Example of design of a drystack masonry structure

Internal wall check

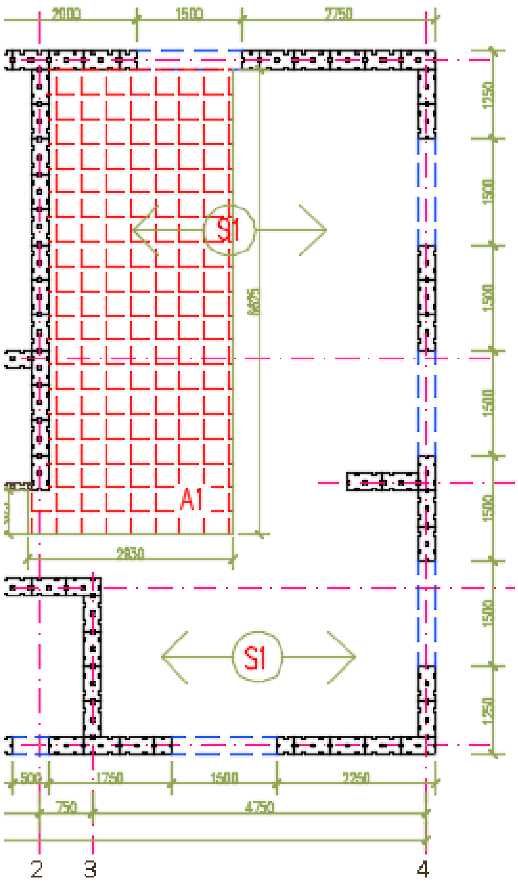
1. Dimension of slab for span 5500 mm

$hs = (\frac{1}{30} \approx \frac{1}{25})l$ therefore hs is considered **200 mm**

By project documentation in the project is used hollow slab “panel Spirol 200” by manufacturer GOLDBECK. Self weight then is specified equal to 258 kg/m^2

2. Area of load

A1=2.75 m * 6.75 m = **18.56 m^2**



3. Calculation of loads

Permanent Load	g_k [kN/m^2]	γ [-]	g_d [kN/m^2]
Self-weight of slab	2.53	1.35	3.42
Roof construction	0.59	1.35	0.8
			∑4.22

Variable Load	q_k [kN/m^2]	γ [-]	q_d [kN/m^2]
Snow load	0.75	1.5	1.13
			∑1.13

$g_{(\text{Self weight of the wall STAVSI})k}$ = V * ρ =0.25 m * 6 m * 3.0 m * 920 kg/m^3=4140 kg = 41.4 kN

g_d = 41.4 kN * 1.35 = **55.89 kN**

N_{ed} = ∑(g_d + q_d)*A+ g_d (s-weight of wall) = (4.22 + 1.13) kN/m^2 *18.56 m^2 +55.89 kN = **155.19 kN**

4. **f_u = 6 MPa**
5. **Strength of brick**

   $f_b = \delta * f_u$

   (by EC 1996-1-1: δ – form factor, related to the brick dimensions)

   For block 500 x 250 x 200 mm δ = 1.1

   f_b = 1.1 * 6 MPa = **6.6 MPa**
6. **Characteristic compressive strength of unreinforced masonry using thin layer mortar (0.5 mm < thickness < 0.3 mm)**

   Use of formula (3.3) for thin mortar masonry

   $f_k = K * f_b^{0.85}$, where K is constant from table 3.3 EC 1996-1-1 and equals 0.8 (masonry unit group 1, thin layer mortar)

   f_k = 0.8 * 6.6 MPa $^{0.85}$ = **5.28 MPa**
7. **Design compressive strength of the masonry**

   $f_d = f_k / \gamma_M$,

   γ_M = 2.0 (from table 5.10)

   f_d=5.28 MPa / 2.0 = **2.64 MPa**
8. **Required area on the footing of the wall. (by estimation)**

   A_{req} = N_{max} / 0.7* f_d, where **N_{max} is load without self-weight = 99.3 kN**

   A_{req} = 99.3 kN / (0.7 * 2.64 MPa) = **0.054** m^2

   Conclusion: wall width satisfies designed dimensions of the block units: 500 x 250 x 200 mm, with minimum foot area A_{eff} = 0.250 m^2

