2. PRELIMINARY DESIGN OF STRUCTURAL ELEMENTS' DIMENSIONS

2.1. USED MATERIAL

Concrete class C30/37: f_{ck}=30 MPa

f_{cd}=20 MPa

Reinforcing bars' diameter: φ=10 mm

Steel class B500 B: f_{yk}=500 MPa

f_{yd}=435 MPa

2.2. DESIGN OF HORIZONTAL LOAD BEARING STRUCTURES - SLAB

Empirical estimation: $hs \geq \frac{1,1}{33} * l_{n,max} = \frac{1,1}{33} * 6300$ mm

$h_s \geq 210$ mm → $h_s = 240$ mm (estimation)

Cover depth: $c = c_{min} + \Delta c_{dev}$

$c_{min} = \max(c_{min,b};\ c_{min,dur};\ 10\text{ mm})$

$c_{min,dur} = 10$ mm (S3, XC1) → $c_{min} = 10$ mm

$c = 10$ mm + 10 mm = 20 mm

Effective depth: $d = h_s - c - \frac{\phi}{2} = 240 - 20 - \frac{10}{2} = \underline{\underline{215\text{ mm}}}$

Span/depth ratio: $\lambda = \frac{l}{d} \leq \lambda_{lim} = \kappa_{c1} * \kappa_{c2} * \kappa_{c3} * \lambda_{d,tab}$

$\kappa_{c1} = 1$, $\kappa_{c2} = 1$, $\kappa_{c3} = 1,25$ (reinf. utilisation 80 %)

$\lambda_{d,tab} = 24,6$ (ρ = 0,5 %)

$d \geq \frac{6,6}{1*1*1,25*24,6} = 0,21463$ m

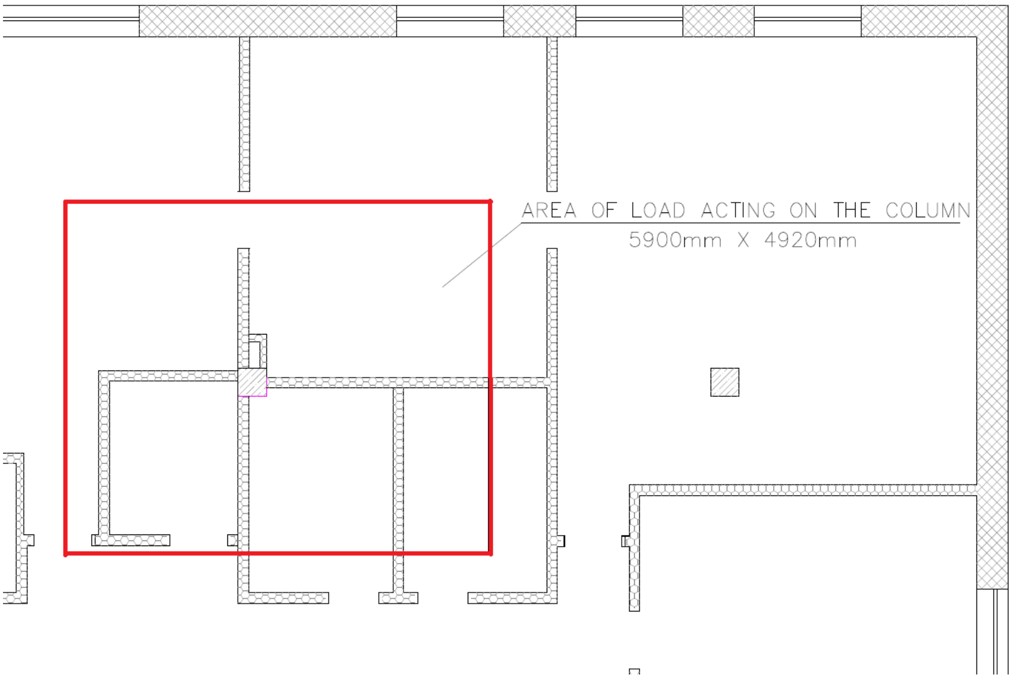
$d \geq 214,63$ mm O.K.

$h_s \geq d + \frac{\phi}{2} + c = 215 + 5 + 20 = 240$ O.K.

CONCLUSION: The slab is designed with thickness 240 mm, cover depth 20 mm, reinforcing bars diameter 10 mm. Its effective depthis 215 mm.

2.3. DESIGN OF VERTICAL LOAD BEARING STRUCTURES - COLUMN

2.3.1. COLUMN'S LOADING AREA



A_{CA} = 4,92 m * 5,9 m = 29,028 m^2

2.3.2. LOADS ACTING ON THE VERTICAL STRUCTURE

2.3.2.1. FLOOR SLAB COMPOSITION

Tiles:	ρ_1=	2300	kg/m^3
	t_1=	10	mm
Anhydrite layer:	ρ_2=	2200	kg/m^3
	t_2=	40	mm
Separation layer:	ρ_3=	500	kg/m^3
	t_3=	1	mm
XPS insulation:	ρ_4=	40	kg/m^3
	t_4=	50	mm
Vapour barrier PE:	ρ_5=	0,19	kg/m^2
Gypsum board 12,5mm:	ρ_6=	12,5	kg/m^2

2.3.2.2. LOAD FROM A FLOOR SLAB – PLANAR LOAD

Permanent:	k-value [kN/m^2]	γ	d-value [kN/m^2]
Self-weight= 25 kN/m^3*0,24 m	6	1,35	8,1
Tiles=23 kN/m^3*0,01m	0,23	1,35	0,3105
Anhydrite= 22 kN/m^3*0,04m	0,88	1,35	1,188
Separation=5 kN/m^3* 0,001m	0,005	1,35	0,00675
XPS=0,4 kN/m^3*0,05m	0,02	1,35	0,027
PE foil=	0,0019	1,35	0,00257
Gypsum board=	0,125	1,35	0,16875
Variable: Live load=	1,5	1,5	2,25
f_s=	8,7619	-	12,0536

category A (residential objects)

CONCLUSION: The total value of load of a floor structure is 8,762 kN/m^2. For further calculations the design value of 12,054 kN/m^2 will be used.

2.3.2.3. ROOF SLAB COMPOSITION

Roof tiles:	ρ_1=	13,4	kg/m^2
Wooden laths, est.:	ρ_2=	50	kg/m^2
Safety waterproofing:	ρ_3=	0,06	kg/m^2
MW insulation:	ρ_4=	150	kg/m^3
	t_4=	220	mm
Vapour barrier:	ρ_5=	0,14	kg/m^2
Gypsum board 12,5 mm:	ρ_6=	12,5	kg/m^2

2.3.2.4. LOAD FROM A ROOF SLAB – PLANAR LOAD

Permanent:	k-value [kN/m^2]	γ	d-value [kN/m^2]
Self-weight=	0,5	1,35	0,675
Roofing=	0,134	1,35	0,1809
Waterproofing=	0,0006	1,35	0,00081
MW=1,5 kN/m^3*0,22m	0,33	1,35	0,4455
Vapour barrier=	0,0014	1,35	0,00189
Gypsum board=	0,125	1,35	0,16875

Variable: Live load=	1	1,5	1,5
Snow load=0,8*0,8*1* 0,7 kN/m2	0,448	1,5	0,672

category H (inaccessible roof)
Prague periphery:
$s=C_e*\mu_1*\mu_2*s_k=0,8*0,8*1*0,7$ kN/m^2

f_r=	2,539	-	3,64485

CONCLUSION: The total value of load of a roof structure is 2,539 kN/m^2. For further calculations the design value of 3,645 kN/m^2 will be used.

