CZECH TECHNICAL UNIVERSITY IN PRAGUE

FACULTY OF CIVIL ENGINEERING

DEPARTMENT OF CONCRETE AND MASONRY STRUCTURES

BACHELOR THESIS

Design of load-bearing structure of an office building with large slab opening

TECHNICAL REPORT

prepared by **Madina Zharas**

supervisor **Ing. Petr Bílý, Ph.D.**

2018-2019

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ZADÁNÍ BAKALÁŘSKÉ PRÁCE

I. OSOBNÍ A STUDIJNÍ ÚDAJE

Příjmení: Zharas Jméno: Madina Osobní číslo: 424557

Zadávající katedra: Katedra betonových a zděných konstrukcí

Studijní program: Civil Engineering

Studijní obor: Building Structures

II. ÚDAJE K BAKALÁŘSKÉ PRÁCI

Název bakalářské práce: Návrh nosné konstrukce administrativní budovy s velkým otvorem ve stropní desce

Název bakalářské práce anglicky: Design of load-bearing structure of an office building with large slab opening

Pokyny pro vypracování:
- Design the principles of the load-bearing system - layout of elements
- Preliminary design of the main load-bearing elements (dimensions of walls, columns, slabs, beams)
- Detailed design of reinforcement of the slab with the large opening
- Detailed design of reinforcement of at least one of the columns
- Structural drawings and reinforcement drawings of selected elements of the structure
- Technical report

Seznam doporučené literatury:
- Textbooks of basic courses of concrete and masonry structures
- Manuals for Scia program - subject Computer aided structural design
- Relevant standards

Jméno vedoucího bakalářské práce: Ing. Petr Bílý, Ph.D.

Datum zadání bakalářské práce: 4. 10. 2018 Termín odevzdání bakalářské práce: 13.1.2019

Údaj uveďte v souladu s datem v časovém plánu příslušného ak. roku

Podpis vedoucího práce Podpis vedoucího katedry

III. PŘEVZETÍ ZADÁNÍ

Beru na vědomí, že jsem povinen vypracovat bakalářskou práci samostatně, bez cizí pomoci, s výjimkou poskytnutých konzultací. Seznam použité literatury, jiných pramenů a jmen konzultantů je nutné uvést v bakalářské práci a při citování postupovat v souladu s metodickou příručkou ČVUT „Jak psát vysokoškolské závěrečné práce" a metodickým pokynem ČVUT „O dodržování etických principů při přípravě vysokoškolských závěrečných prací".

4.10.18

Datum převzetí zadání Podpis studenta(ky)

Acknowledgement

I would like to express sincere thanks to my supervisor Ing. Petr Bílý, Ph.D. for his constant guidance, encouragement, patience and for providing me with advices and ideas for this bachelor thesis. I am also grateful to my family, especially my mom, who gave me a chance to study in Czech Technical University, for their endless support and love. I am also thankful to my partner who supported me throughout my project.

Statement

I hereby declare that I am the original author of this project and I have worked on this bachelor thesis on my own, only with the guidance of my supervisor Ing. Petr Bílý, Ph.D. I confirm that I have not used any sources other than those listed as references. I further declare that this thesis has not been granted to any other institution and has not been presented and published in order to achieve a degree.

In Prague 13th January 2019

Madina Zharas

Abstract

This Bachelor Thesis is a basic structural design of INTOZA office building. All structural and technical design solutions of the main load-bearing structures, building envelope and calculations were made according to Czech and European standards and norms. Detailed calculations of internal forces in each load-bearing element are provided with the help of the FEM (finite element method) software, specifically SCIA Engineer software. Calculation of bending moments in floor slab was provided also manually through DDM (direct design method) to compare and check the results taken from SCIA Engineer. In addition, design of staircase was provided with the bending moments calculated in SCIA Engineer.

Bachelor Thesis is composed of: Concrete part (design of load-bearing elements, calculation of internal forces, formwork drawing, reinforcement drawings of the slab, column, wall and staircase), Buildings Structures part (compositions of the structures, drawings of ground and typical floor plans, sections of the building, typical details of attic, window frame and staircase) and Foundation part (preliminary design of the footing dimensions of each vertical load-bearing element, foundation plan).

Keywords

concrete, load-bearing structure, flat slab, slab opening, reinforced concrete columns, reinforced concrete walls, reinforced concrete pillar, finite element method, reinforcement.

0. GENERAL INFORMATION

Independent civil engineering and design office ATOS-6 and Radim Václavík originally designed an office building in Ostrava, Czech Republic for construction company INTOZA in June 2011.

The building is designed as not only the headquarters of a company, but also for organizing seminars, training and promoting existing and new technologies in the field of energy savings.

The house is conceived in the spirit of the company's philosophy of energy savings, as a model of energetically passive construction. The basic form of the building is the result of a reasonable optimization of the individual basic requirements. Individual elements of the building are selected for optimal price to performance ratio.

Figure 1. INTOZA passive office building

Figure 2. Opening in the slab

Structural design of the building was made for study purpose at Faculty of Civil Engineering, Czech Technical University as a Bachelor Thesis during the study period 1.10.2018 – 13.1.2019. It was created with advanced design procedures of building construction according to the Czech and European norms and standards. This building is located in Karlovy Vary, Czech Republic.

1. BASIC INFORMATION

It is an office building with 5 upper ground floors. The building has rectangular shape with the length of 28.54m and width of 16.95m. Developed area of the building is 483.75m^2 and floor area is 455.04m^2. Clear height of one floor is 3m. Construction height is 3.38m. Total height of the building is 17.7m.

Ground floor consists of the main entrance, corridor, communication area, 6 office rooms, 1 kitchen, 1 printing room, 1 janitor room, 1 technical room, 2 separate bathrooms. The typical floors contain the corridor with an opening in the slab with a diameter 4m, communication area, 6 office rooms, 1 kitchen, 1 printing room, 1 janitor room, 1 technical room, 2 separate bathrooms.

2. STRUCTURAL SYSTEM

The structural system of the building is mixed, column and wall system.

Typical axial distance is 5m and the longest axial distance is 7.4m.

Vertical load bearing structures are reinforced concrete columns with the dimensions of 400x400mm, reinforced concrete pillar with the dimensions of 250x3000mm, reinforced concrete perimeter walls and reinforced concrete walls of communication areas, staircase and elevator, with the thickness of 200mm.

Horizontal load bearing structure is the two-way monolithic concrete flat slab with the thickness of 275 mm.

3. MATERIALS

- **Concrete**

Reinforced concrete slabs:

Concrete class C30/37 – Exposure class XC1 – Structural class S4 – dmax=22mm – Cl<2%

Reinforced concrete columns:

Concrete class C30/37 – Exposure class XC1 – Structural class S3 – dmax=22mm – Cl<2%

Reinforced concrete pillar:

Concrete class C30/37 – Exposure class XC1 – Structural class S3 – dmax=22mm – Cl<2%

Reinforced concrete perimeter walls:

Concrete class C30/37 – Exposure class XC2 – Structural class S4 – dmax=22mm – Cl<2%

Reinforced concrete walls (communication areas):

Concrete class C30/37 – Exposure class XC1 – Structural class S4 – dmax=22mm – Cl<2%

Reinforced concrete foundations:

Concrete class C25/30 – Exposure class XC2 – Structural class S3 – dmax=22mm – Cl<2%

- **Steel**

Reinforcing bars – B500B

- **Plasterboard**

Partition walls:

Gypsum board RIGIPS RB

4. PRELIMINARY DESIGN

4.1. Slab

The main slab is designed as a two-way monolithic flat slab supported by columns (without column heads) and walls. On the slab at typical floors, there is a large circular opening with a diameter of 4000mm. First, we have to estimate the effective depth of the slab.

Empirical estimation: $ds = \left(\frac{1}{35} \sim \frac{1}{30}\right) \times l$,

where *l* is the longest clear span. In my case, *l* = 7150mm.

$$ds = \left(\frac{1}{35} \sim \frac{1}{30}\right) \times 7150mm = 204.3 \sim 233.3mm$$

Effective depth: $ds \geq \frac{l}{Kc1 \times Kc2 \times Kc3 \times \lambda d,tab}$, where:

k_{c1} = 1 – coefficient of cross-section (rectangular cross-section)

k_{c2} = 0.98 – coefficient of span (for $l \geq 7m,\ kc2 = \frac{7}{l}$)

k_{c3} = 1.25 – coefficient of stress in tensile reinforcement (assumed *kc3 = 1.1-1.3*)

$\lambda d, tab$ = 24 – design span to depth ratio obtained from the table below (in case of the slab, we use the reinforcement ratio 0.5%)

$\lambda_{d,tab}$ design span to depth ratio								
Structural system	Reinforcement ratio	Strength class						
		C12/15	C16/20	C20/25	C25/30	C30/37	C40/50	C50/60
Simply supported beam, one-way or two-way spanning simply supported slab	0,5%	16,6	15,8	17,0	18,5	20,5	25,8	32,0
	1,5%	12,2	12,6	13,0	13,5	14,0	15,0	16,0
Cantilever	0,5%	5,8	6,3	6,8	7,4	8,0	10,3	12,8
	1,5%	4,9	5,0	5,2	5,4	5,6	6,0	6,4
Slab supported on columns without beams (flat slab, locally supported slab)	0,5%	17,5	19,0	20,4	22,2	24,0	30,9	38,4
	1,5%	14,6	15,1	15,6	16,2	16,8	18,0	19,2
End span of continuous beam or one-way continuous slab or two-way spanning slab continuous over one long side	0,5%	19,0	20,5	22,1	24,1	26,0	33,5	41,5
	1,5%	15,9	16,4	16,9	17,6	18,0	19,5	20,8
Interior span of continuous beam or one-way or two-way spanning continuous slab	0,5%	21,9	23,7	25,5	27,8	30,8	38,6	48,0
	1,5%	18,3	18,9	19,5	20,3	21,0	22,5	24,0

Effective depth: $ds \geq \frac{7150}{1 \times 0.98 \times 1.25 \times 24} \geq 243.2mm$, so I assumed as ds = 250mm.

Thickness of the slab: $hs = ds + c + \frac{\emptyset}{2}$,

where $\emptyset = 10mm$ is assumed diameter of steel bars and c is concrete cover depth.

Cover depth depends on concrete class, exposure class, design life service, etc.

$$c = c_{min} + \Delta c_{dev}$$

$\Delta c_{dev} = 10mm$ (quality control on construction site necessary for safety)

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

where $c_{min,b} = 10mm$ (depth needed for good mechanical bond between steel and concrete, equal to assumed diameter of steel bars)

$c_{min,dur} = 10mm$ (depth needed for good resistance to different environmental effects) is obtained from the tables below:

Values of $c_{min,dur}$ [mm]							
Structural class	Exposure class related to environmental conditions						
	X0	XC1	XC2/XC3	XC4	XD1/XS1	XD2/XS2	XD3/XS3
S1	10	10	10	15	20	25	30
S2	10	10	15	20	25	30	35
S3	10	10	20	25	30	35	40
S4 (for 50 years)	10	15	25	30	35	40	45
S5	15	20	30	35	40	45	50
S6	20	25	35	40	45	50	55

Structural class								
Criterion	Exposure class related to environmental conditions							
	X0	XC1	XC2	XC3	XC4	XD1/XS1	XD2/XS2	XD3/XS3
Working life 80 years	increase class by 1							
Working life 100 years	increase class by 2							
Concrete class	decrease class by 1 if concrete class is at least:							
	C20/25	C25/30	C30/37	C35/45	C40/50	C40/50	C40/50	C45/55
Member with slab geometry	decrease class by 1							
Special quality control of concrete	decrease class by 1							

$c_{min} = max.(10mm; 10mm; 10mm) = 10mm$

$c = c_{min} + \Delta c_{dev} = 10mm + 10mm = 20mm$

$$hs = ds + c + \frac{\emptyset}{2} = 250 + 20 + \frac{10}{2} = 275mm$$

DESIGN: Thickness of the main slab is equal to $\boldsymbol{h_s}$ **= 275mm.**

Slab is loaded by uniform load, which has to be calculated, see tables below.

Fk – characteristic load

Fd – design load

γ_F – safety factor

SLAB LOAD

Type	Name	Fk [kN/m^2]	γ_F	Fd [kN/m^2]
Permanent (Dead load)	-Surface layer (carpet/ceramic) -Glue layer -Concrete (leveling layer) -Separation foil -Acoustic insulation (EPS/XPS) -Reinforced concrete -Plaster -Partitions	0.2 0.01 1.25 0.01 0.05 0.275*25=6.875 0.06 0.11	1.35 1.35 1.35 1.35 1.35 1.35 1.35 1.35	0.27 0.0135 1.6875 0.0135 0.0675 9.28125 0.081 0.1485
Variable (Live load)		2	1.5	3
Total		$\sum$ = 10.57 kN/m^2		$\sum$ = 14.563 kN/m^2

Snow load:

$$s_k = \mu \times c_e \times c_t \times s$$

μ = 0.8 (for flat roof)

c_e= 1 (for normal topology)

c_t= 1 (for normal conditions)

s = 1.5 kN/m^2 (for III. zone, Karlovy Vary)

$s_k = 0.8 \times 1 \times 1 \times 1.5 = 1.2\ kN/m^2$

ROOF LOAD

Type	Name	Fk [kN/m^2]	γ_F	Fd [kN/m^2]
Permanent (Dead load)	-Gravel -Waterproofing (asphalt) -Waterproofing (asphalt) -Thermal insulation (XPS) -Thermal insulation (XPS) -Reinforced concrete -Gypsum board	0.84 0.025 0.025 0.4 0.4 0.275*25=6.875 0.4	1.35 1.35 1.35 1.35 1.35 1.35 1.35	1.134 0.03375 0.03375 0.54 0.54 9.28125 0.54
Variable (Live load)	-Snow	1.2	1.5	1.8
Total		$\sum$ = 10.17 kN/m^2		$\sum$ = 13.9 kN/m^2

4.2. Wall

To get the sufficient thickness of the wall, we have to calculate the load acting on wall per 1m. I assumed thickness of the wall as 200mm.

$$N_{Ed,w} = \left(4 \times F_{d,slab} \times l\right) + \left(1 \times F_{d,roof} \times l\right) + (5 \times \rho_c \times t_w \times H \times \gamma_F)$$

$\rho_c = 25kN/m^3$ (unit weight of concrete)

l = 3700mm ($\frac{1}{2}$ of the longest span from the wall)

H = 3000mm (clear height of one floor)

$N_{Ed,w} = (4 \times 14.563 \times 3.7) + (1 \times 13.9 \times 3.7) + (5 \times 25 \times 0.2 \times 3 \times 1.35) = 368.21kN/m$

From the below equation, we get the cross-sectional area of the wall per 1m.

$$N_{Rd,w} = 0.8 \times A_c \times f_{cd} + A_s \times \sigma_s \geq N_{Ed,w} \quad \rightarrow \quad A_c \geq \frac{N_{Ed,w}}{0.8 \times f_{cd} + \rho_s \times \sigma_s}$$

$\rho_s = 0.5\%$ (reinforcement ratio)

$\sigma_s = 400MPa$ (stress in the reinforced steel)

$f_{cd} = \frac{f_{ck}}{\gamma_c}$, where

$f_{ck} = 30MPa$ (characteristic strength of concrete, see table below)

$\gamma_c = 1.5$ (partial safety factor for concrete)

		Concrete class								
		C 12/15	C 16/20	C 20/25	C 25/30	C 30/37	C 35/45	C 40/50	C 45/55	C 50/60
compressive strength	f_{ck} [MPa]	12	16	20	25	30	35	40	45	50
	f_{cm} [MPa]	20	24	28	33	38	43	48	53	58
tensile strength	f_{ctm} [MPa]	1,6	1,9	2,2	2,6	2,9	3,2	3,5	3,8	4,1
	$f_{ctk\ 0,05}$ [MPa]	1,1	1,3	1,5	1,8	2	2,2	2,5	2,7	2,9
	$f_{ctk\ 0,95}$ [MPa]	2	2,5	2,9	3,3	3,8	4,2	4,6	4,9	5,3
E_{cm} [GPa]		26	27,5	29	30,5	32	33,5	35	36	37
limiting strain	ε_{cu} . 10^{-4} ‰ [1/]	-3,6	-3,5	-3,4	-3,3	-3,2	-3,1	-3,0	-2,9	-2,8
	ε_{cu} . 10^{-4} ‰ [2/]	-3,5	-3,5	-3,5	-3,5	-3,5	-3,5	-3,5	-3,5	-3,5

[1/] for calculation of resistance
[2/] for calculation of effects of load

$f_{cd} = \frac{30}{1.5} = 20MPa$ (design compressive strength of concrete)

$$A_c \geq \frac{368.21 \times 10^3}{0.8 \times 20 + 0.005 \times 400} \geq 20456.1mm^2/m$$

$A_c = 200 \times 1000 = 200000mm^2/m$

DESIGN: Thickness of the wall is equal to $\boldsymbol{t_w}$**= 200mm.**

4.3. Column

To design the satisfactory dimensions of the columns, we should first calculate the point load acting on a most critical column. Estimated dimensions are b_c = 400mm and h_c= 400m.

$N_{Ed,c} = (4 \times F_{d,slab} \times A) + (1 \times F_{d,roof} \times A) + (\rho_c \times b_c \times h_c \times H \times \gamma_F)$

$A = \left(\frac{7.4}{2} + \frac{5}{2}\right) \times \left(\frac{6.085}{2} + \frac{5.76}{2}\right) = 36.72m^2$ (load area of the column)

$\rho_s = 2\%$ (reinforcement ratio)

$N_{Ed,c} = (4 \times 14.563 \times 36.72) + (1 \times 13.9 \times 36.72) + (25 \times 0.4 \times 0.4 \times 3 \times 1.35)$

$N_{Ed,c} =$ 2665.6 kN

$$N_{Rd,c} = 0.8 \times A_c \times f_{cd} + A_s \times \sigma_s \geq N_{Ed,c} \quad \rightarrow \quad A_c \geq \frac{N_{Ed,c}}{0.8 \times f_{cd} + \rho_s \times \sigma_s}$$

$$A_c \geq \frac{2665.6 \times 10^3}{0.8 \times 20 + 0.02 \times 400} \geq 111067mm^2$$

DESIGN: Dimensions of one column are **400x400mm** → $A_c = 400 \times 400 = 160000mm^2$

4.4. Pillar

It is necessary to design pillar close to slab opening to provide sufficient support of the slab around opening. Same procedure is used as for column. Estimated dimensions are $b_p = 250$mm and h_p= 3000mm.

$$N_{Ed,p} = \left(4 \times F_{d,slab} \times A\right) + \left(1 \times F_{d,roof} \times A\right) + (\rho_c \times b_p \times h_p \times H \times \gamma_F)$$

$A = \left(\frac{7.375}{2} + \frac{6.175}{2}\right) \times \left(\frac{4.665}{2} + \frac{6.085}{2}\right) = 36.42m^2$ (Load area of the pillar)

$$N_{Ed,p} = (4 \times 14.563 \times 36.42) + (1 \times 13.9 \times 36.42) + (25 \times 0.25 \times 3 \times 3 \times 1.35)$$

$$N_{Ed,p} = \text{ 2703.7kN}$$

$$N_{Rd,p} = 0.8 \times A_c \times f_{cd} + A_s \times \sigma_s \geq N_{Ed,c} \quad \rightarrow \quad A_c \geq \frac{N_{Ed,p}}{0.8 \times f_{cd} + \rho_s \times \sigma_s}$$

$$A_c \geq \frac{2703.7 \times 10^3}{0.8 \times 20 + 0.02 \times 400} \geq 112654.2mm^2$$

DESIGN: Dimensions of the pillar are **250x3000mm** → $A_c = 200 \times 3000 = 750000mm^2$

5. CALCULATION OF INTERNAL FORCES

To check the each structure and design the reinforcement any FEM (finite element method) software can be used. To get detailed calculation of bending moments, shear and normal forces I modeled the first floor of the construction in SCIA Engineer software.

