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مثال 2 على التصميم الزلزالي للجدران الحجرية المسلحة

Seismic Design of Reinforced Masonry Walls (Example2)

According to Canadian Concrete Masonry Producers Association CCMPA

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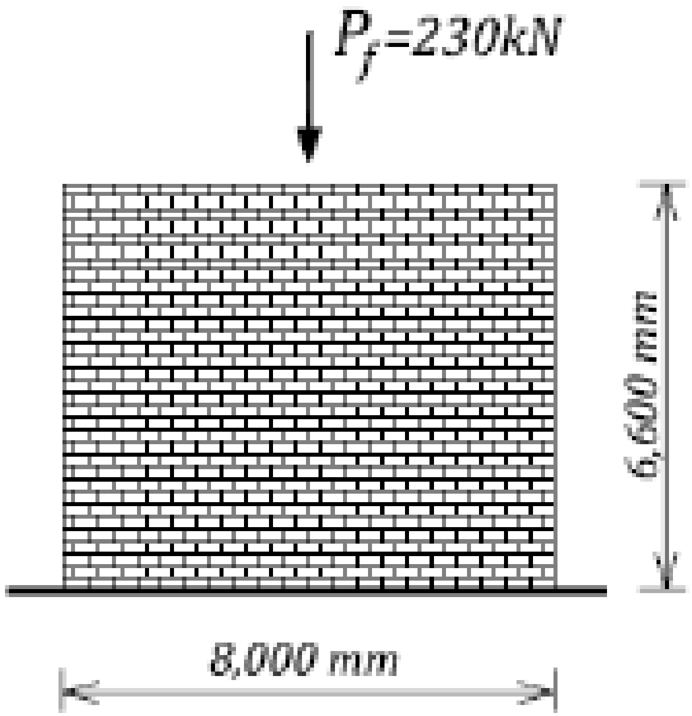
Problem 2

Minimum seismic reinforcement for a squat shear wall

Determine minimum seismic reinforcement according to CSA S304.1-04 for a loadbearing masonry shear wall located in an area with a seismic hazard index $I_E F_a S_a(0.2)$ of 0.66. 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



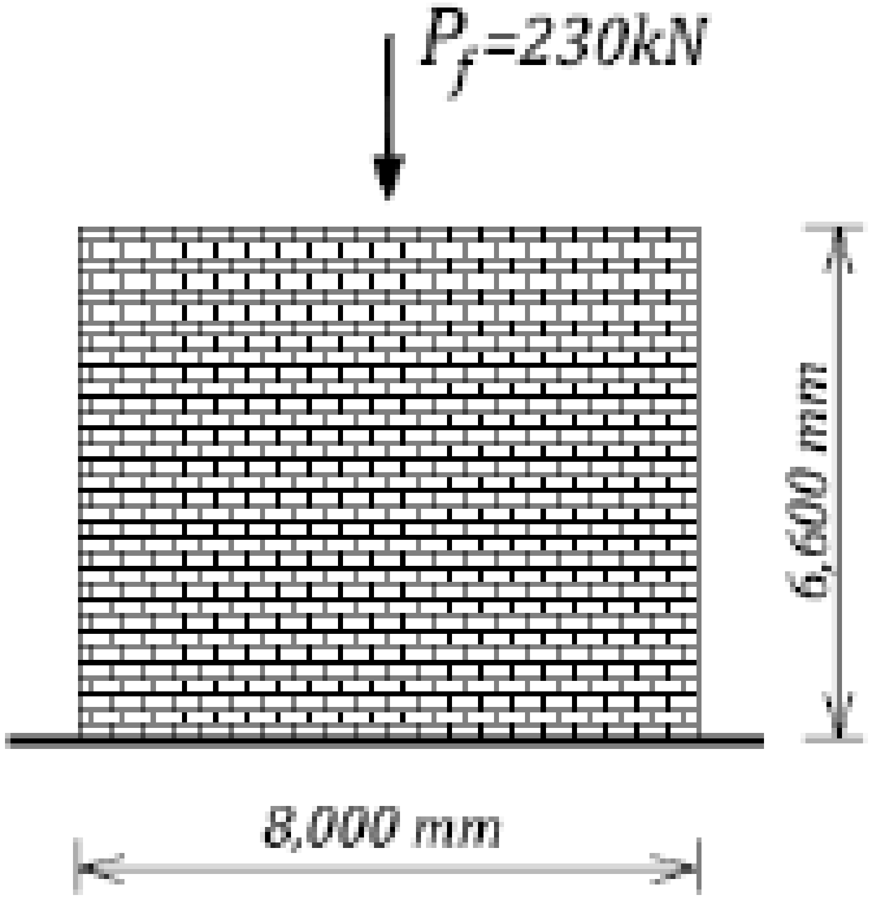
Wall dimensions:
l_w =8000 mm length
h_w = 6600 mm height
t = 190 mm thickness