2.3.2.5. LOAD FROM PARTITIONS – LINEAR LOAD

ρ_p=	500	kg/m^3	*lightweight concrete YTONG*
t_p=	150	mm	
h_p=	3260	mm	*3500 mm-h_s*

$f_{p,k} = (\rho_p /100) * t_p * h_p = (500/100) * 0,15 * 3,26 = 2,445$ kN/m

$f_{p,d} = f_{p,k} * 1,35 = 2,445 * 1,35 = 3,301$ kN/m

$l_{p,x}$=	8260	mm	*length of partitions in x-direction on a column load area*
$l_{p,y}$=	9020	mm	*length of partitions in y-direction on a column load area*

CONCLUSION: The total value of load of partitions is 2,445 kN/m. For further calculations the design value of 3,301 kN/m will be used.

2.3.2.6. LOAD FROM A COLUMN – FORCE LOAD

ρ_c=	2500	kg/m^3	
$d_{c,est}$=	300	mm	*estimated dimension of column*
h_c=	3260	mm	*3500 mm-h_s*

$F_{c,k} = (\rho_c /100) * d_{c,est}^2 * h_c = (2500/100) * 0{,}30^2 * 3{,}26 = 7{,}335$ kN

$F_{c,d} = 1{,}35 * F_{c,k} = 1{,}35 * 7{,}335 = 9{,}9$ kN

CONCLUSION: The total estimated value of load of a column is 7,335 kN. For further calculations the design value of 9,9 kN will be used.

2.3.2.7. LOAD ACTING ON THE MOST LOADED COLUMN

Load	type of load	Live/dead load	K-value	unit	γ	D-value	unit	n of floors of action	calculation	total load for all floors	unit
Floor slab	planar	live + dead	8,7619	kN/m^2	1,35; 1,5	12,054	kN/m^2	4	4*12,054*A_{ca} = 4*12,054*29,028 =	1399,6	kN
Roof slab	planar	live + dead	2,539	kN/m^2	1,35; 1,5	3,645	kN/m^2	1	1*3,645*A_{ca} = 1*3,645*29,028 =	105,8	kN
Partitions	linear	dead	2,445	kN/m	1,35	3,301	kN/m	4	4*3,301* ($l_{p,x}$ + $l_{p,y}$) = 4*3,301* (8,26 + 9,02) =	228,15	kN
Column	force	dead	7,335	kN	1,35	9,9	kN	4	4*9,9 =	39,6	kN

$N_{Ed,col.} = 1399{,}6 + 105{,}8 + 228{,}5 + 39{,}6 = 1773{,}15$ kN

CONCLUSION: The total value of all loads acting on the most loaded column is 1773,15 kN.

2.3.3. COLUMN'S DIMENSIONS

$N_{Rd} \geq N_{Ed}$

$N_{Rd} = 0{,}8 * A_c * f_{cd} + A_s * \sigma_s \geq N_{Ed}$

$A_s = 0,02*A_c$

$\sigma_s = 400$ MPa

$$A_c \geq \frac{N_{Ed}}{0,8*f_{cd} + 0,02*\sigma_s} = \frac{1773,15*10^3}{0,8*20 + 0,02*400}$$

$A_c \geq 73881\ mm^2$

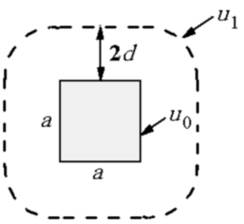
$d_c \geq \sqrt{A_c} = \sqrt{73881} = 272$ mm

dc = 300 mm

$N_{Rd} = 0,8 * 300^2 * 20 + 0,02 * 300^2 * 400 = 2160000$ N = 2160 kN ≥ 1773,15 kN O.K.

CONCLUSION: The designed dimensions of a column are 300 x 300 mm.

2.4. PRELIMINARY CHECK OF PUNCHING OF A COLUMN



$u_0 = 4*a = 4*300 = 1200$ mm

$u_1 = 4*a + 2*\pi * 2*d = 1200 + 2*\pi * 2*215$ (page 1)

$u_1 = 3901,77$ mm

2.4.1. MAXIMUM PUNCHING SHEAR RESISTANCE

$$v_{Ed,0} = \frac{\beta * V_{Ed}}{u_o*d} \leq v_{Rd,max} = 0,4*v*f_{cd}$$

$\beta = 1,15$ (inner column)

$V_{Ed} = F_{s,d}/4 = 1399,6/4 = 349,89$ kN (normal force in the column from one floor)

$$v = 0,6*(1- \frac{f_{ck}}{250}) = 0,6*(1- \frac{30}{250}) = 0,528$$

$$v_{Ed,0} = \frac{1,15*349,89*10^3}{1200*215} = 1,56 \text{ MPa}$$

$v_{Rd,max} = 0,4*0,528*20 = 4,224$ MPa $> v_{Ed,0}$

O.K.

2.4.2. MAXIMUM RESISTANCE WITH REINFORCEMENT

$$v_{Ed,1} = \frac{\beta * V_{Ed}}{u_1 * d} \leq k_{max} * v_{Rd,c}$$

$v_{Rd,c} = C_{Rd,c} * k * \sqrt[3]{(100\rho_l * f_{ck})}$

$k_{max} = 1{,}4575$ *coefficient of maximum resistence, interpolated value*

$C_{Rd,c} = 0{,}12$ *reduction factor*

$k = 1 + \sqrt{\frac{200}{d}} \leq 2{,}0$ *effect of depth*

$k = 1 + \sqrt{\frac{200}{215}} = 1{,}96 < 2{,}0$

$\rho_l = 0{,}005$ *ratio of tensile reinforcement, estimated value*

$v_{Rd,c} = 0{,}12 * 1{,}96 * \sqrt[3]{(100 * 0{,}005 * 30)} = 0{,}581$ MPa

$$v_{Ed,1} = \frac{1{,}15 * 349{,}89 * 10^3}{3901{,}77 * 215} = 0{,}48 \text{ MPa}$$

$k_{max} * v_{Rd,c} = 1{,}4575 * 0{,}581 = 0{,}847$ MPa $> v_{Ed,1}$

O.K.

CONCLUSION: The structure is suitable for punching reinforcement design.

2.5. CHECK OF LOAD-BEARING CAPACITY OF MASONRY WALL

2.5.1. STRENGTH OF MASONRY

The type of bricks used: HELUZ FAMILY 44, dimensions 440 x 247 x 249 mm

$f_u = 8$ MPa *compressive strength*

$f_m = 5$ MPa *strength of mortar*

$f_b = \delta * f_u$

$\delta = 1{,}155$ *effect of bricks' dimensions, value taken from table*

$f_b = 1{,}155 * 8 = 9{,}24$ MPa *compressive strength of the units*

$f_k = K * f_b^{0,7} * f_m^{0,3}$

$K = 0,65$ *coefficient, value taken from table*

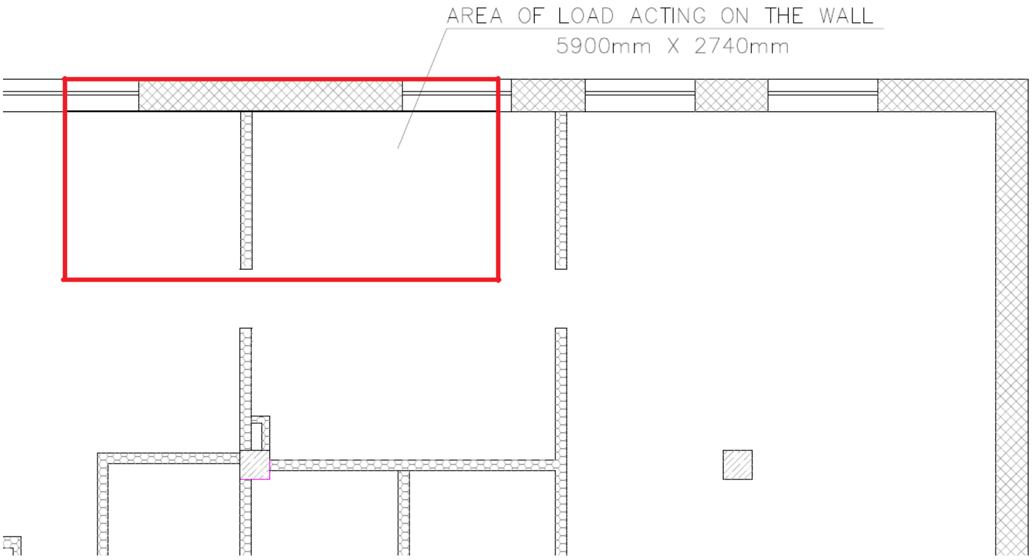
$f_k = 0,65 * 9,24^{0,7} * 5^{0,3} = 4,995$ MPa *characteristic compressive strength of masonry*