First, I modeled the slab with an opening in 3D with respect to all dimensions.

Second, I put all vertical load-bearing elements: columns, pillar and walls with designed dimensions and thicknesses. On the bottom of each structure fixed supports were applied and on the top – custom supports with free end in z(vertical) direction. This was necessary to enable the application of vertical forces from the floors above.

Openings, particularly windows, were not designed on the model, because it would not affect the overall results in the software.

The mesh element size was selected to be the same as the thickness of the slab, i.e. 275mm.

To solve the singularities above the columns and pillar, averaging points were used. The size of the points was selected as 1.5m for columns and 1m for pillar.

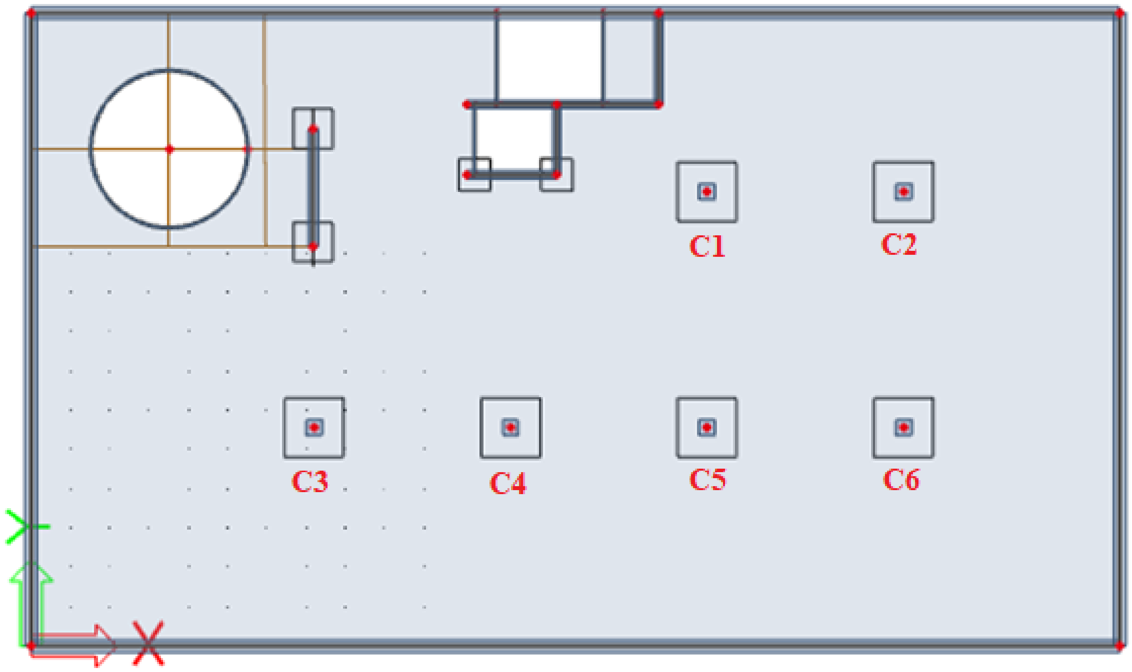


Figure 3. Plan of the structure in SCIA Engineer

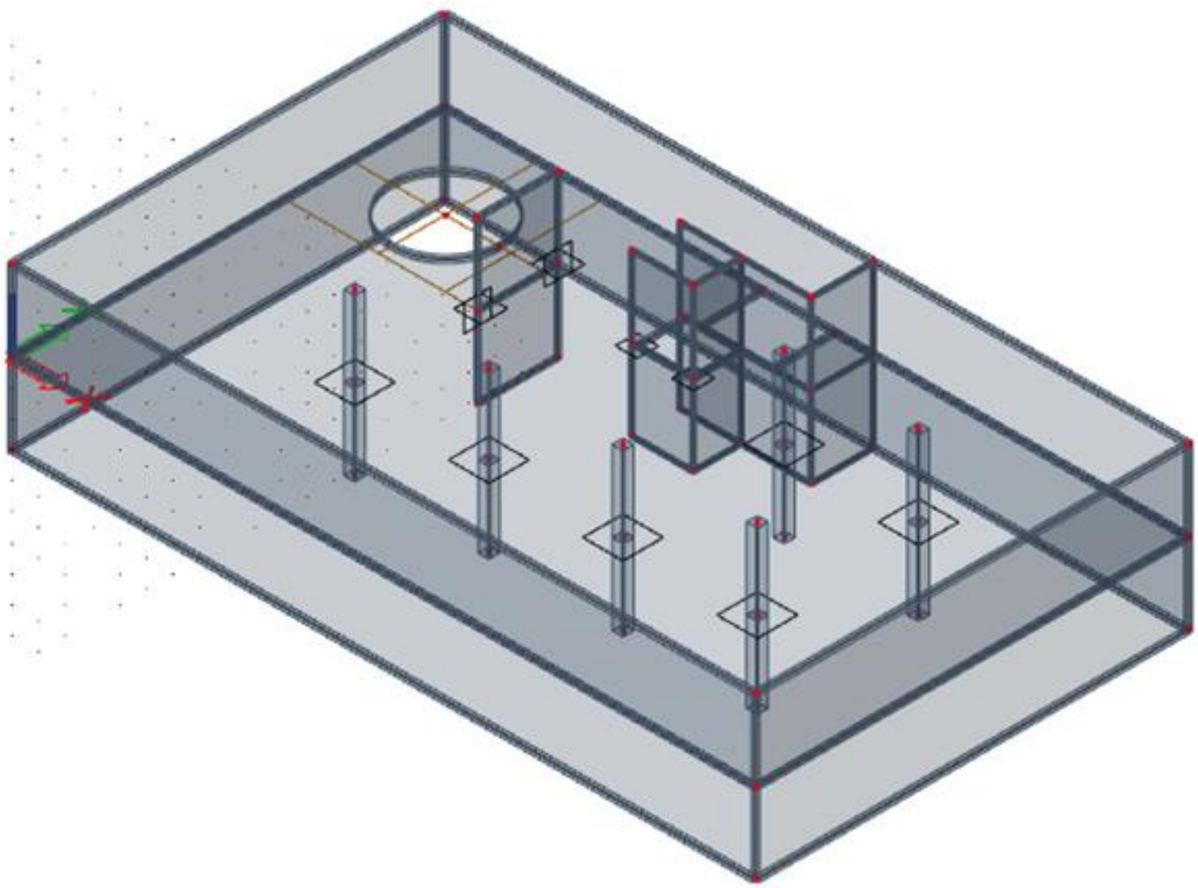


Figure 4. Axonometric view of the structure

After modeling the geometry of the structure, load cases and combinations have to be added. Four load cases were considered:

Load Group	Load Case	Type	[kN/m^2]
LG1 - Permanent	LC 1	Self-weight	-
	LC 2	Other dead load	1.58
	LC 3	Partitions	0.075
LG2 – Variable	LC4	Live load	2

Self-weight is generated automatically by software.

Partitions load:

Density of gypsum board RIGIPS RB = 11.2kg/m^2.

Height of partition walls is 3m.

Length of all partition walls is 108.83m.

Area of the building is 483.75m^2.

$$\frac{0.11kN/m^2 \times 3m \times 108.83m}{483.75m^2} = 0.075kN/m^2$$

Manually calculated loads from upper floors are applied to each vertical element:

- *Load on walls*

$$N_{Ed,w} = (3 \times 14.563 \times 3.7) + (1 \times 13.9 \times 3.7) + (3 \times 25 \times 0.2 \times 3 \times 1.35) = 273.83kN/m$$

- *Load on pillar*

$$A = \left(\frac{7.375}{2} + \frac{6.175}{2}\right) \times \left(\frac{4.665}{2} + \frac{6.085}{2}\right) = 36.42m^2$$

$$N_{Ed,p} = (3 \times 14.563 \times 36.42) + (1 \times 13.9 \times 36.42) + (25 \times 0.25 \times 3 \times 3 \times 1.35) = 2173.33\ kN$$

To get the line load acting on pillar I divided *Ned,p* with 3m, length of the pillar.

$$N_{Ed,p} = \frac{2173.33}{3} = 724.4\ kN/m$$

- *Load on columns*

C1: $A = \left(\frac{1.2}{2} + \frac{5}{2}\right) \times \left(\frac{2.2}{2} + \frac{6}{2}\right) = 12.71m^2$

$$N_{Ed,c,1} = (3 \times 14.563 \times 12.71) + (1 \times 13.9 \times 12.71) + (25 \times 0.4 \times 0.4 \times 3 \times 1.35) = 748.2\ kN$$

$C2\text{:}\quad A = \left(\frac{5}{2} + \frac{5.7}{2}\right) \times \left(\frac{4.75}{2} + \frac{6}{2}\right) = 28.8m^2$

$$N_{Ed,c,2} = (3 \times 14.563 \times 28.8) + (1 \times 13.9 \times 28.8) + (25 \times 0.4 \times 0.4 \times 3 \times 1.35)$$
$$= 1674.8kN$$

$C3\text{:}\quad A = \left(\frac{7.4}{2} + \frac{5}{2}\right) \times \left(\frac{6.085}{2} + \frac{5.76}{2}\right) = 36.72m^2$

$$N_{Ed,c,3} = (3 \times 14.563 \times 36.72) + (1 \times 13.9 \times 36.72) + (25 \times 0.4 \times 0.4 \times 3 \times 1.35)$$
$$= 2130.9kN$$

$C4\text{:}\quad A = \left(\frac{5}{2} + \frac{5}{2}\right) \times \left(\frac{6.49}{2} + \frac{5.76}{2}\right) = 30.63m^2$

$$N_{Ed,c,4} = (3 \times 14.563 \times 30.63) + (1 \times 13.9 \times 30.63) + (25 \times 0.4 \times 0.4 \times 3 \times 1.35)$$
$$= 1780.2kN$$

$C5\text{:}\quad A = \left(\frac{5}{2} + \frac{5}{2}\right) \times \left(\frac{6}{2} + \frac{5.76}{2}\right) = 29.4m^2$

$$N_{Ed,c,5} = (3 \times 14.563 \times 29.4) + (1 \times 13.9 \times 29.4) + (25 \times 0.4 \times 0.4 \times 3 \times 1.35)$$
$$= 1709.3kN$$

$C6\text{:}\quad A = \left(\frac{5}{2} + \frac{5.7}{2}\right) \times \left(\frac{6}{2} + \frac{5.76}{2}\right) = 31.46m^2$

$N_{Ed,c,6} = (3 \times 14.563 \times 31.46) + (1 \times 13.9 \times 31.46) + (25 \times 0.4 \times 0.4 \times 3 \times 1.35) =$ $1825.3kN$

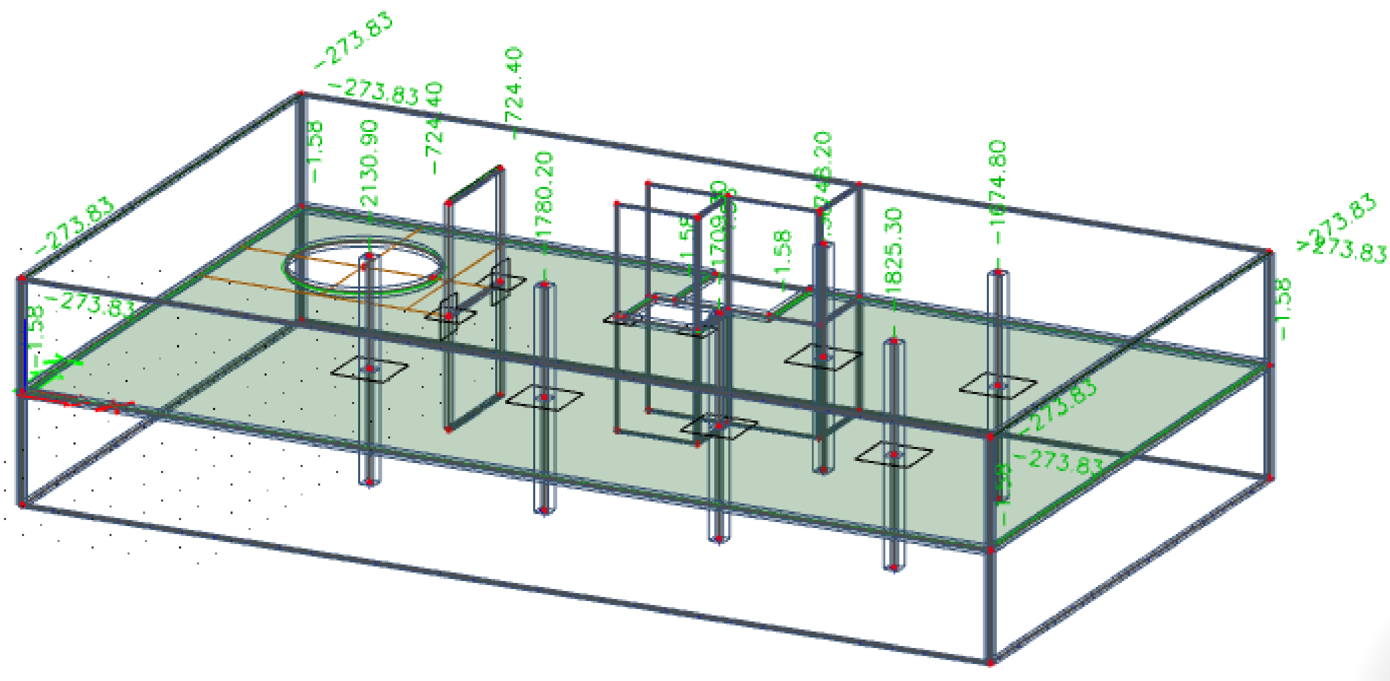


Figure 5. Other dead load with applied vertical loads on vertical structures

In addition, two load combinations were created: ULS (ultimate limit state) and SLS (serviceability limit state). Coefficient for live load in SLS-quasi static load was considered as 0.30.

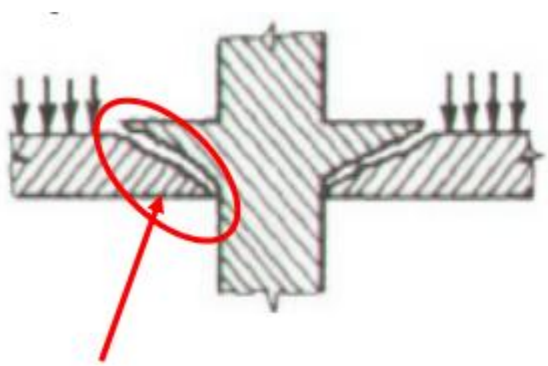
Name	ULS
Description	
Type	Linear - ultimate
Nonlinear combination	
Amplified Sway Moment method	☐ no
Contents of combination	
LC1 - Self weight [-]	1.35
LC2 - Other dead load [-]	1.35
LC3 - Partitions [-]	1.35
LC4 - Live load [-]	1.50

Name	SLS
Description	
Type	Envelope - serviceability
Nonlinear combination	
Contents of combination	
LC1 - Self weight [-]	1.00
LC2 - Other dead load [-]	1.00
LC3 - Partitions [-]	1.00
LC4 - Live load [-]	0.30

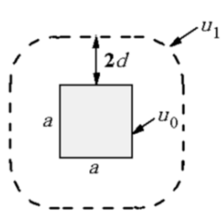
6. DESIGN OF REINFORCEMENT

6.1. Slab

Before designing the main reinforcement of the slab, we have to check if any punching reinforcement is needed. Punching reinforcement is necessary to avoid shear failure around the column.



6.1.1. Preliminary check of punching of the column C3 (the highest load)



$$u_0 = 4a = 4 \times 400 = 1600mm$$
$$u_1 = 4a + 2\pi \times 2d = 1600 + 2\pi \times 2 \times 250 = 4741.6mm$$

We have to check if shear resistance of concrete is satisfactory:

- ***Maximum punching shear resistance***

$$\nu_{Ed,0} = \frac{\beta \times Ved}{u_0 \times d} \leq \nu_{Rd,max} = 0.4 \times \upsilon \times fcd$$

$\upsilon = 0.6 \times \left(1 - \frac{fck}{250}\right) = 0.6 \times \left(1 - \frac{30}{250}\right) = 0.528$ (effect of additional stress)

$\beta = 1.15$ (coefficient expressing the position of the column – for inner column)

0.4 – (effect of shear on compressive strength)

$Ved = 14.563kN/m^2 \times 36.72m^2 = 534.75kN$ (load acting from the slab to column area)

$\nu_{Ed,0} = \frac{\beta \times Ved}{u_0 \times d} = \frac{1.15 \times 534750}{1600 \times 250} = 1.54MPa$ (stress in perimeter u_0)

$\nu_{Rd,max} = 0.4 \times 0.528 \times 20 = 4.224MPa$ (maximum punching shear resistance)

$\nu_{Ed,0} = 1.54MPa < \nu_{Rd,max} = 4.224MPa$ ✓

We have to also check if we can anchor the punching reinforcement in concrete adequately:

- ***Maximum resistance with reinforcement***

$$\nu_{Ed,1} = \frac{\beta \times Ved}{u_1 \times d} \leq k_{max} \times \nu_{Rd,c} = k_{max} \times C_{Rd,c} \times k \times \sqrt[3]{(100 \times \rho_l \times fck)}$$

$k_{max} = \frac{1.45+1.7}{2} = 1.575$ (coefficient of maximum resistance, taken from table below)

	effective depth of the slab	**k_{max}**
S T I R R U P S	*d* ≤ 200 mm	1,45
	200 mm ≤ *d* ≤ 700 mm	interpolation
	d ≥ 700 mm	1,70

$C_{Rd,c} = 0.12$ (reduction factor)

$k = 1 + \sqrt{\frac{200}{d}} \leq 2 = 1 + \sqrt{\frac{200}{250}} = 1.9 < 2$ (effect of depth)

$\rho_l = 0.005$ (estimated reinforcement ratio of tensile reinforcement)

$$\nu_{Ed,1} = \frac{\beta \times Ved}{u_1 \times d} = \frac{1.15 \times 534750}{4741.6 \times 250} = 0.52MPa$$

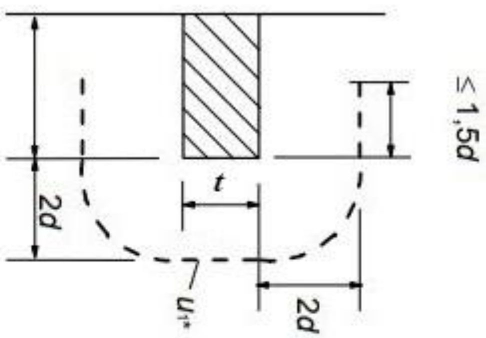
$k_{max} \times C_{Rd,c} \times k \times \sqrt[3]{(100 \times \rho_l \times fck)} = 1.575 \times 0.12 \times 1.9 \times \sqrt[3]{100 \times 0.005 \times 30} = 0.9MPa$

$\nu_{Ed,1} = 0.52MPa < 0.9MPa$ ✓

Both conditions are satisfying, which means that the design of punching reinforcement will be possible in case it is needed.

