Two floor building from YTONG

Building place :

Engineering firm :

General Building Characteristics

Floors   Floors above : 2   Floors below : 0   Masonry type   Confined

Floor heights [m]   Floor type

Gr. floor, floor height=3.00 [m], type=concrete slab d=0.17[m]
1st floor, floor height=3.00 [m], type=concrete slab d=0.17[m]

Structure Loads

Dead on floors   Floor finishing= 0.80[kN/m²], Walls on floors= 0.00[KN/m²]

Live on floors   Live on floors = 2.00[KN/m²], Live on stairs= 5.00[KN/m²]
Live on balconies= 5.00[KN/m²]

Dead on roof   French type tiles+trusses Self weight= 0.70[KN/m²]
Ceiling= 0.60[KN/m²]

Live on roof   Snow= 1.00[KN/m²], Wind= 1.25[KN/m²]
Wind (perpendicular) 1.25xsin²(25°)= 0.18[KN/m²]

Coefficients of action combinations   (psi0)=0.60, (psi1)=0.60, (psi2)=0.30

Masonry Materials   YTONG-M5 0.30

Concrete-Soil   C16/20-S400   qsoil=0.20 [N/mm²,MPA]

General Elements for Earthquake Design

Seismic acceleration   0.160xg

Codes and Regulation which ara used.

Eurocode 6 : ENV 1996-1-1, Design of masonry structures.
Eurocode 2 : ENV 1992-1-1, Design of concrete structures.
Eurocode 8 : ENV 1998-1-1, Design provisions for earthquake resistanse of structures.
Eurocode 1 : ENV 1991-1-1, Basis of design and actions on structures.

YTONG-M5 0.30   **Wall thickness : 0.30 [m]**   Masonry type

Spesific weight : 8.00 [KN/m3]   Weight per m² : 2.40 [KN/m²]   Longitudinal joint: NO

Properties of Masonry Units (Eurocode 6, §3.1)   Masonry units : YTONG 30x25x60

Type of Masonry Units: Aerated concrete EN 771-4   Category I   Group 2a

Dimensions of Masonry Units[mm]: 600 x300 x250   Coefficient delta (table 3-2) = 1.15

Compressive strength : 2.50 [N/mm²] Normalized Compressive Strength fb= 1.15x 2.50= 2.88[N/mm²]

Properties of Mortar (Eurocode 6, §3.2)   Mortar : General purpose-M5

Mortar type: General purpose mortar   Compressive Strength fk: 5.00 [N/mm²]

Characteristic Masonry Strength (Eurocode 6, §3.6)

Compressive Strength fk $f_k = K f_b^{0.65} f_m^{0.25}$ = 1.64 [N/mm²]   (K=0.55)

Shear strength fvko = 0.15 [N/mm²]   Fvk = 0.50fvko+0.40fb   maxfvko= 1.00 [N/mm²]

Modulus of Elasticity (E=1000fk) = 1.64 [GPa=KN/mm²]   Shear Modulus G=40%E

Floor	Buld.El.	Element materias	Dimensions[m]	Pos. (x[m],y[m],a°)
Gr. floo	W1	YTONG-M5 0.30	6.00x 3.00x0.30	2.00,10.00,-90.00
Gr. floo	W2	YTONG-M5 0.30	10.00x 3.00x0.30	12.00,10.00,180.00
Gr. floo	W3	YTONG-M5 0.30	6.00x 3.00x0.30	12.00, 4.00, 90.00
Gr. floo	W4	YTONG-M5 0.30	10.00x 3.00x0.30	2.00, 4.00, 0.00
Gr. floo	W5	YTONG-M5 0.30	6.00x 3.00x0.30	7.00,10.00,-90.00
1st floo	W6	YTONG-M5 0.30	6.00x 3.00x0.30	2.00,10.00,-90.00
1st floo	W7	YTONG-M5 0.30	10.00x 3.00x0.30	12.00,10.00,180.00
1st floo	W8	YTONG-M5 0.30	6.00x 3.00x0.30	12.00, 4.00, 90.00
1st floo	W9	YTONG-M5 0.30	10.00x 3.00x0.30	2.00, 4.00, 0.00
1st floo	W10	YTONG-M5 0.30	6.00x 3.00x0.30	7.00,10.00,-90.00

Floor	Slab	thick. [cm]	Loads [KN/m²]			Area [m²]	Ly/Lx	Kind	Load Factor		Souround Elements
			self w	finish	live				kx	ky	
Gr. floor	P1	20	5.00	0.80	2.00	30.00	1.20		0.84	0.16	W2B-W1-W4A-W5
Gr. floor	P2	20	5.00	0.80	2.00	30.00	1.20		0.84	0.16	W5-W4B-W3-W2A
1st floor	P1	20	5.00	0.70	1.17	30.00	1.20		0.84	0.16	W7B-W6-W9A-W10
1st floor	P2	20	5.00	0.70	1.17	30.00	1.20		0.84	0.16	W10-W9B-W8-W7A

Structural design of slabs (EC2 §2.5 and §5.4.3).

Slabs with side ratio from 0.5 to 2, are computed as carrying load in both directions. The slab design is based on Marcus method, which assumes unit strips in x and y directions with equal center deflections The dead and live loads of each slab, are distributed in the two direction x and y with the coefficients kx and ky. Each direction is solved separately, considering equivalent slab spans.
If the load in one direction is <10% of the plate load the plate is considered as carrying load in one direction only. Reduction of span moments due to torsional resistance is not taken into account.
The support conditions and the continuouity are taken into account with special coefficients for support moments, which are obtained from the solution of continuous beams with equivalent spans.
These coefficients are taken for the most unfavorable load combination in each case.
The minimum support moments minMsdsup are obtained using the moment coefficients of an equivalent beam, with the most unfavorable placement of the living loads, on the plate unit strip.
The maximum support moments maxMsdsup, are obtained accordingly using the moment coefficients of an equivalent beam, with the most favorable placement of living loads on the unit plate strip.
The support moments are reduced by 20% with moment redistribution (EC2 §2.5.3).
The maximum span moment is obtained, from the smallest in absolute value (maxMsdsup) support moments and load combination 1.35g+1.50 q in the span. The loads tranfered on the supporting beams and masonry are obtained by loading with live loads the plates on both sides of the supporting beam or masonry wall.
In the case of slabs carrying load on one direction only, on the side beams which are not taking any load, is considered a minimum load from the plate equal to qL/4 (q:plate load, L:beam span).
The above solution method results in the most unfavorable forces and moments for the evaluation of the slab reinforcements, and the loads carried on the beams and the masonry walls.

Strength Computation (Eurocode 2 EC2 §4.3)

In addition to the reinforcing steel computations, checks are done for the code requirement for slab slenderness (EC2 §5.4.3), and minimum required reinforcement (EC2 §5.4.3).
The check of deflections is omitted as it is covered by the slab slenderness check, (EC2 §4.4.3.2)
The minimum coverage for slab reinforcement is considered 20mm>15mm (EC2 §4.1.3.3).

Gr. floor **Slab: P1** Ly/Lx= 1.20, C16/20-S400, h= 20 cm
Loads: dead g= 5.80 kN/m^2, live q= 2.00 kN/m^2

Direction x-x Lx= 5.00m, Loads: gx= 0.84x 5.80= 4.86, qx= 0.84x 2.00= 1.68[kN/m^2]
Minimum support moment minMsdst=-0.80x(0.125x1.35x 4.86+0.125x1.50x 1.68)x 5.00^2= -22.70 [kNm/m]
Maximum support moment maxMsdst=-0.80x(0.125x1.35x 4.86+0.063x1.50x 1.68)x 5.00^2= -19.58 [kNm/m]
From maxMsdst, for load 1.35x 4.86+1.50x 1.68 maximum span moment and reactions are obtained.
Maximum span moment Msds= 19.43[kNm/m] (V= 9.08x 5.00/2-19.58/ 5.00=18.79,M=0.5x18.79^2/ 9.08=19.43)
Loads on beams, dead gA= 9.72 [kN/m], gB= 14.58 [kN/m]
Loads on Beams, live qA= 3.78 [kN/m], qB= 4.62 [kN/m]
support : Msd=-22.70kNm/m, d=18.0cm, Kd= 3.78, ksi=0.11, ec/es=2.5/20.0, Ks=3.00, As= 3.79cm^2
span : Msd= 19.43kNm/m, d=18.0cm, Kd= 4.08, ksi=0.10, ec/es=2.2/20.0, Ks=2.99, As= 3.23cm^2

Direction y-y Ly= 6.00m, Loads: gy= 0.16x 5.80= 0.94, qy= 0.16x 2.00= 0.32[kN/m^2]
Support moment MsdsupA=MsdsupB=0 [kNm/m]
Span Moment Msds=(1.35x 0.94+1.50x 0.32)x 6.00^2/8= 7.87 [kNm/m]
Loads on beams or walls, dead gA=gB= 0.94x 6.00/2= 2.82 [kN/m].
Loads on beams or walls, live qA=qB= 0.32x 6.00/2= 0.96 [kN/m]
span : Msd= 7.87kNm/m, d=17.0cm, Kd= 6.06, ksi=0.06, ec/es=1.3/20.0, Ks=2.94, As= 1.36cm^2

Slenderness L/d=5.00/0.180=27.78<32.00 (EC2 T4.14),minimum reinforcement minAsxx=2.7, minAsyy=2.5cm^2/m

Span reinforcement x-x:#8/15 (3.33cm^2/m), y-y :#8/19(2.63cm^2/m) (down layer x-x)

Support reinforcement:Righ:#8/13(3.85)

Gr. floor **Slab: P2** Ly/Lx= 1.20, C16/20-S400, h= 20 cm
Loads: dead g= 5.80 kN/m^2, live q= 2.00 kN/m^2

Direction x-x Lx= 5.00m, Loads: gx= 0.84x 5.80= 4.86, qx= 0.84x 2.00= 1.68[kN/m^2]
Minimum support moment minMsdst=-0.80x(0.125x1.35x 4.86+0.125x1.50x 1.68)x 5.00^2= -22.70 [kNm/m]
Maximum support moment maxMsdst=-0.80x(0.125x1.35x 4.86+0.063x1.50x 1.68)x 5.00^2= -19.58 [kNm/m]
From maxMsdst, for load 1.35x 4.86+1.50x 1.68 maximum span moment and reactions are obtained.
Maximum span moment Msds= 19.43[kNm/m] (V= 9.08x 5.00/2-19.58/ 5.00=18.79,M=0.5x18.79^2/ 9.08=19.43)
Loads on beams, dead gA= 14.58 [kN/m], gB= 9.72 [kN/m]
Loads on Beams, live qA= 4.62 [kN/m], qB= 3.78 [kN/m]
support : Msd=-22.70kNm/m, d=18.0cm, Kd= 3.78, ksi=0.11, ec/es=2.5/20.0, Ks=3.00, As= 3.79cm^2
span : Msd= 19.43kNm/m, d=18.0cm, Kd= 4.08, ksi=0.10, ec/es=2.2/20.0, Ks=2.99, As= 3.23cm^2

Direction y-y Ly= 6.00m, Loads: gy= 0.16x 5.80= 0.94, qy= 0.16x 2.00= 0.32[kN/m^2]
Support moment MsdsupA=MsdsupB=0 [kNm/m]
Span Moment Msds=(1.35x 0.94+1.50x 0.32)x 6.00^2/8= 7.87 [kNm/m]
Loads on beams or walls, dead gA=gB= 0.94x 6.00/2= 2.82 [kN/m].
Loads on beams or walls, live qA=qB= 0.32x 6.00/2= 0.96 [kN/m]
span : Msd= 7.87kNm/m, d=17.0cm, Kd= 6.06, ksi=0.06, ec/es=1.3/20.0, Ks=2.94, As= 1.36cm^2

Slenderness L/d=5.00/0.180=27.78<32.00 (EC2 T4.14),minimum reinforcement minAsxx=2.7, minAsyy=2.5cm^2/m

Span reinforcement x-x:#8/15 (3.33cm^2/m), y-y :#8/19(2.63cm^2/m) (down layer x-x)

Support reinforcement:Left:#8/13(3.85)

1st floor Slab: P1 Ly/Lx= 1.20, C16/20-S400, h= 20 cm
Loads: dead g= 5.70 kN/m^2, live q= 1.17 kN/m^2

Direction x-x Lx= 5.00m, Loads: gx= 0.84x 5.70= 4.78, qx= 0.84x 1.17= 0.98[kN/m^2]
Minimum support moment minMsdst=-0.80x(0.125x1.35x 4.78+0.125x1.50x 0.98)x 5.00^2= -19.81 [kNm/m]
Maximum support moment maxMsdst=-0.80x(0.125x1.35x 4.78+0.063x1.50x 0.98)x 5.00^2= -17.98 [kNm/m]
From maxMsdst, for load 1.35x 4.78+1.50x 0.98 maximum span moment and reactions are obtained.
Maximum span moment Msds= 16.58[kNm/m] (V= 7.92x 5.00/2-17.98/ 5.00=16.21,M=0.5x16.21^2/ 7.92=16.58)
Loads on beams, dead gA= 9.56 [kN/m], gB= 14.34 [kN/m]
Loads on Beams, live qA= 2.20 [kN/m], qB= 2.70 [kN/m]
support : Msd=-19.81kNm/m, d=18.0cm, Kd= 4.04, ksi=0.10, ec/es=2.2/20.0, Ks=2.99, As= 3.29cm^2
span : Msd= 16.58kNm/m, d=18.0cm, Kd= 4.42, ksi=0.09, ec/es=1.9/20.0, Ks=2.97, As= 2.74cm^2

Direction y-y Ly= 6.00m, Loads: gy= 0.16x 5.70= 0.92, qy= 0.16x 1.17= 0.19[kN/m^2]
Support moment MsdsupA=MsdsupB=0 [kNm/m]
Span Moment Msds=(1.35x 0.92+1.50x 0.19)x 6.00^2/8= 6.91 [kNm/m]
Loads on beams or walls, dead gA=gB= 0.92x 6.00/2= 2.76 [kN/m].
Loads on beams or walls, live qA=qB= 0.19x 6.00/2= 0.58 [kN/m]
span : Msd= 6.91kNm/m, d=17.0cm, Kd= 6.47, ksi=0.06, ec/es=1.2/20.0, Ks=2.93, As= 1.19cm^2

Slenderness L/d=5.00/0.180=27.78<32.00 (EC2 T4.14),minimum reinforcement minAsxx=2.7, minAsyy=2.5cm^2/m

Span reinforcement x-x:#8/18 (2.78cm^2/m), y-y :#8/19(2.63cm^2/m) (down layer x-x)

Support reinforcement:Righ:#8/15(3.33)

1st floor Slab: P2 Ly/Lx= 1.20, C16/20-S400, h= 20 cm
Loads: dead g= 5.70 kN/m^2, live q= 1.17 kN/m^2

Direction x-x Lx= 5.00m, Loads: gx= 0.84x 5.70= 4.78, qx= 0.84x 1.17= 0.98[kN/m^2]
Minimum support moment minMsdst=-0.80x(0.125x1.35x 4.78+0.125x1.50x 0.98)x 5.00^2= -19.81 [kNm/m]
Maximum support moment maxMsdst=-0.80x(0.125x1.35x 4.78+0.063x1.50x 0.98)x 5.00^2= -17.98 [kNm/m]
From maxMsdst, for load 1.35x 4.78+1.50x 0.98 maximum span moment and reactions are obtained.
Maximum span moment Msds= 16.58[kNm/m] (V= 7.92x 5.00/2-17.98/ 5.00=16.21,M=0.5x16.21^2/ 7.92=16.58)
Loads on beams, dead gA= 14.34 [kN/m], gB= 9.56 [kN/m]
Loads on Beams, live qA= 2.70 [kN/m], qB= 2.20 [kN/m]
support : Msd=-19.81kNm/m, d=18.0cm, Kd= 4.04, ksi=0.10, ec/es=2.2/20.0, Ks=2.99, As= 3.29cm^2
span : Msd= 16.58kNm/m, d=18.0cm, Kd= 4.42, ksi=0.09, ec/es=1.9/20.0, Ks=2.97, As= 2.74cm^2

Direction y-y Ly= 6.00m, Loads: gy= 0.16x 5.70= 0.92, qy= 0.16x 1.17= 0.19[kN/m^2]
Support moment MsdsupA=MsdsupB=0 [kNm/m]
Span Moment Msds=(1.35x 0.92+1.50x 0.19)x 6.00^2/8= 6.91 [kNm/m]
Loads on beams or walls, dead gA=gB= 0.92x 6.00/2= 2.76 [kN/m].
Loads on beams or walls, live qA=qB= 0.19x 6.00/2= 0.58 [kN/m]
span : Msd= 6.91kNm/m, d=17.0cm, Kd= 6.47, ksi=0.06, ec/es=1.2/20.0, Ks=2.93, As= 1.19cm^2

Slenderness L/d=5.00/0.180=27.78<32.00 (EC2 T4.14),minimum reinforcement minAsxx=2.7, minAsyy=2.5cm^2/m

Span reinforcement x-x:#8/18 (2.78cm^2/m), y-y :#8/19(2.63cm^2/m) (down layer x-x)

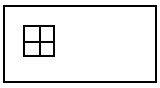
Support reinforcement:Left:#8/15(3.33)

Floor	Slab	thick. [cm]	Lx [m]	Ly [m]	Span Reinforcement		Support Reinforcement			
					x-x	y-y				
Gr. floor	P1	20	5.00	6.00	**Ø8/15b**	**Ø 8/19**		**Ø8/13**		
Gr. floor	P2	20	5.00	6.00	**Ø8/15b**	**Ø 8/19**	**Ø8/13**			
1st floor	P1	20	5.00	6.00	**Ø8/18b**	**Ø 8/19**		**Ø8/15**		
1st floor	P2	20	5.00	6.00	**Ø8/18b**	**Ø 8/19**	**Ø8/15**			

Masonry loads

In each floor the distributed slab and roof loads, and the concentrated loads at the places of floor beam supports, are carried to the masonry walls.

1st floor W6 Dimensions length= 6.00m, height= 3.00m, thickness=0.30m



Position x= 2.00m, y=10.00m, theta=270.00°, wall area= 16.56m²
YTONG-M5 0.30
Self weight of wall Gw= 16.56x 2.4= 39.7 kN
Line load on the wall, dead= 0.0kN/m live= 0.0kN/m

Load from slab P1, dead g= 9.6kN/m, live q= 2.2kN/m, from 0.00m to 6.00m

Total load on wall from floor, dead Gf= 57.4 kN, live Qf= 13.2 kN
Load from wall above dead Ga= 0.0 kN, live Qa= 0.0 kN
Load to wall bellow W1 dead Gb= 97.1 kN, live Qb= 13.2 kN

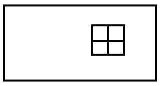
1st floor W7 Dimensions length=10.00m, height= 3.00m, thickness=0.30m

Position x=12.00m, y=10.00m, theta=180.00°, wall area= 27.12m²
YTONG-M5 0.30
Self weight of wall Gw= 27.12x 2.4= 65.1 kN
Line load on the wall, dead= 0.0kN/m live= 0.0kN/m

Load from slab P2, dead g= 2.8kN/m, live q= 0.6kN/m, from 0.00m to 5.00m
Load from slab P1, dead g= 2.8kN/m, live q= 0.6kN/m, from 5.00m to 10.00m

Total load on wall from floor, dead Gf= 27.6 kN, live Qf= 5.8 kN
Load from wall above dead Ga= 0.0 kN, live Qa= 0.0 kN
Load to wall bellow W2 dead Gb= 92.7 kN, live Qb= 5.8 kN

1st floor W8 Dimensions length= 6.00m, height= 3.00m, thickness=0.30m

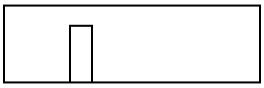


Position x=12.00m, y= 4.00m, theta= 90.00°, wall area= 16.56m²
YTONG-M5 0.30
Self weight of wall Gw= 16.56x 2.4= 39.7 kN
Line load on the wall, dead= 0.0kN/m live= 0.0kN/m

Load from slab P2, dead g= 9.6kN/m, live q= 2.2kN/m, from 0.00m to 6.00m

Total load on wall from floor, dead Gf= 57.4 kN, live Qf= 13.2 kN
Load from wall above dead Ga= 0.0 kN, live Qa= 0.0 kN
Load to wall bellow W3 dead Gb= 97.1 kN, live Qb= 13.2 kN

1st floor W9 Dimensions length=10.00m, height= 3.00m, thickness=0.30m

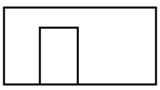


Position x= 2.00m, y= 4.00m, theta= 0.00°, wall area= 28.02m²
YTONG-M5 0.30
Self weight of wall Gw= 28.02x 2.4= 67.2 kN
Line load on the wall, dead= 0.0kN/m live= 0.0kN/m

Load from slab P1, dead g= 2.8kN/m, live q= 0.6kN/m, from 0.00m to 5.00m
Load from slab P2, dead g= 2.8kN/m, live q= 0.6kN/m, from 5.00m to 10.00m

Total load on wall from floor, dead Gf= 27.6 kN, live Qf= 5.8 kN
Load from wall above dead Ga= 0.0 kN, live Qa= 0.0 kN
Load to wall bellow W4 dead Gb= 94.8 kN, live Qb= 5.8 kN

1st floor W10 Dimensions length= 6.00m, height= 3.00m, thickness=0.30m



Position x= 7.00m, y=10.00m, theta=270.00°, wall area= 14.70m²
YTONG-M5 0.30
Self weight of wall Gw= 14.70x 2.4= 35.3 kN
Line load on the wall, dead= 0.0kN/m live= 0.0kN/m

Load from slab P1P2, dead g= 28.7kN/m, live q= 5.4kN/m, from 0.00m to 6.00m

Total load on wall from floor, dead Gf= 172.1 kN, live Qf= 32.4 kN
Load from wall above dead Ga= 0.0 kN, live Qa= 0.0 kN
Load to wall bellow W5 dead Gb= 207.4 kN, live Qb= 32.4 kN

1st floor **Total floor loads**

Total vertical floor loads from walls, permanent G1= 589 kN, live Q1= 70 kN
Total vertical floor loads from columns, permanent G2= 0 kN, live Q2= 0 kN
Total vertical floor loads (floors+walls) permanent Go= 589 kN, live Qo= 70kN
Action combination for vertical loads, total floor loads (1.35xGo+1.50xQo)= 900 kN
Action combination for seismic load, total floor loads (1.00xGo+0.30xQo)= 610 kN
Total floor mass Mo=(1.0xGo+0.30xQo)/9.81= 610.0/9.81= 62 kNsec²/m

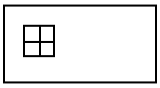
1st floor

Wall	Mass kNsec²/m	xm	ym	x.M	y.M
W6	10.3	2.00	7.00	20.60	72.11
W7	9.6	7.00	10.00	67.39	96.27
W8	10.3	12.00	7.00	123.62	72.11
W9	9.8	7.00	4.00	68.89	39.36
W10	22.1	7.00	7.00	154.93	154.93
Sum	**62.0**			**435.43**	**434.78**

Floor mass center xm=435.43/62.00=7.02m, ym=434.78/62.00=7.01m

For the computation of the floor mass center, we concider masses equivalent to the loads applied at the center of the walls or the columns.

