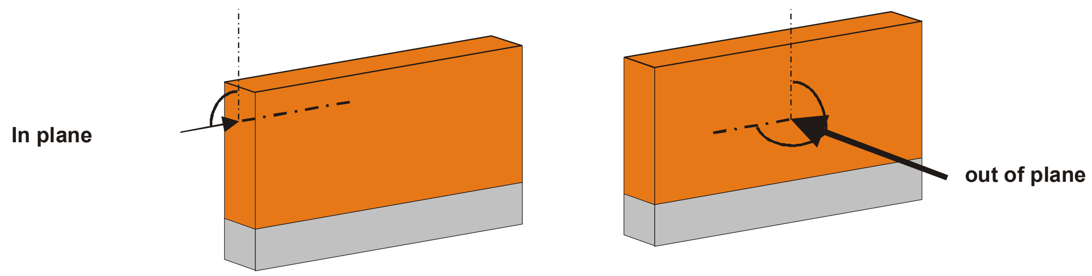
# Unreinforced masonry – shear loading

## 1 Masonry members under shear loading

Types

- In plane
- Out of plane




a) In plane shear b) Out of plane shear

*Figure 1 Types of shear action*

In plane shear loading occurs in connection with the transmission of wind loads and stiffening forces of masonry buildings as well as other lateral forces in plane of walls.
Out of plane shear has to be verified in case of lateral actions perpendicular to the wall area. Typically examples are the wind action perpendicular to the wall or the pressure of earth or loos stock material.

## 2 Bracing of buildings

The layout of the walls in the ground plan of the building should be foreseen in such a way that the sufficient bracing of a building should be guaranteed.
In traditionally buildings the sufficient stiffening can be assumed. The current tendency of economically solutions in residential and office buildings lead to an open ground plan with a minimum of stiffening walls, which makes the verification of stiffening walls often necessary.
The principles of bracing are:

- an available concrete ceiling or ring beam
- more than 3 walls
- the axes should not intersect in one point

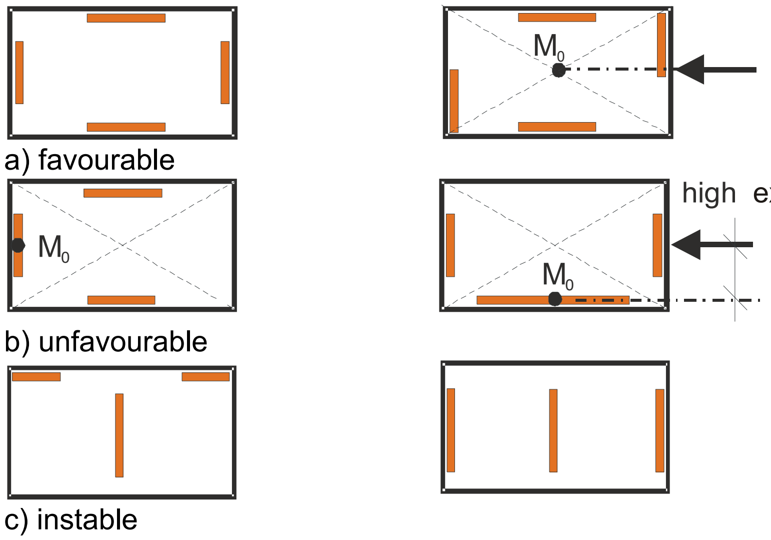



*Figure 2 Arrangement of stiffening walls*

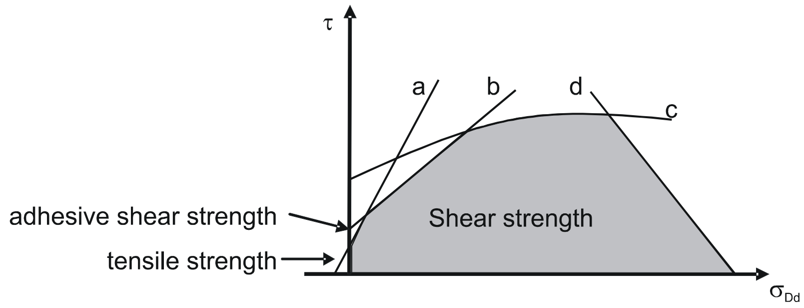
# 3 Verification format

*Table 1 Procedure according to EC 6*

| EN 1996-1-1<br>General rules | EN 1996-3<br>Simplified calculation methods |
|---|---|
| filled head joints:<br>$f_{vk} = f_{vk0} + 0{,}4 \cdot \sigma_d \le \begin{cases} 0{,}065 \cdot f_b \\ f_{vlt} \end{cases}$<br>unfilled head joints:<br>$f_{vk} = 0{,}5 \cdot f_{vk0} + 0{,}4 \cdot \sigma_d \le \begin{cases} 0{,}045 \cdot f_b \\ f_{vlt} \end{cases}$<br>shell bedded masonry:<br>$f_{vk} = \frac{g}{t} \cdot f_{vk0} + 0{,}4 \cdot \sigma_d \le \begin{cases} 0{,}045 \cdot f_b \\ f_{vlt} \end{cases}$<br>$V_{Rd} = f_{vd} \cdot t \cdot l_c$<br>with:<br>$f_{vk0}$: adhesive shear strength without load<br>$\sigma_d$: compressive stress, perpendicular with shear load<br>$f_b$: compressive strength of masonry<br>$f_{klt}$: limit of shear strength<br>$g$: overall with of mortar strips<br>$t$: thickness of the wall | $V_{Rd} = c_v \cdot \left[\frac{l}{2} - e_{Ed}\right] \cdot t \cdot f_{vdo} + 0{,}4 \cdot \frac{N_{Ed}}{\gamma_M} \le 3\left[\frac{l}{2} - e_{Ed}\right] \cdot t \cdot f_{vdu}$<br>with:<br>$e_{Ed} = \frac{M_{Ed}}{N_{Ed}}$<br>$c_v$ = 3 filled head joints<br>$c_v$ = 1,5 unfilled head joints<br>$e_{Ed}$: excentricity of load<br>$t$: thickness of the wall<br>$f_{vd0}$= $f_{vk0}/\gamma_M$<br>$N_{Ed}$: vertical load<br>$l$: length of the wall<br>$f_{vdu}$: ultimate shear strength |
| verification | |
| $V_{Ed} \le V_{Rd}$ | |

## 3.1 Background – theory

Depending on the ratio of shear to the vertical component of compression stress failure initiated differently. The shear fracture failure of masonry is a curve illustration of four modes of failure.