6.1.2. Preliminary check of punching of the pillar

Edge of the pillar must be checked too, because the load is concentrated in the ends of the pillar, so it behaves as a column.



$$u_0 = \mathrm{t} + 3\mathrm{d} = 250 + 3 \times 250 = 1000\mathrm{mm}$$

$$u_1 = u_0 + \frac{2\pi \times 2d}{2} = 1000 + \frac{2\pi \times 2 \times 250}{2} = 2570.8mm$$

- ***Maximum punching shear resistance***

$$v_{Ed,0} = \frac{\beta \times Ved}{u_0 \times d} \leq v_{Rd,max} = 0.4 \times \upsilon \times fcd$$

$$A = \left(\frac{7.375}{2} + \frac{6}{2}\right) \times \left(\frac{4.385}{2} + 0.375\right) = 17.4m^2$$

$Ved = 14.563kN/m^2 \times 17.4m^2 = 253.4kN$ (load acting from the slab to the end of the pillar)

$v_{Ed,0} = \frac{\beta \times Ved}{u_0 \times d} = \frac{1.15 \times 253400}{1000 \times 250} = 1.17MPa$ (stress in perimeter u_0)

$v_{Rd,max} = 0.4 \times 0.528 \times 20 = 4.224MPa$ (maximum punching shear resistance)

$v_{Ed,0} = 1.17MPa < v_{Rd,max} = 4.224MPa$ ✓

- ***Maximum resistance with reinforcement***

$$v_{Ed,1} = \frac{\beta \times Ved}{u_1 \times d} \leq k_{max} \times v_{Rd,c} = k_{max} \times C_{Rd,c} \times k \times \sqrt[3]{(100 \times \rho_l \times fck)}$$

$$v_{Ed,1} = \frac{\beta \times Ved}{u_1 \times d} = \frac{1.15 \times 253400}{2570.8 \times 250} = 0.45MPa$$

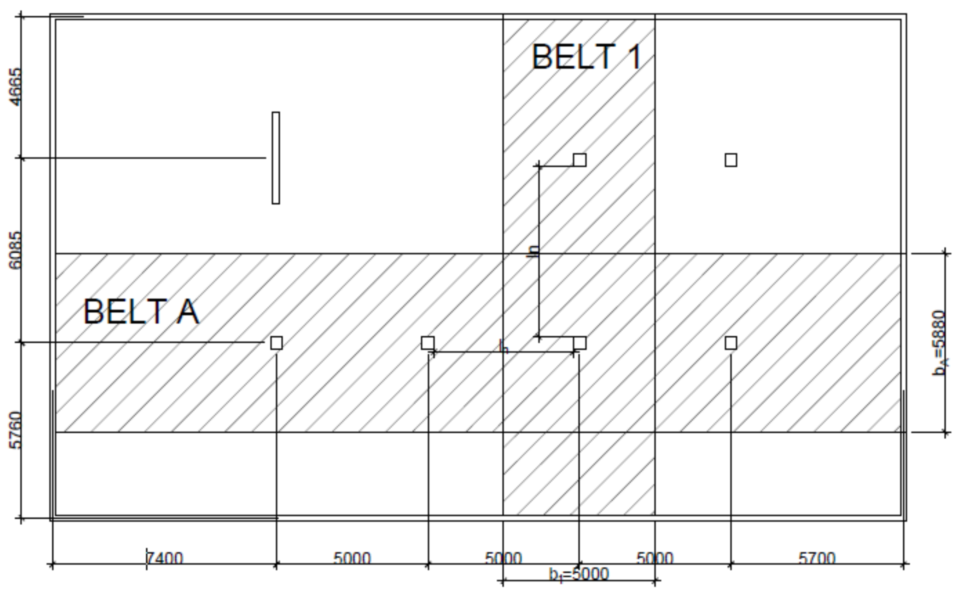
$$k_{max} \times C_{Rd,c} \times k \times \sqrt[3]{(100 \times \rho_l \times fck)} = 1.575 \times 0.12 \times 1.9 \times \sqrt[3]{100 \times 0.005 \times 30} = 0.9MPa$$

$v_{Ed,1} = 0.45MPa < 0.9MPa$ ✓

Both conditions are satisfying, which means that the design of punching reinforcement will be possible in case it is needed.

6.1.3. Manual calculation of bending moments

Before designing the reinforcement, I calculated bending moments manually using direct design method (DDM) to check and compare the results from software. I chose belts in x-direction and y-direction: *belt A* and *belt 1*. Then total moments of outer panel and adjacent inner panel of each belt have to be calculated.



- ***Total moment***

$$M_{tot} = \frac{1}{8} \times f_{d,slab} \times b \times ln^2$$

Panel A_{out}: $Mtot = \frac{1}{8} \times 14.563 \times 5.88 \times 7.1^2 = 539.58kNm$

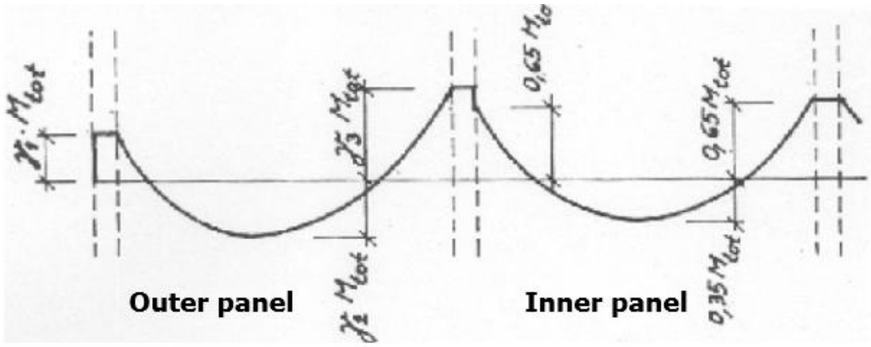
Panel A_{in}: $Mtot = \frac{1}{8} \times 14.563 \times 5.88 \times 4.6^2 = 226.49kNm$

Panel 1_{out}: $Mtot = \frac{1}{8} \times 14.563 \times 5 \times 5.46^2 = 271.34kNm$

Panel 1_{in}: $Mtot = \frac{1}{8} \times 14.563 \times 5 \times 5.6^2 = 285.44kNm$

- ***Total 'positive' and 'negative' moments***

Calculated total moments have to be divided into 'positive' (midspan) and 'negative' (supports) moments in each panel using γ coefficients, which we get from following table.



	Edge of the slab is freely supported on wall	Slab has stiffening beams in all column strips	Slab has not stiffening beams in column strip and		Edge of the slab is fixed
			has not an edge beam	has an edge beam	
γ_1	0,00	0,16	0,26	0,30	0,65
γ_2	0,63	0,57	0,52	0,50	0,35
γ_3	0,75	0,70	0,70	0,70	0,65

Panel A_{out}:

$M_1 = \gamma_1 \times M_f = 0.3 \times 539.58 = 161.87 kNm$

$M_2 = \gamma_2 \times M_f = 0.5 \times 539.58 = 269.79 kNm$

$M_3 = \gamma_3 \times M_f = 0.7 \times 539.58 = 377.71 kNm$

Panel A_{in}:

$M_1 = \gamma_1 \times M_f = 0.65 \times 226.49 = 147.22 kNm$

$M_2 = \gamma_2 \times M_f = 0.35 \times 226.49 = 79.27 kNm$

$M_3 = \gamma_3 \times M_f = 0.65 \times 226.49 = 147.22 kNm$

Panel 1_{out}:

$M_1 = \gamma_1 \times M_f = 0.3 \times 271.34 = 81.4 kNm$

$M_2 = \gamma_2 \times M_f = 0.5 \times 271.34 = 135.67 kNm$

$M_3 = \gamma_3 \times M_f = 0.7 \times 271.34 = 189.94 kNm$

Panel 1_{in}:

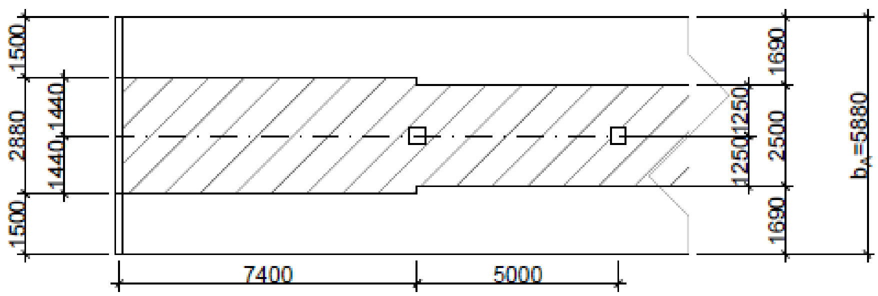
$M_1 = \gamma_1 \times M_f = 0.65 \times 285.44 = 185.54 kNm$

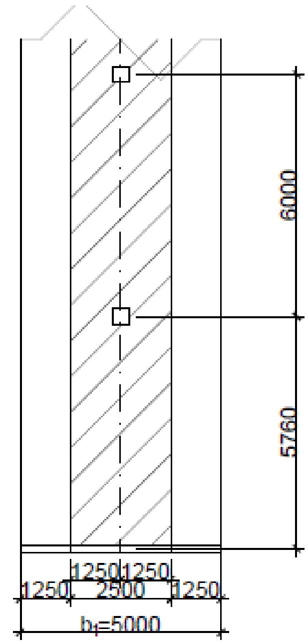
$M_2 = \gamma_2 \times M_f = 0.35 \times 285.44 = 99.9 kNm$

$M_3 = \gamma_3 \times M_f = 0.65 \times 285.44 = 185.54 kNm$

- ***Column and middle strips***

Each belt has to be divided to heavily loaded column strips and less loaded middle strips. The width of the column strip is ¼ of shorter span of adjacent panel in both x and y directions. The width of the middle strip is the rest width of each belt.





Panel A_{out}: column strip – 2880mm, middle strip – 3000mm.

Panel A_{in}: column strip – 2500mm, middle strip – 3380mm.

Panel 1_{out}: column strip – 2500mm, middle strip – 2500mm.

Panel 1_{in}: column strip – 2500mm, middle strip – 2500mm.

- ***Moments in column and middle strips / Moments per 1m***

Total ‘positive’ and ‘negative’ moments have to be divided to moments in column and middle strips using the ω coefficients. In my case ω coefficients are:

1 – for outer support (for moments above the wall)

0.6 – for midspan (for positive moments)

0.75 – for inner support (for moments above the column)

See following table.

TABLE 13.4
Column-strip moment, percent of total moment at critical section

		l_2/l_1		
		0.5	**1.0**	**2.0**
Interior negative moment				
$\alpha_{f1}l_2/l_1 = 0$		75	75	75
$\alpha_{f1}l_2/l_1 \geq 1.0$		90	75	45
Exterior negative moment				
$\alpha_{f1}l_2/l_1 = 0$	$\beta_t = 0$	100	100	100
	$\beta_t \geq 2.5$	75	75	75
$\alpha_{f1}l_2/l_1 \geq 1.0$	$\beta_t = 0$	100	100	100
	$\beta_t \geq 2.5$	90	75	45
Positive moment				
$\alpha_{f1}l_2/l_1 = 0$		60	60	60
$\alpha_{f1}l_2/l_1 \geq 1.0$		90	75	45

Moments in column and middle strips must be divided by the width of each column and middle strip. Calculation provided in the table below.

Moments in column and middle strips

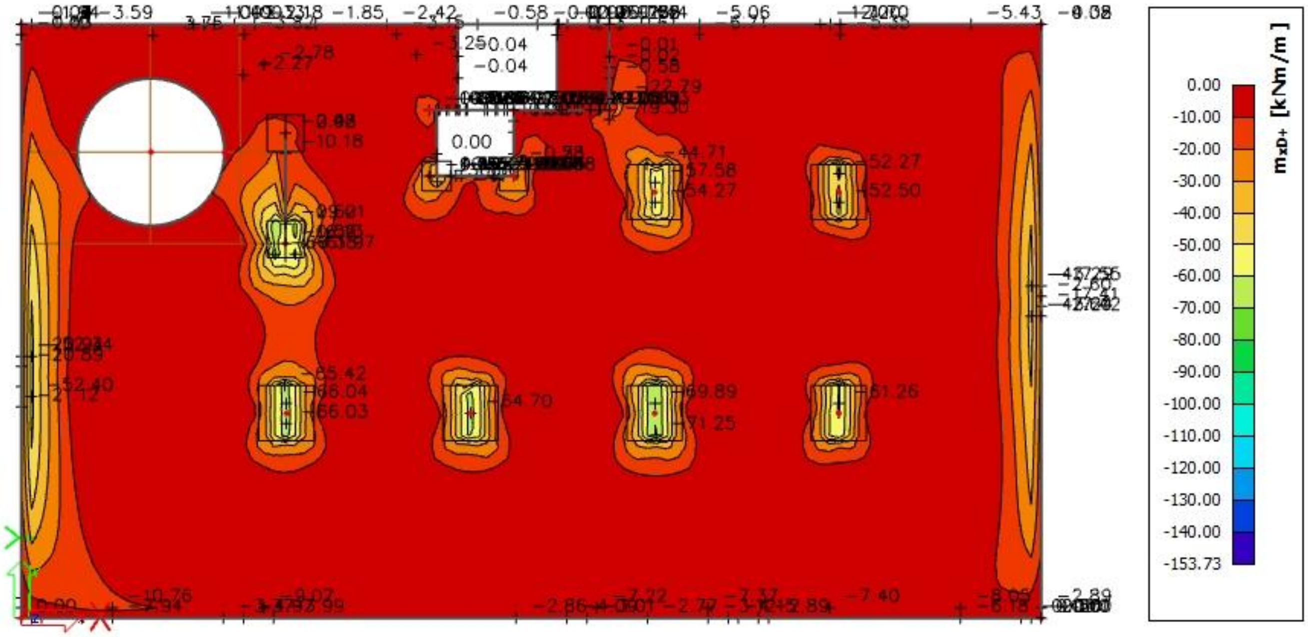
Panel	Cross-section	Positive/negative moment M [kNm]	Strip	ω	Moment in column/ middle strip Mj [kNm]	Width of the strip sj [m]	Moment per 1 m of the slab mj [kNm/m]
Aout	1 (left support)	161.87	no divisi	1	161.87	7.4	21.9
	2 (midspan)	269.79	Column	0.6	161.87	2.88	56.2
			Middle		107.92	3	36.0
	3 (right support)	377.71	Column	0.75	283.28	2.88	98.4
			Middle		94.428	3	31.5
Ain	1 (left support)	147.22	Column	0.75	110.42	2.5	44.2
			Middle		36.805	3.38	10.9
	2 (midspan)	79.27	Column	0.6	47.562	2.5	19.0
			Middle		31.708	3.38	9.4
	3 (right support)	147.22	Column	0.75	110.42	2.5	44.2
			Middle		36.805	3.38	10.9
1out	1 (left support)	81.4	no divisio	1	81.4	5.76	14.1
	2 (midspan)	135.67	Column	0.6	81.402	2.5	32.6
			Middle		54.268	2.5	21.7
	3 (right support)	189.94	Column	0.75	142.46	2.5	57.0
			Middle		47.485	2.5	19.0
1in	1 (left support)	185.54	Column	0.75	139.16	2.5	55.7
			Middle		46.385	2.5	18.6
	2 (midspan)	99.9	Column	0.6	59.94	2.5	24.0
			Middle		39.96	2.5	16.0
	3 (right support)	185.54	Column	0.75	139.16	2.5	55.7
			Middle		46.385	2.5	18.6

6.1.4. Final design

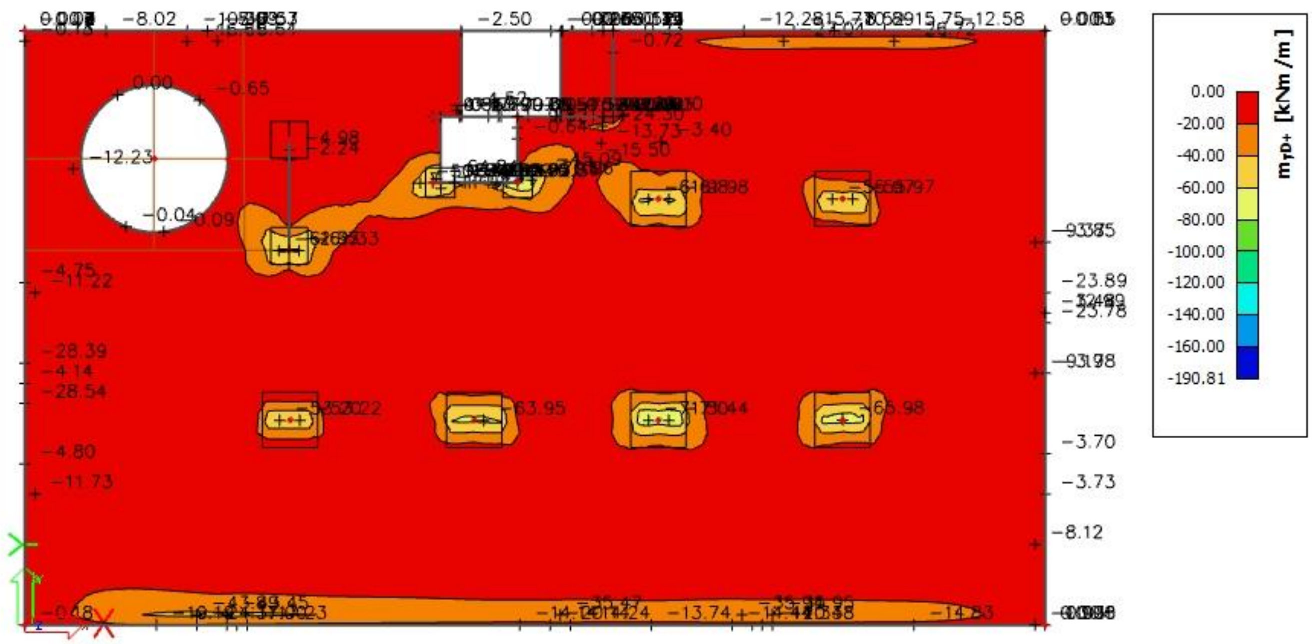
To design the reinforcement of the slab, we use the moments received from FEM software.

