system with walls at only each side of the building, the system is statically determinate in each direction and the total shear on each side can be determined using statics. However, how much shear goes to each of the walls on a side depends on the relative stiffness of the walls, although once yielding occurs the force on each wall depends on the yield strength of the wall.

a) Seismic forces in the N-S direction - no torsional effects (seismic force is assumed to act through the centre of resistance)
Since it is assumed that the roof diaphragm is rigid, the forces are distributed to the walls in proportion to wall stiffness. All walls in the N-S direction have the same geometry (height, length, thickness) and mechanical properties and it can be concluded that these walls have the same stiffness.

As a result, equal shear force will be developed at each side. The force per side is equal to (see the figure):
$0.5V = 0.5*860 = 430$ kN
So, shear force in each of the two walls in the N-S direction is equal to:
$$V_V = \frac{0.5V}{2} = \frac{430}{2} = 215 \text{ kN}$$

b) Seismic forces in the N-S direction taking into account the effect of accidental torsion
The building is symmetrical in plan and so the centre of mass C_M coincides with the centre of resistance C_R (see Section 1.11 for more details on torsional effects). Therefore, there are no actual torsional effects in this building. However, NBC 2015 Cl.4.1.8.11.(9) requires that torsional moments (torques) due to accidental eccentricities must be taken into account in the design. The forces due to accidental torsion can be determined by applying the seismic force at a point offset from the C_R by an accidental eccentricity $e_a = 0.1D_{nx}$, thereby causing the torsional moments equal to
$T_x = \pm V(0.1D_{nx}) = \pm 860*(0.1*64.0) = \pm 5504$ kNm
Note that $D_{nx} = 64.0$ m (equal to the total length of the structure in the East/West direction).

As a result of the accidental torsion, seismic shear forces resisted by each side of the building are different. These forces can be calculated by taking the sum of moments around the C_R (torsional moment created by force must be equal to the sum of moments created by the side forces). The resulting end forces are equal to $0.6V$ and $0.4V$, thereby indicating an increase in the end forces by $0.1V$ due to accidental torsion.

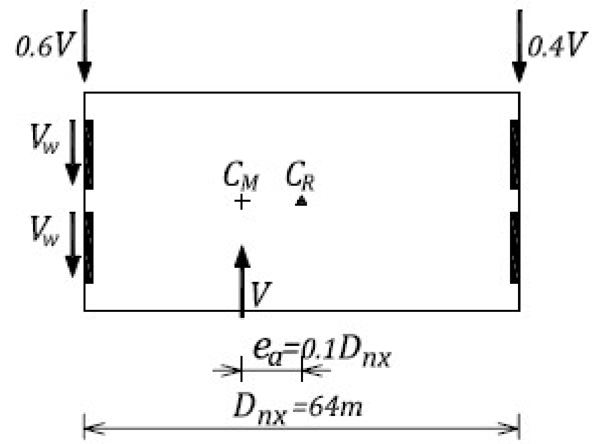
It should be noted that, in this example, accidental torsion would cause forces in the E-W walls as well because of the rigid diaphragm. But a conservative approach is to ignore the contribution of E-W walls and take all the torsional forces on the N-S walls.

The shear force in each N-S wall from accidental torsion is equal to:

$$V_T = \frac{T/D_{nx}}{2} = \frac{5504/64}{2} = 43 \text{ kN}$$

Thus, the maximum shear force in each of the two walls is the sum of the lateral component plus the torsional force,

$$V_W = V_V + V_T = 215 + 43 = 258 \text{ kN}$$



Note that the same result could be obtained by applying the lateral load through a point equal to the accidental eccentricity to one side of the centre of rigidity and then solving for the wall forces using statics (see the figure). This would show that

$$V_W = \frac{V}{2} * 0.6 = \frac{860}{2} * 0.6 = 258 \text{ kN}$$

Therefore, even though this building is symmetrical in plan, the accidental torsion causes increased seismic shear force in each wall of 43 kN, corresponding to a 20% increase compared to the design without torsion. However, this is based on the assumption that the N-S walls resist all the torsion. Walls in the E-W direction would also resist the torsional forces, and in this example the contribution to total torsional stiffness would be roughly the same for the E-W and N-S walls. Thus, one could reduce the torsional forces on the N-S walls by roughly one half.

c) Seismic forces in the E-W walls

Seismic forces in the E-W walls can be determined in a similar manner. Since all walls in the E-W direction have the same geometry (height, length, thickness) and mechanical properties and consequently the same stiffness, the shear force will be equal at the East and West side. The force per side is equal to

$$0.5V = 0.5 * 860 = 430 \text{ kN}$$

- Seismic forces in the E-W walls – torsional effects ignored

Shear force in each E-W wall is equal to (there are seven walls per side):

$$V_V = \frac{0.5V}{7} = \frac{430}{7} = 61 \text{ kN}$$

- Seismic forces in the E-W walls – torsional effects considered:

$$V_W = \frac{V}{7} * 0.6 = \frac{860}{7} * 0.6 = 74 \text{ kN}$$

6. Check whether the structure is torsionally sensitive (see Section 1.11.2).

NBC 2015 Cl. 4.1.8.11(10) requires that the torsional sensitivity B of the structure be determined by comparing the maximum horizontal displacement anywhere on a storey, to the average displacement of that storey. Torsional sensitivity is determined in a similar manner as the effect of accidental torsion, that is, by applying a set of a set of lateral forces at a distance of $\pm 0.1D_{nx}$ from the centre of mass C_M. In case of a rigid diaphragm, displacements are proportional to the forces developed in the walls. Therefore, B can be determined by comparing the forces at the sides of the building with/without the effect of accidental torsion.

The maximum displacement would be proportional to 0.6V, while the displacement on the other side would be proportional to 0.4V. Thus, the average displacement is proportional to 0.5V. Thus

$$B = \frac{0.6V}{0.5V} = 1.2$$

Since B < 1.7, this building is not torsionally sensitive and the equivalent static analysis would have also been allowed under criterion b) as discussed in step 4 above.

7. Discussion

It was assumed at the beginning of this example that the roof structure can be modeled like a rigid diaphragm. If this roof was modeled like a flexible diaphragm, the shear forces in each N-S wall would be equal to 0.5V. From a reliability point of view, it does not seem quite right that the forces are smaller for a flexible diaphragm than a rigid one - it should be the other way around. On the other hand, the flexible diaphragm may have a longer period and the forces would be smaller (see Example 3 for a detailed discussion on rigid and flexible diaphragm models).

<u>EXAMPLE 2</u>: Seismic load calculation for a medium-rise masonry building to NBC 2015

A typical floor plan and vertical elevation are shown below for a four-storey mixed use (commercial/residential) building located at Abbotsford, BC. The ground floor is commercial with a reinforced concrete slab separating it from the residential floors, which have lighter floor system consisting of steel joists supporting a composite steel and concrete deck. The front of the building is mostly glazing, which has no structural application.

First, determine the seismic force for this building according to the NBC 2015 equivalent static force procedure, and a vertical force distribution in the E-W direction. Find the base shear and overturning moment in the E-W walls. Assume that the floors act as rigid diaphragms and that the strong N-S walls can resist the torsion.

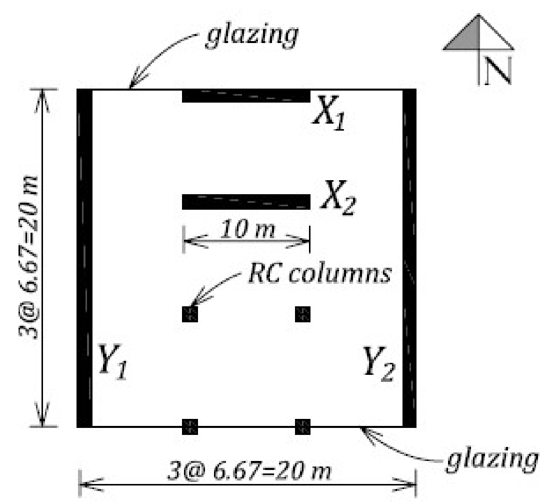
Next, consider the torsional effects in all walls and find the forces in the E-W walls. Compare the seismic forces obtained with and without torsional effects.

For the purpose of weight calculations, use 200 mm blocks for N-S walls and 300 mm blocks for E-W walls. All walls are solid grouted (this is a conservative assumption appropriate for a preliminary design) and the compressive strength f'_m is 10.0 MPa. Grade 400 steel has been used for the reinforcement. The building is of normal importance and is supported on Class C soil. Consider Conventional Construction reinforced masonry shear walls.

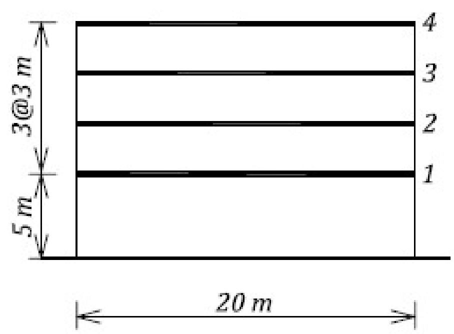
Movement joints are not to be considered in this example. Note that movement joints in the N-S walls would have caused slight changes in the stiffness values of these walls.

Specified loads (note that roof and floor loads include a 1 kPa allowance for partition walls and glazing):
4^{th} floor (roof level) = 3 kPa
2^{nd} and 3^{rd} floor = 4 kPa
1^{st} floor (concrete floor) = 6 kPa
25% snow load = 0.4 kPa

Note: 1 kPa = 1 kN/m^2



Plan



Elevation

SOLUTION:

1. Design assumptions

- Rigid diaphragm
- All walls are solid grouted

2. Calculate the seismic weight W (see NBC 2015 Cl.4.1.8.2)

Wall weight:

N-S walls - 200 mm thick w = 4.18 kPa

E-W walls – 300 mm thick w = 6.38 kPa

Note that, for the purpose of seismic weight calculations, the length of a N-S wall is 20 m, while the length of an E-W wall is 10.0 m.

Seismic weight W_1:

$$W_1 = \left(\frac{5.0m}{2} + \frac{3.0m}{2}\right)(4.18kPa*2*20m + 6.38kPa*2*10.0m) + (6.0kPa)(20m*20m) = 3579kN$$

Seismic weight W_2:

$$W_2 = \left(\frac{3.0m}{2} + \frac{3.0m}{2}\right)(4.18kPa*2*20m + 6.38kPa*2*10.0m) + (4.0kPa)(20m*20m) = 2484kN$$

Seismic weight W_3 (same as W_2) :

$$W_3 = 2484kN$$

Seismic weight W_4:

$$W_4 = \left(\frac{3.0m}{2}\right)(4.18kPa*2*20m + 6.38kPa*2*10.0m) + (3.0kPa + 0.4kPa)(20m*20m) = 1802kN$$

Note that the seismic weight for each floor level is the sum of the wall weights and the floor weight. 25% snow load was included in the roof weight calculation. One-half of the wall height (below and above a certain floor level) was considered in the wall area calculations.

The total seismic weight is equal to

$$W = W_1 + W_2 + W_3 + W_4 = 3579 + 2484 + 2484 + 1802 \cong 10350kN$$

3. Calculate the seismic base shear force (see Section 1.6).

a) Find seismic design parameters used to determine seismic base shear.

- Location: Abbotsford, BC (see NBC 2015 Appendix C)
  - $S_a(0.2)$ = 0.701
  - $S_a(0.5)$ = 0.597
  - $S_a(1.0)$ = 0.350
  - $S_a(2.0)$ = 0.215
  - $S_a(5.0)$ = 0.071
  - PGA_{ref} = 0.306
- Foundation factor – Site Class C and PGA_{ref} = 0.306 (see Tables 1-3 to 1-7)
  - $F(0.2) = F(0.5) = F(1.0) = F(2.0) = F(5.0)$ = 1.0
- Site design spectrum $S(T)$ (see Section 1.4)

For T =0.2 sec: $S(0.2) = F(0.2) \bullet S_a(0.2)$ = 1.0*0.701=0.70 $\boxed{S(0.2)=0.70}$

or $S(0.5) = F(0.5) \bullet S_a(0.5)$ =1.0*0.597=0.60 (larger value governs)

For T =0.5 sec: $S(0.5) = F(0.5) \bullet S_a(0.5)$ =1.0*0.597=0.60 $\boxed{S(0.5)=0.60}$

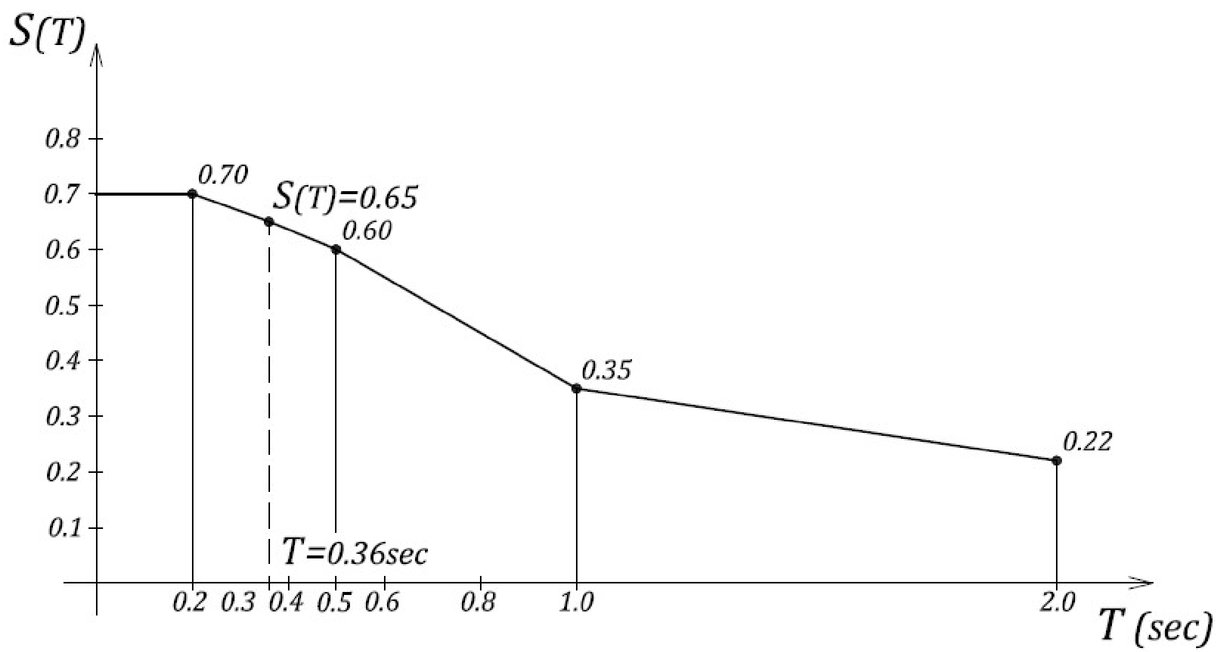
For T =1.0 sec $S(1.0) = F(1.0) \bullet S_a(1.0)$ =1.0*0.35=0.35 $\boxed{S(1.0)=0.35}$

For T =2.0 sec $S(2.0) = F(2.0) \bullet S_a(2.0)$ =1.0*0.215=0.22 $\boxed{S(2.0)=0.22}$

For T =5.0 sec $S(5.0) = F(5.0) \bullet S_a(5.0)$ =1.0*0.071=0.07 $\boxed{S(5.0)=0.07}$

- Building period (T) calculation (NBC 2015 Cl.4.1.8.11.3(c)) – wall structures
  h_n = 14.0 m building height
  $T = 0.05(h_n)^{3/4}$ = 0.36 sec

Building period T = 0.36 sec, so interpolate between $S(0.2)$ and $S(0.5)$, hence $\boxed{S(T) = 0.65}$



- I_E = 1.0 (normal importance building)
- M_v = 1.0 (higher mode factor, equal to 1.0 for $T \leq$ 1.0 sec)
- Building SFRS description: masonry structure – Conventional Construction shear walls can be used for building height of 14 m (see Table 1-13 and NBC 2015 Table 4.1.8.9).

In this case $I_E F_a S_a(0.2)$=1.0*1.0*0.70=0.70, hence 0.35 < $I_E F_a S_a(0.2)$<0.75 thus the maximum building height is 30 m. Hence
R_d = 1.5 and R_o = 1.5

b) Compute the design base shear (NBC 2015 Cl.4.1.8.11).

The design base shear V is determined according to the following equation:

$$V = \frac{S(T) M_v I_E}{R_d R_o} W = \frac{0.70 * 1.0 * 1.0}{1.5 * 1.5} W = 0.31W$$

but should not be less than

$$V_{\min} = \frac{S(4.0) M_v I_E W}{R_d R_o} = \frac{0.12 * 1.0 * 1.0}{1.5 * 1.5} W = 0.05W$$

Note that $S(4.0)$ value (0.15) was obtained by interpolation from the site design spectrum chart $S(T)$.

The design base shear V need not be taken more than greater of the following two values:

$$V_{\max} = \left(\frac{2S(0.2)}{3}\right)\left(\frac{I_E W}{R_d R_o}\right) = \left(\frac{2 * 0.70}{3}\right)\left(\frac{1.0}{1.5 * 1.5}\right) W = 0.21W$$, provided $R_d \geq 1.5$.

and

$$V_{\max} = S(0.5)\left(\frac{I_E W}{R_d R_o}\right) = 0.60\left(\frac{1.0}{1.5 * 1.5}\right) W = 0.27W$$ - this value governs

Therefore, the design seismic base shear is equal to
$V = 0.27W = 0.27 * 10350 \approx 2900$ kN

4. Determine whether the equivalent static procedure can be used (see Section 1.5 and NBC 2015 Cl. 4.1.8.7).

According to the NBC 2015, the dynamic method is the default method, but the equivalent static method can be used if the structure meets <u>any</u> of the following criteria:

(a) is located in a region of low seismic activity where $I_E F_a S_a(0.2) < 0.35$,

In this case, the seismic hazard index is $I_E F_a S_a(0.2)$=1.0*1.0*0.70=0.70 > 0.35 and so this criterion is not satisfied. Note that $F_a = F(0.2) = 1.0$.

(b) is a regular structure less than 60 m in height with period T < 2 seconds in either direction,

This building is clearly less than 60 m in height and the period T < 2 sec (as discussed above). To confirm that this structure is regular, the designer needs to review the irregularities discussed in Section 1.12.1. It can be concluded that this building does not have any of the irregularity types identified by NBC 2015 and so this criterion is satisfied.

(c) has any type of irregularity (other than Type 7 or Type 9 that requires the dynamic method if B >1.7), but is less than 20 m in height with period T < 0.5 seconds in either direction

This is an irregular structure, but it is less than 20 m in height and the period is less than 0.5 sec. The torsional sensitivity B should be checked to confirm that B < 1.7 (see Section 1.11.2).

Since the criterion b) has been satisfied, the design can proceed by using the equivalent static analysis procedure.

5. Seismic force distribution over the building height (see Section 1.9).

According to NBC 2015 Cl. 4.1.8.11.(7), the total lateral seismic force, V , is to be distributed over the building height in accordance with the following formula (see Figure 1-5):

$$F_x = \left(V - F_t\right) \cdot \frac{W_x h_x}{\sum_{i=1}^{n} W_i h_i}$$

where

F_x – seismic force acting at level x

F_t – a portion of the base shear to be applied in addition to force F_n at the top of the building.

In this case, F_t = 0 since the fundamental period is less than 0.7 sec.

Interstorey shear force at level x can be calculated as follows:

$$V_x = F_t + \sum_{x}^{n} F_i$$

Bending moment at level x can be calculated as follows:

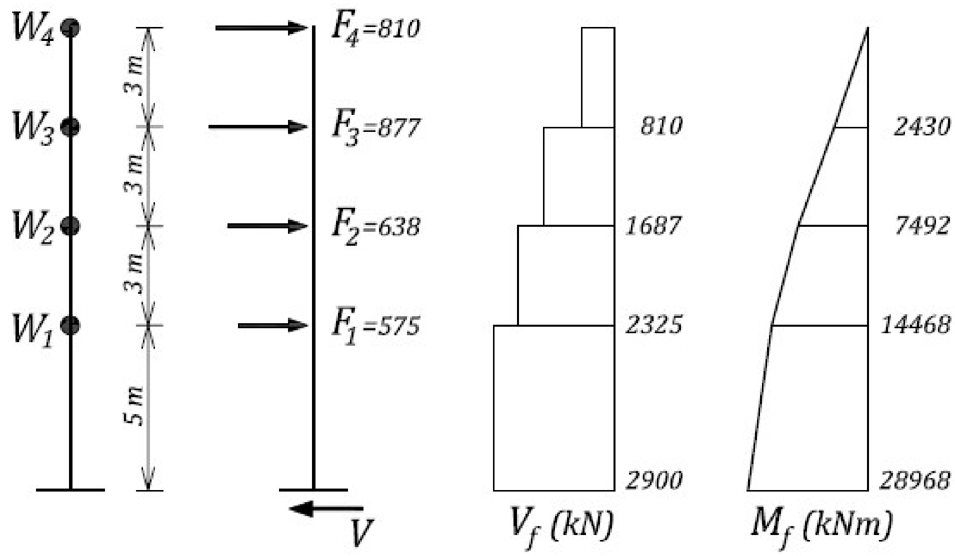
$$M_x = \sum_{i=x}^{n} F_i \left(h_i - h_x\right)$$

These calculations are presented in Table 1.

Table 1. Distribution of Seismic Forces over the Wall Height

Level	h_x (m)	W_x (kN)	$W_x h_x$	F_x (kN)	V_x (kN)	M_x (kNm)
4	14.0	1802	25228	810	810	0
3	11.0	2484	27324	877	1687	2430
2	8.0	2484	19872	638	2325	7492
1	5.0	3579	17895	575	2900	14468
$\sum$		10349	90319	2900		28968

Distribution of seismic forces over the building height and the corresponding shear and moment diagrams are shown on the figure below.



It is important to confirm that the sum of seismic forces F_x over the building height is equal to the base shear

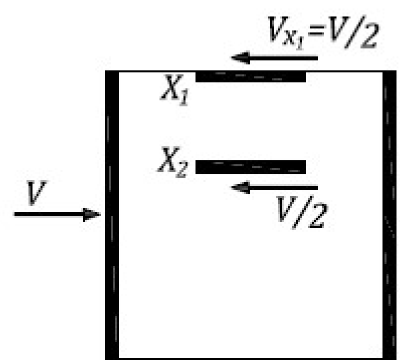
$V_b = V =$ 2900 kN

The bending moment at the base of the building, also called the base bending moment, is equal to

M_b = 28968 ≈ 29000 kNm.

6. Find the seismic forces in the E-W walls – torsional effects ignored.

Due to asymmetric layout of the E-W walls, the centre of mass C_M in the building under consideration does not coincide with the centre of resistance C_R, hence there are torsional effects in all walls. However, since the N-S walls are significantly more rigid compared to the E-W walls, it can be assumed that the N-S walls will resist the torsional effects (see step 8 for a detailed discussion). As a consequence, it can be assumed that the base shear force in the E-W direction is equally divided between the two E-W walls (see the figure), that is,



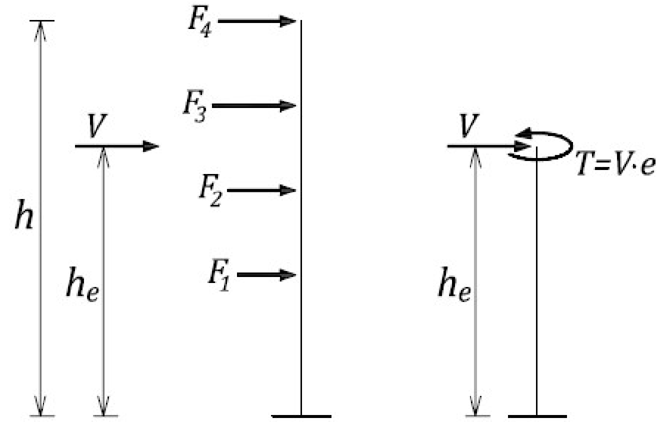
$$V_{xo} = \frac{V}{2} = \frac{2900}{2} = 1450 \text{ kN}$$

Similarly, the base bending moment in each wall is equal to

$$M_{bx} = \frac{M_b}{2} = \frac{29000}{2} = 14500 \text{ kNm}$$

7. Find the seismic forces in the E-W walls – torsional effects considered (see Section 1.11).

To determine the wall forces from the torsional forces a 3-D analysis should be made. Even though the walls are considered uniform over the entire height, the contribution of shear deformation relative to bending deformation is different over the height. An approximate method that does not require a 3-D analysis is to consider the structure as an equivalent single-storey structure. The entire shear is applied at the effective height, h_e, defined as the height at which the shear force V_f must be applied to produce the base moment M_f, that is,



$$h_e = \frac{M_f}{V_f} = \frac{29000}{2900} = 10.0 \text{ m}$$

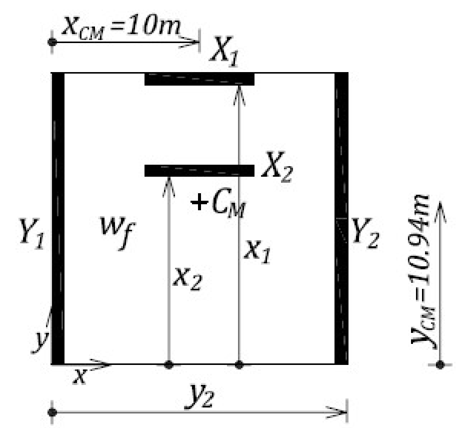
This model, although not strictly correct, will be used to determine the elastic distribution of the torsional forces as well as the displacements. The top displacement of the wall is assumed to be 1.5 times the displacement at the h_e height (see step 8 for displacement calculations).

Torsional moment (torque) is a product of the seismic force and the eccentricity between the centre of resistance (C_R) and the centre of mass (C_M), which will be calculated in the following tables.

First, the centre of mass will be determined, as shown on the figure. The calculations are summarized in Table 2.

Table 2. Calculation of the Centre of Mass (C_M)

Wall	w_i (kN)	x_i (m)	y_i (m)	$w_i * x_i$	$w_i * y_i$
X_1	733.7	10.00	20.00	7337	14674
X_2	733.7	10.00	13.33	7337	9780
Y_1	961.4	0	10.00	0	9614
Y_2	961.4	20.00	10.00	19228	9614
Floors	6960	10.00	10.00	69600	69600
$\sum$	10350			103502	113282



The C_M coordinates can be determined as follows:

$$x_{CM} = \frac{\sum_i w_i * x_i}{\sum_i w_i} = \frac{103502}{10350} = 10.00 \text{ m} \qquad y_{CM} = \frac{\sum_i w_i * y_i}{\sum_i w_i} = \frac{113282}{10350} = 10.94 \text{ m}$$

Next, the centre of resistance (C_R) will be determined, and the calculations are presented in Table 3, although because there are only two equal walls in each direction the C_R will lie between the walls.

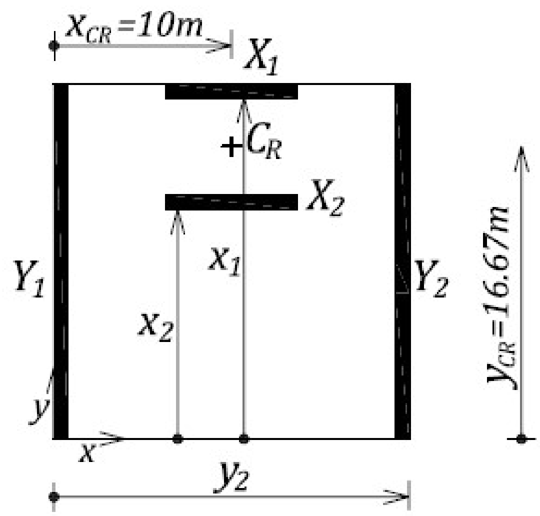
Table 3. Calculation of the Centre of Resistance (C_R)

Wall	t (m)	h/l_w *	$K/(E_m \cdot t)$ **	K_x x10[3] (kN/m)	K_y x10[3] (kN/m)	x_i (m)	y_i (m)	$K_y \cdot x_i$ *10[3]	$K_x \cdot y_i$ *10[3]
X_1	0.29	1.0	0.143	352.5			20.00		7050.0
X_2	0.29	1.0	0.143	352.5			13.33		4699.0
Y_1	0.19	0.5	0.5		807.5	0		0	
Y_2	0.19	0.5	0.5		807.5	20.00		16150.0	
$\sum$				705.0	1615.0			16150.0	11750.0

Notes:

* - $h = h_e$ = 10.0 m effective wall height

** - see Table D-3



Note that the elastic uncracked wall stiffnesses K for individual walls have been determined from Table D-3, by entering appropriate height-to-length ratios. In this design, all walls and piers have been modelled as cantilevers (fixed at the base and free at the top) – see Section C.3 for more details regarding wall stiffness calculations. The modulus of elasticity for masonry is $E_m = 8.5*10^6$ kPa (corresponding to f'_m of 10 MPa).

The C_R coordinates can be determined as follows (see the figure):

$$x_{CR} = \frac{\sum_i K_{yi} * x_i}{\sum_i K_{yi}} = \frac{16150*10^3}{1615*10^3} = 10 \text{ m}$$

$$y_{CR} = \frac{\sum_i K_{xi} * y_i}{\sum_i K_{xi}} = \frac{11750*10^3}{705*10^3} = 16.67 \text{ m}$$

Next, the eccentricity needs to be determined. Since we are looking for the forces in the E-W walls, we need to determine the actual eccentricity in the y direction (e_y), that is,

$$e_y = y_{CR} - y_{CM} = 16.67 - 10.94 = 5.73 \text{ m}$$

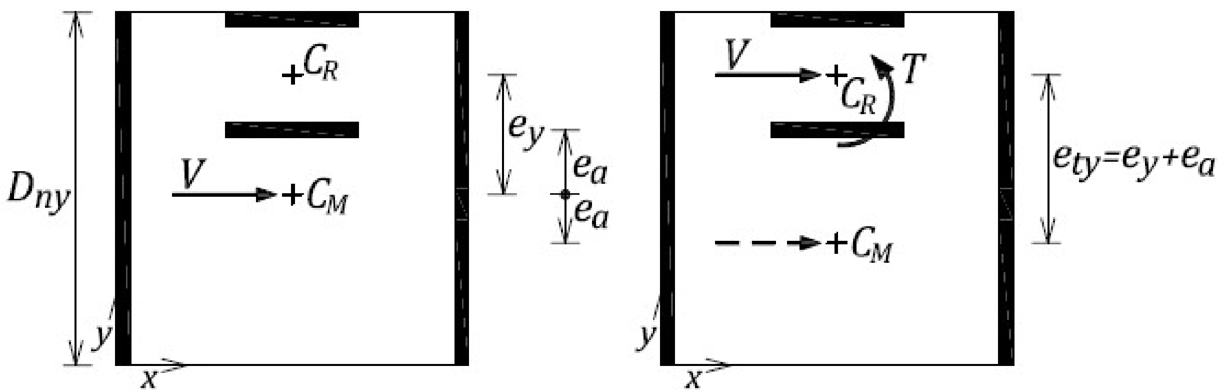
In addition, the accidental eccentricity needs to be considered, that is,

$$e_a = \pm 0.1 D_{ny} = \pm 0.1 * 20 = \pm 2.0 \text{ m}$$

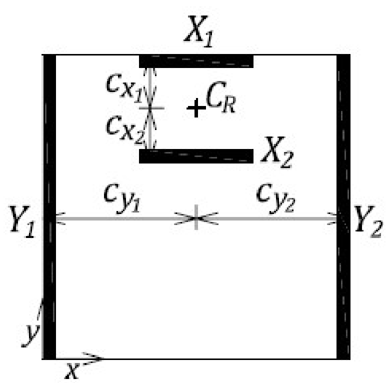
The total maximum eccentricity in the y-direction is equal to

$e_{ty1} = e_y + e_a = 5.73 + 2.0 = 7.73$ m
or
$e_{ty2} = e_y - e_a = 5.73 - 2.0 = 3.73$ m
Note that the latter value does not govern and will not be considered in further calculations.

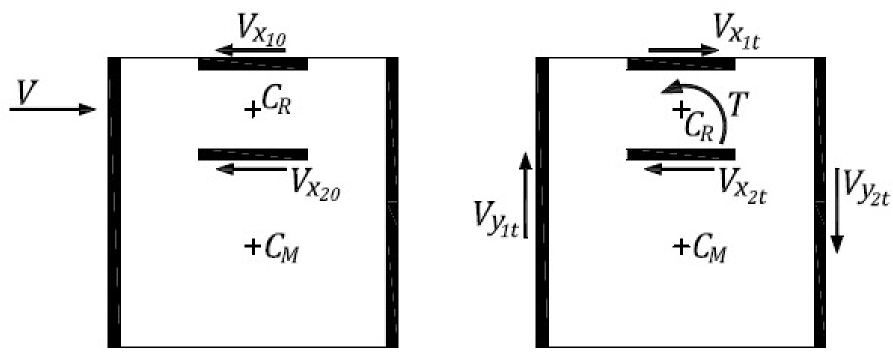
Torsional moment is determined as a product of the shear force and the eccentricity, that is,
$T = V * e_{ty1} = 2900 * 7.73 = 22417$ kNm
Torsional effects are illustrated on the figure below.



Seismic force in each wall has two components: translational (no torsional effects) and torsional, that is,
$V_i = V_{io} + V_{it}$
where
$V_{io} = V * \frac{K_i}{\sum K_i}$ translational component
and
$V_{it} = \frac{T * c_i}{J} * K_i$ torsional component
$J = \sum K_{xi} \cdot c_{xi}^2 + \sum K_{yi} \cdot c_{yi}^2 = 169 * 10^6$ torsional stiffness (see Table 4)
c_{xi}, c_{yi} - distance of the wall centroid from the centre of resistance (C_R) (see the figure below)



Translational and torsional force components for the individual walls are shown below.



Translational Force Components   Torsional Force Components

Calculation of translational and torsional forces is presented in Table 4.

Table 4. Seismic Shear Forces in the Walls due to Seismic Load in the E-W Direction

Wall	K_x *10³ (kN/m)	K_y *10³ (kN/m)	c_i (m)	$\sum K_i \cdot c_i^2$ *10⁶	$\frac{K_x}{\sum K_x}$	V_{xo} (kN)	V_{xt} (kN)	V_{total} (kN)
X_1	352.5		-3.33	3.84	0.5	1450	-154	1296
X_2	352.5		3.33	3.84	0.5	1450	154	1604
Y_1		807.5	-10.00	80.80			-1070	-1070
Y_2		807.5	10.00	80.80			1070	1070
$\sum$	705.0	1615.0		169.0				

It can be concluded from the above table that the maximum force in the E-W direction is equal to 1604 kN. This is an increase of only 11% as compared to the total force of 1450 kN obtained ignoring torsional effects.