Failure due to
a) gaping of bed joint
b) friction of bed joint
c) unit tension
d) masonry compression

*Figure 3 failure modes of a shear wall*

The failure of a masonry wall depends on the strength of the unit and mortar. Around the unit exist different areas of compressive stress and shear.

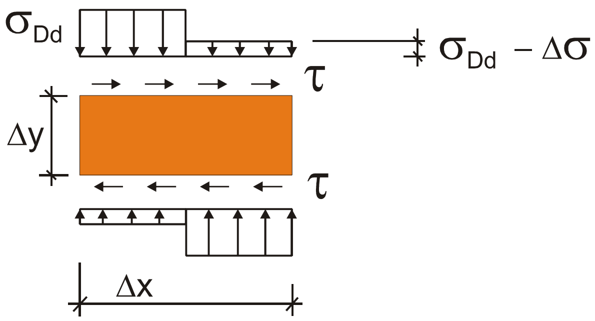



*Figure 4 equilibrium of forces at a masonry unit*

# 4 Procedure

## 4.1 In plane shear loading

Schedule
A) Actions

1. Determination of wind action
2. Determination of bracing forces
3. Distribution of forces on the stiffening walls (see Figure 5)
4. Determination of normal stresses/forces and compressed area
    a. Bending action due to in plane lateral forces and
    b. determination of the compressed area

c. Bending due to deflection of the ceilings

B) Resistance

5. Determination of shear strength and/or shear resistance

C) Verification

6. Design action ≤ Design resistance

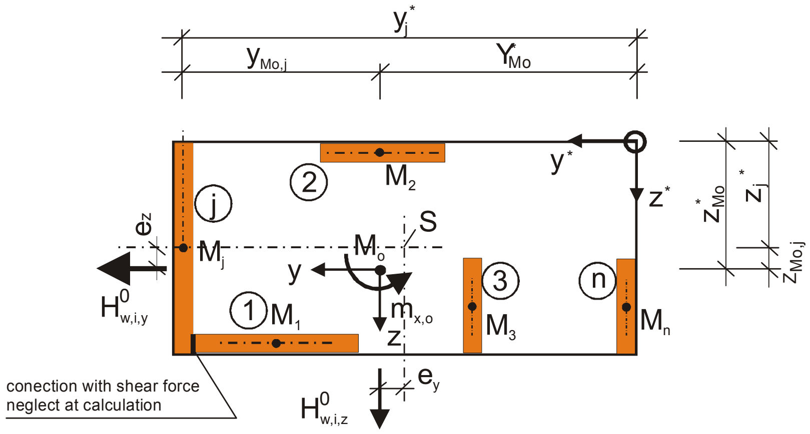



*Figure 5 Determination of the shear centre in the ground plan*

## 4.2 Out of plane shear loading

Schedule

A) Actions

1. Determination of wind action
2. Determination of normal stresses/forces and compressed area
    a. Bending action due to in plane lateral forces and
    b. determination of the compressed area