For design of top reinforcement we take negative moments and for bottom reinforcement positive moments must be used. To design reinforcement around slab opening I made sections around the opening and through the opening in both axis.

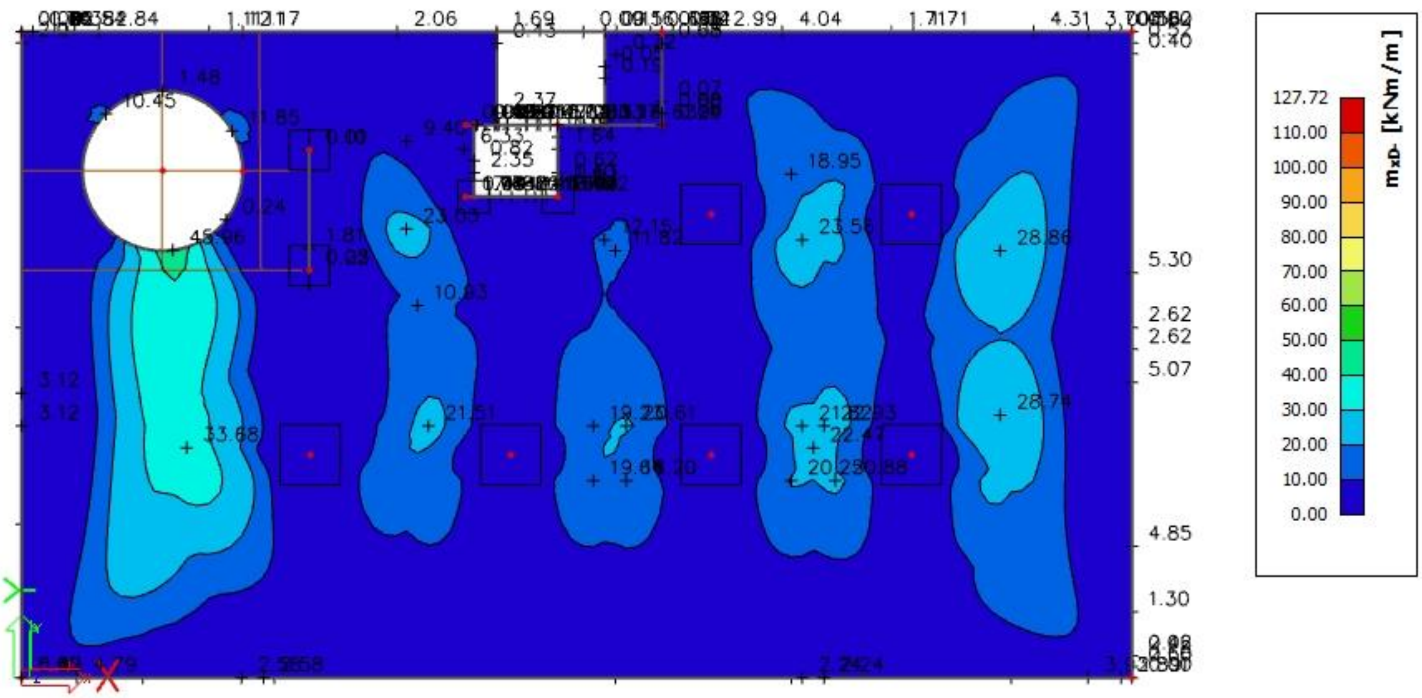
- Negative moments in x-direction (above columns and walls):



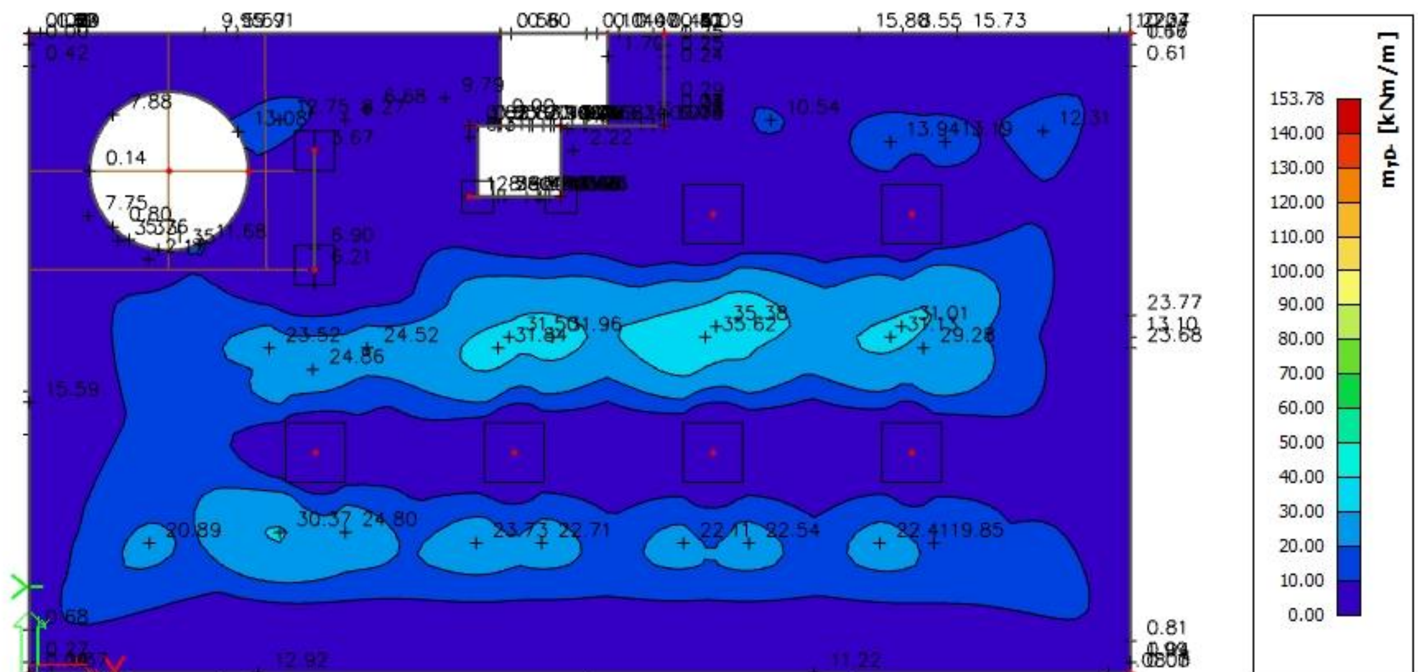
- Negative moments in y-direction (above columns and walls):



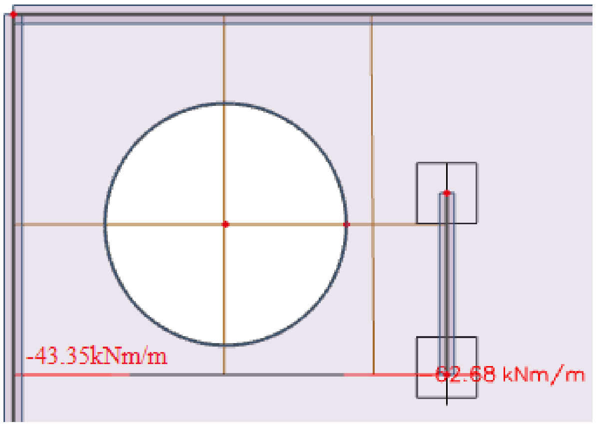
- Positive moments in x-direction (in midspans):

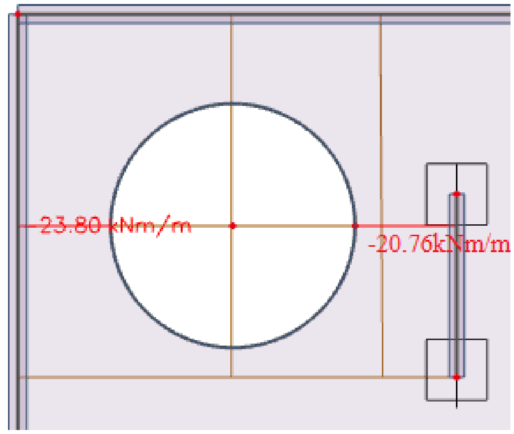


- Positive moments in y-direction (in midspans):

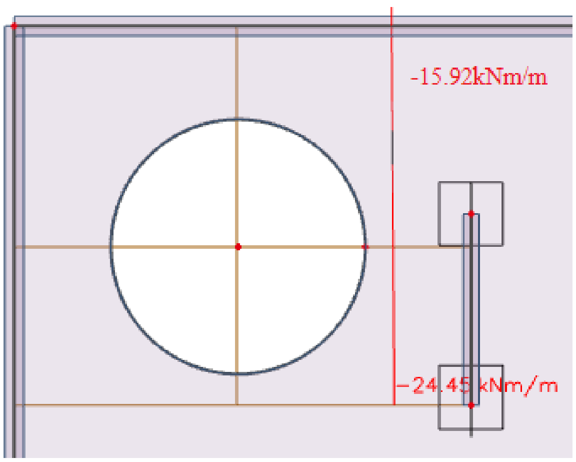


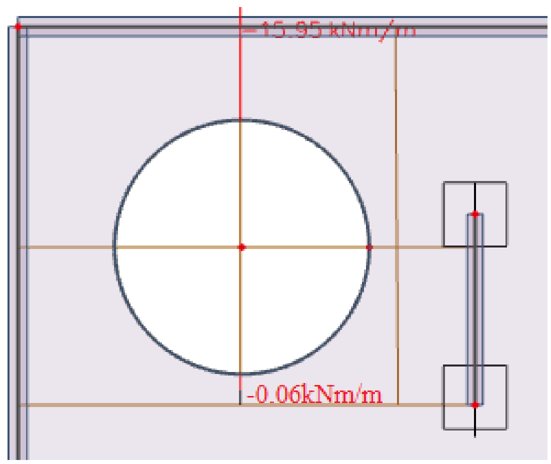
- Negative moments in x-direction around and through the slab opening:



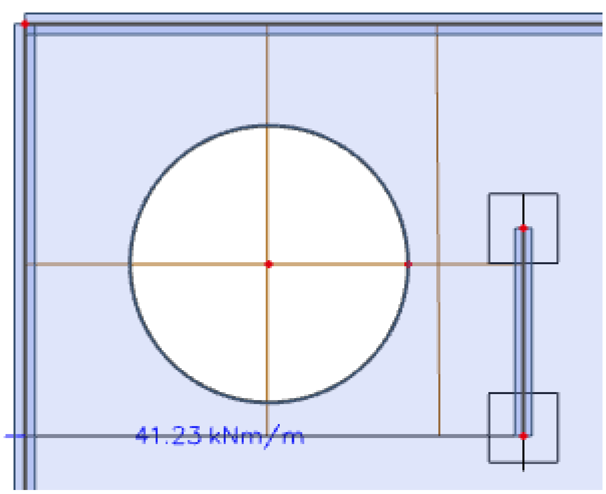


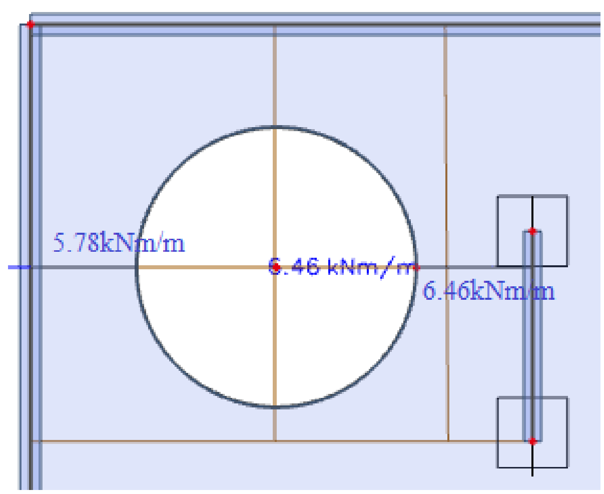
- Negative moments in y-direction around and through the slab opening:



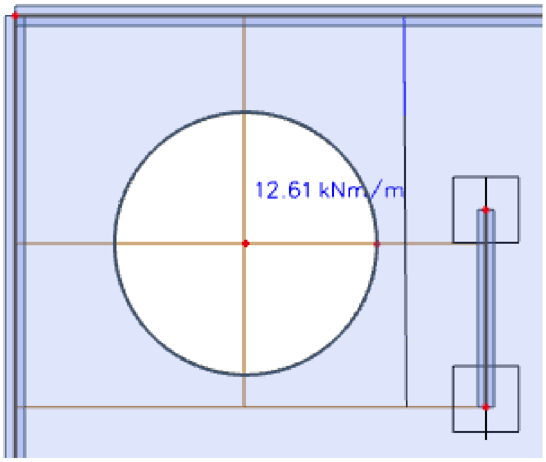


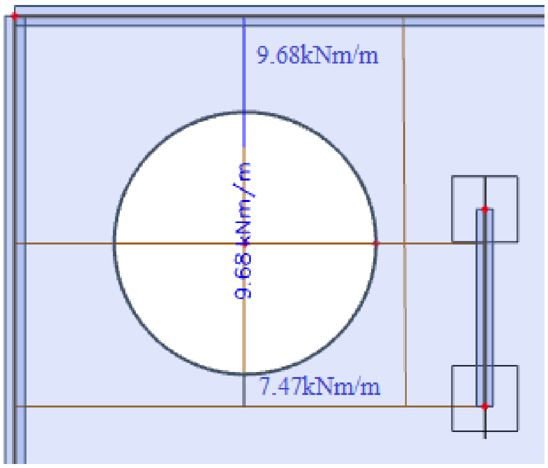
- Positive moments in x-direction around and through the slab opening:





- Positive moments in y-direction around and through the slab opening:





- ***Area of reinforcement***

Area of the reinforcement will be received through following formula.

$$a_{s,prov} \geq a_{s,req} = \frac{m_{ed}}{z \times f_{yd}}$$

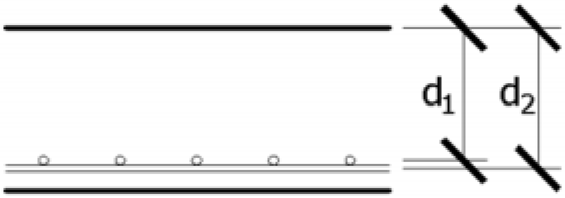
$z = 0.9 \times d$ (lever arm of internal forces)

$f_{yk} = 500MPa$ (characteristic strength of steel)

$\gamma_s = 1.15$ (partial safety factor for steel)

$f_{yd} = \frac{f_{yk}}{\gamma_s} = \frac{500}{1.15} = 435MPa$ (design yield strength of steel)

For two-way slab, there are two different effective depths:



$d_1 = h_s - c - \emptyset - \frac{\emptyset}{2} = 275 - 20 - 10 - 5 = 240mm$

$d_2 = h_s - c - \frac{\emptyset}{2} = 275 - 20 - 5 = 250mm$

$\emptyset = 10mm$ (assumed diameter of steel bars)

- ***Minimum reinforcement***

Brittle failure check:

$$a_{s,prov} \geq a_{s,min,1} = max.\left(0.26 \times \frac{f_{ctm}}{f_{yk}} \times b \times d; 0.0013 \times b \times d\right)$$

$f_{ctm} = 2.9MPa$ (mean tensile strength of concrete C30/37)

b – width of the slab, per 1m

Excessive cracking check:

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

$f_{ct,eff}$ – mean value of the tensile strength of concrete effective at the time when the first cracks may occur. In my case, $f_{ct,eff} = f_{ctm}$

$k_c = 0.4$

$k = 1$ – coefficients describing stress distribution in the cross-section

$A_{ct} = 0.5 \times b \times d$ – area of concrete within tensile zone at the first crack

$\sigma_s = f_{yk}$ – maximum stress permitted in the reinforcement immediately after formation of the crack

- ***Check of the design***

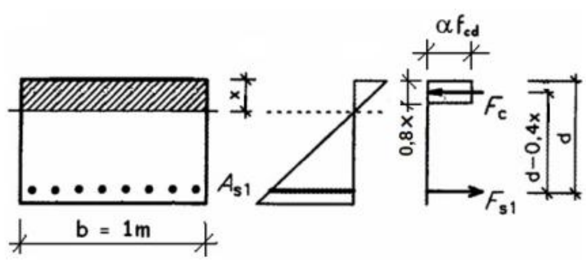
Thickness of compression zone of concrete cross-section :

$$x = \frac{a_{s,prov} \times f_{yd}}{0.8 \times b \times f_{cd}}$$

$z = d - 0.4 \times x$ – real value of lever arm of internal forces

$m_{Rd} = a_{s,prov} \times f_{yd} \times z$ – resistant moment

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



- ***Detailing rules***

Relative height of compressed zone:

$$\xi = \frac{x}{d} \leq 0.45$$

Spacing of the steel bars:

$s \leq min.(2 \times h_s; 250mm)$

All of these conditions above must be checked. Calculation is provided in following table. Table is also available in bigger scale with attached drawings.