$f_d = f_k/\gamma_m$

$\gamma_m = 2$ *partial factor for material, designed mortar*

$f_d = 4,995/2 = 2,498$ MPa *design compressive strength of masonry*

2.5.2. LOADS ACTING ON MASONRY WALL



$A_{WA} = 5,9 * 2,74 = 16,166$ m^2 *area of load acting on the wall*

2.5.2.1. LOAD FROM SLABS – ROOF AND TYPICAL FLOORS – PLANAR LOAD

NOTE: The values are the same as in *2.3.2. LOADS ACTING ON THE VERTICAL STRUCTURE*.

2.5.2.2. LOAD FROM PARTITIONS – LINEAR LOAD

ρ_p=	500	kg/m^3	*lightweight concrete YTONG*
t_p=	150	mm	
h_p=	3260	mm	*3500 mm-h_s*

$f_{p,k} = (\rho_p /100) * t_p * h_p = (500/100) * 0,15 * 3,26 = 2,445$ kN/m

$f_{p,d} = f_{p,k} * 1,35 = 2,455 * 1,35 = 3,301$ kN/m

$l_{p,x}$=	0	mm	*length of partitions in x-direction on a wall load area*
$l_{p,y}$=	2300	mm	*length of partitions in y-direction on a wall load area*

NOTE: The values of characterisitc and design partition linear load are the same as in *2.3.2. LOADS ACTING ON THE VERTICAL STRUCTURE.*

2.5.2.3. LOAD FROM MASONRY – LINEAR LOAD

ρ_w=	650	kg/m^3	*density of the material*
t_w=	440	mm	
h_w=	3260	mm	*3500 mm-h_s*

$f_{w,k} = (\rho_w /100) * t_w * h_w = (650/100) * 0,44 * 3,26 = 9,324$ kN/m

$f_{w,d} = f_{w,k} * 1,35 = 9,324 * 1,35 = 12,587$ kN/m

$l_{w,x}$=	5900	mm	*length of walls in x-direction on a wall load area*
$l_{w,y}$=	0	mm	*length of wall in y-direction on a wall load area*

CONCLUSION: The total value of load of masonry walls is 9,324 kN/m. For further calculations the design value of 12,587 kN/m will be used.

2.5.2.4. LOAD ACTING ON THE MOST LOADED PART OF MASONRY WALL

Load	**type of load**	**Live/dead load**	**K-value**	**unit**	**γ**	**D-value**	**unit**	**n of floors of action**	**calculation**	**total load for all floors**	**unit**
Floor slab	planar	live + dead	8,7619	kN/m^2	1,35; 1,5	12,054	kN/m^2	3	$3*12,054*A_w =$ $3*12,054*16,166 =$	584,6	kN
Roof slab	planar	live + dead	2,539	kN/m^2	1,35; 1,5	3,645	kN/m^2	1	$1*3,645*A_w =$ $1*3,645*16,166 =$	58,9	kN
Partitions	linear	dead	2,445	kN/m	1,35	3,301	kN/m	2	$2*3,301* (l_{p,x} + l_{p,y}) =$ $2*3,301* (0 + 2,3) =$	15,2	kN
Masonry	linear	dead	9,324	kN/m	1,35	12,587	kN/m	2	$2*12,587* (l_{w,x} + l_{w,y})=$ $2*12,587* (5,9 + 0) =$	148,5	kN

$N_{Ed,wall.}$ = 584,6 + 58,9 + 15,2 + 148,5 = 807,2 kN

CONCLUSION: The total value of all loads acting on the most loaded part of masonry wall is 807,2 kN.

2.5.3. CHECK OF RESISTANCE OF THE MASONRY WALL

2.5.3.1. SLENDERNESS RATIO

$h_{ef}/t \leq 27$

$h_{ef} = \rho_n * h_w$

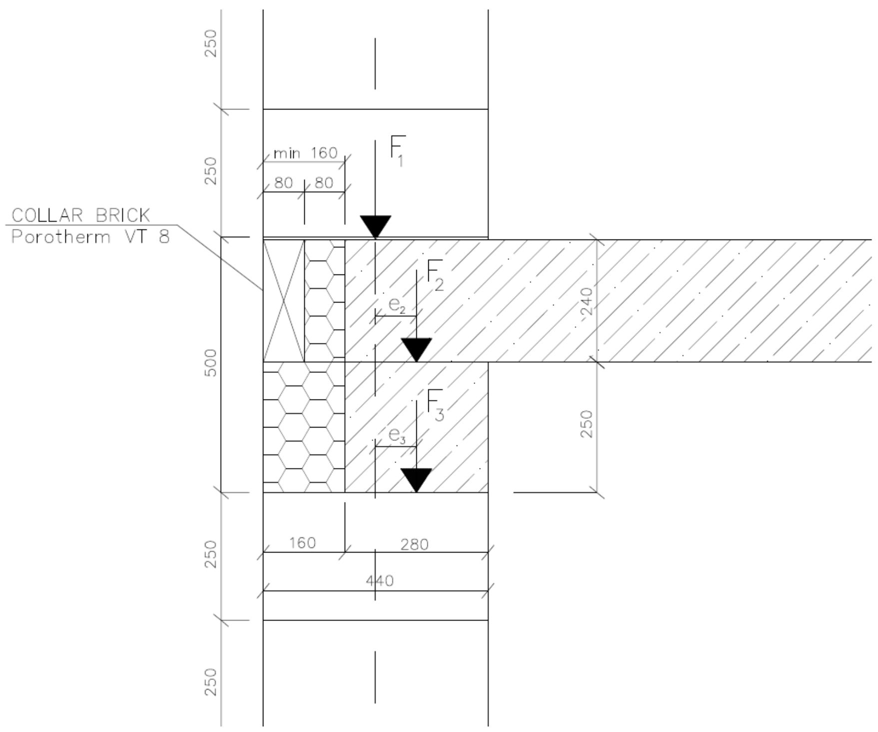
$\rho_n = 0,75$ *factor of supporting the wall at its top and bottom*

$h_w = 3260$ mm *3500 – h_s*

h_{ef} = 0,75 * 3260 = 2445 mm = 2,445 m

h_{ef}/t = 2,445/0,44 = 5,557 < 27 O.K.