Gr. floor **W1** Dimensions length= 6.00m, height= 3.00m, thickness=0.30m



Position x= 2.00m, y=10.00m, theta=270.00°, wall area= 16.56m²
YTONG-M5 0.30
Self weight of wall Gw= 16.56x 2.4= 39.7 kN
Line load on the wall, dead= 0.0kN/m live= 0.0kN/m

Load from slab P1, dead g= 9.7kN/m, live q= 3.8kN/m, from 0.00m to 6.00m

Total load on wall from floor, dead Gf= 58.3 kN, live Qf= 22.7 kN
Load from wall above W6 dead Ga= 97.1 kN, live Qa= 13.2 kN
Load to wall bellow dead Gb= 195.1 kN, live Qb= 35.9 kN

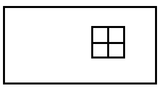
Gr. floor **W2** Dimensions length=10.00m, height= 3.00m, thickness=0.30m

Position x=12.00m, y=10.00m, theta=180.00°, wall area= 27.12m²
YTONG-M5 0.30
Self weight of wall Gw= 27.12x 2.4= 65.1 kN
Line load on the wall, dead= 0.0kN/m live= 0.0kN/m

Load from slab P2, dead g= 2.8kN/m, live q= 1.0kN/m, from 0.00m to 5.00m
Load from slab P1, dead g= 2.8kN/m, live q= 1.0kN/m, from 5.00m to 10.00m

Total load on wall from floor, dead Gf= 28.2 kN, live Qf= 9.6 kN
Load from wall above W7 dead Ga= 92.7 kN, live Qa= 5.8 kN
Load to wall bellow dead Gb= 186.0 kN, live Qb= 15.4 kN

Gr. floor **W3** Dimensions length= 6.00m, height= 3.00m, thickness=0.30m

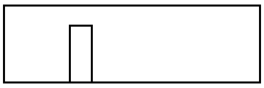


Position x=12.00m, y= 4.00m, theta= 90.00°, wall area= 16.56m²
YTONG-M5 0.30
Self weight of wall Gw= 16.56x 2.4= 39.7 kN
Line load on the wall, dead= 0.0kN/m live= 0.0kN/m

Load from slab P2, dead g= 9.7kN/m, live q= 3.8kN/m, from 0.00m to 6.00m

Total load on wall from floor, dead Gf= 58.3 kN, live Qf= 22.7 kN
Load from wall above W8 dead Ga= 97.1 kN, live Qa= 13.2 kN
Load to wall bellow dead Gb= 195.1 kN, live Qb= 35.9 kN

Gr. floor **W4** Dimensions length=10.00m, height= 3.00m, thickness=0.30m

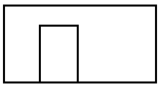


Position x= 2.00m, y= 4.00m, theta= 0.00°, wall area= 28.02m²
YTONG-M5 0.30
Self weight of wall Gw= 28.02x 2.4= 67.2 kN
Line load on the wall, dead= 0.0kN/m live= 0.0kN/m

Load from slab P1, dead g= 2.8kN/m, live q= 1.0kN/m, from 0.00m to 5.00m
Load from slab P2, dead g= 2.8kN/m, live q= 1.0kN/m, from 5.00m to 10.00m

Total load on wall from floor, dead Gf= 28.2 kN, live Qf= 9.6 kN
Load from wall above W9 dead Ga= 94.8 kN, live Qa= 5.8 kN
Load to wall bellow dead Gb= 190.2 kN, live Qb= 15.4 kN

Gr. floor **W5** Dimensions length= 6.00m, height= 3.00m, thickness=0.30m



Position x= 7.00m, y=10.00m, theta=270.00°, wall area= 14.70m²
YTONG-M5 0.30
Self weight of wall Gw= 14.70x 2.4= 35.3 kN
Line load on the wall, dead= 0.0kN/m live= 0.0kN/m

Load from slab P1P2, dead g= 29.2kN/m, live q= 9.2kN/m, from 0.00m to 6.00m

Total load on wall from floor, dead Gf= 175.0 kN, live Qf= 55.5 kN
Load from wall above W10 dead Ga= 207.4 kN, live Qa= 32.4 kN
Load to wall bellow dead Gb= 417.7 kN, live Qb= 87.9 kN

Gr. floor **Total floor loads**

Total vertical floor loads from walls, permanent G1= 595 kN, live Q1= 120 kN
Total vertical floor loads from columns, permanent G2= 0 kN, live Q2= 0 kN
Total vertical floor loads (floors+walls) permanent Go= 595 kN, live Qo= 120kN
Action combination for vertical loads, total floor loads (1.35xGo+1.50xQo)= 983 kN
Action combination for seismic load, total floor loads (1.00xGo+0.30xQo)= 631 kN
Total floor mass Mo=(1.0xGo+0.30xQo)/9.81= 631.0/9.81= 64 kNsec²/m

Gr. floor

Wall	Mass kNsec²/m	xm	ym	x.M	y.M
W1	10.7	2.00	7.00	21.37	74.79
W2	9.8	7.00	10.00	68.63	98.04
W3	10.7	12.00	7.00	128.21	74.79
W4	10.0	7.00	4.00	70.13	40.07
W5	23.1	7.00	7.00	161.94	161.94
Sum	**64.0**			**450.28**	**449.63**

Floor mass center xm=450.28/64.00=7.04m, ym=449.63/64.00=7.03m

For the computation of the floor mass center, we concider masses equivalent to the loads applied at the center of the walls or the columns.

Computation of seismic loads on the walls

The distribution of horizontal seismic forces on the walls, is done after the evaluation of the masonry wall stiffness using finite element analysis of each masonry wall, with the application of an horizontal relative unit movement between the top and bottom of the wall.
Rectangular plane stress elements with for nodes, are used in the finite element solution of each wall.
For each wall the approximate horizontal stiffness (without openings) is computed as 1/(h3/12EI+1.2h/GA)

General Elements for Earthquake Design

Soil Category	Horizontal groung acceleration axg= 0.160g
Building importance coefficient	1.00
Building system	Confined q=2.0
Foundation factor	1.00

Approximate basic buiding natural period T=0.09*H*(H/(H+L))½*(1/L)½= 0.10sec

Horizontal groung acceleration Rd(T)=axg= 0.160g

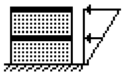
Total mass of the structure = (62+ 64)= 126 $kNsec^2/m$

Base shear force Vo= 126x0.160x9.81= 198 kN

Vertical distribution of seismic forces.

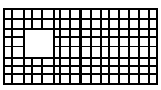
(computation of shear center in each floor)

Floor	Mass[$kNsec^2/m$]	Zi[m]	m.Zi	horizontal force Fi[kN]
1st floor	62.00	6.00	372.00	198x372.0/ 564.00= 130.60
Gr. floor	64.00	3.00	192.00	198x192.0/ 564.00= 67.40
sums	126.00		564.00	198.00



Following is shown, analytically for each floor and masonry wall, the distribution of seismic forces on the walls according to their relative horizontal stiffness which are computed with a finite element analysis for each masonry wall.

1st floor **W6** Dimensions length= 6.00m, height= 3.00m, thickness=0.30m



Computation of horizontal wall stiffness in the wall plane

The wall is divided in 128 rectangular plane stress finite elements.
Nodes in a grid 9x17 , total 153 nodes.
Wall stiffness from finite element analysis K= 0.261 GN/m
Approximate wall stiffness (without openings) = 0.302 GN/m
Stiffness (x-x) Kx= 0.000 GN/m, (y-y) Ky= 0.261 GN/m

Seismic forces based on equivalent static horizontal loads
Wall eccentricities from building elastic axis, ex=4.96[m], ey=0.17[m]

Horizontal force Fx

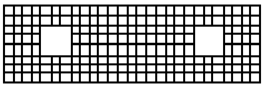
Seismic direction x-x= 130.60x 0.000/ 0.945+ 0.17x 8x 0.000/ 21.375= 0.00[kN]
Seismic direction y-y= 0.00x 0.000/ 0.945+ 0.17x 24x 0.000/ 21.375= 0.00[kN]

Horizontal force Fy

Seismic direction x-x= 0.00x 0.261/ 0.720+ 4.96x 8x 0.261/ 21.375= 0.51[kN]
Seismic direction y-y= 130.60x 0.261/ 0.720+ 4.96x 24x 0.261/ 21.375= 48.82[kN]

Considering effect of seismic forces in x and y directions
$(expFx)^2$= 0.00^2+ 0.00^2, expFx= 0.00 [kN]
$(expFy)^2$= 0.51^2+ 48.82^2, expFy= 48.83 [kN]
Maximum resulting seismic forces according
maxFx= 0.00+0.30x 0.00= 0.00 [kN]
maxFy= 48.82+0.30x 0.51= 48.98 [kN]
Maximum horizontal force along the wall F=48.98 kN

1st floor **W7** Dimensions length=10.00m, height= 3.00m, thickness=0.30m



Computation of horizontal wall stiffness in the wall plane

The wall is divided in 216 rectangular plane stress finite elements.
Nodes in a grid 9x28 , total 252 nodes.
Wall stiffness from finite element analysis K= 0.445 GN/m
Approximate wall stiffness (without openings) = 0.529 GN/m
Stiffness (x-x) Kx= 0.445 GN/m, (y-y) Ky= 0.000 GN/m

Seismic forces based on equivalent static horizontal loads
Wall eccentricities from building elastic axis, ex=0.04[m], ey=3.17[m]

Horizontal force Fx

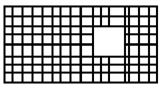
Seismic direction x-x= 130.60x 0.445/ 0.945+ 3.17x 8x 0.445/ 21.375= 62.06[kN]
Seismic direction y-y= 0.00x 0.445/ 0.945+ 3.17x 24x 0.445/ 21.375= 1.62[kN]

Horizontal force Fy

Seismic direction x-x= 0.00x 0.000/ 0.720+ 0.04x 8x 0.000/ 21.375= 0.00[kN]
Seismic direction y-y= 130.60x 0.000/ 0.720+ 0.04x 24x 0.000/ 21.375= 0.00[kN]

Considering effect of seismic forces in x and y directions
$(expFx)^2 = 62.06^2 + 1.62^2$, expFx= 62.08 [kN]
$(expFy)^2 = 0.00^2 + 0.00^2$, expFy= 0.00 [kN]
Maximum resulting seismic forces according
maxFx= 62.06+0.30x 1.62= 62.54 [kN]
maxFy= 0.00+0.30x 0.00= 0.00 [kN]
Maximum horizontal force along the wall F=62.54 kN

1st floor **W8** Dimensions length= 6.00m, height= 3.00m, thickness=0.30m



Computation of horizontal wall stiffness in the wall plane

The wall is divided in 128 rectangular plane stress finite elements.
Nodes in a grid 9x17 , total 153 nodes.
Wall stiffness from finite element analysis K= 0.255 GN/m
Approximate wall stiffness (without openings) = 0.302 GN/m
Stiffness (x-x) Kx= 0.000 GN/m, (y-y) Ky= 0.255 GN/m

Seismic forces based on equivalent static horizontal loads
Wall eccentricities from building elastic axis, ex=5.04[m], ey=0.17[m]

Horizontal force Fx

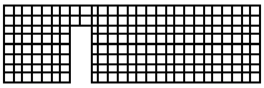
Seismic direction x-x= 130.60x 0.000/ 0.945+ 0.17x 8x 0.000/ 21.375= 0.00[kN]
Seismic direction y-y= 0.00x 0.000/ 0.945+ 0.17x 24x 0.000/ 21.375= 0.00[kN]

Horizontal force Fy

Seismic direction x-x= 0.00x 0.255/ 0.720+ 5.04x 8x 0.255/ 21.375= 0.51[kN]
Seismic direction y-y= 130.60x 0.255/ 0.720+ 5.04x 24x 0.255/ 21.375= 47.73[kN]

Considering effect of seismic forces in x and y directions
$(expFx)^2 = 0.00^2 + 0.00^2$, expFx= 0.00 [kN]
$(expFy)^2 = 0.51^2 + 47.73^2$, expFy= 47.73 [kN]
Maximum resulting seismic forces according
maxFx= 0.00+0.30x 0.00= 0.00 [kN]
maxFy= 47.73+0.30x 0.51= 47.88 [kN]
Maximum horizontal force along the wall F=47.88 kN

1st floor **W9** Dimensions length=10.00m, height= 3.00m, thickness=0.30m



Computation of horizontal wall stiffness in the wall plane

The wall is divided in 216 rectangular plane stress finite elements.
Nodes in a grid 9x28 , total 252 nodes.
Wall stiffness from finite element analysis K= 0.500 GN/m
Approximate wall stiffness (without openings) = 0.529 GN/m
Stiffness (x-x) Kx= 0.500 GN/m, (y-y) Ky= 0.000 GN/m

Seismic forces based on equivalent static horizontal loads
Wall eccentricities from building elastic axis, ex=0.04[m], ey=2.83[m]

Horizontal force Fx

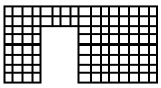
Seismic direction x-x= 130.60x 0.500/ 0.945+ 2.83x 8x 0.500/ 21.375= 69.66[kN]
Seismic direction y-y= 0.00x 0.500/ 0.945+ 2.83x 24x 0.500/ 21.375= 1.62[kN]

Horizontal force Fy

Seismic direction x-x= 0.00x 0.000/ 0.720+ 0.04x 8x 0.000/ 21.375= 0.00[kN]
Seismic direction y-y= 130.60x 0.000/ 0.720+ 0.04x 24x 0.000/ 21.375= 0.00[kN]

Considering effect of seismic forces in x and y directions
(expFx)^2= 69.66^2+ 1.62^2, expFx= 69.68 [kN]
(expFy)^2= 0.00^2+ 0.00^2, expFy= 0.00 [kN]
Maximum resulting seismic forces according
maxFx= 69.66+0.30x 1.62= 70.14 [kN]
maxFy= 0.00+0.30x 0.00= 0.00 [kN]
Maximum horizontal force along the wall F=70.14 kN

1st floor **W10** Dimensions length= 6.00m, height= 3.00m, thickness=0.30m



Computation of horizontal wall stiffness in the wall plane

The wall is divided in 128 rectangular plane stress finite elements.
Nodes in a grid 9x17 , total 153 nodes.
Wall stiffness from finite element analysis K= 0.204 GN/m
Approximate wall stiffness (without openings) = 0.302 GN/m
Stiffness (x-x) Kx= 0.000 GN/m, (y-y) Ky= 0.204 GN/m

Seismic forces based on equivalent static horizontal loads
Wall eccentricities from building elastic axis, ex=0.04[m], ey=0.17[m]

Horizontal force Fx

Seismic direction x-x= 130.60x 0.000/ 0.945+ 0.17x 8x 0.000/ 21.375= 0.00[kN]
Seismic direction y-y= 0.00x 0.000/ 0.945+ 0.17x 24x 0.000/ 21.375= 0.00[kN]

Horizontal force Fy

Seismic direction x-x= 0.00x 0.204/ 0.720+ 0.04x 8x 0.204/ 21.375= 0.00[kN]
Seismic direction y-y= 130.60x 0.204/ 0.720+ 0.04x 24x 0.204/ 21.375= 37.01[kN]

Considering effect of seismic forces in x and y directions
(expFx)^2= 0.00^2+ 0.00^2, expFx= 0.00 [kN]
(expFy)^2= 0.00^2+ 37.01^2, expFy= 37.01 [kN]
Maximum resulting seismic forces according
maxFx= 0.00+0.30x 0.00= 0.00 [kN]
maxFy= 37.01+0.30x 0.00= 37.01 [kN]
Maximum horizontal force along the wall F=37.01 kN

Floor Shear Center (SC)

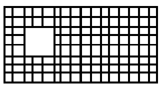
Wall	Kx[GN/m]	Ky[GN/m]	x[m]	y[m]	x.Ky	y.Kx	x^2.Ky	y^2.Kx
W6	0.000	0.261	2.00	7.00	0.522	0.000	1.044	0.000
W7	0.445	0.000	7.00	10.00	0.000	4.450	0.000	44.500
W8	0.000	0.255	12.00	7.00	3.060	0.000	36.720	0.000
W9	0.500	0.000	7.00	4.00	0.000	2.000	0.000	8.000
W10	0.000	0.204	7.00	7.00	1.428	0.000	9.996	0.000
Sum	**0.945[GN/m]**	**0.720[GN/m]**			**5.010**	**6.450**	**47.760**	**52.500**

Shear center x= 5.010/ 0.720= 6.96 m , y= 6.450/ 0.945= 6.83 m

Torsional resistanse of floor Ip=47.760+52.500-6.96?x0.720-6.83?x0.945=21.375[GNm]

The horizontal diaphragm of 1st floor is concidered the place of building elastic center (level approximate 0.8H). Building elastic axis at Po x=6.96[m], y=6.83[m]
Floor eccentricities eox=|7.02-6.96|=0.06[m], eoy=|7.01-6.83|=0.19[m]
Taking into account increase of eccentricities by a factor 0.00%
efx=1.00x0.06=0.06[m], erx=1.00x0.06=0.06[m], efy=1.00x0.19=0.19[m], ery=1.00x0.19=0.19[m]
Design eccenticities
maximum ex= 0.06[m], minimum ex= 0.06[m]
maximum ey= 0.19[m], minimum ey= 0.19[m]
Maximum rotational moments of horizontal floor load due to load eccentricities
Horizontal load direction x-x maxMzx= 0.06x 131= 8[kNm]
Horizontal load direction y-y maxMzy= 0.19x 131= 24[kNm]
Approximate relative horizontal floor movement dx=0.001x 130.6/ 0.945= 0.138 mm
Approximate relative horizontal floor movement dy=0.001x 130.6/ 0.720= 0.181 mm

Gr. floor **W1** Dimensions length= 6.00m, height= 3.00m, thickness=0.30m



Computation of horizontal wall stiffness in the wall plane
The wall is divided in 128 rectangular plane stress finite elements.
Nodes in a grid 9x17 , total 153 nodes.
Wall stiffness from finite element analysis K= 0.261 GN/m
Approximate wall stiffness (without openings) = 0.302 GN/m
Stiffness (x-x) Kx= 0.000 GN/m, (y-y) Ky= 0.261 GN/m

Seismic forces based on equivalent static horizontal loads
Wall eccentricities from building elastic axis, ex=4.96[m], ey=0.17[m]

Horizontal force Fx

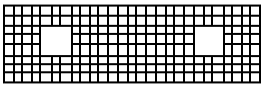
Seismic direction x-x= 198.00x 0.000/ 0.945+ 0.17x 15x 0.000/ 21.375= 0.00[kN]
Seismic direction y-y= 0.00x 0.000/ 0.945+ 0.17x 40x 0.000/ 21.375= 0.00[kN]

Horizontal force Fy

Seismic direction x-x= 0.00x 0.261/ 0.720+ 4.96x 15x 0.261/ 21.375= 0.93[kN]
Seismic direction y-y= 198.00x 0.261/ 0.720+ 4.96x 40x 0.261/ 21.375= 74.17[kN]

Considering effect of seismic forces in x and y directions
$(expFx)^2$= 0.00^2+ 0.00^2, expFx= 0.00 [kN]
$(expFy)^2$= 0.93^2+ 74.17^2, expFy= 74.18 [kN]
Maximum resulting seismic forces according
maxFx= 0.00+0.30x 0.00= 0.00 [kN]
maxFy= 74.17+0.30x 0.93= 74.45 [kN]
Maximum horizontal force along the wall F=74.45 kN

Gr. floor **W2** Dimensions length=10.00m, height= 3.00m, thickness=0.30m



Computation of horizontal wall stiffness in the wall plane

The wall is divided in 216 rectangular plane stress finite elements.
Nodes in a grid 9x28 , total 252 nodes.
Wall stiffness from finite element analysis K= 0.445 GN/m
Approximate wall stiffness (without openings) = 0.529 GN/m
Stiffness (x-x) Kx= 0.445 GN/m, (y-y) Ky= 0.000 GN/m

Seismic forces based on equivalent static horizontal loads
Wall eccentricities from building elastic axis, ex=0.04[m], ey=3.17[m]

Horizontal force Fx

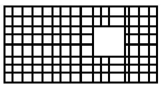
Seismic direction x-x= 198.00x 0.445/ 0.945+ 3.17x 15x 0.445/ 21.375= 94.25[kN]
Seismic direction y-y= 0.00x 0.445/ 0.945+ 3.17x 40x 0.445/ 21.375= 2.62[kN]

Horizontal force Fy

Seismic direction x-x= 0.00x 0.000/ 0.720+ 0.04x 15x 0.000/ 21.375= 0.00[kN]
Seismic direction y-y= 198.00x 0.000/ 0.720+ 0.04x 40x 0.000/ 21.375= 0.00[kN]

Considering effect of seismic forces in x and y directions
(expFx)^2= 94.25^2+ 2.62^2, expFx= 94.29 [kN]
(expFy)^2= 0.00^2+ 0.00^2, expFy= 0.00 [kN]
Maximum resulting seismic forces according
maxFx= 94.25+0.30x 2.62= 95.03 [kN]
maxFy= 0.00+0.30x 0.00= 0.00 [kN]
Maximum horizontal force along the wall F=95.03 kN

Gr. floor **W3** Dimensions length= 6.00m, height= 3.00m, thickness=0.30m



Computation of horizontal wall stiffness in the wall plane

The wall is divided in 128 rectangular plane stress finite elements.
Nodes in a grid 9x17 , total 153 nodes.
Wall stiffness from finite element analysis K= 0.255 GN/m
Approximate wall stiffness (without openings) = 0.302 GN/m
Stiffness (x-x) Kx= 0.000 GN/m, (y-y) Ky= 0.255 GN/m

Seismic forces based on equivalent static horizontal loads
Wall eccentricities from building elastic axis, ex=5.04[m], ey=0.17[m]

Horizontal force Fx

Seismic direction x-x= 198.00x 0.000/ 0.945+ 0.17x 15x 0.000/ 21.375= 0.00[kN]
Seismic direction y-y= 0.00x 0.000/ 0.945+ 0.17x 40x 0.000/ 21.375= 0.00[kN]