DESIGN AND CHECK OF BENDING REINFORCEMENT OF THE SLAB																			
		DESIGN											Check						
		Med	d	z	as,rqd	as,min1	as,min2	mber of reb	Design	ø	max spacin	as,prov	x	ξ	z	mRd	mRd>mE	ξ < 0,45	spacing
Panel	Cross-section	[kNm/m]	[mm]	[mm]	[mm2]	[mm2]	[mm2]			[mm]	[mm]	[mm2]	[kNm/m]	[mm]	[mm]	[mm2]	[mm2]		[mm2]
	GENERAL	20.0	250	225	204.44	377.00	290.00	5.00	5ø10 /200	10	200	392.50	10.67	0.042665	245.73	41.94	OK	OK	OK
m,x + (negative)	Pillar	66.4	250	225	678.52	377.00	290.00	9.00	9ø10 /111	10	111.1111	706.50	19.20	0.076797	242.32	74.44	OK	OK	OK
	C1	57.6	250	225	588.57	377.00	290.00	8.00	8ø10 /125	10	125	628.00	17.07	0.068264	243.17	66.40	OK	OK	OK
	C2	52.5	250	225	536.65	377.00	290.00	7.00	7ø10 /143	10	142.8571	549.50	14.93	0.059731	244.03	58.30	OK	OK	OK
	C3	66.0	250	225	675.05	377.00	290.00	9.00	9ø10 /111	10	111.1111	706.50	19.20	0.076797	242.32	74.44	OK	OK	OK
	C4	64.7	250	225	661.35	377.00	290.00	9.00	9ø10 /111	10	111.1111	706.50	19.20	0.076797	242.32	74.44	OK	OK	OK
	C5	71.3	250	225	728.30	377.00	290.00	10.00	10ø10 /100	10	100	785.00	21.33	0.08533	241.47	82.42	OK	OK	OK
	C6	61.3	250	225	626.19	377.00	290.00	8.00	8ø10 /125	10	125	628.00	17.07	0.068264	243.17	66.40	OK	OK	OK
	Elevator corner	36.4	250	225	371.67	377.00	290.00	5.00	5ø10 /200	10	200	392.50	10.67	0.042665	245.73	41.94	OK	OK	OK
	Left support (wall)	52.4	250	225	535.62	377.00	290.00	7.00	7ø10 /143	10	142.8571	549.50	14.93	0.059731	244.03	58.30	OK	OK	OK
	Right support (wall)	43.3	250	225	442.50	377.00	290.00	6.00	6ø10 / 167	10	166.6667	471.00	12.80	0.051198	244.88	50.15	OK	OK	OK
m,y + (negative)	Pillar	61.4	240	216	653.24	361.92	278.40	9.00	9ø10 /111	10	111.1111	706.50	19.20	0.079996	232.32	71.37	OK	OK	OK
	C1	62.0	240	216	659.95	361.92	278.40	9.00	9ø10 /111	10	111.1111	706.50	19.20	0.079996	232.32	71.37	OK	OK	OK
	C2	56.0	240	216	595.95	361.92	278.40	8.00	8ø10 /125	10	125	628.00	17.07	0.071108	233.17	63.67	OK	OK	OK
	C3	53.2	240	216	566.67	361.92	278.40	8.00	8ø10 /125	10	125	628.00	17.07	0.071108	233.17	63.67	OK	OK	OK
	C4	64.0	240	216	680.92	361.92	278.40	9.00	9ø10 /111	10	111.1111	706.50	19.20	0.079996	232.32	71.37	OK	OK	OK
	C5	71.5	240	216	761.31	361.92	278.40	10.00	10ø10 /100	10	100	785.00	21.33	0.088885	231.47	79.00	OK	OK	OK
	C6	66.0	240	216	702.54	361.92	278.40	9.00	9ø10 /111	10	111.1111	706.50	19.20	0.079996	232.32	71.37	OK	OK	OK
	Elevator corner	71.0	240	216	755.56	361.92	278.40	10.00	10ø10 /100	10	100	785.00	21.33	0.088885	231.47	79.00	OK	OK	OK
	Top support (wall)	27.0	240	216	287.91	361.92	278.40	5.00	5ø10 /200	10	200	392.50	10.67	0.044442	235.73	40.23	OK	OK	OK
	Bottom support (wall)	43.9	240	216	467.33	361.92	278.40	6.00	6ø10 / 167	10	166.6667	471.00	12.80	0.053331	234.88	48.10	OK	OK	OK
m,x - (positive)	1st midspan	33.7	250	225	344.27	377.00	290.00	5.00	5ø10 /200	10	200	392.50	10.67	0.042665	245.73	41.94	OK	OK	OK
	2nd midspan	23.0	250	225	235.41	377.00	290.00	5.00	5ø10 /200	10	200	392.50	10.67	0.042665	245.73	41.94	OK	OK	OK
	3rd midspan	20.6	250	225	210.67	377.00	290.00	5.00	5ø10 /200	10	200	392.50	10.67	0.042665	245.73	41.94	OK	OK	OK
	4th midspan	23.6	250	225	240.83	377.00	290.00	5.00	5ø10 /200	10	200	392.50	10.67	0.042665	245.73	41.94	OK	OK	OK
	5th midspan	28.9	250	225	295.00	377.00	290.00	5.00	5ø10 /200	10	200	392.50	10.67	0.042665	245.73	41.94	OK	OK	OK
m,y - (positive)	1st midspan	30.4	240	216	323.37	361.92	278.40	5.00	5ø10 /200	10	200	392.50	10.67	0.044442	235.73	40.23	OK	OK	OK
	2nd midspan	35.6	240	216	379.27	361.92	278.40	5.00	5ø10 /200	10	200	392.50	10.67	0.044442	235.73	40.23	OK	OK	OK
opening + (negative)	x - bottom section (right)	62.7	250	225	640.70	377.00	290.00	9.00	9ø10 /111	10	111.1111	706.50	19.20	0.076797	242.32	74.44	OK	OK	OK
	x - bottom section (left)	43.4	250	225	443.12	377.00	290.00	6.00	6ø10 / 167	10	166.6667	471.00	12.80	0.051198	244.88	50.15	OK	OK	OK
	x - section through (right)	20.8	250	225	212.20	377.00	290.00	5.00	5ø10 /200	10	200	392.50	10.67	0.042665	245.73	41.94	OK	OK	OK
	x - section through (left)	23.8	250	225	243.28	377.00	290.00	5.00	5ø10 /200	10	200	392.50	10.67	0.042665	245.73	41.94	OK	OK	OK
	y - bottom section (right)	15.9	240	216	169.51	361.92	278.40	5.00	5ø10 /200	10	200	392.50	10.67	0.044442	235.73	40.23	OK	OK	OK
	y - bottom section (left)	24.5	240	216	260.34	361.92	278.40	5.00	5ø10 /200	10	200	392.50	10.67	0.044442	235.73	40.23	OK	OK	OK
	y - section through	16.0	240	216	169.83	361.92	278.40	5.00	5ø10 /200	10	200	392.50	10.67	0.044442	235.73	40.23	OK	OK	OK
opening - (positive)	x - bottom section	41.2	250	225	421.45	377.00	290.00	6.00	6ø10 / 167	10	166.6667	471.00	12.80	0.051198	244.88	50.15	OK	OK	OK
	x - section through (right)	6.5	250	225	66.03	377.00	290.00	5.00	5ø10 /200	10	200	392.50	10.67	0.042665	245.73	41.94	OK	OK	OK
	x - section through (left)	5.8	250	225	59.08	377.00	290.00	5.00	5ø10 /200	10	200	392.50	10.67	0.042665	245.73	41.94	OK	OK	OK
	y - bottom section	12.6	240	216	134.27	361.92	278.40	5.00	5ø10 /200	10	200	392.50	10.67	0.044442	235.73	40.23	OK	OK	OK
	y - section through (right)	9.7	240	216	103.28	361.92	278.40	5.00	5ø10 /200	10	200	392.50	10.67	0.044442	235.73	40.23	OK	OK	OK
	y - section through (left)	7.5	240	216	79.54	361.92	278.40	5.00	5ø10 /200	10	200	392.50	10.67	0.044442	235.73	40.23	OK	OK	OK

6.1.5. Anchorage length

Anchorage length is the length needed to transmit the forces from bars to concrete safely avoiding longitudinal cracks. I designed anchorage length for bottom and top reinforcements.

Top reinforcement:

$$l_{b,d} = \alpha_1 \times \alpha_2 \times \alpha_3 \times \alpha_4 \times \alpha_5 \times l_{b,req} \geq l_{b,min} = max.(0.3 \times l_{b,req}; 10\emptyset; 100mm)$$

$\alpha_1 = 1$ (effect of form of the bars)

$\alpha_2 = 1$ (effect of concrete minimum cover depth)

$\alpha_3 = 1$ (effect of confinement by transverse reinforcement)

$\alpha_4 = 1$ (effect of influence of one or more welded transverse bars)

$\alpha_5 = 1$ (effect of the transverse pressure)

α coefficients are taken from the table below.

Table - coefficients α_1, α_2, α_3, α_4 and α_5

Influencing factor	Type of anchorage	Reinforcement bar In tension	Reinforcement bar In compression
Shape of bars	Straight	$\alpha_1 = 1{,}0$	$\alpha_1 = 1{,}0$
	Other than straight (see Figure 8.1 (b), (c) and (d)	$\alpha_1 = 0{,}7$ if $c_d > 3\phi$ otherwise $\alpha_1 = 1{,}0$ (see Figure 1for values of c_d)	$\alpha_1 = 1{,}0$
Concrete cover	Straight	$\alpha_2 = 1 - 0{,}15\,(c_d - \phi)/\phi$ $\geq 0{,}7$ $\leq 1{,}0$	$\alpha_2 = 1{,}0$
	Other than straight (see Figure 8.1 (b), (c) and (d))	$\alpha_2 = 1 - 0{,}15\,(c_d - 3\phi)/\phi$ $\geq 0{,}7$ $\leq 1{,}0$ (see figure 1 for values of c_d)	$\alpha_2 = 1{,}0$
Confinement by transverse reinforcement not welded to main reinforcement	All types	$\alpha_3 = 1 - K\lambda$ $\geq 0{,}7$ $\leq 1{,}0$	$\alpha_3 = 1{,}0$
Confinement by welded transverse reinforcement*	All types, position and size as specified in Figure 8.1 (e)	$\alpha_4 = 0{,}7$	$\alpha_4 = 0{,}7$
Confinement by transverse pressure	All types	$\alpha_5 = 1 - 0{,}04p$ $\geq 0{,}7$ $\leq 1{,}0$	-

where:

λ $= (\Sigma A_{st} - \Sigma A_{st,min})/ A_s$

ΣA_{st} cross-sectional area of the transverse reinforcement along the design anchorage length l_{bd}

$\Sigma A_{st,min}$ cross-sectional area of the minimum transverse reinforcement

$= 0{,}25\, A_s$ for beams and 0 for slabs

A_s area of a single anchored bar with maximum bar diameter

K values shown in figure

p transverse pressure [MPa] at ultimate limit state along l_{bd}

For direct supports l_{bd} may be taken less than $l_{b,min}$ provided that there is at least one transverse wire welded within the support. This should be at least 15 mm from the face of the support.

Basic anchorage length:

$$l_{b,req} = \frac{\emptyset}{4} \times \frac{\sigma_{sd}}{f_{bd}}$$

$\sigma_{sd} = f_{yd} -$ stress in the reinforcement

$f_{bd} = 2.25 \times \eta_1 \times \eta_2 \times f_{ctd}$

$\eta_1 = 1$ (coefficeint expressing position of steel bars during concreting)

$\eta_2 = 1$ (coefficient expressing diameter of bars)

$f_{ctd} = \frac{f_{ctk,0.05}}{1.5} = \frac{2}{1.5} = 1.333MPa$ (design value of concrete tensile strength)

$f_{bd} = 2.25 \times 1 \times 1 \times 1.333 = 3MPa$ (design value of ultimate bond stress)

$$l_{b,req} = \frac{10}{4} \times \frac{435}{3} = 362.5mm$$

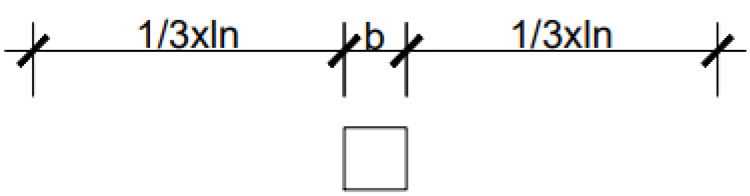
$l_{b,d} = 1 \times 1 \times 1 \times 1 \times 1 \times 362.5 \geq l_{b,min} = max.(0.3 \times 362.5; 10 \times 10; 100mm)$

$l_{b,d} = 362.5 \geq l_{b,min} = max.(108.75; 100; 100mm) = 108.75mm$

$l_{b,d} = 365mm$ – design anchorage length of reinforcement bars at the edge, near the walls

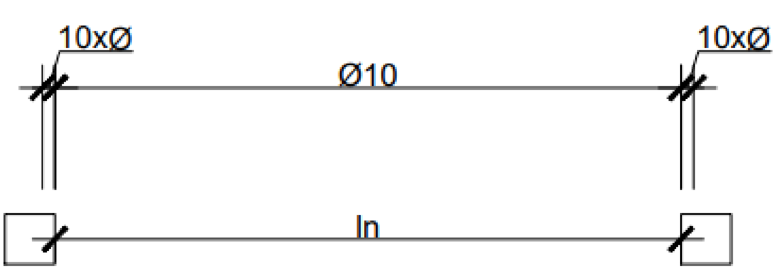
For reinforcement bars around columns, pillar and slab opening we can assume the total length of the bar including the anchorage length as 1/3 of clear span l_n from both sides of the structure.

Top reinforcement:



Bottom reinforcement:

For lower reinforcement we can assume anchorage length as 10Ø.



6.1.6. Punching reinforcement

Now we have to check if punching reinforcement is needed. For column C3 (the highest load):

- ***Resistance without reinforcement***

$$v_{Ed,1} = \frac{\beta \times Ved}{u_1 \times d} \leq v_{Rd,c} = C_{Rd,c} \times k \times \sqrt[3]{(100 \times \rho_l \times fck)}$$

$\rho_l = \sqrt{\rho_{lx} \times \rho_{ly}} = \sqrt{\frac{706}{250\times1000} \times \frac{628}{250\times1000}} = 0.0027$ (reinforcement ratio of tensile reinforcement, values of $a_{s,prov}$ taken as the designed longitudinal reinforcement of the slab above column C3)

$$v_{Ed,1} = \frac{\beta \times Ved}{u_1 \times d} = \frac{1.15 \times 534750}{4741.6 \times 250} = 0.52MPa$$

$$v_{Rd,c} = C_{Rd,c} \times k \times \sqrt[3]{(100 \times \rho_l \times fck)} = 0.12 \times 1.9 \times \sqrt[3]{100 \times 0.0027 \times 30} = 0.46MPa$$

$$v_{Ed,1} = 0.52MPa > 0.46MPa$$

The condition is not satisfying, which means that punching reinforcement should be designed in this column. As the difference is quite small and no punching reinforcement is needed for the other supports (see further calculations), we will increase the amount of longitudinal reinforcement in column C3 instead of designing the punching reinforcement. Instead of 9 Ø10/m in x direction and 8 Ø10/m in y-direction, we will use 10 Ø12/m in both directions. Then we will have:

$$\rho_l = \sqrt{\rho_{lx} \times \rho_{ly}} = \sqrt{\frac{1130}{250\times1000} \times \frac{113}{250\times1000}} = 0.0045$$

$$v_{Rd,c} = C_{Rd,c} \times k \times \sqrt[3]{(100 \times \rho_l \times fck)} = 0.12 \times 1.9 \times \sqrt[3]{100 \times 0.0045 \times 30} = 0.54MPa$$

$$v_{Ed,1} = 0.52MPa < 0.54\,MPa \quad \checkmark$$

Now the condition is satisfying without the punching reinforcement.

For column C6 (the column with the second highest load):

- ***Resistance without reinforcement***

$$v_{Ed,1} = \frac{\beta \times Ved}{u_1 \times d} \leq v_{Rd,c} = C_{Rd,c} \times k \times \sqrt[3]{(100 \times \rho_l \times fck)}$$

$Ved = 14.563kN/m^2 \times 31.46m^2 = 458.15kN$ (load acting from the slab to column area)

$\rho_l = \sqrt{\rho_{lx} \times \rho_{ly}} = \sqrt{\frac{628}{250\times1000} \times \frac{706}{250\times1000}} = 0.0027$ (reinforcement ratio of tensile reinforcement, values of $a_{s,prov}$ taken as the designed longitudinal reinforcement of the slab above column C6, see section 6.1.4)

$$v_{Ed,1} = \frac{\beta \times Ved}{u_1 \times d} = \frac{1.15 \times 458150}{4741.6 \times 250} = 0.44MPa$$

$$v_{Rd,c} = C_{Rd,c} \times k \times \sqrt[3]{(100 \times \rho_l \times fck)} = 0.12 \times 1.9 \times \sqrt[3]{100 \times 0.0027 \times 30} = 0.46MPa$$

$$v_{Ed,1} = 0.44MPa < 0.46MPa \quad \checkmark$$

The condition is satisfying, which means no punching reinforcement is needed for other columns than C3.