B) Resistance

3. Determination of shear strength and/or shear resistance

C) Verification

4. Design action ≤ Design resistance

# 5 Example for in plane shear

## 5.1 Building

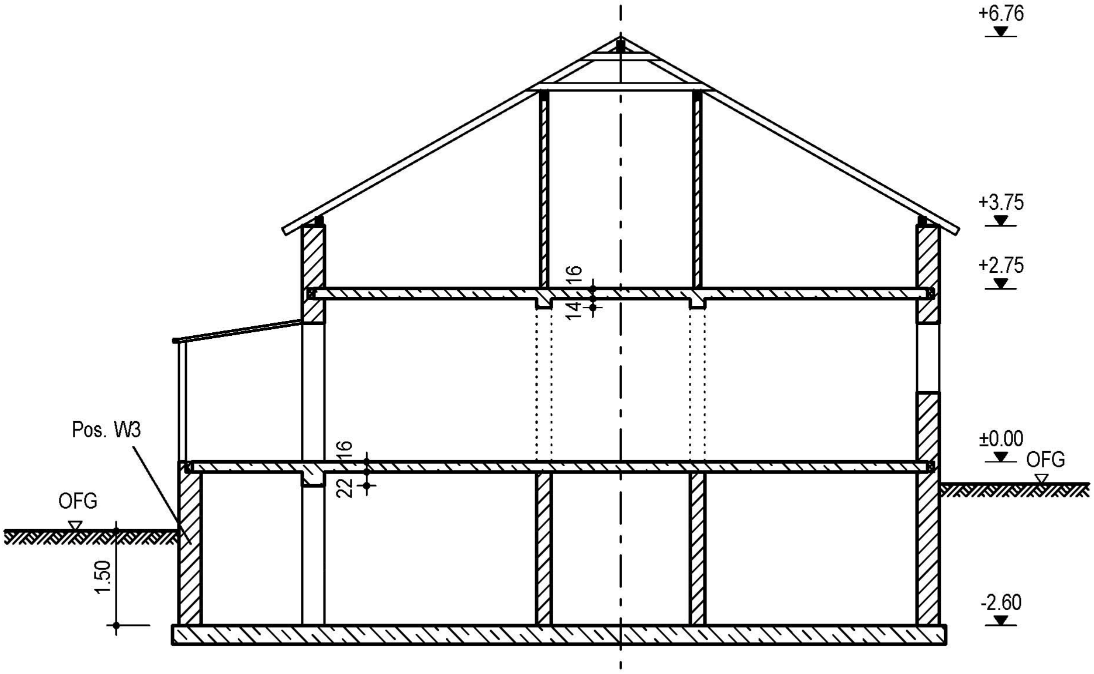



*Figure 6 longitudinal section A-A*

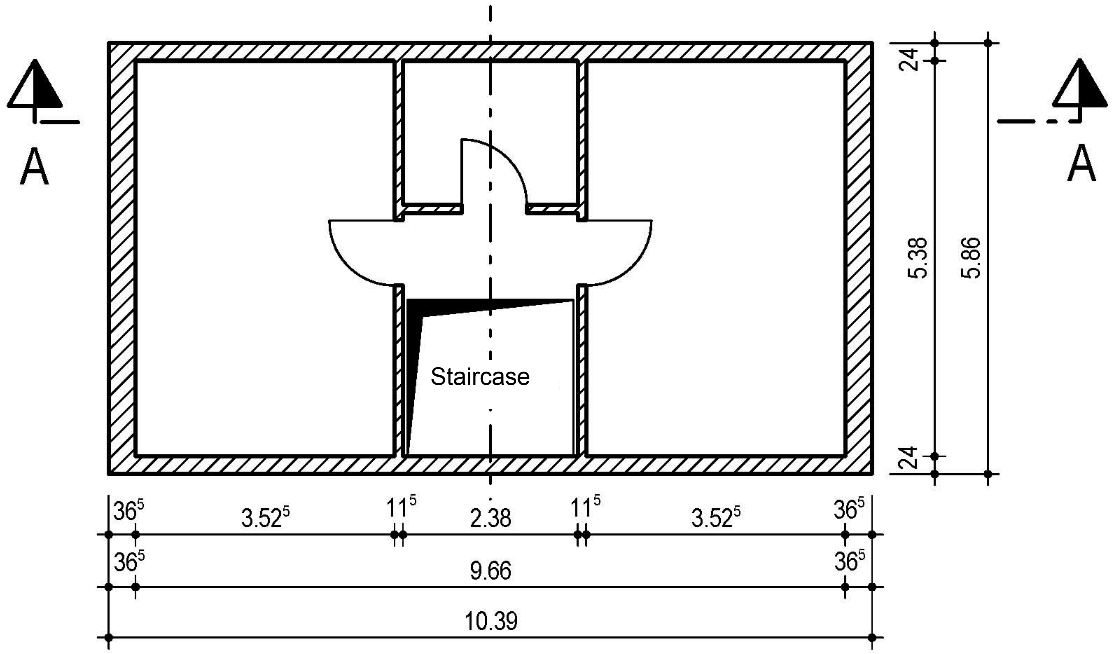



*Figure 7 top floor plan*

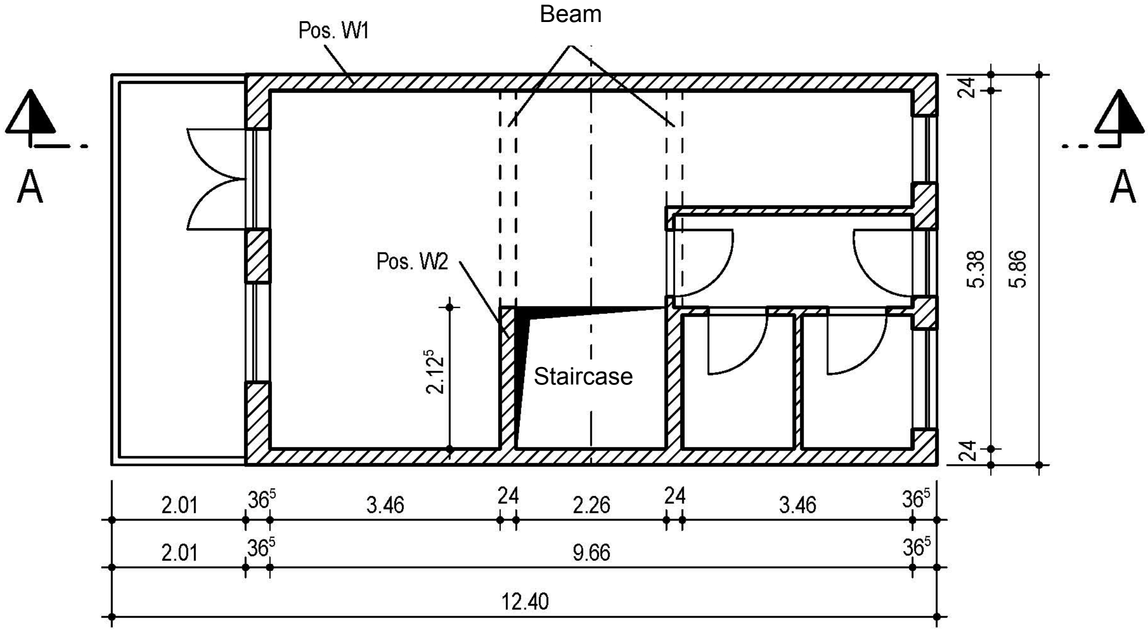



*Figure 8 first floor plan*

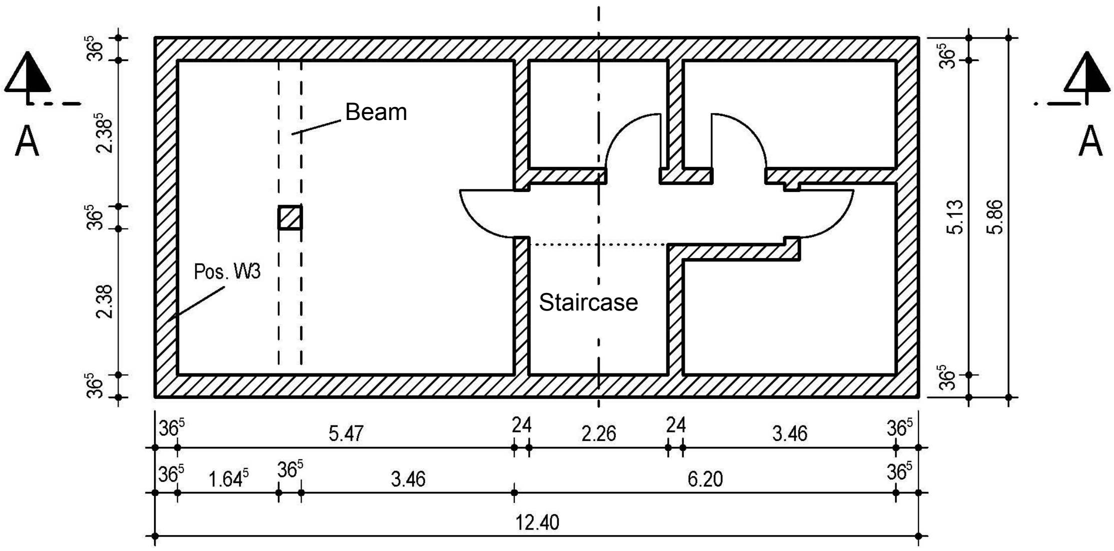



*Figure 9 basement plan*

## 5.2 Pos. W2 interior wall acc. to part 1-1

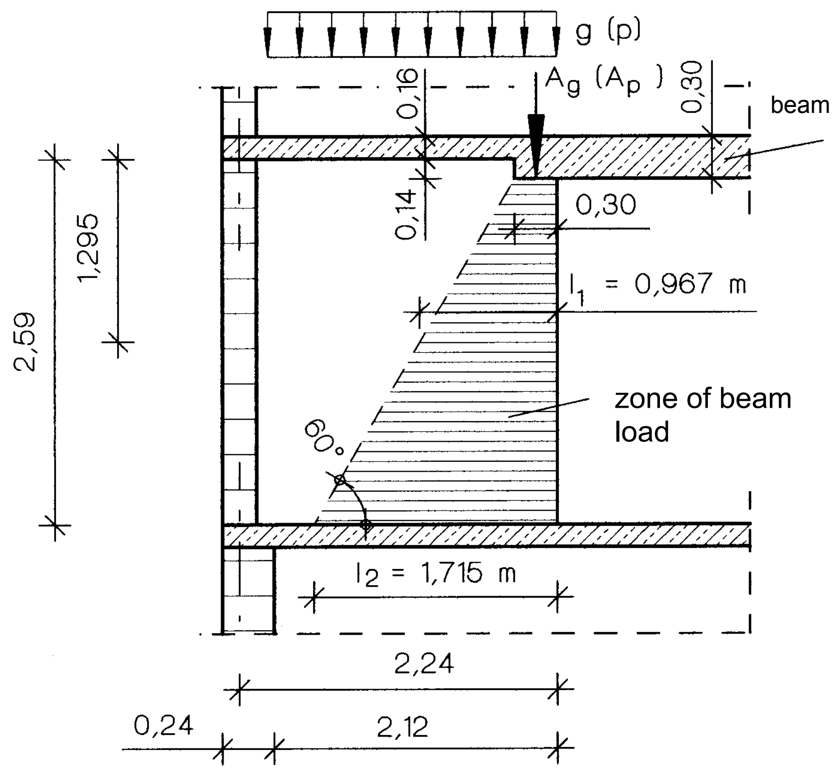
### 5.2.1 Geometry

| | | | | |
|---|---|---|---|---|
| thickness of the wall | $t$ | = | 0,24 | m |
| length of the wall | $l$ | = | 2,24 | m |
| span length of the slab | $l_1$ | = | 3,60 | m |