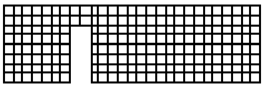
Horizontal force Fy

Seismic direction x-x= 0.00x 0.255/ 0.720+ 5.04x 15x 0.255/ 21.375= 0.92[kN]
Seismic direction y-y= 198.00x 0.255/ 0.720+ 5.04x 40x 0.255/ 21.375= 72.51[kN]

Considering effect of seismic forces in x and y directions
(expFx)^2= 0.00^2+ 0.00^2, expFx= 0.00 [kN]
(expFy)^2= 0.92^2+ 72.51^2, expFy= 72.51 [kN]
Maximum resulting seismic forces according
maxFx= 0.00+0.30x 0.00= 0.00 [kN]
maxFy= 72.51+0.30x 0.92= 72.78 [kN]
Maximum horizontal force along the wall F=72.78 kN

Gr. floor **W4** Dimensions length=10.00m, height= 3.00m, thickness=0.30m

Computation of horizontal wall stiffness in the wall plane



The wall is divided in 216 rectangular plane stress finite elements.
Nodes in a grid 9x28 , total 252 nodes.
Wall stiffness from finite element analysis K= 0.500 GN/m
Approximate wall stiffness (without openings) = 0.529 GN/m
Stiffness (x-x) Kx= 0.500 GN/m, (y-y) Ky= 0.000 GN/m

Seismic forces based on equivalent static horizontal loads
Wall eccentricities from building elastic axis, ex=0.04[m], ey=2.83[m]

Horizontal force Fx

Seismic direction x-x= 198.00x 0.500/ 0.945+ 2.83x 15x 0.500/ 21.375= 105.77[kN]
Seismic direction y-y= 0.00x 0.500/ 0.945+ 2.83x 40x 0.500/ 21.375= 2.62[kN]

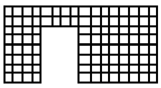
Horizontal force Fy

Seismic direction x-x= 0.00x 0.000/ 0.720+ 0.04x 15x 0.000/ 21.375= 0.00[kN]
Seismic direction y-y= 198.00x 0.000/ 0.720+ 0.04x 40x 0.000/ 21.375= 0.00[kN]

Considering effect of seismic forces in x and y directions
$(expFx)^2 = 105.77^2 + 2.62^2$, expFx= 105.81 [kN]
$(expFy)^2 = 0.00^2 + 0.00^2$, expFy= 0.00 [kN]
Maximum resulting seismic forces according
maxFx= 105.77+0.30x 2.62= 106.56 [kN]
maxFy= 0.00+0.30x 0.00= 0.00 [kN]
Maximum horizontal force along the wall F=106.56 kN

Gr. floor **W5** Dimensions length= 6.00m, height= 3.00m, thickness=0.30m

Computation of horizontal wall stiffness in the wall plane



The wall is divided in 128 rectangular plane stress finite elements.
Nodes in a grid 9x17 , total 153 nodes.
Wall stiffness from finite element analysis K= 0.204 GN/m
Approximate wall stiffness (without openings) = 0.302 GN/m
Stiffness (x-x) Kx= 0.000 GN/m, (y-y) Ky= 0.204 GN/m

Seismic forces based on equivalent static horizontal loads
Wall eccentricities from building elastic axis, ex=0.04[m], ey=0.17[m]

Horizontal force Fx

Seismic direction x-x= 198.00x 0.000/ 0.945+ 0.17x 15x 0.000/ 21.375= 0.00[kN]
Seismic direction y-y= 0.00x 0.000/ 0.945+ 0.17x 40x 0.000/ 21.375= 0.00[kN]

Horizontal force Fy

Seismic direction x-x= 0.00x 0.204/ 0.720+ 0.04x 15x 0.204/ 21.375= 0.01[kN]
Seismic direction y-y= 198.00x 0.204/ 0.720+ 0.04x 40x 0.204/ 21.375= 56.12[kN]

Considering effect of seismic forces in x and y directions
$(expFx)^2 = 0.00^2 + 0.00^2$, expFx= 0.00 [kN]
$(expFy)^2 = 0.01^2 + 56.12^2$, expFy= 56.12 [kN]
Maximum resulting seismic forces according
maxFx= 0.00+0.30x 0.00= 0.00 [kN]
maxFy= 56.12+0.30x 0.01= 56.12 [kN]
Maximum horizontal force along the wall F=56.12 kN

Floor Shear Center (SC)

Wall	Kx[GN/m]	Ky[GN/m]	x[m]	y[m]	x.Ky	y.Kx	x².Ky	y².Kx
W1	0.000	0.261	2.00	7.00	0.522	0.000	1.044	0.000
W2	0.445	0.000	7.00	10.00	0.000	4.450	0.000	44.500
W3	0.000	0.255	12.00	7.00	3.060	0.000	36.720	0.000
W4	0.500	0.000	7.00	4.00	0.000	2.000	0.000	8.000
W5	0.000	0.204	7.00	7.00	1.428	0.000	9.996	0.000
Sum	**0.945[GN/m]**	**0.720[GN/m]**			**5.010**	**6.450**	**47.760**	**52.500**

Shear center x= 5.010/ 0.720= 6.96 m , y= 6.450/ 0.945= 6.83 m

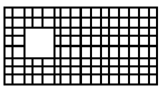
Torsional resistanse of floor Ip=47.760+52.500-6.96?x0.720-6.83?x0.945=21.375[GNm]

The horizontal diaphragm of 1st floor is concidered the place of building elastic center (level approximate 0.8H). Building elastic axis at Po x=6.96[m], y=6.83[m]
Floor eccentricities eox=|7.04-6.96|=0.08[m], eoy=|7.03-6.83|=0.20[m]
Taking into account increase of eccentricities by a factor 0.00%
efx=1.00x0.08=0.08[m], erx=1.00x0.08=0.08[m], efy=1.00x0.20=0.20[m], ery=1.00x0.20=0.20[m]
Design ecenticities
maximum ex= 0.08[m], minimum ex= 0.08[m]
maximum ey= 0.20[m], minimum ey= 0.20[m]
Maximum rotational moments of horizontal floor load due to load eccentricities
Horizontal load direction x-x maxMzx= 0.08x 198= 15[kNm]
Horizontal load direction y-y maxMzy= 0.20x 198= 40[kNm]
Approximate relative horizontal floor movement dx=0.001x 198.0/ 0.945= 0.210 mm
Approximate relative horizontal floor movement dy=0.001x 198.0/ 0.720= 0.275 mm

Design strength of masonry (Eurocode 6, §4)

The design of masonry walls is done in the ultimate limit state based on EC6, §4.
The load combinations which are used are :
a) Verification of strength in compression for load 1.35xg+1.50xq, Nsd<=Nrd (EC6 §4.4.2)
b) Verification of strength in compression for load 1.0xg+0.30xq+Earthquake, Nsd<=Nrd (EC6 §4.4.2)
c) Verification of shear strength for load 1.0xg+0.30xq+Earthquake, Vsd<=Vrd (EC 6 §4.5.3)
The slenderness ratio for the masonry walls is also checked hef/tef<27 (EC 6 §4.4.6.)
and verification of strength if the places of concentrated loads according to EC 6 §4.4.8
Verifications are also done for the requirement on geometry, thickness, slenderness,
wall height and thickness, according Eurocode 6.
In any case the design loads Nsd, or Vsd are computed as load per unit length of the masonry wall
from the maximum stresses which are computed from the finite element analysis of the masonry wall.
The eccentricities for reduction factors (Phi)i and (Phi)m are computed with accuracy
from the structural loads, based on EC 6 §4.4.3. The moments Mi at the top
of each masonry wall are computed from the slab loads (EC 6 Annex. C). The eccentricities eh are
computed from the maximum computed horizontal wall relative movement due to earthquake loading.
The effective wall height is computed (EC 6 §4.4.4.3) taking into account the wall restraint
conditions at the top an bottom of the masonry wall. For the free edges due to openings on the walls
the coefficients (ro)3, and (ro)4 are taken to be (ro)3=(ro)4=1 as most unfavorable.
The out of plane eccentricity due to imperfections is taken as es=hef/450 (EC 6 §4.4.7.2)

1st floor **W6** Dimensions length= 6.00m, height= 3.00m, thickness=0.30m



YTONG-M5 0.30
Category of execution A (EC-6, Annex ?)
Category of masonry units construction I (EC 6, §3.1.1)
Compressive strength of masonry fk= 1.64 N/mm^2

Partial safety factors for material properties (gammaM)=1.70 (EC6, Table.2.3)
Effective Length hef=(ro)xh=0.75x 3.00= 2.25 m (EC 6, §4.4.4.3)
Slenderness ratio hef/tef= 2.25/0.30= 7.50<27 OK (EC 6, §4.4.6)

Strength verification in axial compression, load 1.35xg+1.50xq (EC 6, §4.4.2)

Masonry strength verification at the top of the masonry
Vertical design load at the top Ni=(1.35x57.4+1.50x13.2)/6.00= 16.22 kN/m
Average compression design stress at the top fsdo=0.001x16.22/0.30= 0.054 N/mm^2
fsdo= 0.054<0.25N/mm^2 so (EC 6 Annex C.2) the eccentricity is reduced by (1-k/4)=0.66
Design bending moment at the top (EC 6, Annex C.1) Mi= 4.56 kNm/m
Eccentricity based on EC 6 Annex C.1 Mi/Ni= 4.56/ 16.22=0.28120 m =0.94x(wall thichkness)
The eccentricity is >40% of the masonry wall thickness. It is computed as in EC 6 Annex C.4
bearing depth =0.20x0.30=0.06m. Eccentricity of load at the top Mi/Ni=0.12000 m
Eccentricity at the top due to horizontal loads ehi=0.00000 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity at the top (EC 6 §4.4.3) ei=Mi/Ni+ehi+es =0.12000+0.00000+0.00500=0.12500 m
Minimum eccendricity (EC6, §4.4.3) ei=0.05t=0.05x0.30=0.01500 m
Reduction factor at the top (EC 6, §4.4.3) (Phi)i=1-2ei/t=1-2x0.12500/0.30=0.17
Design vertical load resistance Nrd=(Phi)im.t.fk/(gammaM)=1000x0.17x0.30x1.64/1.70= 49.1kN/m
Nsd= 16.2 < 49.1=Nrd. The ultimate limit state for vertical loading is verified
Masonry strength verification at the middle fifth of the height
Vertical design load in the middle fifth of the height Nm=(16.22+0.40x1.35x39.7/6.00)= 19.79 kN/m
Average compressive design stress in the middle fifth of the height fsdo=0.001x19.79/0.30=0.066 N/mm^2
Eccentricity of floor load in the middle fifth of the height Mm/Nm=0.20x0.12000x16.2/19.8=0.01967 m
Eccentricity in the middle fifth of the height due to horizontal forces ehm=0.00000 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity due to loads em=(Mm/Nm)+ehm+es=0.01967+0.00000+0.00500=0.02467 m
The slenderness ratio is <=15 so (EC 6 §4.4.3.2) eccentricity due to creep ek=0
Eccentricity in the middle fifth of the height (EC 6 §4.4.3) em=em+ek=0.02467+0.00000=0.02467 m
Minimum eccentricity (EC 6, §4.4.3) em=0.05t=0.05x0.30=0.01500 m
Reduction factor in the middle fifth of the height (EC 6, §4.4.3) (Phi)m=1-2em/t=1-2x0.025/0.30=0.84

Vertical design load at wall base Ni=(1.35x97.1+1.50x13.2)/6.00= 25.15 kN/m
Avearge compressive design stress at wall base qsdo=0.001x25.15/0.30= 0.084 N/mm²
Eccentricity of load at wall base Mi/Ni=0.12000x16.2/25.1=0.07738 m
Eccentricity at wall base due to horizontal loads ehi=0.00000 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity at wall base (EC 6 §4.4.3) ei=Mi/Ni+ehi+es =0.07738+0.00000+0.00500=0.08238 m
Minimum eccendricity (EC6, §4.4.3) ei=0.05t=0.05x0.30=0.01500 m
Reduction factor at wall base (EC 6, §4.4.3) (Phi)i=1-2ei/t=1-2x0.08238/0.30=0.45
Design vertical load resistance Nrd=(Phi)im.t.fk/(gammaM)=1000x0.45x0.30x1.64/1.70= 129.9kN/m
Maximum compressive stress from finite element analysis maxfn=-0.134 N/mm²
(the maximum stress which is obtained from finite element analysis is for stress regions outside the regions of stresses concentrations at the floor beam supports, which are checked separately)
Maximum design vertical load per unit length Nsd=1000x0.30x 0.134= 40.2 kN/m
Nsd= 40.2 < 129.9=Nrd. The ultimate limit state for vertical loading is verified

Strength verification in axial compression, load 1.00xg+0.30xq+Earthquake (EC 6, §4.4.2)

Masonry strength verification at the top of the masonry

Vertical design load at the top Ni=(1.00x57.4+0.30x13.2)/6.00= 10.23 kN/m
Average compression design stress at the top fsdo=0.001x10.23/0.30= 0.034 N/mm²
fsdo= 0.034<0.25N/mm² so (EC 6 Annex C.2) the eccentricity is reduced by (1-k/4)=0.66
Design bending moment at the top (EC 6, Annex C.1) Mi= 2.92 kNm/m
Eccentricity based on EC 6 Annex C.1 Mi/Ni= 2.92/ 10.23=0.28553 m =0.95x(wall thichkness)
The eccentricity is >40% of the masonry wall thickness. It is computed as in EC 6 Annex C.4 bearing depth =0.20x0.30=0.06m. Eccentricity of load at the top Mi/Ni=0.12000 m
Eccentricity at the top due to horizontal loads ehi=0.00018 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity at the top (EC 6 §4.4.3) ei=Mi/Ni+ehi+es =0.12000+0.00018+0.00500=0.12518 m
Minimum eccendricity (EC6, §4.4.3) ei=0.05t=0.05x0.30=0.01500 m
Reduction factor at the top (EC 6, §4.4.3) (Phi)i=1-2ei/t=1-2x0.12518/0.30=0.17
Design vertical load resistance Nrd=(Phi)im.t.fk/(gammaM)=1000x0.17x0.30x1.64/1.70= 49.1kN/m
Nsd= 10.2 < 49.1=Nrd. The ultimate limit state for vertical loading is verified

Masonry strength verification at the middle fifth of the height

Vertical design load in the middle fifth of the height Nm=(10.23+0.40x1.00x39.7/6.00)= 12.87 kN/m
Average compressive design stress in the middle fifth of the height fsdo=0.001x12.87/0.30=0.043 N/mm²
Eccentricity of floor load in the middle fifth of the height Mm/Nm=0.20x0.12000x10.2/12.9=0.01907 m
Eccentricity in the middle fifth of the height due to horizontal forces ehm=0.00009 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity due to loads em=(Mm/Nm)+ehm+es=0.01907+0.00009+0.00500=0.02416 m
The slenderness ratio is <=15 so (EC 6 §4.4.3.2) eccentricity due to creep ek=0
Eccentricity in the middle fifth of the height (EC 6 §4.4.3) em=em+ek=0.02416+0.00000=0.02416 m
Minimum eccentricity (EC 6, §4.4.3) em=0.05t=0.05x0.30=0.01500 m
Reduction factor in the middle fifth of the height (EC 6, §4.4.3) (Phi)m=1-2em/t=1-2x0.024/0.30=0.84
Vertical design load at wall base Ni=(1.00x97.1+0.30x13.2)/6.00= 16.84 kN/m
Avearge compressive design stress at wall base qsdo=0.001x16.84/0.30= 0.056 N/mm²
Eccentricity of load at wall base Mi/Ni=0.12000x10.2/16.8=0.07286 m
Eccentricity at wall base due to horizontal loads ehi=0.00018 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity at wall base (EC 6 §4.4.3) ei=Mi/Ni+ehi+es =0.07286+0.00018+0.00500=0.07804 m
Minimum eccendricity (EC6, §4.4.3) ei=0.05t=0.05x0.30=0.01500 m
Reduction factor at wall base (EC 6, §4.4.3) (Phi)i=1-2ei/t=1-2x0.07804/0.30=0.48
Design vertical load resistance Nrd=(Phi)im.t.fk/(gammaM)=1000x0.48x0.30x1.64/1.70= 138.6kN/m
Maximum compressive stress from finite element analysis maxfn=-0.187 N/mm²
(the maximum stress which is obtained from finite element analysis is for stress regions outside the regions of stresses concentrations at the floor beam supports, which are checked separately)
Maximum design vertical load per unit length Nsd=1000x0.30x 0.187= 56.1 kN/m
Nsd= 56.1 < 138.6=Nrd. The ultimate limit state for vertical loading is verified

Strength verification in shear, load 1.00xg+0.30xq+Earthquake (EC 6.§4.5.3)

Maximum shearing stress from finite element solution (tau)max= 0.053 N/mm^2
(the maximum stress which is obtained from finite element analysis is for stress regions outside the regions of stresses concentrations at the floor beam supports, which are checked separately)
Maximum design shear load per unit length Vsd=1000x0.30x 0.053= 16.0 KN/m
Characteristic shear strength fvk=0.70x(fvko+0.4xfd) (EC 6 §3.6.3 and 3.6.3.(8))
fvk=0.70x(0.150+0.4x 0.187)= 0.157 N/mm^2, maxfvk= 1.000 N/mm^2
Design shear resistance of masonry Vrd=fvk.t./(gammaM) (EC 6 §4.5.3)
Vrd=1000x 0.157x0.30/1.70= 27.8. kN/m
Vsd= 16.0 < 27.8=Vrd. The ultimate limit state for shear loading is verified.

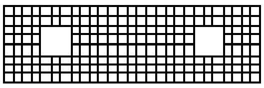
Verification of regions with concentrated loads (EC 6, §4.4.8)

From the finite element solution is obtained at the support positions of the floor beams at the top the maximum compressive stresses. Because at the wall top allways exists a bond beam from reinforced concrete, we check the stresses in the finite elements a line bellow.
The maximum compressive stress in the region of stress concentration is fsdmax= 0.143N/mm^2
This maximum stress 0.143 is < fk/(gammaM)= 1.64/1.7= 0.96
The strength requirements for concentrated loads are verified (EC 6 §4.4.8)

Top beam of reinforced concrete (EC 6 §5.2)

On the top of the wall and the top of the openings, some small tensile stresses are taken from reinforced concrete bond beams or lintels at these places 30x20 (width x height in cm), with minimum reinforcement 4#12 (stirrups #8/20) which satisfies the minimum code requirements.

1st floor **W7** Dimensions length=10.00m, height= 3.00m, thickness=0.30m



YTONG-M5 0.30
Category of execution A (EC-6, Annex ?)
Category of masonry units construction I (EC 6, §3.1.1)
Compressive strength of masonry fk= 1.64 N/mm^2

Partial safety factors for material properties (gammaM)=1.70 (EC6, Table.2.3)
Effective Length hef=(ro)xh=0.75x 3.00= 2.25 m (EC 6, §4.4.4.3)
Slenderness ratio hef/tef= 2.25/0.30= 7.50<27 OK (EC 6, §4.4.6)

Strength verification in axial compression, load 1.35xg+1.50xq (EC 6, §4.4.2)

Masonry strength verification at the top of the masonry

Vertical design load at the top Ni=(1.35x27.6+1.50x5.8)/10.00= 4.60 kN/m
Average compression design stress at the top fsdo=0.001x4.60/0.30= 0.015 N/mm^2
fsdo= 0.015<0.25N/mm^2 so (EC 6 Annex C.2) the eccentricity is reduced by (1-k/4)=0.71
Design bending moment at the top (EC 6, Annex C.1) Mi= 1.53 kNm/m
Eccentricity based on EC 6 Annex C.1 Mi/Ni= 1.53/ 4.60=0.33298 m =1.11x(wall thichkness)
The eccentricity is >40% of the masonry wall thickness. It is computed as in EC 6 Annex C.4 bearing depth =0.20x0.30=0.06m. Eccentricity of load at the top Mi/Ni=0.12000 m
Eccentricity at the top due to horizontal loads ehi=0.00000 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity at the top (EC 6 §4.4.3) ei=Mi/Ni+ehi+es =0.12000+0.00000+0.00500=0.12500 m
Minimum eccendricity (EC6, §4.4.3) ei=0.05t=0.05x0.30=0.01500 m
Reduction factor at the top (EC 6, §4.4.3) (Phi)i=1-2ei/t=1-2x0.12500/0.30=0.17
Design vertical load resistance Nrd=(Phi)im.t.fk/(gammaM)=1000x0.17x0.30x1.64/1.70= 49.1kN/m
Nsd= 4.6 < 49.1=Nrd. The ultimate limit state for vertical loading is verified

Masonry strength verification at the middle fifth of the height

Vertical design load in the middle fifth of the height Nm=(4.60+0.40x1.35x65.1/10.00)= 8.11 kN/m
Average compressive design stress in the middle fifth of the height fsdo=0.001x8.11/0.30=0.027 N/mm^2
Eccentricity of floor load in the middle fifth of the height Mm/Nm=0.20x0.12000x4.6/8.1=0.01360 m
Eccentricity in the middle fifth of the height due to horizontal forces ehm=0.00000 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity due to loads em=(Mm/Nm)+ehm+es=0.01360+0.00000+0.00500=0.01860 m
The slenderness ratio is <=15 so (EC 6 §4.4.3.2) eccentricity due to creep ek=0
Eccentricity in the middle fifth of the height (EC 6 §4.4.3) em=em+ek=0.01860+0.00000=0.01860 m
Minimum eccentricity (EC 6, §4.4.3) em=0.05t=0.05x0.30=0.01500 m
Reduction factor in the middle fifth of the height (EC 6, §4.4.3) (Phi)m=1-2em/t=1-2x0.019/0.30=0.88

Vertical design load at wall base Ni=(1.35x92.7+1.50x5.8)/10.00= 13.38 kN/m
Avearge compressive design stress at wall base qsdo=0.001x13.38/0.30= 0.045 N/mm^2
Eccentricity of load at wall base Mi/Ni=0.12000x4.6/13.4=0.04121 m
Eccentricity at wall base due to horizontal loads ehi=0.00000 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity at wall base (EC 6 §4.4.3) ei=Mi/Ni+ehi+es =0.04121+0.00000+0.00500=0.04621 m
Minimum eccendricity (EC6, §4.4.3) ei=0.05t=0.05x0.30=0.01500 m
Reduction factor at wall base (EC 6, §4.4.3) (Phi)i=1-2ei/t=1-2x0.04621/0.30=0.69
Design vertical load resistance Nrd=(Phi)im.t.fk/(gammaM)=1000x0.69x0.30x1.64/1.70= 199.2kN/m
Maximum compressive stress from finite element analysis maxfn=-0.064 N/mm^2
(the maximum stress which is obtained from finite element analysis is for stress regions outside the regions of stresses concentrations at the floor beam supports, which are checked separately)
Maximum design vertical load per unit length Nsd=1000x0.30x 0.064= 19.1 kN/m
Nsd= 19.1 < 199.2=Nrd. The ultimate limit state for vertical loading is verified