   Hence design force F_d = Aeff * 0.7 * f_d = 0.25 m^2 * 0.7 * 2.64 MPa = **462 kN**

9. **Slenderness ratio**

h_{ef} / t_{ef} $\leq$ 27, where h_{ef} = ρ_n * h_p (h_p – clear height of wall and ρ_n – 0.75 for walls restrained at both ends); t_{ef} = min (b;t). Hence h_{ef} = 3.0 m * 0.75=2.25 m; t_{ef} = min(500; 250) = 250mm

2.25/0.25 = 9 $\leq$ 27 (not slender)

10. **Check of cross sections**

A – top of the wall – eccentric load, high vertical load ($N_{ed,A}$ = N_{max})

B – effect of slenderness, possibility of buckling ($N_{ed,B}$ = N_{max} + ½ s-weight of the wall)

- **Vertical resistance A**

Eccentricity of design vertical load

Assume $N_{ed,A}$ = **N_{max} = 99.3 kN**

$N_{rd,i}$ = Φ_i * A * f_d , where Φm is capacity reduction factor.

Φi = 1 – 2 e_i / t, where e_i = e_{if} + e_{ia} + e_{ia} $\geq$ 0.05*t is total eccentricity at the top of the wall

e_{if} – eccentricity due to loads (M_{id} / N_{id} = 0)

e_{fm} – eccentricity due to horizontal load is 0

e_{ia} – initial eccentricity = h_{ef} / 450 = 2.25 m/450 = 0.005 m (5 mm)

e_i = 0 + 0 + e_k $\geq$ 0.05*t

e_i is 0.05 * t at the top and bottom of the wall, which are the minimum eccentricity design values to be used.

Therefore Φi = 1 – 2 e_i / t equals

Φi=1 – 2 * 0.05*t / t = **0.9**

$N_{rd,i}$ = Φ_i * A * f_d = 0.9 * 0.25m^2 * 2.64 MPa = 594 kN > $N_{Ed,A}$ = 99.3 kN

Conclusion: The wall **satisfies** the requirements for total eccentricity at the top and the bottom of the wall.

- **Vertical resistance B**

Eccentricity of design vertical load

Assume $N_{ed,B}$ = **N_{max} + ½ of the s-weight = 127.24 kN**

$N_{rd,m}$ = Φ_i * A * f_d , where Φm is capacity reduction factor (table meaning)

e_{mk}/t = e_{mf}/t + e_{ma}/t + e_k/t $\geq$ 0.05 is relative eccentricity at the middle of the wall

e_{mf} – eccentricity due to loads (M_{id} / N_{id} = 0)

e_{ma} – initial eccentricity h_{ef} / 450 = 2.25 m/450 = 0.005 m (5 mm)

e_k – eccentricity due to creep = 0

0.05 is the minimum relative eccentricity.

$e_{mk}/t = (0 + 0.005 + 0)/t \geq 0.05$

e_{mk}/t is 0.05 t at the middle of the wall, which is the minimum eccentricity design value to be used. Hence by table $h_{ef} / t_{ef} = 9$; $e_{mk} = 0.05$

Therefore Φi = **0.85**

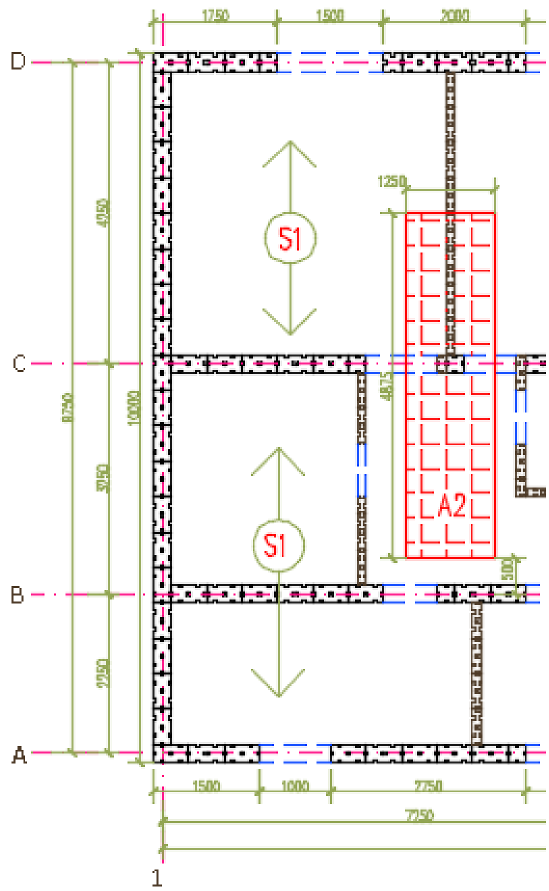
$N_{rd,m} = \Phi_i * A * f_d = 0.85 * 0.25m^2 * 2.64$ MPa = 561 kN > $N_{Ed,B}$ = 127.24 kN