For the pillar:

- ***Resistance without reinforcement***

$$v_{Ed,1} = \frac{\beta \times Ved}{u_1 \times d} \leq v_{Rd,c} = C_{Rd,c} \times k \times \sqrt[3]{(100 \times \rho_l \times fck)}$$

$\rho_l = \sqrt{\rho_{lx} \times \rho_{ly}} = \sqrt{\frac{706}{250\times1000} \times \frac{706}{250\times1000}} = 0.0028$ (reinforcement ratio of tensile reinforcement, values of $a_{s,prov}$ taken as the designed longitudinal reinforcement of the slab above the pillar, see section 6.1.4)

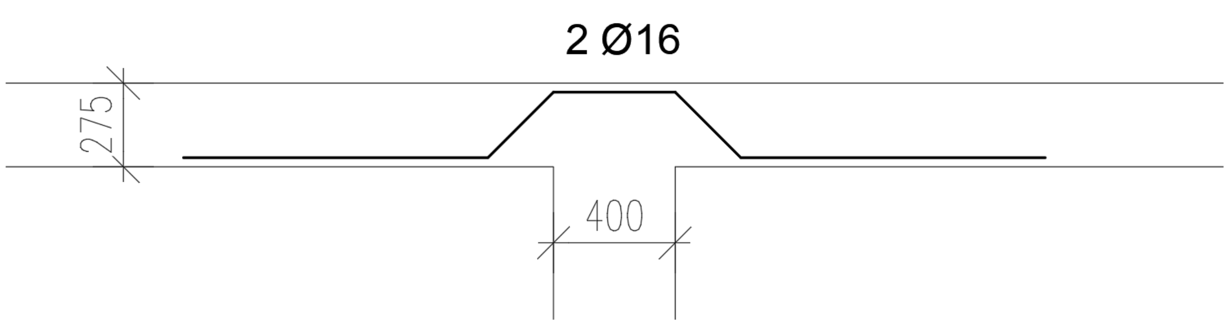
$$v_{Ed,1} = \frac{\beta \times Ved}{u_1 \times d} = \frac{1.15 \times 253400}{2570.8 \times 250} = 0.45MPa$$

$$v_{Rd,c} = C_{Rd,c} \times k \times \sqrt[3]{(100 \times \rho_l \times fck)} = 0.12 \times 1.9 \times \sqrt[3]{100 \times 0.0028 \times 30} = 0.46MPa$$

$$v_{Ed,1} = 0.45MPa < 0.46MPa \quad \checkmark$$

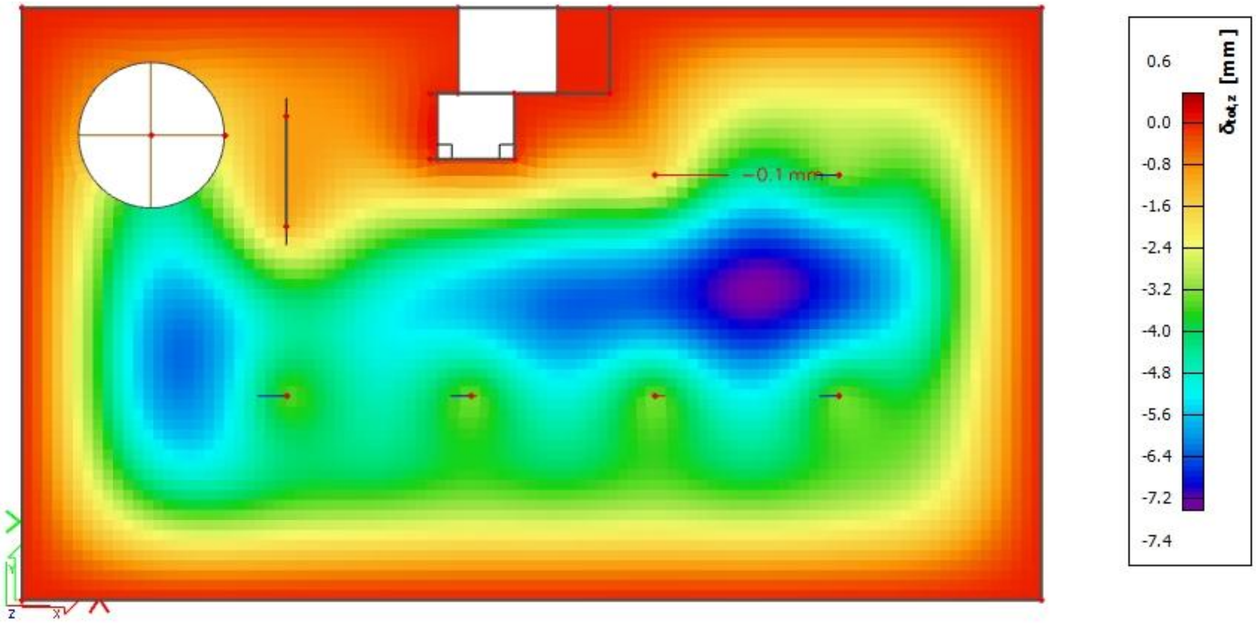
The condition is satisfying, which means no punching reinforcement is needed for the pillar.

For safety reasons, two 16 mm bended bars will be added above each column and pillar end in both directions as safety punching reinforcement.



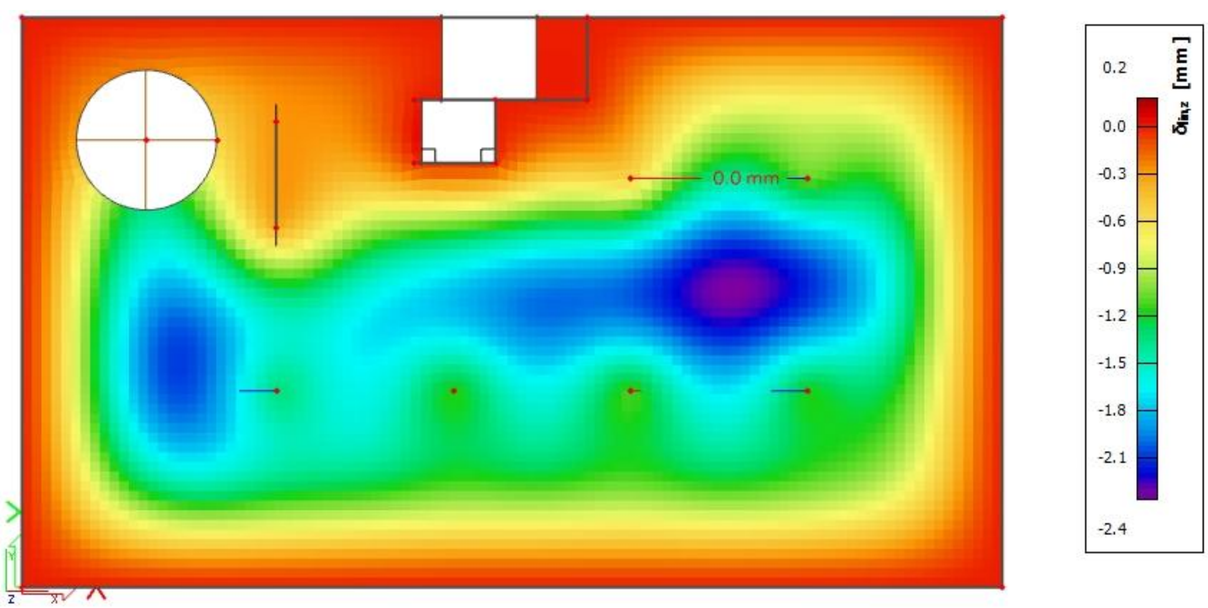
6.1.7. Check of the deflections

Total deflection δ_{tot} :



$$\delta_{tot} = 7.4mm \leq \frac{L_{max}}{250} = \frac{7400}{250} = 29.6mm \ \checkmark$$

Linear deflection δ_{lin} :



Total deflection should be approximately 3 times bigger than linear deflection.

$\delta_{lin} = 2.4mm \times 3 = 7.2mm < \delta_{tot} = 7.4mm$ ✓

6.2. Wall

To design reinforcement in the wall, following equation can be checked:

$$A_{s,req} = \frac{N_{Ed,w} - 0.8 \times A_c \times f_{cd}}{\sigma_s}$$

$N_{Ed,w} = 368.21 kN/m$

t_w= 200mm

b = 1000mm (per 1m)

$f_{cd} = 20MPa$

$\sigma_s = 400MPa$

$A_{s,req} = \frac{368.21 \times 10^3 - 0.8 \times 200 \times 1000 \times 20}{400} = -7079.5 mm^2/m$

$A_{s,req} = -7079.5 mm^2/m < 0$,so minimum design of reinforcement $4 \times \emptyset 8 mm/m$ can be used → $A_s = 201 mm^2/m$

$N_{Rd,w} = 0.8 \times A_c \times f_{cd} + A_s \times \sigma_s = 0.8 \times 200 \times 1000 \times 20 + 201 \times 400$

$N_{Rd,w} = 3280.4 kN/m > N_{Ed,w} = 368.21 kN/m$ ✓

I designed reinforcement of the wall based on detailing rules.

Vertical reinforcement:

$0.002 \times a_c \leq a_{s,v} \leq 0.04 \times a_c$

$0.002 \times 200000 \leq a_{s,v} \leq 0.04 \times 200000$

$400 mm^2/m \leq a_{s,v} \leq 8000 mm^2/m$

$a_{s,v} = 400 mm^2/m \rightarrow 4 \times \emptyset 8 mm/m$ on each surface (2×200mm^2/m)

$s_v \leq min.(3 \times t; 400mm) \rightarrow s_v = 250mm$ (spacing)

Horizontal reinforcement:

$a_{s,h} \geq max.(0.25 \times a_{s,v}; 0.001 \times a_c)$

$a_{s,h} \geq max.(100 mm^2/m; 200 mm^2/m) = 200 mm^2/m$

$a_{s,h} = 300 mm^2/m \rightarrow 3 \times \emptyset 8 mm/m$ on each surface (2×150mm^2/m)

$s_h \leq 400mm \rightarrow s_v = 333mm$ (spacing)

6.2.1. Lapping length

Lapping length is the length needed to transmit forces from one rebar to another rebar. It depends on the shape of the bar, concrete cover and spacing between bars, on presence of transverse reinforcement and transverse pressure forces.

$$l_{0,d} = \alpha_1 \times \alpha_2 \times \alpha_3 \times \alpha_5 \times \alpha_6 \times l_{b,req} \geq l_{0,min} = max.(0.3 \times \alpha_6 \times l_{b,req}; 15\emptyset; 200mm)$$

$\alpha_6 = 1.5$ (coefficient expressing amount of lapped reinforcement, > 50% in my case)

Table: Values of the coefficient α_6

Percentage of lapped bars relative to the total cross-section area	< 25%	33%	50%	>50%
α_6	1	1,15	1,4	1,5
Note: Intermediate values may be determined by interpolation.				

Basic anchorage length:

$$l_{b,req} = \frac{\emptyset}{4} \times \frac{\sigma_{sd}}{f_{bd}}$$

$\sigma_{sd} = f_{yd} -$ stress in the reinforcement

$\eta_1 = 1$ (coefficeint expressing position of steel bars during concreting)

$\eta_2 = 1$ (coefficient expressing diameter of bars)

$f_{ctd} = \frac{f_{ctk,0.05}}{1.5} = \frac{2}{1.5} = 1.333MPa$ (design value of concrete tensile strength)

$f_{bd} = 2.25 \times \eta_1 \times \eta_2 \times f_{ctd} = 2.25 \times 1 \times 1 \times 1.333 = 3MPa$ (design value of bond stress between steel and concrete)

$$l_{b,req} = \frac{8}{4} \times \frac{435}{3} = 290mm$$

$$l_{0,d} = 1 \times 1 \times 1 \times 1 \times 1.5 \times 290 \geq l_{0,min} = max.(0.3 \times 1.5 \times 290; 15 \times 8; 200mm)$$

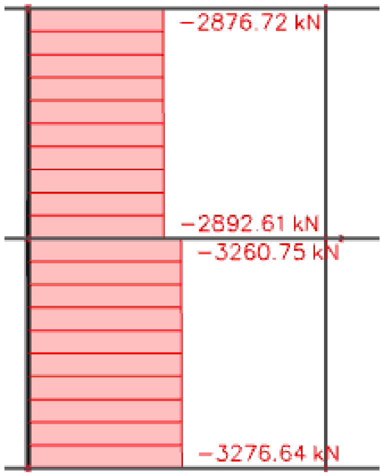
$$l_{0,d} = 435mm \geq l_{0,min} = max.(130.5; 120; 200mm) = 200mm \checkmark$$

$l_{0,d} = 450mm$ (design lapping length of horizontal wall reinforcement)

$s = 167mm$ (spacing in lapping area)

6.3. Column

To design column reinforcement, I took normal forces from SCIA Engineer in the most loaded column C3.



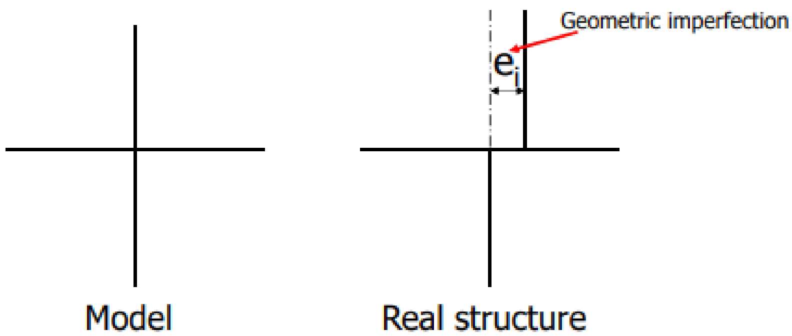
$N_{Ed} = -3276.64kN$ (highest normal force on the bottom of the column)

$h = 3m$

$400 \times 400mm$

6.3.1. Geometric imperfections

First, we should calculate geometric imperfections, which cause additional bending moments on real structure.



$$e_i = \theta_0 \times \alpha_h \times \alpha_m \times \frac{l_0}{2}$$

$\theta_0 = \frac{1}{200}$ (basic value of imperfection)

$\alpha_h = \frac{2}{\sqrt{h}}$ (height reduction factor) h – clear length of the column

$\alpha_m = \sqrt{0.5 \times (1 + \frac{1}{m})}$ (reduction factor for number of members)

$m = 4$ (number of columns in one frame)

$l_0 = 0.8 \times h$ (effective length of the column)

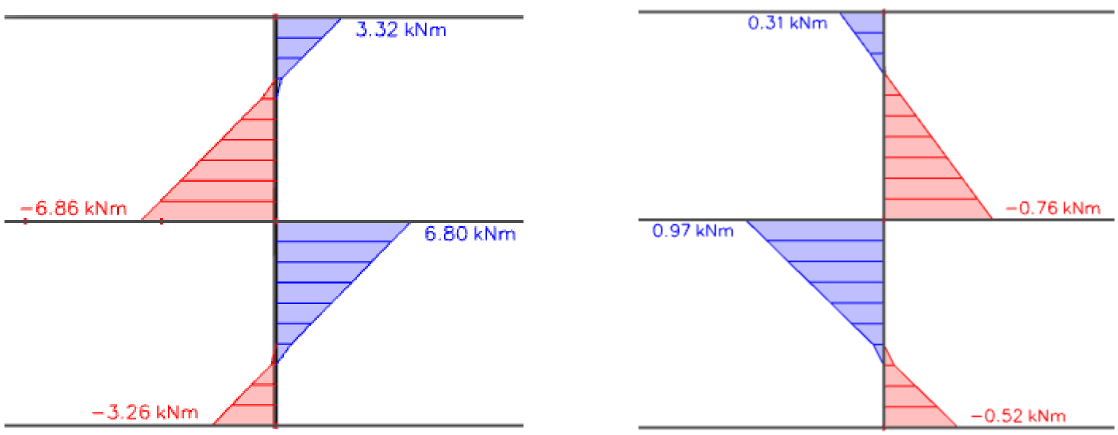
$$e_i = \frac{1}{200} \times \frac{2}{\sqrt{3}} \times \sqrt{0.5 \times (1 + \frac{1}{4})} \times \frac{0.8 \times 3}{2} = 5.48 \times 10^{-3} m$$

$M_{imp} = N_{Ed} \times e_i -$ additional moment due to geometric imperfection

$M_{imp} = 3276.64 kN \times 5.48 \times 10^{-3} m = 18 kNm$ (in the foot of the column)

$M_{imp} = 3260.75 kN \times 5.48 \times 10^{-3} m = 17.9 kNm$ (in the head of the column)

Then I calculated bending moments with the influence of geometric imperfections in the head and foot of the column for ULS combination in both y and z directions. Real bending moments from SCIA Engineer were used.



COMB	M[kNm]	Head of the column	Foot of the column
	$\lvert M_{imp} \rvert$	17.9	18
ULS (Y)	M_{Ed}	6.8	-3.26
	$\mathbf{M_{Ed,I}}$	**24.7**	**-21.26**
ULS (Z)	M_{Ed}	0.97	-0.52
	$\mathbf{M_{Ed,I}}$	**18.87**	**-18.52**

6.3.2. Slenderness of the column

Slenderness of the column must be checked by following expressions.

$$\lambda = \frac{l}{i}$$

$I = \frac{b_c \times h_c{}^3}{12} = \frac{0.4^4}{12} = 2.1333 \times 10^{-3} m^4$ (moment of inertia)

$i = \sqrt{\frac{I}{A_c}} = \sqrt{\frac{2.1333 \times 10^{-3}}{0.16}} = 0.115 m$ (radius of gyration)

$\lambda = \frac{0.8 \times H_c}{i} = \frac{0.8 \times 3}{0.115} = 20.87$ (slenderness of the column)

We have to calculate limiting slenderness of the column to check the slenderness.

$$\lambda_{lim} = \frac{20 \times A \times B \times C}{\sqrt{n}} \leq 75$$

$A = 0.7$ (effect of creep)

$B = 1.1$ (effect of reinforcement ratio)

$C = 1.7 - r_m$ (effect of bending moments)

$r_m = \frac{M_{01}}{M_{02}}$

M_{01} and M_{02} are bending moments in the head and foot of the column, $|M_{02}| > |M_{01}|$

$r_m = \frac{-21.26}{24.7} = -0.86$

$C = 1.7 - (-0.86) = 2.56$

$n = \frac{N_{Ed}}{A_c \times f_{cd}} = \frac{3276640}{400^2 \times 20} = 1$ (relative normal force)

$$\lambda_{lim} = \frac{20 \times 0.7 \times 1.1 \times 2.56}{\sqrt{1}} \leq 75$$

$$\lambda_{lim} = 39.424 \leq 75 \quad \checkmark \quad \text{for ULS (Y)}$$

$r_m = \frac{-0.52}{0.97} = -0.54$

$C = 1.7 - (-0.54) = 2.24$

$$\lambda_{lim} = \frac{20 \times 0.7 \times 1.1 \times 2.24}{\sqrt{1}} \leq 75$$

$$\lambda_{lim} = 34.5 \leq 75 \quad \checkmark \quad \text{for ULS (Z)}$$

I will use the worst case, lowest value of λ_{lim}.

$\lambda = 20.87 < \lambda_{lim} = 34.5$ ✓ column is robust

6.3.3. Final design

Two different methods can be used for design of reinforcement. 1st is estimation with the presumption of uniformly distributed compression over the whole cross-section and 2nd is chart for design of symmetrical reinforcement.