2.5.3.2. ECCENTRICITY DUE TO LOADS



F_1 *force from the structure above (excl. the connected slab)*

F_1 = 1* roof load + 2* slab load + 2* masonry wall load + 2* partition load

F_1 = 58,9 kN + 2* (584,6 kN /3) + 148,5 kN + 15,2 kN = 612,3 kN

e_1 = 0 mm

F_2 *force from the connected slab*

F_2 = 1* slab load = 584,6 kN /3 = 194,87 kN

e_2 = 80 mm = 0,08 m

F_3 *force from concrete beam*

F_2 = (0,28 m * 0,25 m * 5,9 m) * 25 kN/m^3 = 10,325 kN

e_2 = 80 mm = 0,08 m

NOTE: The design values of force loads are taken from *2.5.2. LOADS ACTING ON MASONRY WALL*.

$$e_f = \frac{\Sigma F_i * e_i}{\Sigma F_i} = \frac{F_1 * e_1 + F_2 * e_2 + F_3 * e_3}{F_1 + F_2 + F_3}$$

$$e_f = \frac{612{,}3 * 0 + 194{,}87 * 0{,}08 + 10{,}325 * 0{,}08}{612{,}3 + 194{,}87 + 10{,}325} = \frac{16{,}4156}{817{,}495} = 0{,}0201$$

$e_a = h_{ef}/450 = 2{,}445/450 = 0{,}00543$ *initial eccentricity*

$e_i = e_f + e_a = 0{,}0201 + 0{,}00543 = 0{,}02553$ *total eccentricity*

$$\phi_i = 1 - \frac{2*e_i}{t} = 1 - \frac{2*0{,}02553}{0{,}44} = 0{,}0884$$

capacity redution factor

2.5.3.3. RESISTANCE AT THE TOP OF THE WALL

$N_{Ed,A} = N_{Ed,wall}$ = 807,2 kN

NOTE: The design value of force load acting on the top of the wall is taken from *2.5.2. LOADS ACTING ON MASONRY WALL*.

$N_{Rd,i}= \phi_i * A * f_d$

$A = t * l_{w,x} = 0,44 * 5,9 = 2,596\ m^2$

$f_d = 2,498$ MPa

NOTE: The value of design compressive strength of masonry is taken from *1.5.1. STREGTH OF MASONRY*.

$N_{Rd,i}= 0,884 * 2,596 * 2,498*10^3 = 5726,1\ kN > N_{Ed,A}$ O.K.

2.5.3.4. RESISTANCE AT THE MIDDLE OF THE WALL

$N_{Ed,B} = N_{Ed,wall}$ + ½ wall = 807,2 kN + (148,5kN/2)/2 = 844,3 kN

NOTE: The design value of force load of a masonry wall is taken from *2.5.2. LOADS ACTING ON MASONRY WALL.*

$N_{Rd,m}= \phi_m * A * f_d$

$e_{mf} = e_f *0,5 = 0,0201 * 0,5 = 0,01005$

$$\frac{e_{mk}}{t} = \frac{e_{mf} + e_a + e_k}{t}$$

relative eccentricity

$e_k \approx 0$ *eccentricity due to creep*

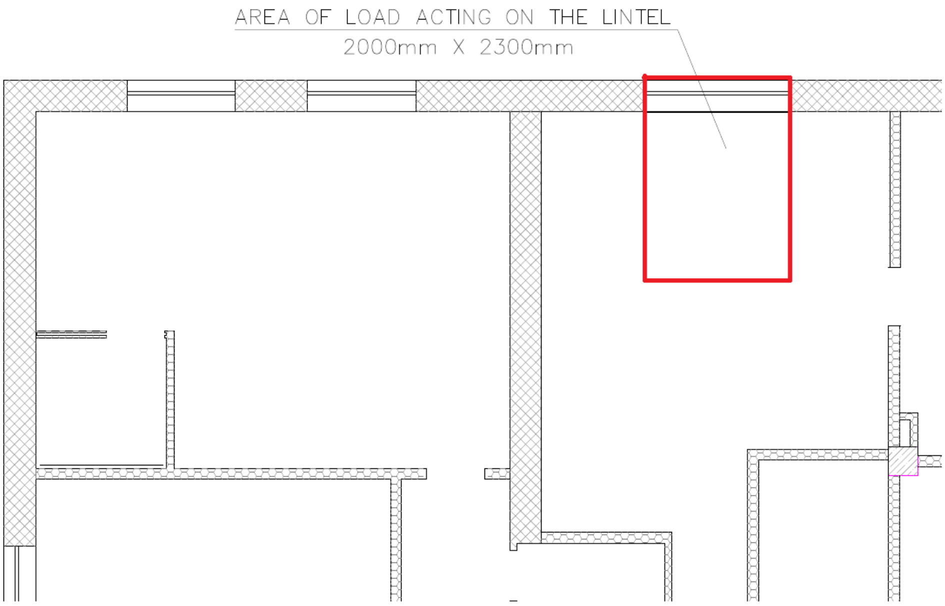
$$\frac{e_{mk}}{t} = \frac{0,01005 + 0,00543 + 0}{0,44} = 0,0352$$

$\phi_m= 0,885$ *capacity reduction factor, value taken from table, estimated eccentricity 0,05*

$N_{Rd,m}= 0,885* 2,596 * 2,498*10^3 = 5739\ kN > N_{Ed,B}$ O.K.

CONCLUSION: The masonry wall is able to withstand the loading.

2.6. CHECK OF LOAD-BEARING CAPACITY OF LINTELS



$A_L = 2 * 2,3 = 4,6\ m^2$ *area of load acting on the lintel*

2.6.1. LOAD ACTING ON THE LINTEL

$f_{lintel,d}$ = 1* slab + 1*wall above the lintel = $f_{s,d} * l_s + \rho_{m,d} * t * h$

l_s = 2300 mm *length of slab acting on the lintel*

$\rho_{m,d}$ = (650 kg/m³ / 100)*1,35 = 6,5 kN/m³ * 1,35 = 8,775 kN/ m³

volume weight of masonry

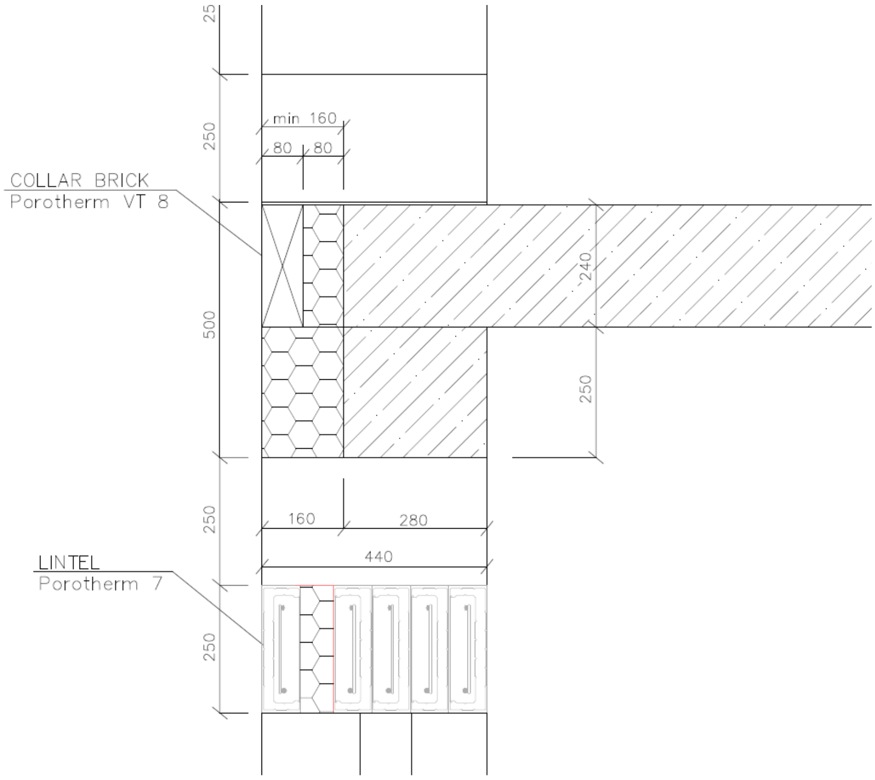
h = 1750 mm *vertical distance between the openings, taken from given dimensions of the openings*

$f_{lintel,d}$ = 12,054 kN/m² * 2,3 m + 8,775 kN/ m³ *0,44 m* 1,75 m = 34,481 kN/m

CONCLUSION: The design value of total load acting on the lintel subjected to check is 34,481 kN/m.