Strength verification in axial compression, load 1.00xg+0.30xq+Earthquake (EC 6, §4.4.2)

Masonry strength verification at the top of the masonry

Vertical design load at the top Ni=(1.00x27.6+0.30x5.8)/10.00= 2.93 kN/m
Average compression design stress at the top fsdo=0.001x2.93/0.30= 0.010 N/mm^2
fsdo= 0.010<0.25N/mm^2 so (EC 6 Annex C.2) the eccentricity is reduced by (1-k/4)=0.71
Design bending moment at the top (EC 6, Annex C.1) Mi= 0.98 kNm/m
Eccentricity based on EC 6 Annex C.1 Mi/Ni= 0.98/ 2.93=0.33261 m =1.11x(wall thichkness)
The eccentricity is >40% of the masonry wall thickness. It is computed as in EC 6 Annex C.4 bearing depth =0.20x0.30=0.06m. Eccentricity of load at the top Mi/Ni=0.12000 m
Eccentricity at the top due to horizontal loads ehi=0.00018 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity at the top (EC 6 §4.4.3) ei=Mi/Ni+ehi+es =0.12000+0.00018+0.00500=0.12518 m
Minimum eccendricity (EC6, §4.4.3) ei=0.05t=0.05x0.30=0.01500 m
Reduction factor at the top (EC 6, §4.4.3) (Phi)i=1-2ei/t=1-2x0.12518/0.30=0.17
Design vertical load resistance Nrd=(Phi)im.t.fk/(gammaM)=1000x0.17x0.30x1.64/1.70= 49.1kN/m
Nsd= 2.9 < 49.1=Nrd. The ultimate limit state for vertical loading is verified

Masonry strength verification at the middle fifth of the height

Vertical design load in the middle fifth of the height Nm=(2.93+0.40x1.00x65.1/10.00)= 5.54 kN/m
Average compressive design stress in the middle fifth of the height fsdo=0.001x5.54/0.30=0.018 N/mm^2
Eccentricity of floor load in the middle fifth of the height Mm/Nm=0.20x0.12000x2.9/5.5=0.01272 m
Eccentricity in the middle fifth of the height due to horizontal forces ehm=0.00009 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity due to loads em=(Mm/Nm)+ehm+es=0.01272+0.00009+0.00500=0.01781 m
The slenderness ratio is <=15 so (EC 6 §4.4.3.2) eccentricity due to creep ek=0
Eccentricity in the middle fifth of the height (EC 6 §4.4.3) em=em+ek=0.01781+0.00000=0.01781 m
Minimum eccentricity (EC 6, §4.4.3) em=0.05t=0.05x0.30=0.01500 m
Reduction factor in the middle fifth of the height (EC 6, §4.4.3) (Phi)m=1-2em/t=1-2x0.018/0.30=0.88
Vertical design load at wall base Ni=(1.00x92.7+0.30x5.8)/10.00= 9.44 kN/m
Avearge compressive design stress at wall base qsdo=0.001x9.44/0.30= 0.031 N/mm^2
Eccentricity of load at wall base Mi/Ni=0.12000x2.9/9.4=0.03728 m
Eccentricity at wall base due to horizontal loads ehi=0.00018 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity at wall base (EC 6 §4.4.3) ei=Mi/Ni+ehi+es =0.03728+0.00018+0.00500=0.04246 m
Minimum eccendricity (EC6, §4.4.3) ei=0.05t=0.05x0.30=0.01500 m
Reduction factor at wall base (EC 6, §4.4.3) (Phi)i=1-2ei/t=1-2x0.04246/0.30=0.72
Design vertical load resistance Nrd=(Phi)im.t.fk/(gammaM)=1000x0.72x0.30x1.64/1.70= 207.9kN/m
Maximum compressive stress from finite element analysis maxfn=-0.122 N/mm^2
(the maximum stress which is obtained from finite element analysis is for stress regions outside the regions of stresses concentrations at the floor beam supports, which are checked separately)
Maximum design vertical load per unit length Nsd=1000x0.30x 0.122= 36.6 kN/m
Nsd= 36.6 < 207.9=Nrd. The ultimate limit state for vertical loading is verified

Strength verification in shear, load 1.00xg+0.30xq+Earthquake (EC 6.§4.5.3)

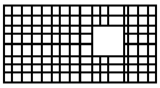
Maximum shearing stress from finite element solution (tau)max= 0.037 N/mm^2
(the maximum stress which is obtained from finite element analysis is for stress regions outside the regions of stresses concentrations at the floor beam supports, which are checked separately)
Maximum design shear load per unit length Vsd=1000x0.30x 0.037= 11.0 KN/m
Characteristic shear strength fvk=0.70x(fvko+0.4xfd) (EC 6 §3.6.3 and 3.6.3.(8))
fvk=0.70x(0.150+0.4x 0.122)= 0.139 N/mm^2, maxfvk= 1.000 N/mm^2
Design shear resistance of masonry Vrd=fvk.t./(gammaM) (EC 6 §4.5.3)
Vrd=1000x 0.150x0.30/1.70= 26.5. kN/m
Vsd= 11.0 < 26.5=Vrd. The ultimate limit state for shear loading is verified.

Top beam of reinforced concrete (EC 6 §5.2)

On the top of the wall and the top of the openings, some small tensile stresses are taken from reinforced concrete bond beams or lintels at these places 30x20 (width x height in cm), with minimum reinforcement 4#12 (stirrups #8/20) which satisfies the minimum code requirements.

1st floor W8

Dimensions length= 6.00m, height= 3.00m, thickness=0.30m



YTONG-M5 0.30
Category of execution A (EC-6, Annex ?)
Category of masonry units construction I (EC 6, §3.1.1)
Compressive strength of masonry fk= 1.64 N/mm^2

Partial safety factors for material properties (gammaM)=1.70 (EC6, Table.2.3)
Effective Length hef=(ro)xh=0.75x 3.00= 2.25 m (EC 6, §4.4.4.3)
Slenderness ratio hef/tef= 2.25/0.30= 7.50<27 OK (EC 6, §4.4.6)

Strength verification in axial compression, load 1.35xg+1.50xq (EC 6, §4.4.2)

Masonry strength verification at the top of the masonry

Vertical design load at the top Ni=(1.35x57.4+1.50x13.2)/6.00= 16.22 kN/m
Average compression design stress at the top fsdo=0.001x16.22/0.30= 0.054 N/mm^2
fsdo= 0.054<0.25N/mm^2 so (EC 6 Annex C.2) the eccentricity is reduced by (1-k/4)=0.66
Design bending moment at the top (EC 6, Annex C.1) Mi= 4.56 kNm/m
Eccentricity based on EC 6 Annex C.1 Mi/Ni= 4.56/ 16.22=0.28120 m =0.94x(wall thichkness)
The eccentricity is >40% of the masonry wall thickness. It is computed as in EC 6 Annex C.4 bearing depth =0.20x0.30=0.06m. Eccentricity of load at the top Mi/Ni=0.12000 m
Eccentricity at the top due to horizontal loads ehi=0.00000 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity at the top (EC 6 §4.4.3) ei=Mi/Ni+ehi+es =0.12000+0.00000+0.00500=0.12500 m
Minimum eccendricity (EC6, §4.4.3) ei=0.05t=0.05x0.30=0.01500 m
Reduction factor at the top (EC 6, §4.4.3) (Phi)i=1-2ei/t=1-2x0.12500/0.30=0.17
Design vertical load resistance Nrd=(Phi)im.t.fk/(gammaM)=1000x0.17x0.30x1.64/1.70= 49.1kN/m
Nsd= 16.2 < 49.1=Nrd. The ultimate limit state for vertical loading is verified

Masonry strength verification at the middle fifth of the height

Vertical design load in the middle fifth of the height Nm=(16.22+0.40x1.35x39.7/6.00)= 19.79 kN/m
Average compressive design stress in the middle fifth of the height fsdo=0.001x19.79/0.30=0.066 N/mm^2
Eccentricity of floor load in the middle fifth of the height Mm/Nm=0.20x0.12000x16.2/19.8=0.01967 m
Eccentricity in the middle fifth of the height due to horizontal forces ehm=0.00000 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity due to loads em=(Mm/Nm)+ehm+es=0.01967+0.00000+0.00500=0.02467 m
The slenderness ratio is <=15 so (EC 6 §4.4.3.2) eccentricity due to creep ek=0
Eccentricity in the middle fifth of the height (EC 6 §4.4.3) em=em+ek=0.02467+0.00000=0.02467 m
Minimum eccentricity (EC 6, §4.4.3) em=0.05t=0.05x0.30=0.01500 m
Reduction factor in the middle fifth of the height (EC 6, §4.4.3) (Phi)m=1-2em/t=1-2x0.025/0.30=0.84

Vertical design load at wall base Ni=(1.35x97.1+1.50x13.2)/6.00= 25.15 kN/m
Avearge compressive design stress at wall base qsdo=0.001x25.15/0.30= 0.084 N/mm^2
Eccentricity of load at wall base Mi/Ni=0.12000x16.2/25.1=0.07738 m
Eccentricity at wall base due to horizontal loads ehi=0.00000 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity at wall base (EC 6 §4.4.3) ei=Mi/Ni+ehi+es =0.07738+0.00000+0.00500=0.08238 m
Minimum eccendricity (EC6, §4.4.3) ei=0.05t=0.05x0.30=0.01500 m
Reduction factor at wall base (EC 6, §4.4.3) (Phi)i=1-2ei/t=1-2x0.08238/0.30=0.45
Design vertical load resistance Nrd=(Phi)im.t.fk/(gammaM)=1000x0.45x0.30x1.64/1.70= 129.9kN/m
Maximum compressive stress from finite element analysis maxfn=-0.136 N/mm^2
(the maximum stress which is obtained from finite element analysis is for stress regions outside the regions of stresses concentrations at the floor beam supports, which are checked separately)
Maximum design vertical load per unit length Nsd=1000x0.30x 0.136= 40.7 kN/m
Nsd= 40.7 < 129.9=Nrd. The ultimate limit state for vertical loading is verified

Strength verification in axial compression, load 1.00xg+0.30xq+Earthquake (EC 6, §4.4.2)

Masonry strength verification at the top of the masonry

Vertical design load at the top Ni=(1.00x57.4+0.30x13.2)/6.00= 10.23 kN/m
Average compression design stress at the top fsdo=0.001x10.23/0.30= 0.034 N/mm^2
fsdo= 0.034<0.25N/mm^2 so (EC 6 Annex C.2) the eccentricity is reduced by (1-k/4)=0.66
Design bending moment at the top (EC 6, Annex C.1) Mi= 2.92 kNm/m
Eccentricity based on EC 6 Annex C.1 Mi/Ni= 2.92/ 10.23=0.28553 m =0.95x(wall thichkness)
The eccentricity is >40% of the masonry wall thickness. It is computed as in EC 6 Annex C.4 bearing depth =0.20x0.30=0.06m. Eccentricity of load at the top Mi/Ni=0.12000 m
Eccentricity at the top due to horizontal loads ehi=0.00018 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity at the top (EC 6 §4.4.3) ei=Mi/Ni+ehi+es =0.12000+0.00018+0.00500=0.12518 m
Minimum eccendricity (EC6, §4.4.3) ei=0.05t=0.05x0.30=0.01500 m
Reduction factor at the top (EC 6, §4.4.3) (Phi)i=1-2ei/t=1-2x0.12518/0.30=0.17
Design vertical load resistance Nrd=(Phi)im.t.fk/(gammaM)=1000x0.17x0.30x1.64/1.70= 49.1kN/m
Nsd= 10.2 < 49.1=Nrd. The ultimate limit state for vertical loading is verified

Masonry strength verification at the middle fifth of the height

Vertical design load in the middle fifth of the height Nm=(10.23+0.40x1.00x39.7/6.00)= 12.87 kN/m
Average compressive design stress in the middle fifth of the height fsdo=0.001x12.87/0.30=0.043 N/mm^2
Eccentricity of floor load in the middle fifth of the height Mm/Nm=0.20x0.12000x10.2/12.9=0.01907 m
Eccentricity in the middle fifth of the height due to horizontal forces ehm=0.00009 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity due to loads em=(Mm/Nm)+ehm+es=0.01907+0.00009+0.00500=0.02416 m
The slenderness ratio is <=15 so (EC 6 §4.4.3.2) eccentricity due to creep ek=0
Eccentricity in the middle fifth of the height (EC 6 §4.4.3) em=em+ek=0.02416+0.00000=0.02416 m
Minimum eccentricity (EC 6, §4.4.3) em=0.05t=0.05x0.30=0.01500 m
Reduction factor in the middle fifth of the height (EC 6, §4.4.3) (Phi)m=1-2em/t=1-2x0.024/0.30=0.84
Vertical design load at wall base Ni=(1.00x97.1+0.30x13.2)/6.00= 16.84 kN/m
Avearge compressive design stress at wall base qsdo=0.001x16.84/0.30= 0.056 N/mm^2
Eccentricity of load at wall base Mi/Ni=0.12000x10.2/16.8=0.07286 m
Eccentricity at wall base due to horizontal loads ehi=0.00018 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity at wall base (EC 6 §4.4.3) ei=Mi/Ni+ehi+es =0.07286+0.00018+0.00500=0.07804 m
Minimum eccendricity (EC6, §4.4.3) ei=0.05t=0.05x0.30=0.01500 m
Reduction factor at wall base (EC 6, §4.4.3) (Phi)i=1-2ei/t=1-2x0.07804/0.30=0.48
Design vertical load resistance Nrd=(Phi)im.t.fk/(gammaM)=1000x0.48x0.30x1.64/1.70= 138.6kN/m
Maximum compressive stress from finite element analysis maxfn=-0.189 N/mm^2
(the maximum stress which is obtained from finite element analysis is for stress regions outside the regions of stresses concentrations at the floor beam supports, which are checked separately)
Maximum design vertical load per unit length Nsd=1000x0.30x 0.189= 56.8 kN/m
Nsd= 56.8 < 138.6=Nrd. The ultimate limit state for vertical loading is verified

Strength verification in shear, load 1.00xg+0.30xq+Earthquake (EC 6.§4.5.3)

Maximum shearing stress from finite element solution (tau)max= 0.065 N/mm^2
(the maximum stress which is obtained from finite element analysis is for stress regions outside the regions of stresses concentrations at the floor beam supports, which are checked separately)
Maximum design shear load per unit length Vsd=1000x0.30x 0.065= 19.6 KN/m
Characteristic shear strength fvk=0.70x(fvko+0.4xfd) (EC 6 §3.6.3 and 3.6.3.(8))
fvk=0.70x(0.150+0.4x 0.189)= 0.158 N/mm^2, maxfvk= 1.000 N/mm^2
Design shear resistance of masonry Vrd=fvk.t./(gammaM) (EC 6 §4.5.3)
Vrd=1000x 0.158x0.30/1.70= 27.9. kN/m
Vsd= 19.6 < 27.9=Vrd. The ultimate limit state for shear loading is verified.

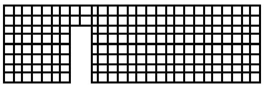
Verification of regions with concentrated loads (EC 6, §4.4.8)

From the finite element solution is obtained at the support positions of the floor beams at the top the maximum compressive stresses. Because at the wall top allways exists a bond beam from reinforced concrete, we check the stresses in the finite elements a line bellow.
The maximum compressive stress in the region of stress concentration is fsdmax= 0.151N/mm^2
This maximum stress 0.151 is < fk/(gammaM)= 1.64/1.7= 0.96
The strength requirements for concentrated loads are verified (EC 6 §4.4.8)

Top beam of reinforced concrete (EC 6 §5.2)

On the top of the wall and the top of the openings, some small tensile stresses are taken from reinforced concrete bond beams or lintels at these places 30x20 (width x height in cm), with minimum reinforcement 4#12 (stirrups #8/20) which satisfies the minimum code requirements.

1st floor **W9** Dimensions length=10.00m, height= 3.00m, thickness=0.30m



YTONG-M5 0.30
Category of execution A (EC-6, Annex ?)
Category of masonry units construction I (EC 6, §3.1.1)
Compressive strength of masonry fk= 1.64 N/mm^2

Partial safety factors for material properties (gammaM)=1.70 (EC6, Table.2.3)
Effective Length hef=(ro)xh=0.75x 3.00= 2.25 m (EC 6, §4.4.4.3)
Slenderness ratio hef/tef= 2.25/0.30= 7.50<27 OK (EC 6, §4.4.6)

Strength verification in axial compression, load 1.35xg+1.50xq (EC 6, §4.4.2)

Masonry strength verification at the top of the masonry

Vertical design load at the top Ni=(1.35x27.6+1.50x5.8)/10.00= 4.60 kN/m
Average compression design stress at the top fsdo=0.001x4.60/0.30= 0.015 N/mm^2
fsdo= 0.015<0.25N/mm^2 so (EC 6 Annex C.2) the eccentricity is reduced by (1-k/4)=0.71
Design bending moment at the top (EC 6, Annex C.1) Mi= 1.53 kNm/m
Eccentricity based on EC 6 Annex C.1 Mi/Ni= 1.53/ 4.60=0.33298 m =1.11x(wall thichkness)
The eccentricity is >40% of the masonry wall thickness. It is computed as in EC 6 Annex C.4
bearing depth =0.20x0.30=0.06m. Eccentricity of load at the top Mi/Ni=0.12000 m
Eccentricity at the top due to horizontal loads ehi=0.00000 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity at the top (EC 6 §4.4.3) ei=Mi/Ni+ehi+es =0.12000+0.00000+0.00500=0.12500 m
Minimum eccendricity (EC6, §4.4.3) ei=0.05t=0.05x0.30=0.01500 m
Reduction factor at the top (EC 6, §4.4.3) (Phi)i=1-2ei/t=1-2x0.12500/0.30=0.17
Design vertical load resistance Nrd=(Phi)im.t.fk/(gammaM)=1000x0.17x0.30x1.64/1.70= 49.1kN/m
Nsd= 4.6 < 49.1=Nrd. The ultimate limit state for vertical loading is verified

Masonry strength verification at the middle fifth of the height

Vertical design load in the middle fifth of the height Nm=(4.60+0.40x1.35x67.2/10.00)= 8.22 kN/m
Average compressive design stress in the middle fifth of the height fsdo=0.001x8.22/0.30=0.027 N/mm^2
Eccentricity of floor load in the middle fifth of the height Mm/Nm=0.20x0.12000x4.6/8.2=0.01341 m
Eccentricity in the middle fifth of the height due to horizontal forces ehm=0.00000 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity due to loads em=(Mm/Nm)+ehm+es=0.01341+0.00000+0.00500=0.01841 m
The slenderness ratio is <=15 so (EC 6 §4.4.3.2) eccentricity due to creep ek=0
Eccentricity in the middle fifth of the height (EC 6 §4.4.3) em=em+ek=0.01841+0.00000=0.01841 m
Minimum eccentricity (EC 6, §4.4.3) em=0.05t=0.05x0.30=0.01500 m
Reduction factor in the middle fifth of the height (EC 6, §4.4.3) (Phi)m=1-2em/t=1-2x0.018/0.30=0.88

Vertical design load at wall base Ni=(1.35x94.8+1.50x5.8)/10.00= 13.67 kN/m
Avearge compressive design stress at wall base qsdo=0.001x13.67/0.30= 0.046 N/mm²
Eccentricity of load at wall base Mi/Ni=0.12000x4.6/13.7=0.04035 m
Eccentricity at wall base due to horizontal loads ehi=0.00000 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity at wall base (EC 6 §4.4.3) ei=Mi/Ni+ehi+es =0.04035+0.00000+0.00500=0.04535 m
Minimum eccendricity (EC6, §4.4.3) ei=0.05t=0.05x0.30=0.01500 m
Reduction factor at wall base (EC 6, §4.4.3) (Phi)i=1-2ei/t=1-2x0.04535/0.30=0.70
Design vertical load resistance Nrd=(Phi)im.t.fk/(gammaM)=1000x0.70x0.30x1.64/1.70= 202.1kN/m
Maximum compressive stress from finite element analysis maxfn=-0.058 N/mm²
(the maximum stress which is obtained from finite element analysis is for stress regions outside the regions of stresses concentrations at the floor beam supports, which are checked separately)
Maximum design vertical load per unit length Nsd=1000x0.30x 0.058= 17.3 kN/m
Nsd= 17.3 < 202.1=Nrd. The ultimate limit state for vertical loading is verified

Strength verification in axial compression, load 1.00xg+0.30xq+Earthquake (EC 6, §4.4.2)

Masonry strength verification at the top of the masonry

Vertical design load at the top Ni=(1.00x27.6+0.30x5.8)/10.00= 2.93 kN/m
Average compression design stress at the top fsdo=0.001x2.93/0.30= 0.010 N/mm²
fsdo= 0.010<0.25N/mm² so (EC 6 Annex C.2) the eccentricity is reduced by (1-k/4)=0.71
Design bending moment at the top (EC 6, Annex C.1) Mi= 0.98 kNm/m
Eccentricity based on EC 6 Annex C.1 Mi/Ni= 0.98/ 2.93=0.33261 m =1.11x(wall thichkness)
The eccentricity is >40% of the masonry wall thickness. It is computed as in EC 6 Annex C.4 bearing depth =0.20x0.30=0.06m. Eccentricity of load at the top Mi/Ni=0.12000 m
Eccentricity at the top due to horizontal loads ehi=0.00018 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity at the top (EC 6 §4.4.3) ei=Mi/Ni+ehi+es =0.12000+0.00018+0.00500=0.12518 m
Minimum eccendricity (EC6, §4.4.3) ei=0.05t=0.05x0.30=0.01500 m
Reduction factor at the top (EC 6, §4.4.3) (Phi)i=1-2ei/t=1-2x0.12518/0.30=0.17
Design vertical load resistance Nrd=(Phi)im.t.fk/(gammaM)=1000x0.17x0.30x1.64/1.70= 49.1kN/m
Nsd= 2.9 < 49.1=Nrd. The ultimate limit state for vertical loading is verified

Masonry strength verification at the middle fifth of the height

Vertical design load in the middle fifth of the height Nm=(2.93+0.40x1.00x67.2/10.00)= 5.62 kN/m
Average compressive design stress in the middle fifth of the height fsdo=0.001x5.62/0.30=0.019 N/mm²
Eccentricity of floor load in the middle fifth of the height Mm/Nm=0.20x0.12000x2.9/5.6=0.01253 m
Eccentricity in the middle fifth of the height due to horizontal forces ehm=0.00009 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity due to loads em=(Mm/Nm)+ehm+es=0.01253+0.00009+0.00500=0.01762 m
The slenderness ratio is <=15 so (EC 6 §4.4.3.2) eccentricity due to creep ek=0
Eccentricity in the middle fifth of the height (EC 6 §4.4.3) em=em+ek=0.01762+0.00000=0.01762 m
Minimum eccentricity (EC 6, §4.4.3) em=0.05t=0.05x0.30=0.01500 m
Reduction factor in the middle fifth of the height (EC 6, §4.4.3) (Phi)m=1-2em/t=1-2x0.018/0.30=0.88
Vertical design load at wall base Ni=(1.00x94.8+0.30x5.8)/10.00= 9.65 kN/m
Avearge compressive design stress at wall base qsdo=0.001x9.65/0.30= 0.032 N/mm²
Eccentricity of load at wall base Mi/Ni=0.12000x2.9/9.7=0.03647 m
Eccentricity at wall base due to horizontal loads ehi=0.00018 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity at wall base (EC 6 §4.4.3) ei=Mi/Ni+ehi+es =0.03647+0.00018+0.00500=0.04165 m
Minimum eccendricity (EC6, §4.4.3) ei=0.05t=0.05x0.30=0.01500 m
Reduction factor at wall base (EC 6, §4.4.3) (Phi)i=1-2ei/t=1-2x0.04165/0.30=0.72
Design vertical load resistance Nrd=(Phi)im.t.fk/(gammaM)=1000x0.72x0.30x1.64/1.70= 207.9kN/m
Maximum compressive stress from finite element analysis maxfn=-0.127 N/mm²
(the maximum stress which is obtained from finite element analysis is for stress regions outside the regions of stresses concentrations at the floor beam supports, which are checked separately)
Maximum design vertical load per unit length Nsd=1000x0.30x 0.127= 38.2 kN/m
Nsd= 38.2 < 207.9=Nrd. The ultimate limit state for vertical loading is verified

Strength verification in shear, load 1.00xg+0.30xq+Earthquake (EC 6.§4.5.3)

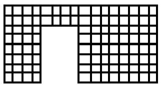
Maximum shearing stress from finite element solution (tau)max= 0.034 N/mm^2
(the maximum stress which is obtained from finite element analysis is for stress regions outside the regions of stresses concentrations at the floor beam supports, which are checked separately)
Maximum design shear load per unit length Vsd=1000x0.30x 0.034= 10.3 KN/m
Characteristic shear strength fvk=0.70x(fvko+0.4xfd) (EC 6 §3.6.3 and 3.6.3.(8))
fvk=0.70x(0.150+0.4x 0.127)= 0.141 N/mm^2, maxfvk= 1.000 N/mm^2
Design shear resistance of masonry Vrd=fvk.t./(gammaM) (EC 6 §4.5.3)
Vrd=1000x 0.150x0.30/1.70= 26.5. kN/m
Vsd= 10.3 < 26.5=Vrd. The ultimate limit state for shear loading is verified.