1st method:

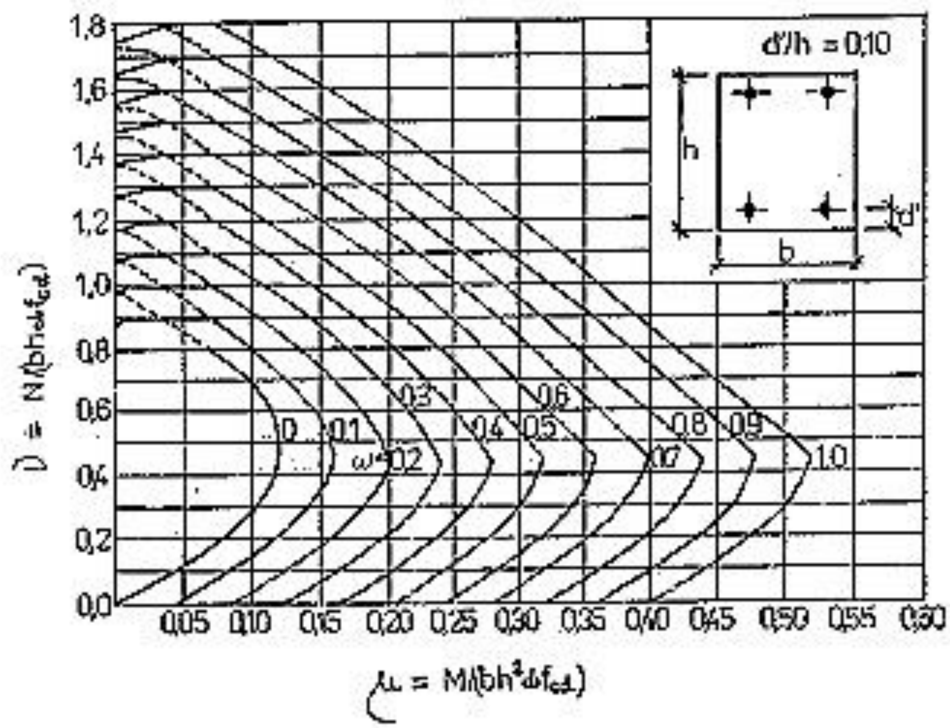
$$A_{s,req,1} = \frac{N_{Ed} - 0.8 \times A_c \times f_{cd}}{\sigma_s} = \frac{3276640 - 0.8 \times 400^2 \times 20}{400} = 1791.6mm^2$$

2nd method:

$\mu = \frac{M_{Ed,I}}{b \times h^2 \times f_{cd}}$ (relative bending moment)

$v = \frac{N_{Ed}}{b \times h \times f_{cd}}$ (relative normal force)

Through these values we can get ω coefficient from the chart below.



$$A_{s,req,2} = \frac{\omega \times A_c \times f_{cd}}{f_{yd}}$$

$$\mu_{head} = \frac{24.7 \times 10^6}{400^3 \times 20} = 0.019$$

$$\mu_{foot} = \frac{21.26 \times 10^6}{400^3 \times 20} = 0.017$$

$$v_{head} = \frac{3260.75 \times 10^3}{400^2 \times 20} = 1$$

$$v_{foot} = \frac{3276.64 \times 10^3}{400^2 \times 20} = 1$$

According to received values of μ and v, ω coefficient will be equal to 0. See the chart above. There is no need to check ULS (Z) because moments in this combination are smaller than moments in ULS (Y), which will give me smaller values of μ and v.

$$A_{s,req,2} = 0$$

$$A_{s,req} = max.\left(A_{s,req.1}; A_{s,req,2}\right) = max.(1791.6mm^2; 0) = 1791.6mm^2$$

$$A_{s,prov} \geq A_{s,req} = 1791.6mm^2$$

I will design $8 \times \emptyset 18 \rightarrow A_{s,prov} = 2036mm^2$

- *Check of detailing rules:*

$$A_{s,prov} \geq A_{s,min} = max.\left(0.1 \times \frac{N_{Ed}}{f_{yd}}; 0.002 \times A_c\right)$$

$$= max.\left(0.1 \times \frac{3276.64 \times 10^3}{435}; 0.002 \times 400^2\right) = max.(753.3; 320)$$

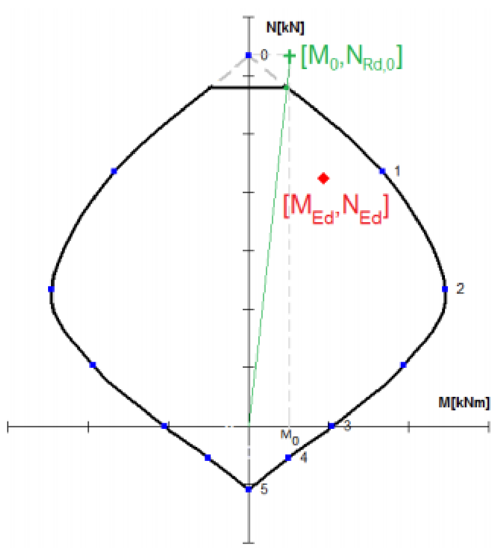
$$A_{s,prov} = 2036mm^2 > A_{s,min} = 753.3mm^2 \ \checkmark$$

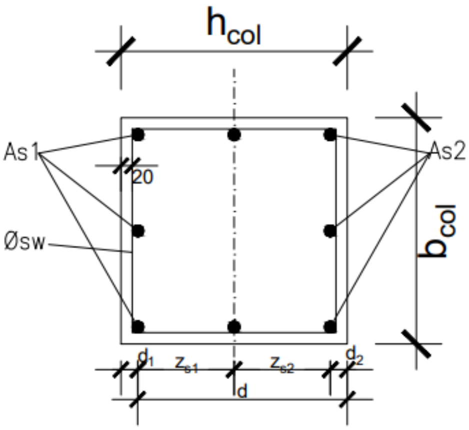
$$A_{s,prov} \leq A_{s,max} = 0.04 \times A_c = 0.04 \times 400^2 = 6400$$

$$A_{s,prov} = 2036mm^2 < A_{s,max} = 6400mm^2 \ \checkmark$$

6.3.4. Interaction diagram

Check of the column can be provided by illustration of the interaction between axial forces N and bending moments M acting in column cross-section at important points.





$b_{col} = h_{col} = 400\text{mm}$

$c = 20\text{mm}$ (cover depth)

$\emptyset_{sw} = 10\text{mm}$ (estimated diameter of stirrups)

$\emptyset_{s} = 18\text{mm}$ (main reinforcement)

$$d = h_{col} - c - \emptyset_{sw} - \frac{\emptyset_s}{2} = 400 - 20 - 10 - 9 = 361mm$$

$$z_{s1} = z_{s2} = \frac{1}{2} \times (h_{col} - 2c - 2\emptyset_{sw} - \emptyset_s) = \frac{1}{2} \times (400 - 2 \times 20 - 2 \times 10 - 18) = 161mm$$

$$d_1 = d_2 = \frac{h_{col}}{2} - z_{c1} = 200 - 161 = 39mm$$

$$A_{s1} = A_{s2} = 3 \times \emptyset 18mm = 763.41mm^2$$

$$A_s = 8 \times \emptyset 18mm = 2036mm^2$$

$f_{cd} = 20MPa$ (design compressive strength of the concrete)

$f_{yd} = 435MPa$ (design yield strength of the steel)

$A_c = 160000mm^2$ (area of the cross-section of the column)

$\sigma_s = 400MPa$ (stress in reinforcement; in my case, $f_{yd} \geq 400MPa$)

$\varepsilon_{cd} = 0.0035$ (limit strain of concrete)

$E_s = 210000MPa$ (Young's elastic modulus of steel)

- **Point 0 (pure compression)**

Resistance of normal force is maximum at this point.

$$N_{Rd,0} = F_c + F_{s1} + F_{s2} = b_{col} \times h_{col} \times f_{cd} + A_s \times \sigma_s$$

$$N_{Rd,0} = 400 \times 400 \times 20 + 2036 \times 400 = 4014.4kN$$

$$M_{Rd,0} = F_{s1} \times z_{s1} - F_{s2} \times z_{s2} = (A_{s1} \times z_{s1} - A_{s2} \times z_{s2}) \times \sigma_s$$

$$M_{Rd,0} = 0kNm$$

- **Point 1 (strain in tensile reinforcement is 0 ε_{s1}=0)**

Whole cross-section is compressed.

$$N_{Rd,1} = F_c + F_{s2} = 0.8 \times b_{col} \times d \times f_{cd} + A_{s2} \times f_{yd}$$

$$N_{Rd,0} = 0.8 \times 400 \times 361 \times 20 + 763.41 \times 435 = 2642.5kN$$

$$M_{Rd,1} = F_c \times z_c + F_{s2} \times z_{s2} = 0.8 \times b_{col} \times d \times f_{cd} \times \left(\frac{h_{col}}{2} - 0.4 \times d\right) + A_{s2} \times z_{s2} \times f_{yd}$$

$$M_{Rd,1} = 0.8 \times 400 \times 361 \times 20 \times \left(\frac{400}{2} - 0.4 \times 361\right) + 763.41 \times 161 \times 435 = 181.9kNm$$

0.8 – factor expressing the difference between real and idealized stress distribution

- **Point 2 (stress in tensile reinforcement is on yield limit σ_{s1}=f_{yd})**

Resistance of bending moment is maximum at this point.

$$\xi_{bal,1} = \frac{700}{700 + f_{yd}} = \frac{700}{700 + 435} = 0.617$$

$$x_{bal,1} = \xi_{bal,1} \times d = 0.617 \times 361 = 222.74mm$$

$$\frac{\varepsilon_{cd}}{x_{bal,1}} = \frac{\varepsilon_{s2}}{x_{bal,1} - d_2} \quad \rightarrow \quad \varepsilon_{s2} = \varepsilon_{cd} \times \left(1 - \frac{d_2}{x_{bal,1}}\right) = 0.0035 \times \left(1 - \frac{39}{222.74}\right) = 0.00289$$

$$\varepsilon_{yd} = \frac{f_{yd}}{E_s} = \frac{435}{210000} = 0.002071$$

$\varepsilon_{s2} = 0.00289 > \varepsilon_{yd} = 0.002071$, we assume σ_{s2} = f_{yd} = 435MPa (stress in compressed reinforcement)

$$N_{Rd,2} = F_c + F_{s2} - F_{s1} = 0.8 \times b_{col} \times x_{bal,1} \times f_{cd} + A_{s2} \times \sigma_{s2} - A_{s1} \times f_{yd}$$

$$N_{Rd,2} = 0.8 \times 400 \times 22.74 \times 20 + 763.41 \times 435 - 763.41 \times 435 = 1425.5kN$$

$$\begin{aligned} M_{Rd,2} &= F_c \times z_c + F_{s2} \times z_{s2} + F_{s1} \times z_{s1} \\ &= 0.8 \times b_{col} \times x_{bal,1} \times f_{cd} \times \left(\frac{h_{col}}{2} - 0.4 \times x_{bal,1}\right) + A_{s2} \times \sigma_{s2} \times z_{s2} + A_{s1} \times f_{yd} \times z_{s1} \end{aligned}$$

$$\begin{aligned} M_{Rd,2} &= 0.8 \times 400 \times 222.74 \times 20 \times \left(\frac{400}{2} - 0.4 \times 222.74\right) + 763.41 \times 435 \times 161 + 763.41 \\ &\quad \times 435 \times 161 = 265kNm \end{aligned}$$

- **Point 3 (pure bending)**

Normal force is equal to 0.

$N_{Rd,3} = F_c + F_{s1} - F_{s2} = 0kN$

$$M_{Rd,3} = F_c \times z_c + F_{s2} \times z_{s2} + F_{s1} \times z_{s1}$$
$$= 0.8 \times b_{col} \times x \times f_{cd} \times \left(\frac{h_{col}}{2} - 0.4 \times x\right) + A_{s2} \times \sigma_{s2} \times z_{s2} + A_{s1} \times f_{yd} \times z_{s1}$$

We can find σ_{s2} through derivation of quadratic equation:

$${\sigma_{s2}}^2 \times A_{s2} - \sigma_{s2} \times \left(A_{s1} \times f_{yd} + A_{s2} \times \varepsilon_{cd} \times E_s\right) + \varepsilon_{cd} \times E_s \times \left(A_{s1} \times f_{yd} - 0.8 \times b_{col} \times f_{cd} \times d_2\right) = 0$$

$${\sigma_{s2}}^2 \times 763.41 - \sigma_{s2} \times (763.41 \times 435 + 763.41 \times 0.0035 \times 210000) + 0.0035 \times 210000$$
$$\times (763.41 \times 435 - 0.8 \times 400 \times 20 \times 39) = 0$$

$${\sigma_{s2}}^2 \times 763.41 - \sigma_{s2} \times 893189.7 + 60625262.25 = 0$$

$\sigma_{s2,1} = 72.35MPa$ (stress in compressed reinforcement)

$\sigma_{s2,2} = 1097.7MPa > f_{yk} = 500MPa$ so I took σ_{s2} = 72.35MPa.

$x = \frac{A_{s1} \times f_{yd} - A_{s2} \times \sigma_{s2}}{0.8 \times b_{col} \times f_{cd}} = \frac{763.41 \times 435 - 763.41 \times 72.35}{0.8 \times 400 \times 20} = 43.3mm$ (height of compressed zone)

$$M_{Rd,3} = 0.8 \times 400 \times 43.3 \times 20 \times \left(\frac{400}{2} - 0.4 \times 43.3\right) + 763.41 \times 72.35 \times 161 + 763.41 \times 72.35 \times 161$$

$M_{Rd,3} = 113kNm$

- **Point 4 (strain in compressed reinforcement is 0 ε_{s2}=0)**

Whole cross-section is in tension.

$N_{Rd,4} = F_{s1} = A_{s1} \times f_{yd} = 763.41 \times 435 = 561.1kN$

$M_{Rd,4} = F_{s1} \times z_{s1} = 561.1 \times 161 = 53.5kNm$

- **Point 5 (pure tension)**

Ending moment is equal to 0.

$N_{Rd,5} = F_{s1} + F_{s2} = A_s \times f_{yd} = 2036 \times 435 = 885.7kN$

$M_{Rd,5} = F_{s1} \times z_{s1} - F_{s2} \times z_{s2} = 0kNm$

Minimum eccentricity has to be considered:

$$e_0 = max.\left(\frac{h_{col}}{30}; 20mm\right) = max.(13.333mm; 20mm) = 20mm$$

And minimum bending moment has to be calculated:

$$M_0 = N_{Rd,0} \times e_0 = 4014.4 \times 0.02 = 80.3kNm$$

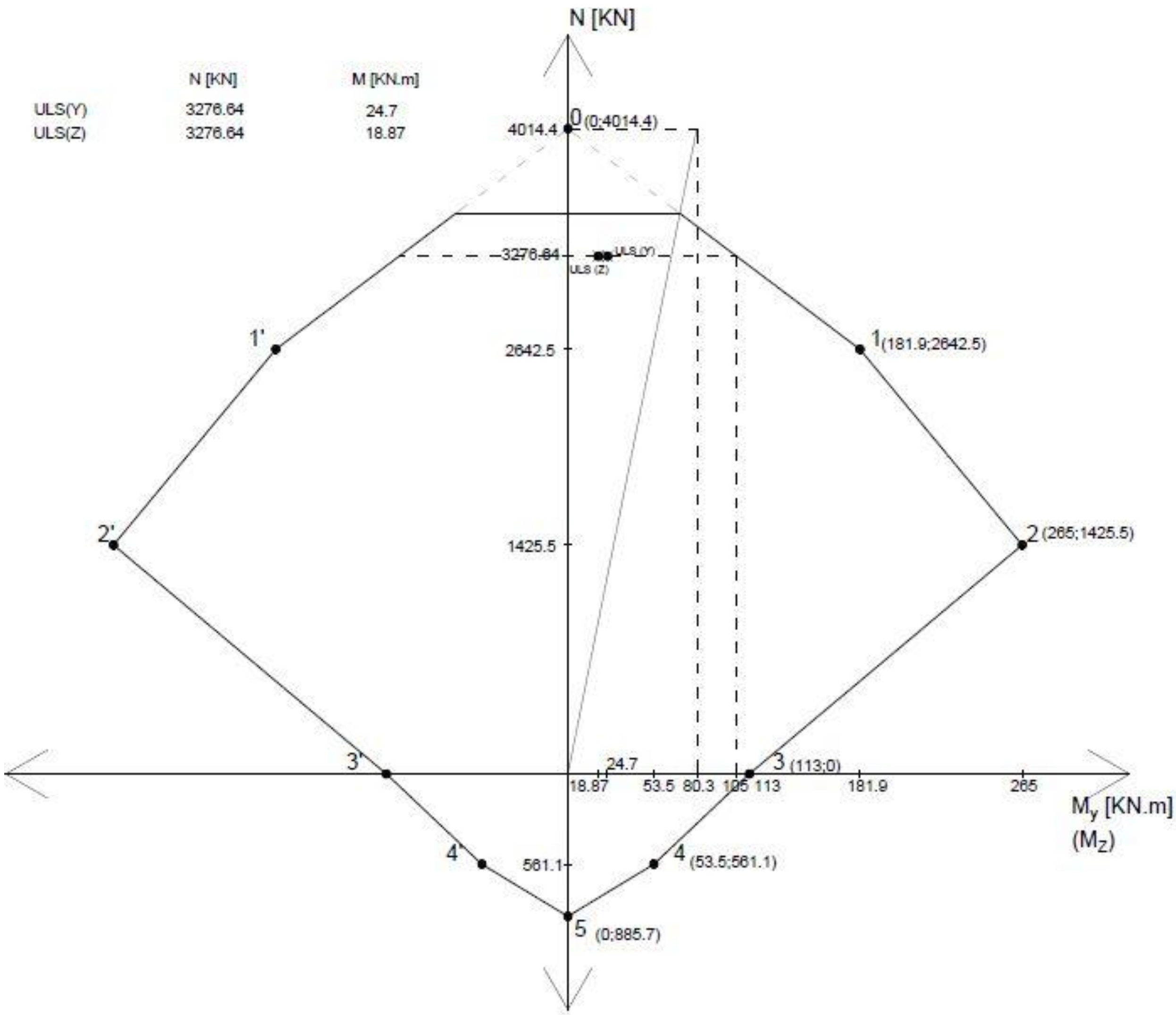


Figure 6. Interaction diagram (due to the symmetry of reinforcement, the shape is the same for both y and z directions)

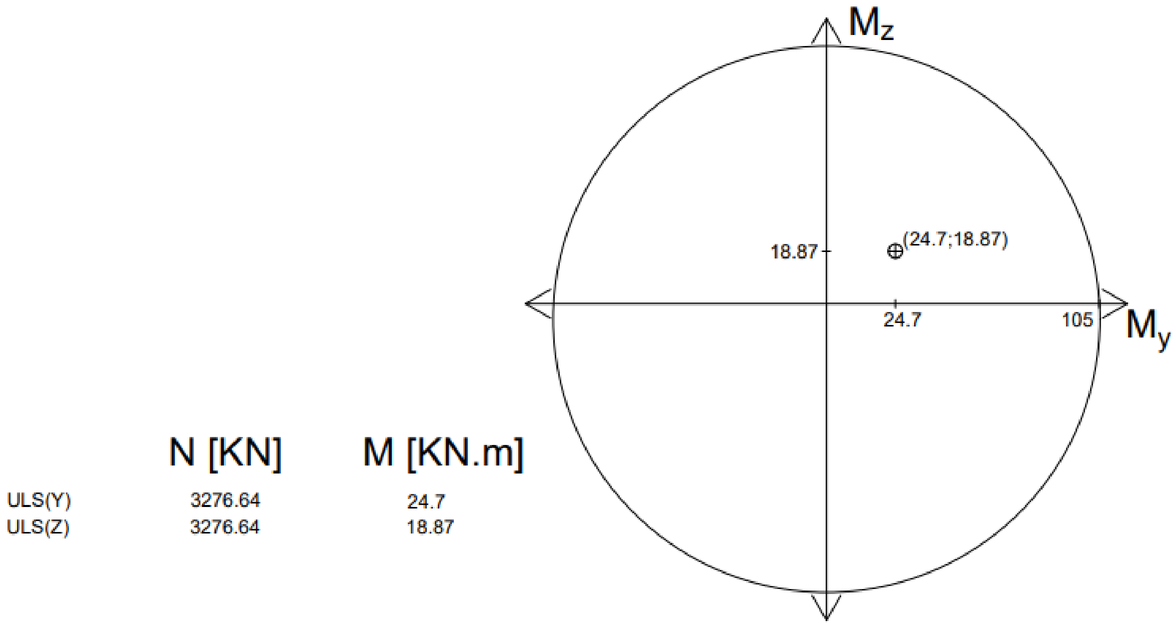


Figure 7. Horizontal section of 3D interaction diagram in the level of ULS(Y)

The points representing the actual load of the column are located inside the diagram, which means the column is satisfying.