2.6.2. DESIGNED LINTEL PARAMETERS

The used lintels are Porotherm 7, length 2500 mm, bedding 250 mm on each side, clear span 2000 mm. In the given structure there is 5 of them used above the opening.



q_d = 40 kN/m > $f_{lintel,d}$ load-bearing capacity of lintels if 4 of them are used

O.K.

CONCLUSION: The lintels are able to withstand the loading.

3. DESIGN OF SLAB REINFORCEMENT

3.1. CALCULATION IN SCIA ENGINEER SOFTWARE

3.1.1. MODEL

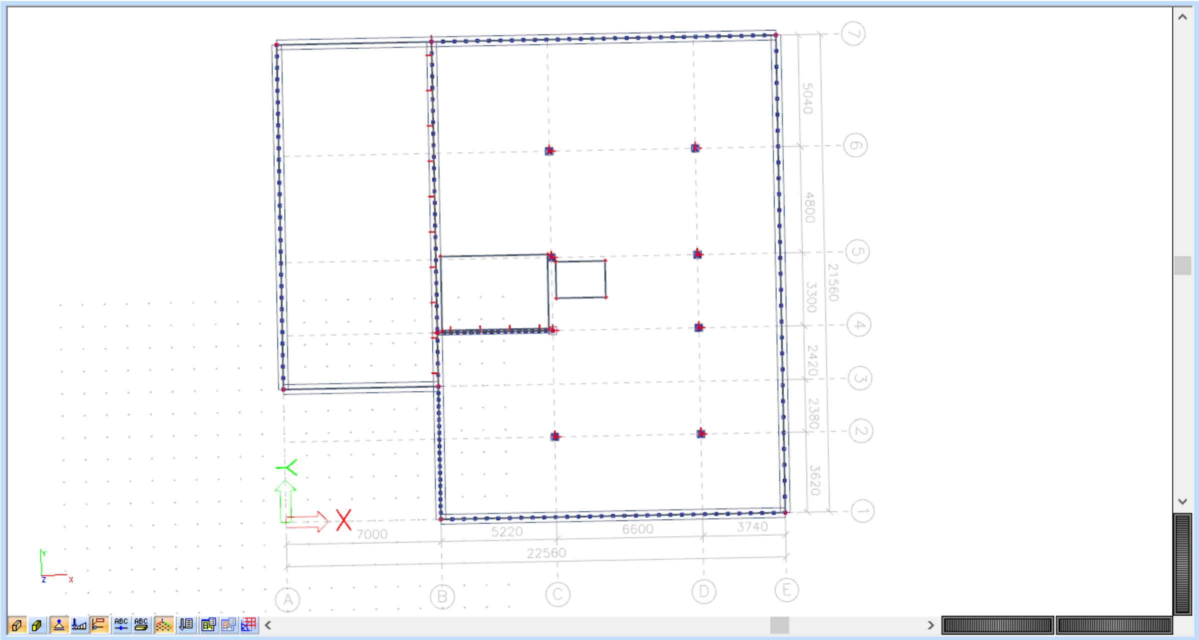


Fig. 1 Ground plan of model with invisible structure structure mesh