It can be noted that the contribution of E-W walls to the overall torsional moment T of 22417 kNm is not significant (see Table 4).

$T_{E-W} = 154kN * 3.3m + 154kN * 3.3m \cong 1017kNm$

because

$T_{E-W} / T = 1017 / 22417 = 0.045 \approx 5\%$

this shows that the E-W walls contribute only 5% to the overall torsional moment.

The contribution of N-S walls to the overall torsional moment is as follows:

$T_{N-S} = 1070kN * 10m + 1070kN * 10m = 21400kNm$

and

$T_{N-S} / T = 21400 / 22417 \approx 95\%$

and

$T = T_{E-W} + T_{N-S} = 1017 + 21400 \approx 22417kNm$ (this is also a check for the torsional forces)

Therefore, the assumption that the torsional effects are resisted by N-S walls only is reasonable, since these walls contribute approximately 95% to the overall torsional resistance.

8. Calculate the displacements at the roof level (consider torsional effects).
Approximate deflections in the E-W walls can be determined according to the procedure outlined below. It should be noted that the force distribution calculations have been performed using elastic wall stiffnesses obtained from Table D-3. It is expected that the walls are going to crack during earthquake ground shaking; this will cause a drop in the wall stiffnesses. For the purpose of deflection calculations, we are going to use a reduction in the elastic stiffness (K) value to account for the effect of cracking.

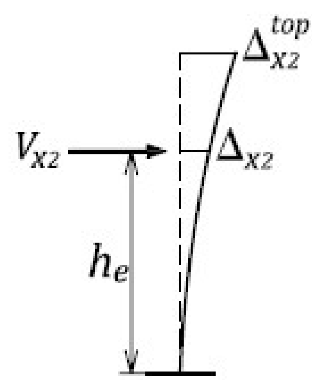
a) The reduced stiffness to account for the effect of cracking (see Section 2.5.4)
The reduced stiffness for walls X_1 *and* X_2 will be determined according to Section 2.5.4 (S304-14 Cl.16.3.3), that is,

$$I_e = I_g \left[0.3 + P_s / \left(A_g f'_m\right)\right]$$

Here,
$P_s = (2*6.67*6.67)(3.0 + 2*4.0 + 6.0) = 1513$ kN (axial force due to dead load in wall X_2)
$A_g = \left(290*10^3\right)*10.0 = 290*10^4$ mm^2 (gross cross-sectional area for 290 mm block wall, solid grouted, length 10.0 m; see Table D-1 for A_e values for the unit wall length)
f'_m =10.0 MPa
Since
$$0.3 + P_s / \left(A_g f'_m\right) = 0.3 + 1513*10^3 / \left(10.0*290*10^4\right) = 0.35$$
It appears that
$$\frac{I_e}{I_g} = 0.35$$
thus
$$K_{ce} = \left(\frac{I_e}{I_g}\right) K_c = 0.35 K_c$$
where K_c is elastic uncracked stiffness. In this case, stiffness is taken as proportional to the ratio of moment of inertia values because the wall is expected to behave in flexure-dominant manner (otherwise a ratio of cross-sectional areas could be used – see Example 3).

b) The translational displacement in the walls X_1 ***and*** X_2 ***can be calculated as follows***

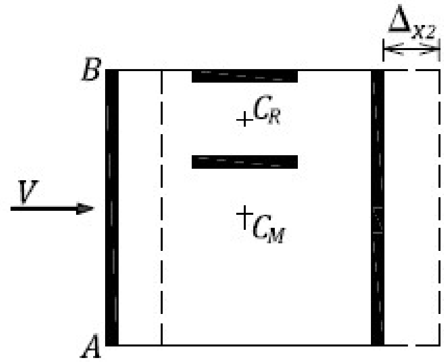
$$\Delta_{X20} = \frac{V_{X2o}}{0.35 K_{X2}} = \frac{1450kN}{0.35*352.5*10^3 kN/m} = 11.8mm$$
According to NBC 2015 Cl. 4.1.8.13, these deflections need to be multiplied by the $R_d R_o / I_E$ ratio (see Section 1.13). In this case, I_E = 1.0, and so
$$\Delta_{X20} = \left(11.8mm\right) R_d R_o = 11.8*1.5*1.5 = 26.6mm$$



Since the previous analysis assumed that the seismic force acts at the effective height h_e, the displacement at the top of the wall will be larger (see the figure). The top displacement can be calculated by deriving the displacement value at the tip of the cantilever; alternatively, an approximate factor of 1.5 can be used as follows:

$\Delta_{X20}^{top} = 1.5 * \Delta_{x2} = 1.5 * 26.6mm \approx 40.0mm$

Since this is a rigid diaphragm, it can be assumed that the translational displacements are equal at a certain floor level – let us use point A at the South-East corner as a reference (see the figure).



c) The torsional displacements can be calculated as follows:

Torsional rotation of the building θ can be determined as follows, considering the reduced torsional stiffness to account for cracking (same as discussed in step a) above):

$\theta = \frac{T}{J} = \frac{22417kNm}{0.35*169*10^6} = 3.79*10^{-4}$ rad

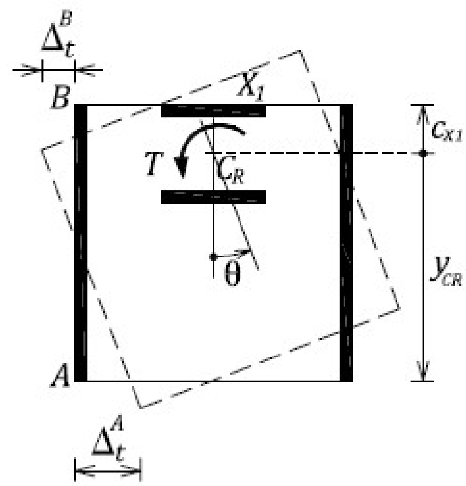
where (see the step 7 calculations)

$T = 22417$ kNm torsional moment

$J = 169*10^6$ elastic torsional stiffness

The maximum torsional displacement at the South-East corner in the X direction (see point A on the figure):

$\Delta^A_t = \theta * Y_{CR} = 3.79*10^{-4} * 16.67m = 6.3mm$



Similarly, as above, these displacements need to be multiplied by $R_d R_o / I_E$ and also by 1.5 to determine the displacement at the top of the roof, and so

$\Delta^{A\ top}_t = 1.5 * 6.3 * R_d R_o \approx 22mm$

d) Finally, the total maximum displacement at the roof level (at point A) is equal to:

$\Delta^A_{\max} = \Delta_{X2}^{top} + \Delta^{A\ top}_t = 40 + 22 = 62mm$

9. Check whether the building is torsionally sensitive.

NBC 2015 Cl. 4.1.8.11(10) requires that the torsional sensitivity B of the structure be determined by comparing the maximum horizontal displacement anywhere on a storey to the average displacement of that storey (see Section 1.11.2). This should be done for every storey, but in this case will only be done for the one storey as the remaining storeys will have similar B values because of the vertical uniformity of the walls. Torsional sensitivity is determined in a similar manner like the effect of accidental torsion, that is, by applying a set of lateral forces at a distance of $\pm 0.1D_{nx}$ from the centre of mass C_M. Since the purpose of this evaluation is to compare deflections at certain locations relative to one another, it is not critical to use cracked wall stiffnesses.

In this case, the total maximum displacement at point A was determined in step 8 above, that is,

$\Delta^A_{\max} = 62mm$

We need to determine the displacement at other corner (point B), that is, the minimum displacement. This can be done as follows:

Translational component:

$$\Delta^{B}{}_{0} = \Delta_{X20}{}^{top} = 40mm$$

Torsional component:

$$\Delta_t = \theta * c_{X1} = 3.79*10^{-4}*3.3m \approx 1.3mm$$

These displacements need to be multiplied by $R_d R_o / I_E$ and also by 1.5 to determine the displacement at the top of the roof, and so

$$\Delta^{B}{}_{t} = 1.3*1.5*R_d R_o \approx 5mm$$

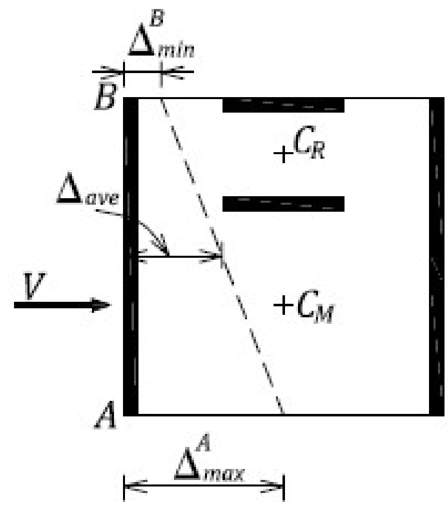
Since the direction of torsional displacements is opposite from the translational displacements, it follows that

$$\Delta^{B}{}_{\min} = \Delta^{B}{}_{o} - \Delta^{B}{}_{t} = 40 - 5 = 35mm$$

The average displacement at the roof level in the E-W direction (see the figure showing the displacement components):

$$\Delta_{ave} = \frac{\Delta^{A}{}_{\max} + \Delta^{B}{}_{\min}}{2} = \frac{62+35}{2} = 49mm$$

$$B = \frac{\Delta_{\max}}{\Delta_{ave}} = \frac{62.0}{49.0} = 1.27$$



Since B <1.7, this building is not considered to be torsionally sensitive. In general buildings with the main force resisting elements located around the exterior of the building will not be torsionally sensitive.

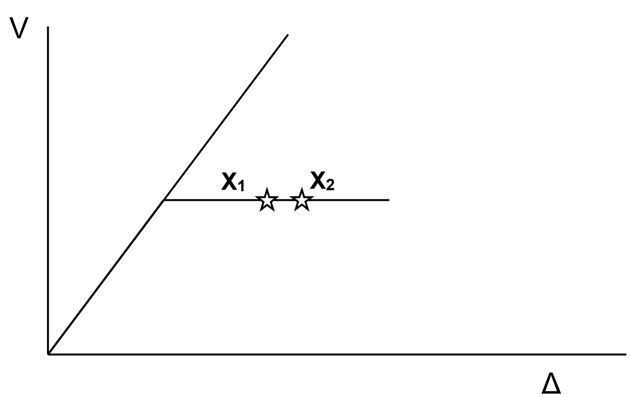
10. Discussion

A couple of important issues related to this design example will be discussed in this section.

a) Why should the N-S walls be considered to resist entire torsional effects?

The distribution of forces to the various elements in the structure is generally based on the relative elastic stiffnesses of the elements, unless the diaphragms are considered to be flexible and then the forces are distributed on the basis of contributory masses. The present example structure with four floors of concrete construction can be considered as having rigid diaphragms, and an elastic analysis was performed to determine the wall forces due to the torsional effects. Because the N-S walls are so much longer and stiffer than the E-W walls, and more widely separated, it is expected that they will resist most of the torque from the eccentricity. However, since we are designing the structures to respond inelastically, the distribution of forces from an elastic analysis should always be questioned. An argument is presented below to show that if the forces in the E-W walls are designed to be equal, they will not contribute to the torsional resistance.

The elastic torsional analysis for the forces in the E-W direction result in additional forces of ±154 kN in the E-W walls and ±1070 kN in the N-S walls (see Table 4). If all the torque is resisted by the N-S walls, the force in these walls would be ±1120 kN (an increase of only 50 kN).



For the earthquake load in the E-W direction the E-W walls must resist the total base shear in this direction and so they will have reached their yield strength and progressed along the flat portion of the shear/displacement curve as shown in the figure (assuming they have equal strength). The torsional load will have caused a small rotation of the diaphragms and so wall X_2 will have a slightly larger displacement than wall X_1, as shown on the figure. Had the walls remained elastic, the shear in wall X_2 would then be greater than wall X_1 and this would contribute to the torsional resistance. However, in the nonlinear case, they both have the same shear resistance and so do not contribute to the torsional resistance. Thus, in this example, all the torsion should be resisted by the longer N-S walls. The N-S walls are designed to resist the loads in the N-S direction but also to provide the torsional resistance from the loads in the E-W direction. However, it is highly unlikely that the maximum forces in the N-S walls from the two directions would occur at the same time, and practice has been to consider only 30% of the loads in one direction when combining with the loads in the other direction. Thus, the forces in the N-S walls at the time of the maximum torsional forces from the N-S direction could reach the yield level on one side, but the torsional displacement on the other side would be in the opposite direction, so the wall force would be much reduced in the other direction. The two N-S forces then provide a torque to resist the torsional motion. Although this resisting torque may not be as large as the elastic analysis would predict, the result would not be failure, but only slightly larger torsional displacements.

b) Application of the "100%+30%" rule

In the calculation of total wall seismic forces including the torsional effects (see step 7 above), the effect of seismic loads in E-W direction only was taken into consideration when calculating the forces in E-W walls. However, it is a good practice to consider the "100+30%" rule that requires the forces in any element that arise from 100% of the loads in one direction be combined with 30% of the loads in the orthogonal direction (for more details refer to NBC 4.1.8.8.(1)c and the commentary portion in Section 1.11.3).

Let us determine the forces in one of the E-W walls, e.g. wall X_2, by applying the "100+30%" rule. If only 100% of the force in the E-W direction is considered, the total force in the wall is equal to (see Table 4):

$$V_{X2}^{E-W} = V_{X2o} + V_{X2t} = 1450 + 154 = 1604kN$$

If the seismic load is applied in the N-S direction, the torsional moment would be determined based on the accidental eccentricity e_a (since the building is symmetrical in that direction), and so the torsional force in the wall X_2 can be prorated by the ratio of torsional eccentricities in the E-W and N-S directions as follows,

$$V_{X2}^{N-S} = V_{X2t} * \frac{e_a}{e_y} = 154 * \frac{2.0m}{7.73m} = 39.8 \approx 40kN$$

The total seismic force in the wall X_2 due to 100% of the load in E-W direction and 30% of the load in the N-S direction can be determined as

$$V_{X2} = V_{X2}^{E-W} + 0.3V_{X2}^{N-S} = 1604 + 0.3 * 40 = 1616kN$$

It can be concluded that the difference between the force of 1616 kN (when the "100+30%" rule is applied) and the force of 1604 kN (when the rule is ignored) is insignificant.

However, it can be shown that the "100+30%" rule would significantly influence the forces in the N-S walls. When the seismic force acts in the E-W direction, the force in the N-S wall (e.g. wall Y_1) due to torsional effects is equal to (see Table 4)

$$V_{Y1}^{E-W} = 1070kN$$

When the seismic force acts in the N-S direction, the total force in the wall Y_1 (including the effect of accidental torsion) can be determined as (see Example 1 for a detailed discussion on accidental torsion)

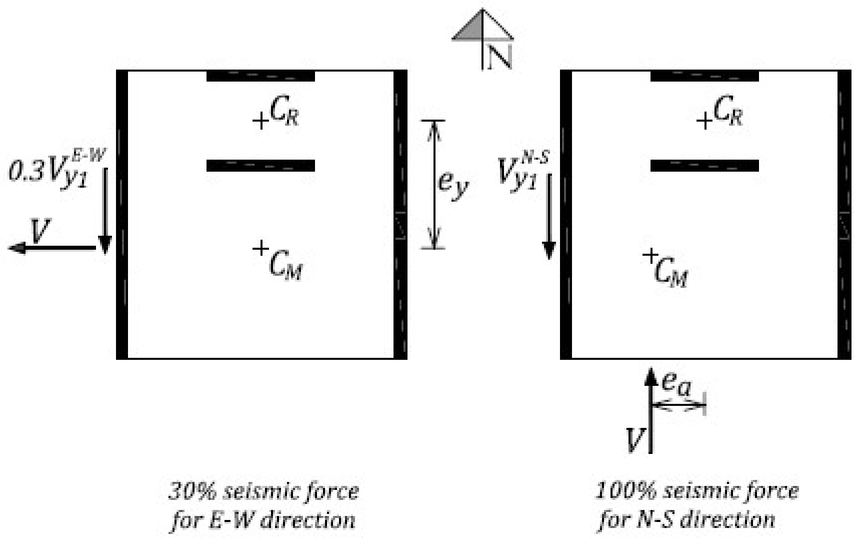
$$V_{Y1}^{N-S} = 0.6 * V = 0.6 * 2900 = 1740kN$$

So, if we apply the "100+30%" rule to 100% of the force in the N-S direction and 30% of the force in the E-W direction the resulting total force is equal to

$$V_{Y1} = V_{Y1}^{N-S} + 0.3V_{Y1}^{E-W} = 1740 + 0.3 * 1070 = 2061kN$$

In this case, it can be concluded that the difference between the force of 2061 kN (when the "100+30%" rule is applied) and the force of 1740 kN (when the rule is ignored) is significant (around 18%). This is illustrated on the figure below.

For those cases where there is a large eccentricity in one direction and the torsional forces are mainly resisted by elements in the other direction, the contribution from the "100+30%" rule can be significant.



<u>EXAMPLE 3</u>: Seismic load distribution in a masonry building considering both rigid and flexible diaphragm alternatives

Consider a single-storey commercial building located in Nanaimo, BC on a Class C site. The building plan and relevant elevations are shown on the figure below. The building has an open north-west façade consisting mostly of glazing. The roof elevation is at 4.8 m above the foundation. The roof structure is supported by 240 mm reinforced block masonry walls and steel columns on the north-west side. Masonry properties should be determined based on 20 MPa block strength and Type S mortar (use f'_m of 10.0 MPa). Grade 400 steel has been used for the reinforcement.

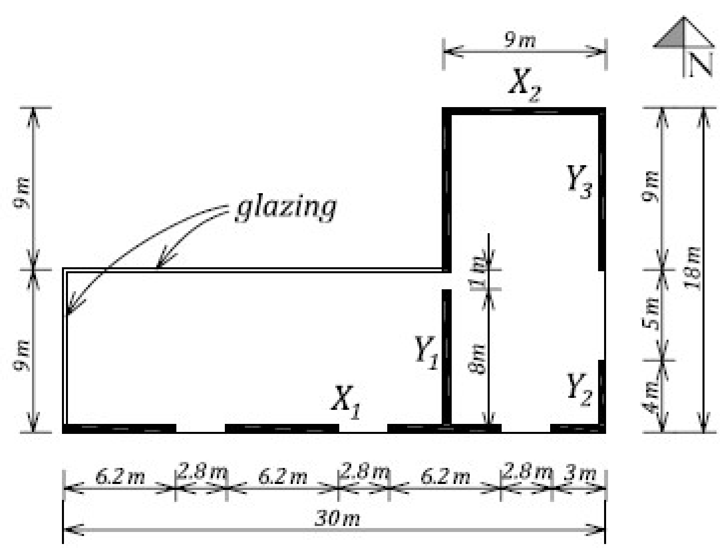
Masonry walls should be treated as "conventional construction" according to NBC 2015 and CSA S304-14. A preliminary seismic design has shown that the total seismic base shear force for the building is equal to $V = 700$ kN. This force was determined based on the total seismic weight W of 2340 kN and the seismic coefficient equal to 0.3, that is, $V = 0.3W$.

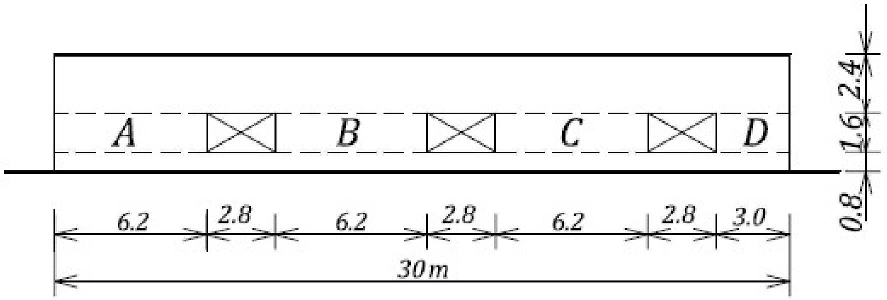
This example will determine the seismic forces in the N-S walls (Y_1 to Y_3) due to seismic force acting in the N-S direction for the following two cases:
a) Rigid roof diaphragm (consider torsional effects), and
b) Flexible roof diaphragm.

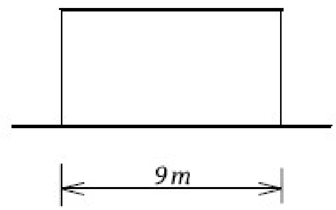
Finally, the wall forces obtained in parts a) and b) will be compared and the differences will be discussed.

Note that both flexible and rigid diaphragms are considered to have the same weight, although this would be unlikely in a real design application. Also, the columns located on the north-west side are neglected in the seismic design calculations.

Specified loads:
roof = 3.5 kPa
25% snow load = 0.6 kPa
wall weight = 5.38 kPa (240 mm blocks solid grouted; this is a conservative assumption)

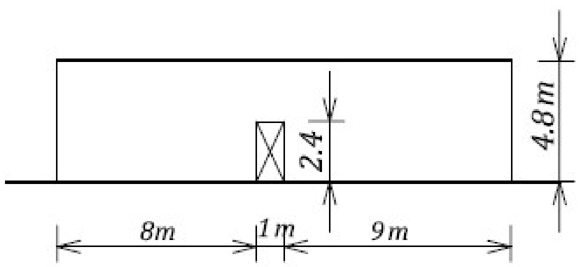


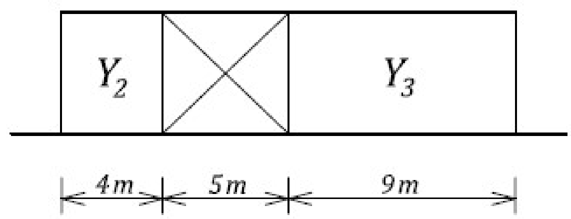




Wall X_1

Wall X_2





Wall Y_1

Walls Y_2 & Y_3

SOLUTION:

a) Rigid diaphragm

Torsional moment (torque) is a product of the seismic force and the eccentricity between the centre of resistance (C_R) and the centre of mass (C_M). The coordinates of the centre of mass will be determined taking into account the influence of wall masses, the upper half of which are supported laterally by the roof. The calculations are summarized in Table 1 below. Note that the centroid of the roof area is determined by dividing the roof plan into two rectangular sections.

Table 1. Calculation of the Centre of Mass (C_M)

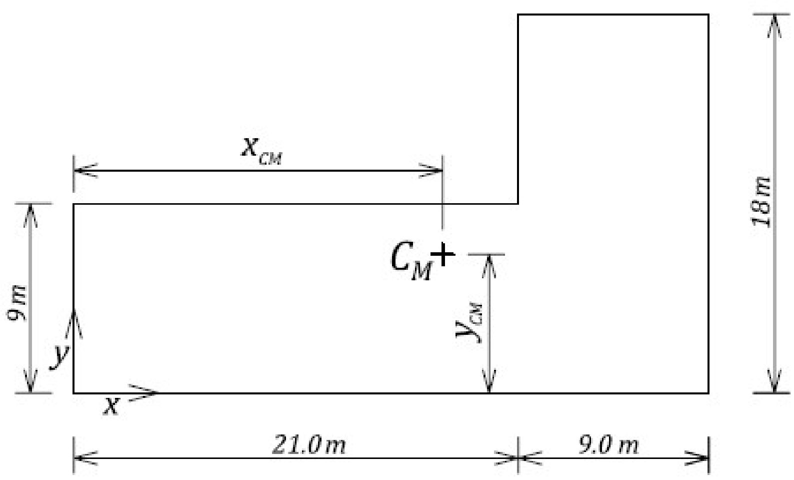
Wall	W_i (kN)	X_i (m)	Y_i (m)	$W_i * X_i$	$W_i * Y_i$
X1	387	15.00	0.00	5810	0
X2	116	25.50	18.00	2963	2092
Y1	232	21.00	9.00	4880	2092
Y2	52	30.00	2.00	1548	103
Y3	116	30.00	13.50	3486	1569
Roof 1	1107	15.00	4.50	16605	4982
Roof 2	332	25.50	13.50	8466	4482
	2343			43759	15319

The C_M coordinates have been determined from the table as follows (see the figure below):

$$x_{CM} = \frac{\sum_i W_i * X_i}{\sum_i W_i} = \frac{43757.02}{2343.86} = 18.68 \text{ m}$$

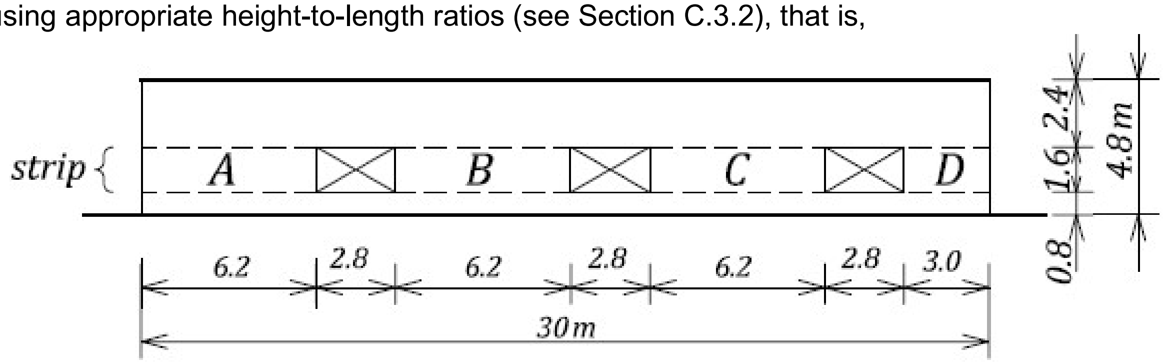
$$y_{CM} = \frac{\sum_i W_i * Y_i}{\sum_i W_i} = \frac{15324.38}{2343.86} = 6.54 \text{ m}$$

Next, the coordinates of the centre of resistance (C_R) will be determined. Wall X_1 has several openings and the overall wall stiffness is determined using the method explained in Section C.3.3 by considering the deflections of the following components for a unit load (see the figure below):



- solid wall with 4.8 m height and 30 m length – cantilever (Δ_{solid})
- an interior strip with 1.6 m height (equal to the opening height) and 30 m length – cantilever (Δ_{strip})
- piers A, B, C, and D – cantilevered (Δ_{ABCD}) (the stiffness of the piers A, B, C, and D is summed and the inverse taken as Δ_{ABCD})

The stiffness of each component is based on the following equation for the cantilever model by using appropriate height-to-length ratios (see Section C.3.2), that is,



Wall X_1

$$\frac{K}{E_m * t} = \frac{1}{\left(\frac{h}{l}\right)\left[4\left(\frac{h}{l}\right)^2 + 3\right]}$$

The overall wall deflection is determined from the combined pier deflections, as follows:

$$\Delta_{X1} = \Delta_{solid} - \Delta_{strip} + \Delta_{ABCD}$$

Note that the strip deflection is subtracted from the solid wall deflections - this removes the entire portion of the wall containing all the openings, which is then replaced with the deflection of the four piers.

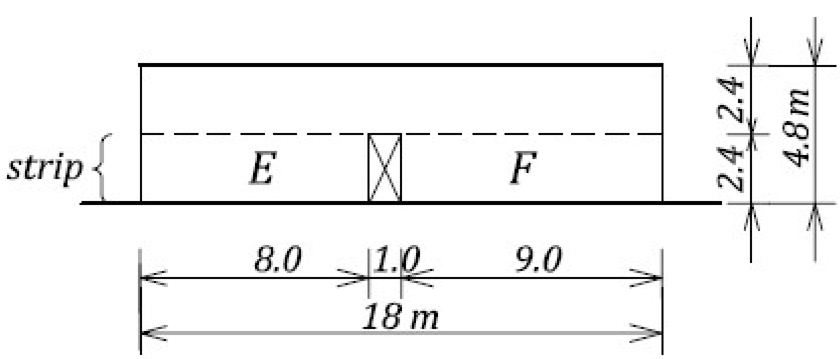
Finally, the stiffness of the wall X_1 is equal to the reciprocal of the deflection (see Table 2), as follows

$$K_{X1} = \frac{1}{\Delta_{X1}} = 1.71$$

Table 2. Wall X_1 Stiffness Calculations

Wall	t (m)	h (m)	l (m)	**End conditions**	h/l	$K/(E*t)$	**Displacement**	$K_{final}/(E*t)$
Solid	0.24	4.8	30.0	cant	0.160	2.015	0.496	
Opening strip	0.24	1.6	30.0	cant	0.053	6.226	-0.161	
X1A	0.24	1.6	6.2	cant	0.258	1.186		
X1B	0.24	1.6	6.2	cant	0.258	1.186		
X1C	0.24	1.6	6.2	cant	0.258	1.186		
X1D	0.24	1.6	3.0	cant	0.533	0.453		
					$\sum$ (ABCD)	4.012	0.249	
							0.585	1.709

The stiffness of wall Y_1 is determined in the same manner (see the figure below). The calculations are summarized in Table 3.



Wall Y_1

Table 3. Wall Y_1 Stiffness Calculations

Wall	t (m)	h (m)	l (m)	End conditions	h/l	$K/(E*t)$	Displacement	$K_{final}/(E*t)$
Solid	0.24	4.8	18	cant	0.267	1.142	0.876	
Opening strip	0.24	2.4	18	cant	0.133	2.442	-0.409	
Pier E	0.24	2.4	8	cant	0.300	0.992		
Pier F	0.24	2.4	9	cant	0.267	1.142		
					sum(EF)	2.134	0.469	
							0.935	1.070

Next, the centre of resistance (C_R) will be determined, and the calculations are presented in Table 4.

Table 4. Calculation of the Centre of Resistance (C_R)

Wall	t (m)	h (m)	l (m)	End cond.	h/l	$\frac{K}{E*t}$	K_x (kN/m)	K_y (kN/m)	X_i (m)	Y_i (m)	K_y*X_i	K_x*Y_i
X1	0.24					1.709*	3.49E+06	0	15	0		0.00E+00
X2	0.24	4.8	9	cant	0.53	0.453	9.24E+05	0	25.5	18		1.66E+07
Y1	0.24					1.070**	0	2.18E+06	21	0	4.58E+07	
Y2	0.24	4.8	4	cant	1.20	0.095	0	1.94E+05	30	0	5.82E+06	
Y3	0.24	4.8	9	cant	0.53	0.453	0	9.24E+05	30	0	2.77E+07	
							4.41E+06	3.30E+06			7.94E+07	1.66E+07

Notes:
* - see Table 2
** - see Table 3

Note that all walls and piers in this example were modelled as cantilevers (fixed at the base and free at the top). For more discussion related to modelling of masonry walls and piers for seismic loads see Section C.3. The modulus of elasticity for masonry is taken as $E_m = 8.5*10^6$ kPa (corresponding to f'_m of 10 MPa).

The C_R coordinates can be determined as follows (see the figure on the next page):

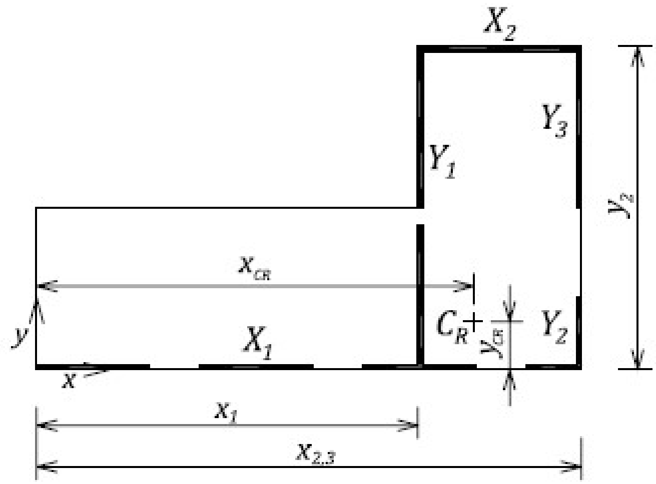
$$x_{CR} = \frac{\sum_i K_{yi} * x_i}{\sum_i K_{yi}} = \frac{7.94*10^7}{3.30*10^6} = 24.05 \text{ m}$$

$$y_{CR} = \frac{\sum_i K_{xi} * y_i}{\sum_i K_{xi}} = \frac{1.66*10^7}{4.41*10^6} = 3.77 \text{ m}$$

Next, the eccentricity needs to be determined. Since we are considering the seismic load effects in the N-S direction, we need to determine the actual eccentricity in the x-direction (e_x), that is,
$e_x = x_{CR} - x_{CM} = 24.05 - 18.68 = 5.37$ m

In addition, an accidental eccentricity needs to be considered, as follows:

$e_a = \pm 0.1 D_{nx} = \pm 0.1 * 30 = \pm 3.0$ m



The total maximum eccentricity in the x-direction assumes the following two values depending on the sign of the accidental eccentricity, that is,

$e_{x1} = e_x + e_a = 5.37 + 3.0 = 8.37$ m

$e_{x2} = e_x - e_a = 5.37 - 3.0 = 2.37$ m

The torsional moment is determined as a product of the shear force and the eccentricity, that is,

$T_1 = V * e_{x1} = 700 * 8.37 \approx 5860$ kNm

$T_2 = V * e_{x2} = 700 * 2.37 \approx 1660$ kNm

The seismic force in each wall can be determined as the sum of the two components: translational (no torsional effects) and torsional, that is,

$V_i = V_{io} + V_{it}$

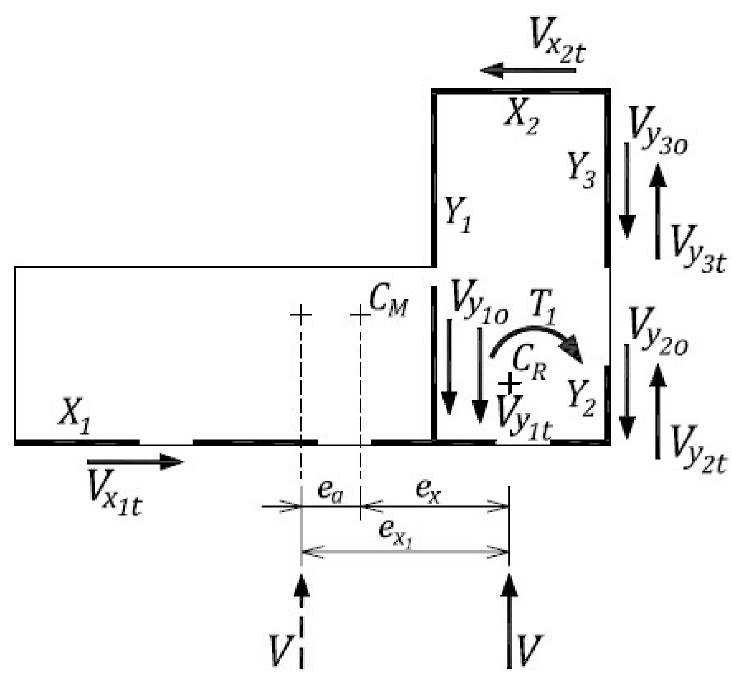
where

$V_{io} = V * \dfrac{K_i}{\sum K_i}$ translational component

$V_{it} = \dfrac{T * c_i}{J} * K_i$ torsional component

$J = \sum K_{xi} \cdot c_{xi}^{\ 2} + \sum K_{yi} \cdot c_{yi}^{\ 2} = 2.97 * 10^8$ torsional rigidity (see Table 5)

c_{xi}, c_{yi} - distance of the wall centroid from the centre of resistance (C_R)



The calculation of translational and torsional forces is presented in Table 5. Translational and torsional force components due to the eccentricity e_{x1} and the torsional moment T_1 are shown on the figure. Note that the torque T_1 causes rotation in the same direction like the force V (showed by the dashed line) around point C_R (this is illustrated on Figure 1-8). The wall forces shown on the diagram are in the directions to resist the shear V and torque T_1, thus on wall Y1 the translational force and torsional force act in the same direction, while in walls Y2 and Y3 these forces act in the opposite direction. The calculation of the forces is presented in Table 5 where the sign convention has horizontal wall forces positive to the left and vertical forces positive down, resulting in negative values for the torsional forces in walls X1, Y2 and Y3.