### 5.2.2 Material parameters

clay bricks, group 2, $f_b$ = 15 N/mm²
mortar M 2,5, $f_m$ = 2,5 N/mm²

### 5.2.3 Loading




$l = (1{,}295-0.14)/\tan 60°+0{,}30 = 0{,}967\text{m}$
$l = (2{,}59-0{,}14)/\tan 60°+0{,}30 = 1{,}715\text{m}$

*Figure 10 Loading and load distribution for Pos. W2 due to the beam*

Design value of the actions $S_E$ see EC 1[6.4.3.2 and annex A1.2] with the following simplification.

Consideration of all unfavourable variable actions:

$$\sum\left(\gamma_{Gj}\cdot G_{kj}\right)+1{,}5\cdot\sum Q_{k,i}$$

Load case combinations see Figure 11.

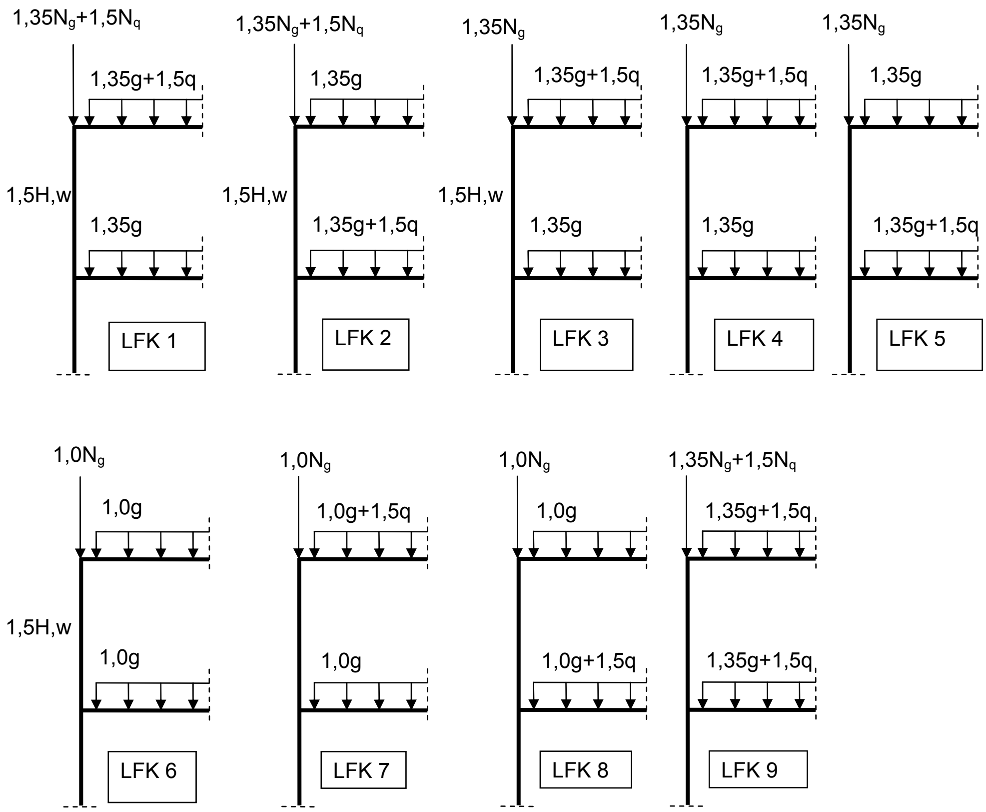



*Figure 11 Load case combinations for Pos. W2*

Load case combination 1 is verified. The numerical values for load case combination 6 are declared in brackets.