Top beam of reinforced concrete (EC 6 §5.2)

On the top of the wall and the top of the openings, some small tensile stresses are taken from reinforced concrete bond beams or lintels at these places 30x20 (width x height in cm), with minimum reinforcement 4#12 (stirrups #8/20) which satisfies the minimum code requirements.

1st floor W10

Dimensions length= 6.00m, height= 3.00m, thickness=0.30m



YTONG-M5 0.30
Category of execution A (EC-6, Annex ?)
Category of masonry units construction I (EC 6, §3.1.1)
Compressive strength of masonry fk= 1.64 N/mm^2

Partial safety factors for material properties (gammaM)=1.70 (EC6, Table.2.3)
Effective Length hef=(ro)xh=0.75x 3.00= 2.25 m (EC 6, §4.4.4.3)
Slenderness ratio hef/tef= 2.25/0.30= 7.50<27 OK (EC 6, §4.4.6)

Strength verification in axial compression, load 1.35xg+1.50xq (EC 6, §4.4.2)

Masonry strength verification at the top of the masonry

Vertical design load at the top Ni=(1.35x172.1+1.50x32.4)/6.00= 46.82 kN/m
Average compression design stress at the top fsdo=0.001x46.82/0.30= 0.156 N/mm^2
fsdo= 0.156<0.25N/mm^2 so (EC 6 Annex C.2) the eccentricity is reduced by (1-k/4)=0.50
Design bending moment at the top (EC 6, Annex C.1) Mi= 0.00 kNm/m
Eccentricity based on EC 6 Annex C.1 Mi/Ni= 0.00/ 46.82=0.00000 m =0.00x(wall thichkness)
Eccentricity at the top due to horizontal loads ehi=0.00000 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity at the top (EC 6 §4.4.3) ei=Mi/Ni+ehi+es =0.00000+0.00000+0.00500=0.00500 m
Minimum eccendricity (EC6, §4.4.3) ei=0.05t=0.05x0.30=0.01500 m
Reduction factor at the top (EC 6, §4.4.3) (Phi)i=1-2ei/t=1-2x0.01500/0.30=0.90
Design vertical load resistance Nrd=(Phi)im.t.fk/(gammaM)=1000x0.90x0.30x1.64/1.70= 259.8kN/m
Nsd= 46.8 < 259.8=Nrd. The ultimate limit state for vertical loading is verified

Masonry strength verification at the middle fifth of the height

Vertical design load in the middle fifth of the height Nm=(46.82+0.40x1.35x35.3/6.00)= 50.00 kN/m
Average compressive design stress in the middle fifth of the height fsdo=0.001x50.00/0.30=0.167 N/mm^2
Eccentricity of floor load in the middle fifth of the height Mm/Nm=0.20x0.00000x46.8/50.0=0.00000 m
Eccentricity in the middle fifth of the height due to horizontal forces ehm=0.00000 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity due to loads em=(Mm/Nm)+ehm+es=0.00000+0.00000+0.00500=0.00500 m
The slenderness ratio is <=15 so (EC 6 §4.4.3.2) eccentricity due to creep ek=0
Eccentricity in the middle fifth of the height (EC 6 §4.4.3) em=em+ek=0.00500+0.00000=0.00500 m
Minimum eccentricity (EC 6, §4.4.3) em=0.05t=0.05x0.30=0.01500 m
Reduction factor in the middle fifth of the height (EC 6, §4.4.3) (Phi)m=1-2em/t=1-2x0.015/0.30=0.90

Vertical design load at wall base Ni=(1.35x207.4+1.50x32.4)/6.00= 54.77 kN/m
Avearge compressive design stress at wall base qsdo=0.001x54.77/0.30= 0.183 N/mm²
Eccentricity of load at wall base Mi/Ni=0.00000x46.8/54.8=0.00000 m
Eccentricity at wall base due to horizontal loads ehi=0.00000 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity at wall base (EC 6 §4.4.3) ei=Mi/Ni+ehi+es =0.00000+0.00000+0.00500=0.00500 m
Minimum eccendricity (EC6, §4.4.3) ei=0.05t=0.05x0.30=0.01500 m
Reduction factor at wall base (EC 6, §4.4.3) (Phi)i=1-2ei/t=1-2x0.01500/0.30=0.90
Design vertical load resistance Nrd=(Phi)im.t.fk/(gammaM)=1000x0.90x0.30x1.64/1.70= 259.8kN/m
Maximum compressive stress from finite element analysis maxfn=-0.349 N/mm²
(the maximum stress which is obtained from finite element analysis is for stress regions outside the regions of stresses concentrations at the floor beam supports, which are checked separately)
Maximum design vertical load per unit length Nsd=1000x0.30x 0.349= 104.6 kN/m
Nsd= 104.6 < 259.8=Nrd. The ultimate limit state for vertical loading is verified

Strength verification in axial compression, load 1.00xg+0.30xq+Earthquake (EC 6, §4.4.2)

Masonry strength verification at the top of the masonry

Vertical design load at the top Ni=(1.00x172.1+0.30x32.4)/6.00= 30.30 kN/m
Average compression design stress at the top fsdo=0.001x30.30/0.30= 0.101 N/mm²
fsdo= 0.101<0.25N/mm² so (EC 6 Annex C.2) the eccentricity is reduced by (1-k/4)=0.50
Design bending moment at the top (EC 6, Annex C.1) Mi= 0.00 kNm/m
Eccentricity based on EC 6 Annex C.1 Mi/Ni= 0.00/ 30.30=0.00000 m =0.00x(wall thichkness)
Eccentricity at the top due to horizontal loads ehi=0.00018 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity at the top (EC 6 §4.4.3) ei=Mi/Ni+ehi+es =0.00000+0.00018+0.00500=0.00518 m
Minimum eccendricity (EC6, §4.4.3) ei=0.05t=0.05x0.30=0.01500 m
Reduction factor at the top (EC 6, §4.4.3) (Phi)i=1-2ei/t=1-2x0.01500/0.30=0.90
Design vertical load resistance Nrd=(Phi)im.t.fk/(gammaM)=1000x0.90x0.30x1.64/1.70= 259.8kN/m
Nsd= 30.3 < 259.8=Nrd. The ultimate limit state for vertical loading is verified

Masonry strength verification at the middle fifth of the height

Vertical design load in the middle fifth of the height Nm=(30.30+0.40x1.00x35.3/6.00)= 32.66 kN/m
Average compressive design stress in the middle fifth of the height fsdo=0.001x32.66/0.30=0.109 N/mm²
Eccentricity of floor load in the middle fifth of the height Mm/Nm=0.20x0.00000x30.3/32.7=0.00000 m
Eccentricity in the middle fifth of the height due to horizontal forces ehm=0.00009 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity due to loads em=(Mm/Nm)+ehm+es=0.00000+0.00009+0.00500=0.00509 m
The slenderness ratio is <=15 so (EC 6 §4.4.3.2) eccentricity due to creep ek=0
Eccentricity in the middle fifth of the height (EC 6 §4.4.3) em=em+ek=0.00509+0.00000=0.00509 m
Minimum eccentricity (EC 6, §4.4.3) em=0.05t=0.05x0.30=0.01500 m
Reduction factor in the middle fifth of the height (EC 6, §4.4.3) (Phi)m=1-2em/t=1-2x0.015/0.30=0.90
Vertical design load at wall base Ni=(1.00x207.4+0.30x32.4)/6.00= 36.19 kN/m
Avearge compressive design stress at wall base qsdo=0.001x36.19/0.30= 0.121 N/mm²
Eccentricity of load at wall base Mi/Ni=0.00000x30.3/36.2=0.00000 m
Eccentricity at wall base due to horizontal loads ehi=0.00018 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity at wall base (EC 6 §4.4.3) ei=Mi/Ni+ehi+es =0.00000+0.00018+0.00500=0.00518 m
Minimum eccendricity (EC6, §4.4.3) ei=0.05t=0.05x0.30=0.01500 m
Reduction factor at wall base (EC 6, §4.4.3) (Phi)i=1-2ei/t=1-2x0.01500/0.30=0.90
Design vertical load resistance Nrd=(Phi)im.t.fk/(gammaM)=1000x0.90x0.30x1.64/1.70= 259.8kN/m
Maximum compressive stress from finite element analysis maxfn=-0.263 N/mm²
(the maximum stress which is obtained from finite element analysis is for stress regions outside the regions of stresses concentrations at the floor beam supports, which are checked separately)
Maximum design vertical load per unit length Nsd=1000x0.30x 0.263= 78.8 kN/m
Nsd= 78.8 < 259.8=Nrd. The ultimate limit state for vertical loading is verified

Strength verification in shear, load 1.00xg+0.30xq+Earthquake (EC 6.§4.5.3)

Maximum shearing stress from finite element solution (tau)max= 0.069 N/mm²
(the maximum stress which is obtained from finite element analysis is for stress regions outside the regions of stresses concentrations at the floor beam supports, which are checked separately)
Maximum design shear load per unit length Vsd=1000x0.30x 0.069= 20.6 KN/m
Characteristic shear strength fvk=0.70x(fvko+0.4xfd) (EC 6 §3.6.3 and 3.6.3.(8))
fvk=0.70x(0.150+0.4x 0.263)= 0.179 N/mm², maxfvk= 1.000 N/mm²
Design shear resistance of masonry Vrd=fvk.t./(gammaM) (EC 6 §4.5.3)
Vrd=1000x 0.179x0.30/1.70= 31.5. kN/m
Vsd= 20.6 < 31.5=Vrd. The ultimate limit state for shear loading is verified.

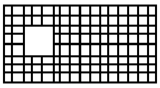
Verification of regions with concentrated loads (EC 6, §4.4.8)

From the finite element solution is obtained at the support positions of the floor beams at the top the maximum compressive stresses. Because at the wall top allways exists a bond beam from reinforced concrete, we check the stresses in the finite elements a line bellow.
The maximum compressive stress in the region of stress concentration is fsdmax= 0.400N/mm^2
This maximum stress 0.400 is < fk/(gammaM)= 1.64/1.7= 0.96
The strength requirements for concentrated loads are verified (EC 6 §4.4.8)

Top beam of reinforced concrete (EC 6 §5.2)

On the top of the wall and the top of the openings, some small tensile stresses are taken from reinforced concrete bond beams or lintels at these places 30x20 (width x height in cm), with minimum reinforcement 4#12 (stirrups #8/20) which satisfies the minimum code requirements.

Gr. floor **W1** Dimensions length= 6.00m, height= 3.00m, thickness=0.30m



YTONG-M5 0.30
Category of execution A (EC-6, Annex ?)
Category of masonry units construction I (EC 6, §3.1.1)
Compressive strength of masonry fk= 1.64 N/mm^2

Partial safety factors for material properties (gammaM)=1.70 (EC6, Table.2.3)
Effective Length hef=(ro)xh=0.75x 3.00= 2.25 m (EC 6, §4.4.4.3)
Slenderness ratio hef/tef= 2.25/0.30= 7.50<27 OK (EC 6, §4.4.6)

Strength verification in axial compression, load 1.35xg+1.50xq (EC 6, §4.4.2)

Masonry strength verification at the top of the masonry

Vertical design load at the top Ni=(1.35x155.4+1.50x35.9)/6.00= 43.94 kN/m
Average compression design stress at the top fsdo=0.001x43.94/0.30= 0.146 N/mm^2
fsdo= 0.146<0.25N/mm^2 so (EC 6 Annex C.2) the eccentricity is reduced by (1-k/4)=0.83
Design bending moment at the top (EC 6, Annex C.1) Mi= 4.64 kNm/m
Eccentricity based on EC 6 Annex C.1 Mi/Ni= 4.64/ 43.94=0.10563 m =0.35x(wall thichkness)
Eccentricity at the top due to horizontal loads ehi=0.00000 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity at the top (EC 6 §4.4.3) ei=Mi/Ni+ehi+es =0.10563+0.00000+0.00500=0.11063 m
Minimum eccendricity (EC6, §4.4.3) ei=0.05t=0.05x0.30=0.01500 m
Reduction factor at the top (EC 6, §4.4.3) (Phi)i=1-2ei/t=1-2x0.11063/0.30=0.26
Design vertical load resistance Nrd=(Phi)im.t.fk/(gammaM)=1000x0.26x0.30x1.64/1.70= 75.1kN/m
Nsd= 43.9 < 75.1=Nrd. The ultimate limit state for vertical loading is verified

Masonry strength verification at the middle fifth of the height

Vertical design load in the middle fifth of the height Nm=(43.94+0.40x1.35x39.7/6.00)= 47.51 kN/m
Average compressive design stress in the middle fifth of the height fsdo=0.001x47.51/0.30=0.158 N/mm^2
Eccentricity of floor load in the middle fifth of the height Mm/Nm=0.20x0.10563x43.9/47.5=0.01954 m
Eccentricity in the middle fifth of the height due to horizontal forces ehm=0.00000 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity due to loads em=(Mm/Nm)+ehm+es=0.01954+0.00000+0.00500=0.02454 m
The slenderness ratio is <=15 so (EC 6 §4.4.3.2) eccentricity due to creep ek=0
Eccentricity in the middle fifth of the height (EC 6 §4.4.3) em=em+ek=0.02454+0.00000=0.02454 m
Minimum eccentricity (EC 6, §4.4.3) em=0.05t=0.05x0.30=0.01500 m
Reduction factor in the middle fifth of the height (EC 6, §4.4.3) (Phi)m=1-2em/t=1-2x0.025/0.30=0.84
Vertical design load at wall base Ni=(1.35x195.1+1.50x35.9)/6.00= 52.87 kN/m
Avearge compressive design stress at wall base qsdo=0.001x52.87/0.30= 0.176 N/mm^2
Eccentricity of load at wall base Mi/Ni=0.10563x43.9/52.9=0.08778 m
Eccentricity at wall base due to horizontal loads ehi=0.00000 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity at wall base (EC 6 §4.4.3) ei=Mi/Ni+ehi+es =0.08778+0.00000+0.00500=0.09278 m
Minimum eccendricity (EC6, §4.4.3) ei=0.05t=0.05x0.30=0.01500 m
Reduction factor at wall base (EC 6, §4.4.3) (Phi)i=1-2ei/t=1-2x0.09278/0.30=0.38
Design vertical load resistance Nrd=(Phi)im.t.fk/(gammaM)=1000x0.38x0.30x1.64/1.70= 109.7kN/m
Maximum compressive stress from finite element analysis maxfn=-0.314 N/mm^2
(the maximum stress which is obtained from finite element analysis is for stress regions outside the regions of stresses concentrations at the floor beam supports, which are checked separately)
Maximum design vertical load per unit length Nsd=1000x0.30x 0.314= 94.2 kN/m
Nsd= 94.2 < 109.7=Nrd. The ultimate limit state for vertical loading is verified

Strength verification in axial compression, load 1.00xg+0.30xq+Earthquake (EC 6, §4.4.2)

Masonry strength verification at the top of the masonry

Vertical design load at the top Ni=(1.00x155.4+0.30x35.9)/6.00= 27.70 kN/m
Average compression design stress at the top fsdo=0.001x27.70/0.30= 0.092 N/mm²
fsdo= 0.092<0.25N/mm² so (EC 6 Annex C.2) the eccentricity is reduced by (1-k/4)=0.83
Design bending moment at the top (EC 6, Annex C.1) Mi= 2.74 kNm/m
Eccentricity based on EC 6 Annex C.1 Mi/Ni= 2.74/ 27.70=0.09899 m =0.33x(wall thichkness)
Eccentricity at the top due to horizontal loads ehi=0.00027 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity at the top (EC 6 §4.4.3) ei=Mi/Ni+ehi+es =0.09899+0.00027+0.00500=0.10427 m
Minimum eccendricity (EC6, §4.4.3) ei=0.05t=0.05x0.30=0.01500 m
Reduction factor at the top (EC 6, §4.4.3) (Phi)i=1-2ei/t=1-2x0.10427/0.30=0.30
Design vertical load resistance Nrd=(Phi)im.t.fk/(gammaM)=1000x0.30x0.30x1.64/1.70= 86.6kN/m
Nsd= 27.7 < 86.6=Nrd. The ultimate limit state for vertical loading is verified

Masonry strength verification at the middle fifth of the height

Vertical design load in the middle fifth of the height Nm=(27.70+0.40x1.00x39.7/6.00)= 30.34 kN/m
Average compressive design stress in the middle fifth of the height fsdo=0.001x30.34/0.30=0.101 N/mm²
Eccentricity of floor load in the middle fifth of the height Mm/Nm=0.20x0.09899x27.7/30.3=0.01807 m
Eccentricity in the middle fifth of the height due to horizontal forces ehm=0.00014 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity due to loads em=(Mm/Nm)+ehm+es=0.01807+0.00014+0.00500=0.02321 m
The slenderness ratio is <=15 so (EC 6 §4.4.3.2) eccentricity due to creep ek=0
Eccentricity in the middle fifth of the height (EC 6 §4.4.3) em=em+ek=0.02321+0.00000=0.02321 m
Minimum eccentricity (EC 6, §4.4.3) em=0.05t=0.05x0.30=0.01500 m
Reduction factor in the middle fifth of the height (EC 6, §4.4.3) (Phi)m=1-2em/t=1-2x0.023/0.30=0.85
Vertical design load at wall base Ni=(1.00x195.1+0.30x35.9)/6.00= 34.31 kN/m
Avearge compressive design stress at wall base qsdo=0.001x34.31/0.30= 0.114 N/mm²
Eccentricity of load at wall base Mi/Ni=0.09899x27.7/34.3=0.07990 m
Eccentricity at wall base due to horizontal loads ehi=0.00027 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity at wall base (EC 6 §4.4.3) ei=Mi/Ni+ehi+es =0.07990+0.00027+0.00500=0.08518 m
Minimum eccendricity (EC6, §4.4.3) ei=0.05t=0.05x0.30=0.01500 m
Reduction factor at wall base (EC 6, §4.4.3) (Phi)i=1-2ei/t=1-2x0.08518/0.30=0.43
Design vertical load resistance Nrd=(Phi)im.t.fk/(gammaM)=1000x0.43x0.30x1.64/1.70= 124.1kN/m
Maximum compressive stress from finite element analysis maxfn=-0.321 N/mm²
(the maximum stress which is obtained from finite element analysis is for stress regions outside the regions of stresses concentrations at the floor beam supports, which are checked separately)
Maximum design vertical load per unit length Nsd=1000x0.30x 0.321= 96.2 kN/m
Nsd= 96.2 < 124.1=Nrd. The ultimate limit state for vertical loading is verified

Strength verification in shear, load 1.00xg+0.30xq+Earthquake (EC 6.§4.5.3)

Maximum shearing stress from finite element solution (tau)max= 0.085 N/mm²
(the maximum stress which is obtained from finite element analysis is for stress regions outside the regions of stresses concentrations at the floor beam supports, which are checked separately)
Maximum design shear load per unit length Vsd=1000x0.30x 0.085= 25.4 KN/m
Characteristic shear strength fvk=0.70x(fvko+0.4xfd) (EC 6 §3.6.3 and 3.6.3.(8))
fvk=0.70x(0.150+0.4x 0.321)= 0.195 N/mm², maxfvk= 1.000 N/mm²
Design shear resistance of masonry Vrd=fvk.t./(gammaM) (EC 6 §4.5.3)
Vrd=1000x 0.195x0.30/1.70= 34.4. kN/m
Vsd= 25.4 < 34.4=Vrd. The ultimate limit state for shear loading is verified.