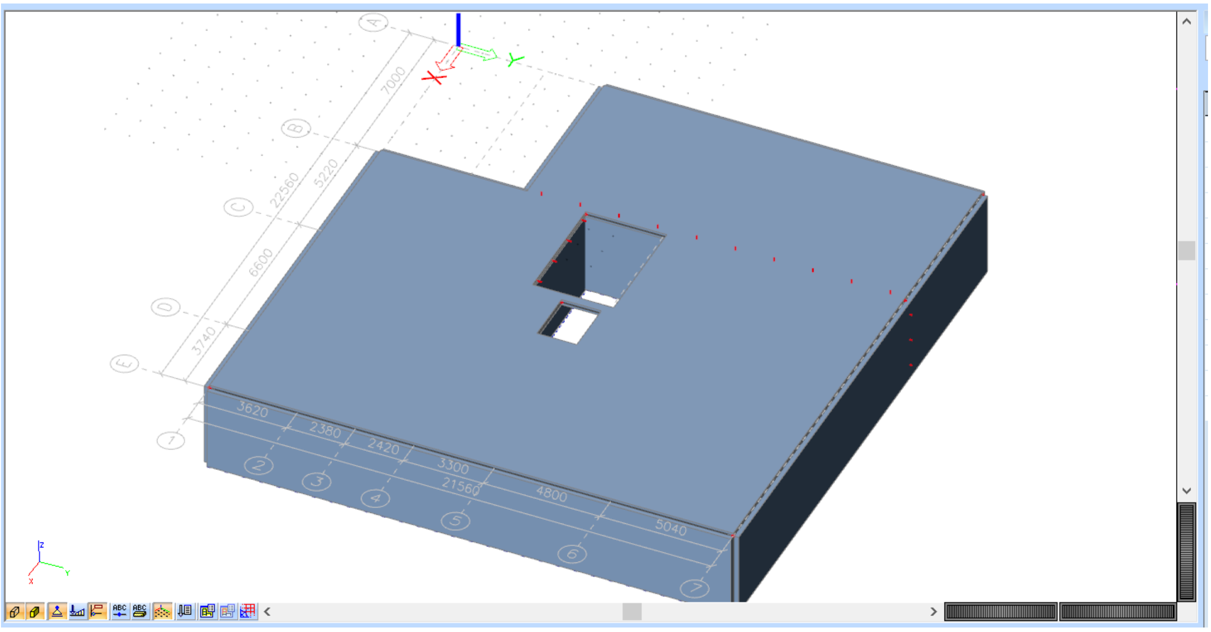


Fig. 2 Bird's-eye view of model

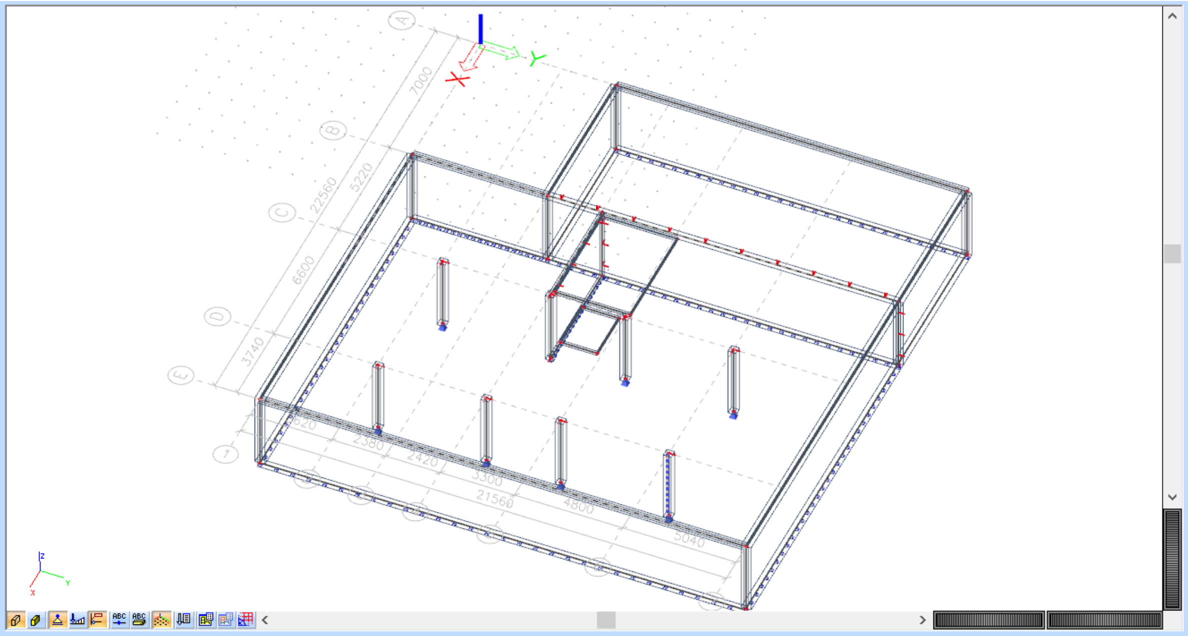


Fig. 3 Bird's-eye view of model with invisible structure mesh

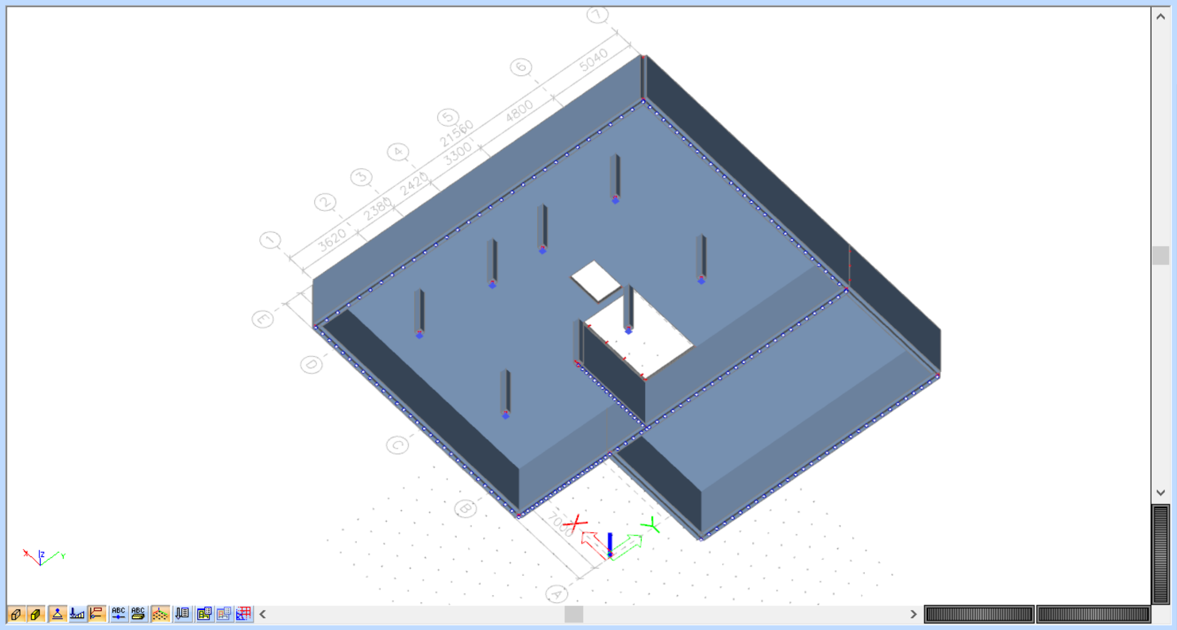


Fig. 4 Worm's-eye view of model

3.1.2. LOADS

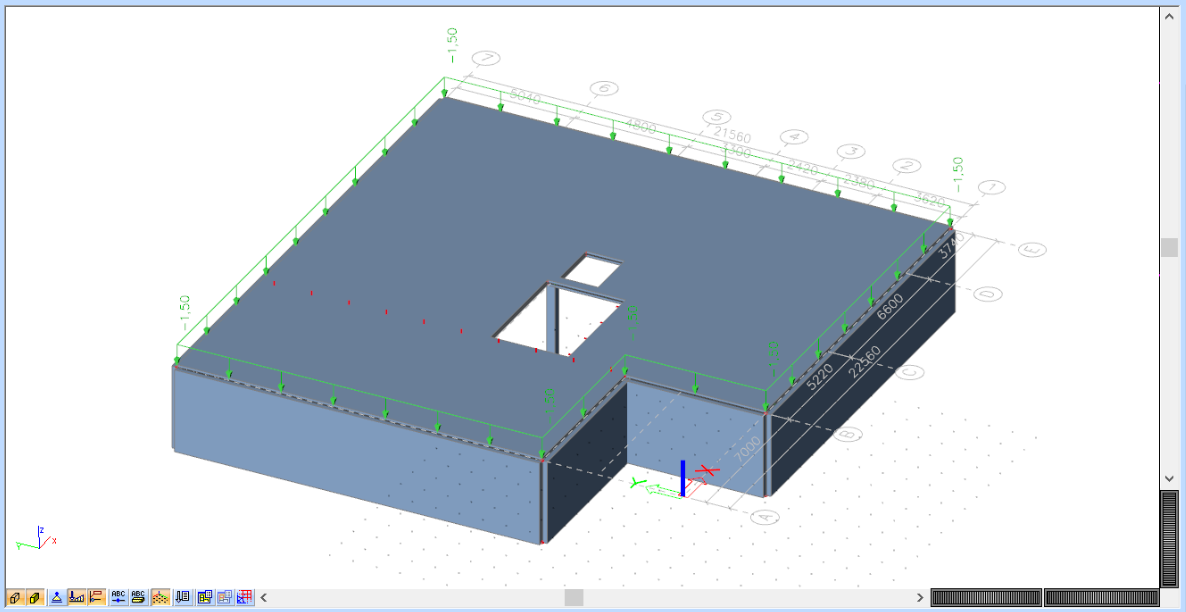


Fig.5 Varible load applied on the model

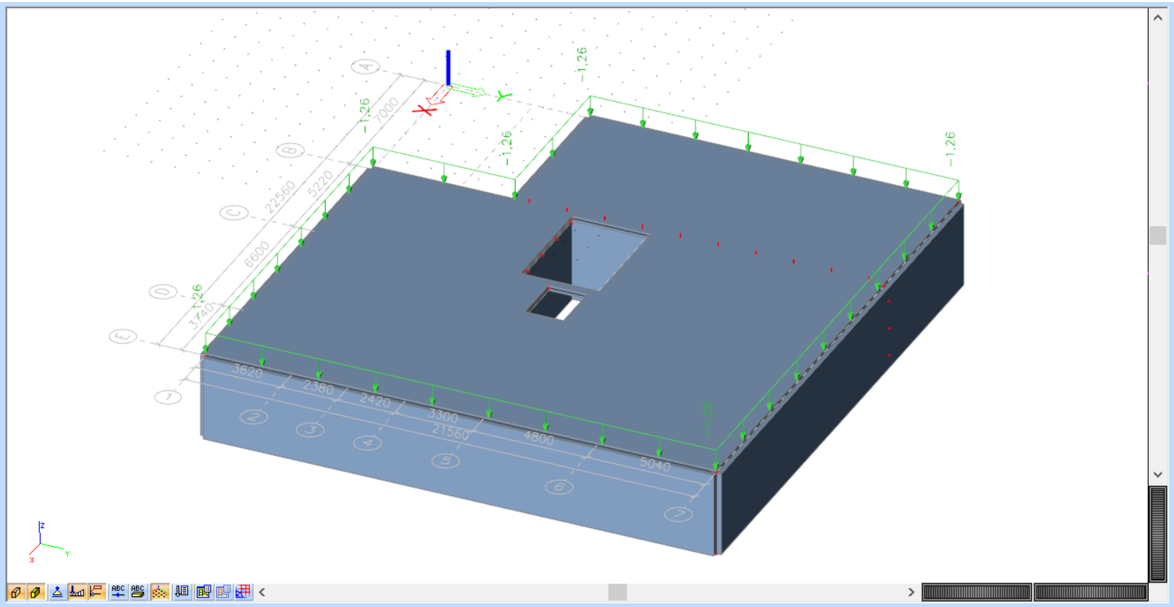


Fig. 6 Permanent load applied on the model

NOTE: SCIA Engineering SW calculates the value of self-weight of the structure by itself.