Conclusion: The wall **satisfies** the requirements for relative eccentricity at the middle of the wall.

Internal column check

1. Area of load

A=3.875 m * 1.25 m = **4.84 m²**



2. Calculation of loads

Permanent Load	g_k [kN/m²]	γ [-]	g_d [kN/m²]
Self-weight of slab	2.53	1.35	3.42
Roof construction	0.59	1.35	0.8

			∑4.22
Variable Load	q_k [kN/m^2]	γ [-]	q_d [kN/m^2]
Snow load	0.75	1.5	1.13
			∑1.13

$g_{(\text{Self weight of the pillar STAVSI})k}$ = V * ρ =0.25 m * 0.25 m * 3.0 m * 920 kg/m^3=173 kg = 1.73 kN

g_d = 1.73 kN * 1.35 = **2.34 kN**

N_{ed} = ∑(g_d + q_d)*A+s-weight of column = (4.22 + 1.13) kN/m^2 *4.84 m^2 + 2.34= **28.23 kN**

3. **f_u = 6 MPa**
4. **Strength of brick**

   $f_b = \delta * f_u$

   (by EC 1996-1-1: δ – form factor, related to the brick dimensions)

   For block 250 x 250 x 200 mm δ = 1.1

   f_b = 1.1 * 6 MPa = **6.6 MPa**
5. **Characteristic compressive strength of unreinforced masonry using thin layer mortar (0.5 mm < thickness < 0.3 mm)**

   Use of formula (3.3) for thin mortar masonry

   $f_k = K * f_b^{0.85}$, where K is constant from table 3.3 EC 1996-1-1 and equals 0.8 (masonry unit group 1, thin layer mortar)

   f_k = 0.8 * 6.6 MPa $^{0.85}$ = **5.28 MPa**
6. **Design compressive strength of the masonry**

   $f_d = f_k / \gamma_{M,}$

   γ_M = 2.0 (from table 5.10)

   f_d=5.28 MPa / 2.0 = **2.64 MPa**
7. **Required area on the footing of the wall. (by estimation)**

   A_{req} = N_{max} / 0.7* f_d, where **N_{max} is load without self-weight = 25.9 kN**

   A_{req} = 25.9 kN / (0.7 * 2.64 MPa) = **0.014** m^2

   Conclusion: wall width satisfies designed dimensions of the block units: 250 x 250 x 200 mm, with minimum foot area A_{eff} = 0.063 m^2

Hence design force F_d = Aeff * 0.7 * f_d = 0.014 m^2 * 0.7 * 2.64 MPa = **116.42 kN**

8. Slenderness ratio

h_{ef} / t_{ef} $\leq$ 27, where h_{ef} = ρ_n * h_p (h_p – clear height of wall and ρ_n – 0.75 for walls and pillars restrained at both ends); t_{ef} = min (b;t). Hence h_{ef} = 3.0 m * 0.75=2.25 m; t_{ef} = min(250; 250) = 250mm

2.25/0.25 = 9 $\leq$ 27 (not slender)

9. Check of cross sections

A – top of the wall – eccentric load, high vertical load ($N_{ed,A}$ = N_{max})

B – effect of slenderness, possibility of buckling ($N_{ed,B}$ = N_{max} + ½ s-weight of the pillar)

- **Vertical resistance A**

Eccentricity of design vertical load

Assume $N_{ed,A}$ = **N_{max} = 25.9 kN**

$N_{rd,i}$ = Φ_i * A * f_d , where Φm is capacity reduction factor.