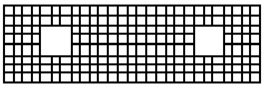
Verification of regions with concentrated loads (EC 6, §4.4.8)

From the finite element solution is obtained at the support positions of the floor beams at the top the maximum compressive stresses. Because at the wall top allways exists a bond beam from reinforced concrete, we check the stresses in the finite elements a line bellow.
The maximum compressive stress in the region of stress concentration is fsdmax= 0.337N/mm²
This maximum stress 0.337 is < fk/(gammaM)= 1.64/1.7= 0.96
The strength requirements for concentrated loads are verified (EC 6 §4.4.8)

Top beam of reinforced concrete (EC 6 §5.2)

On the top of the wall and the top of the openings, some small tensile stresses are taken from reinforced concrete bond beams or lintels at these places 30x20 (width x height in cm), with minimum reinforcement 4#12 (stirrups #8/20) which satisfies the minimum code requirements.

Gr. floor **W2** Dimensions length=10.00m, height= 3.00m, thickness=0.30m



YTONG-M5 0.30
Category of execution A (EC-6, Annex ?)
Category of masonry units construction I (EC 6, §3.1.1)
Compressive strength of masonry fk= 1.64 N/mm^2

Partial safety factors for material properties (gammaM)=1.70 (EC6, Table.2.3)
Effective Length hef=(ro)xh=0.75x 3.00= 2.25 m (EC 6, §4.4.4.3)
Slenderness ratio hef/tef= 2.25/0.30= 7.50<27 OK (EC 6, §4.4.6)

Strength verification in axial compression, load 1.35xg+1.50xq (EC 6, §4.4.2)

Masonry strength verification at the top of the masonry
Vertical design load at the top Ni=(1.35x120.9+1.50x15.4)/10.00= 18.63 kN/m
Average compression design stress at the top fsdo=0.001x18.63/0.30= 0.062 N/mm^2
fsdo= 0.062<0.25N/mm^2 so (EC 6 Annex C.2) the eccentricity is reduced by (1-k/4)=0.86
Design bending moment at the top (EC 6, Annex C.1) Mi= 1.43 kNm/m
Eccentricity based on EC 6 Annex C.1 Mi/Ni= 1.43/ 18.63=0.07669 m =0.26x(wall thichkness)
Eccentricity at the top due to horizontal loads ehi=0.00000 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity at the top (EC 6 §4.4.3) ei=Mi/Ni+ehi+es =0.07669+0.00000+0.00500=0.08169 m
Minimum eccendricity (EC6, §4.4.3) ei=0.05t=0.05x0.30=0.01500 m
Reduction factor at the top (EC 6, §4.4.3) (Phi)i=1-2ei/t=1-2x0.08169/0.30=0.46
Design vertical load resistance Nrd=(Phi)im.t.fk/(gammaM)=1000x0.46x0.30x1.64/1.70= 132.8kN/m
Nsd= 18.6 < 132.8=Nrd. The ultimate limit state for vertical loading is verified

Masonry strength verification at the middle fifth of the height
Vertical design load in the middle fifth of the height Nm=(18.63+0.40x1.35x65.1/10.00)= 22.15 kN/m
Average compressive design stress in the middle fifth of the height fsdo=0.001x22.15/0.30=0.074 N/mm^2
Eccentricity of floor load in the middle fifth of the height Mm/Nm=0.20x0.07669x18.6/22.1=0.01290 m
Eccentricity in the middle fifth of the height due to horizontal forces ehm=0.00000 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity due to loads em=(Mm/Nm)+ehm+es=0.01290+0.00000+0.00500=0.01790 m
The slenderness ratio is <=15 so (EC 6 §4.4.3.2) eccentricity due to creep ek=0
Eccentricity in the middle fifth of the height (EC 6 §4.4.3) em=em+ek=0.01790+0.00000=0.01790 m
Minimum eccentricity (EC 6, §4.4.3) em=0.05t=0.05x0.30=0.01500 m
Reduction factor in the middle fifth of the height (EC 6, §4.4.3) (Phi)m=1-2em/t=1-2x0.018/0.30=0.88
Vertical design load at wall base Ni=(1.35x186.0+1.50x15.4)/10.00= 27.42 kN/m
Avearge compressive design stress at wall base qsdo=0.001x27.42/0.30= 0.091 N/mm^2
Eccentricity of load at wall base Mi/Ni=0.07669x18.6/27.4=0.05211 m
Eccentricity at wall base due to horizontal loads ehi=0.00000 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity at wall base (EC 6 §4.4.3) ei=Mi/Ni+ehi+es =0.05211+0.00000+0.00500=0.05711 m
Minimum eccendricity (EC6, §4.4.3) ei=0.05t=0.05x0.30=0.01500 m
Reduction factor at wall base (EC 6, §4.4.3) (Phi)i=1-2ei/t=1-2x0.05711/0.30=0.62
Design vertical load resistance Nrd=(Phi)im.t.fk/(gammaM)=1000x0.62x0.30x1.64/1.70= 179.0kN/m
Maximum compressive stress from finite element analysis maxfn=-0.145 N/mm^2
(the maximum stress which is obtained from finite element analysis is for stress regions outside the regions of stresses concentrations at the floor beam supports, which are checked separately)
Maximum design vertical load per unit length Nsd=1000x0.30x 0.145= 43.5 kN/m
Nsd= 43.5 < 179.0=Nrd. The ultimate limit state for vertical loading is verified

Strength verification in axial compression, load 1.00xg+0.30xq+Earthquake (EC 6, §4.4.2)

Masonry strength verification at the top of the masonry
Vertical design load at the top Ni=(1.00x120.9+0.30x15.4)/10.00= 12.55 kN/m
Average compression design stress at the top fsdo=0.001x12.55/0.30= 0.042 N/mm^2
fsdo= 0.042<0.25N/mm^2 so (EC 6 Annex C.2) the eccentricity is reduced by (1-k/4)=0.86
Design bending moment at the top (EC 6, Annex C.1) Mi= 0.85 kNm/m
Eccentricity based on EC 6 Annex C.1 Mi/Ni= 0.85/ 12.55=0.06743 m =0.22x(wall thichkness)

Eccentricity at the top due to horizontal loads ehi=0.00027 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity at the top (EC 6 §4.4.3) ei=Mi/Ni+ehi+es =0.06743+0.00027+0.00500=0.07270 m
Minimum eccendricity (EC6, §4.4.3) ei=0.05t=0.05x0.30=0.01500 m
Reduction factor at the top (EC 6, §4.4.3) (Phi)i=1-2ei/t=1-2x0.07270/0.30=0.52
Design vertical load resistance Nrd=(Phi)im.t.fk/(gammaM)=1000x0.52x0.30x1.64/1.70= 150.1kN/m
Nsd= 12.6 < 150.1=Nrd. The ultimate limit state for vertical loading is verified

Masonry strength verification at the middle fifth of the height

Vertical design load in the middle fifth of the height Nm=(12.55+0.40x1.00x65.1/10.00)= 15.16 kN/m
Average compressive design stress in the middle fifth of the height fsdo=0.001x15.16/0.30=0.051 N/mm²
Eccentricity of floor load in the middle fifth of the height Mm/Nm=0.20x0.06743x12.6/15.2=0.01117 m
Eccentricity in the middle fifth of the height due to horizontal forces ehm=0.00014 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity due to loads em=(Mm/Nm)+ehm+es=0.01117+0.00014+0.00500=0.01631 m
The slenderness ratio is <=15 so (EC 6 §4.4.3.2) eccentricity due to creep ek=0
Eccentricity in the middle fifth of the height (EC 6 §4.4.3) em=em+ek=0.01631+0.00000=0.01631 m
Minimum eccentricity (EC 6, §4.4.3) em=0.05t=0.05x0.30=0.01500 m
Reduction factor in the middle fifth of the height (EC 6, §4.4.3) (Phi)m=1-2em/t=1-2x0.016/0.30=0.89
Vertical design load at wall base Ni=(1.00x186.0+0.30x15.4)/10.00= 19.06 kN/m
Avearge compressive design stress at wall base qsdo=0.001x19.06/0.30= 0.064 N/mm²
Eccentricity of load at wall base Mi/Ni=0.06743x12.6/19.1=0.04440 m
Eccentricity at wall base due to horizontal loads ehi=0.00027 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity at wall base (EC 6 §4.4.3) ei=Mi/Ni+ehi+es =0.04440+0.00027+0.00500=0.04967 m
Minimum eccendricity (EC6, §4.4.3) ei=0.05t=0.05x0.30=0.01500 m
Reduction factor at wall base (EC 6, §4.4.3) (Phi)i=1-2ei/t=1-2x0.04967/0.30=0.67
Design vertical load resistance Nrd=(Phi)im.t.fk/(gammaM)=1000x0.67x0.30x1.64/1.70= 193.4kN/m
Maximum compressive stress from finite element analysis maxfn=-0.218 N/mm²
(the maximum stress which is obtained from finite element analysis is for stress regions outside the regions of stresses concentrations at the floor beam supports, which are checked separately)
Maximum design vertical load per unit length Nsd=1000x0.30x 0.218= 65.4 kN/m
Nsd= 65.4 < 193.4=Nrd. The ultimate limit state for vertical loading is verified

Strength verification in shear, load 1.00xg+0.30xq+Earthquake (EC 6.§4.5.3)

Maximum shearing stress from finite element solution (tau)max= 0.062 N/mm²
(the maximum stress which is obtained from finite element analysis is for stress regions outside the regions of stresses concentrations at the floor beam supports, which are checked separately)
Maximum design shear load per unit length Vsd=1000x0.30x 0.062= 18.5 KN/m
Characteristic shear strength fvk=0.70x(fvko+0.4xfd) (EC 6 §3.6.3 and 3.6.3.(8))
fvk=0.70x(0.150+0.4x 0.218)= 0.166 N/mm², maxfvk= 1.000 N/mm²
Design shear resistance of masonry Vrd=fvk.t./(gammaM) (EC 6 §4.5.3)
Vrd=1000x 0.166x0.30/1.70= 29.3. kN/m
Vsd= 18.5 < 29.3=Vrd. The ultimate limit state for shear loading is verified.

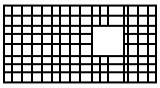
Verification of regions with concentrated loads (EC 6, §4.4.8)

From the finite element solution is obtained at the support positions of the floor beams at the top the maximum compressive stresses. Because at the wall top allways exists a bond beam from reinforced concrete, we check the stresses in the finite elements a line bellow.
The maximum compressive stress in the region of stress concentration is fsdmax= 0.155N/mm²
This maximum stress 0.155 is < fk/(gammaM)= 1.64/1.7= 0.96
The strength requirements for concentrated loads are verified (EC 6 §4.4.8)

Top beam of reinforced concrete (EC 6 §5.2)

On the top of the wall and the top of the openings, some small tensile stresses are taken from reinforced concrete bond beams or lintels at these places 30x20 (width x height in cm), with minimum reinforcement 4#12 (stirrups #8/20) which satisfies the minimum code requirements.

Gr. floor **W3** Dimensions length= 6.00m, height= 3.00m, thickness=0.30m



YTONG-M5 0.30
Category of execution A (EC-6, Annex ?)
Category of masonry units construction I (EC 6, §3.1.1)
Compressive strength of masonry fk= 1.64 N/mm^2

Partial safety factors for material properties (gammaM)=1.70 (EC6, Table.2.3)
Effective Length hef=(ro)xh=0.75x 3.00= 2.25 m (EC 6, §4.4.4.3)
Slenderness ratio hef/tef= 2.25/0.30= 7.50<27 OK (EC 6, §4.4.6)

Strength verification in axial compression, load 1.35xg+1.50xq (EC 6, §4.4.2)

Masonry strength verification at the top of the masonry
Vertical design load at the top Ni=(1.35x155.4+1.50x35.9)/6.00= 43.94 kN/m
Average compression design stress at the top fsdo=0.001x43.94/0.30= 0.146 N/mm^2
fsdo= 0.146<0.25N/mm^2 so (EC 6 Annex C.2) the eccentricity is reduced by (1-k/4)=0.83
Design bending moment at the top (EC 6, Annex C.1) Mi= 4.64 kNm/m
Eccentricity based on EC 6 Annex C.1 Mi/Ni= 4.64/ 43.94=0.10563 m =0.35x(wall thichkness)
Eccentricity at the top due to horizontal loads ehi=0.00000 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity at the top (EC 6 §4.4.3) ei=Mi/Ni+ehi+es =0.10563+0.00000+0.00500=0.11063 m
Minimum eccendricity (EC6, §4.4.3) ei=0.05t=0.05x0.30=0.01500 m
Reduction factor at the top (EC 6, §4.4.3) (Phi)i=1-2ei/t=1-2x0.11063/0.30=0.26
Design vertical load resistance Nrd=(Phi)im.t.fk/(gammaM)=1000x0.26x0.30x1.64/1.70= 75.1kN/m
Nsd= 43.9 < 75.1=Nrd. The ultimate limit state for vertical loading is verified

Masonry strength verification at the middle fifth of the height
Vertical design load in the middle fifth of the height Nm=(43.94+0.40x1.35x39.7/6.00)= 47.51 kN/m
Average compressive design stress in the middle fifth of the height fsdo=0.001x47.51/0.30=0.158 N/mm^2
Eccentricity of floor load in the middle fifth of the height Mm/Nm=0.20x0.10563x43.9/47.5=0.01954 m
Eccentricity in the middle fifth of the height due to horizontal forces ehm=0.00000 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity due to loads em=(Mm/Nm)+ehm+es=0.01954+0.00000+0.00500=0.02454 m
The slenderness ratio is <=15 so (EC 6 §4.4.3.2) eccentricity due to creep ek=0
Eccentricity in the middle fifth of the height (EC 6 §4.4.3) em=em+ek=0.02454+0.00000=0.02454 m
Minimum eccentricity (EC 6, §4.4.3) em=0.05t=0.05x0.30=0.01500 m
Reduction factor in the middle fifth of the height (EC 6, §4.4.3) (Phi)m=1-2em/t=1-2x0.025/0.30=0.84
Vertical design load at wall base Ni=(1.35x195.1+1.50x35.9)/6.00= 52.87 kN/m
Avearge compressive design stress at wall base qsdo=0.001x52.87/0.30= 0.176 N/mm^2
Eccentricity of load at wall base Mi/Ni=0.10563x43.9/52.9=0.08778 m
Eccentricity at wall base due to horizontal loads ehi=0.00000 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity at wall base (EC 6 §4.4.3) ei=Mi/Ni+ehi+es =0.08778+0.00000+0.00500=0.09278 m
Minimum eccendricity (EC6, §4.4.3) ei=0.05t=0.05x0.30=0.01500 m
Reduction factor at wall base (EC 6, §4.4.3) (Phi)i=1-2ei/t=1-2x0.09278/0.30=0.38
Design vertical load resistance Nrd=(Phi)im.t.fk/(gammaM)=1000x0.38x0.30x1.64/1.70= 109.7kN/m
Maximum compressive stress from finite element analysis maxfn=-0.316 N/mm^2
(the maximum stress which is obtained from finite element analysis is for stress regions outside the regions of stresses concentrations at the floor beam supports, which are checked separately)
Maximum design vertical load per unit length Nsd=1000x0.30x 0.316= 94.8 kN/m
Nsd= 94.8 < 109.7=Nrd. The ultimate limit state for vertical loading is verified

Strength verification in axial compression, load 1.00xg+0.30xq+Earthquake (EC 6, §4.4.2)

Masonry strength verification at the top of the masonry
Vertical design load at the top Ni=(1.00x155.4+0.30x35.9)/6.00= 27.70 kN/m
Average compression design stress at the top fsdo=0.001x27.70/0.30= 0.092 N/mm^2
fsdo= 0.092<0.25N/mm^2 so (EC 6 Annex C.2) the eccentricity is reduced by (1-k/4)=0.83
Design bending moment at the top (EC 6, Annex C.1) Mi= 2.74 kNm/m
Eccentricity based on EC 6 Annex C.1 Mi/Ni= 2.74/ 27.70=0.09899 m =0.33x(wall thichkness)

Eccentricity at the top due to horizontal loads ehi=0.00027 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity at the top (EC 6 §4.4.3) ei=Mi/Ni+ehi+es =0.09899+0.00027+0.00500=0.10427 m
Minimum eccendricity (EC6, §4.4.3) ei=0.05t=0.05x0.30=0.01500 m
Reduction factor at the top (EC 6, §4.4.3) (Phi)i=1-2ei/t=1-2x0.10427/0.30=0.30
Design vertical load resistance Nrd=(Phi)im.t.fk/(gammaM)=1000x0.30x0.30x1.64/1.70= 86.6kN/m
Nsd= 27.7 < 86.6=Nrd. The ultimate limit state for vertical loading is verified

Masonry strength verification at the middle fifth of the height

Vertical design load in the middle fifth of the height Nm=(27.70+0.40x1.00x39.7/6.00)= 30.34 kN/m
Average compressive design stress in the middle fifth of the height fsdo=0.001x30.34/0.30=0.101 N/mm²
Eccentricity of floor load in the middle fifth of the height Mm/Nm=0.20x0.09899x27.7/30.3=0.01807 m
Eccentricity in the middle fifth of the height due to horizontal forces ehm=0.00014 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity due to loads em=(Mm/Nm)+ehm+es=0.01807+0.00014+0.00500=0.02321 m
The slenderness ratio is <=15 so (EC 6 §4.4.3.2) eccentricity due to creep ek=0
Eccentricity in the middle fifth of the height (EC 6 §4.4.3) em=em+ek=0.02321+0.00000=0.02321 m
Minimum eccentricity (EC 6, §4.4.3) em=0.05t=0.05x0.30=0.01500 m
Reduction factor in the middle fifth of the height (EC 6, §4.4.3) (Phi)m=1-2em/t=1-2x0.023/0.30=0.85
Vertical design load at wall base Ni=(1.00x195.1+0.30x35.9)/6.00= 34.31 kN/m
Avearge compressive design stress at wall base qsdo=0.001x34.31/0.30= 0.114 N/mm²
Eccentricity of load at wall base Mi/Ni=0.09899x27.7/34.3=0.07990 m
Eccentricity at wall base due to horizontal loads ehi=0.00027 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity at wall base (EC 6 §4.4.3) ei=Mi/Ni+ehi+es =0.07990+0.00027+0.00500=0.08518 m
Minimum eccendricity (EC6, §4.4.3) ei=0.05t=0.05x0.30=0.01500 m
Reduction factor at wall base (EC 6, §4.4.3) (Phi)i=1-2ei/t=1-2x0.08518/0.30=0.43
Design vertical load resistance Nrd=(Phi)im.t.fk/(gammaM)=1000x0.43x0.30x1.64/1.70= 124.1kN/m
Maximum compressive stress from finite element analysis maxfn=-0.342 N/mm²
(the maximum stress which is obtained from finite element analysis is for stress regions outside the regions of stresses concentrations at the floor beam supports, which are checked separately)
Maximum design vertical load per unit length Nsd=1000x0.30x 0.342= 102.5 kN/m
Nsd= 102.5 < 124.1=Nrd. The ultimate limit state for vertical loading is verified

Strength verification in shear, load 1.00xg+0.30xq+Earthquake (EC 6.§4.5.3)

Maximum shearing stress from finite element solution (tau)max= 0.110 N/mm²
(the maximum stress which is obtained from finite element analysis is for stress regions outside the regions of stresses concentrations at the floor beam supports, which are checked separately)
Maximum design shear load per unit length Vsd=1000x0.30x 0.110= 33.1 KN/m
Characteristic shear strength fvk=0.70x(fvko+0.4xfd) (EC 6 §3.6.3 and 3.6.3.(8))
fvk=0.70x(0.150+0.4x 0.342)= 0.201 N/mm², maxfvk= 1.000 N/mm²
Design shear resistance of masonry Vrd=fvk.t./(gammaM) (EC 6 §4.5.3)
Vrd=1000x 0.201x0.30/1.70= 35.4. kN/m
Vsd= 33.1 < 35.4=Vrd. The ultimate limit state for shear loading is verified.

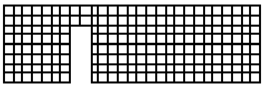
Verification of regions with concentrated loads (EC 6, §4.4.8)

From the finite element solution is obtained at the support positions of the floor beams at the top the maximum compressive stresses. Because at the wall top allways exists a bond beam from reinforced concrete, we check the stresses in the finite elements a line bellow.
The maximum compressive stress in the region of stress concentration is fsdmax= 0.357N/mm²
This maximum stress 0.357 is < fk/(gammaM)= 1.64/1.7= 0.96
The strength requirements for concentrated loads are verified (EC 6 §4.4.8)

Top beam of reinforced concrete (EC 6 §5.2)

On the top of the wall and the top of the openings, some small tensile stresses are taken from reinforced concrete bond beams or lintels at these places 30x20 (width x height in cm), with minimum reinforcement 4#12 (stirrups #8/20) which satisfies the minimum code requirements.