Table 5. Seismic Shear Forces in the Walls due to Seismic Load in the N-S Direction

Wall	K_i (kN/m)	c_i (m)	$K_i * c_i^2$	$K_y / \sum K_y$	V_o (kN)	V_{1t} (kN)	V_{1total} (kN)	V_{2t} (kN)	V_{2total} (kN)	V_{govern} (kN)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
X1	3.49E+06	-3.77	4.96E+07			-260	-260	-74	-74	260
X2	9.24E+05	14.23	1.87E+08			260	260	74	74	260
$\sum K_x$	4.41E+06									
Y1	2.18E+06	3.05	2.03E+07	0.66	463	131	594	37	500	594
Y2	1.94E+05	-5.95	6.87E+06	0.06	41	-23	18	-6	35	35
Y3	9.24E+05	-5.95	3.27E+07	0.28	196	-109	87	-31	165	165
$\sum K_y$	3.30E+06			1.00	700					
		$\sum K_i * c_i^2$	2.97E+08							

It should be noted that there are two total seismic forces for each wall in the N-S direction (corresponding to torsional moments T_1 and T_2) – see columns (8) and (10) in Table 5. The governing force to be used for design is equal to the larger of these two forces, as shown in column (11) of Table 5. Note that, in some cases, torsional forces have a negative sign and cause a reduction in the total seismic force, like in the case of walls Y2 and Y3.

b) Flexible diaphragm

It is assumed in this example that flexible diaphragms are not capable of transferring significant torsional forces to the walls perpendicular to the direction of the inertia forces. Therefore, the wall forces are determined as diaphragm reactions, assuming that diaphragms D1 and D2 act as beams spanning between the walls, as shown on the figure below. The diaphragm loads include the inertia loads of the walls supported laterally by the diaphragm. The SFRS wall inertia forces are added to the forces supporting the diaphragms to get the total wall load. The seismic coefficient of 0.3 will be used in these calculations (as defined at the beginning of this example).

Shear forces in the walls Y_{1a} and Y_2 (diaphragm D1):

Seismic force in the diaphragm D1 is due to the roof seismic weight and the wall X_1 inertia load:

$$V_{D1} = 0.3 * [(9m * 30m) * (3.5kPa + 0.6kPa) + 2.4m * 30m * 5.38kPa] = 448kN$$

The diaphragm is considered as a beam with the reactions at the locations of walls Y_{1a} and Y_2, that is,

$$R_{Y1a} = 448kN * 15m/9m = 747kN$$

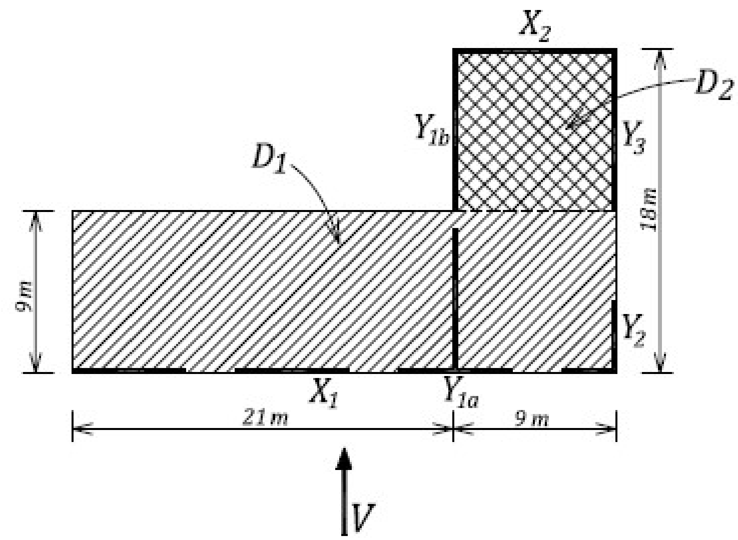
and

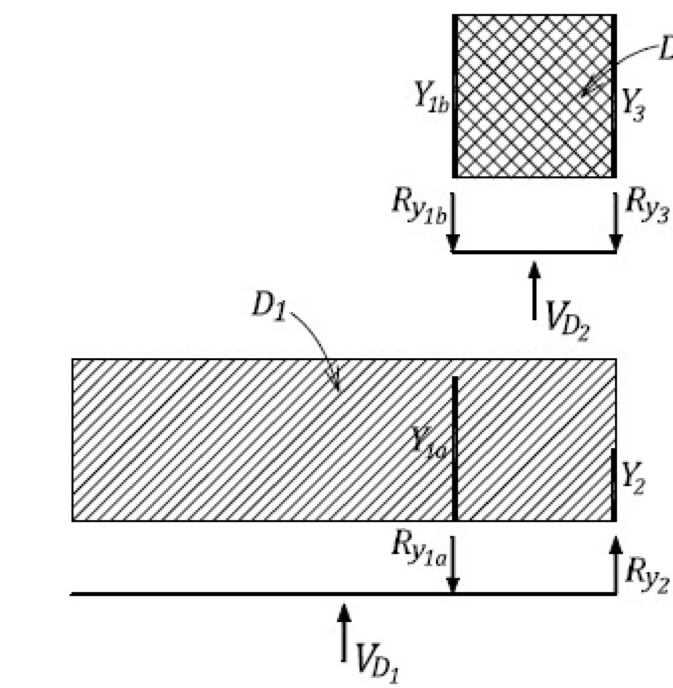
$R_{Y2} = V_{D1} - R_{Y1a} = 448 - 747 = -299kN$ (opposite direction from R_{Y1a} is required to satisfy equilibrium)

The total force in each wall is obtained when the wall inertia load is added to the diaphragm reaction, that is,

$V_{Y1a} = R_{Y1a} + V_w = 747 + 0.3 * 2.4m * 9m * 5.38kPa = 782kN$

$V_{Y2} = R_{Y2} + V_w = -299 + 0.3 * 2.4m * 4m * 5.38kPa = -284kN$ (note: this force has opposite direction from force V_{Y1a})





Shear forces in the walls Y_{1b} and Y_3 (diaphragm D2):

Seismic force in the diaphragm D2 is due to the roof seismic weight and the wall X_2 inertia load:

$V_{D2} = 0.3 * [(9m * 9m) * (3.5kPa + 0.6kPa) + 2.4m * 9m * 5.38kPa] = 134.5kN$

The diaphragm is considered as a beam with the reactions at the locations of walls Y_{1b} and Y_3, that is,

$R_{Y1b} = R_{Y3} = 134.5/2 = 67.3kN$

The total force in each wall is obtained when the wall inertia load is added to the diaphragm reaction, that is,

$V_{Y1b} = R_{Y1b} + V_w = 67 + 0.3 * 2.4m * 9m * 5.38kPa = 102kN$

$V_{Y3} = R_{Y3} + V_w = 67 + 0.3 * 2.4m * 9m * 5.38kPa = 102kN$

<u>Total shear force in wall Y_1</u>:

The total seismic force in the wall Y_1 is equal to

$V_{Y1} = V_{Y1a} + V_{Y1b} = 782 + 102 = 884kN$

<u>Shear forces in walls Y_2 and Y_3</u>:

The total shear force in the combined walls Y_2 and Y_3 is equal to

$V_{Y23} = V_{Y2} + V_{Y3} = -284 + 102 = -182kN$

This force will then be distributed to these walls in proportion to the wall stiffness, as follows (the wall stiffnesses are presented in Table 4):

$$V_{Y2} = \frac{K_{Y2}}{K_{Y2} + K_{Y3}} * V_{Y23} = \frac{1.94 * 10^5}{1.94 * 10^5 + 9.24 * 10^5} * (-182) = 0.17 * (-182) = -32kN$$

$V_{Y3} = V_{Y23} - V_{Y2} = -182 - (-32) = -150kN$

The comparison

Shear forces in the walls Y_1 to Y_3 obtained in parts a) and b) of this example are summarized on the figure below. A comparison of the shear forces is presented in Table 6.

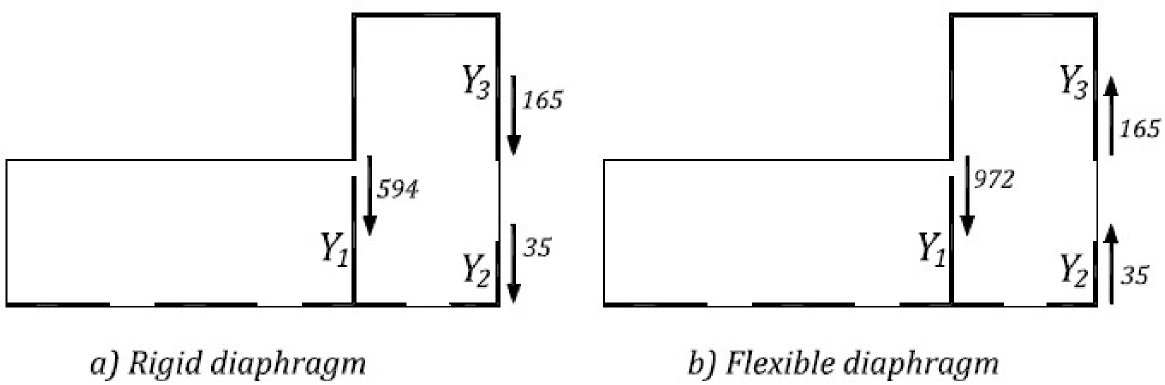


Table 6. Shear Forces in the Walls Y_1 to Y_3 for Rigid and Flexible Diaphragms

Wall	Shear forces (kN)	
	Rigid diaphragm (part a)	Flexible diaphragm (part b)
Y_1	594	972 (884)
Y_2	35	35 (32)
Y_3	165	165 (150)

Note that, for the flexible diaphragm case, values in the brackets are actual forces. These values are increased by 10 % to account for accidental eccentricity.

It can be observed from the table that the flexible diaphragm assumption results in the same seismic forces for the walls Y_2 and Y_3, and an increase in the wall Y_1 force.

Deflection calculations

A fundamental question related to diaphragm design is: when should a diaphragm be modeled as a rigid or a flexible one? This is discussed in Section 1.11.4. A possible way for comparing the extent of diaphragm flexibility is through deflections. The deflection calculations for the rigid and flexible diaphragm case are presented below.

- ***Rigid diaphragm (see Example 2, step 8 for a similar calculation)***

The deflection will be calculated for point A as this should be the maximum. First, a reduction in the wall stiffness to account for the effect of cracking will be determined following the approach presented in Section 2.5.4 (S304-14 Cl.16.3.3), that is,

$$A_e = A_g\left[0.3 + P_s / \left(A_g f'_m\right)\right]$$

Here,

$P_s = 9.0*(9.0/2)*3.5 = 142$ kN (axial force due to dead load in wall X_2)

$A_e = (240*10^3)*9.0 = 216*10^4$ mm² (effective cross-sectional area for 240 mm block wall, solid grouted, length 9.0 m; see Table D-1 for A_e values for the unit wall length)

f'_m =10.0 MPa

Since

$$0.3 + P_s / \left(A_g f'_m\right) = 0.3 + 142*10^3 / \left(10.0*216*10^4\right) = 0.31$$

It appears that

$$\frac{A_e}{A_g} = 0.31$$

Because the behaviour of low-rise shear walls is expected to be shear dominant and so stiffness is proportional to cross-sectional area; thus

$$K_{ce} = \left(\frac{A_e}{A_g}\right)K_c = 0.31K_c$$

where K_c is elastic uncracked stiffness

Next, the translational displacement at point A can be calculated as follows:

$$\Delta^A{}_0 = \frac{V}{0.31\sum K_Y} = \frac{700kN}{0.31*3.3*10^6 kN/m} = 0.68mm$$

Subsequently, the torsional displacement at point A will be determined. Torsional rotation of the building θ can be found from the following equation:

$$\theta = \frac{T}{J} = \frac{5860kNm}{0.31*297*10^6} = 6.36*10^{-5} \text{ rad}$$

where (see the torsional calculations performed in part a) of this example)

$T = 5860$ kNm torsional moment

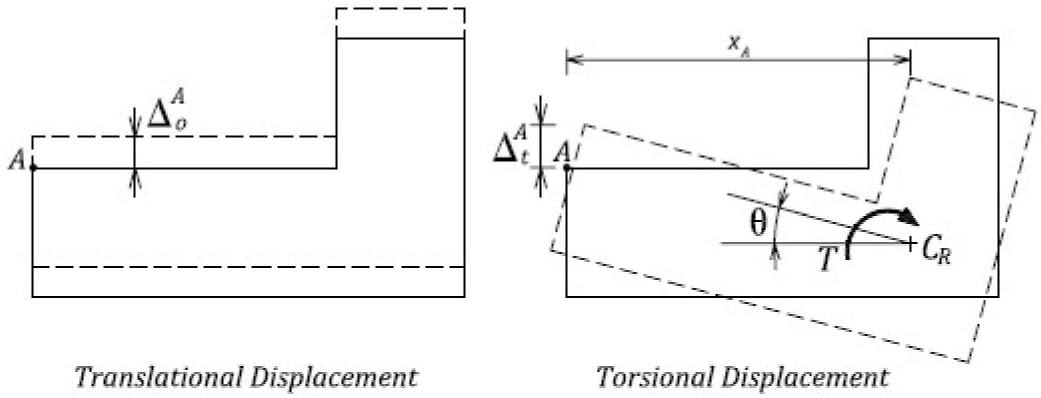
$J = 297 * 10^6$ elastic torsional stiffness (this value is reduced by 0.5 to take into account the cracking in the walls)

The torsional displacement at point A:

$\Delta^A{}_t = \theta * x_A = 6.36 * 10^{-5} * 24.05m = 1.53mm$

The total displacement at point A is can be found as follows (note that the displacements need to be multiplied by $R_d R_o / I_E$ ratio, where I_E = 1.0):

$\Delta^A{}_{\max} = \left(\Delta^A{}_0 + \Delta^A{}_t\right) * R_d R_o = (0.68 + 1.53) * 1.5 * 1.5 = 5.0mm$



- **Flexible diaphragm**

As a first approximation the calculation will consider a 21 m long diaphragm portion as a cantilever beam, as shown in the figure on the next page. This is an approximate model since the diaphragm is not fully fixed at that point, but the model is simple and useful for checking magnitude of deformations in a flexible diaphragm for this structure. The total shear force is equal to:

$V_D = 0.3 * [(9m * 21m) * (3.5kPa + 0.6kPa) + 2.4m * 21m * 5.38kPa] = 314kN$

and the equivalent uniform load is equal to

$v_D = V_D / L = 15.0$ kN/m

where

$L = 21.0$ m diaphragm length for the cantilevered portion

The real deflection will be larger since the diaphragm acting as a cantilever is not fully fixed at the wall Y_1, and walls Y_1, Y_2, and Y_3 also deflect; both effects provide some rotation at the fixed end of the cantilever.

Consider a plywood diaphragm with the following properties:

$E = 1500$ MPa plywood modulus of elasticity

$G = 600$ MPa plywood shear modulus

$t_D = 25.4$ mm (1” plywood thickness)

$A = b * t_D = 9.0m * 0.0254m = 0.23$ m^2

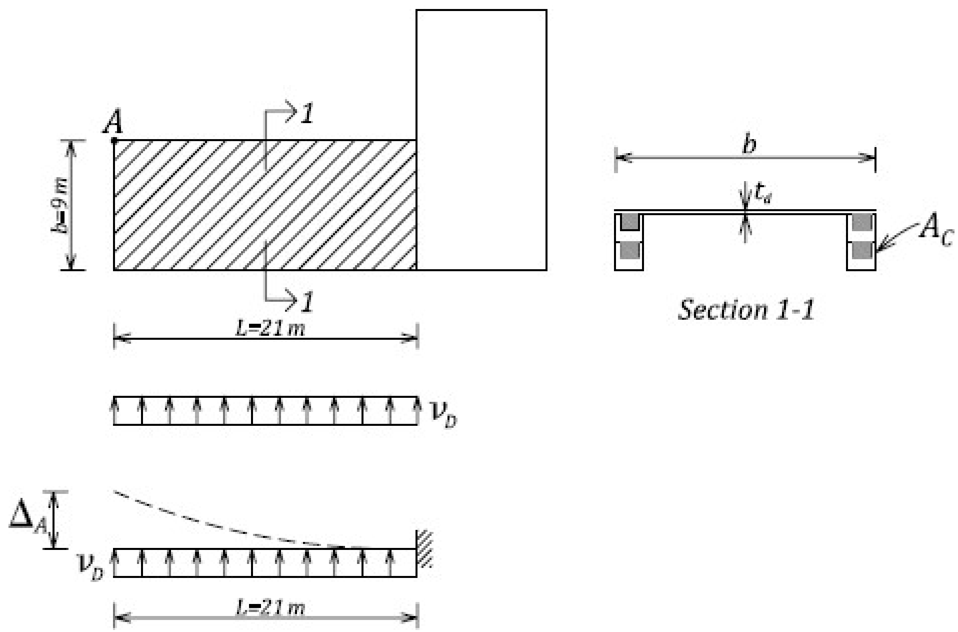
Let us assume that the two courses of grouted bond beam block act as a chord member, as shown on the figure on the next page. The roof-to-wall connection is achieved by means of nails driven into the anchor plate and hooked steel anchors welded to the plate embedded into the masonry. The corresponding moment of inertia around the centroid of the diaphragm can be found as follows:

$$I = 2 * A_c * \left(\frac{b}{2}\right)^2 = 2 * 0.096 * \left(\frac{9.0}{2}\right)^2 = 3.89 \text{ m}^4$$

where

$A_c = 2 * (0.24m * 0.2m) = 0.096$ m^2 chord area (two grouted 240 mm blocks)

$E_m = 8.5 * 10^6 \, kPa$ masonry modulus of elasticity based on f'_m = 10.0 MPa (solid grouted 20 MPa blocks and Type S mortar)



The total displacement at point A is equal to the combination of flexural and shear component, that is,

$$\Delta^A = \frac{v_D * L^4}{8E * I} + \frac{1.2V_D * L}{2 * A * G} = \frac{15.0 * (21.0)^4}{8 * 8.5 * 10^6 * 3.89} + \frac{1.2 * 314 * 21.0}{2 * 0.23 * 600 * 10^3} = (11.0 + 29.0) * 10^{-3} = 40 * 10^{-3} m = 40mm$$

The total displacement at point A is can be found by multiplying the above displacement by $R_d R_o / I_E$ ratio, that is,

$$\Delta^A{}_{\max} = \Delta^A * R_d R_o = 40 * 1.5 * 1.5 = 90mm$$

A quick check of the additional deflection caused by rotation at the fixed end of the cantilever indicates that an additional 50 mm could be expected at point A. Thus, the total displacement would be about 140 mm.

By comparing the displacements for the rigid and flexible diaphragm model, it can be observed that the difference is significant:

$\Delta^A{}_{\max} = 5mm$ rigid diaphragm model

$\Delta^A{}_{\max} = 90mm$ flexible diaphragm model

Had the flexible diaphragm been used, the lateral drift ratio at point A would be equal to:

$$DR = \frac{\Delta_{\max}}{h_w} = \frac{90}{4800} = 0.019 = 1.9 \text{ \%}$$

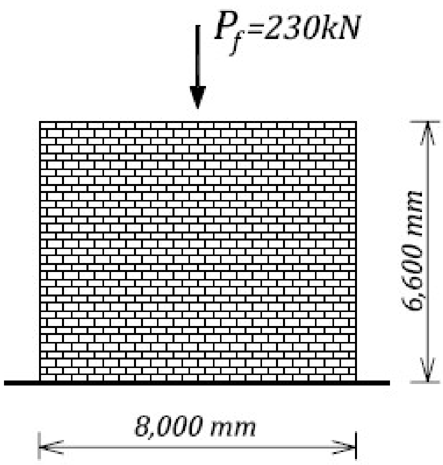
The drift is within the NBC 2015 limit of 2.5% (see Section 1.13); however, a flexible diaphragm would not be an ideal solution for this design – a rigid diaphragm would be the preferred solution.

Discussion
In this example, seismic forces were determined for the N-S walls due to seismic load acting in the N-S direction. It should be noted, however, that there is a significant eccentricity causing torsional effects in the E-W walls due to seismic load acting in the E-W direction – these calculations were not included in this example.

<u>EXAMPLE 4a</u>: Minimum seismic reinforcement for a <u>squat</u> shear wall

Determine minimum seismic reinforcement according to CSA S304-14 for a loadbearing masonry shear wall located in an area with a seismic hazard index $I_E F_a S_a(0.2)$ of 0.80. The wall is subjected to axial dead load (including its own weight) of 230 kN.

Use 200 mm hollow concrete blocks of 15 MPa unit strength and Type S mortar. Grade 400 steel reinforcing bars (yield strength f_y = 400 MPa) and cold-drawn galvanized wire (ASWG) joint reinforcement are used for this design.



Wall dimensions:
l_w =8000 mm length
h_w = 6600 mm height
t = 190 mm thickness

<u>SOLUTION:</u>
The purpose of this example is to demonstrate how the minimum seismic reinforcement area should be determined and distributed in horizontal and vertical direction. Once the reinforcement has been selected in terms of its area and distribution, the flexural and shear resistance of the wall will be determined and the capacity design issues discussed, as well as the seismic safety implications of vertical and horizontal reinforcement distribution.

1. Material properties
Steel (both reinforcing bars and joint reinforcement):
f_y = 400 MPa ϕ_s = 0.85
Note that the cold-drawn galvanized wire has higher yield strength than Grade 400 steel, but it will be ignored for the small area included.
Masonry:
ϕ_m = 0.6
Assume partially grouted masonry. For 15MPa blocks and Type S mortar, it follows from Table 4 of S304-14 that
f'_m = 9.8 MPa
Based on Note 3 to Table 4, this f'_m value is normally used for hollow block masonry but can also be used for partially grouted masonry if the grouted area is not considered.

2. Find the minimum seismic reinforcement area and spacing (see Section 2.6.9 and Table 2-3).

Since $I_E F_a S_a(0.2)$=0.80 > 0.35, minimum seismic reinforcement must be provided (S304-14 Cl.16.4.5.1).

Seismic reinforcement area

Loadbearing walls, including shear walls, shall be reinforced horizontally and vertically with steel having a minimum area of

$A_{s\,min} = 0.002A_g$ = 0.002*(190*10^3 mm^2/m) = 380 mm^2/m

for 190 mm block walls, where

A_g =(1000mm)*(190mm)=190*10^3 mm^2/m gross cross-sectional area for a unit wall length of 1 m

Minimum area in each direction (one-third of the total area):

$A'_{h\,min} = A'_{v\,min} = 0.00067A_g = \frac{A_{s\,min}}{3} = \frac{380}{3} = 127$ mm^2/m

Thus the minimum total vertical reinforcement area

$A_{v\,min} = 127 * l_w$ = (127 mm^2/m)(8 m) = 1016 mm^2

In distributing seismic reinforcement, the designer may be faced with the dilemma: should more reinforcement be placed in the vertical or in the horizontal direction? In theory, 1/3rd of the total amount of reinforcement can be placed in one direction and the remainder in the other direction. In this example, less reinforcement will be placed in the vertical direction, and more in the horizontal direction. The rationale for this decision will be explained later in this example.

Vertical reinforcement (area and distribution) (see Table 2-3):

Since $I_E F_a S_a(0.2)$=0.80 > 0.75, according to S304-14 Cl.16.4.5.3 spacing of vertical reinforcing bars shall not exceed the lesser of:

- $6(t+10)$ =6(190+10)=1200 mm
- 1200 mm

Therefore, the maximum permitted spacing of vertical reinforcement is equal to $s =$ 1200 mm.

Since the maximum permitted bar spacing is 1200 mm, a minimum of 8 bars are required (note that the total wall length is 8000 mm). Therefore, let us use 8-15M bars, so

A_v = 8*200 =1600 mm^2

(note that the resulting reinforcement spacing is going to be less than 1200 mm, which is the upper limit prescribed by S304-14).

The corresponding vertical reinforcement area per metre length is

$A'_v = \frac{A_v}{l_w} * 1000$ = 200 mm^2/m > $A'_{v\,min}$ =127 mm^2/m   OK

It should be noted that the requirements for spacing of vertical reinforcement have been relaxed for Conventional Construction masonry walls at sites where 0.35 ≤ $I_E F_a S_a(0.2)$<0.75 (see Table 2-3).

Horizontal reinforcement (area and distribution) (see Table 2-3):
Let us consider a combination of joint reinforcement and bond beam reinforcement. According to S304-14 Cl.16.4.5.4, where both types of reinforcement are used, the maximum spacing of bond beams is 2400 mm and of joint reinforcement is 400 mm, so the following reinforcement arrangement is considered:

- 9 Ga. ladder reinforcement @ 400 mm spacing, and
- 2-15M bond beam reinforcement @ 2200 mm (1/3rd of the overall wall height). The area of ladder reinforcement (2 wires) is equal to 22.4mm^2, and the area of a 15M bar is 200 mm^2. So, the total area of horizontal reinforcement per metre of wall height is

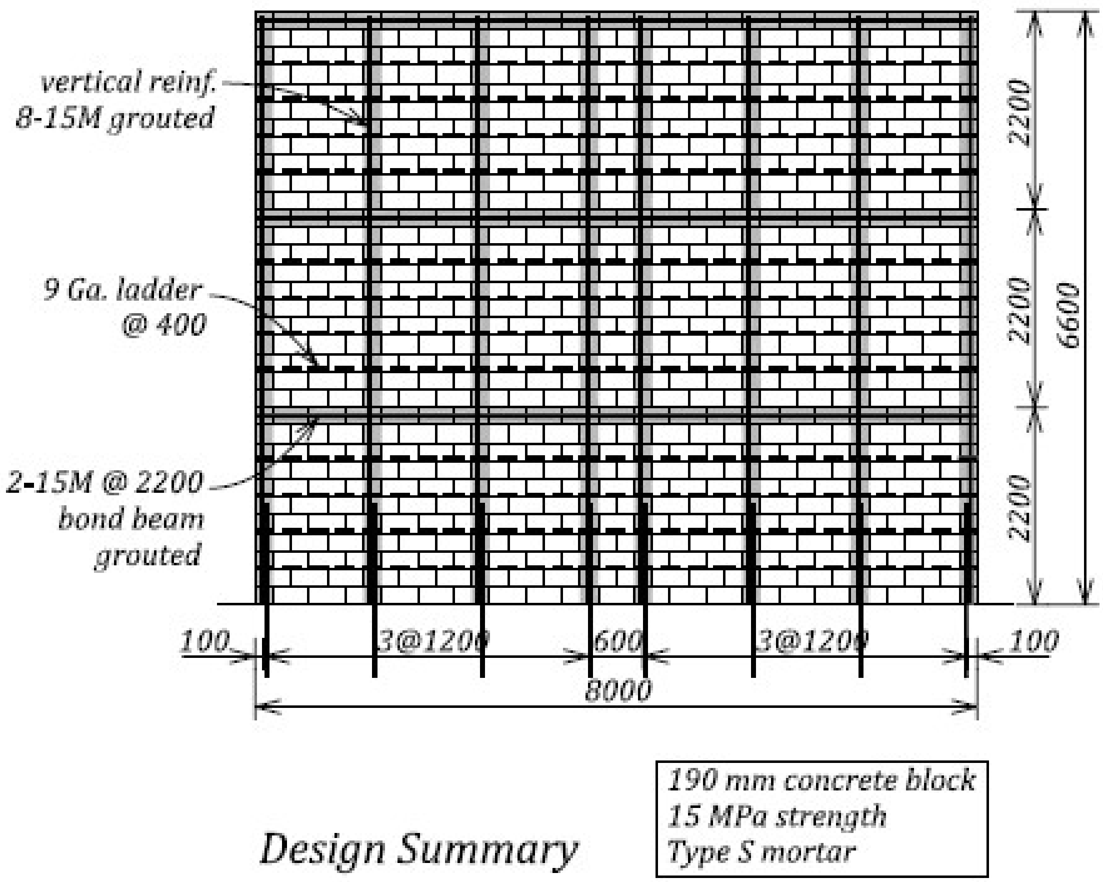
$A'_h = \left(\frac{22.4}{400} + \frac{400}{2200} \right) * 1000 = 238$ mm^2/m > $A'_{h\,\min}$ =127 mm^2/m   OK

So, the total area of horizontal and vertical reinforcement is
$A_s = A'_v + A'_h = 200 + 238$ =438 mm^2/m   > $A_{s\,\min}$ =380 mm^2/m   OK

Note that the total area (438 mm^2/m) exceeds the S304-14 minimum requirements (380 mm^2/m) by about 10%. It is difficult to select reinforcement that exactly meets the requirements, and also a reserve in reinforcement area provides additional safety for seismic effects.

3. Check whether the vertical reinforcement meets the minimum requirements for loadbearing walls (S304-14 Cl.10.15.1.1 – see Table 2-3).
Since this is a shear wall, but also a loadbearing wall, pertinent reinforcement requirements would need to be checked, however the check is omitted from this example since it does not govern in seismic zones.

4. Design summary
The reinforcement arrangement for the wall under consideration is summarized below.

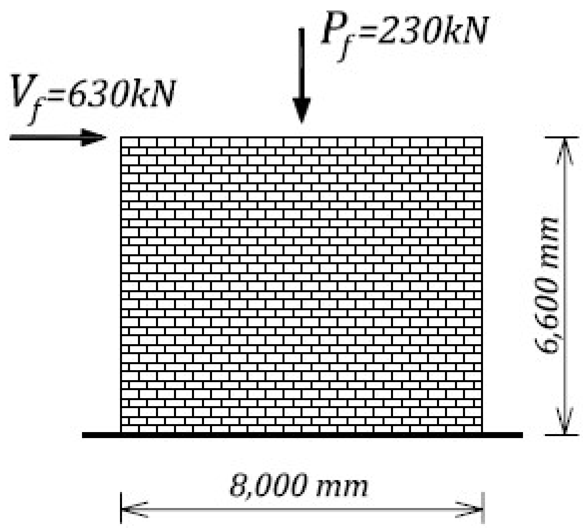


Design Summary

EXAMPLE 4b: Seismic design of a Conventional Construction squat shear wall

Design a single-storey squat concrete block shear wall shown in the figure below according to NBC 2015 and CSA S304-14 seismic requirements for Conventional Construction reinforced masonry walls. The building site is located at the site supported by Site Class C soil, and the seismic hazard index $I_E F_a S_a(0.2)$ is 0.66. The wall is subjected to a total dead load of 230 kN (including the wall self-weight) and an in-plane seismic force of 630 kN. Consider the wall to be solid grouted. Neglect the out-of-plane effects in this design.

Use 200 mm hollow concrete blocks of 15 MPa unit strength and Type S mortar. Grade 400 steel reinforcing bars (yield strength f_y = 400 MPa) and cold-drawn galvanized wire (ASWG) joint reinforcement are used for this design.



Wall dimensions:
l_w =8000 mm length
h_w = 6600 mm height
t = 190 mm thickness

SOLUTION:

1. Material properties
Steel (both reinforcing bars and joint reinforcement):
ϕ_s = 0.85 f_y = 400 MPa
Masonry:
ϕ_m = 0.6
S304-14 Table 4, 15 MPa concrete blocks and Type S mortar:
f'_m = 7.5 MPa (assume solid grouted masonry)

2. Load analysis
The wall needs to be designed for the following load effects:

- P_f = 230 kN axial load
- V_f = 630 kN seismic shear force
- $M_f = V_f * h$ = 630*6.6 ≈ 4160 kNm overturning moment at the base of the wall

Note that, according to NBC 2015 Table 4.1.3.2, load combination for the dead load and seismic effects is 1.0*D + 1.0*E.

3. Minimum CSA S304-14 seismic reinforcement (see Section 2.6.9 and Table 2-3)
Since $I_E F_a S_a(0.2)$ = 0.66 > 0.35, minimum seismic reinforcement is required (S304-14 Cl.16.4.5.1). See Example 4a for a detailed calculation of the S304-14 minimum seismic reinforcement.

4. Design for the combined axial load and flexure
A design for the combined effects of axial load and flexure will be performed using two different procedures: i) by considering uniformly distributed vertical reinforcement, and ii) by considering concentrated and distributed reinforcement.

Distributed wall reinforcement (see Section C.1.1.2)
This procedure assumes uniformly distributed vertical reinforcement over the wall length. The total vertical reinforcement area can be estimated, and the estimate can be revised until the moment resistance value is sufficiently large. After a few trial estimates, the total area of vertical reinforcement was determined as

A_{vt} = 3200 mm^2 > 1016 mm^2 (minimum seismic reinforcement) - OK

Try 16-15M bars for vertical reinforcement.
The wall is subjected to axial load

P_f = 230 kN

The approximate moment resistance for the wall section is given by:

$\alpha_1 = 0.85 \qquad \beta_1 = 0.8$

$$\omega = \frac{\phi_s f_y A_{vt}}{\phi_m f'_m l_w t} = \frac{0.85 * 400 * 3200}{0.6 * 7.5 * 8000 * 190} = 0.159$$

$$\alpha = \frac{P_f}{\phi_m f'_m l_w t} = \frac{230 * 10^3}{0.6 * 7.5 * 8000 * 190} = 0.034$$

$$c = \frac{\omega + \alpha}{2\omega + \alpha_1 \beta_1} l_w = \frac{0.159 + 0.034}{2 * 0.159 + 0.85 * 0.8}(8000) = 1547 \text{ mm}$$

$$M_r = 0.5\phi_s f_y A_{vt} l_w \left(1 + \frac{P_f}{\phi_s f_y A_{vt}}\right)\left(1 - \frac{c}{l_w}\right) = 0.5 * 0.85 * \frac{400}{1000} * 3200 * \frac{8000}{1000}\left(1 + \frac{230 * 10^3}{0.85 * 400 * 3200}\right)\left(1 - \frac{1544}{8000}\right)$$

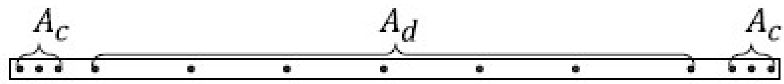
$M_r = 4253$ kNm > $M_f = 4160$ kNm OK

Distributed and concentrated wall reinforcement (see Section C.1.1.1)

This procedure assumes the same total reinforcement area, but the concentrated reinforcement is provided at the wall ends, and the remaining reinforcement is distributed over the wall length.