- **vertical loads**

  beam

  $A_g$=56,6 kN; $A_p$=16,9 kN

  angle of load distribution 60° (Figure 10)

  dead load of the wall

  $g_{wall} = 4{,}68 kN/m^2$

  roof loads

  $N_{g,roof} = 22{,}96 kN/m$, $N_{q,roof} = 6{,}47 kN/m$

*Table 2 configuration of the characteristic loads Pos. W2*

| | $N_g$ [kN/m] | $N_q$ [kN/m] |
|---|---|---|
| top of the wall | 22,96 | 6,47 |
| middle of the wall | 87,55 | 23,95 |
| bottom of the wall | 68,08 | 16,33 |

slab loading
slab over first floor

$g = 5{,}50 kN/m$, $q = 1{,}50 kN/m$

slab over basement

$g = 5{,}50 kN/m$, $q = 2{,}75 kN/m$

- **horizontal loads**
  - wind

design values of the horizontal loads
(proportion of the sum of second moments of area 0,456)
wind upper floor

$w_1 = 1{,}5 \cdot 0{,}456 \cdot (0{,}40 + 0{,}25) \cdot 10{,}36 = 4{,}606 kN/m$ (4,606kN/m)

wind first floor

$w_2 = 1{,}5 \cdot 0{,}456 \cdot (0{,}40 + 0{,}25) \cdot 12{,}44 = 5{,}531 kN/m$ (5,531kN/m)

- the structure is inclined due to deviation from the vertical

Determination of forces of obliquity (bracing forces in principle see Figure 12) with the unfavourable load case g+q (peak values of the vertical loads)

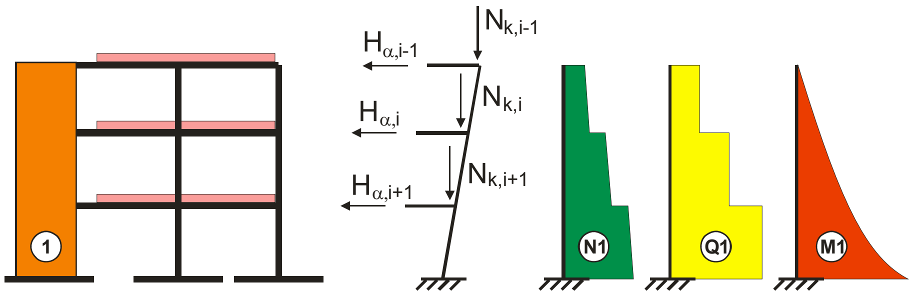



*Figure 12 Forces due to unintentional obliquity (principle sketch)*

EC 6 –5.3 – assumption that the structure is inclined at an angel $\nu = \frac{1}{100\sqrt{h_{tot}}}$ radians to the vertical; $h_{tot}$ is the total height of the structure in metres.

$$\nu = \frac{1}{100\sqrt{7{,}85}} = 3{,}569 \cdot 10^{-3}$$

$H_{Ln} = \nu \cdot F_n$

$H_{L1} = 3{,}569 \cdot 10^{-3} \cdot 520{,}43 = 1{,}86 kN$ (1,86kN)

$H_{L2} = 3{,}569 \cdot 10^{-3} \cdot 1127{,}58 = 4{,}47 kN$ (4,47kN)

proportion of the loads from obliquity:

$H_{L1} = 0{,}456 \cdot 1{,}86 = 0{,}848 kN$ (0,848kN)

$H_{L2} = 0{,}456 \cdot 4{,}47 = 2{,}038 kN$ (2,038kN)

Total shear force in longitudinal direction of the wall:
$V = 0{,}848 + 2{,}038 + 4{,}606 \cdot 2{,}50 + 5{,}531 \cdot 2{,}75 = 29{,}61\,kN$ (29,61kN)

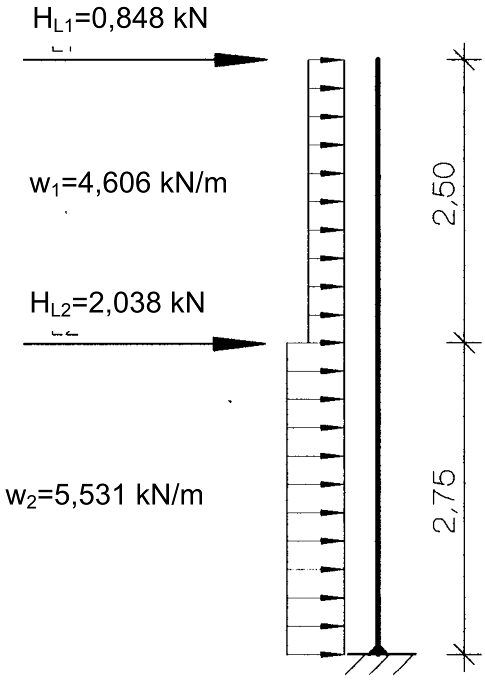



*Figure 13 loads from wind and obliquity*

### 5.2.4 bending moments

- **moments as a result of vertical loads**
  - characteristic compressive strength of masonry

    compressive strength of the unit $f_b$= 15 N/mm²
    compressive strength of the mortar $f_m$= 2,5 N/mm²
    $f_k = K \cdot f_b^{0,7} \cdot f_m^{0,3} = 0{,}45 \cdot 15^{0,7} \cdot 2{,}5^{0,3} = 3{,}94\ N/mm^2$
  - modulus of elasticity
    $E_4 = E_{c0m} = 30500 MN/m^2$
    $E_1 = E_2 = 1000 \cdot f_k = 39400 MN/m^2$
  - second moment of area

$$I_4 = I_B = \frac{1{,}00 \cdot 0{,}16^3}{12} = 3{,}413 \cdot 10^{-4} m^4$$

$$I_1 = \frac{1{,}00 \cdot 0{,}115^3}{12} = 1{,}267 \cdot 10^{-4} m^4$$

$$I_2 = \frac{1{,}00 \cdot 0{,}24^3}{12} = 1{,}152 \cdot 10^{-3} m^4$$

  - stiffness factor
    $n = 4$ members fixed at both ends
    $n = 3$ otherwise

- slab loading

  Load case combination 1 is verified with $1{,}35 \cdot g + 1{,}5 \cdot q$. The numerical values for load case combination 6 are declared in brackets.

  slab over first floor

  $q_4 = 1{,}35 \cdot 5{,}50 + 1{,}5 \cdot 1{,}50 = 9{,}675 kN/m^2$ (5,500kN/m²)

  slab over basement

  $q_4 = 1{,}35 \cdot 5{,}50 = 7{,}425 kN/m^2$ (5,500kN/m²)