Gr. floor **W4**   Dimensions length=10.00m, height= 3.00m, thickness=0.30m



YTONG-M5 0.30
Category of execution A (EC-6, Annex ?)
Category of masonry units construction I (EC 6, §3.1.1)
Compressive strength of masonry fk= 1.64 N/mm^2

Partial safety factors for material properties (gammaM)=1.70 (EC6, Table.2.3)
Effective Length hef=(ro)xh=0.75x 3.00= 2.25 m (EC 6, §4.4.4.3)
Slenderness ratio hef/tef= 2.25/0.30= 7.50<27 OK (EC 6, §4.4.6)

Strength verification in axial compression, load 1.35xg+1.50xq (EC 6, §4.4.2)

Masonry strength verification at the top of the masonry
Vertical design load at the top Ni=(1.35x123.0+1.50x15.4)/10.00= 18.91 kN/m
Average compression design stress at the top fsdo=0.001x18.91/0.30= 0.063 N/mm^2
fsdo= 0.063<0.25N/mm^2 so (EC 6 Annex C.2) the eccentricity is reduced by (1-k/4)=0.86
Design bending moment at the top (EC 6, Annex C.1) Mi= 1.43 kNm/m
Eccentricity based on EC 6 Annex C.1 Mi/Ni= 1.43/ 18.91=0.07554 m =0.25x(wall thichkness)
Eccentricity at the top due to horizontal loads ehi=0.00000 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity at the top (EC 6 §4.4.3) ei=Mi/Ni+ehi+es =0.07554+0.00000+0.00500=0.08054 m
Minimum eccendricity (EC6, §4.4.3) ei=0.05t=0.05x0.30=0.01500 m
Reduction factor at the top (EC 6, §4.4.3) (Phi)i=1-2ei/t=1-2x0.08054/0.30=0.46
Design vertical load resistance Nrd=(Phi)im.t.fk/(gammaM)=1000x0.46x0.30x1.64/1.70= 132.8kN/m
Nsd= 18.9 < 132.8=Nrd. The ultimate limit state for vertical loading is verified

Masonry strength verification at the middle fifth of the height
Vertical design load in the middle fifth of the height Nm=(18.91+0.40x1.35x67.2/10.00)= 22.54 kN/m
Average compressive design stress in the middle fifth of the height fsdo=0.001x22.54/0.30=0.075 N/mm^2
Eccentricity of floor load in the middle fifth of the height Mm/Nm=0.20x0.07554x18.9/22.5=0.01268 m
Eccentricity in the middle fifth of the height due to horizontal forces ehm=0.00000 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity due to loads em=(Mm/Nm)+ehm+es=0.01268+0.00000+0.00500=0.01768 m
The slenderness ratio is <=15 so (EC 6 §4.4.3.2) eccentricity due to creep ek=0
Eccentricity in the middle fifth of the height (EC 6 §4.4.3) em=em+ek=0.01768+0.00000=0.01768 m
Minimum eccentricity (EC 6, §4.4.3) em=0.05t=0.05x0.30=0.01500 m
Reduction factor in the middle fifth of the height (EC 6, §4.4.3) (Phi)m=1-2em/t=1-2x0.018/0.30=0.88
Vertical design load at wall base Ni=(1.35x190.2+1.50x15.4)/10.00= 27.99 kN/m
Avearge compressive design stress at wall base qsdo=0.001x27.99/0.30= 0.093 N/mm^2
Eccentricity of load at wall base Mi/Ni=0.07554x18.9/28.0=0.05105 m
Eccentricity at wall base due to horizontal loads ehi=0.00000 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity at wall base (EC 6 §4.4.3) ei=Mi/Ni+ehi+es =0.05105+0.00000+0.00500=0.05605 m
Minimum eccendricity (EC6, §4.4.3) ei=0.05t=0.05x0.30=0.01500 m
Reduction factor at wall base (EC 6, §4.4.3) (Phi)i=1-2ei/t=1-2x0.05605/0.30=0.63
Design vertical load resistance Nrd=(Phi)im.t.fk/(gammaM)=1000x0.63x0.30x1.64/1.70= 181.9kN/m
Maximum compressive stress from finite element analysis maxfn=-0.128 N/mm^2
(the maximum stress which is obtained from finite element analysis is for stress regions outside the regions of stresses concentrations at the floor beam supports, which are checked separately)
Maximum design vertical load per unit length Nsd=1000x0.30x 0.128= 38.4 kN/m
Nsd= 38.4 < 181.9=Nrd. The ultimate limit state for vertical loading is verified

Strength verification in axial compression, load 1.00xg+0.30xq+Earthquake (EC 6, §4.4.2)

Masonry strength verification at the top of the masonry
Vertical design load at the top Ni=(1.00x123.0+0.30x15.4)/10.00= 12.76 kN/m
Average compression design stress at the top fsdo=0.001x12.76/0.30= 0.043 N/mm^2
fsdo= 0.043<0.25N/mm^2 so (EC 6 Annex C.2) the eccentricity is reduced by (1-k/4)=0.86
Design bending moment at the top (EC 6, Annex C.1) Mi= 0.85 kNm/m
Eccentricity based on EC 6 Annex C.1 Mi/Ni= 0.85/ 12.76=0.06632 m =0.22x(wall thichkness)

Eccentricity at the top due to horizontal loads ehi=0.00027 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity at the top (EC 6 §4.4.3) ei=Mi/Ni+ehi+es =0.06632+0.00027+0.00500=0.07159 m
Minimum eccendricity (EC6, §4.4.3) ei=0.05t=0.05x0.30=0.01500 m
Reduction factor at the top (EC 6, §4.4.3) (Phi)i=1-2ei/t=1-2x0.07159/0.30=0.52
Design vertical load resistance Nrd=(Phi)im.t.fk/(gammaM)=1000x0.52x0.30x1.64/1.70= 150.1kN/m
Nsd= 12.8 < 150.1=Nrd. The ultimate limit state for vertical loading is verified

Masonry strength verification at the middle fifth of the height

Vertical design load in the middle fifth of the height Nm=(12.76+0.40x1.00x67.2/10.00)= 15.45 kN/m
Average compressive design stress in the middle fifth of the height fsdo=0.001x15.45/0.30=0.051 N/mm^2
Eccentricity of floor load in the middle fifth of the height Mm/Nm=0.20x0.06632x12.8/15.4=0.01096 m
Eccentricity in the middle fifth of the height due to horizontal forces ehm=0.00014 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity due to loads em=(Mm/Nm)+ehm+es=0.01096+0.00014+0.00500=0.01609 m
The slenderness ratio is <=15 so (EC 6 §4.4.3.2) eccentricity due to creep ek=0
Eccentricity in the middle fifth of the height (EC 6 §4.4.3) em=em+ek=0.01609+0.00000=0.01609 m
Minimum eccentricity (EC 6, §4.4.3) em=0.05t=0.05x0.30=0.01500 m
Reduction factor in the middle fifth of the height (EC 6, §4.4.3) (Phi)m=1-2em/t=1-2x0.016/0.30=0.89
Vertical design load at wall base Ni=(1.00x190.2+0.30x15.4)/10.00= 19.48 kN/m
Avearge compressive design stress at wall base qsdo=0.001x19.48/0.30= 0.065 N/mm^2
Eccentricity of load at wall base Mi/Ni=0.06632x12.8/19.5=0.04344 m
Eccentricity at wall base due to horizontal loads ehi=0.00027 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity at wall base (EC 6 §4.4.3) ei=Mi/Ni+ehi+es =0.04344+0.00027+0.00500=0.04872 m
Minimum eccendricity (EC6, §4.4.3) ei=0.05t=0.05x0.30=0.01500 m
Reduction factor at wall base (EC 6, §4.4.3) (Phi)i=1-2ei/t=1-2x0.04872/0.30=0.68
Design vertical load resistance Nrd=(Phi)im.t.fk/(gammaM)=1000x0.68x0.30x1.64/1.70= 196.3kN/m
Maximum compressive stress from finite element analysis maxfn=-0.218 N/mm^2
(the maximum stress which is obtained from finite element analysis is for stress regions outside the regions of stresses concentrations at the floor beam supports, which are checked separately)
Maximum design vertical load per unit length Nsd=1000x0.30x 0.218= 65.3 kN/m
Nsd= 65.3 < 196.3=Nrd. The ultimate limit state for vertical loading is verified

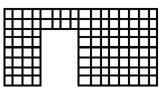
Strength verification in shear, load 1.00xg+0.30xq+Earthquake (EC 6.§4.5.3)

Maximum shearing stress from finite element solution (tau)max= 0.053 N/mm^2
(the maximum stress which is obtained from finite element analysis is for stress regions outside the regions of stresses concentrations at the floor beam supports, which are checked separately)
Maximum design shear load per unit length Vsd=1000x0.30x 0.053= 16.0 KN/m
Characteristic shear strength fvk=0.70x(fvko+0.4xfd) (EC 6 §3.6.3 and 3.6.3.(8))
fvk=0.70x(0.150+0.4x 0.218)= 0.166 N/mm^2, maxfvk= 1.000 N/mm^2
Design shear resistance of masonry Vrd=fvk.t./(gammaM) (EC 6 §4.5.3)
Vrd=1000x 0.166x0.30/1.70= 29.3. kN/m
Vsd= 16.0 < 29.3=Vrd. The ultimate limit state for shear loading is verified.

Top beam of reinforced concrete (EC 6 §5.2)

On the top of the wall and the top of the openings, some small tensile stresses are taken from reinforced concrete bond beams or lintels at these places 30x20 (width x height in cm), with minimum reinforcement 4#12 (stirrups #8/20) which satisfies the minimum code requirements.

Gr. floor **W5** Dimensions length= 6.00m, height= 3.00m, thickness=0.30m



YTONG-M5 0.30
Category of execution A (EC-6, Annex ?)
Category of masonry units construction I (EC 6, §3.1.1)
Compressive strength of masonry fk= 1.64 N/mm^2

Partial safety factors for material properties (gammaM)=1.70 (EC6, Table.2.3)
Effective Length hef=(ro)xh=0.75x 3.00= 2.25 m (EC 6, §4.4.4.3)
Slenderness ratio hef/tef= 2.25/0.30= 7.50<27 OK (EC 6, §4.4.6)

Strength verification in axial compression, load 1.35xg+1.50xq (EC 6, §4.4.2)

Masonry strength verification at the top of the masonry

Vertical design load at the top Ni=(1.35x382.4+1.50x87.9)/6.00= 108.02 kN/m
Average compression design stress at the top fsdo=0.001x108.02/0.30= 0.360 N/mm²
fsdo= 0.360>0.25N/mm² so (EC 6 Annex C.2) the eccentricity is not reduced by (1-k/4)
Design bending moment at the top (EC 6, Annex C.1) Mi= 0.00 kNm/m
Eccentricity based on EC 6 Annex C.1 Mi/Ni= 0.00/108.02=0.00000 m =0.00x(wall thichkness)
Eccentricity at the top due to horizontal loads ehi=0.00000 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity at the top (EC 6 §4.4.3) ei=Mi/Ni+ehi+es =0.00000+0.00000+0.00500=0.00500 m
Minimum eccendricity (EC6, §4.4.3) ei=0.05t=0.05x0.30=0.01500 m
Reduction factor at the top (EC 6, §4.4.3) (Phi)i=1-2ei/t=1-2x0.01500/0.30=0.90
Design vertical load resistance Nrd=(Phi)im.t.fk/(gammaM)=1000x0.90x0.30x1.64/1.70= 259.8kN/m
Nsd= 108.0 < 259.8=Nrd. The ultimate limit state for vertical loading is verified

Masonry strength verification at the middle fifth of the height

Vertical design load in the middle fifth of the height Nm=(108.02+0.40x1.35x35.3/6.00)=111.19 kN/m
Average compressive design stress in the middle fifth of the height fsdo=0.001x111.19/0.30=0.371 N/mm²
Eccentricity of floor load in the middle fifth of the height Mm/Nm=0.20x0.00000x108.0/111.2=0.00000 m
Eccentricity in the middle fifth of the height due to horizontal forces ehm=0.00000 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity due to loads em=(Mm/Nm)+ehm+es=0.00000+0.00000+0.00500=0.00500 m
The slenderness ratio is <=15 so (EC 6 §4.4.3.2) eccentricity due to creep ek=0
Eccentricity in the middle fifth of the height (EC 6 §4.4.3) em=em+ek=0.00500+0.00000=0.00500 m
Minimum eccentricity (EC 6, §4.4.3) em=0.05t=0.05x0.30=0.01500 m
Reduction factor in the middle fifth of the height (EC 6, §4.4.3) (Phi)m=1-2em/t=1-2x0.015/0.30=0.90
Vertical design load at wall base Ni=(1.35x417.7+1.50x87.9)/6.00= 115.96 kN/m
Avearge compressive design stress at wall base qsdo=0.001x115.96/0.30= 0.387 N/mm²
Eccentricity of load at wall base Mi/Ni=0.00000x108.0/116.0=0.00000 m
Eccentricity at wall base due to horizontal loads ehi=0.00000 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity at wall base (EC 6 §4.4.3) ei=Mi/Ni+ehi+es =0.00000+0.00000+0.00500=0.00500 m
Minimum eccendricity (EC6, §4.4.3) ei=0.05t=0.05x0.30=0.01500 m
Reduction factor at wall base (EC 6, §4.4.3) (Phi)i=1-2ei/t=1-2x0.01500/0.30=0.90
Design vertical load resistance Nrd=(Phi)im.t.fk/(gammaM)=1000x0.90x0.30x1.64/1.70= 259.8kN/m
Maximum compressive stress from finite element analysis maxfn=-0.749 N/mm²
(the maximum stress which is obtained from finite element analysis is for stress regions outside the regions of stresses concentrations at the floor beam supports, which are checked separately)
Maximum design vertical load per unit length Nsd=1000x0.30x 0.749= 224.7 kN/m
Nsd= 224.7 < 259.8=Nrd. The ultimate limit state for vertical loading is verified

Strength verification in axial compression, load 1.00xg+0.30xq+Earthquake (EC 6, §4.4.2)

Masonry strength verification at the top of the masonry

Vertical design load at the top Ni=(1.00x382.4+0.30x87.9)/6.00= 68.13 kN/m
Average compression design stress at the top fsdo=0.001x68.13/0.30= 0.227 N/mm²
fsdo= 0.227<0.25N/mm² so (EC 6 Annex C.2) the eccentricity is reduced by (1-k/4)=0.66
Design bending moment at the top (EC 6, Annex C.1) Mi= 0.00 kNm/m
Eccentricity based on EC 6 Annex C.1 Mi/Ni= 0.00/ 68.13=0.00000 m =0.00x(wall thichkness)
Eccentricity at the top due to horizontal loads ehi=0.00027 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity at the top (EC 6 §4.4.3) ei=Mi/Ni+ehi+es =0.00000+0.00027+0.00500=0.00528 m
Minimum eccendricity (EC6, §4.4.3) ei=0.05t=0.05x0.30=0.01500 m
Reduction factor at the top (EC 6, §4.4.3) (Phi)i=1-2ei/t=1-2x0.01500/0.30=0.90
Design vertical load resistance Nrd=(Phi)im.t.fk/(gammaM)=1000x0.90x0.30x1.64/1.70= 259.8kN/m
Nsd= 68.1 < 259.8=Nrd. The ultimate limit state for vertical loading is verified

Masonry strength verification at the middle fifth of the height

Vertical design load in the middle fifth of the height Nm=(68.13+0.40x1.00x35.3/6.00)= 70.48 kN/m
Average compressive design stress in the middle fifth of the height fsdo=0.001x70.48/0.30=0.235 N/mm²
Eccentricity of floor load in the middle fifth of the height Mm/Nm=0.20x0.00000x68.1/70.5=0.00000 m
Eccentricity in the middle fifth of the height due to horizontal forces ehm=0.00014 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity due to loads em=(Mm/Nm)+ehm+es=0.00000+0.00014+0.00500=0.00514 m
The slenderness ratio is <=15 so (EC 6 §4.4.3.2) eccentricity due to creep ek=0
Eccentricity in the middle fifth of the height (EC 6 §4.4.3) em=em+ek=0.00514+0.00000=0.00514 m
Minimum eccentricity (EC 6, §4.4.3) em=0.05t=0.05x0.30=0.01500 m
Reduction factor in the middle fifth of the height (EC 6, §4.4.3) (Phi)m=1-2em/t=1-2x0.015/0.30=0.90
Vertical design load at wall base Ni=(1.00x417.7+0.30x87.9)/6.00= 74.01 kN/m
Avearge compressive design stress at wall base qsdo=0.001x74.01/0.30= 0.247 N/mm²
Eccentricity of load at wall base Mi/Ni=0.00000x68.1/74.0=0.00000 m
Eccentricity at wall base due to horizontal loads ehi=0.00027 m
Accidental eccentricity (EC 6.§4.4.7.2) es=hef/450= 2.25/450=0.00500 m
Eccentricity at wall base (EC 6 §4.4.3) ei=Mi/Ni+ehi+es =0.00000+0.00027+0.00500=0.00528 m
Minimum eccendricity (EC6, §4.4.3) ei=0.05t=0.05x0.30=0.01500 m
Reduction factor at wall base (EC 6, §4.4.3) (Phi)i=1-2ei/t=1-2x0.01500/0.30=0.90
Design vertical load resistance Nrd=(Phi)im.t.fk/(gammaM)=1000x0.90x0.30x1.64/1.70= 259.8kN/m
Maximum compressive stress from finite element analysis maxfn=-0.478 N/mm²
(the maximum stress which is obtained from finite element analysis is for stress regions outside the regions of stresses concentrations at the floor beam supports, which are checked separately)
Maximum design vertical load per unit length Nsd=1000x0.30x 0.478= 143.5 kN/m
Nsd= 143.5 < 259.8=Nrd. The ultimate limit state for vertical loading is verified

Strength verification in shear, load 1.00xg+0.30xq+Earthquake (EC 6.§4.5.3)

Maximum shearing stress from finite element solution (tau)max= 0.122 N/mm²
(the maximum stress which is obtained from finite element analysis is for stress regions outside the regions of stresses concentrations at the floor beam supports, which are checked separately)
Maximum design shear load per unit length Vsd=1000x0.30x 0.122= 36.6 KN/m
Characteristic shear strength fvk=0.70x(fvko+0.4xfd) (EC 6 §3.6.3 and 3.6.3.(8))
fvk=0.70x(0.150+0.4x 0.478)= 0.239 N/mm², maxfvk= 1.000 N/mm²
Design shear resistance of masonry Vrd=fvk.t./(gammaM) (EC 6 §4.5.3)
Vrd=1000x 0.239x0.30/1.70= 42.2. kN/m
Vsd= 36.6 < 42.2=Vrd. The ultimate limit state for shear loading is verified.

Verification of regions with concentrated loads (EC 6, §4.4.8)

From the finite element solution is obtained at the support positions of the floor beams at the top the maximum compressive stresses. Because at the wall top allways exists a bond beam from reinforced concrete, we check the stresses in the finite elements a line bellow.
The maximum compressive stress in the region of stress concentration is fsdmax= 0.861N/mm²
This maximum stress 0.861 is < fk/(gammaM)= 1.64/1.7= 0.96
The strength requirements for concentrated loads are verified (EC 6 §4.4.8)

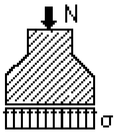
Top beam of reinforced concrete (EC 6 §5.2)

On the top of the wall and the top of the openings, some small tensile stresses are taken from reinforced concrete bond beams or lintels at these places 30x20 (width x height in cm), with minimum reinforcement 4#12 (stirrups #8/20) which satisfies the minimum code requirements.

Masonry wall foundation

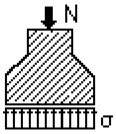
Allowable soil pressure qsoil,all= 0.20 [MPa=N/mm²]

Masonry foundation: W1   Dimensions length= 6.00m, height= 3.00m, thickness=0.30m



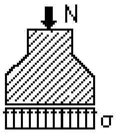
Total vertical load =1.35x 195.1+1.50x 35.9= 317.2 [kN]
Load per unit wall length +foot self weight= 55 [kN/m]
For footing width=0.50[m], maximum soil pressure qsoil=0.001x55/0.50=0.11[MPa]
maxqsoil=0.11<0.20=allowable soil pressure [MPa]

Masonry foundation: W2   Dimensions length=10.00m, height= 3.00m, thickness=0.30m



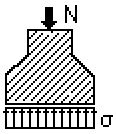
Total vertical load =1.35x 186.0+1.50x 15.4= 274.2 [kN]
Load per unit wall length +foot self weight= 29 [kN/m]
For footing width=0.50[m], maximum soil pressure qsoil=0.001x29/0.50=0.06[MPa]
maxqsoil=0.06<0.20=allowable soil pressure [MPa]

Masonry foundation: W3   Dimensions length= 6.00m, height= 3.00m, thickness=0.30m



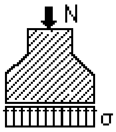
Total vertical load =1.35x 195.1+1.50x 35.9= 317.2 [kN]
Load per unit wall length +foot self weight= 55 [kN/m]
For footing width=0.50[m], maximum soil pressure qsoil=0.001x55/0.50=0.11[MPa]
maxqsoil=0.11<0.20=allowable soil pressure [MPa]

Masonry foundation: W4   Dimensions length=10.00m, height= 3.00m, thickness=0.30m

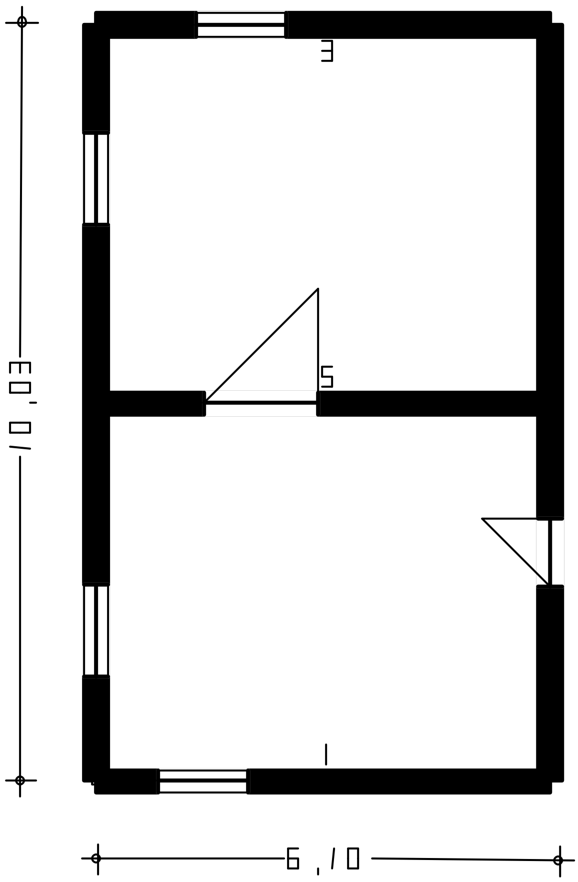


Total vertical load =1.35x 190.2+1.50x 15.4= 279.9 [kN]
Load per unit wall length +foot self weight= 30 [kN/m]
For footing width=0.50[m], maximum soil pressure qsoil=0.001x30/0.50=0.06[MPa]
maxqsoil=0.06<0.20=allowable soil pressure [MPa]

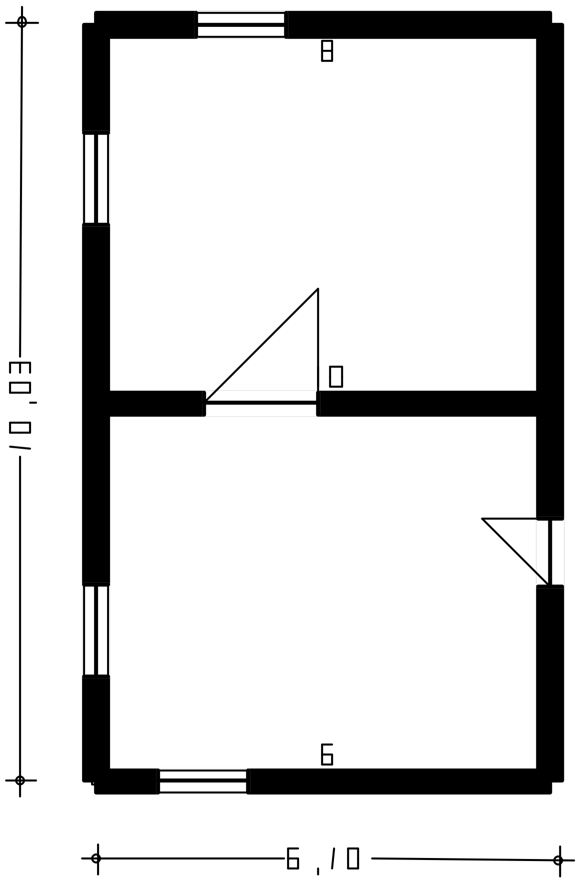
Masonry foundation: W5   Dimensions length= 6.00m, height= 3.00m, thickness=0.30m