Φi = 1 – 2 e_i / t, where e_i = e_{if} + e_{ia} + e_{ia} $\geq$ 0.05*t is total eccentricity at the top of the wall

e_{if} – eccentricity due to loads (M_{id} / N_{id} = 0)

e_{fm} – eccentricity due to horizontal load is 0

e_{ia} – initial eccentricity = h_{ef} / 450 = 2.25 m/450 = 0.005 m (5 mm)

e_i = 0 + 0 + e_k $\geq$ 0.05*t

e_i is 0.05 * t at the top and bottom of the wall, which are the minimum eccentricity design values to be used.

Therefore Φi = 1 – 2 e_i / t equals

Φi=1 – 2 * 0.05*t / t = **0.9**

$N_{rd,i}$ = Φ_i * A * f_d = 0.9 * 0.063m^2 * 2.64 MPa = 149.7 kN > $N_{Ed,A}$ = 25.9 kN

Conclusion: The pillar **satisfies** the requirements for total eccentricity at the top and the bottom of the wall.

- **Vertical resistance B**

Eccentricity of design vertical load

Assume $N_{ed,B}$ = **N_{max} + ½ of the s-weight = 27.06 kN**

$N_{rd,m}$ = Φ_i * A * f_d , where Φm is capacity reduction factor (table meaning)

e_{mk}/t = e_{mf}/t + e_{ma}/t + e_k/t $\geq$ 0.05 is relative eccentricity at the middle of the wall

e_{mf} – eccentricity due to loads (M_{id} / N_{id} = 0)

e_{ma} – initial eccentricity h_{ef} / 450 = 2.25 m/450 = 0.005 m (5 mm)

e_k – eccentricity due to creep = 0

0.05 is the minimum relative eccentricity.

$e_{mk}/t = (0 + 0.005 + 0)/t \geq 0.05$

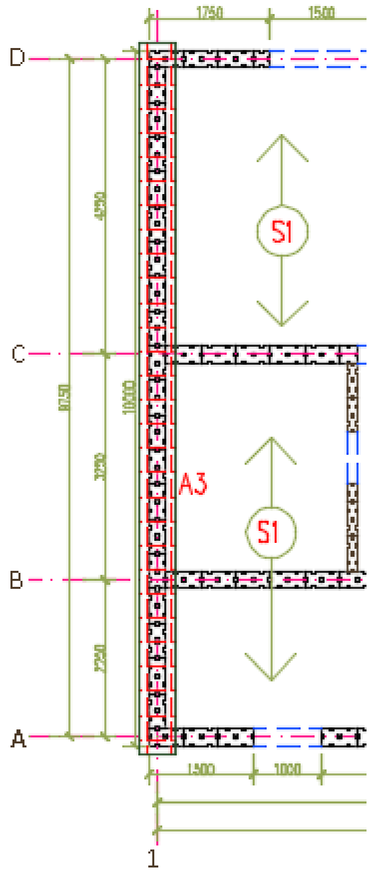
e_{mk}/t is 0.05 t at the middle of the wall, which is the minimum eccentricity design value to be used. Hence by table h_{ef} / t_{ef} = 9; e_{mk} = 0.05

Therefore Φi = **0.85**

$N_{rd,m} = \Phi_i * A * f_d$ = 0.85 * 0.063m^2 * 2.64 MPa = 141.37 kN > $N_{Ed,B}$ = 27.06 kN

Conclusion: The wall **satisfies** the requirements for relative eccentricity at the middle of the wall.

External non-load bearing wall exposed to wind load check A3



Non load bearing masonry is laterally loaded, vertical load is not sufficient to avoid possibility of flexural failure. The wall is considered as two-way slab.