Bending moment at the top of the wall

Simplified frame model

$$M_{full} = -\frac{9{,}675 \cdot 3{,}60^2}{12} = -10{,}449 kNm/m \qquad (-5{,}94 kNm/m)$$

with

$$E_{1M} \cdot I_{1M} = 3940 \cdot 1{,}152 \cdot 10^{-3} = 4{,}5389\, MNm^2$$

$$E_{2M} \cdot I_{2M} = 3940 \cdot 1{,}267 \cdot 10^{-4} = 0{,}4992\, MNm^2$$

$$E_B \cdot I_B = 30000 \cdot 3{,}413 \cdot 10^{-4} = 10{,}2390\, MNm^2$$

$$M_1 = \frac{\frac{4 \cdot 4{,}5389}{2{,}75}}{\frac{4 \cdot 4{,}5389}{2{,}75} + \frac{3 \cdot 0{,}4992}{3{,}00} + \frac{4 \cdot 10{,}2390}{3{,}60}} \cdot (-10{,}449)$$

$$M_1 = 0{,}357 \cdot (-10{,}449) = -3{,}733 kNm/m \qquad (-2{,}121 kNm/m)$$

$$k_m = \frac{\frac{4 \cdot 10{,}2390}{3{,}60}}{\frac{4 \cdot 4{,}5389}{2{,}75} + \frac{3 \cdot 0{,}4992}{3{,}00}} = 1{,}602 < 2{,}0$$

design value

$$M_o = (1 - \frac{k_m}{4}) \cdot |M_1| = 2{,}238 kNm/m \quad (1{,}273 kNm/m)$$

Bending moment at the bottom of the wall

$$M_{full} = -\frac{7{,}425 \cdot 3{,}60^2}{12} = -8{,}019 kNm/m$$

$$M_2 = 0{,}280 \cdot 8{,}019 = 2{,}243 kNm/m \qquad (1{,}663 kNm/m)$$

$$k_m = \frac{\frac{4 \cdot 10{,}2390}{3{,}60}}{\frac{4 \cdot 4{,}5389}{2{,}60} + \frac{3 \cdot 0{,}4992}{2{,}75}} = 0{,}837 < 2{,}0$$

Bending moment at the bottom of the wall

$M_2 = 0{,}280 \cdot 8{,}019 = 2{,}243 kNm/m$ (1,663kNm/m)

$k = 0{,}837 < 2{,}0$ Reduction factor

$M_2 = 0{,}791 \cdot 2{,}243 = 1{,}774 kNm/m$ (1,315kNm/m)

moment in the middle of the wall (from the moments at the top and the bottom of the wall)

$$M_m = (2{,}238 + 1{,}774) \cdot \frac{1}{2} - 2{,}238 = -0{,}232 kNm/m \quad (0{,}021 kNm/m)$$

design value

$M_m = |M_m| = 0{,}232 kNm/m \ (0{,}021 kNm/m)$

Corresponding normal force in the middle of the wall considering the load propagation of the beam

### 5.2.5 Design values

- Design values of the actions

top of the wall

$$N_{Ed} = N_K = 1{,}35 \cdot 22{,}96 + 1{,}5 \cdot 6{,}47 = 40{,}70 kN/m \ (22{,}96 kN/m)$$

middle of the wall

$$N_m = 1{,}35 \cdot 87{,}55 + 1{,}5 * 23{,}95 = 154{,}12 kN/m$$

$$(N_m = 87{,}55 kN/m)$$

from wind and obliquity

$$M = 0{,}848 \cdot (2{,}50 + 1{,}375) + 2{,}038 \cdot 1{,}375 + 4{,}606 \cdot 2{,}5 \cdot 2{,}625 + 5{,}531 \cdot 1{,}375 \cdot 0{,}688$$

$$M = 41{,}547 kNm \ (41{,}547 kNm)$$

$$N = 1{,}35 \cdot [(22{,}96 + 4{,}68 \cdot 1{,}375) \cdot 2{,}24 + 56{,}6] + 1{,}5 \cdot [6{,}47 \cdot 2{,}24 + 16{,}9]$$

$$N = 212{,}39 kN \ (122{,}44 kN)$$

$$e = \frac{41{,}547}{212{,}39} = 0{,}196 m \ (0{,}339 m) < 0{,}373 m = \frac{2{,}24}{6}$$

(section uncracked!)

$$N_\sigma = \pm \frac{6 \cdot M}{L^2} = \frac{6 \cdot 41{,}547}{2{,}24^2} = \pm 49{,}682 kN/m \ (\pm 49{,}682 kN/m)$$

$$N_{Ed} = N_m + N_\sigma = 154{,}12 + 49{,}682 = 203{,}80 kN/m \ (137{,}23 kN/m)$$

bottom of the wall

$$N_F = 1{,}35 \cdot 68{,}08 + 1{,}5 \cdot 16{,}33 = 116{,}40 kN/m \ (68{,}08 kN/m)$$

from wind and obliquity

$$M = 0{,}848 \cdot (2{,}50 + 2{,}75) + 2{,}038 \cdot 2{,}75 + 4{,}606 \cdot 2{,}5 \cdot 4{,}00 + 5{,}531 \cdot 2{,}75 \cdot 1{,}375$$

$$M = 77{,}031 kNm \ (77{,}031 kNm)$$

$$N = 1{,}35 \cdot [(22{,}96 + 12{,}12) \cdot 2{,}24 + 56{,}6] + 1{,}5 \cdot [6{,}47 \cdot 2{,}24 + 16{,}9]$$

$$N = 229{,}585 kN \ (135{,}182 kN)$$

$$e = \frac{77{,}031}{229{,}585} = 0{,}336 m < 0{,}373 m = \frac{2{,}24}{6} = \frac{L}{6} \text{(section uncracked!)}$$

$$\begin{pmatrix} 0{,}570 m > 0{,}373 m = \frac{2{,}24}{6} = \frac{L}{6} \\ 0{,}570 m < 0{,}747 m = \frac{2{,}24}{3} = \frac{L}{3} \end{pmatrix} \Rightarrow \text{(section partial cracked!)}$$

Length of the compressed section for load case combination 6:

$$L_c = 3 \cdot c = 3 \cdot (\frac{L}{2} - e) = 3 \cdot (\frac{2{,}24}{2} - 0{,}570) = 1{,}650 m$$

$$N_\sigma = \pm \frac{6 \cdot M}{L^2} = \frac{6 \cdot 77{,}031}{2{,}24^2} = \pm 92{,}113 kN/m$$

$$(N_\sigma = \pm \frac{6 \cdot M}{L_c^2} = \frac{6 \cdot 77{,}031}{1{,}650^2} = \pm 169{,}765 kN/m)$$

$$N_{Ed} = N_F + N_\sigma = 116{,}40 + 92{,}113 = 208{,}51 kN/m \ (237{,}85 kN/m)$$