Solution 2

1. Load analysis:

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

- P_f = 230 kN axial load



Wall dimensions:

l_w =8000 mm length

h_w = 6600 mm height

t = 190 mm thickness

2. Material properties:

Steel (both reinforcing bars and joint reinforcement):

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.

$\phi_s = 0.85 \quad f_y = 400\ \text{MPa}$

Masonry:

$\phi_m = 0.6$

15 MPa concrete blocks and Type S mortar. from Table 4 of S304.1-04:

f'_m **= 9.8 Mpa** (assume partially grouted masonry)

h_w = 6600 mm height, l_w= 8000 mm length, Then:

$$\frac{h_w}{l_w} = \frac{6600}{8000} = 0.825 < 1 \quad \Rightarrow \quad \boxed{\text{Squat shear wall}}$$

seismic design requirements for squat shear walls should be followed.

3. Minimum seismic reinforcement requirements

the seismic hazard index $I_E F_a S_a (0.2)$ is 0.66.

$$I_E F_a S_a(0.2) = 0.66 > 0.35$$

Thus, it is required to provide minimum seismic reinforcement

1. Seismic reinforcement area:

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

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

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 1m.

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

$$A'_{h\,min} = A'_{v\,min} = 0.00067A_g = \frac{A_{s\,min}}{3}$$

$A`_{hmin}=A`_{vmin}=380/3=127mm^2/m$

Thus the minimum total vertical reinforcement area:

$A_{vmin}= 127* l_w= (127mm^2/m)(8m)=1016mm^2$

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.

2. Vertical reinforcement (area and distribution):

spacing of vertical reinforcing bars shall not exceed the lesser of:

- $6(t+10)=6(190+10)=1200mm$
- 1200mm
- $L_w/4=8000/4=2000mm$.

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 **8T16**, so

$$A_v = 201 * 8 = 1608\ mm^2$$

$$S \leq \frac{(8000-200)}{7} = 1114.3 = 1115mm$$

The corresponding vertical reinforcement area per metre length is

$$A`_v = \frac{A_v}{l_w} * 1000 = 200mm^2/m > A`_{vmin} = 127mm^2/m$$

3. Horizontal reinforcement (area and distribution):

Considering a combination of joint reinforcement and bond beam reinforcement.

According to S304.1 Cl.10.15.2.6,

the maximum spacing of

bond beams is 2400 mm.

joint reinforcement is 400 mm.

so the following reinforcement arrangement is considered:

- 9 Ga. ladder reinforcement @ 400 mm spacing, and
- 2T16 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 16 bar is 200 mm^2. So, the total area of horizontal reinforcement per metre of wall height is

$$A`_h = \left(\frac{22.4}{400} + \frac{400}{2200}\right) * 1000 = 238 mm^2/m > A`_{hmin} = 127 mm^2/m \quad Ok$$

So, the total area of horizontal and vertical reinforcement is

$$A_s = A`_v + A`_h = 200 + 238 = 438 mm^2/m > A_{smin} = 380 mm^2/m \quad OK$$

4. Determine the flexural resistance of the wall section

Moment capacity for rectangular wall sections with distributed vertical reinforcement

$$M_r = 0.5\phi_s f_y A_w l_w \left(1 + \frac{P_f}{\phi_s f_y A_w}\right)\left(1 - \frac{c}{l_w}\right)$$

where

A_w - the total area of distributed vertical reinforcement

c - neutral axis depth

2470

4. Determine the flexural resistance of the wall section

Moment capacity for rectangular wall sections with distributed vertical reinforcement

