Design check of masonry wall

Virtual strip: s1

Code: **Eurocode**

EN 1996-1-1:2005+A1:2013

Load case: **Linear,(Auto) Critical**

Parametres of the masonry strip

Height of the wall strip: $h_{total} = 8.000$ m

Width of the wall strip: $b = 3000.0$ mm

Storeys: 2

Alignment: Axis

Calculation of additional bending moments: Hinged model

Analysis: Axial force-Bending-Shear

1st storey - Design calculation results

Critical combination: [1.35*0.85*self-weight+1.35*0.85*dead load] {1.5*horizontal force} (1.5*0.7*var1)

Coefficient for seismic forces: **1.0**

Material: **Solid Clay Brick M2.5 G**

Design compressive strength: $f_d = f_k / \gamma_M = 4/2.000 = 2$ N/mm²

Wall thickness: $t = 250.0$ mm

Storey height: $\Sigma H = 4.000$ m

Clear height of the wall: $h = 4.000$ m

Buckling length: $h_{ef} = \rho_n \cdot h = 1.000 \cdot 4.000 = 4.000$ m

Maximum value for relative eccentricity: $e_{rel,max} = 0.400$

Basic data for the calculation of additional bending moment:

Storey above: $N_{Top} = -348.778$ kN; $\Delta e_{Axis} = 0$ mm

Slab reaction: $N_{Slab} = -243.321$ kN; $e_{Slab} = 0$ mm

A) Result summary:

Critical section: **Mid cross-section - m_v**

Utilization: $\eta_{max} = 0.870$ **passed!**

Result of the sections

Top cross-section - 1_v

Critical check: $N - M_y - M_z$

Utilization: $\eta_{max} = 0.564$ **passed!**

Mid cross-section - m_v

Critical check: $N - M_y - M_z$

Utilization: $\eta_{max} = 0.870$ **passed!**

Bottom cross-section - 2_v

Critical check: $N - M_y - M_z$

Utilization: $\eta_{max} = 0.869$ **passed!**

B) Detailed results:

Top cross-section - 1_v

Relative position of the reference cross-section: 4.000 m

Design internal forces:

$N_{1d} = -592.098$ kN

$V_{y,1} = 62.737$ kN

$M_{y,1} = 0$ kNm; $\Delta M_{y1} = 0$ kNm

$\Sigma M_{1d,y} = M_{y,1} + \Delta M_{y1} = 0 + 0 = 0$ kNm

$M_{z,1} = 127.046$ kNm

Calculation of design eccentricity:

Eccentricities perpendicular to the plane of the wall:

Effective thickness: $t_{eff} = 250.0$ mm

Initial eccentricity: $e_{init,z} = h_{ef}/450 = 4.000/450 = 8.9$ mm

Minimum eccentricity: $e_{min} = 0.05 \cdot t_{eff} = 0.05 \cdot 250.0 = 12.5$ mm

Eccentricity of axial force relative to the axis of the wall:

$e_{0,z} = \Sigma M_{1d,y}/N_{1d} = 0/(-592.098) = 0$ mm

Eccentricity of axial force relative to the axis of the effective wall thickness:

$e_{0,1,z} = e_{0,z} - \Delta_z = 0 - 0 = 0$ mm

Design eccentricity:

$e_{1,z} = \max(e_{0,1,z} + e_{init,z}, e_{min}) = \max(0 + 8.9, 12.5) = 12.5$ mm

Relative eccentricity: $e_{rel,1,z} = |e_{1,z}|/t_{eff} = |12.5|/250.0 = 0.050$

Eccentricities in the plane of the wall:

Initial eccentricity: $e_{init,y} = 0.5$ mm

2nd order eccentricity in local y direction: $e_{\Delta 2,y} = 0$ mm

$e_{0,y} = -1 \cdot M_{z,1}/N_{1d} = -1 \cdot 127.046/(-592.098) = 214.6$ mm

Design eccentricity:

$e_{1,y} = e_{0,y} + e_{init,y} + e_{\Delta 2,y} = 214.6 + 0.5 + 0 = 215.0$ mm

Relative eccentricity: $e_{rel,1,y} = |e_{1,y}|/b = |215.0|/3000.0 = 0.072$

Stability check : $(N - M_y - M_z)$

Load-bearing capacity:

$$\Phi_{1,y} = 1 - 2 \cdot \frac{|e_{1,z}|}{t_{eff}} = 1 - 2 \cdot \frac{|12.5|}{250.0} = 0.900$$

$$\Phi_{1,z} = 1 - 2 \cdot \frac{|e_{1,y}|}{b} = 1 - 2 \cdot \frac{|215.0|}{3000.0} = 0.857$$

$N_{Rd,1} = \Phi_{1,y} \cdot \Phi_{1,z} \cdot t_{eff} \cdot f_d \cdot b = 0.900 \cdot 0.857 \cdot 250.0 \cdot 2 \cdot 3000.0 = 1049.502$ kN

$N_{Rd,1} \geq N_{1d} \rightarrow$ **passed!**

Utilization: $\eta_1 = |N_{1d}|/N_{Rd,1} = |(-592.098)|/1049.502 = 0.564$

Shear check : interaction check - $V(N-M_y-M_z)$

Relative eccentricity: $e_{rel,1,y} = 0.072$

Length of the compressed part of the wall: $l_c = \Phi_{1,z} \cdot b = 0.857 \cdot 3000.0 = 2569.9$ mm

Reduced wall thickness: $t_{nom} = \Phi_{1,y} \cdot t_{eff} = 0.900 \cdot 250.0 = 225.0$ mm

Design compressive stress: $\sigma_d = \dfrac{|N_{1d}|}{l_c \cdot t_{nom}} = \dfrac{|(-592.098)|}{2569.9 \cdot 225.0} = 1$ N/mm²

Design shear strength:

Mortar joint: Filled joints

$f_{vk} = \min(f_{vk0} + 0.4 \cdot \sigma_d, 0.065 \cdot f_b) = \min(0 + 0.4 \cdot 1, 0.065 \cdot 10) = 1$ N/mm²

$f_{vd} = f_{vk} / \gamma_M = 1/2.000 = 0$ N/mm²

Load-bearing capacity:

$V_{Rd,1v} = f_{vd} \cdot t_{nom} \cdot l_c = 0 \cdot 225.0 \cdot 2569.9 = 176.243$ kN

$V_{Rd,1v} \geq V_{1d,y} \rightarrow$ **passed!**

Utilization: $\eta_1 = |V_{1d,y}| / V_{Rd,1v} = |62.737| / 176.243 = 0.356$

Mid cross-section - m_v

Relative position of the reference cross-section: 2.000 m

Design internal forces:

$N_{md} = -627.079$ kN

$V_{y,m} = 63.073$ kN

$M_{y,m} = 0$ kNm; $\Delta M_{ym} = 0$ kNm

$\Sigma M_{md,y} = M_{y,m} + \Delta M_{ym} = 0 + 0 = 0$ kNm

$M_{z,m} = 252.011$ kNm

Calculation of design eccentricity:

Eccentricities perpendicular to the plane of the wall:

Initial eccentricity: $e_{init,z} = h_{ef} / 450 = 4.000/450 = 8.9$ mm

Minimum eccentricity: $e_{min} = 0.05 \cdot t = 0.05 \cdot 250.0 = 12.5$ mm

Eccentricity of axial force relative to the axis of the wall:

$e_{0,m,z} = \Sigma M_{md,y} / N_{md} = 0/(-627.079) = 0$ mm

Eccentricity due to loads:

$e_{m,z} = e_{0,m,z} + e_{init,z} = 0 + 8.9 = 8.9$ mm

Slenderness: $\lambda = \dfrac{h_{ef}}{t} = \dfrac{4.000}{250.0} = 16.000$

$\lambda \geq \lambda_C \rightarrow$ Eccentricity due to creep: $e_k = 0.002 \cdot \phi_\infty \cdot \dfrac{h_{ef}}{t} \cdot \sqrt{t \cdot e_{m,z}} = 0.002 \cdot 1.000 \cdot \dfrac{4.000}{250.0} \cdot \sqrt{250.0 \cdot 8.9} = 1.5$ mm

Design eccentricity:

$e_{mk} = \max(e_{m,z} + e_k, e_{min}) = \max(8.9 + 1.5, 12.5) = 12.5$ mm

Relative eccentricity: $e_{rel,m,z} = |e_{mk}| / t = |12.5| / 250.0 = 0.050$

Eccentricities in the plane of the wall:

Initial eccentricity: $e_{init,y}$ = 1.0 mm

2nd order eccentricity in local y direction: $e_{\Delta 2,y}$ = 0 mm

$$e_{0,y} = -1 \cdot M_{z,m} / N_{md} = -1 \cdot 252.011 / (-627.079) = 401.9 \text{ mm}$$

Design eccentricity:

$$e_{m,y} = e_{0,y} + e_{init,y} + e_{\Delta 2,y} = 401.9 + 1.0 + 0 = 402.9 \text{ mm}$$

Relative eccentricity: $e_{rel,m,y} = |e_{m,y}| / b = |402.9| / 3000.0 = 0.134$

Stability check : $(N - M_y - M_z)$

Load-bearing capacity:

$$A_1 = 1 - 2 \cdot \frac{|e_{mk}|}{t} = 1 - 2 \cdot \frac{|12.5|}{250.0} = 0.900$$

$$\lambda = \frac{h_{ef}}{t} \cdot \sqrt{\frac{f_k}{E}} = \frac{4.000}{250.0} \cdot \sqrt{\frac{4}{3630}} = 0.506$$

$$u = \frac{\lambda - 0.063}{0.73 - 1.17 \cdot |e_{mk}| / t} = \frac{0.506 - 0.063}{0.73 - 1.17 \cdot |12.5| / 250.0} = 0.660$$

$$\Phi_{m,y} = A_1 \cdot e^{\frac{-u^2}{2}} = 0.900 \cdot e^{\frac{-0.660^2}{2}} = 0.724$$

$$\Phi_{m,z} = 1 - 2 \cdot \frac{|e_{m,y}|}{b} = 1 - 2 \cdot \frac{|402.9|}{3000.0} = 0.731$$

$$N_{Rd,m} = \Phi_{m,y} \cdot \Phi_{m,z} \cdot t \cdot f_d \cdot b = 0.724 \cdot 0.731 \cdot 250.0 \cdot 2 \cdot 3000.0 = 720.848 \text{ kN}$$

$N_{Rd,m} \geq N_{md} \rightarrow$ **passed!**

Utilization: $\eta_m = |N_{md}| / N_{Rd,m} = |(-627.079)| / 720.848 = 0.870$

Shear check : interaction check - $V(N - M_y - M_z)$

Relative eccentricity: $e_{rel,m,y}$ = 0.134

Length of the compressed part of the wall: $l_c = \Phi_{m,z} \cdot b = 0.731 \cdot 3000.0 = 2194.2$ mm

Reduced wall thickness: $t_{nom} = \Phi_{m,y} \cdot t_{eff} = 0.724 \cdot 250.0 = 181.0$ mm

Design compressive stress: $\sigma_d = \dfrac{|N_{md}|}{l_c \cdot t_{nom}} = \dfrac{|(-627.079)|}{2194.2 \cdot 181.0} = 2$ N/mm²

Design shear strength:

Mortar joint: Filled joints

$f_{vk} = \min(f_{vk0} + 0.4 \cdot \sigma_d, 0.065 \cdot f_b) = \min(0 + 0.4 \cdot 2, 0.065 \cdot 10) = 1$ N/mm²

$f_{vd} = f_{vk} / \gamma_M = 1 / 2.000 = 0$ N/mm²

Load-bearing capacity:

$$V_{Rd,mv} = f_{vd} \cdot t_{nom} \cdot l_c = 0 \cdot 181.0 \cdot 2194.2 = 129.078 \text{ kN}$$

$V_{Rd,mv} \geq V_{md,y} \rightarrow$ **passed!**

Utilization: $\eta_m = |V_{md,y}| / V_{Rd,mv} = |63.073| / 129.078 = 0.489$

Bottom cross-section - 2_v

Relative position of the reference cross-section: 0 m

Design internal forces:

$N_{2d} = -657.143$ kN

$V_{y,2} = 60.457$ kN

$M_{y,2} = 0$ kNm; $\Delta M_{y2} = 0$ kNm

$\Sigma M_{2d,y} = M_{y,2} + \Delta M_{y2} = 0 + 0 = 0$ kNm

$M_{z,2} = 376.247$ kNm

Calculation of design eccentricity:

Eccentricities perpendicular to the plane of the wall:

Effective thickness: $t_{eff} = 250.0$ mm

Initial eccentricity: $e_{init,z} = h_{ef}/450 = 4.000/450 = 8.9$ mm

Minimum eccentricity: $e_{min} = 0.05 \cdot t_{eff} = 0.05 \cdot 250.0 = 12.5$ mm

Eccentricity of axial force relative to the axis of the wall:

$e_{0,z} = \Sigma M_{2d,y}/N_{2d} = 0/(-657.143) = 0$ mm

Eccentricity of axial force relative to the axis of the effective wall thickness:

$e_{0,2,z} = e_{0,z} - \Delta_z = 0 - 0 = 0$ mm

Design eccentricity:

$e_{2,z} = \max(e_{0,2,z} + e_{init,z}, e_{min}) = \max(0 + 8.9, 12.5) = 12.5$ mm

Relative eccentricity: $e_{rel,2,z} = |e_{2,z}|/t_{eff} = |12.5|/250.0 = 0.050$

Eccentricities in the plane of the wall:

Initial eccentricity: $e_{init,y} = 1.5$ mm

2nd order eccentricity in local y direction: $e_{\Delta 2,y} = 0$ mm

$e_{0,y} = -1 \cdot M_{z,2}/N_{2d} = -1 \cdot 376.247/(-657.143) = 572.5$ mm

Design eccentricity:

$e_{2,y} = e_{0,y} + e_{init,y} + e_{\Delta 2,y} = 572.5 + 1.5 + 0 = 574.1$ mm

Relative eccentricity: $e_{rel,2,y} = |e_{2,y}|/b = |574.1|/3000.0 = 0.191$

Stability check : $(N - M_y - M_z)$

Load-bearing capacity:

$$\Phi_{2,y} = 1 - 2 \cdot \frac{|e_{2,z}|}{t_{eff}} = 1 - 2 \cdot \frac{|12.5|}{250.0} = 0.900$$

$$\Phi_{2,z} = 1 - 2 \cdot \frac{|e_{2,y}|}{b} = 1 - 2 \cdot \frac{|574.1|}{3000.0} = 0.617$$

$N_{Rd,2} = \Phi_{2,y} \cdot \Phi_{2,z} \cdot t_{eff} \cdot f_d \cdot b = 0.900 \cdot 0.617 \cdot 250.0 \cdot 2 \cdot 3000.0 = 756.250$ kN

$N_{Rd,2} \geq N_{2d} \rightarrow$ **passed!**

Utilization: $\eta_2 = |N_{2d}|/N_{Rd,2} = |(-657.143)|/756.250 = 0.869$

Shear check : interaction check - $V(N - M_y - M_z)$

Relative eccentricity: $e_{rel,2,y} = 0.191$

Length of the compressed part of the wall: $l_c = \Phi_{2,z} \cdot b = 0.617 \cdot 3000.0 = 1851.9$ mm

Reduced wall thickness: $t_{nom} = \Phi_{2,y} \cdot t_{eff} = 0.900 \cdot 250.0 = 225.0$ mm

Design compressive stress: $\sigma_d = \dfrac{|N_{2d}|}{l_c \cdot t_{nom}} = \dfrac{|(-657.143)|}{1851.9 \cdot 225.0} = 2$ N/mm²

Design shear strength:

Mortar joint: Filled joints

$f_{vk} = \min(f_{vk0} + 0.4 \cdot \sigma_d,\ 0.065 \cdot f_b) = \min(0 + 0.4 \cdot 2,\ 0.065 \cdot 10) = 1$ N/mm²

$f_{vd} = f_{vk} / \gamma_M = 1 / 2.000 = 0$ N/mm²

Load-bearing capacity:

$V_{Rd,2v} = f_{vd} \cdot t_{nom} \cdot l_c = 0 \cdot 225.0 \cdot 1851.9 = 135.417$ kN

$V_{Rd,2v} \geq V_{2d,y} \rightarrow$ **passed!**

Utilization: $\eta_2 = |V_{2d,y}| / V_{Rd,2v} = |60.457| / 135.417 = 0.446$