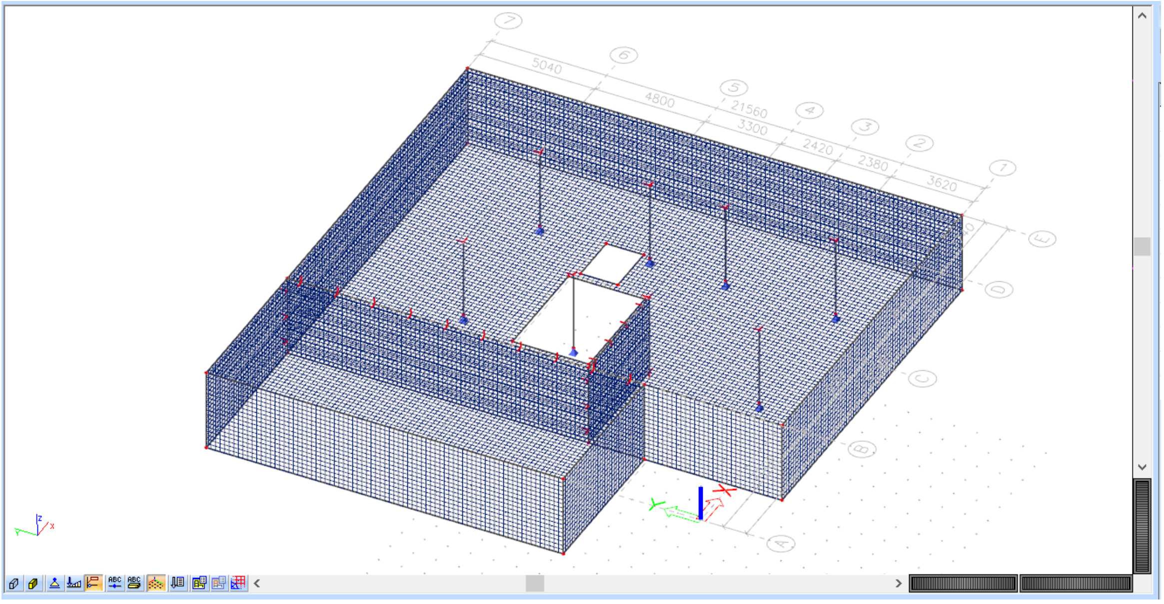


Fig. 7 Mesh dividing the structure into many separate elements, for which the calculation is then performed

3.1.3. RESULTS

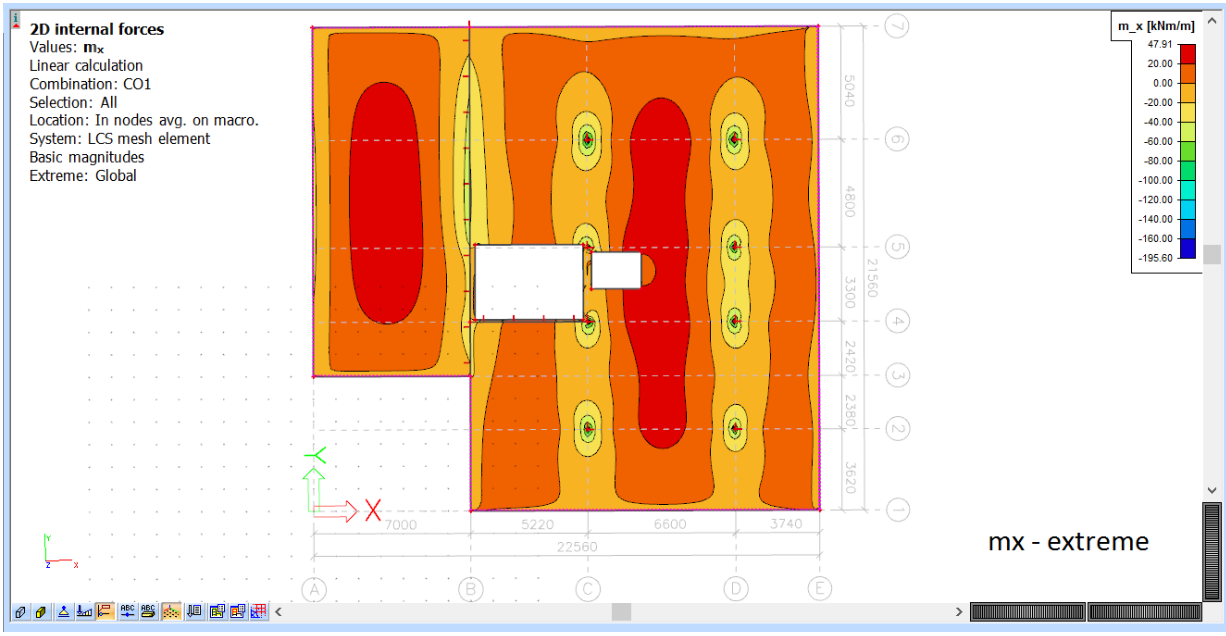


Fig. 8 The envelope of bending moments appearing in the slab structure in the x-direction. These values are then used for further calculation of bending reinforcement.

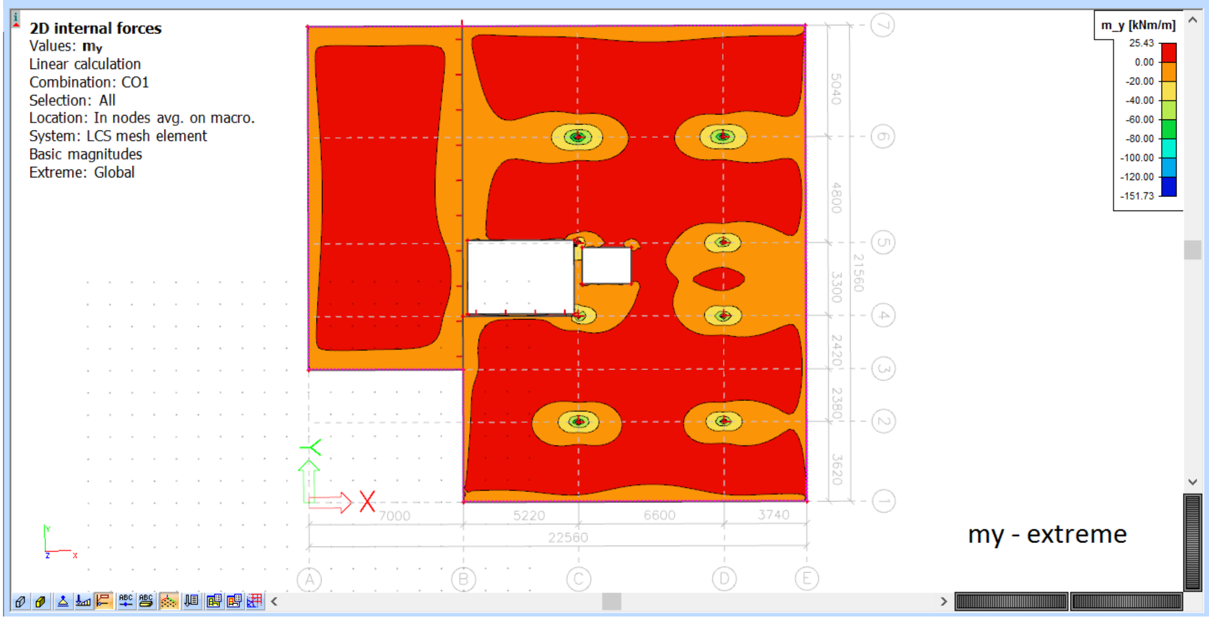


Fig. 9 The envelope of bending moments appearing in the slab structure in the y-direction. These values are then used for further calculation of bending reinforcement.