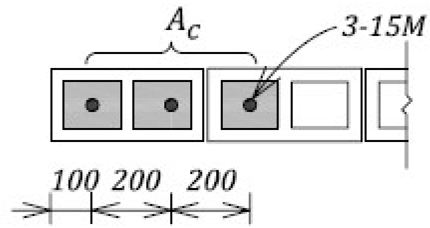
A_{vt} = 3200 mm^2

Concentrated reinforcement area at each wall end (3-15M bars in total, 1-15M in last 3 cells):
A_c = 600 mm^2



Distributed reinforcement
A_d =3200-2*600=2000 mm^2

Distance from the wall end to the centroid of concentrated reinforcement
$d' = 300$ mm



The compression zone depth a:

$$a = \frac{P_f + \phi_s f_y A_d}{0.85\phi_m f'_m\, t} = \frac{230*10^3 + 0.85*400*2000}{0.85*0.6*7.5*190} \quad = 1252 \text{ mm}$$

The masonry compression resultant C_r:

$$C_m = \left(0.85\phi_m f'_m\right)\left(t \cdot a\right) = (0.85*0.6*7.5)(190*1252) = 910 \text{ kN}$$

The factored moment resistance M_r will be determined by summing up the moments around the centroid of the wall section as follows (see equation (3) in Section C.1.1.1)

$$M_r = \left[C_m (l_w - a)/2 + 2\left(\phi_s f_y A_c\right)\left(l_w/2 - d'\right)\right]*10^{-6}$$

$$= \left[910*10^3*(8000-1252)/2 + 2*\left(0.85*400*600\right)\left(8000/2 - 300\right)\right]*10^{-6} \quad M_r = 4580 \text{ kNm}$$

The second procedure was used as a reference (to confirm the results of the first procedure). Both procedure give similar M_r values (4253 kNm and 4580 kNm by the first and second procedure respectively).

5. Find the minimum required factored shear resistance (see Section 2.6.5 and S304-14 Cl.16.5.4)

Cl.16.5.4 requires that the factored shear resistance, V_r, for a Conventional Construction shear wall should be greater than the shear due to effects of factored loads, but not less than i) the shear corresponding to the development of factored moment capacity, M_r, or ii) shear corresponding to the lateral seismic load (base shear), where earthquake effects were calculated using $R_d R_o$=1.3.

The first requirement is based on the Capacity Design approach (see Section 2.5.1 for more details). For Conventional Construction shear walls, the shear capacity should exceed the shear corresponding to the nominal moment capacity, as follows

$M_r = 4253$ kNm

The shear force V_{rb} corresponding to the overturning moment M_r is equal to

$V_{rb} = \frac{M_r}{h} = \frac{4253}{6.6} =$ 645 kN

The second requirement gives an "almost elastic" factored base shear force for the wall, which is equal to

$V_{fe} = \frac{V_f \cdot R_d \cdot R_o}{1.3} = \frac{630 \cdot 1.5 \cdot 1.5}{1.3} = 1090$ kN

The smaller of these two values should be used, hence

$V_{rd} = 645$ kN

6. Find the diagonal tension shear resistance (see Section 2.3.2 and S304-14 Cl.10.10.2.1).

Masonry shear resistance (V_m):

$b_w = 190$ mm overall wall thickness

$d_v \approx 0.8 l_w = 6400$ mm effective wall depth

$\gamma_g = 1.0$ solid grouted wall

$P_d = 0.9 P_f$ = 207 kN

$v_m = 0.16(2 - \frac{M_f}{V_f d_v})\sqrt{f'_m}$ = 0.44 MPa

$\frac{M_f}{V_f d_v} = \frac{4160}{630 * 6.4}$ = 1.03 ≈ 1.0

$V_m = \phi_m (v_m b_w d_v + 0.25 P_d)\gamma_g$ = 0.6(0.44*190*6400+0.25*207*10³)*1.0 = 352 kN

Steel shear resistance V_s (2-15M bond beam reinforcement at 1200 mm spacing):

$V_s = 0.6\phi_s A_v f_y \frac{d_v}{s} = 0.6 * 0.85 * \frac{400}{1000} * 400 * \frac{6400}{1200}$ = 435 kN

Total shear resistance

$V_r = V_m + V_s = 352 + 435 = 787$ kN

The factored shear resistance exceeds the minimum required factored shear resistance, that is,

$V_r = 787$ kN > $V_{rd} = 645$ kN OK

This is a squat shear wall because $\frac{h_w}{l_w} = \frac{6600}{8000} = 0.825 \le 1.0$. Maximum shear allowed on the section is (S304-14 Cl.10.10.2.1)

$\max V_r = 0.4\phi_m \sqrt{f'_m} b_w d_v \gamma_g (2 - \frac{h_w}{l_w}) = 939$ kN

Since

$V_r < \max V_r$ OK

Note that a solid grouted wall is required, that is, $\gamma_g = 1.0$. A partially grouted wall would have $\gamma_g = 0.5$, so its shear capacity would not be adequate for this design.

7. Sliding shear resistance (see Section 2.3.3)

The factored in-plane sliding shear resistance V_r is determined as follows.

μ = 1.0 for a masonry-to-masonry or masonry-to-roughened concrete sliding plane

A_s = 3200 mm^2 total area of vertical wall reinforcement

$T_y = \phi_s A_s f_y$ = 0.85*3200*400 = 1088 kN

P_d = 207 kN

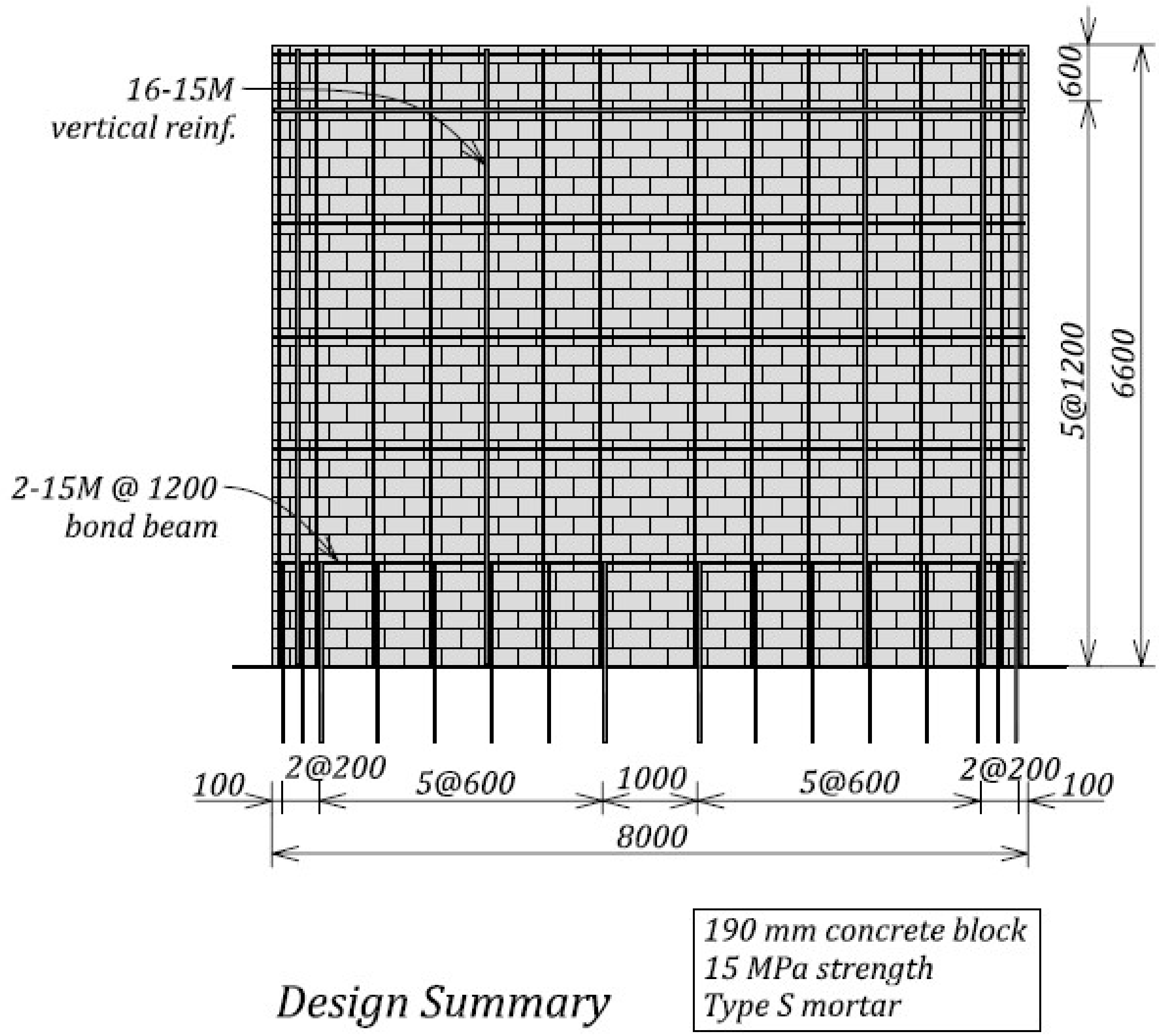
$P_2 = P_d + T_y$ = 207+1088 = 1295 kN

$V_r = \phi_m \mu P_2$ = 0.6*1.0*1295=777 kN

$V_r = 777$ kN > $V_{rd} = 645$ kN   OK

8. Design summary

The reinforcement arrangement for the wall under consideration is shown in the figure below. Note that the wall is solidly grouted. A bond beam (transfer beam) is provided atop the wall to ensure uniform shear transfer along the entire length (see Section 2.3.2.2).



9. Discussion

It is important to consider all possible behaviour modes and identify the one that governs in this design. There are three shear forces:

a) $V_{rd} = 645$ kN minimum required factored shear resistance

b) $V_r = 787$ kN diagonal tension shear resistance

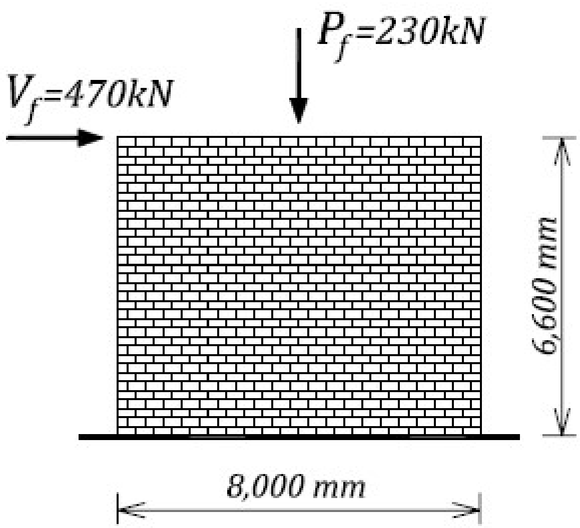
c) $V_r = 777$ kN sliding shear resistance

Since the minimum required factored shear resistance is smallest of the three values, it can be concluded that the flexural failure mechanism is critical in this case, which is desirable for seismic design.

Note that S304-14 Cl.10.2.8 prescribes the use of a reduced effective depth d for the flexural design of squat shear walls. This example deals with seismic design, and the wall reinforcement is expected to yield in tension, this provision was not followed since it would lead to a non-conservative design; instead, the actual effective depth was used for flexural design.

EXAMPLE 4c: Seismic design of a Moderately Ductile squat shear wall
Design a single-storey squat concrete block shear wall shown on the figure below according to NBC 2015 and CSA S304-14 seismic requirements for moderately ductile squat shear walls (note that the same shear wall was designed in Example 4b as a conventional construction). The building site is located in Ottawa, ON and the seismic hazard index $I_E F_a S_a(0.2)$ is 0.66. The wall is subjected to the total dead load of 230 kN (including the wall self-weight) and the in-plane seismic force of 470 kN; this reflects the higher R_d value of 2.0 that can be used for walls with Moderate Ductility. Consider the wall to be solid grouted. Neglect the out-of-plane effects in this design.

Use 200 mm hollow concrete blocks of 15 MPa unit strength and Type S mortar. Grade 400 steel reinforcing bars (yield strength f_y = 400 MPa) and cold-drawn galvanized wire (ASWG) joint reinforcement are used for this design.



Wall dimensions:
l_w =8000 mm length
h_w = 6600 mm height
t = 190 mm thickness

Note that the h/t ratio exceeds the S304.1 limit of 20 for moderately ductile squat shear walls (Cl.10.16.6.3).

SOLUTION:
Since

$$\frac{h_w}{l_w} = \frac{6600}{8000} = 0.825 \leq 1.0$$

this is a squat shear wall. The wall is to be designed as a moderately ductile squat shear wall, and NBC 2015 Table 4.1.8.9 specifies the following R_d and R_o values (see Table 1-13):
R_d = 2.0 and R_o = 1.5
The seismic shear force of 470 kN for a wall with moderate ductility ($R_d = 2.0$) was obtained by prorating the force of 630 kN from Example 4b which corresponded to a shear wall with conventional construction ($R_d = 1.5$), as follows

$$V_f = 630 * \frac{1.5}{2.0} \approx 470 \text{ kN}$$

1. Material properties
Steel (both reinforcing bars and joint reinforcement):
ϕ_s = 0.85 f_y = 400 MPa
Masonry:

ϕ_m = 0.6
From S304-14 Table 4, 15 MPa concrete blocks and Type S mortar:
f'_m = 7.5 MPa (assume solid grouted masonry)

2. Load analysis

The wall needs to be designed for the following load effects:

- P_f = 230 kN axial load
- V_f = 470 kN seismic shear force
- $M_f = V_f * h$ = 470*6.6 ≈ 3100 kNm overturning moment at the base of the wall

Note that, according to NBC 2015 Table 4.1.3.2, the load combination for the dead load and seismic effects is 1.0*D + 1.0*E.

3. Minimum S304-14 seismic reinforcement (see Section 2.6.9 and Table 2-3)

Since $I_E F_a S_a(0.2)$ = 0.66 > 0.35, minimum seismic reinforcement is required (Cl.16.4.5.1). See Example 4a for a detailed calculation of the S304-14 minimum seismic reinforcement.

4. Design for the combined axial load and flexure (see Section C.1.1.2).

A design for the combined effects of axial load and flexure will be performed by assuming uniformly distributed vertical reinforcement over the wall length. After a few trial estimates, the total area of vertical reinforcement was determined as

A_{vt} = 2200 mm^2 > 1016 mm^2 (minimum seismic reinforcement) - OK

and so 11-15M reinforcing bars can be used for vertical reinforcement in this design (total area of 2200 mm^2).

The wall is subjected to axial load P_f = 230 kN. Note that the load factor for the load combination with earthquake load is equal to 1.0.

The moment resistance for the wall section can be determined from the following equations (see Example 4b):

$\alpha_1 = 0.85 \quad \beta_1 = 0.8 \quad \omega = 0.109 \quad \alpha = 0.034 \quad c = 1273$ mm

$$M_r = 0.5\phi_s f_y A_{vt} l_w \left(1 + \frac{P_f}{\phi_s f_y A_{vt}}\right)\left(1 - \frac{c}{l_w}\right) = 0.5 * 0.85 * \frac{400}{1000} * 2200 * \frac{8000}{1000}\left(1 + \frac{230 * 10^3}{0.85 * 400 * 2200}\right)\left(1 - \frac{1273}{8000}\right)$$

$M_r \cong 3290$ kNm > $M_f = 3100$ kNm OK

5. Height/thickness ratio check (see Section 2.6.4)

S304-14 prescribes the following height-to-thickness (h/t) limit for the compression zone in moderately ductile squat shear walls (Cl.16.7.4):

$h/(t+10) < 20$, unless it can be shown for lightly loaded walls that a more slender wall is satisfactory for out-of-plane stability.

For this example,
h = 6600 mm (unsupported wall height)
t = 190 mm actual wall thickness

So,
$h/(t+10) = 6600/(190+10) = 33 > 20$
The height-to-thickness ratio for this wall exceeds the S304-14 limits by a significant margin. However, S304-14 permits the height-to-thickness restrictions for moderately ductile squat shear walls to be relaxed, provided that the designer can show that the out-of-plane wall stability is satisfactory.

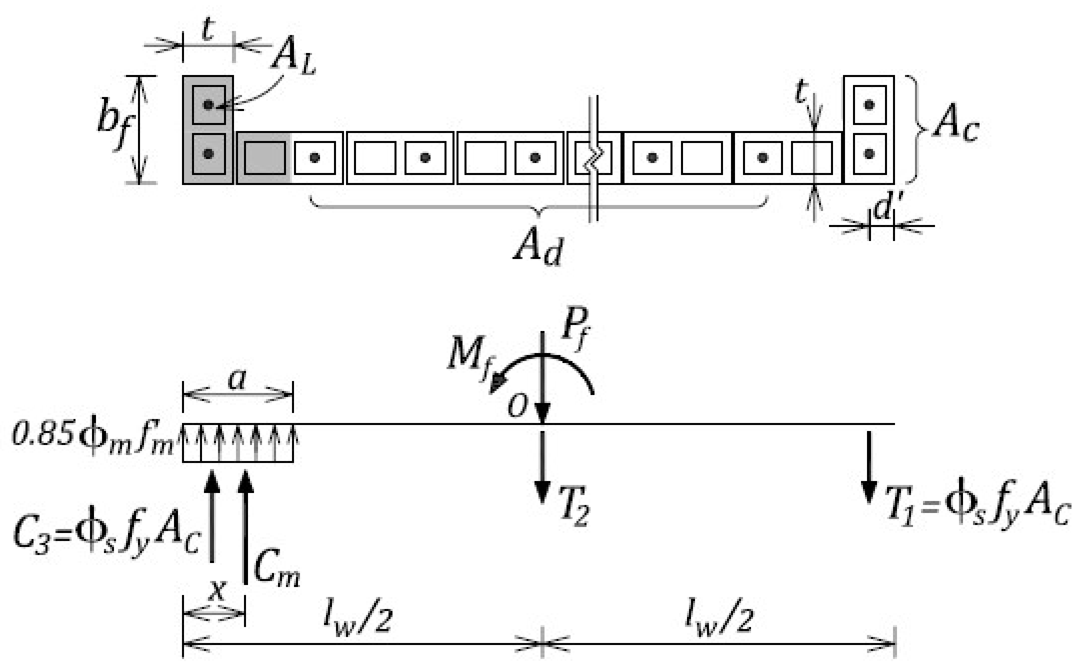
This is a lightly loaded wall in a single-storey building. The total dead load is 230 kN, which corresponds to the compressive stress of

$$f_c = \frac{P_f}{l_w t} = \frac{230*10^3}{8000*190} = 0.15 \text{ MPa}$$

This stress corresponds to only 2% of the masonry compressive strength f'_m which is equal to 7.5 MPa. In general, a compressive stress below 0.1 f'_m (equal to 0.75 MPa in this case) is considered to be very low.

The recommendations included in the commentary to Section 2.6.4 will be followed here. A possible solution involves the provision of flanges at the wall ends. The out-of-plane stability of the compression zone must be confirmed for this case.

Try an effective flange width $b_f = 390$ mm. The wall section and the internal force distribution is shown on the figure below.



This procedure assumes the same total reinforcement area A_{vt} as determined in step 4, but the concentrated reinforcement is provided at the wall ends, while the remaining reinforcement is distributed over the wall length.

A_{vt} = 2200 mm^2

Concentrated reinforcement area (2-15M bars at each wall end):

A_c = 400 mm^2

Distributed reinforcement area:

A_d =2200-2*400=1400 mm^2

Distance from the wall end to the centroid of concentrated reinforcement A_c:

$d' = 100$ mm

- Check the buckling resistance of the compression zone.

The area of the compression zone A_L:

$$A_L = \frac{P_f + \phi_s f_y A_d}{0.85\phi_m f'_m} = \frac{230*10^3 + 0.85*400*1400}{0.85*0.6*7.5} = 1.846*10^5 \text{ mm}^2$$

The depth of the compression zone a:

$$a = \frac{A_L - b_f * t + t^2}{t} = \frac{1.846*10^5 - (390*190) + 190^2}{190} = 772 \text{ mm}$$

The neutral axis depth:

$$c = \frac{a}{0.8} = 965 \text{ mm}$$

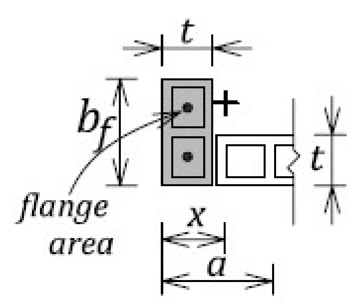
The centroid of the masonry compression zone:

$$x = \frac{t*\left(a^2/2\right) + (b_f - t)\left(t^2/2\right)}{A_L} = 326 \text{ mm}$$

In this case, the compression zone is L-shaped, however only the flange area will be considered for the buckling resistance check (see the shaded area shown on the figure below). This is a conservative approximation and it is considered to be appropriate for this purpose, since the gross moment of inertia is used.

Gross moment of inertia for the flange only:

$$I_{xg} = \frac{t*b_f^{\ 3}}{12} = \frac{190*390^3}{12} = 9.39*10^8 \text{ mm}^4$$



The buckling strength for the compression zone will be determined according to S304-14 Cl.10.7.4.3, as follows:

$$P_{cr} = \frac{\pi^2 \varphi_{er} E_m I}{\left(1+0.5\beta_d\right)\left(kh\right)^2} = 1017 \text{ kN}$$

where

$\phi_{er} = 0.75$

$k = 1.0$ pin-pin support conditions

$\beta_d = 0$ assume 100% seismic live load

$h = 6600$ mm unsupported wall height

$E_m = 850 f'_m = 6375$ MPa modulus of elasticity for masonry

- Find the resultant compression force (including the concrete and steel component).

$$P_{fb} = C_m + \phi_s f_y A_c = 706*10^3 + 0.85*400*400 = 842 \text{ kN}$$

where

$$C_m = \left(0.85\phi_m f'_m\right)A_L = (0.85*0.6*7.5)(1.846*10^5) = 706 \text{ kN}$$

- Confirm that the out-of-plane buckling resistance is adequate.

Since

$P_{fb} = 842$ kN < $P_{cr} = 1017$ kN

it can be concluded that the out-of-plane buckling resistance is adequate and so the flanged section can be used for this design. This is in compliance with S304-14 Cl.16.7.4, despite the fact that the h/t ratio for this wall is 33, which exceeds the S304-14-prescribed limit of 20.

4a. Design the flanged section for the combined axial load and flexure – consider distributed and concentrated wall reinforcement (see Section C.1.1.1).

The key design parameters for this calculation were determined in step 5 above. The factored moment resistance M_r will be determined by summing up the moments around the centroid of the wall section as follows

$$M_r = C_m(l_w/2 - x) + 2(\phi_s f_y A_c)(l_w/2 - d') = 706*10^3*(8000/2 - 326) + 2*(0.85*400*400)*(8000/2 - 100)$$

$M_r = 3655*10^6\ Nmm = 3655$ kNm

Since

$M_r = 3655$ kNm > $M_f = 3100$ kNm OK

6. Find the minimum required factored shear resistance (see Section 2.6.5 and S304-14 Cl.16.7.3.2)

Cl.16.7.3.2 requires that the factored shear resistance, V_r, for a Moderately Ductile squat shear wall should be greater than the shear due to effects of factored loads, but not less than i) the shear corresponding to the development of factored moment resistance, M_r, or ii) shear corresponding to the lateral seismic load (base shear), where earthquake effects were calculated using $R_d R_o$=1.3.

The first requirement is based on the Capacity Design approach (see Section 2.5.1 for more details). For Moderately Ductile shear walls, the shear capacity should exceed the shear corresponding to the factored moment resistance. In this case, the factored moment resistance is equal to

$M_r = 3655$ kNm

The shear force at the top of the wall that would cause an overturning moment equal to M_r is

$$V_{rb} = \frac{M_r}{h_w} = \frac{3655}{6.6} = 554 \text{ kN}$$

The second requirement gives an "almost elastic" factored base shear force for the wall, which is equal to

$$V_{fe} = \frac{V_f \cdot R_d \cdot R_o}{1.3} = \frac{470 \cdot 2.0 \cdot 1.5}{1.3} = 1085 \text{ kN}$$

The smaller of these two values should be used, hence

$V_{rd} = 554$ kN

7. The diagonal tension shear resistance (see Section 2.3.2 and S304-14 Cl.10.10.2.1)

Masonry shear resistance (V_m):

$b_w = 190$ mm overall wall thickness

$d_v \approx 0.8 l_w = 6400$ mm effective wall depth

$\gamma_g = 1.0$ solid grouted wall

$P_d = 0.9P_f$ = 207 kN

$v_m = 0.16(2 - \frac{M_f}{V_f d_v})\sqrt{f'_m}$ = 0.44 MPa

$\frac{M_f}{V_f d_v} = \frac{3100}{470 * 6.4}$ = 1.03 ≈ 1.0

$V_m = \phi_m (v_m b_w d_v + 0.25P_d)\gamma_g$ = 0.6(0.44*190*6400+0.25*207*10³)*1.0 = 352 kN

Steel shear resistance V_s:

Assume 2-15M bond beam reinforcement at 1200 mm spacing, so

$A_v = 400$ mm²

$s = 1200$ mm

Horizontal reinforcement area per metre:

$A_h{}' = \frac{A_v}{s} * 1000 = \frac{400}{1200} * 1000 = 333$ mm²/m

$V_s = 0.6\phi_s A_v f_y \frac{d_v}{s} = 0.6 * 0.85 * \frac{400}{1000} * 400 * \frac{6400}{1200}$ = 435 kN

Total diagonal shear resistance

$V_r = V_m + V_s = 352 + 435 = 787$ kN

The factored shear resistance exceeds the minimum required factored shear resistance, that is,

$V_r = 787$ kN > $V_{rd} = 554$ kN OK

Maximum shear allowed on the section is (S304-14 Cl.10.10.2.2)

$\max V_r = 0.4\phi_m \sqrt{f'_m} b_w d_v \gamma_g (2 - \frac{h_w}{l_w}) = 939$ kN

Since

$V_r < \max V_r$ OK

Note that S304-14 Cl.16.7.3.1 requires that the method by which the shear force is applied to the wall shall be capable of applying shear force uniformly over the wall length. This can be achieved by providing a continuous bond beam at the top of the wall, as discussed in Section 2.3.2.2 (see Figure 2-16).

8. Sliding shear resistance (see Section 2.3.3)

The factored in-plane sliding shear resistance V_r is determined as follows.

μ = 1.0 for a masonry-to-masonry or masonry-to-roughened concrete sliding plane

A_s = 2200 mm² total area of vertical wall reinforcement

$T_y = \phi_s A_s f_y$ = 0.85*2200*400 = 748 kN

P_d = 207 kN

$P_2 = P_d + T_y$ = 207+748 = 955 kN

$V_r = \phi_m \mu P_2$ = 0.6*1.0*955 = 573 kN

$V_r = 573$ kN > $V_{rd} = 554$ kN OK

Note that $V_r = 573$ kN < $V_r = 787$ kN for diagonal tension (this indicates that the sliding shear resistance governs over the diagonal tension shear resistance).

9. Minimum reinforcement requirements for Moderately Ductile squat shear walls (see Section 2.6.10)

S304-14 Cl.16.7.5 prescribes the following requirements for the amount of reinforcement in Moderately Ductile squat shear walls:

Horizontal reinforcement ratio ρ_h

ρ_h should be greater than the minimum value set by S304-14 Cl.16.7.5:

$$\rho_{h\min} = \frac{V_f}{b_w \cdot h_w \cdot \phi_s \cdot f_y} = \frac{470*10^3}{190*6600*0.85*400} = 1.10*10^{-3}$$

and the value determined in accordance with Cl.10.10.2 based on the shear resistance requirements

$$\rho_{hshear} = \frac{A_h}{b_w * h_w} = \frac{2131}{190*6600} = 1.70*10^{-3}$$

where A_h is the total area of horizontal reinforcement along the wall height, that is,

$A_h = A_h' \bullet d_v = 333 \bullet 6.4 = 2131$ mm^2

where

$A_h' = 333$ mm^2/m (see step 6)

In this case,

$\rho_{h\min}$ =1.10*10^{-3} < ρ_{hshear} =1.70*10^{-3}

This indicates that the S304-14 shear resistance requirement governs. The amount of horizontal reinforcement (2-15M bond beam reinforcement bar at 1200 mm spacing) is adequate.

Vertical reinforcement ratio ρ_v

Minimum $\rho_{v\min}$ value set by S304-14 Cl.16.7.5:

$$\rho_{v\min} \geq \rho_{h\min} - \frac{P_s}{\phi_s \cdot b_w \cdot l_w \cdot f_y} = 1.10*10^{-3} - \frac{230*10^3}{0.85*190*8000*400} = 0.65*10^{-3}$$

where $P_s = P_f = 230$ kN. Actual vertical reinforcement ratio ρ_{vflex} based on the flexural design requirements (see step 4):

$$\rho_{vflex} = \frac{A_{vt}}{l_w * t} = \frac{2200}{8000*190} = 1.447*10^{-3}$$

Since

ρ_{vflex} = 1.447*10^{-3} > $\rho_{v\min}$ = 0.65*10^{-3}

It appears that the amount of vertical reinforcement determined based on the flexural design requirements (11-15M) governs. It can be concluded that the minimum S304-14 reinforcement requirements for Moderately Ductile shear walls have been satisfied.

10. Shear resistance at the web-to-flange interface (see Section C.2 and Cl.7.11).
The factored shear stress at the web-to-flange interface is equal to the larger of horizontal and vertical shear stress, as shown below.
Horizontal shear:

$$v_f = \frac{V_{rd}}{t_e l_w} = \frac{554*10^3}{190*8000} = 0.36 \text{ MPa}$$

where t_e = 190 mm (effective wall thickness)

Vertical shear (caused by the resultant compression force P_{fb} calculated in Step 5):

$$v_f = \frac{P_{fb}}{b_w * h_w} = \frac{842*10^3}{190*6600} = 0.67 \text{ MPa} \qquad \text{governs}$$

Factored shear strength for bonded interfaces (S304-14 Cl.7.11.1):

$$v_m = 0.16\phi_m\sqrt{f'_m} = 0.26 \text{ MPa}$$

Since

$v_f = 0.67$ MPa > $v_m = 0.26$ MPa

shear reinforcement at the web-to-flange interface is required. Since the horizontal reinforcement consists of 2-15M bars @ 1200 mm spacing, both bars can be extended into the flange (90° hook), and so

$$v_s = \frac{\varphi_s A_s f_y}{s \cdot t_e} = \frac{0.85*2*200*400}{1200*190} = 0.60 \text{ MPa}$$

The total shear resistance

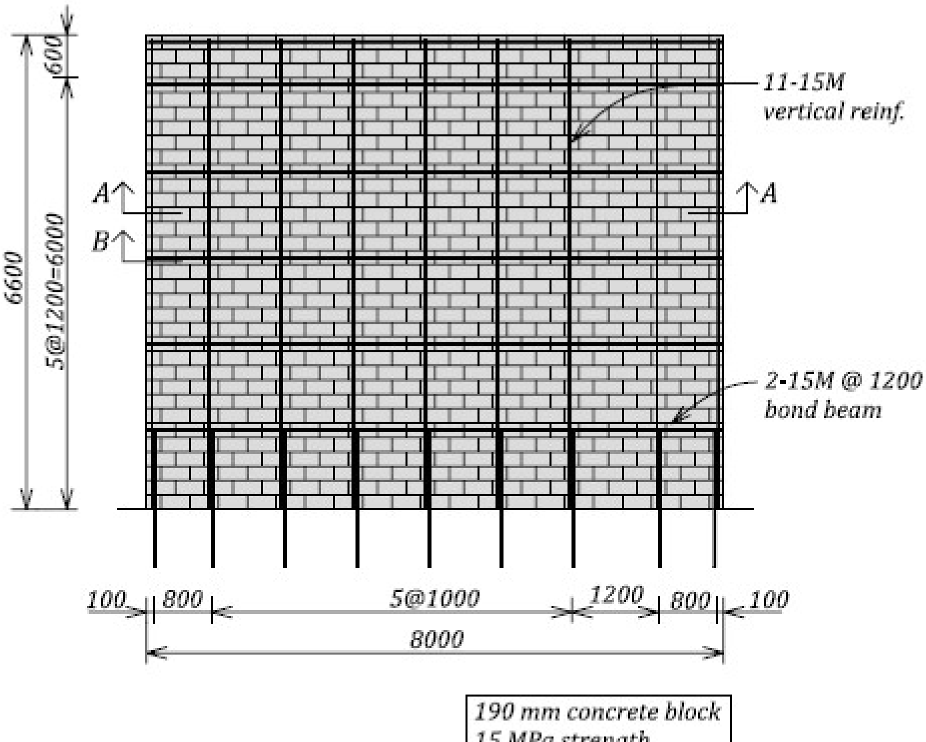
$$v_r = v_m + v_s = 0.26 + 0.60 = 0.86 \text{ MPa}$$

Since

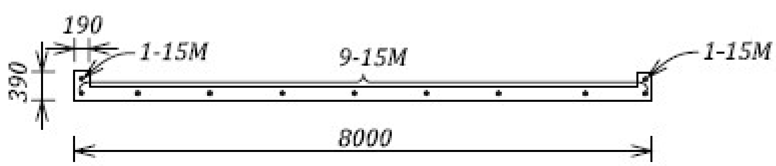
$v_f = 0.67$ MPa < $v_r = 0.86$ MPa

the shear resistance at the web-to-flange interface is satisfactory.

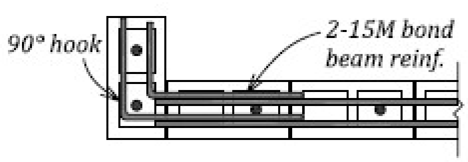
11. Design summary
The reinforcement arrangement for the wall under consideration is shown in the figure below. Note that the wall is solid grouted.