$$\alpha_1 = 0.85 \quad \beta_1 = 0.8$$

$$\omega = \frac{\phi_s f_y A_{vt}}{\phi_m f'_m l_w t} = \frac{0.85 * 400 * 1600}{0.6 * 9.8 * 8000 * 190} = 0.061$$

$$\alpha = \frac{P_f}{\phi_m f'_m l_w t} = \frac{230 * 10^3}{0.6 * 9.8 * 8000 * 190} = 0.026$$

$$c = \frac{\omega + \alpha}{2\omega + \alpha_1 \beta_1} l_w = \frac{0.061 + 0.026}{2 * 0.061 + 0.85 * 0.8}(8000) = 868 \text{ mm}$$

$$M_r = 0.5 \phi_s f_y A_{vt} l_w \left(1 + \frac{P_f}{\phi_s f_y A_{vt}}\right)\left(1 - \frac{c}{l_w}\right) = 0.5 * 0.85 * \frac{400}{1000} * 1600 * \frac{8000}{1000}\left(1 + \frac{230 * 10^3}{0.85 * 400 * 1600}\right)\left(1 - \frac{868}{8000}\right)$$

$$M_r = 2762 \text{ kNm}$$

5. Find the diagonal tension shear resistance

Moment capacity for rectangular wall sections with distributed vertical reinforcement

Find the masonry shear resistance (V_m):

$b_w = 190$ mm overall wall thickness

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

$\gamma_g = 0.5$ partially grouted wall

$P_d = 0.9 P_f = 207$ kN

$v_m = 0.16(2 - \frac{M_f}{V_f d_v})\sqrt{f'_m} = 0.5$ MPa $\qquad \frac{M_f}{V_f d_v} = 1.0$

$V_m = \phi_m (v_m b_w d_v + 0.25 P_d)\gamma_g = 0.6(0.5*190*6400+0.25*207*10^3)*0.5 = 198$ kN

5. Find the diagonal tension shear resistance

Moment capacity for rectangular wall sections with distributed vertical reinforcement

Find the steel shear resistance (V_s):

$$V_s = 0.6\phi_s\left(\sum A_v f_y \frac{d_v}{s}\right) = 0.6 * 0.85 * 608.8 = 310 \text{ kN}$$

where the shear reinforcement includes 9 Ga. joint reinforcement spaced at 400 mm, and 2T16 bond beam reinforcement at 2200 mm spacing, and so

$$\sum A_v f_y \frac{d_v}{s} = \frac{22.4}{1000} * 400 * \frac{6400}{400} + \frac{400}{1000} * 400 * \frac{6400}{2200} = 608.8 \text{ kN}$$

The total diagonal shear resistance is equal to

$$V_r = V_m + V_s = 198 + 310 = 508 \text{ kN}$$

Maximum shear allowed on the section is

$$\max V_r = 0.4\phi_m \sqrt{f'_m} b_w d_v \gamma_g (2 - \frac{h_w}{l_w}) = 537 \text{ kN}$$

Since

$V_r < \max V_r$ OK

6. Sliding shear resistance

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

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

A_s = 1600 mm^2 total area of vertical wall reinforcement

$T_y = \phi_s A_s f_y$ = 0.85*1600*400 = 544 kN

P_d = 207 kN

$$P_2 = P_d + T_y = 207+544 = 751 \text{ kN}$$

$$V_r = \phi_m \mu P_2 = 0.6*1.0*751=451 \text{ kN}$$

7. Capacity design check

Seismic design philosophy considers that it is desirable to design structural members such that the more ductile flexural failure takes place before the more brittle shear failure has been initiated. This is known as capacity design approach.

In this case, the factored moment resistance is equal to

$M_r = 2762 \ \text{kNm}$

The nominal moment resistance can be estimated as follows

$$M_n = \frac{M_r}{\phi_s} = \frac{2762}{0.85} = 3249 \ \text{kNm}$$

Shear force at the top of the wall that would cause the overturning moment equal to M_n is equal to

$$V_{nb} = \frac{M_n}{h_w} = \frac{3249}{6.6} = 492 \ \text{kN}$$

7. Capacity design check

To ensure that flexural failure takes place before the diagonal shear failure, it is required that

$$V_{mb} \leq V_r$$

Since

$$V_{mb} = 492 < V_r = 508 \text{ kN}$$

7. Design Summary