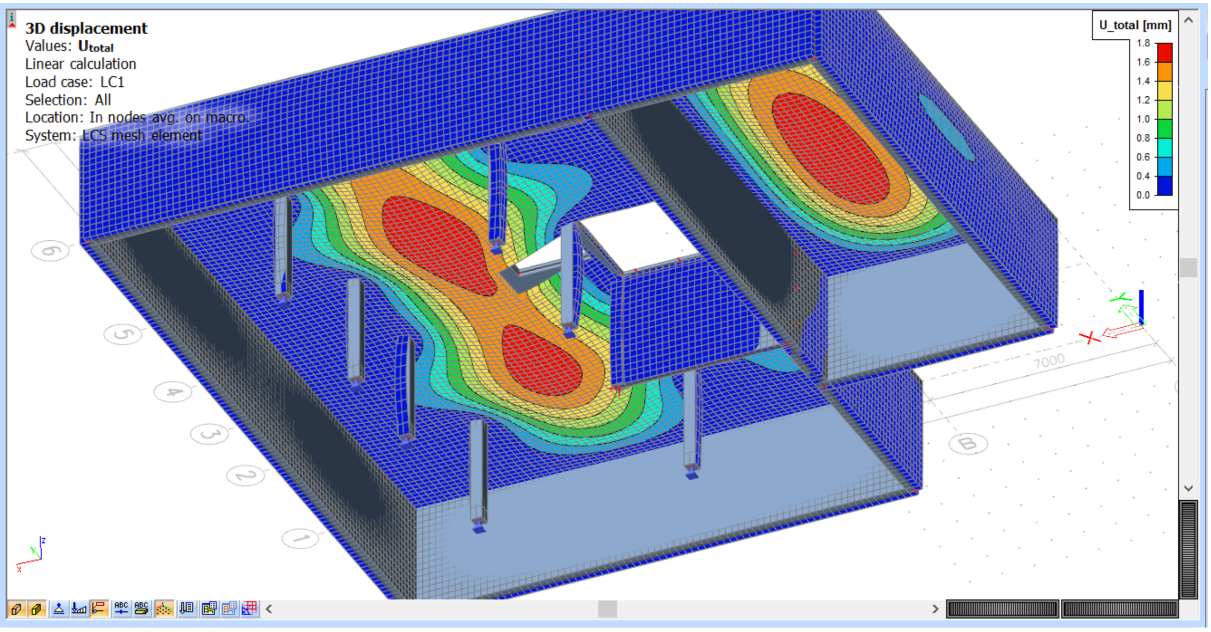


Fig. 10 Scheme of total displacement appearing in the structure. The maximum displacement was calculated with value of 1.8 mm.

NOTE: The results of bending moments are depicted in the attached *Annex file 1. Diagrams of bending moment envelopes.*

3.2. DESIGN OF SLAB'S REINFORCEMENT

3.2.1. CALCULATION OF MINIMUM REINFORCEMENT

Cross-sectional area of bending reinforcement:

$$a_{s,prov} \geq a_{s,rqd} = \frac{m_{Ed}}{z*f_{yd}}$$

$z=0,9*d$ *lever arm of internal forces – estimation*

Brittle failure precaution:

$$a_{s,prov} \geq a_{s,min,1} = \max\,(0,26*(f_{ctm}/f_{yk})*b*d\ ;\ 0,0013*b*d)$$

$f_{ctm} = 2,9$ MPa *C30/37*

$b = 1$ m *width of the slab, in this case equal to 1 m*

Excessive cracking precaution:

$$a_{s,prov} \geq a_{s,min,2} = \frac{k_c*k*f_{ct,eff}*A_{ct}}{\sigma_s}$$

$k_c = 0,4$ and $k = 1,0$ *coefficients describing stress distributions in the cross-section*

$f_{ct,eff} = f_{ctm} = 2,9$ MPa *mean value of tensile strength in time when first cracks might appear*

$\sigma_s = f_{yk} = 500$ MPa *maximum stress inside the reinforcement after acracks appear*

$A_{ct} = 0,5*b*d$ *area of concrete within tensile zone at the first crack*

NOTE: The values of $a_{s,min,2}$ will not be specified in further calculations due to the fact that these values are negligibly small compared to other calculated limits of minimum reinforcement.

3.2.2. CHECK OF THE DESIGN

Height of compressed zone of concrete cross-section:

$$x = \frac{a_{s,prov}*f_{yd}}{0,8*b*f_{cd}}$$

$z = d - 0{,}4*x$ *actual value of lever arm of internal forces*

$m_{Rd} = a_{s,prov} * f_{yd} * z$

$m_{Rd} \geq m_{Ed}$

3.2.3. DETAILING RULES

Relative height of compressed zone:

$\zeta = x/d \leq 0{,}45$

Spacing of rebars:

$s \leq \min(2*h_s\ ;\ 250\ \text{mm})$

$2*h_s = 480$ mm

$s \leq 250$ mm

NOTE: The values calculated in 3.2.1. Calculation of minimum reinforcement, 3.2.2. Check of the design and 3.2.3. Detailing rules are specified in attached *Annex file 2. Table of bending reinforcement design*.

3.3. DETAILED CHECK OF PUNCHING

$$v_{Ed,1} = \frac{\beta * V_{Ed}}{u_1 * d} \leq v_{Rd,c}$$

$$v_{Rd,c} = \max\left[C_{Rd,c} * k * \sqrt[3]{(100 * \rho_l * f_{ck})}\ ;\ 0{,}035 * \sqrt[2]{(k^3 * f_{ck})}\right]$$

$$d = \frac{d_x + d_y}{2}$$

dx = 240 mm – 20 mm – (16 mm/2) = 212 mm *reinf. ϕ 16 á 105 mm*

dy = 240 mm – 20 mm – 16 mm – (14 mm/2) = 197 mm *reinf. ϕ 14 á 95 mm*

$$d = \frac{212 + 197}{2} = 204{,}5 \text{ mm}$$

$\rho_l = \sqrt[2]{(\rho_{lx} + \rho_{ly})} \leq 0{,}02$

$\rho_{lx} = \frac{a_{s,x}}{1000 * d_x}$, $\rho_{ly} = \frac{a_{s,y}}{1000 * d_y}$

$a_{s,x} = 1915\ mm^2$ *reinf. ϕ 16 á 105 mm*

$a_{s,y} = 1620\ mm^2$ *reinf. ϕ 14 á 95 mm*

$\rho_{lx} = \frac{1915}{1000*212} = 0{,}00903$

$\rho_{ly} = \frac{1620}{1000*197} = 0{,}00822$

$\rho_l = \sqrt[2]{(0{,}00903 + 0{,}00822)} = 0{,}00861 \leq 0{,}02$

$C_{Rd,c} = 0{,}12$ *reduction factor*

$k = 1+\sqrt{\frac{200}{d}} \leq 2{,}0$ *effect of depth*

$k = 1+\sqrt{\frac{200}{204{,}5}} = 1{,}99 < 2{,}0$

$C_{Rd,c} *k* \sqrt[3]{(100 * \rho_l * f_{ck})} = 0{,}12*1{,}99*\sqrt[3]{(100 * 0{,}00861 * 30)} = 0{,}706\ MPa$

$0{,}035* \sqrt[2]{(k^3 * f_{ck})} = 0{,}035 * \sqrt[2]{(1{,}99^3 * 30)} = 0{,}538\ MPa$

$\max [C_{Rd,c} *k* \sqrt[3]{(100 * \rho_l * f_{ck})} ; 0{,}035* \sqrt[2]{(k^3 * f_{ck})}] = 0{,}706\ MPa = v_{Rd,c}$

$v_{Ed,1} = \frac{1{,}15*349{,}89*10^3}{3901{,}77*204{,}5} = 0{,}5\ MPa$

$v_{Rd,c} = 0{,}706\ MPa > v_{Ed,1} = 0{,}5\ MPa$ O.K.

CONCLUSION: The detailed check of punching confirmed that no punching reinforcement design is needed.

NOTE: Some value used in this calculation are taken from *2.4. Preliminary check of punching of a column.*