Design Summary



Section A-A



Section B-B

11. Discussion

It is important to consider all possible behaviour modes and identify the one that governs in this design. There are three shear forces:

a) $V_{rd} = 554$ kN minimum required factored shear resistance

b) $V_r = 787$ kN diagonal tension shear resistance

c) $V_r = 573$ kN sliding shear resistance

Since the minimum required factored shear resistance is smallest of the three values, it can be concluded that the flexural failure mechanism is critical in this case, which is desirable for seismic design.

Note that S304-14 Cl.10.2.8 prescribes the use of reduced effective depth d for flexural design of squat shear walls. Since this example deals with seismic design and essentially all the wall reinforcement is expected to yield in tension, this provision was not used as it is expected to result in additional vertical reinforcement, which would increase the moment capacity and possibly lead to a more brittle diagonal shear failure.

Note that the S304-14 ductility check is not prescribed for Moderately Ductile squat shear walls.

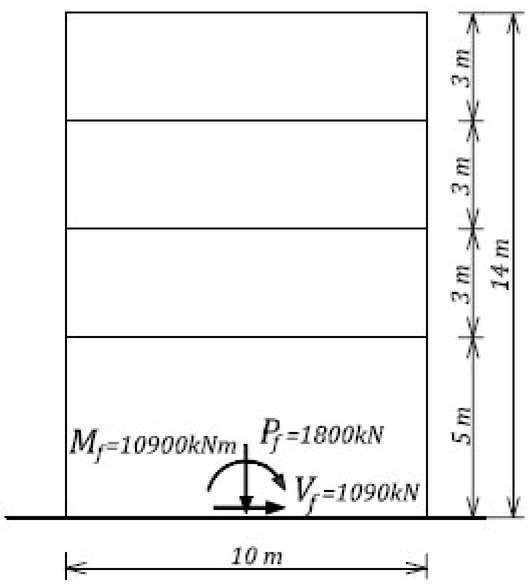
This example shows that an addition of flanges can be effective in preventing the out-of-plane buckling of Moderately Ductile squat shear walls. This is in compliance with S304-14 Cl.16.7.4, despite the fact that the h/t ratio for this wall is 33, which exceeds the S304-14-prescribed limit of 20.

The last two examples provide an opportunity for comparing the total amount of vertical reinforcement required for a squat shear wall of conventional construction (Example 4b) and a moderately ductile squat shear wall (this example). It is noted that the moderately ductile wall has less vertical reinforcement (11-15M bars) than a similar wall of conventional construction (16-15M bars); this reduction amounts to approximately 30%.

EXAMPLE 5a: Seismic design of a Moderately Ductile flexural (non-squat) shear wall
Perform the seismic design of a shear wall shown in the figure below. The wall is a part of a four-storey building located in Montreal, QC (City Hall) where the seismic hazard index, $I_E F_a S_a(0.2)$, is 0.60. The design needs to meet the requirements for Moderately Ductile Shear Wall SFRS according to NBC 2015.

The section at the base of the wall is subjected to a previously calculated total dead load of 1,800 kN (including the wall self-weight), an in-plane seismic shear force of 1090 kN, and an overturning moment of 10,900 kNm. The elastic lateral displacement at the top of the wall is 15 mm. Select the wall dimensions (length and thickness) and the reinforcement, such that the CSA S304-14 seismic design requirements for Moderately Ductile shear walls are satisfied. Due to architectural constraints, the wall length should not exceed 10 m, and 190 mm standard blocks should preferably be used.

Use hollow concrete blocks of 20 MPa unit strength and Type S mortar. Grade 400 steel reinforcement (yield strength f_y = 400 MPa) is used for this design.



SOLUTION:

1. Material properties and wall dimensions
Material properties for steel (both reinforcing bars and joint reinforcement):
ϕ_s = 0.85 f_y = 400 MPa
and masonry:
From S304-14 Table 4, for 20 MPa concrete blocks and Type S mortar:
f'_m = 10.0 MPa (assume solid grouted masonry)
ϕ_m = 0.6
Wall dimensions:
Overall height $h_w = 14$ m

Length $l_w = 10$ m

2. Load analysis

The section at the base of the wall needs to be designed for the following load effects:

- P_f = 1800 kN axial load
- V_f = 1090 kN seismic shear force
- M_f = 10900 kNm overturning moment

This is a Moderately Ductile shear wall, and NBC 2015 Table 4.1.8.9 specifies the following R_d and R_o values:
R_d = 2.0 and R_o = 1.5

3. Height/thickness ratio check (S304-14 Cl.16.8.3, see Section 2.6.4)

S304-14 prescribes the following height-to-thickness (h/t) limit for the compression zone in Moderately Ductile shear walls:
$h/(t+10) < 20$
For this example,
h = 5000 mm (the largest unsupported wall height)
So,
$t \geq h/20 - 10 = 240$ mm
This means that a rectangular wall section with 240 mm thickness could be used. However, S304-14 Cl.16.8.3 permits the use of a more slender wall if the wall is lightly loaded (axial stress less than $0.1f'_m$), and it can be proven that out-of-plane stability can be maintained under seismic effects.

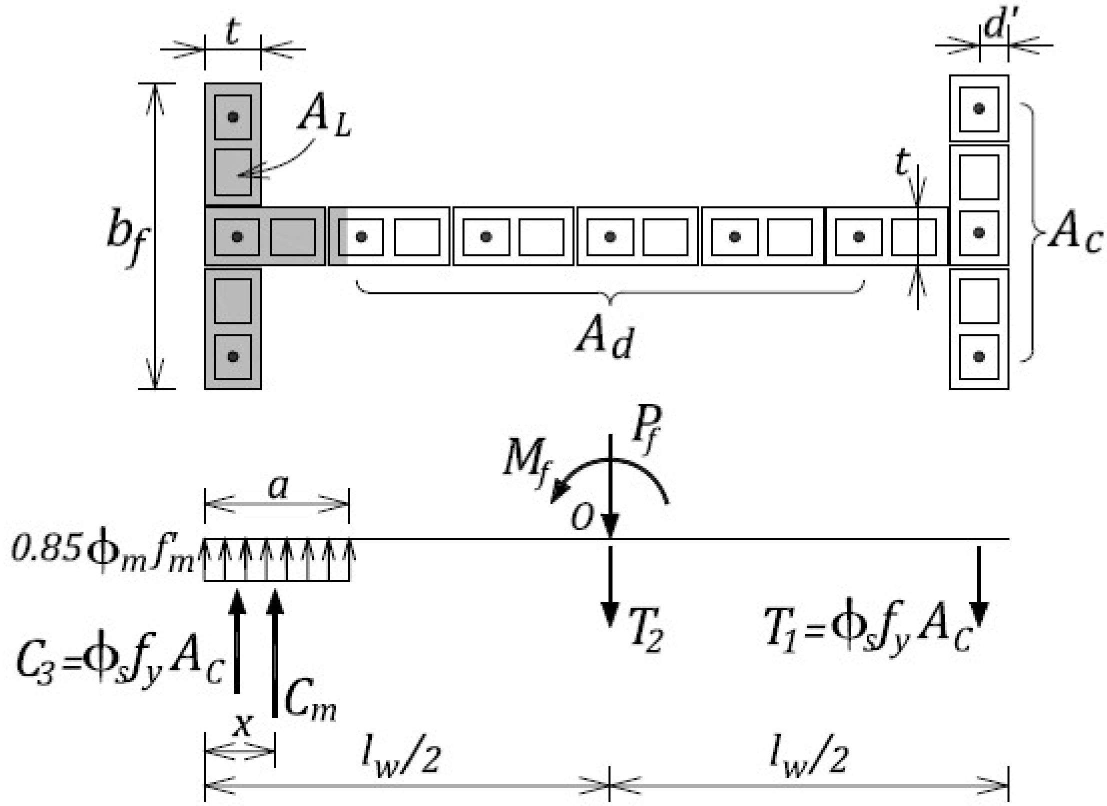
Let us consider $t = 190$ mm (standard concrete blocks) – this will result in $h/(t+10) = 25 > 20$.

In this case, the axial stress level is

$$\frac{P_f}{l_w * t * f'_m} = \frac{1800*10^3}{10000*190*10} = 0.095 < 0.1$$

The Commentary to Section 2.6.4 proposes an approach for verifying the out-of-plane stability of masonry shear walls with flanged ends. Let us assume a 1000 mm wide flange at each wall end, because S304-14 Cl.16.8.3.4 states that the minimum flange width of $0.2h$ (= 1000 mm for a 5m unsupported wall height at the first storey level) is required to ensure out-of-plane stability in ductile shear walls.

The effective flange width
$b_f = 1000$ mm
The wall section and the internal force distribution is shown in the figure below.



This procedure assumes that the concentrated reinforcement (area A_c) is provided at the wall ends (flanges), while the remaining reinforcement (area A_d) is distributed over the wall length. After a few trial estimates, the total area of vertical reinforcement A_{vt} was determined as follows

A_{vt} = 2800 mm^2

Concentrated reinforcement area (3-15M bars at each flange):

A_c = 600 mm^2

Distributed reinforcement area:

A_d =2800-2*600= 1600 mm^2

Distance from the wall end to the centroid of concentrated reinforcement A_c:

$d' = 95$ mm

- Check the buckling resistance of the compression zone.

The area of the compression zone A_L:

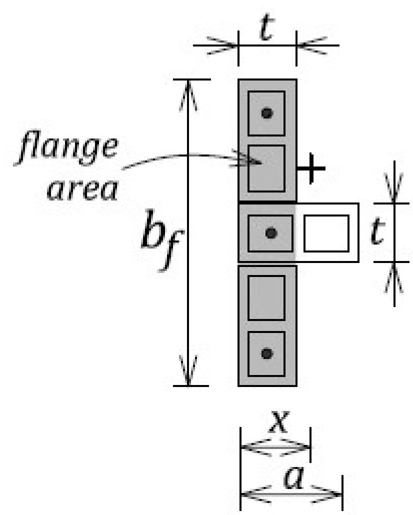
$$A_L = \frac{P_f + \phi_s f_y A_d}{0.85\phi_m f'_m} = \frac{1800*10^3 + 0.85*400*1600}{0.85*0.6*10.0} = 4.6*10^5 \text{ mm}^2$$

Check whether the neutral axis falls in the web. Since the flange area is

$A_f = b_f * t = 1.9*10^5$ mm^2

It is obvious that the area of compression zone is greater than the flange area, hence the neutral axis falls in the web. The depth of the compression zone a is:

$$a = \frac{A_L - b_f * t + t^2}{t} = \frac{4.6*10^5 - (1000*190) + 190^2}{190} = 1610$$
mm



The neutral axis depth:

$$c = \frac{a}{0.8} = 2011 \text{ mm}$$

The centroid of the masonry compression zone:

$$x = \frac{t*\left(a^2/2\right) + (b_f - t)\left(t^2/2\right)}{A_L} = 567 \text{ mm}$$

In this case, the compression zone is T-shaped, however only the flange area will be considered for the buckling resistance check (see the shaded area shown in the figure). This is a conservative approximation, and it is considered to be appropriate for this purpose, since the gross moment of inertia is used.

Gross moment of inertia for the flange only:

$$I_{xg} = \frac{t*b_f^{\ 3}}{12} = \frac{190*1000^3}{12} = 1.58*10^{10} \text{ mm}^4$$

The buckling strength for the compression zone will be determined according to S304-14 Cl. 10.7.4.3, as follows:

$$P_{cr} = \frac{\pi^2 \phi_{er} E_m I_{xg}}{\left(1+0.5\beta_d\right)\left(kh\right)^2} = 26566 \text{ kN}$$

where

$\phi_{er} = 0.75$

$k = 1.0$ pin-pin support conditions

$\beta_d = 0$ assume 100% seismic live load

$h = 5000$ mm unsupported wall height

$E_m = 850 f'_m = 8500$ MPa modulus of elasticity for masonry

- *Find the resultant compression force (including the concrete and steel component).*

$$P_{fb} = C_m + \phi_s f_y A_c = 2346*10^3 + 0.85*400*600 = 2550 \text{ kN}$$

where

$$C_m = \left(0.85\phi_m f'_m\right)A_L = (0.85*0.6*10.0)(4.6*10^5) = 2346 \text{ kN}$$

- Confirm that the out-of-plane buckling resistance is adequate.

Since

$P_{fb} = 2550$ kN < $P_{cr} = 26566$ kN

it can be concluded that the out-of-plane buckling resistance is adequate. The flanged section can be used for this design.

Note that S304-14 Cl.16.8.3.4 prescribes a relaxed (h/t *<30*) limit for flanged shear walls provided that the neutral axis depth meets the following simplified requirement (see Figure 2-28):

$c^* \leq 3t = 3*190 = 570$ mm

Note that $3t$ denotes the distance from the inside of a wall flange to the point of zero strain. So the total neutral axis depth (distance from the extreme compression fibre to the point of zero strain) is equal to

$c = c^* + t = 570 + 190 = 760$ mm

The neutral axis depth determined above is as follows

$c = 2011$ mm > 760 mm

It can be concluded that the S304-14 simplified (h/t) check performed above is not satisfied, and that a detailed verification is required (as presented above), to confirm the wall stability.

4. Design the flanged section for the combined axial load and flexure – consider distributed and concentrated wall reinforcement (see Section C.1.1.1).

The key design parameters for this calculation were determined in step 3 above. The factored moment resistance M_r will be determined by summing up the moments around the centroid of the wall section as follows

$$M_r = C_m(l_w/2 - x) + 2(\phi_s f_y A_c)(l_w/2 - d') = 2346*10^3*(10000/2 - 567) + 2*(0.85*400*600)*(10000/2 - 95)$$

$M_r = 12392$ kNm > $M_f = 10900$ kNm OK

5. Perform the S304-14 ductility check (see Section 2.6.3).

To satisfy the S304-14 ductility requirements for Moderately Ductile shear walls (Cl.16.8.7), the neutral axis depth ratio (c/l_w) should be less than the following limit:

$c/l_w \leq 0.15$ when $h_w/l_w \geq 5$

In this case,

$$\frac{h_w}{l_w} = 1.4 < 5$$

Also, the neutral axis depth

c = 2011 mm

and so

$c/l_w = 2011/10000 = 0.2 > 0.15$

Therefore, the simplified S304-14 ductility requirement is <u>not satisfied</u>. Consequently, a detailed ductility check according to S304-14 Cl.16.8.8 needs to be performed. It is required to determine the rotational demand θ_{id} and the rotational capacity θ_{ic}, and to confirm that the capacity exceeds the demand.

The rotational demand depends on the elastic lateral displacement at the top of the wall, which is given as

$\Delta_{f1} = 15$ mm

The overstrength factor must be at least equal to 1.3 and can be determined from the following equation:

$$\gamma_w = \frac{M_n}{M_f} = \frac{14034}{10900} = 1.29 < 1.3 \qquad \gamma_w = 1.3$$

In this case, the nominal moment capacity is equal to M_n = 14034 kNm, which was calculated in the same manner as the factored moment resistance M_r, except that unit values of material resistance factors $\phi_m = \phi_s = 1.0$ were used.

The S304-14 minimum rotational demand is θ_{min} = 0.003 for Moderately Ductile shear walls (Cl.16.8.8.2). The actual value is determined from the following equation:

$$\theta_{id} = \frac{\left(\Delta_{f1} R_o R_d - \Delta_{f1} \gamma_w\right)}{h_w - \dfrac{\ell_w}{2}} = \frac{\left(15 \cdot 2.0 \cdot 1.5 - 15 \cdot 1.30\right)}{\left(14.0 - \dfrac{10.0}{2}\right) \cdot 10^3} = 2.83 \cdot 10^{-3}$$

This is less than θ_{min} = 0.003, hence

$$\theta_{id} = \theta_{\min} = 3.0 \cdot 10^{-3}$$

The rotational capacity can be calculated as follows (and should not exceed 0.025)

$$\theta_{ic} = \left(\frac{\varepsilon_{mu} l_w}{2c} - 0.002\right) = \left(\frac{0.0025 \cdot 10000}{2 \cdot 2011} - 0.002\right) = 4.22 \cdot 10^{-3}$$

Since the rotational capacity θ_{iic} is greater than rotational demand θ_{id}, it can be concluded that the S304-14 ductility requirements have been satisfied.

6. Minimum required factored shear resistance (see Section 2.6.5 and S304-14 Cl.16.8.9.2)

Cl.16.8.9.2 requires that the factored shear resistance, V_r, for a Moderately Ductile shear wall should be greater than the shear due to the effects of factored loads, but not less than i) the shear corresponding to the development of the nominal moment capacity, M_n, or ii) shear corresponding to the lateral seismic load (base shear), where earthquake effects were calculated using $R_d R_o$=1.3.

The first requirement is based on the Capacity Design approach (see Section 2.5.1 for more details). For Moderately Ductile shear walls, the shear capacity should exceed the shear corresponding to the nominal moment capacity, as follows

$M_n = 14034$ kNm

The shear force resultant acts at the effective height h_e, the distance from the base of the wall to the resultant of all the seismic forces acting at the floor levels. Note that h_e can be determined as follows

$$h_e = \frac{M_f}{V_f} = 10.0 \text{ m}$$

The shear force V_{nb} corresponding to the overturning moment M_n is equal to

$$V_{nb} = \frac{M_n}{h_e} = \frac{14034}{10.0} = 1403 \text{ kN}$$

The second requirement gives an "almost elastic" factored base shear force for the wall, which is equal to

$$V_{fe} = \frac{V_f \cdot R_d \cdot R_o}{1.3} = \frac{1090 \cdot 2.0 \cdot 1.5}{1.3} = 2510 \text{ kN}$$

The smaller of these two values should be used, hence

$V_{rd} = 1403$ kN

7. The diagonal tension shear resistance (see Sections 2.3.2 and 2.6.5 and S304-14 Cl.10.10.2.1 and 16.8.9.1)

Masonry shear resistance (V_m):

$b_w = 190$ mm overall wall thickness

$d_v \approx 0.8 l_w = 8000$ mm   effective wall depth

$\gamma_g = 1.0$ solid grouted wall

Although the seismic hazard index $I_E F_a S_a(0.2) = 0.6 > 0.35$, partial grouting in the plastic hinge zone of Moderately Ductile shear walls is permitted by S304-14 Cl.16.8.5.2, because the wall has an aspect ratio $\frac{h_w}{l_w} = 1.4 < 2$, and is subjected to low axial stress (less than $0.1 f'_m$).

However, this design requires full grouting within the plastic hinge zone due to the significant shear demand.

$P_d = 0.9 P_f$ = 1620 kN

Since $\frac{M_f}{V_f d_v} = \frac{10900}{1090 * 8.0}$ = 1.25 > 1.0 use $\frac{M_f}{V_f d_v} = 1.0$ in the equation for masonry shear resistance below

$v_m = 0.16(2 - \frac{M_f}{V_f d_v})\sqrt{f'_m}$ = 0.51 MPa

$V_m = \phi_m (v_m b_w d_v + 0.25 P_d)\gamma_g$ = 0.6(0.51*190*8000+0.25*1620*10^3)*1.0 = 704 kN

To find the steel shear resistance V_s, assume 2-15M bond beam reinforcing bars at 600 mm spacing (this should provide some allowance in the shear strength to satisfy capacity design), thus

$A_v = 400$ mm^2

$s = 600$ mm

$V_s = 0.6 \phi_s A_v f_y \frac{d_v}{s} = 0.6 * 0.85 * \frac{400}{1000} * 400 * \frac{8000}{600}$ = 1088 kN

According to Cl.16.8.9.1, there is a 25% reduction in the masonry shear resistance contribution for Moderately Ductile shear walls, and so

$V_r = 0.75 V_m + V_s = 0.75 * 704 + 1088 = 1616$ kN > $V_{rd} = 1403$ kN   OK

Maximum shear allowed on the section is (S304-14 Cl.10.10.2.1)

$\max V_r = 0.4 \phi_m \sqrt{f'_m} b_w d_v \gamma_g = 1154$ kN < V_r

It can be concluded that the above maximum shear resistance requirement has not been satisfied. It would be required to increase either wall thickness or length to satisfy this requirement. It is recommended to perform this check at an early stage of the design.

8. Sliding shear resistance (see Sections 2.3.3 and 2.6.7 and S304-14 Cl.10.10.5.1)

The factored in-plane sliding shear resistance V_r is determined as follows:

μ = 1.0 for a masonry-to-masonry or masonry-to-roughened concrete sliding plane

A_s = 2800 mm^2 total area of vertical wall reinforcement

For Moderately Ductile shear walls, all vertical reinforcement should be accounted for in the T_y calculations (Cl.10.10.5.1), (also see Figure 2-17)

$T_y = \phi_s A_s f_y = 0.85 * 2800 * 400$ = 952 kN

P_d = 1620 kN

$C = P_d + T_y$ = 1620+952 = 2572 kN

$V_r = \phi_m \mu C$ = 0.6*1.0*2572 = 1543 kN

$V_r = 1543$ kN > $V_{rd} = 1403$ kN OK

9. Shear resistance at the web-to-flange interface (see Section C.2 and S304-14 Cl.7.11).

The factored shear stress at the web-to-flange interface is equal to the larger of the horizontal and vertical shear stress, as shown below.

Horizontal shear can be determined as follows:

$$v_f = \frac{V_{rd}}{t_e l_w} = \frac{1403 * 10^3}{190 * 10000} = 0.74 \text{ MPa}$$

where t_e = 190 mm (effective wall thickness)

Vertical shear over the entire wall height (caused by the resultant compression force P_{fb} calculated in Step 3):

$$v_f = \frac{P_{fb}}{b_w * h_w} = \frac{2550 * 10^3}{190 * 14000} = 0.96 \text{ MPa} \quad \text{governs}$$

Factored masonry shear strength for bonded interfaces (S304-14 Cl.7.11.1):

$v_m = 0.16 \phi_m \sqrt{f'_m} = 0.30$ MPa

Since

$v_f = 0.96$ MPa > $v_m = 0.30$ MPa

it is required to provide additional shear reinforcement at the web-to-flange interface. The horizontal reinforcement consists of 2-15M bars @ 600 mm spacing (bond beam reinforcement) and both bars can be extended into the flange (90° hook). These bars will provide shear resistance at the interface. Therefore,

$$v_s = \frac{\phi_s A_s f_y}{s \cdot t_e} = \frac{0.85 * 2 * 200 * 400}{600 * 190} = 1.19 \text{ MPa}$$

The total shear resistance

$v_r = v_m + v_s = 0.30 + 1.19 = 1.49$ MPa > $v_f = 0.96$ MPa OK

10. S304-14 seismic detailing requirements for Moderately Ductile shear walls – plastic hinge region

According to Cl.16.8.4, the required height of the plastic hinge region for Moderately Ductile shear walls must be greater than (see Table 2-5)

$h_p = l_2 / 2 = 5.0$ m

or

$h_p = h_w / 6 = 14.0 / 6 = 2.3$ m

(note that h_w denotes the total wall height)

So, $h_p = 5.0$ m governs

The reinforcement detailing requirements for the plastic hinge region of Moderately Ductile shear walls are as follows (see Table 2-4 and Figure 2-40):

1. ***The wall in the plastic hinge region must be solid grouted (Cl.16.6.2****) (the relaxation under Cl.16.8.5.2 does not apply in this case).*
2. ***Horizontal reinforcement requirements***

   a) Reinforcement spacing should not exceed the following limits (Cl.16.8.5.4):

   $s \le 1200$ mm or

   $s \le l_w/2 = 10000/2 = 5000$ mm

   Since the lesser value governs, the maximum permitted spacing is

   $s \le 1200$ mm

   According to the design, the horizontal reinforcement spacing is 600 mm, hence OK.

   b) Detailing requirements

   Horizontal reinforcement shall not be lapped within (Cl.16.8.5.4)

   600 mm or

   $l_w/5 = 2000$ mm

   whichever is greater, from the ends of the wall. In this case, the reinforcement should not be lapped within the distance 2000 mm from the end of the wall. The horizontal reinforcement can be lapped at the wall half-length. Lap splice lengths within the plastic hinge region are required to be at least $1.5l_d$ (Cl. 16.8.5.5).

   Horizontal reinforcement shall be (Cl.16.8.5.4):

   i) provided by reinforcing bars only (no joint reinforcement!);

   ii) continuous over the length of the wall (can be lapped in the centre), and

   iii) have at least 90° hooks at the ends of the wall.

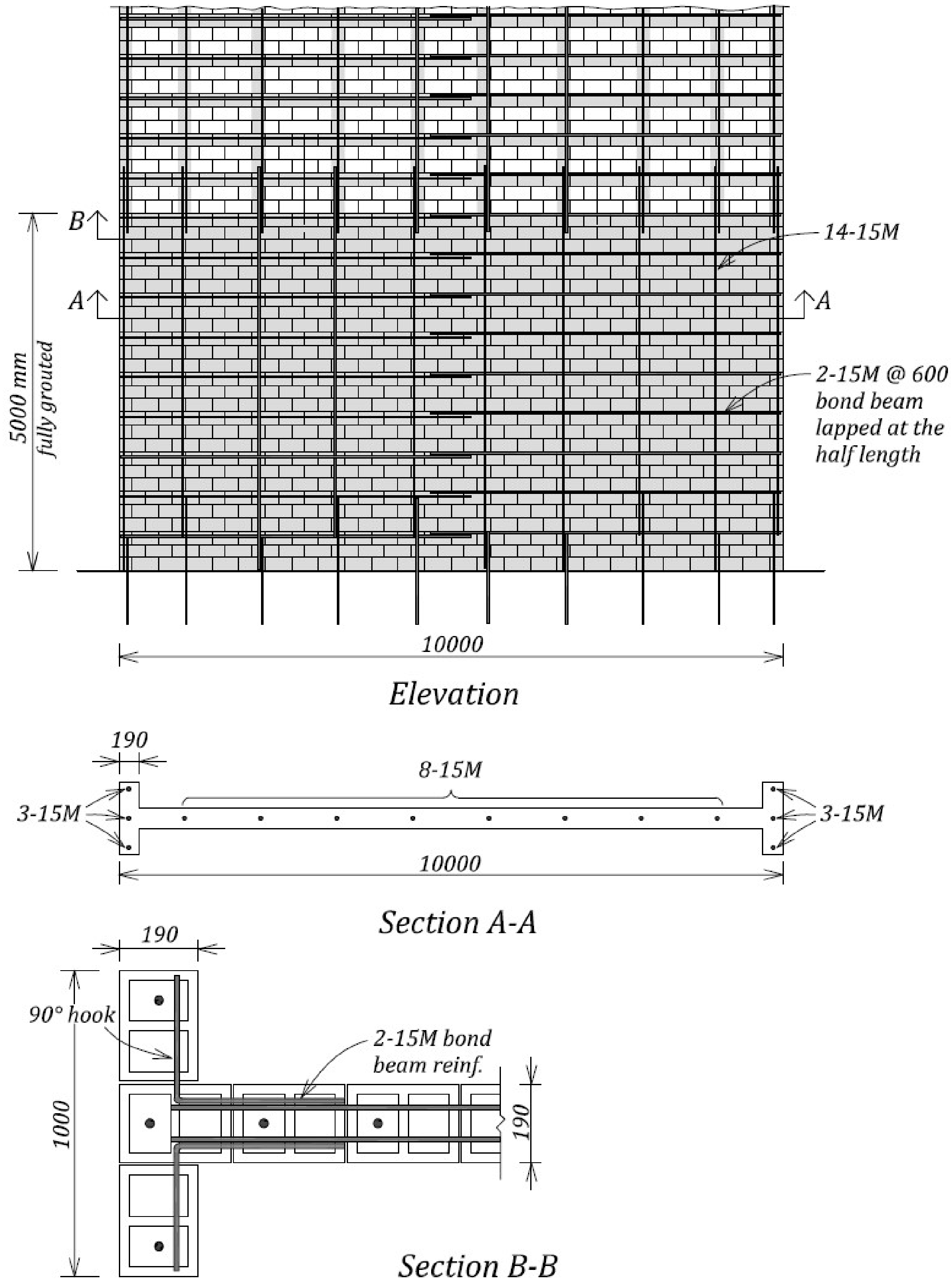
   All these requirements will be complied with, as shown on the design summary drawing.

3. Vertical reinforcement requirements (Cl.16.8.5.1)

Unlike Ductile shear walls there are no specific lapping restrictions for vertical reinforcement in the plastic hinge zone of Moderately Ductile shear walls. Lap splice lengths within the plastic hinge region are required to be at least $1.5l_d$ (Cl.16.8.5.5).

11. Design summary

The reinforcement arrangement for the wall under consideration is summarized in the figure below. Note that Moderately Ductile shear walls must be solid grouted in the plastic hinge region, except for certain specific cases. But they may be partially grouted outside the plastic hinge region (this depends on the design forces).



12. Discussion

It is important to consider all possible behaviour modes, and to identify the one that governs in this design. The following shear resistance values need to be considered:

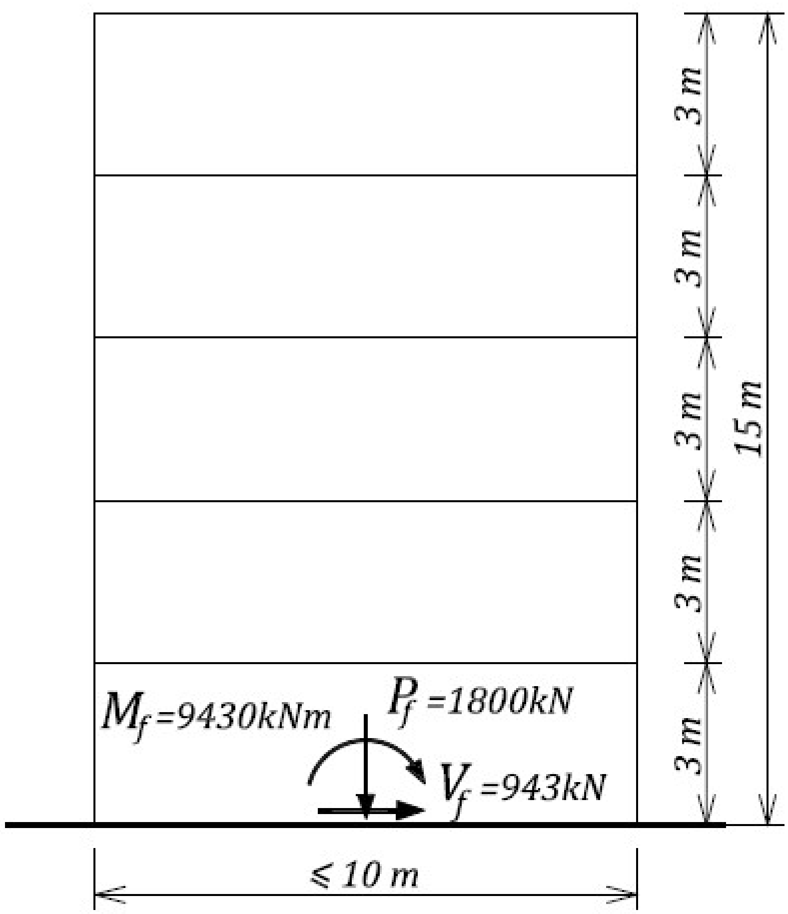
1. $V_r = 1616$ kN diagonal tension shear resistance
2. $V_r = 1543$ kN sliding shear resistance
3. $V_{rd} = 1403$ kN minimum required shear resistance to achieve ductile flexural behaviour

It can be concluded that the minimum required shear force corresponding to the flexural failure mechanism is the smallest, so the flexural failure mechanism governs in this case, which is a requirement for the Capacity Design approach for Moderately Ductile shear walls.

EXAMPLE 5b: Seismic design of a Ductile shear wall with a rectangular cross-section
Perform the seismic design of a shear wall shown in the figure below. The wall is five-stories high, with a total height of 15 m. It is part of a building located in Vancouver, BC (City Hall), where the seismic hazard index, $I_E F_a S_a(0.2)$, is 0.85. The design needs to meet the requirements for a Ductile Shear Wall SFRS according to NBC 2015.

The section at the base of the wall is subjected to a previously calculated total dead load of 1800 kN, an in-plane seismic shear force of 943 kN, and an overturning moment of 9430 kNm. The elastic lateral displacement at the top of the wall is 13 mm. Select the wall dimensions (length and thickness), and the reinforcement so that the CSA S304-14 seismic design requirements for Ductile shear walls are satisfied. Due to architectural constraints, the wall length should not exceed 10 m, and a standard rectangular wall section should be used.

Use hollow concrete blocks of 30 MPa unit strength and Type S mortar. Consider the wall as solid grouted. Grade 400 steel reinforcement (yield strength f_y = 400 MPa) is used for this design.



SOLUTION:

1. Material properties and wall dimensions

Material properties for steel (both reinforcing bars and joint reinforcement):

ϕ_s = 0.85 f_y = 400 MPa

and masonry:

From S304-14 Table 4, for 30 MPa concrete blocks and Type S mortar:

f'_m = 13.5 MPa (assume solid grouted masonry)

ϕ_m = 0.6

Wall dimensions:

Overall height $h_w = 15$ m

Wall length considered for initial calculations: $l_w = 10$ m

2. Load analysis

The section at the base of the wall needs to be designed for the following load effects:

- P_f = 1800 kN axial load
- V_f = 943 kN seismic shear force
- M_f = 9430 kNm overturning moment

For Ductile shear walls (NBC 2015 Table 4.1.8.9 – see Section 1.7) it is required that R_d = 3.0 and R_o = 1.5.

According to S304-14 Cl.16.9.2, the height/length aspect ratio for Ductile walls needs to be greater than 1.0. In this case,

$$\frac{h_w}{l_w} \geq \frac{15000}{10000} = 1.5 > 1.0 \quad \text{OK}$$

3. Determine the required wall thickness based on the S304-14 height-to-thickness requirements (Cl.16.9.3, see Section 2.6.4)

S304-14 prescribes the following height-to-thickness (h/t) limit for the compression zone in Ductile shear walls:

$h/(t+10) < 12$

For this example, h = 3000 mm (unsupported wall height)

So,

$t \geq h/12 - 10 = 240$ mm

Therefore, in this case the minimum acceptable wall thickness is

$t = 240$ mm

Note that it would be possible to use a smaller wall thickness (190 mm) if $c \leq 4b_w$ or $c \leq 0.3l_w$ (Cl.16.9.3.3 relaxing provision $h/(t+10) < 16$). The requirement $c \leq 4b_w = 4 \cdot 190 = 760$ mm would require a very small neutral axis depth which would be difficult to achieve in this case. Therefore a 240 mm wall thickness will be used in this design.