1. **Characteristic wind load w_k [kN/m^2]**

   $w_k = q_b * c_e * c_{pe}$, where q_b – reference mean velocity pressure [kN/m^2]

q_b = ½ * ρ * v_b^2 (ρ-air density – 1.25 kg/m^3; v_b – basic wind velocity from a map in EC 1991-1-4 = 25 m/s)

hence q_b = 0.5 * 1.25 kg/m^3 * 25^2 m/s = 390 Pa (N/m^2) = **0.39 kN/m^2**

then c_e – exposure factor, by figure 4.2 of EC for III terrain category and 3.5 m height of the building =**1.3**

next c_{pe} – pressure coefficient chapter 7 of EC figure 7.5, where e = min(b or 2h) = **7** as the least of 9.25 m **-** b -crosswind dimension and 2* height. e < d. Then by table 7.1 c_{pe} is determined as crossing of h/d = 3.5 m/12.75 m = 0.27 and zone A; hence c_{pe} **= -1.2**

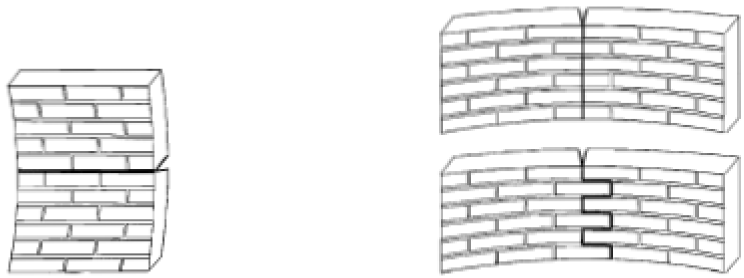
w_k = q_b * c_e * c_{pe} = 0.39 * 1.3 * (-1.2) = -0.61 [kN/m^2]

2. **Design wind load w_k [kN/m^2]**

   w_d= w_k*1.5 =0.91 [kN/m^2]

3. **Flexural strength having a plane of failure parallel to the bed joints f_{xk1}, and plane of failure perpendicular to bed joints f_{xk2}, EC 6 chapter 3.6.3.**

   For thin mortared wall STAVSI block has **f_{xk1}= 0.2 MPa (200 kN/m^2), f_{xk2}=0.3 MPa (300 kN/m^2)**



a) plane of failure parallel to bed joints, f_{xk1} b) plane of failure perpendicular to bed joints, f_{xk2}

4. **Flexural strength of masonry**

   f_{xd1} = f_{xk1}/ γ_M + σ_d

   f_{xd2} = f_{xk2}/ γ_M , where γ_M = 2.0 (from table 5.10)

   σ_d – stress from the vertical loading in the critical cross-section

   σ_d = ρ_m / 1000 * h/2 = 9.2 kN/m^3 / (1000) * 3.0 m / 2 = **13.8 kN/m^2**

   Hence

   f_{xd1} = f_{xk1}/ γ_M + σ_d = 200 kN/m^2 / 2 + 13.8 kN/m^2 = **113.8 kN/m^2**

   f_{xd2} = f_{xk2}/ γ_M = 300 kN/m^2 / 2 = **150 kN/m^2**

5. **Design bending moment**

- **Plane of failure parallel to the bed joints**

  $M_{Ed,x}$= α*w_db*L^2,** there we calculate μ is orthogonal ratio of flexural strength

μ= f_{xd1}/ f_{xd2}=0.113/0.15**=0.76**. Next from the chapter 5.5.5 α (bending moment coefficient) can be obtained and equal to **0.01** (h/L = 3.0/9.25 =0.33; and μ=0.67)

Wall support conditions E (simply supported on 4 edges)

$M_{Ed,x}$= α*w_db*L^2** =0.01*1 m*0.91 [kN/m^2]*(9.25 m)^2 = **0.78 kNm**

- **Plane of failure perpendicular to the bed joints**

  $M_{Ed,y}$= μ*α*w_db*L^2 =** 0.76*0.01*1 m*0.91 [kN/m^2]*(9.25 m)^2 = **0.59 kNm**

6. Resistance moment

$M_{Rd,x}$ = f_{xd1}*z, where z-elastic section modulus per 1 meter of the wall [m^3/m] and

$M_{Rd,y}$ = f_{xd2}*z

Z=b*t^2/6=1 m*0.25^2 m^2/6 = 0.01 m^3

$M_{Rd,x}$ = f_{xd1}*z =113.8 kN/m^2 * 0.01 m^3= **1.138 kNm > $M_{Ed,x}$= 0.78 kNm**

$M_{Rd,y}$ = f_{xd2}*z=150 kN/m^2 * 0.01 m^3= **1.5 kNm > $M_{Ed,y}$= 0.59 kNm**

Conclusion: the wall thickness is **adequate,** it can resist flexural forces **sufficiently**