- Design values of the resistance

eccentricity

top of the wall

$$e_0 = \frac{2,238}{40,70} = 0,055\text{m} \ \ (0,055\text{m})$$

middle of the wall

$$e_m = \frac{0,232}{203,80} = 0,0011\text{m} \ \ (0,0001\text{m})$$

bottom of the wall

$$e_u = \frac{1,774}{208,51} = 0,0085\text{m} \ \ (0,0055\text{m})$$

Initial eccentricity

$$e_{init} = \frac{h_{ef}}{450} = \frac{2,26}{450} = 0,00502\text{m} \qquad \text{[EC 6-5.5.1]}$$

with $h_{ef} = 0,871 \cdot 2,59 = 2,26\text{m}$ [EC 6-6.1.4(10)]

wall supported on three sides, $l = 2,24\text{m} < 3,60\text{m} = 15 \cdot 0,24$ .

$\rho_2 = 1,0$ , load eccentricity at the top of the wall

$$e_o = 0,055\text{m} \ \ (0,031\text{m}) < 0,06\text{m} = \frac{0,24}{4} = \frac{t}{4}$$

$$\rho_3 = \frac{1}{1 + \left(\frac{1,0 \cdot 2,59}{3 \cdot 2,24}\right)^2} \cdot 1,0 = 0,871 > 0,3$$

resultant eccentricity

Top of the wall

$$e_i = 0,055 + 0,00502 = 0,060\text{m} \ \ (0,060\text{m}) > 0,012\text{m} = 0,05 \cdot 0,24$$

middle of the wall

$$e_{mk} = 0,0011 + 0,00502 = 0,0061\text{m} \ \ (0,0051) < 0,012\text{m} = 0,05 \cdot 0,24$$

$\Rightarrow 0,05 \cdot t$ becomes decisive

bottom of the wall

$$e_i = 0,0085 + 0,00502 = 0,0135\text{m} > 0,012\text{m} = 0,05 \cdot 0,24 \qquad (0,0105\text{m}<0,012\text{m})$$

Reduction factors for slenderness and eccentricity [EC 6-6.1.3]:

Top of the wall

$$\Phi_K = 1 - 2 \cdot \frac{0,060}{0,24} = 0,5 \ \ (0,5)$$

middle of the wall

$$\Phi_m = A_1 \cdot e^{-\frac{u^2}{2}}$$

with
$$u = \frac{\frac{2,26}{0,24} - 2}{23 - 37 \cdot \frac{0,012}{0,24}} = \frac{7,417}{21,15} = 0,350 \ \ (0,350)$$

$$A_1 = 1 - 2 \cdot \frac{0,012}{0,24} = 0,900 \ \ (0,900)$$

$$\Phi_m = 0{,}900 \cdot e^{-\frac{0{,}350^2}{2}} = 0{,}847 \;\; (0{,}847)$$

bottom of the wall

$$\Phi_B = 1 - 2 \cdot \frac{0{,}0135}{0{,}24} = 0{,}888 \;\; (0{,}900)$$

Design values of the resistance [EC 6-6.1.2]:
Top of the wall

$$N_{Rd} = 0{,}5 \cdot \frac{0{,}24 \cdot 3{,}94}{1{,}7} = 278{,}91 kN/m \;\; (278{,}91 kN/m)$$

middle of the wall

$$N_{Rd} = 0{,}847 \cdot \frac{0{,}24 \cdot 3{,}94}{1{,}7} = 471{,}13 kN/m \;\; (471{,}13 kN/m)$$

bottom of the wall

$$N_{Rd} = 0{,}888 \cdot \frac{0{,}24 \cdot 3{,}94}{1{,}7} = 493{,}94 kN/m \;\; (500{,}61 kN/m)$$

### 5.2.6 Verification to vertical loading

[EC 6-6.1.2]:

$N_{Ed} \leq N_{Rd}$

Top of the wall

$N_{Ed} = 40{,}70 kN/m < 278{,}91 kN/m = N_{Rd}$

$(N_{Ed} = 22{,}96 kN/m < 278{,}91 kN/m = N_{Rd})$

middle of the wall

$N_{Ed} = 203{,}80 kN/m < 471{,}13 kN/m = N_{Rd}$

$(N_{Ed} = 137{,}232 kN/m < 471{,}13 kN/m = N_{Rd})$

bottom of the wall

$N_{Ed} = 208{,}51 kN/m < 493{,}94 kN/m = N_{Rd}$

$(N_{Ed} = 237{,}85 kN/m < 500{,}61 kN/m = N_{Rd})$

### 5.2.7 Verification to shear loading

- Design value of the shear resistance

$$V_{Rd} = \frac{f_{vk} \cdot t \cdot l_c}{\gamma_M}$$

with $f_{vk}$ shear strength
$t$ thickness of the wall
$l_c$ length of the wall
$\gamma_M$ safety factor

$f_{vk} = f_{vko} + 0{,}4 \cdot \sigma_d$

$f_{vko} = 0{,}20 MN/m^2$ [EC6-Tab. 3.4] für Mauersteingruppe 2, Mörtel M2,5

Length of the compressed section and design compressive stress for load case combination 1+6:

$L = 2{,}24 m \;\; (L_c = 1{,}650 m)$

$$vorh\sigma_d = \frac{\min N}{t \cdot l_c} = \frac{0{,}229585}{0{,}24 \cdot 2{,}24} = 0{,}427 MN/m^2 \;\; (0{,}341 MN/m^2)$$

Characteristic value of shear resistance

$f_{vk} = 0{,}20 + 0{,}4 \cdot 0{,}427 = 0{,}371\,MN/m^2$ (0,336MN/m²)

$< 0{,}065 \cdot f_b = 0{,}065 \cdot 15 = 0{,}975\,MN/m^2$

$< 1{,}0\,MN/m^2 = f_{vlt}$ limit value for $f_{vk}$ (possibly defined at the National Annex)

Design value of the shear resistance:

$$V_{Rd} = \frac{0{,}371 \cdot 0{,}24 \cdot 2{,}24}{1{,}7} = 117{,}3\,kN\ \ (106{,}7kN)$$

- Shear force for load case combination 1 + 6

Q=V=29,61kN (29,61kN)

- verification

$V_{Ed} = 29{,}61kN < 117{,}3kN = V_{Rd}$

$(29{,}61kN < 106{,}7kN)$ .