4. Determine the wall length based on the shear design requirements.
Designers may be requested to determine the wall dimensions (length and thickness) based on the design loads. In this case, the thickness is governed by the height-to-thickness ratio requirements, and the length can be determined from the maximum shear resistance for the wall section. The shear resistance for flexural walls cannot exceed the following limit (S304-14 Cl.10.10.2.1):

$V_r \le \max V_r = 0.4\phi_m \sqrt{f'_m} b_w d_v \gamma_g$

$\gamma_g = 1.0$ solid grouted wall (required for plastic hinge zone)

$b_w = 240$ mm overall wall thickness

$d_v \approx 0.8 l_w = 8000$ mm effective wall depth

Set

$V_r = V_f = 943$ kN

and so

$$l_w > \frac{V_f}{0.4\phi_m \sqrt{f'_m} b_w (0.8)\gamma_g} = \frac{943 * 10^3}{0.4 * 0.6 * \sqrt{13.5} * 240 * 0.8 * 1.0} = 5570 \text{ mm}$$

Therefore, based on the shear design requirements the designer could select a wall length of 5.7 m. However, a preliminary capacity design check indicated that a minimum wall length of nearly 10 m was required, thus try

$l_w = 10000$ mm

which gives

$\max V_r = 1690$ kN

5. Minimum S304-14 seismic reinforcement requirements (see Table 2-3). Since $I_E F_a S_a(0.2)$ = 0.85 > 0.35, it is required to provide minimum seismic reinforcement (S304-14 Cl.16.4.5). See Example 4a for a detailed discussion on the S304-14 minimum seismic reinforcement requirements.

6. Design the wall for the combined effect of axial load and flexure (see Section C.1.1.2).
Design for the combined effects of axial load and flexure by assuming uniformly distributed vertical reinforcement over the wall length.

The amount of vertical reinforcement can be estimated from the ductility requirements for Ductile shear walls (S304-14 Cl.16.8.8). The goal for the S304-14 detailed ductility check is to confirm that the rotational capacity exceeds the rotational demand in the plastic hinge zone. Based on the minimum rotational demand requirements (θ_{min} = 0.004), the c/l_w ratio should not exceed 0.2 for Ductile Shear Walls (see Section 2.6.3). An approach for estimating the maximum amount of vertical reinforcement required for predefined c/l_w ratio for walls with distributed reinforcement is presented in Section 2.6.3, and its application will be illustrated next.

The main input parameter is the level of axial compression stress relative to compressive strength f'_m, that is,

$$\frac{f}{f'_m} = \frac{P_f}{f'_m l_w t} = \frac{1800 \cdot 10^3}{13.5 \cdot 10000 \cdot 240} = 0.055$$

From Fig. 2-27 (see below), for the given axial stress level of 0.055 (vertical axis), and assuming c/l_w =0.2 (horizontal axis) it is possible to determine the corresponding ω value;

$$\omega = 0.06$$

The required vertical reinforcement ratio can be determined from ω as follows:

$$\rho_v = \frac{\omega \phi_m f'_m}{\phi_s f_y} = \frac{0.06 \cdot 0.6 \cdot 13.5}{0.85 \cdot 400} = 0.00143$$

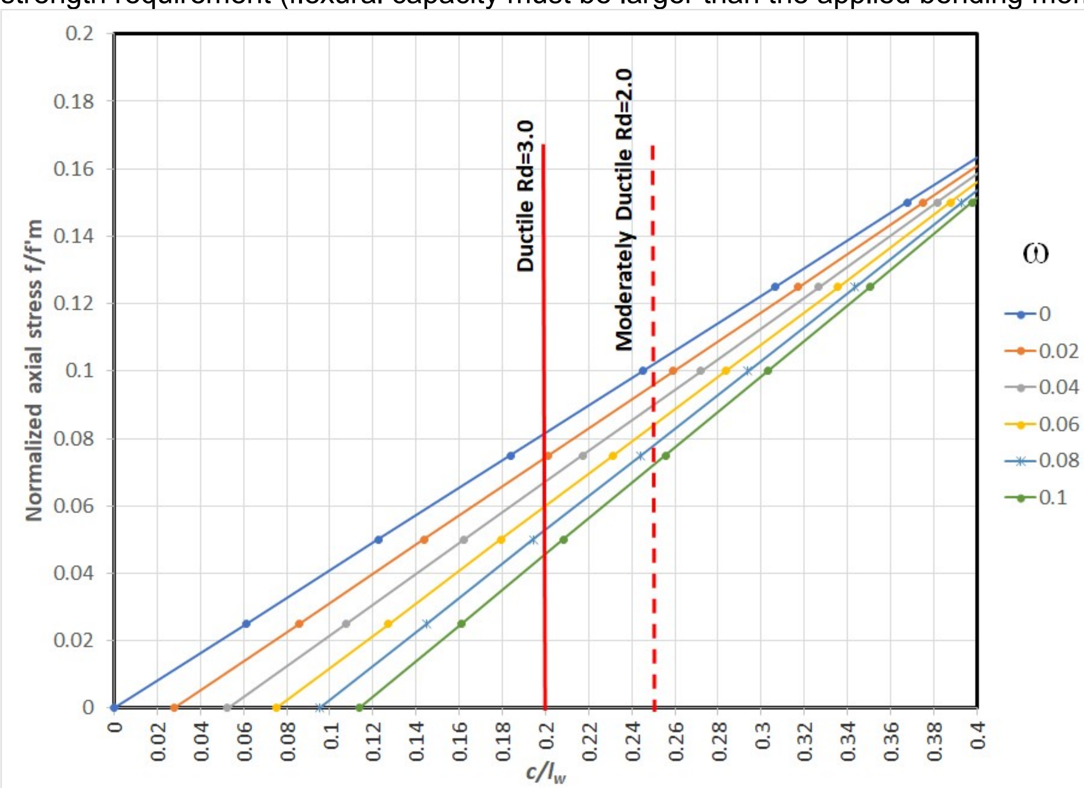
Since the vertical reinforcement ratio is equal to

$$\rho_v = \frac{A_{vt}}{t * l_w}$$

The maximum required area of vertical reinforcement can be determined as follows

$$A_{vt} = \rho_v \cdot t \cdot l_w = 0.00143 \cdot 240 \cdot 10000 = 3432 \text{ mm}^2$$

Since this is the maximum amount from the ductility perspective, the goal is to select an amount of reinforcement less than the maximum and confirm that the amount is sufficient to satisfy the strength requirement (flexural capacity must be larger than the applied bending moment).



The proposed area of vertical reinforcement is as follows:

A_{vt} = 2800 mm^2

In total, 14 vertical reinforcing bars are used in this design: 4-15 M reinforcing bars as concentrated reinforcement (2-15M bars at each end) plus 10-15M bars as distributed reinforcement, and the average spacing is equal to

$$s \leq \frac{10000 - 200}{13} = 753 \quad \text{mm}$$

Since 2-15M bars are concentrated at each end, the amount of concentrated reinforcement is
A_c = 400 mm^2

And the amount of distributed reinforcement is
$A_d = A_{vt} - 2A_c =$ 2000 mm^2

For Ductile shear walls, S304-14 Cl.16.9.5.3 notes that the amount of concentrated reinforcement at each wall end should not exceed 25% of the distributed reinforcement. Since
$A_c / A_d = 400/2000 = 0.2$ < 0.25 OK

It is also required to check the maximum reinforcement area per S304-14 Cl.10.15.2 (see Table 2-3).
Since $s = 753mm$ < $4t = 4*240 = 960$ mm
$A_{s\max} = 0.02 A_g = 0.02(240*10^3) = 4800$ mm^2/m

This is significantly larger than the estimated area of vertical reinforcement.

The wall is subjected to axial load P_f = 1800 kN. The moment resistance for the wall section can be determined from the following equations (see Section C.1.1.2):
$\alpha_1 = 0.85 \quad \beta_1 = 0.8 \quad \omega = 0.05 \quad \alpha = 0.09 \quad c \approx 1820$ mm

$$M_r = 0.5\phi_s f_y A_{vt} l_w \left(1 + \frac{P_f}{\phi_s f_y A_{vt}}\right)\left(1 - \frac{c}{l_w}\right) = 0.5*0.85*\frac{400}{1000}*2800*\frac{10000}{1000}\left(1 + \frac{1800*10^3}{0.85*400*2800}\right)\left(1 - \frac{1820}{10000}\right)$$

$M_r = 11300$ kNm > $M_f = 9430$ kNm OK

Note that
$c/l_w = 1820/10000 = 0.18 < 0.2$

Therefore, the S304-14 minimum rotational demand requirement for Ductile shear walls is satisfied.

7. Perform the S304-14 ductility check (see Section 2.6.3).

To satisfy the S304-14 ductility requirements for Ductile shear walls (Cl.16.9.7), the neutral axis depth ratio (c/l_w), should be less than the following limit:
$c/l_w \le 0.125$ when $h_w / l_w \ge 5$

In this case,
$\frac{h_w}{l_w} = 1.5 < 5$ Also, the neutral axis depth
c = 1820 mm
and so
$c/l_w = 1820/10000 = 0.18 > 0.125$

Therefore, the simplified S304-14 ductility requirement is not satisfied. Consequently, a detailed ductility check according to S304-14 Cl.16.8.8 needs to be performed. It is required to determine the rotational demand θ_{id} and the rotational capacity θ_{ic}, and to confirm that the capacity exceeds the demand.

The rotational demand depends on the elastic lateral displacement at the top of the wall, which is given as
$\Delta_{f1} = 13$ mm

The overstrength factor must be at least equal to 1.3, and can be determined from the following equation:

$$\gamma_w = \frac{M_n}{M_f} = \frac{12800}{9430} = 1.36$$

In this case, the nominal moment capacity is equal to M_n = 12,800 kNm, which was calculated in the same manner as the factored moment resistance M_r, except that unit values of material resistance factors $\phi_m = \phi_s = 1.0$ were used.

The S304-14 minimum rotational demand is θ_{min} = 0.004 for Ductile shear walls. The actual value is determined from the following equation:

$$\theta_{id} = \frac{\left(\Delta_{f1} R_o R_d - \Delta_{f1}\gamma_w\right)}{h_w - \frac{\ell_w}{2}} = \frac{\left(13 \cdot 3.0 \cdot 1.5 - 13 \cdot 1.36\right)}{\left(15.0 - \frac{10.0}{2}\right) \cdot 10^3} = 4.08 \cdot 10^{-3}$$

This is greater than θ_{min} = 0.004, so the actual rotational demand will be used.
The rotational capacity can be calculated as follows (and should not exceed 0.025)

$$\theta_{ic} = \left(\frac{\varepsilon_{mu} l_w}{2c} - 0.002\right) = \left(\frac{0.0025 \cdot 10000}{2 \cdot 1820} - 0.002\right) = 4.87 \cdot 10^{-3}$$

Since the rotational capacity θ_{ic} is greater than rotational demand θ_{id}, it can be concluded that the S304-14 ductility requirements have been satisfied.

8. Minimum required factored shear resistance (see Section 2.6.5 and S304-14 Cl.16.9.8.3)

Cl.16.9.8.3 requires that the factored shear resistance, V_r, should be greater than the shear due to effects of factored loads, but not less than i) the shear corresponding to the development of probable moment capacity, M_p, or ii) the shear corresponding to the lateral seismic load (base shear), where earthquake effects were calculated using $R_d R_o$=1.3.

The first requirement is based on the Capacity Design approach (see Section 2.5.1 for more details). For Ductile shear walls, the shear capacity should exceed the shear corresponding to the probable moment capacity, as follows

$M_p = 13900$ kNm

The shear force resultant acts at the effective height h_e, that is, the distance from the base of the wall to the resultant of all seismic forces acting at the floor levels. Note that h_e can be determined as follows

$$h_e = \frac{M_f}{V_f} = 10.0 \text{ m}$$

The shear force V_{pb} corresponding to the overturning moment M_p is equal to

$$V_{pb} = \frac{M_p}{h_e} = \frac{13900}{10.0} = 1390 \text{ kN}$$

The second requirement gives an "almost elastic" factored base shear force for the wall, which is equal to

$$V_{fe} = \frac{V_f \cdot R_d \cdot R_o}{1.3} = \frac{943 \cdot 3.0 \cdot 1.5}{1.3} = 3264 \text{ kN}$$

The smaller of these two values should be used, hence

$V_{rd} = 1390$ kN

9. Diagonal tension shear resistance (see Sections 2.3.2 and 2.6.5 and S304-14 Cl.10.10.2.1 and Cl.16.9.8.1)

Masonry shear resistance (V_m):

$b_w = 240$ mm overall wall thickness

$d_v \approx 0.8 l_w = 8000$ mm effective wall depth

$\gamma_g = 1.0$ solid grouted wall

$P_d = 0.9 P_f$ = 1620 kN

Since

$\frac{M_f}{V_f d_v} = \frac{9430}{943 * 8.0}$ = 1.25 > 1.0 use $\frac{M_f}{V_f d_v} = 1.0$ in the equation for masonry shear resistance below

$v_m = 0.16(2 - \frac{M_f}{V_f d_v})\sqrt{f'_m}$ = 0.59 MPa

$V_m = \phi_m (v_m b_w d_v + 0.25 P_d)\gamma_g$ = 0.6(0.59*240*8000+0.25*1620*10^3)*1.0 = 920 kN

The required steel shear resistance can be found from the following equation (see Section 2.6.5 and S304-14 Cl.16.9.8.1) (note 50% reduction of V_m)

$V_r = 0.5 V_m + V_s \geq V_{rd}$

hence

$V_s = V_{rd} - 0.5 V_m = 1390 - 0.5 \cdot 920 = 930$ kN

The required amount of reinforcement can be found from the following equation

$$\frac{A_v}{s} = \frac{V_s}{0.6 \phi_s f_y d_v} = \frac{930 * 10^3}{0.6 * 0.85 * 400 * 8000} = 0.57$$

Try 2-15M bond beam reinforcing bars at 600 mm spacing ($A_v = 400$ mm^2 and $s = 600$ mm):

$\frac{A_v}{s} = \frac{400}{600} = 0.67$ > 0.57 OK

Steel shear resistance V_s:

$V_s = 0.6 \phi_s A_v f_y \frac{d_v}{s} = 0.6 * 0.85 * \frac{400}{1000} * 400 * \frac{8000}{600}$ = 1088 kN

Total diagonal shear resistance:

$V_r = 0.5 V_m + V_s = 0.5 \cdot 920 + 1088 = 1548$ kN > $V_{rd} = 1390$ kN OK

Maximum shear allowed on the section is (S304-14 Cl.10.10.2.1)

$\max V_r = 0.4 \phi_m \sqrt{f'_m} b_w d_v \gamma_g = 1690$ kN

Since
$V_r = 1548$ kN < $\max V_r = 1690$ kN   OK
In conclusion, the diagonal shear design requirement has been satisfied.

10. Sliding shear resistance (see Sections 2.3.3 and 2.6.7 and S304-14 Cl.10.10.5.1 and 16.9.8.2)

The factored in-plane sliding shear resistance V_r is determined as follows:
μ = 1.0 for a masonry-to-masonry or masonry-to-roughened concrete sliding plane
A_s = 2800 mm^2 total area of vertical wall reinforcement
For Ductile shear walls, only the vertical reinforcement in the tension zone should be accounted for in the T_y calculations (S304-14 Cl.16.9.8.2), and so (see Figure 2-17b)

$$T_y = \phi_s A_s f_y \left(\frac{l_w - c}{l_w} \right) = 0.85 * 2800 * 400 * \left(\frac{10000 - 1820}{10000} \right) = 779 \text{ kN}$$

P_d = 1620 kN
$C = P_d + T_y$ = 1620+779 = 2399 kN
$V_r = \phi_m \mu C$ = 0.6*1.0*2399 = 1440 kN
$V_r = 1440$ kN > $V_{rd} = 1390$ kN   OK

11. S304-14 seismic detailing requirements for Ductile shear walls – plastic hinge region

According to Cl.16.9.4, the required height of the plastic hinge region for Ductile shear walls is (see Table 2-5)
$h_p = 0.5 l_w + 0.1 h_w = 0.5 \cdot 10000 + 0.1 \cdot 15000 = 6500$ mm
However
$0.8 l_w \le h_p \le 1.5 l_w$
Since
$0.8 l_w = 8000$ mm > 6500 mm
It follows that
$h_p = 0.8 l_w = 8.0$ m governs.

The reinforcement detailing requirements for the plastic hinge region of Ductile shear walls are as follows (see Table 2-4 and Figure 2-41):

1. *The wall in the plastic hinge region must be solid grouted (Cl.16.6.2).*
2. *Horizontal reinforcement requirements:*
   a) Reinforcement spacing should not exceed the following limits (Cl.16.9.5.4):
      $s \le 600$ mm or
      $s \le l_w/2 = 10000/2 = 5000$ mm
      Since the lesser value governs, the maximum permitted spacing is
      $s \le 600$ mm
      According to the design, the horizontal reinforcement spacing is 600 mm, hence OK.
   b) Detailing requirements
      Horizontal reinforcement shall not be lapped within (Cl.16.9.5.4)
      600 mm or

$l_w/5$ = 2000 mm

whichever is greater, from the end of the wall. In this case, the reinforcement should not be lapped within 2000 mm from the end of the wall. The horizontal reinforcement can be lapped at the wall half-length.

Horizontal reinforcement shall be (Cl.16.9.5.4):
i) provided by reinforcing bars only (no joint reinforcement!);
ii) continuous over the length of the wall (can be lapped in the centre), and
iii) have 180° hooks around the vertical reinforcing bars at the ends of the wall.

3. Vertical reinforcement requirements:

a) Reinforcement spacing should not exceed the following limits (Cl.16.9.5.3):

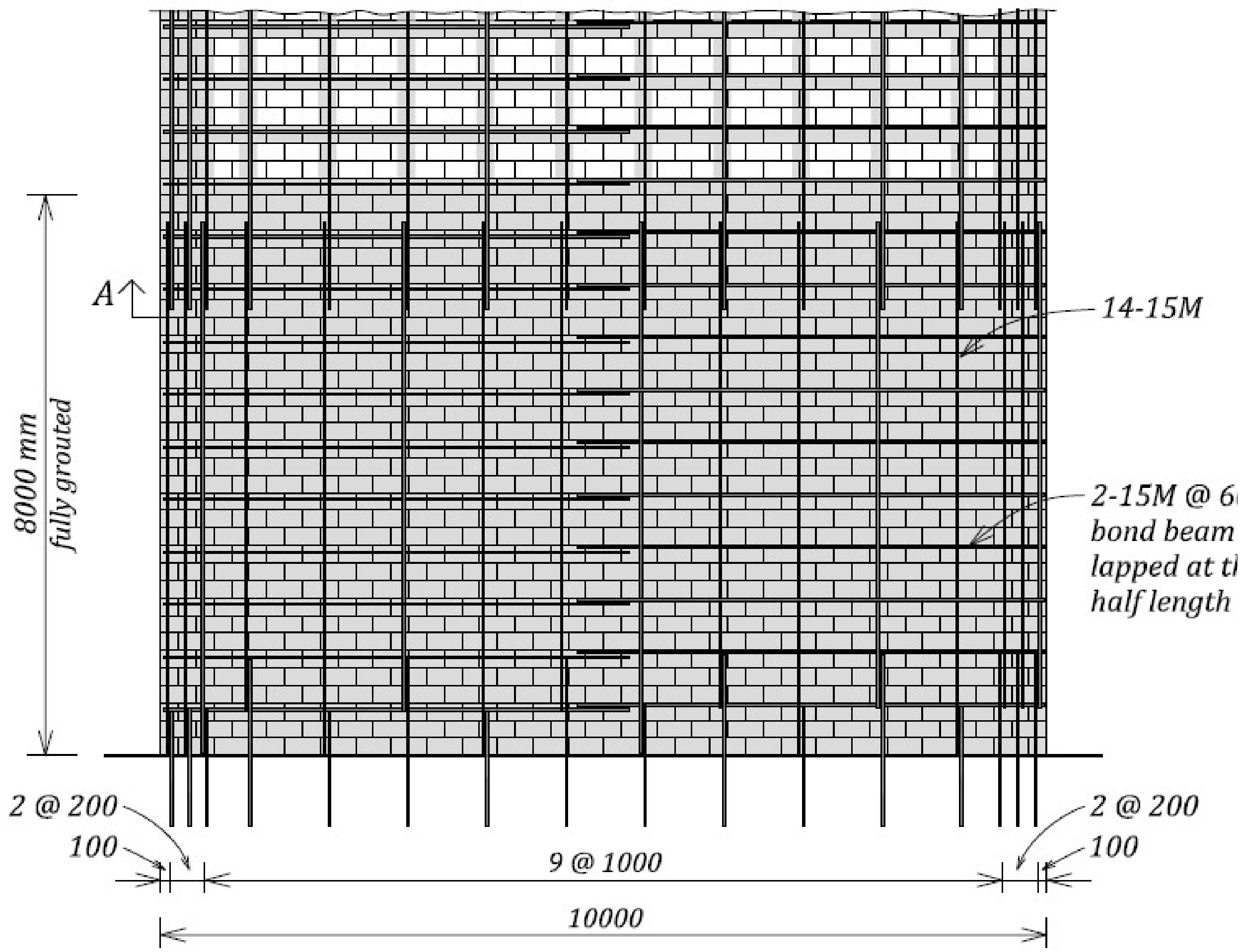
$s \leq l_w/4 = 10000/4 = 2500$ mm, but need not be less than 400 mm, or the minimum seismic requirements specified in Cl.16.4.5.3, which states that $s \leq 1200$ mm (this value governs since the wall thickness is 240 mm). Since the lesser value governs, the maximum permitted spacing is $s \leq 1200$ mm.

b) Detailing requirements

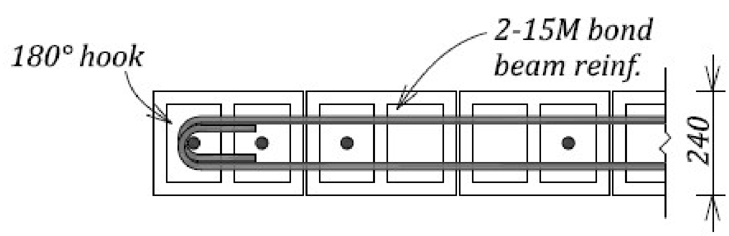
At any section within the plastic hinge region, no more than half of the area of vertical reinforcement may be lapped (Cl.16.9.5.2).

12. Design summary

The reinforcement arrangement for the wall under consideration is summarized in the figure below. Note that a Ductile shear wall must be solid grouted in plastic hinge region, but it may be partially grouted outside the plastic hinge region (depending on the design forces).



Elevation



Section A-A

13. Discussion

It is important to consider all possible behaviour modes and identify the one that governs in this design. The following shear resistance values need to be considered:

4. $V_r = 1548$ kN diagonal tension shear resistance
5. $V_r = 1440$ kN sliding shear resistance
6. $V_{rd} = 1390$ kN minimum required shear resistance to achieve ductile flexural behaviour

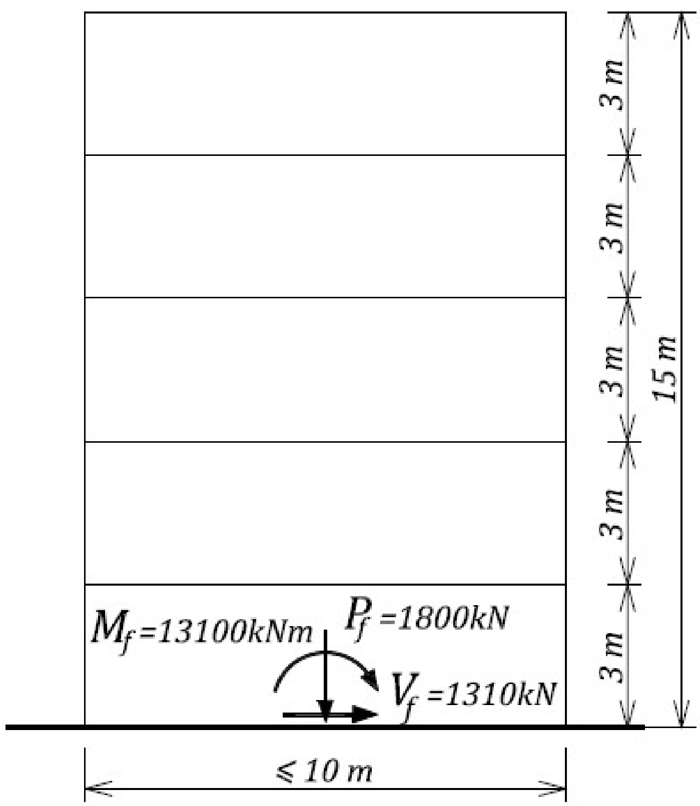
It can be concluded that the minimum required shear force corresponding to the flexural failure mechanism is the smallest (1390 kN), so it governs in this case, which is a requirement for the Capacity Design approach for Ductile RM shear walls.

EXAMPLE 5c: Seismic design of a Ductile shear wall with Boundary Elements

Perform the seismic design of the same shear wall designed in Example 5b. The building is located in Victoria, BC where the seismic hazard index, $I_E F_a S_a(0.2)$, is 1.3. The design needs to meet the requirements for a Ductile Shear Wall SFRS according to NBC 2015.

The section at the base of the wall is subjected to a previously calculated total dead load of 1800 kN, an in-plane seismic shear force of 1310 kN, and an overturning moment of 13100 kNm. The elastic lateral displacement at the top of the wall is 18 mm. Select the wall dimensions (length and thickness) and the reinforcement, so that the CSA S304-14 seismic design requirements for Ductile shear walls are satisfied. Due to architectural constraints, the wall length should not exceed 10 m. The wall may have standard rectangular section, or alternatively, boundary elements may be provided at wall ends if required by design.

Use hollow concrete blocks of 30 MPa unit strength and Type S mortar. Consider the wall as solid grouted. Grade 400 steel reinforcement (yield strength f_y = 400 MPa) is used for this design.



SOLUTION:

As the first attempt, the wall will be designed with a rectangular cross-section, and boundary elements will be provided only if a rectangular section cannot be used.

1. Material properties and wall dimensions

Material properties for steel (both reinforcing bars and joint reinforcement):

ϕ_s = 0.85 f_y = 400 MPa

and masonry:

From S304-14 Table 4, for 30 MPa concrete blocks and Type S mortar:

f'_m = 13.5 MPa (assume solid grouted masonry)

ϕ_m = 0.6

Wall dimensions:

Overall height $h_w = 15$ m

Wall length considered for initial calculations: $l_w = 10$ m

2. Load analysis

The section at the base of the wall needs to be designed for the following load effects:

- P_f = 1800 kN axial load
- V_f = 1310 kN seismic shear force
- M_f = 13100 kNm overturning moment

For Ductile shear walls (NBC 2015 Table 4.1.8.9 – see Section 1.7), it is required that R_d = 3.0 and R_o = 1.5.

According to S304-14 Cl.16.9.2, the height/length aspect ratio for Ductile walls needs to be greater than 1.0. In this case,

$$\frac{h_w}{l_w} \geq \frac{15000}{10000} = 1.5 > 1.0 \quad \text{OK}$$

3. Determine the required wall thickness based on the S304-14 height-to-thickness requirements (Cl.16.9.3, see Section 2.6.4)

S304-14 prescribes the following height-to-thickness (h/t) limit for the compression zone in Ductile shear walls:

$h/(t+10) < 12$

For this example,

h = 3000 mm (unsupported wall height)

So,

$t \geq h/12 - 10 = 240$ mm

Therefore, in this case the minimum acceptable wall thickness is

$t = 240$ mm

4. Minimum S304-14 seismic reinforcement requirements (see Table 2-2)

Since $I_E F_a S_a(0.2)$ = 1.3 > 0.35, it is required to provide minimum seismic reinforcement (S304-14 Cl.16.4.5). See Example 4a for a detailed discussion on the S304-14 minimum seismic reinforcement requirements.

5. Design the wall for the combined effect of axial load and flexure (see Section C.1.1.2).

The total area of vertical reinforcement has been estimated as follows:
A_{vt} = 6000 mm^2

The wall is subjected to axial load P_f = 1800 kN. The moment resistance for the wall section can be determined from the following equations (see Section C.1.1.2):
$\alpha_1 = 0.85 \quad \beta_1 = 0.8 \quad \omega = 0.09 \quad \alpha = 0.08 \quad c \approx 1910$ mm

$$M_r = 0.5\phi_s f_y A_{vt} l_w \left(1 + \frac{P_f}{\phi_s f_y A_{vt}}\right)\left(1 - \frac{c}{l_w}\right) = 0.5 * 0.85 * \frac{400}{1000} * 6000 * \frac{10000}{1000}\left(1 + \frac{1800*10^3}{0.85*400*6000}\right)\left(1 - \frac{1910}{10000}\right)$$

$M_r = 15500$ kNm > $M_f = 13100$ kNm OK

6. Perform the S304-14 ductility check (see Section 2.6.3).
To satisfy the S304-14 ductility requirements for Ductile shear walls (Cl.16.9.7), the neutral axis depth ratio (c/l_w) should be less than the following limit:
$c/l_w \le 0.125$ when $h_w/l_w \ge 5$
In this case,

$$\frac{h_w}{l_w} = 1.5 < 5$$

Also, the neutral axis depth
c = 1910 mm
and so
$c/l_w = 1910/10000 = 0.19 > 0.125$
Therefore, the simplified S304-14 ductility requirement is not satisfied. Consequently, a detailed ductility check according to S304-14 Cl.16.8.8 needs to be performed. It is required to determine the rotational demand θ_{id} and the rotational capacity θ_{ic}, and to confirm that the capacity exceeds the demand.

The rotational demand depends on the elastic lateral displacement at the top of the wall, which is given as
$\Delta_{f1} = 18$ mm
The overstrength factor must be at least equal to 1.3 and can be determined from the following equation:

$$\gamma_w = \frac{M_n}{M_f} = \frac{18200}{13100} = 1.39$$

In this case, the nominal moment capacity is equal to M_n = 18,200 kNm, which was calculated in the same manner as the factored moment resistance M_r, except that unit values of material resistance factors $\phi_m = \phi_s = 1.0$ were used.

Based on the S304-14 rotational demand requirement, the minimum rotational demand θ_{min} = 0.004 for Ductile shear walls. The actual value is determined from the following equation:

$$\theta_{id} = \frac{\left(\Delta_{f1} R_o R_d - \Delta_{f1}\gamma_w\right)}{h_w - \frac{\ell_w}{2}} = \frac{\left(18\cdot 3.0\cdot 1.5 - 18\cdot 1.39\right)}{\left(15.0 - \frac{10.0}{2}\right)\cdot 10^3} = 5.60\cdot 10^{-3}$$

This is greater than θ_{min} = 0.004, so the actual rotational demand will be used.
The rotational capacity can be calculated as follows (and should not exceed 0.025)

$$\theta_{ic} = \left(\frac{\varepsilon_{mu} l_w}{2c} - 0.002\right) = \left(\frac{0.0025\cdot 10000}{2\cdot 1910} - 0.002\right) = 4.53\cdot 10^{-3}$$

Since the rotational capacity is less than the rotational demand, it can be concluded that the S304-14 ductility requirements have not been satisfied. The design will be continued by providing boundary elements at wall ends, and following the pertinent S304-14 provisions for Ductile shear walls with increased compressive strain beyond the 0.0025 limit (S304-14 Cl.16.10). It is proposed that an overall wall length of 9 m be used, which is less than the maximum length (10 m) per design requirements.

7. Determine the minimum required thickness for the boundary elements and the wall based on the S304-14 height-to-thickness requirements (Cl.16.9.3, see Section 2.6.8.3)

S304-14 prescribes the following height-to-thickness (h/t) limit for the compression zone in Ductile shear walls with boundary elements (for the zone between the compression face to one-half of the compression zone depth, see Figure 2-35):

$h/(t+10) < 12$

For this example,

h = 3000 mm (unsupported wall height)

So

$t \geq h/12 - 10 = 240$ mm

Therefore, in this case the minimum acceptable wall thickness of the boundary element is 240 mm, however a larger size will be selected since larger number of vertical reinforcing bars need to be provided, that is,

$t_b = 390$ mm

The maximum required thickness of the wall web is

$t \geq h/16 - 10 = 178$ mm

Therefore, a 190 mm wall thickness could be used for this design based on the height/thickness requirements, however a larger thickness is required to meet the shear resistance requirements, therefore

$t = 240$ mm

will be used in this design.

8. Design the wall for the combined effect of axial load and flexure (see Section C.1.1.1).

The proposed wall length $l_w = 9000$ mm is less than the maximum permitted value (10000 mm).

The proposed dimensions of boundary elements are:

$l_b = 790$ mm length

$t_b = 390$ mm thickness

These dimensions will be verified at a later stage.

The design procedure assumes that the concentrated reinforcement (area A_c) is provided at each boundary element, while the remaining reinforcement (area A_d) is distributed over the wall web. After a few trial estimates, the total area of vertical reinforcement A_{vt} was determined as follows

A_{vt} = 5200 mm^2

Concentrated reinforcement in the boundary elements (8-15M bars at each boundary element):

A_c = 1600 mm^2

Check if this amount is sufficient based on S304-14 Cl.16.11.8:

$A_c \geq 0.00075 * t * l_w = 0.00075 * 240 * 9000 = 1620$ mm^2

The proposed area is slightly less than the required area, but the difference is insignificant.

Distributed reinforcement in the wall:

A_d = 5200-2*1600 = 2000 mm^2

Distance from the wall end to the centroid of concentrated reinforcement A_c:

$d' = l_b/2 = 395$ mm

The area of the compression zone A_L:

$$A_L = \frac{P_f + \phi_s f_y A_d}{0.85\phi_m f'_m} = \frac{1800*10^3 + 0.85*400*2000}{0.85*0.6*13.5} = 3.6*10^5 \text{ mm}^2$$

If the area of the compression zone exceeds the area of boundary element, it follows that the neutral axis falls in the wall web (as opposed to the boundary element). In this case the area of boundary element is

$A_g = t_b * l_b = 390*790 = 3.08*10^5$ mm^2

Since

$A_L > A_g$

it follows that the neutral axis falls in the web. The compression zone depth a can be determined from the following equation:

$$a = \frac{A_L - b_f * l_f}{t} + l_f = \frac{3.6*10^5 - 390*790}{240} + 790 = 1010 \text{ mm}$$

The neutral axis depth is

$$c = \frac{a}{0.8} = 1259 \text{ mm}$$

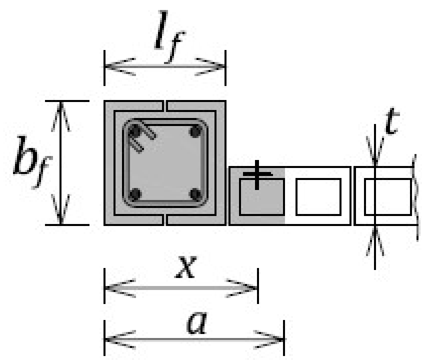
The centroid of the masonry compression zone:

$$x = \frac{b_f * l_f * \left(a - \frac{l_f}{2}\right) + \left(a - l_f\right)^2 * t/2}{A_L} = \frac{390*790*(1010 - \frac{790}{2}) + (1010-790)^2 * 240/2}{3.6*10^5} = 539$$

The resultant of masonry compression stress is

$$C_m = \left(0.85\phi_m f'_m\right)A_L = (0.85*0.6*13.5)(3.6*10^5) = 2480 \text{ kN}$$

Finally, the factored moment resistance of the wall section is



$$M_r = C_m(l_w/2 - x) + 2(\phi_s f_y A_c)(l_w/2 - d') = 2.48*1$$

$$+2(0.85*400*1600)(9000/2-395) = 14300 \text{ kNm}$$

$M_r = 14300$ kNm > $M_f = 13100$ kNm OK

Note that

$c/l_w = 1259/9000 = 0.14 < 0.2$

therefore the S304-14 minimum rotational demand requirement for Ductile shear walls is satisfied.