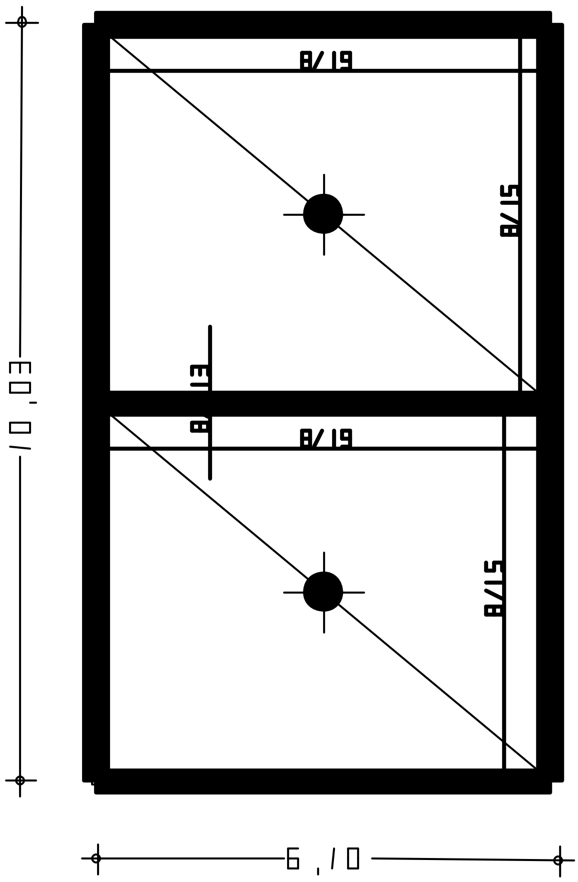
Total vertical load =1.35x 417.7+1.50x 87.9= 695.7 [kN]
Load per unit wall length +foot self weight= 118 [kN/m]
For footing width=0.59[m], maximum soil pressure qsoil=0.001x118/0.59=0.20[MPa]
maxqsoil=0.20<0.20=allowable soil pressure [MPa]



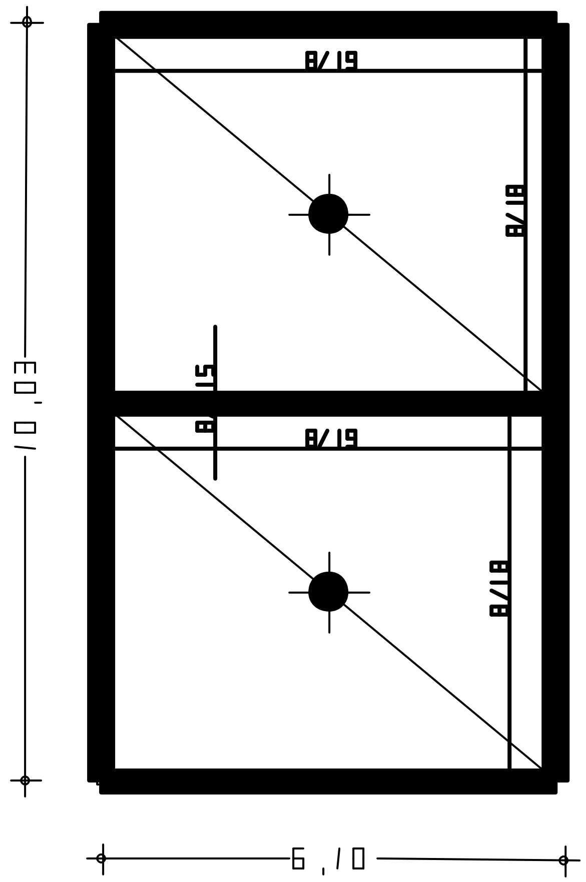
POSITION		SUBJECT	Plan of ground floor
PROJECT	Two floor building from YTONG	SCALE	1:100
DESIGN		DATE	23/8/2004



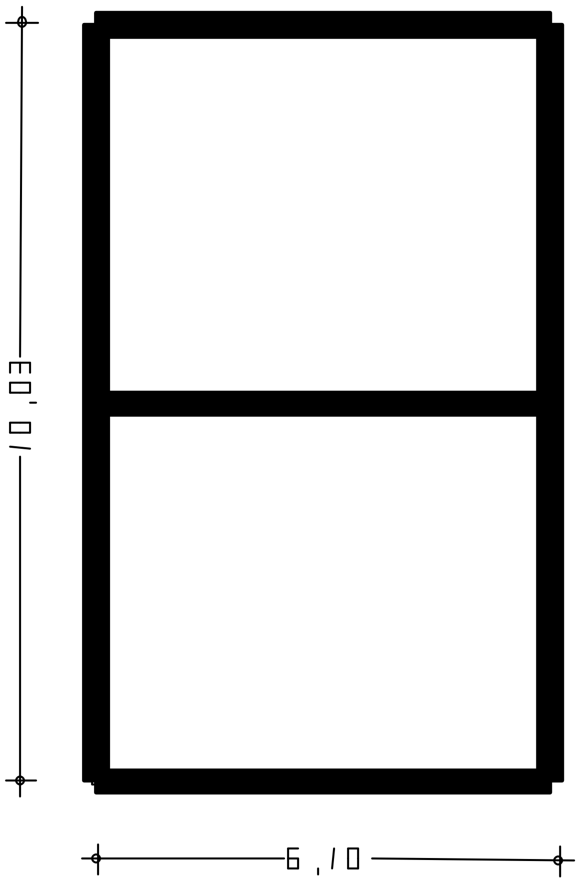
POSITION		SUBJECT	Plan of 1st floor
PROJECT	Two floor building from YTONG	SCALE	1:100
DESIGN		DATE	23/8/2004



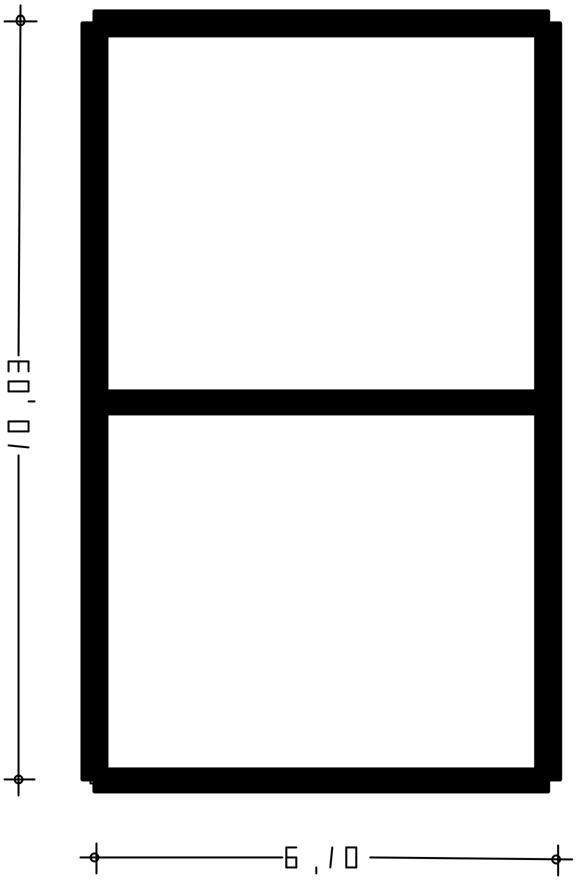
POSITION		SUBJECT	Slab reinforcement of ground floor
PROJECT	Two floor building from YTONG	SCALE	1:100
DESIGN		DATE	23/8/2004



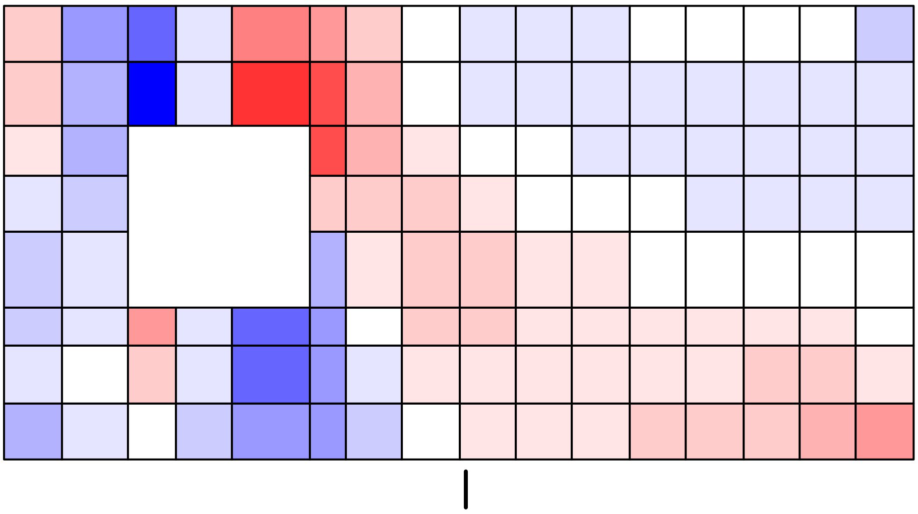
POSITION		SUBJECT	Slab reinforcement of 1st floor
PROJECT	Two floor building from YTONG	SCALE	1:100
DESIGN		DATE	23/8/2004



POSITION		SUBJECT	Beam reinforcement of ground floor
PROJECT	Two floor building from YTONG	SCALE	1:100
DESIGN		DATE	23/8/2004



POSITION		SUBJECT	Beam reinforcement of 1st floor
PROJECT	Two floor building from YTONG	SCALE	1:100
DESIGN		DATE	23/8/2004



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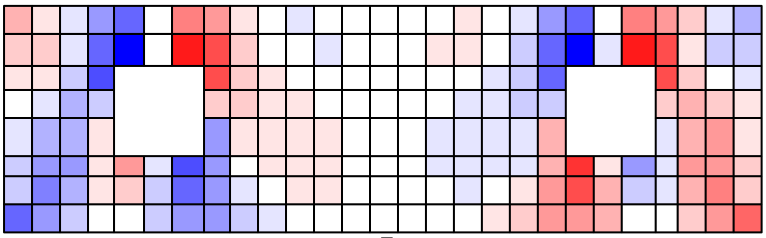
PROJECT	Two floor building from YTONG
POSITION	
DESIGN	
SUBJECT	Load 1.35g+1.50q tau xy of wall W1
SCALE	1:50
DATE	23/8/2004

N/mm²



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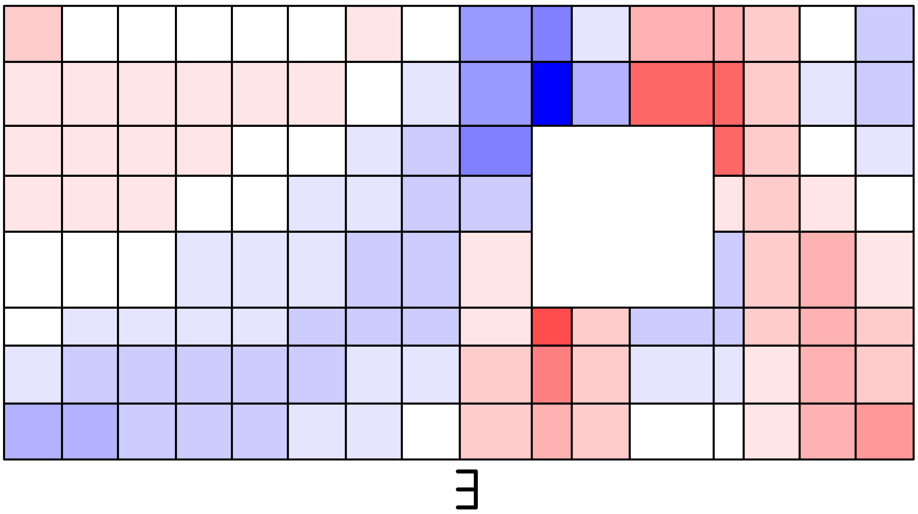
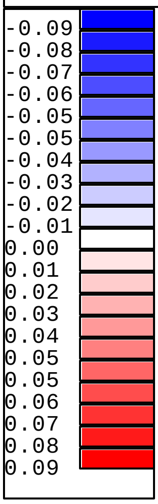
N/mm²

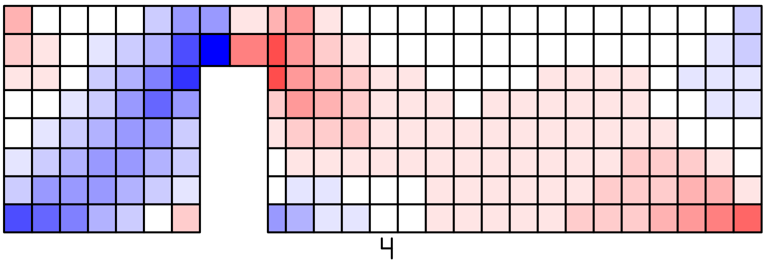
PROJECT	Two floor building from YTONG
POSITION	
DESIGN	
SUBJECT	Load 1.35g+1.50q tau xy of wall W2
SCALE	1:100
DATE	23/8/2004





N/mm²

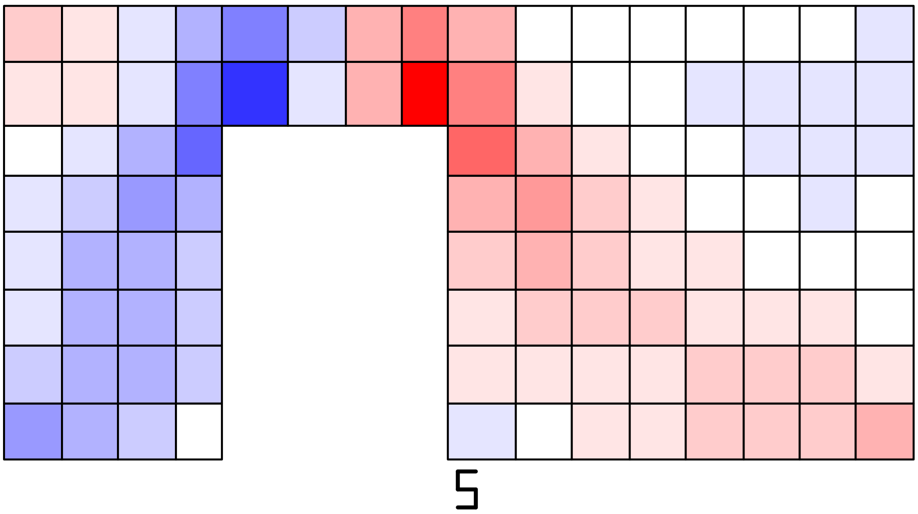
PROJECT	Two floor building from YTONG
POSITION	
DESIGN	
SUBJECT	Load 1.35g+1.50q tau xy of wall W3
SCALE	1:50
DATE	23/8/2004



-0.03
-0.03
-0.02
-0.02
-0.02
-0.01
-0.01
-0.01
-0.01
-0.00
0.00
0.00
0.01
0.01
0.01
0.01
0.02
0.02
0.02
0.03
0.03

N/mm²

PROJECT	Two floor building from YTONG
POSITION	
DESIGN	
SUBJECT	Load 1.35g+1.50q tau xy of wall W4
SCALE	1:100
DATE	23/8/2004



-0.27
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-0.22
-0.19
-0.16
-0.14
-0.11
-0.08
-0.05
-0.03
0.00
0.03
0.05
0.08
0.11
0.14
0.16
0.19
0.22
0.25
0.27

N/mm²

PROJECT	Two floor building from YTONG
POSITION	
DESIGN	
SUBJECT	Load 1.35g+1.50q tau xy of wall W5
SCALE	1:50
DATE	23/8/2004

TECHNICAL REPORT

Solution methodology and dimensioning

The Design of the masonry buildings is based on the assumption that the maximum part of the vertical and horizontal loads are taken from the masonry.
The concrete floor design in vertical loads is done considering the beams as space grillage and by moving the loads so to obtain the worse loading conditions.The concrete slabs are solved with the method of Marcus. The horizontal seismic forces on each floor considered as equivalent static loads. The distribution of the seismic forces on the walls is done after the computation of the wall stiffness using finite element analysis. The wall stresses are also computed using finite element analysis.
The dimensioning of the concrete elements (slabs, beams, columns, footings) is based on Eurocode 2. The masonry dimensioning is done using the Eurocode 6. The timber roof is dimensioned using Eurocode 5.

Slabs

The design of concrete slabs is based on Marcus method. This method is based on the solution of unit plate strips located at midspans, with equal deflections at the plate centers. From this assumption is obtained the plate load distribution in the two main plate directions. The advantage of the plate torsional resistance is not taken into account. Each plate strip is solved as a continuous beam. The solution is obtained through specific coefficients which are obtained from the solution of continuous beams of equal spans. These coefficients are taken such as to obtain the maximum internal forces in each case. The minimum (maximum in absolute value) support bending moments are obtained using the most unfavorable position of live loads in an equivalent continuous beam. Correspondingly the maximum (minimum in absolute value) support moments are obtained using the most favorable position of live loads, and from these support moments are obtained the maximum span Moments with additional span loading 1.35g+1.50 q.
The loads transferred on the beams and walls are obtained for loading with live load both slabs on the left and right side of the beam or wall. In the case of slabs with span ratio over 2, or load factor <0.10, the load is transferred only in one direction. In this case the beam which does not take load from the slab is loaded with a minimum uniform load equal to wL/4 where w=1.35g+1.50q. (g, q dead and live load of the plate, L the beam span).
The design for ultimate strength is done according to Eurocode 2 §4.3.1. The design for serviceability conditions is base on control of the slenderness ratio (EC2 §4.4.2.3). In addition the minimum steel reinforcement requirements are verified. The minimum cover for steel reinforcement is set to 20 mm which satisfies the code requirements (EC2 § 4.1.3.2) for dry or humid environment.

Beams

The concrete floor beam system is designed as a system of beam grid. The structural analysis is done with finite elements. The finite elements are beams with 3 degrees of freedom per node, rotations around x-x and y-y axis and vertical displacement along the z-z axis. The grid is supported on the walls and the columns. When the wall is not parallel to the beam axis the rotations are zero. For the computation of the beam stiffness the effective flange width is taken 0.70L/10 for each beam flange (left or right).
The solution is done for unit uniform loads on each span of the grid. The most unfavorable load combinations are obtained with combination of the unit loads results (1.35g and 1.50 q). The solution is done with Gauss method for symmetric banded matrices.
The dimensioning of beams is done based on Eurocode 2. For the design the support bending moments are taken at a distance 10 cm from the support

(wall or column) axis. The design shearing force values are taken at a distance d (beam height) from the support face (EC2 §4.3.2.3). The effective flange width is taken 0.70L/10 for each beam flange left or right. The minimum reinforcing steel coverage is set to 50 mm which satisfies the code requirements (EC2 §4.1.3.2) for dry or humid environment. We use only straight reinforcing steel bars, and the shear force is taken only with vertical stirrups. The minimum requirements for steel reinforcement are verified. The verification of crack width requirements and maximum deformations are done according to (EC2 §4.4.2).

Masonry

The masonry walls are carrying most of the vertical and all the horizontal loads. The computation of the horizontal seismic forces for each floor level is based on equivalent static loads. The vertical distribution of the seismic loads is reverse triangular.
The distribution of the total horizontal floor force on the masonry walls is done using the stiffness of each wall. This stiffness depends on the wall dimensions and the dimensions and positions of the openings. The wall stiffness is computed wit a finite element analysis of each wall, for unit relative displacement between the top and bottom wall ends. After the computation of the horizontal loads the evaluation of the internal stresses of the walls is done also with a finite element analysis, for the various load combinations.
The design for the masonry is done for the ultimate limit state based on Eurocode 6, chapter 4. All the checks for loading cases 1.35g+1.50q, and 1.00g+0.30q+earthquake, are done for compression, and shear. In addition verification of slenderness ratio requirements and checks for strength at stress concentrations are performed according to Eurocode 6.

These checks are:

Nsd<Nrd, Nrd =design vertical load resistance (Eurocode 6 §4.4.1).
Nsd Vertical design load, which is evaluated as vertical load per unit length from the maximum compressive stresses, obtained from the finite element solution (the regions of stress concentrations at beam supports are excluded).

Nrd=Fi,m t fk/?M

- **Fi,m** is the capacity reduction factor, which takes into account the effects of slenderness and eccentricity of the loading. The eccentricities for the computation of capacity reduction factors are computed from the loads on the slabs and beams based on Eurocode6 §4.4.3 and appendix C.
- **t** : is the wall thickness,
- **fk** : is the characteristic compressive strength of the masonry which is obtained based on Eurocode 6 chapter 3, for each masonry type depending on the masonry units, and the masonry mortar.
- **?M** : is the partial safety factor for the material and is obtained according to Eurocode 6 table 2.3.

The slenderness ratio check performed based on Eurocode 6 §4.4.4.3. The effective height of the wall is taken **hef=?h h**. The coefficient ? is computed for partial or complete restrain on the top and bottom of the wall and we consider ?3=?4=1 for vertical wall edges, as most unfavorable.
The shear verification is done according to Eurocode 6 §4.5.3.
Vsd<Vrd.
Vsd is the applied shear load which is computed as horizontal force per unit length from the maximum shearing stresses obtained from finite element analysis (excluding stress concentrations at beam supports),.
Vrd=fvk t L /?M
The maximum compressive stresses obtained from finite element analysis at the places of beam supports are verified according to §4.4.8 to be less than fk/fM.

Seismic Design

The seismic design is based on equivalent static loads at the level of each floor.
The total seismic force is defined proportional to the total vertical load, by a factor defined as the ratio of the horizontal seismic ground acceleration to the acceleration of gravity g. The distribution of the seismic force over the building height, is a reverse triangular distribution.
At each floor the eccentricity of the horizontal loading is computed. The horizontal load of each floor is applied to the mass center of the floor, and the building is assumed to rotate around an elastic axis. The elastic axis is defined as the axis passing through the elastic center of the floor which is more near to the level 0.8H, where H is the building height.