## 5.3 Pos. W2 interior wall – simplified method acc. to part 3

### 5.3.1 Requirements

- the building has not more than three stores above ground level;
- the walls are fixed either through the ceiling or through appropriate constructions, such as ring beams with sufficient rigidity;
- load depth of the ceiling and the roof on the wall is at least 2 / 3 of wall thickness, but not less than 85 mm;
- the floor height is not higher than 3.0 m;
- the smallest dimensions in the building floor plan is at least 1 / 3 of building height;
- the characteristic values of the variable loads on the ceiling and the roof are not more than 5.0 kN / m²;
- the largest span of the ceiling is 6.0 m;

The conditions are complied with.

### 5.3.2 Verification to shear loading

- Design value of the shear resistance

$$V_{Rd} = c_v \cdot \left[\frac{l}{2} - e_{Ed}\right] \cdot t \cdot f_{vdo} + 0{,}4 \cdot \frac{N_{Ed}}{\gamma_M} \leq 3\left[\frac{l}{2} - e_{Ed}\right] \cdot t \cdot f_{vdu}$$

with:

$c_v = 3$ (filled head joints)

Excentricity from wind and obliquity for load case 1

$M = 0{,}848 \cdot (2{,}50 + 2{,}75) + 2{,}038 \cdot 2{,}75 + 4{,}606 \cdot 2{,}5 \cdot 4{,}00 + 5{,}531 \cdot 2{,}75 \cdot 1{,}375$

$M = 77{,}031kNm\ \ (77{,}031kNm)$

$N = 1{,}35 \cdot [(22{,}96 + 12{,}12) \cdot 2{,}24 + 56{,}6] + 1{,}5 \cdot [6{,}47 \cdot 2{,}24 + 16{,}9]$

$N = 229{,}585kN\ \ (135{,}182kN)$

$$e_{Ed} = \frac{77{,}031}{229{,}585} = 0{,}336m < 0{,}373m = \frac{2{,}24}{6} = \frac{L}{6} \text{(section uncracked!)}$$

$$V_{Rd} = 3 \cdot \left[\frac{2,24}{2} - 0,376\right] \cdot 0,24 \cdot \frac{0,2}{1,7} + 0,4 \cdot \frac{0,229}{1,7} \leq 3\left[\frac{2,24}{2} - 0,376\right] \cdot 0,24 \cdot \frac{0,371}{1,7}$$

$$V_{Rd} = 180\text{kN} \leq \underline{117\text{kN}}$$

- verification

$$V_{Ed} = 29,61\text{kN} < 117\text{kN} = V_{Rd}$$

.

# 6 Example for out of plane shear

## 6.1 Pos. W1 exterior wall

### 6.1.1 Geometry

| | | | | |
|---|---|---|---|---|
| thickness of the wall | t | = | 0,24 | m |
| length of the wall | l | = | 10,36 | m |
| span length of the slab | $l_1$ | = | 2,59 | m |

### 6.1.2 Material parameters

clay bricks, group 2, $f_b$ = 15 N/mm²
mortar M 2,5, $f_m$ = 2,5 N/mm²

### 6.1.3 Loading

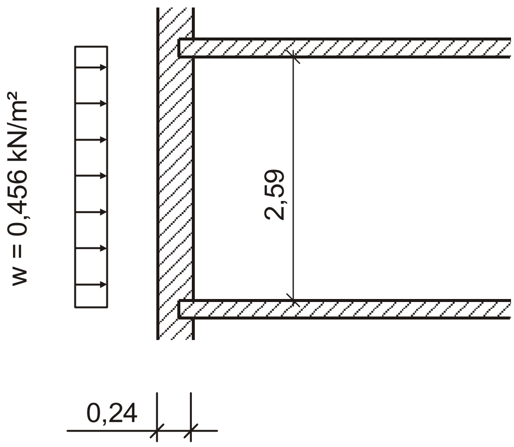



*Figure 14 Wind loading for Pos. W1*

- **vertical loads**

  jamb wall

  $A_g$=4,68 kN/m

  dead load of the wall

  $g_{wall} = 4,68\text{kN/m}^2$

  roof loads

$N_{g,roof} = 1{,}29 kN/m$, $N_{q,roof} = 1{,}56 kN/m$

- **horizontal loads**
  - wind

    $w_d = 1{,}5 \cdot 0{,}456 = 0{,}684 kN/m^2$

### 6.1.4 Shear force as result of wind

$V_d = 0{,}684 \cdot 2{,}59 / 2 = 0{,}89 kN/m$

### 6.1.5 Verification to shear loading

- Design value of the shear resistance

$$V_{Rd} = \frac{f_{vk} \cdot l \cdot t_c}{\gamma_M}$$

with $f_{vk}$ shear strength
$t_c$ thickness of the compression zone of the wall
$l$ length of the wall
$\gamma_M$ safety factor

$f_{vk} = f_{vko} + 0{,}4 \cdot \sigma_d$

$f_{vko} = 0{,}20 MN/m^2$ [EC6-Tab. 3.4] unit group 2, mortar M2,5

No moments from load on the top of the wall, because support of the ceiling is parallel to its span direction.

$$e = e_{lim} = 0{,}05 \cdot 0{,}24 m = 0{,}012 < \frac{l}{6} = 0{,}04 m \text{ (section uncracked!)}$$

$t_c = t = 0{,}24 m$

$\min N = 4{,}68 + 1{,}29 = 5{,}97$ kN/m

$$\text{vorh}\sigma_d = \frac{\min N}{t \cdot l} = \frac{0{,}00597}{0{,}24 \cdot 1} = 0{,}025 MN/m^2$$

Characteristic value of shear resistance

$f_{vk} = 0{,}20 + 0{,}4 \cdot 0{,}025 = 0{,}21 MN/m^2 < 0{,}065 \cdot f_b = 0{,}065 \cdot 15 = 0{,}975 MN/m^2$

Design value of the shear resistance:

$$V_{Rd} = \frac{0{,}21 \cdot 0{,}24 \cdot 1{,}00}{1{,}7} = 29{,}6 kN/m$$

- verification

$V_{Ed} = 0{,}89 kN < 29{,}6 kN = V_{Rd}$