9. Determine the size of boundary elements (see Section 2.6.8.3).

The proposed thickness of a boundary element is

$t_b = 390$ mm

and the proposed length is

$l_b = 790$ mm

Note that the length of a boundary element should not be less than the largest of the following three values (Cl.16.11.2):

$l_b \geq (c - 0.1l_w, c/2, c(\varepsilon_{mu} - 0.0025)/\varepsilon_{mu})$

The selection of the length is an iterative process, since it is required to perform a design for axial load and flexure in order to determine the neutral axis depth *c*, hence

$c - 0.1l_w = 1259 - 0.1*9000 = 359$ mm

$c/2 = 1259/2 = 630$ mm

The larger of these two values will govern, that is,

$l_b \geq 630$ mm

Hence, the proposed value of 790 mm is OK. Note that the third criterion is as follows

$l_b \geq c(\varepsilon_{mu} - 0.0025)/\varepsilon_{mu}$

Cannot be followed at this stage because ε_{mu} is not known.

10. Perform the S304-14 ductility check (see Section 2.6.3).

To satisfy the S304-14 ductility requirements for Ductile shear walls (Cl.16.9.7), the neutral axis depth ratio (c/l_w) should be less than the following limit:

$c/l_w \leq 0.125$ when $h_w/l_w \geq 5$

In this case,

$$\frac{h_w}{l_w} = 1.67 < 5$$

Also, the neutral axis depth

c = 1259 mm

and so

$c/l_w = 1259/9000 = 0.14 > 0.125$

Therefore, the simplified S304-14 ductility requirement is <u>not satisfied</u>. Consequently, a detailed ductility check according to S304-14 Cl.16.8.8 needs to be performed. It is required to determine the rotational demand θ_{id} and the rotational capacity θ_{ic}, and to confirm that the capacity exceeds the demand.

The rotational demand depends on the elastic lateral displacement at the top of the wall, which is given as

$\Delta_{f1} = 18$ mm

The overstrength factor must be at least equal to 1.3 and can be determined from the following equation:

$$\gamma_w = \frac{M_n}{M_f} = \frac{16600}{13100} = 1.27 < 1.3$$

Hence,

$\gamma_w = 1.3$

In this case, the nominal moment capacity is equal to M_n = 16,600 kNm, which was calculated in the same manner as the factored moment resistance M_r, except that unit values of material resistance factors $\phi_m = \phi_s = 1.0$ were used.

The S304-14 minimum rotational demand is θ_{min} = 0.004 for Ductile shear walls. The actual value is determined from the following equation:

$$\theta_{id} = \frac{\left(\Delta_{f1} R_o R_d - \Delta_{f1}\gamma_w\right)}{h_w - \frac{\ell_w}{2}} = \frac{\left(18 \cdot 3.0 \cdot 1.5 - 18 \cdot 1.30\right)}{\left(15.0 - \frac{9.0}{2}\right) \cdot 10^3} = 5.49 \cdot 10^{-3}$$

This is greater than θ_{min} = 0.004, so the actual rotational demand will be used.

The required maximum compressive strain value can be determined from the following equation (see Section 2.6.8.2)

$$\varepsilon_{mu} \geq \left(\theta_{id} + 0.002\right)\frac{2c}{l_w} = (5.49 * 10^{-3} + 0.002)\frac{2 * 1259}{9000} = 0.0021$$

Note that

$l_b \geq c(\varepsilon_{mu} - 0.0025)/\varepsilon_{mu}$

However, this criterion cannot be applied since ε_{mu} is less than 0.0025.

11. Minimum required factored shear resistance (see Section 2.6.5 and S304-14 Cl.16.10.4.3)

Cl.16.10.4.3 requires that the factored shear resistance, V_r, should be greater than the shear due to the effects of factored loads, but not less than i) the shear corresponding to the development of probable moment capacity, M_p, or ii) the shear corresponding to the lateral seismic load (base shear), where earthquake effects were calculated using $R_d R_o$ = 1.3.

The first requirement is based on the Capacity Design approach (see Section 2.5.1 for more details). For Ductile shear walls, the shear capacity should exceed the shear corresponding to the probable moment capacity, as follows

$M_p = 18600$ kNm

The shear force resultant acts at the effective height h_e, that is, the distance from the base of the wall to the resultant of all seismic forces acting at the floor levels. Note that h_e can be determined as follows

$$h_e = \frac{M_f}{V_f} = 10.0 \text{ m}$$

The shear force V_{pb} corresponding to the overturning moment M_p is equal to

$$V_{pb} = \frac{M_p}{h_e} = \frac{18600}{10.0} = 1860 \text{ kN}$$

The second requirement gives an “almost elastic” factored base shear force for the wall, which is equal to

$$V_{fe} = \frac{V_f \cdot R_d \cdot R_o}{1,.3} = \frac{1310 \cdot 3.0 \cdot 1.5}{1.3} = 4535 \text{ kN}$$

The smaller of these two values should be used, hence

$V_{rd} = 1860$ kN

12. Diagonal tension shear resistance (see Section 2.6.5 and S304-14 Cl.10.10.2.1)

Masonry shear resistance (V_m):

$b_w = 240$ mm overall wall thickness

$d_v \approx 0.8 l_w = 7200$ mm effective wall depth

$\gamma_g = 1.0$ solid grouted wall

$P_d = 0.9 P_f$ = 1620 kN

$$v_m = 0.16(2 - \frac{M_f}{V_f d_v})\sqrt{f'_m} = 0.59 \text{ MPa}$$

Since

$$\frac{M_f}{V_f d_v} = \frac{13100}{1310 * 7.2} = 1.39 > 1.0$$

use $\frac{M_f}{V_f d_v} = 1.0$

$V_m = \phi_m (v_m b_w d_v + 0.25 P_d)\gamma_g$ = 0.6(0.59*240*7200+0.25*1620*10^3)*1.0 = 852 kN

The required steel shear resistance can be found from the following equation (see Section 2.6.5 and S304-14 Cl.16.10.4.1)

$$V_r = (0.0025/(2\varepsilon_{mu}))V_m + V_s \geq V_{rd}$$

Since

$$0.0025/(2\varepsilon_{mu}) = 0.0025/(2 * 0.0021) = 0.59$$

Then

$V_s = V_{rd} - 0.59V_m = 1860 - 0.59*852 = 1357$ kN

The required amount of reinforcement can be found from the following equation

$$\frac{A_v}{s} = \frac{V_s}{0.6\phi_s f_y d_v} = \frac{1357*10^3}{0.6*0.85*400*7200} = 0.92$$

Try 2-20M bond beam reinforcing bars at 600 mm spacing ($A_v = 600$ mm^2 and $s = 600$ mm):

$$\frac{A_v}{s} = \frac{600}{600} = 1.0 > 0.92 \quad \text{OK}$$

Steel shear resistance V_s:

$$V_s = 0.6\phi_s A_v f_y \frac{d_v}{s} = 0.6*0.85*\frac{400}{1000}*600*\frac{7200}{600} = 1470 \text{ kN}$$

Total diagonal shear resistance:

$V_r = 0.59V_m + V_s = 0.59 \cdot 852 + 1470 = 1973$ kN > $V_{rd} = 1860$ kN OK

Maximum shear allowed on the section is (S304-14 Cl.10.10.2.1)

$$\max V_r = 0.4\phi_m \sqrt{f'_m} b_w d_v \gamma_g = 1520 \text{ kN}$$

Since

$V_r = 1973$ kN > $\max V_r = 1520$ kN

the above maximum shear resistance requirement has not been satisfied. It would be required to increase either wall thickness or length to satisfy this requirement. It is recommended to perform this check at an early stage of the design.

13. Sliding shear resistance (see Sections 2.3.3 and 2.6.7, and S304-14 Cl.10.10.5.1 and 16.10.4.2)

The factored in-plane sliding shear resistance V_r is determined as follows:

μ = 1.0 for a masonry-to-masonry or masonry-to-roughened concrete sliding plane

A_s = 5200 mm^2 total area of vertical wall reinforcement

For Ductile shear walls, only the vertical reinforcement in the tension zone should be accounted for in the T_y calculations (S304-14 Cl.16.10.4.2), (also see Figure 2-17b)

$$T_y = \phi_s A_s f_y \left(\frac{l_w - c}{l_w}\right) = 0.85*5200*400*\left(\frac{9000-1259}{9000}\right) = 1520 \text{ kN}$$

P_d = 1620 kN

$C = P_d + T_y$ = 1620+1520 = 3140 kN

$V_r = \phi_m \mu C$ = 0.6*1.0*3140 = 1884 kN

$V_r = 1884$ kN > $V_{rd} = 1860$ kN OK

14. Shear at the interface (see Section 2.6.8.4 and S304-14 Cl.16.11.10)

It is required to check whether the horizontal wall reinforcement is sufficient to resist the vertical shear stresses at the boundary element interface. The shear flow demand is based on the design shear force transferred over the storey height, that is,

$$V_{sf} = \frac{V_{rd}}{h} = \frac{1860}{3.0} = 620 \text{ kN/m}$$

The shear flow resistance is as follows (Cl.16.11.10)

$V_{fr} = \phi_m \mu F_s$

The resistance provided by horizontal reinforcement (2-20M bars at 600 mm spacing) is as follows

$V_{fr} = \phi_m \mu F_s = 0.6 * 1.0 * 340 = 204$ kN/m

Where

$F_s = \phi_s f_y (A_v / s) = 0.85 * 400 * (600/600) = 340$ kN/m

is the shear flow resistance provided by the horizontal reinforcement. Since

$V_{fr} < V_{sf}$

it follows that additional horizontal reinforcement is required to satisfy the requirement. Let us assume that 2-20M bars (total area 600 mm^2) will be provided at 200 mm spacing throughout the wall height at the first-floor level, that is,

$F_s = \phi_s f_y (A_v / s) = 0.85 * 400 * (600/200) = 1020$ kN/m

$V_{fr} = \phi_m \mu F_s = 0.6 * 1.0 * 1020 = 612$ kN/m

This shear flow resistance approximately satisfies the shear flow demand. The difference (620-612=8 kN/m) is 1% of the total demand, which is insignificant.

15. Detailing of boundary elements (see Section 2.6.8.5 and S304-14 Cl.16.11)

1) Regular ties and buckling prevention ties within the plastic hinge zone

Dimensions of a boundary element:

$l_b = 790$ mm length

$t_b = 390$ mm thickness

$A_g = l_b * t_b = 790 * 390 = 3.08 * 10^5$ mm^2

For the rectangular hoop reinforcement, the minimum area A_{sh} in each principal direction should not be less than the larger of the following (S304-14 Cl.16.11.6):

$$A_{sh} = 0.2 k_n k_{p1} \frac{A_g}{A_{ch}} \frac{f'_m}{f_{yh}} s \cdot h_c$$

or

$$A_{sh} = 0.09 \frac{f'_m}{f_{yh}} s \cdot h_c$$

where

$$k_n = \frac{n_l}{n_l - 2} = \frac{8}{8 - 2} = 1.33$$

$n_l = 8$ number of supported bars around the perimeter of a boundary element

$k_{p1} = 0.1 + 30\varepsilon_{mu} = 0.1 + 30 * 0.0021 = 0.163$

$A_{ch} = 290 * 690 = 2.0 * 10^5$ mm^2

is the area of the confined core and $h_c = 690$ mm is the larger dimension of the confined core (the dimension in other direction is 290 mm)

The maximum spacing of buckling prevention ties within the plastic hinge zone should not exceed the lesser of (S304-14 Cl.16.11.4)

$s \le (6d_b, 24d_{tie}, t_b/2)$

Where d_b is longitudinal bar diameter, and d_{tie} is the tie diameter, hence

$6d_b = 6*15 = 90$ mm

$24d_{tie} = 24*10 = 240$ mm

$t_b/2 = 390/2 = 195$ mm

Hence,

$s \le 90$ mm governs

Assume

$s = 80$ mm

The required area of tie reinforcement in boundary elements should be at least equal to the larger of

$$A_{sh} = 0.2k_n k_{p1}\frac{A_g}{A_{ch}}\frac{f'_m}{f_{yh}}s \cdot h_c = 0.2*1.33*0.163*\frac{3.08*10^5}{2.0*10^5}\frac{13.5}{400}*80*690 = 124 \text{ mm}^2$$

or

$$A_{sh} = 0.09\frac{f'_m}{f_{yh}}s \cdot h_c = 0.09\frac{13.5}{400}*80*690 = 168 \text{ mm}^2$$

Hence

$A_{sh} = 168$ mm^2 governs

This area of reinforcement can be achieved through 3-10M bars (total area 300 mm^2): two bars are a part of a regular tie enclosing the boundary element, plus a cross tie supporting intermediate bars.

2) Regular ties and buckling prevention ties outside the plastic hinge zone

The maximum spacing of buckling prevention ties outside the plastic hinge zone should not exceed the lesser of (S304-14 Cl.12.2.1)

$s \le (16d_b, 48d_{tie}, t_b)$

Where d_b is longitudinal bar diameter, and d_{tie} is the tie diameter, hence

$16d_b = 16*15 = 240$ mm

$48d_{tie} = 48*10 = 480$ mm

$t_b = 390$ mm

Hence,

$s \le 240$ mm governs

Assume

$s = 240$ mm

3) Vertical reinforcement: detailing

At any section within the plastic hinge region, no more than half of the area of vertical reinforcement may be lapped (S304-14 Cl.16.11.9).

16. The S304-14 seismic detailing requirements for Ductile shear walls – plastic hinge region

According to Cl.16.10.3, the required height of the plastic hinge region for Ductile shear walls is (see Table 2-5)

$h_p = 0.5l_w + 0.1h_w = 0.5 \cdot 9000 + 0.1 \cdot 15000 = 6000$ mm

However

$l_w \le h_p \le 2.0l_w$

Since

$l_w = 9000$ mm > 6000 mm

It follows that

$h_p = l_w = 9.0$ m governs.

The reinforcement detailing requirements for the plastic hinge region of Ductile shear walls are as follows (see Table 2-4 and Figure 2-41):

1. *The wall in the plastic hinge region must be solid grouted (Cl.16.6.2).*
2. *Horizontal reinforcement requirements:*

   a) Reinforcement spacing should not exceed the following limits (Cl.16.9.5.4):

   $s \le 600$ mm or

   $s \le l_w/2 = 9000/2 = 4500$ mm

   Since the lesser value governs, the maximum permitted spacing is

   $s \le 600$ mm

   According to the design, the horizontal reinforcement spacing is 600 mm, hence OK.

   b) Detailing requirements

   Horizontal reinforcement shall not be lapped within (Cl.16.9.5.4)

   600 mm or

   $l_w/5$ = 1800 mm

   whichever is greater, from the end of the wall. In this case, the reinforcement should not be lapped within the distance 1800 mm from the end of the wall. The horizontal reinforcement can be lapped at the wall half-length.

   Horizontal reinforcement shall be (Cl.16.9.5.4):

   i) provided by reinforcing bars only (no joint reinforcement!);

   ii) continuous over the length of the wall (can be lapped in the centre), and

   iii) have 180° hooks around the vertical reinforcing bars at the ends of the wall.
3. *Vertical reinforcement requirements:*

   a) Reinforcement spacing should not exceed the following limits (Cl.16.9.5.3):

   $s \le l_w/4 = 9000/4 = 2250$ mm, but need not be less than 400 mm

   or the minimum seismic requirements specified in Cl.16.4.5.3, which states that

   $s \le 1200$ mm (this value governs since the wall thickness is 240 mm).

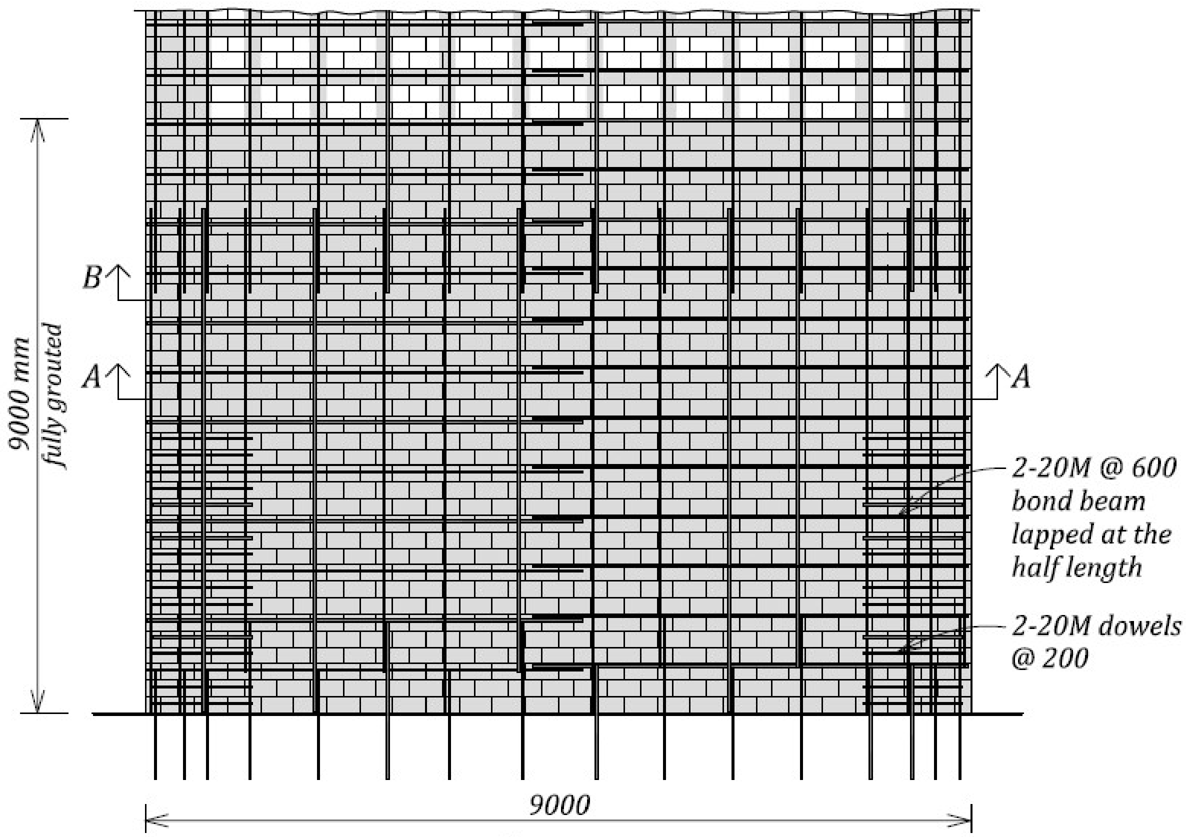
   Since the lesser value governs, the maximum permitted spacing is $s \le 1200$ mm.

   b) Detailing requirements

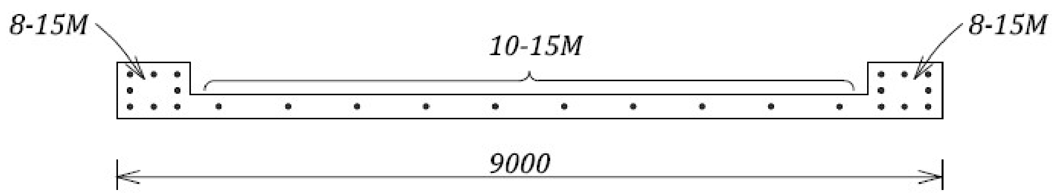
At any section within the plastic hinge region, no more than half of the area of vertical reinforcement may be lapped (Cl.16.9.5.2).

17. Design summary

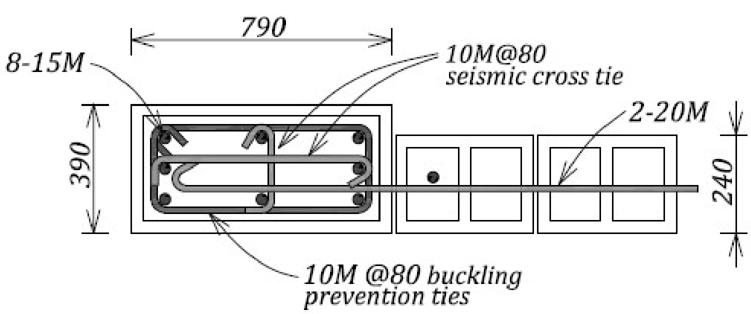
The reinforcement arrangement for the wall under consideration is summarized in the figure below. Note that a Ductile shear wall must be solid grouted in plastic hinge region, but it may be partially grouted outside the plastic hinge region (depending on the design forces).



Elevation



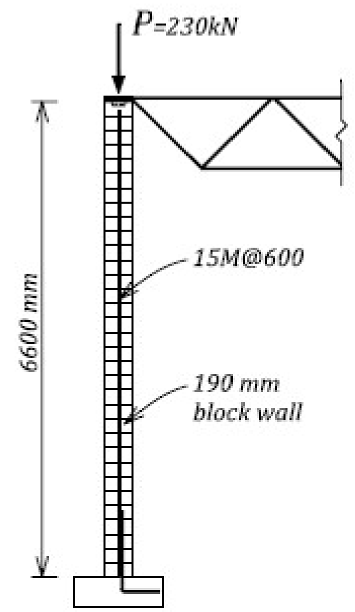
Section A-A



Section B-B

EXAMPLE 6 a: Design of a loadbearing wall for out-of-plane seismic effects

Verify the out-of-plane seismic resistance of the loadbearing block wall designed for in-plane loads in Example 4b, according to NBC 2015 and CSA S304-14 requirements. The wall is a part of a single-storey warehouse building located in Burnaby, BC, with soil corresponding to Site Class D. The wall is 8 m long and 6.6 m high, and is subjected to a total dead load of 230 kN (including its self-weight). The wall is constructed with 200 mm hollow concrete blocks of 15 MPa unit strength, Type S mortar, and solid grouting. The wall is reinforced with 15M Grade 400 vertical rebars at 600 mm on centre spacing. The slenderness effects outlined in S304-14 will not be considered in this design.



SOLUTION:

1. Material properties

Steel (both reinforcing bars and joint reinforcement):

ϕ_s = 0.85 f_y = 400 MPa

Masonry:

ϕ_m = 0.6

S304-14 Table 4, 15 MPa concrete blocks and Type S mortar:

f'_m = 7.5 MPa (assume solid grouted masonry)

2. Determine the out-of-plane seismic load according to NBC 2015 (see Section 2.7.7.3).

This design requires the calculation of seismic load V_p for parts of buildings and nonstructural components according to NBC 2015 Cl.4.1.8.18. First, seismic design parameters need to be determined as follows:

- Location: Burnaby, BC (NBC 2015 Appendix C)

  $S_a(0.2)$ = 0.768 and PGA_{ref} = 0.50
- Foundation factors

$F_a = F(0.2) = 0.9$ and Site Class D for PGA_{ref} = 0.50 (from Table 1-3 or NBC 2015 Table 4.1.8.4.B)

- I_E = 1.0 normal importance building

Find S_p (horizontal force factor for part or portion of a building and its anchorage per NBC 2015, Table 4.1.8.18, Case 1)

$C_p = 1.0 \quad A_r = 1.0 \quad R_p = 2.5 \quad A_x = 3.0$ ($h_x = h_n$ top floor)

$S_p = C_p A_r A_x / R_p = 1.0 \cdot 1.0 \cdot 3.0/2.5 = 1.2$

$0.7 < S_p < 4.0$ O.K.

- W_p = 4.0 kN/m^2 unit weight of the 190 mm block wall (solid grouted)

Seismic load V_p can be calculated as follows:

$V_p = 0.3 F_a S_a (0.2) I_E S_p W_p$ =0.3*0.9*0.69*1.0*1.2*(4.0 kN/m^2) = 0.99 kN/m^2 ≈ 1.0 kN/m^2

3. Determine the effective compression zone width (b) for the out-of-plane design (see Section 2.4.2).

According to S304-14 Cl.10.6.1, the effective compression zone width (b) should be taken as the lesser of the following two values (see Figure 2-19):

$b = s = 600$ mm spacing of vertical reinforcement

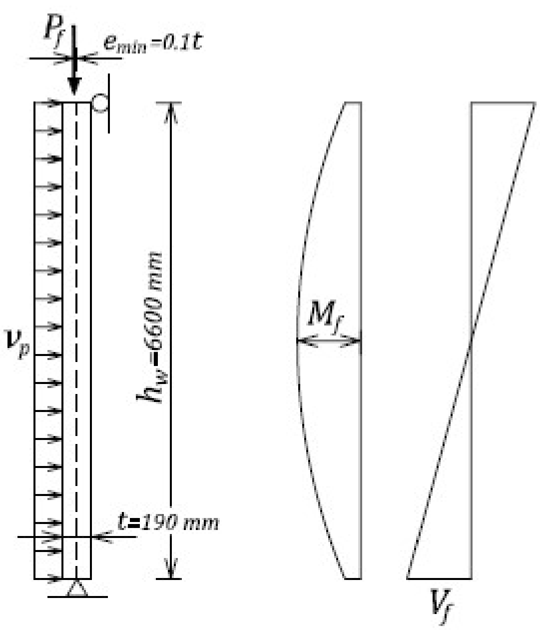
or

$b = 4t = 4*190 = 760$ mm

All design calculations in this example will be performed considering a vertical wall strip of width $b = 600$ mm.

4. Find the design shear force and the bending moment.

The wall will be modeled as a simple beam with pin supports at the base and top. The loads on the wall consist of axial load due to roof load and wall self-weight, plus the seismic out-of-plane load. The roof load and wall self-weight create moments due to minimum axial load eccentricity.



- Axial load per wall width equal to $b = 600$ mm:

$P_f = \frac{P}{l_w} * b = \frac{230kN}{8m} * 0.6 = 17.25 \approx 17.0$ kN

- Minimum eccentricity (S304-14 Cl.10.7.2)

$e_{\min} = 0.1t =$ 0.019 m

- Out-of-plane seismic load per wall width equal to $b = 600$ mm:

$v_p = 1.0 * 0.6 = 0.6$ kN/m

- Design bending moment (at the midheight):

$M_f = p * e_{\min} + \frac{v_p * h_w^2}{8} = 17 * 0.019 + \frac{0.6 * 6.6^2}{8}$

= 3.59 ≈ 3.6 kNm

5. Check whether the wall resistance for the combined effect of axial load and bending is adequate (see Section C.1.2).

This can be verified from a P-M interaction diagram which can be developed using the EXCEL© software (or commercially available masonry design software). Relevant tables used to develop the diagram are presented below, while the detailed theoretical background is outlined in Section C.1.2. Note that the design width is equal to $b = 600mm$.

Table 1. Design Parameters

Design parameter	Unit	Symbol	Value
Wall thickness	mm	t	190
Design width	mm	b	600
Masonry maximum strain		EPSm	0.003
Masonry strength	MPa	f'm	7.5
Steel yield strength	MPa	fy	400
Steel modulus of elasticity	MPa	Es	200000
Effective depth	mm	d	95
(c/d)balanced			0.6
Reinforcement area	mm^2/b	As	200
Material resistance-masonry		Fim	0.6
Material resistance-steel		Fis	0.85
X- factor		X	1
BETA1		BETA1	0.8
Effective area	mm^2	Ae	114000

In this case, the reinforcement is placed at the centre of the wall and so

$$d = \frac{t}{2} = \frac{190}{2} = 95 \text{ mm}$$

The neutral axis depth corresponding to a balanced condition (onset of yielding in the steel and maximum compressive strain in masonry) can be determined from the following proportion

$$\frac{c_b}{d - c_b} = \frac{\varepsilon_m}{\varepsilon_y}$$

For $\varepsilon_m = 0.003$ and $\varepsilon_y = 0.002$ it follows that

$$c_b = 0.6d$$

The area of vertical reinforcement per width $b = 600$ mm can be determined as follows:

$$A_s = \frac{A_b}{s} * b = \frac{200}{600} * 600 = 200 \text{ mm}^2$$ (15M@ 600 mm reinforcement)

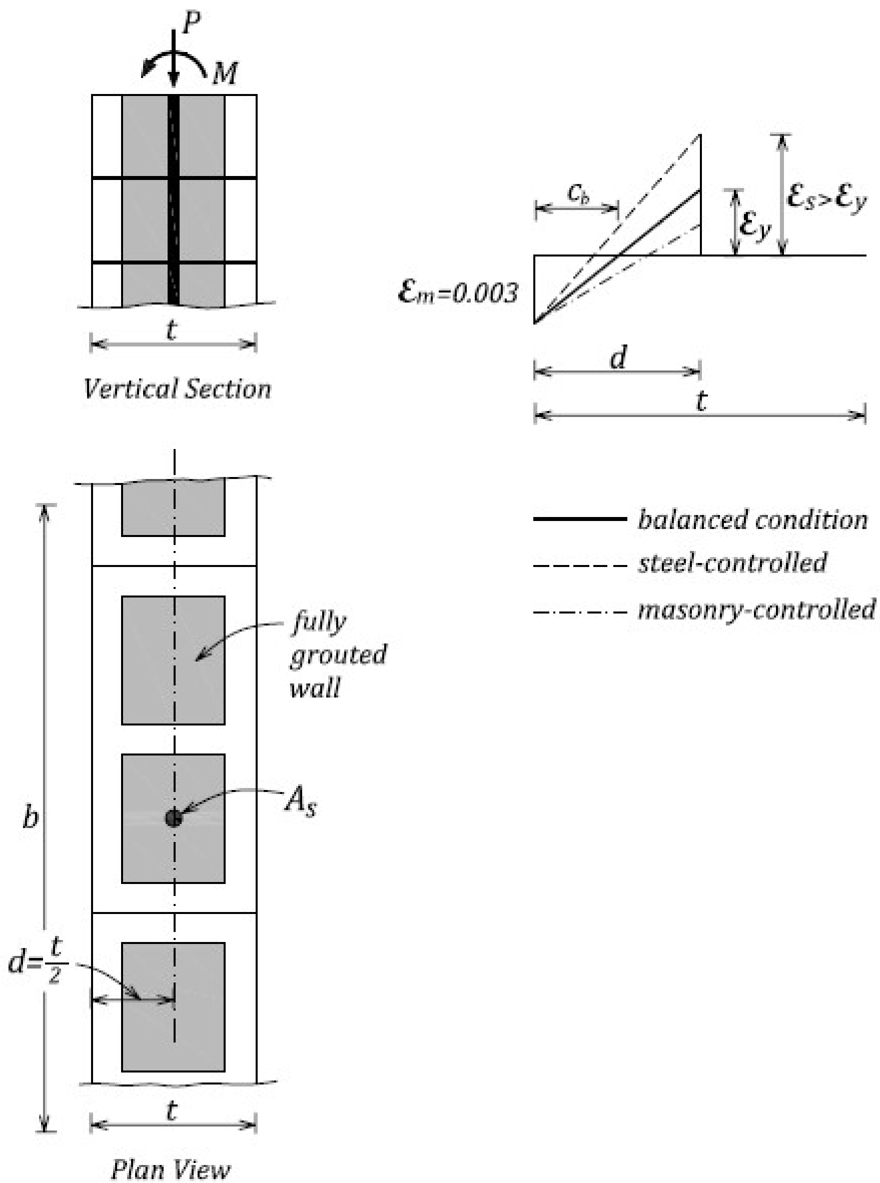
To determine whether the wall can carry the combined effect of axial load and bending moment, it is useful to construct an axial load-moment interaction diagram (also known as P-M interaction diagram). The P-M interaction diagram for this example was developed using Microsoft

EXCEL® spreadsheet, but other methods or computer programs are also available. The results of the calculations are presented in Table 2.

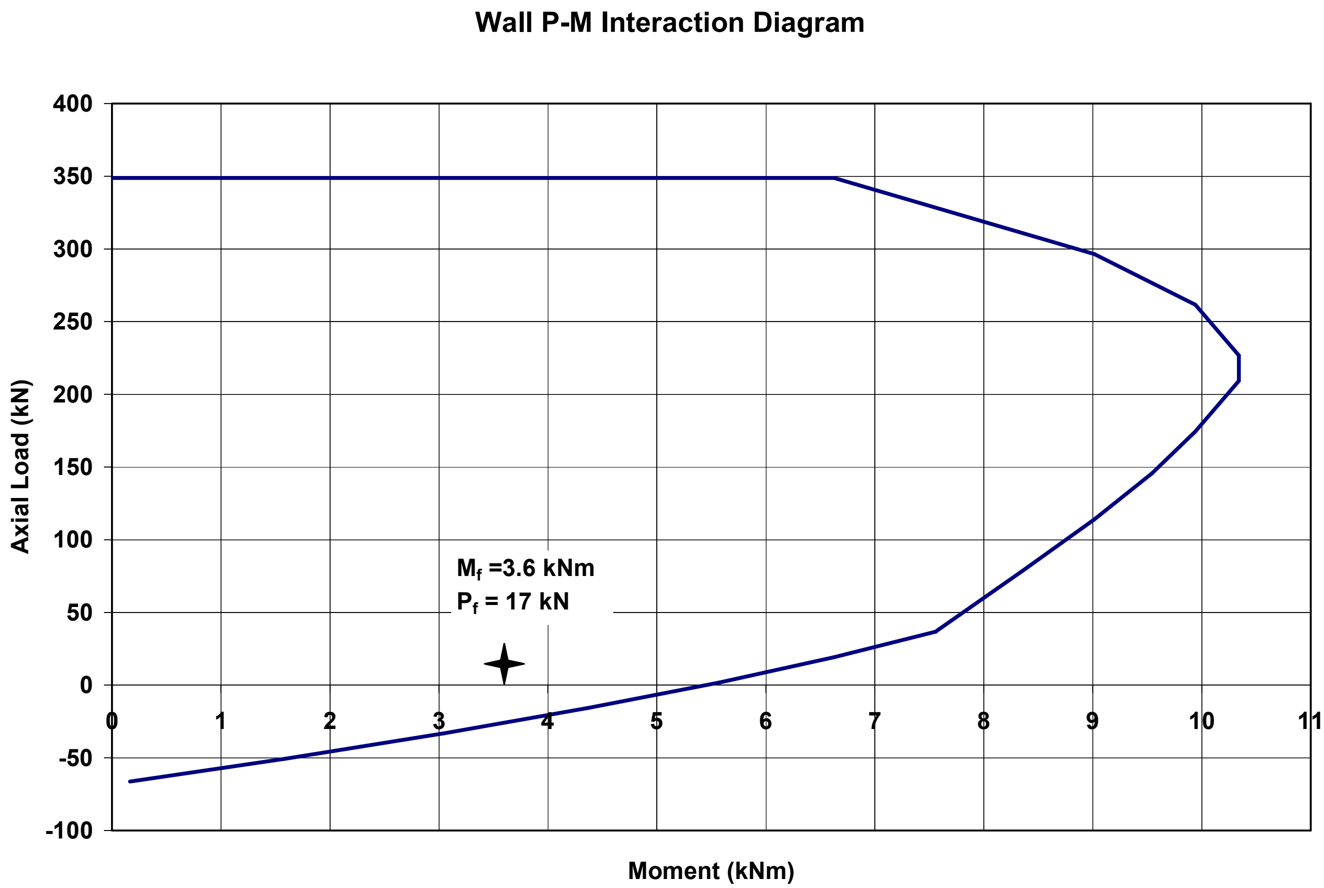
Table 2. P-M Interaction Diagram Values

		c/d	c	C_m	EPSs	T_r	M_r	P_r
			mm	N		N	kNm	kN
Points controlled by steel $c<c_b$		0.01	0.95	1744.2	0.02	68000	0.16504	-66.256
		0.1	9.5	17442	0.02	68000	1.59071	-50.558
		0.2	19	34884	0.02	68000	3.04886	-33.116
		0.3	28.5	52326	0.02	68000	4.37445	-15.674
		0.4	38	69768	0.02	68000	5.56749	1.768
		0.5	47.5	87210	0.02	68000	6.62796	19.21
		0.6	57	104652	0.02	68000	7.55587	36.652
Points controlled by masonry $c>c_b$		0.6	57	104652	0.002	68000	7.55587	36.652
		0.7	66.5	122094	0.00129	43714.3	8.35123	78.3797
		0.8	76	139536	0.00075	25500	9.01403	114.036
		0.9	85.5	156978	0.00033	11333.3	9.54426	145.645
Full section under compression		1	95	174420	0	0	9.94194	174.42
		1.2	114	209304	-0.0005	-17000	10.3396	209.304
		1.3	123.5	226746	-0.0007	-23538	10.3396	226.746
		1.5	142.5	261630	-0.001	-34000	9.94194	261.63
		1.7	161.5	296514	-0.0012	-42000	9.01403	296.514
		2	190	348840	-0.0015	-51000	6.62796	348.84
Pure compression							0	348.84

The three basic cases considered in the development of the interaction diagram (steel-controlled behaviour, masonry-controlled behaviour, and the balanced condition) are illustrated on the figure below. For more detailed explanation related to the development of P-M interaction diagrams refer to Section C.1.2.



The P-M interaction diagram showing the point of interest ($M_f = 3.6$ kNm and $P_f = 17$ kN) is shown below. It is obvious that the wall resistance to combined effects of axial load and out-of-plane bending is adequate for the given design loads and the reinforcement determined in Example 4b.



6. Check whether the out-of-plane shear resistance of the wall is adequate (S304-14 Cl.10.10.3, see Section 2.4.2).

Design shear force at the support per wall width $b = 600$ mm:

$$V_f = \frac{v_p * h_w}{2} = \frac{0.6 * 6.6}{2} \approx 2.0 \text{ kN}$$

According to S304-14 Cl.10.10.3, the factored out-of-plane shear resistance (V_r) shall be taken as follows

$$V_r = \phi_m (v_m \cdot b \cdot d + 0.25 P_d)$$

where

$v_m = 0.16\sqrt{f'_m}$ = 0.44 MPa (f'_m = 7.5 MPa for solid grouted 15 MPa block)

$d = 95$ mm effective depth (to the block mid-depth)

$b = 600$ mm effective compression zone width

The axial load P_d can be determined as

$$P_d = 0.9 P_f = 0.9 * 17.25 = 15.5 \text{ kN}$$

(note that the load has been prorated in proportion to the effective compression zone width b).

So,

$$V_r = 0.6 * (0.44 * 600 * 95 + 0.25 * 15500) = 17.4 \text{ kN}$$

Since

$V_f = 2.0$ kN < $V_r = 17.4$ kN   OK

Maximum shear allowed on the section is

$\max V_r = 0.4\phi_m \sqrt{f'_m}(b * d) = 0.4 * 0.6 * \sqrt{7.5} * (600 * 95) = 37.5$ kN   OK

7. Check the sliding shear resistance (see Section 2.4.3).
The factored out-of-plane sliding shear resistance V_r is determined according to S304-14 Cl.10.10.5.2, as follows:

μ = 1.0 for a masonry-to-masonry or masonry-to-roughened concrete sliding plane

A_s = 200 mm^2 area of vertical reinforcement per wall width $b = 600$ mm

$T_y = \phi_s A_s f_y$ = 0.85*200*400 = 68 kN

$P_d = 0.9 P_f = 15.5$ kN

$P_2 = P_d + T_y$ = 15.5+68 = 83.5 kN

$V_r = \phi_m \mu P_2$= 0.6*1.0*83.5= 50.0 kN

$V_r = 50.0$ kN > $V_f = 2.0$ kN   OK

Note that the sliding shear resistance does not govern in this case, however this mechanism often governs the in-plane shear resistance.

8. Conclusion
It can be concluded that the out-of-plane seismic resistance for this wall is satisfactory. This wall seems to be overdesigned for the out-of-plane resistance because the in-plane seismic design governs (this is a common scenario in design practice).

EXAMPLE 6 b: Design of a nonloadbearing wall for out-of-plane seismic effects

Consider the same masonry wall discussed in Example 6a, but in this example treat is as a nonloadbearing wall. The wall is 8 m long and 6.6 m high and is constructed using 200 mm hollow concrete blocks of 15 MPa unit strength and Type S mortar. Verify the out-of-plane seismic resistance of the wall according to NBC 2015 and CSA S304-14 seismic requirements.

Consider the following two cases:
a) unreinforced wall, and
b) reinforced partially grouted wall (use Grade 400 steel reinforcement for this design).

Use the seismic load determined in Example 6a, that is, $v_p = 1.0$ kN/m^2.

SOLUTION:

Material properties

Steel (both reinforcing bars and joint reinforcement):

ϕ_s = 0.85 f_y = 400 MPa

Masonry:

ϕ_m = 0.6

Compression resistance (S304-14 Table 4, 15 MPa concrete blocks and Type S mortar):

f'_m = 9.8 MPa (ungrouted, or partially grouted ignoring grout area)

Tension resistance normal to bed joint (S304-14 Table 5):

f_t = 0.4 MPa (ungrouted)

Find the design shear force and the bending moment.

The wall will be modeled as a simple beam with pin supports at the base and the top. The wall height is $h_w = 6.6$ m. A unit wall strip (width $b = 1000$ mm) will be considered for this design.

The forces on the wall consist of the axial load due to the wall self-weight and the bending moment due to seismic out-of-plane load (NBC 2015 load combination 1xD+1xE).

- Factored axial load per width b of 1.0 m:

wall weight w = 2.46 kN/m^2 (ungrouted 190 mm block wall)

$$P_f = w * \frac{h_w}{2} * b = (2.46) * \frac{6.6}{2} * 1.0 = 8.1 \text{ kN/m}$$

- Out-of-plane seismic load per width b of 1.0 m:

$v_p = 1.0$ kN/m

- Factored bending moment (at the midheight):

$$M_f = \frac{v_p * h_w^2}{8} = \frac{1.0 * 6.6^2}{8} \approx 5.5 \text{ kNm/m}$$

- Factored shear force (at the support):

$$V_f = \frac{v_p * h_w}{2} = \frac{1.0 * 6.6}{2} \approx 3.3 \text{ kN/m}$$

a) Unreinforced wall

Check whether the wall resistance to the combined effect of axial load and bending is adequate (see Section 2.7.1.3).

Find the load eccentricity:

$$e = \frac{M_f}{P_f} = \frac{5.5kNm}{8.1kN} = 0.68m = 680mm$$

According to S304-14 Cl.7.2.1, an unreinforced masonry wall is to be designed as uncracked if $e > 0.33t$

where t denotes the wall thickness ($t = 190mm$)

$0.33t = 0.33 * 190 = 63mm$

In this case,

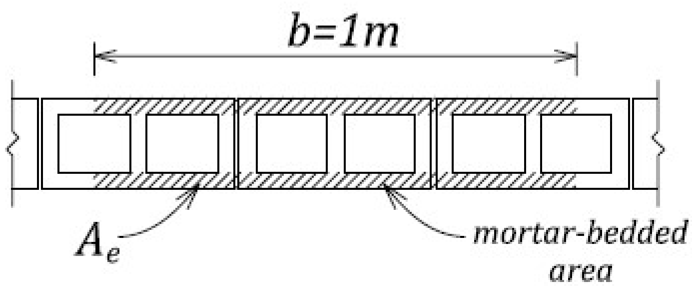
$e = 680mm > 0.33t = 63mm$

so the wall will be designed as uncracked (i.e. the maximum tensile stress is less than the allowable value) according to S304-14 Cl.7.2. The design procedure is explained in Section 2.7.1.3.

First, we need to determine properties for the effective wall section for a width $b = 1000$ mm. For a hollow 190 mm wall, the values obtained from Table D-1 are as follows:

$A_e = 75.4 * 10^3$ mm²/m effective cross-sectional area

$S_e = 4.66 * 10^6$ mm³/m section modulus of effective cross-sectional area



The maximum compression stress at the wall face can be calculated as follows:

$$\max f_c = \frac{P_f}{A_e} + \frac{M_f}{S_e} = \frac{8.1 * 10^3}{75.4 * 10^3} + \frac{5.5 * 10^6}{4.66 * 10^6} = 0.107 + 1.18 = 1.29MPa$$

The allowable value is equal to

$\phi_m f'_m = 0.6 * 9.8 = 5.9MPa$

Since

$\max f_c = 1.29MPa < 5.9MPa$

it follows that the maximum compression stress is less than the allowable value.

Find the maximum tensile stress as follows:

$$\max f_t = \frac{P_f}{A_e} - \frac{M_f}{S_e} = \frac{8.1 * 10^3}{75.4 * 10^3} - \frac{5.5 * 10^6}{4.66 * 10^6} = 0.107 - 1.18 = -1.07MPa$$

The allowable value is equal to

$-\phi_m f_t = -0.6 * 0.4 = -0.24MPa$

Since

$\max f_t = -1.07MPa < -0.24MPa$
it follows that the maximum tensile stress exceeds the allowable value, which is not acceptable.

In this design, the tensile stress criterion is not going to be satisfied even if the wall thickness is increased to 290 mm. Therefore, a reinforced masonry wall is required in this case. Also, reinforcement in this wall is mandatory since the wall is to be constructed at Ottawa, ON, where the seismic hazard index $I_E F_a S_a(0.2)$=1.0*1.0*0.66=0.66 > 0.35. Therefore, the design will proceed considering a reinforced nonloadbearing wall.

b) Reinforced wall

i. Find the minimum seismic reinforcement for nonloadbearing walls (see Section 2.7.4).
According to S304-14 Cl.16.4.5.2a, if $0.35 \le I_E F_a S_a(0.2) \le 0.75$ nonloadbearing walls shall be reinforced in one or more directions with reinforcing steel having a minimum total area of

$A_{stotal} = 0.0005A_g$

The reinforcement may be placed in one direction, provided that it is located to reinforce the wall adequately against lateral loads and spans between lateral supports.

$A_{stotal} = 0.0005A_g$ = 0.0005*(190*10^3 mm^2) = 95 mm^2/m

where

A_g =(1000mm)*(190mm)=190*10^3 mm^2 gross cross-sectional area per metre of wall length

Let us choose 15M vertical reinforcement (area 200 mm^2) at 1200 mm spacing which is the maximum spacing allowed (1200 mm).
The area of reinforcement per metre of wall length is

$$A_s = 200 * \frac{1000}{1200} = 167 \text{ mm}^2/\text{m} > 95 \text{ mm}^2/\text{m} \quad \text{OK}$$

ii. Determine the effective compression zone width (b) for the out-of-plane design (see Section 2.4.2).
The wall resistance will be determined considering a strip equal to the bar spacing s =1200 mm, as follows:

$$P_f = 8.1 * \frac{1.2}{1.0} = 9.7 \text{ kN}$$

$$M_f = 5.5 * \frac{1.2}{1.0} = 6.6 \text{ kNm}$$

$$V_f = 3.3 * \frac{1.2}{1.0} = 4.0 \text{ kN}$$

iii. Check whether the wall resistance to the combined effect of axial load and bending is adequate (see Section C.1.2).
Since this is a partially grouted wall, its flexural resistance will be determined using a T-section model.

According to S304-14 Cl.10.6.1, the effective compression zone width (b) should be taken as the lesser of the following two values (see Figure 2-19):

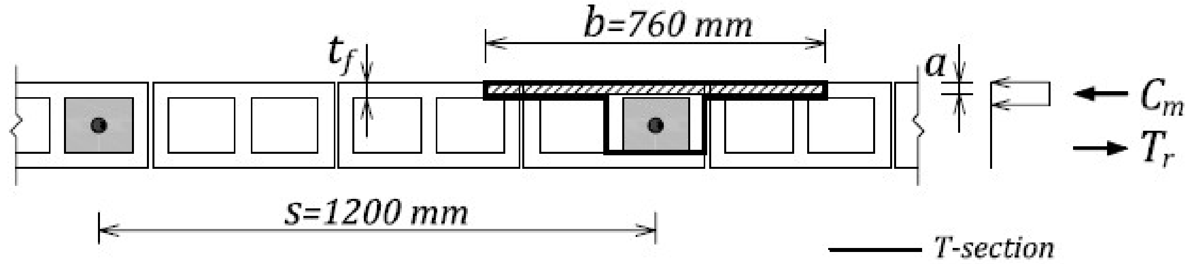
$b = s = 1200$ mm

or

$b = 4t = 4*190 = 760$ mm

Therefore, $b = 760$ mm will be used as the width of the masonry compression zone.

A typical wall cross-section is shown on the figure below. Note that the face shell thickness is 38 mm (typical for a hollow block masonry unit). The same value can be obtained from Table D-1, considering the case of an ungrouted 200 mm block wall.



Since the reinforcement is placed at the centre of the wall, the effective depth is equal to

$d = \frac{t}{2} = \frac{190}{2} = 95$ mm

The reinforcement area used for the design needs to be determined as follows:

$A_s = A_b = 200$ mm^2

The internal forces will be determined as follows (see Figure C-9):

$T_r = \phi_s f_y A_s = 0.85 * 400 * 200 = 68000$ N

Since

$C_m = P_f + T_r = 9700 + 68000 = 77700$ N

and

$C_m = (0.85\phi_m f'_m)(b \cdot a)$

the depth of the compression stress block a can be determined as follows

$a = \frac{C_m}{0.85\phi_m f'_m b} = \frac{77700}{0.85 * 0.6 * 9.8 * 760} = 20$ mm

Since

$a = 20mm < t_f = 38mm$

the neutral axis is located in the face shell (flange). The moment resistance around the centroid of the wall section can be determined as follows

$M_r = C_m(d - a/2) = 77700 * (95 - 20/2) = 6.6$ kNm

Since

$M_r = 6.6$ kNm = $M_f = 6.6$ kNm

it follows that the wall flexural resistance is adequate. However, the reinforcement spacing could be reduced to s =1000 mm to allow for an additional safety margin (the revised moment resistance calculations are omitted from this example).

iv. Check whether the out-of-plane shear resistance of the wall is adequate (see Section 2.4.2).

According to S304-14 Cl.10.10.3, the factored out-of-plane shear resistance (V_r) shall be taken as follows

$V_r = \phi_m (v_m \cdot b \cdot d + 0.25 P_d)$ where

$v_m = 0.16\sqrt{f'_m}$ = 0.50 MPa

$d = 95$ mm effective depth

$b \approx 200$ mm web width - equal to the grouted cell width (156 mm) plus the thickness of the adjacent webs (26 mm each)

The axial load P_d can be determined as

$P_d = 0.9P_f = 0.9 * 9.7 = 8.7$ kN

Thus,

$V_r = 0.6 * (0.50 * 200 * 95 + 0.25 * 8700) = 7.0$ kN

Since

$V_f = 4.0$ kN < $V_r = 7.0$ kN OK

Maximum shear allowed on the section is

$\max V_r = 0.4\phi_m \sqrt{f'_m}(b * d) = 0.4 * 0.6 * \sqrt{9.8} * (200 * 95) = 14.3$ kN OK

v. Check the sliding shear resistance (see Section 2.4.3).

The factored in-plane sliding shear resistance V_r is determined according to S304-14 Cl.10.10.5.2, as follows:

μ = 1.0 for a masonry-to-masonry or masonry-to-roughened concrete sliding plane

A_s = 200 mm^2 area of vertical reinforcement at 1.2 m spacing

$T_y = \phi_s A_s f_y$ = 0.85*200*400 = 68.0 kN

P_d = 8.7 kN

$P_2 = P_d + T_y$ = 8.7+68.0 = 76.7 kN

$V_r = \phi_m \mu P_2$ = 0.6*1.0*76.7 = 46.0 kN

$V_r = 46.0$ kN > $V_f = 4.0$ kN OK

vi. Conclusion

It can be concluded that the out-of-plane seismic resistance of this nonloadbearing wall is satisfactory. It should be noted that the flexural resistance governs in this design. The required amount of vertical reinforcement (15M@1200 mm) corresponds to the following area per metre length

$$A_s = A_b * \frac{1000}{s} = 167 \text{ mm}^2$$

which is significantly larger than the minimum seismic reinforcement prescribed by S304-14, that is, $A_{stotal} = 95$ mm^2/m. Note that 15M@1200 mm is also the minimum vertical reinforcement that meets the minimum spacing requirements using typical15M bars.

Also, since horizontal reinforcement does not contribute to out-of-plane wall resistance, it was not considered in this example. However, provision of 9 Ga. horizontal ladder reinforcement at 400 mm spacing could be considered to improve the overall seismic performance of the wall.

It should be noted that, in exterior walls the mortar-bedded joints could be significantly affected by the presence of aesthetic joint finishes characterized by deeper grooves (e.g. raked joints); some of the grooves are up to 10 mm deep. The designer should consider this effect in the calculation of the compression zone depth.

EXAMPLE 7: Seismic design of masonry veneer ties

Perform the seismic design for tie connections for a 4.8 m high concrete block veneer wall in a school gymnasium in Montréal, Quebec. The building is founded on Site Class C. The design should be performed to the requirements of NBC 2015, CSA S304-14, and CSA A370-14. Consider the following two types of the veneer backup:
a) Concrete block wall (a rigid backup), and
b) Steel stud wall with 400 mm steel stud spacing (a flexible backup).
c) Evaluate the minimum tie strength requirements for the rigid and flexible backup.

SOLUTION:

This design problem requires the calculation of seismic load V_p for nonstructural elements according to NBC 2015 Cl.4.1.8.18 (for more details see Section 2.7.7.3). Note that the wind load could govern in a tie design for many site locations in Canada, however wind load calculations were omitted for this seismic design example.

First, seismic design parameters need to be determined as follows:

- Location: Montréal (City Hall), Quebec (NBC 2015 Appendix C)
  $S_a(0.2)$ = 0.595 and PGA_{ref} = 0.379
- Foundation factor
  $F_a = F(0.2) = 1.0$ and Site Class C for PGA_{ref} = 0.379 (from Table 1-3 or NBC 2015 Table 4.1.8.4.B)
- I_E = 1.3 school (high importance building)

At this point, it would be appropriate to check whether the seismic design of ties is required for this design. According to NBC 2015 Cl.4.1.8.18.2, seismic design of ties is required when the seismic hazard index $I_E F_a S_a(0.2) \geq 0.35$ (and also for post-disaster buildings in lower seismic regions). In this case,

$I_E F_a S_a(0.2)$= 1.3*0.88*0.69=0.79 $\geq$ 0.35

Therefore, seismic design is required.

- Find S_p (horizontal force factor for part or portion of a building and its anchorage per NBC 2015, Table 4.1.8.18, Case 8)

$S_p = C_p A_r A_x / R_p = 1.0 \cdot 1.0 \cdot 3.0/1.5 = 2.0$

where

$A_x = 1 + 2h_x/h_n = 3.0$ for top of wall worst case

Since $0.7 < S_p < 4.0$ O.K.

- W_p = 1.8 kN/m^2 unit weight of the veneer masonry (concrete blocks)

Seismic load V_p can be calculated as follows:

$V_p = 0.3 F_a S_a(0.2) I_E S_p W_p$=0.3*1.0*0.595*1.3*2.0*(1.8 kN/m^2) =0.85 kN/m^2

Note that the above load is determined per m^2 of the wall surface area.

a) Concrete block backup (rigid)

Assume the maximum tie spacing permitted according to S304-14 Cl.9.1.3 of 600 mm vertically and 820 mm horizontally (see Section 2.7.7.2), resulting in a tributary tie area for a concrete backup wall of

A = 0.82*0.60 = 0.49 m^2

The required factored tie capacity should exceed the factored tie load, that is,

$V_f \geq V_p * A$ = (0.85 kN/m^2)*(0.49 m^2) = 0.42 kN

Alternatively, for a given tie capacity, a tie spacing could be determined based on the maximum tributary area calculated from V_p and the factored tie capacity V_f, that is,

$A \leq V_f / V_p$

b) Steel stud backup (flexible)

Since the steel stud is a flexible backup, a tie must be able to resist 40% of the tributary lateral load on a vertical line of ties (S304-14 Cl.9.1.3.3, see Section 2.7.7.3):

$V_f \geq 0.4 * V_p * A_t$ = 0.4*(0.85 kN/m^2)*(1.92m^2) = 0.65 kN

where A_t = 0.4m*4.8m = 1.92 m^2 is tributary area on a vertical line of ties based on a probable 0.4 m horizontal tie spacing, and 4.8 m wall height

According to the same S304-14 clause, the tie must also be able to resist a load corresponding to double the tributary area on a tie, that is,

$V_f = 2 * V_p * A$ = 2*(0.85 kN/m^2)*(0.4m*0.6m) = 0.41 kN

Note that the tributary area was based on a 0.4 m stud spacing, and the maximum vertical tie spacing of 0.6 m prescribed by S304-14 Cl.9.1.3.1.

In conclusion, the tie design load for the flexible veneer backup is V_f = 0.65 kN.

c) Minimum strength requirements

CSA A370-14 Cl.8.1 prescribes minimum <u>ultimate</u> tensile/compressive tie strength of 1 kN. In order to obtain the ultimate tie strength, the factored strength needs to be divided by the resistance factor ϕ. According to CSA A370-14 Cl.9.4.2.1.2, the resistance factor is 0.9 for tie material strength, or 0.6 for embedment failure, failure of fasteners, or buckling failure of the connection. It is conservative to use lower resistance factor in determining the ultimate tie strength V_{ult}.

- For the steel stud backup:

$V_r \geq V_f =$ 0.65 kN

thus the ultimate strength can be determined as follows

$$V_{ult} = \frac{V_r}{\phi} = \frac{0.65}{0.6} = 1.08 \text{ kN}$$

This value is slightly higher than the minimum of 1 kN prescribed by CSA A370-04 and governs.

- For the concrete block backup:

$V_r \geq V_f =$ 0.42 kN

thus the ultimate strength can be determined as follows

$$V_{ult} = \frac{V_r}{\phi} = \frac{0.42}{0.6} = 0.7 \text{ kN}$$

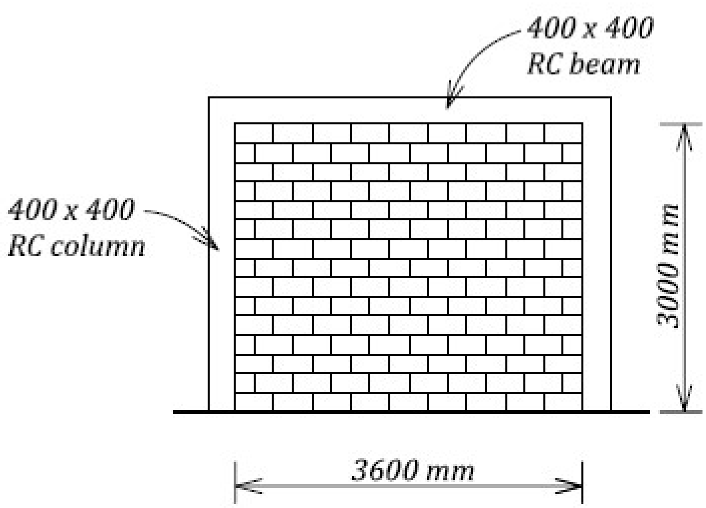
This value is less than the minimum of 1 kN, so the minimum requirement governs.

EXAMPLE 8: Seismic design of a masonry infill wall

A single-storey reinforced concrete frame structure is shown in the figure below. The frame is infilled with an unreinforced, ungrouted concrete block wall panel that is in full contact with the frame. The wall is built using 190 mm hollow blocks and Type S mortar.

a) Model the infill as an equivalent diagonal compression strut. Determine the strut dimensions according to CSA S304-14 assuming the infill-frame interaction.

b) Assuming that the infill wall provides the total lateral resistance, determine the maximum lateral load that the infilled frame can resist. Consider the following three failure mechanisms: strut compression failure, diagonal tension resistance, and sliding shear resistance.



Given:

E_f =25000 MPa concrete frame modulus of elasticity

f'_m = 9.8 MPa hollow block masonry, from 15 MPa block strength and Type S mortar (Table 4, CSA S304-14)

SOLUTION:

a) Find the diagonal strut properties.

- Key properties for the masonry wall and the concrete frame

Concrete frame:

E_f =25000 MPa

Beam and column properties:

$$I_b = I_c = \frac{(400)^4}{12} = 2.133*10^9 \text{ mm}^4$$

Masonry:

$$E_m = 850 f'_m = 850*9.8 = 8330 \text{ MPa}$$

Effective wall thickness (face shells only):

$t_e = 75$ mm (Table D-1, 200 mm hollow block wall)

- Diagonal strut geometry (see Section 2.7.2 and S304-14 Cl.7.13)

$h = 3000$ mm

$l = 3600$ mm

Find θ (angle of diagonal strut measured from the horizontal):

$$\tan(\theta) = \frac{h}{l} = \frac{3000}{3600} = 0.833 \qquad \theta = 39.8^\circ$$

Length of the diagonal:

$$l_d = \sqrt{h^2 + l^2} = \sqrt{3000^2 + 3600^2} = 4686 \text{ mm}$$

Find the strut width (see Figure 2-46):

$$\alpha_h = \frac{\pi}{2}\left(\frac{4E_f I_c h}{E_m t_e \sin 2\theta}\right)^{1/4} = \frac{\pi}{2}\left(\frac{4 * 25000 * 2.133 * 10^9 * 3000}{8330 * 75 * \sin(2 * 39.8^\circ)}\right)^{1/4} = 1587$$

$$\alpha_L = \pi\left(\frac{4E_f I_b l}{E_m t_e \sin 2\theta}\right)^{1/4} = \pi\left(\frac{4 * 25000 * 2.133 * 10^9 * 3600}{8330 * 75 * \sin(2 * 39.8^\circ)}\right)^{1/4} = 3322$$

Strut width:

$$w = \sqrt{{\alpha_h}^2 + {\alpha_L}^2} = \sqrt{(1587)^2 + (3322)^2} = 3682 \text{ mm}$$

Effective diagonal strut width w_e for the compressive resistance calculation should be taken as the least of (Cl.7.13.3.3)

$w_e = w/2 = 3682/2 = 1841$ mm

or

$w_e = l_d/4 = 4686/4 = 1172$ mm

thus

$w_e = 1172 \approx 1170$ mm

The design length of the diagonal strut l_s should be equal to (Cl.7.13.3.4.4)

$l_s = l_d - w/2 = 4686 - 3682/2 = 2845$ mm

b) Determine the maximum lateral load which the infilled frame can resist assuming that the infill wall provides the total lateral resistance.

- Diagonal strut: compression resistance (Cl.7.13.3.4.3 and Section 2.7.2)

The compression strength of the diagonal strut $P_{r\max}$ is equal to the compression strength of masonry times the effective cross-sectional area, that is,

$$P_{r\max} = (0.85 \chi \phi_m f'_m) \cdot A_e$$

where

ϕ_m = 0.6

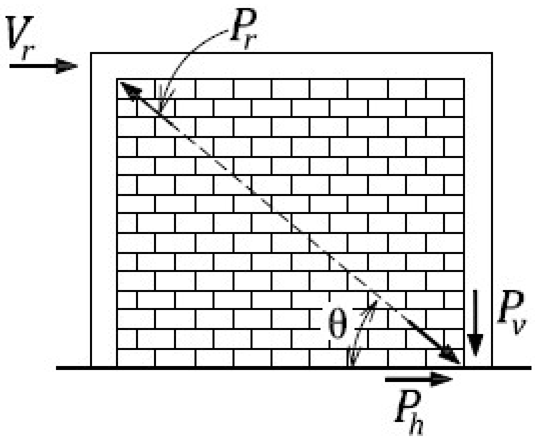
$\chi = 0.5$ the masonry compressive strength parallel to bed joints

$A_e = t_e * w_e = 75 * 1170 = 87750$ mm^2 the effective cross-sectional area

$$P_{r\max} = 0.85 * 0.5 * 0.6 * 9.8 * 87750 = 219.3 \text{ kN}$$

The corresponding lateral force is equal to the horizontal component of the strut compression force P_h, that is, (see the figure below)

$P_h = P_{r\max} * \cos(\theta) = 219.3 * \cos(39.8) = 168.0$ kN



Before proceeding with the design, slenderness effects should also be checked. First, the slenderness ratio needs to be determined as follows (S304-14 Cl.7.7.5):

$$\frac{k * l_s}{t} = \frac{1.0 * 2845}{190} = 15.0$$

where

$k = 1.0$ assume pin-pin support conditions

$l_s = 2845$ mm design length for the diagonal strut

$t = 190$ mm overall wall thickness

The strut is concentrically loaded, but the minimum eccentricity needs to be taken into account, that is,

$e_1 = e_2 = 0.1 * t = 19$ mm

Since

$\frac{k * l_s}{t} = 15.0 > 10 - 3.5 e_1 / e_2 = 6.5$ and $\frac{k * l_d}{t} < 30.0$

the slenderness effects need to be considered.

The critical axial compressive force for the diagonal strut P_{cr} will be determined according to S304-14 Cl.7.7.6.3 as follows:

$$P_{cr} = \frac{\pi^2 \phi_{er} E_m I_{eff}}{(1 + 0.5\beta_d)(kl_d)^2} = 1380 \text{ kN}$$

where

$\phi_{er} = 0.65$

$\beta_d = 0$ assume 100% seismic live load

$E_m = 8330$ MPa modulus of elasticity for masonry

$I_{eff} = 0.4 I_o = 209 * 10^6$ mm^4

where

$I_o = \dfrac{1170 * \left[190^3 - (190 - 75.4)^3\right]}{12} = 522 * 10^6$ mm^4 moment of inertia of the effective cross-sectional area based on the effective diagonal strut width $w_e = 1170$ mm and the effective wall thickness $t_e = 75.4$ mm (face shells only).

Since
$P_{r\max} = 219.3$ kN < $P_{cr} = 1380$ kN
it follows that compression failure governs over buckling failure.

- The diagonal tension shear resistance (see Section 2.3.2 and S304-14 Cl.10.10.2).

Find the masonry shear resistance (V_m):

$b_w = 190$ mm overall wall thickness

$d_v \approx 0.8 l_w = 2880$ mm effective wall depth

$\gamma_g = 0.5$ ungrouted wall

$P_d = 0$ (ignore self-weight)

$v_m = 0.16\sqrt{f'_m}$ = 0.5 MPa

$V_m = \phi_m (v_m b_w d_v + 0.25 P_d)\gamma_g$ = 0.6(0.5*190*2880+0)*0.5 ≈ 82.0 kN

This is a squat shear wall because $\dfrac{h_w}{l_w} = \dfrac{3000}{3600} = 0.83 < 1.0$. In this case, there is no need to find the maximum permitted shear resistance per S304-14 Cl.10.10.2.1 $\max V_r$ because it is not going to control for an unreinforced wall without gravity load.

- Sliding shear resistance (see Section 2.7.1 and Cl.7.10.5)

$V_{rs} = 0.16\phi_m \sqrt{f'_m} A_{uc} + \phi_m \mu P_1$

The factored in-plane sliding shear resistance V_r is determined as follows.

μ = 1.0 for a masonry-to-masonry or masonry-to-roughened concrete sliding plane

$A_{uc} = t_e \cdot d_v = 75 * 2880 = 216000$ mm^2 uncracked portion of the effective wall cross-sectional area

The compressive force in masonry acting normal to the sliding plane is normally taken as P_d plus an additional component, equal to 90% of the factored vertical component of the compressive force resulting from the diagonal strut action P_v (see the figure on the previous page).

$P_1 = P_d + 0.9 * P_v$

where

$P_v = V_{rs} * \tan(\theta)$

thus

$P_1 = 0 + 0.9 * V_{rs} \tan(\theta)$

The sliding shear resistance can be determined from the following equation

$V_{rs} = 0.16\phi_m \sqrt{f'_m} A_{uc} + \phi_m \mu (0.9 * V_{rs} \tan(\theta))$

or

$$V_{rs} = \frac{0.16\phi_m \sqrt{f'_m} A_{uc}}{1-\phi_m * \mu * 0.9 * \tan(\theta)} = \frac{0.16 * 0.6 * \sqrt{9.8} * 216000}{1-0.6*1.0*0.9*\tan(39.8^\circ)} = 118.0 \text{ kN}$$

- Discussion

It is important to consider all possible behaviour modes and identify the one that governs in this design. The following three lateral forces should be considered:

a) $P_h = 168$ kN shear force corresponding to the strut compression failure

b) $V_m = 82$ kN diagonal tension shear resistance

c) $V_{rs} = 118$ kN sliding shear resistance

It could be concluded that the diagonal tension shear resistance governs, however once diagonal tension cracking takes place, the strut mechanism forms. Therefore, the maximum shear force developed in an infill wall corresponds either to the strut compression resistance or the sliding shear resistance (see the discussion in Section 2.7.2). In this case, sliding shear resistance governs and so $V_{r\max} = V_{rs} = 118$ kN.

It should be noted that the maximum shear force developed in the infill $V_{r\max}$ will be transferred to the adjacent reinforced concrete columns, which need to be designed for shear. This is not the scope of the masonry design, however the designer should always consider the entire lateral load path and the force transfer between the structural components.

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