6.3.5. Column ties (stirrups) and lapping length

We have to design column tie, which helps to avoid buckling of reinforcement.

$$\emptyset_{tie} \geq max.\left(\frac{\emptyset_s}{4}; 6mm\right) = max.\left(\frac{18}{4}; 6mm\right) = max.(4.5mm; 6mm) = 6mm$$

$$\emptyset_{tie} = 10mm$$

To close the tie or stirrup the length of ends can be considered as $10\emptyset$.

- *Basic spacing:*

$$s_1 \leq min.(20\emptyset; min.(b_{col}; h_{col}); 400mm) = min.(360; 400; 400mm) = 360mm$$

$$s_1 = 350mm$$

- *Spacing in lapping area:*

$$s_2 \leq 0.6 \times s_1 = 0.6 \times 350 = 210mm$$

$$s_2 = 200mm$$

We also use s_2 below the slab in distance of $max.(b_{col}; h_{col}) = 400mm$.

Calculation of the lapping length is the same as for wall.

$$l_{0,d} = \alpha_1 \times \alpha_2 \times \alpha_3 \times \alpha_5 \times \alpha_6 \times l_{b,req} \geq l_{0,min} = max.(0.3 \times \alpha_6 \times l_{b,req}; 15\emptyset; 200mm)$$

Basic anchorage length:

$$l_{b,req} = \frac{\emptyset}{4} \times \frac{\sigma_{sd}}{f_{bd}} = \frac{18}{4} \times \frac{435}{3} = 652.5mm$$

$$l_{0,d} = 1 \times 1 \times 1 \times 1 \times 1.5 \times 652.5 \geq l_{0,min} = max.(0.3 \times 1.5 \times 652.5; 15 \times 18; 200mm)$$

$$l_{0,d} = 978.75mm \geq l_{0,min} = max.(293.625; 270; 200mm) = 293.625mm$$

$l_{0,d} = 980mm$ (design lapping length of column reinforcement) ✓

6.4. Pillar

Since pillar behaves like column, slenderness of the pillar must be checked before designing the reinforcement.

- *Slenderness of the pillar:*

$$\lambda = \frac{l}{i}$$

$I = \frac{b_p \times {h_p}^3}{12} = \frac{3 \times 0.25^3}{12} = 0.0039m^4$ (moment of inertia)

$i = \sqrt{\frac{I}{A_c}} = \sqrt{\frac{0.0039}{0.25 \times 3}} = 0.072m$ (radius of gyration)

$\lambda = \frac{0.8 \times H_c}{i} = \frac{0.8 \times 3}{0.072} = 33.3$ (slenderness of the pillar)

$$\lambda_{lim} = \frac{20 \times A \times B \times C}{\sqrt{n}} \leq 75$$

$A = 0.7$ (effect of creep)

$B = 1.1$ (effect of reinforcement ratio)

$C = 1.7 - r_m$ (effect of bending moments)

$$r_m = \frac{M_{01}}{M_{02}}$$

For loads I took the resultant of reactions acting on the bottom of the pillar and bending moments on the head and foot of the pillar from SCIA Engineer.

$$r_m = \frac{2.97}{34.96} = 0.085$$

$$C = 1.7 - 0.085 = 1.615$$

$n = \frac{N_{Ed}}{A_c \times f_{cd}} = \frac{3559670}{250 \times 3000 \times 20} = 0.2373$ (relative normal force)

$$\lambda_{lim} = \frac{20 \times 0.7 \times 1.1 \times 1.615}{\sqrt{0.2373}} \leq 75$$

$\lambda_{lim} = 51.1 \leq 75$ ✓

$\lambda = 33.3 < \lambda_{lim} = 51.1$ ✓

To design reinforcement in edges of the pillar, following estimation of uniformly distributed compression over the whole cross-section can be checked.

$$A_{s,req} = \frac{N_{Ed,p} - 0.8 \times A_c \times f_{cd}}{\sigma_s}$$

$$A_{s,req} = \frac{3559.6 \times 10^3 - 0.8 \times 250 \times 1000 \times 20}{400} = -1100.83mm^2$$

$A_{s,req} < 0$, so just the reinforcement according to the detailing rules is needed. The same reinforcement as for walls will be used.

7. DESIGN OF STAIRCASE

7.1. Geometry of the staircase

- Construction height of the floor h_k = 3380 mm
- Thickness of the main slab h_s = 275 mm
- Thickness of the floor structure h_f = 110 mm
- Thickness of cladding of the stairs h_c = 20 mm

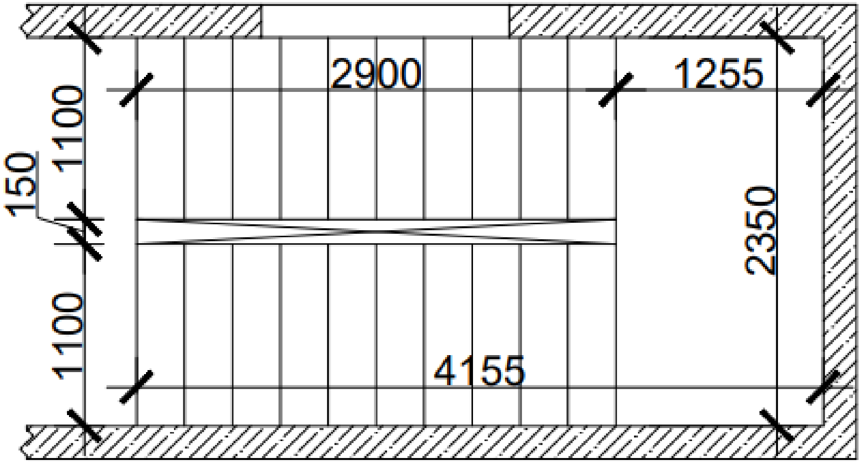
Ideal height of one step is 170mm.

$n = \frac{3380mm}{170mm} = 19.88$ → 20 steps (2 flights, 10 steps)

- Height of one step $h = \frac{3380}{20} = 169mm$
- Width of one step $b = 630 - 2 \times h = 630 - 2 \times 169 = 292mm$
- Slope of the staircase $\alpha = \arctan\left(\frac{h}{b}\right)$

DESIGN: 2 flights, 10 steps in each flight, h**=169mm, b=290mm, α=30.2°**

- Width of flight – 1100mm, length of the flight – 2900mm
- Width of the gap between flights – 150mm
- Width of the landing – 1275mm, length of the landing – 1255mm
- Width of the staircase – 1100mm*2+150mm = 2350mm

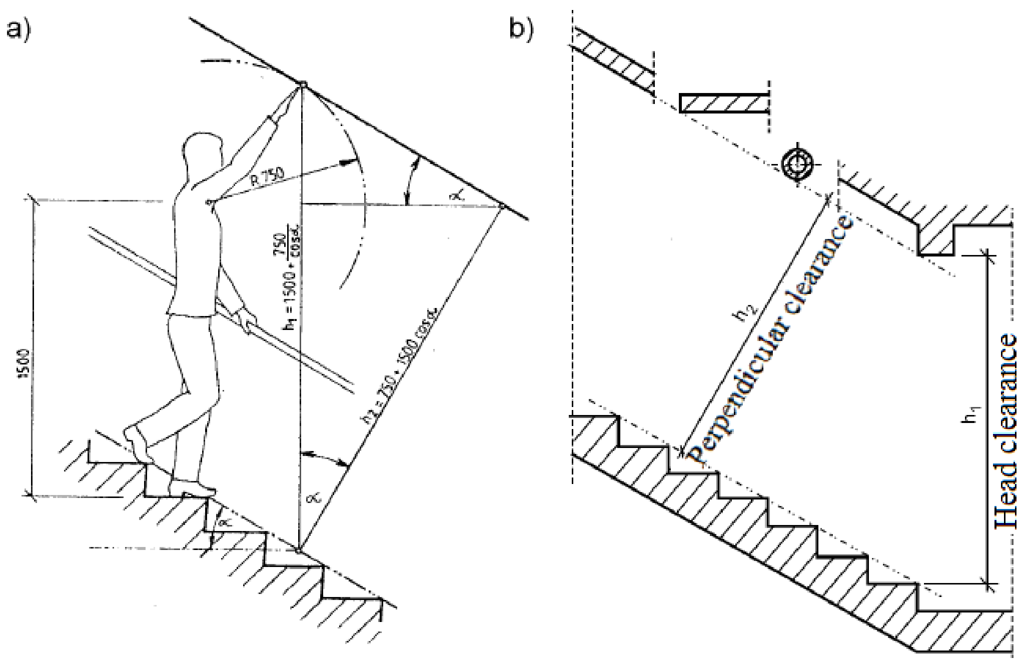


Perpendicular and head clearance of the staircase:

- Head clearance has to be more than $1500 + \frac{750}{\cos(30.2^o)} = 2368\, mm > 2100mm$

  $h_1 = h_k - h_s - h_f - h = 3380 - 275 - 110 - 169 = 2826\, mm$ ✓
- Perpendicular clearance has to be more than $750 + 1500 \times cos(30.2°) = 2046\, mm > 1900mm$

  $h_2 = h_1 \times cos\alpha = 2826 \times cos(30.2°) = 2442mm$ ✓



Preliminary check of the depth of the slab:

- The staircase is considered as one-way simply supported slab with the span of 4155mm.
- The depth should be at least 4155mm/25 = 166.2mm.
- The depth of landing is same as the main slab thickness – 275mm.
- The depth of flight is 230mm.

  $275\, mm > 180\, mm\ and\ 230mm > 180\, mm$ ✓

7.2. Calculation of loads

Landing:

Type	F_k[kN/m^2]	γ_F	F_d[kN/m^2]
Slab	0.275*25=6.875	1.35	9.28125
Floor	1	1.35	1.35
Live load	3.5	1.5	5.25
Total			$\sum$ = 15.881kN/m^2

Flight:

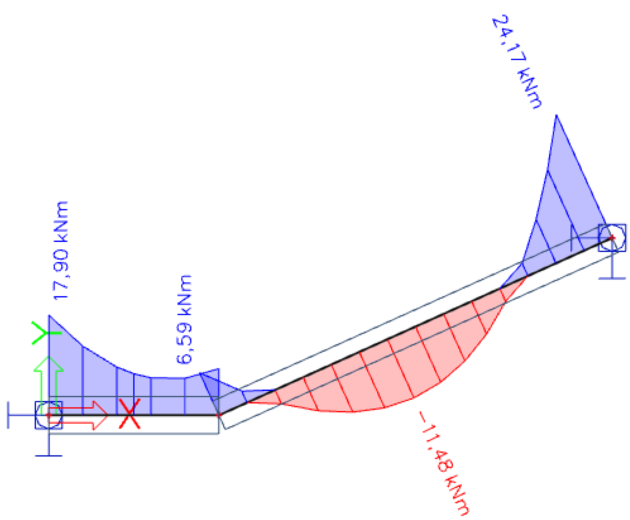
Type	F_k[kN/m^2]	γ_F	F_d[kN/m^2]
Slab	$\frac{0.23}{\cos(30.2^o)} \times 25 = 6.7$	1.35	9.045
Cladding	$0.5 \times \frac{169+290}{290} = 0.8$	1.35	1.08
Steps	$\frac{0.169}{2} \times 25 = 2.1125$	1.35	2.851875
Live load	3.5	1.5	5.25
Total			$\sum$ = 18.227kN/m^2

$$f_{d,l} = \frac{15.881kN}{m^2} \times 1.275m = 20.3kN/m$$

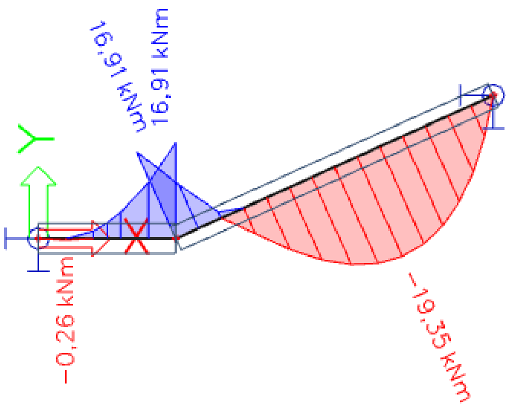
$$f_{d,f} = 18.227kN/m^2 \times \cos(30.2^o) \times 1.1m = 17.33kN/m$$

I designed the staircase in SCIA Engineer to get real bending moments acting on the structure. I modeled landing and flight as beams with all designed cross-section dimensions with two types supports (first fixed, the hinged).

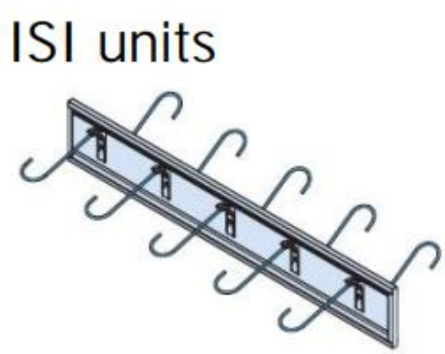
Fixed supports were added to get moments in the supports.

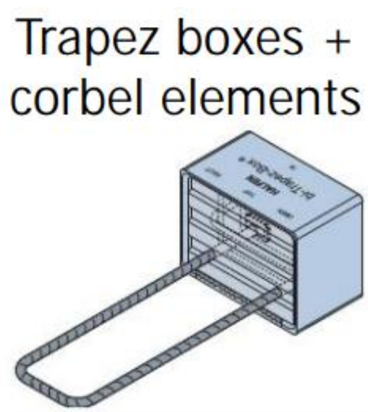


Then I changed supports to hinged to receive moments in midspans.



In the structure staircase will be supported by ISI units, trapez boxes and corbel elements, which is needed for sound isolation.





7.3. Design of reinforcement

Design procedure is the same as for one-way slab.

- ***Landing (in supports)***

$\emptyset = 10mm$ (assumption)

h_s = 275mm

$$d = 275 - 20 - \frac{10}{2} = 250mm$$

$$\mu = \frac{M_{Ed}}{b \times d^2 \times f_{cd}} = \frac{17.9 \times 10^6}{1000 \times 250^2 \times 20} = 0.014 \ \rightarrow \ \zeta = 0.995$$

$$a_{s,req} = \frac{M_{Ed}}{\zeta \times d \times f_{yd}} = \frac{17.9 \times 10^6}{0.995 \times 250 \times 435} = 165.4mm^2$$

DESIGN: $5 \times \emptyset 10mm \rightarrow a_{s,prov} = 393mm^2$

Detailing rules:

- $a_{s,prov} \geq a_{s,min,1} = max.\left(0.26 \times \frac{f_{ctm}}{f_{yk}} \times b \times d; 0.0013 \times b \times d\right) = max.(0.26 \times \frac{2.9}{500} \times 1000 \times 250; 0.0013 \times 1000 \times 250)$

$a_{s,prov} \geq a_{s,min,1} = max.(377mm^2; 325mm^2) = 377mm^2$

$a_{s,prov} = 393mm^2 \geq a_{s,min.1} = 377mm^2$ ✓

- $a_{s,prov} \geq a_{s,min,2} = \frac{k_c \times k \times f_{ct,eff} \times A_{ct}}{\sigma_s} = \frac{0.4 \times 1 \times 2.9 \times 0.5 \times 1000 \times 250}{500} = 290mm^2$

$a_{s,prov} = 393mm^2 \geq a_{s,min,2} = 290mm^2$ ✓

- Spacing:
  $s_{max} \leq min.(2 \times h; 250) = 250mm$
  $s_a = \frac{b - 2 \times c - 5 \times \emptyset}{4} = \frac{1255 - 2 \times 20 - 5 \times 10}{4} = 296.3mm$
  $s_a > s_{max}$, which means we have to add more bars

NEW DESIGN: $7 \times \emptyset 10mm \rightarrow a_{s,prov} = 550mm^2,\ s = 190\ mm$

Check:

$$x = \frac{a_{s,prov} \times f_{yd}}{0.8 \times b \times f_{cd}} = \frac{550 \times 435}{0.8 \times 1000 \times 20} = 15mm$$

$z = d - 0.4 \times x = 250 - 0.4 \times 15 = 244mm$

$m_{Rd} = a_{s,prov} \times f_{yd} \times z = 393 \times 435 \times 244 = 42kNm$

$m_{Rd} = 42kNm \geq m_{Ed} = 17.9Nm$ ✓

- ***Flight (in supports)***

$\emptyset = 10mm$ (assumption)

h_s = 230mm

$$d = 230 - 20 - \frac{10}{2} = 205mm$$

$$\mu = \frac{M_{Ed}}{b \times d^2 \times f_{cd}} = \frac{24.17 \times 10^6}{1100 \times 205^2 \times 20} = 0.03 \ \rightarrow \ \zeta = 0.985$$

$$a_{s,req} = \frac{M_{Ed}}{\zeta \times d \times f_{yd}} = \frac{24.17 \times 10^6}{0.985 \times 205 \times 435} = 275.2mm^2$$

DESIGN: $5 \times \emptyset 10mm \ \rightarrow \ a_{s,prov} = 393mm^2$

Detailing rules:

- $a_{s,prov} \geq a_{s,min,1} = max.\left(0.26 \times \frac{f_{ctm}}{f_{yk}} \times b \times d; 0.0013 \times b \times d\right) = max.(0.26 \times \frac{2.9}{500} \times 1100 \times 205; 0.0013 \times 1100 \times 205)$

$a_{s,prov} \geq a_{s,min,1} = max.(340.1mm^2; 293.2mm^2) = 340.1mm^2$

$a_{s,prov} = 393mm^2 \geq a_{s,min.1} = 340.1mm^2$ ✓

- $a_{s,prov} \ \geq a_{s,min,2} \ = \ \frac{k_c \times k \times f_{ct,eff} \times A_{ct}}{\sigma_s} = \frac{0.4 \times 1 \times 2.9 \times 0.5 \times 1100 \times 205}{500} = 261.6mm^2$

$a_{s,prov} = 393mm^2 \geq a_{s,min,2} = 261.6mm^2$ ✓

- Spacing:
  $s_{max} \leq min.(2 \times h; 250) = 250mm$
  $s_a = \dfrac{b - 2 \times c - 5 \times \emptyset}{4} = \dfrac{1100 - 2 \times 20 - 5 \times 10}{4} = 252.5mm$
  $s_a > s_{max}$, which means we have to add more bars

NEW DESIGN: $6 \times \emptyset 10mm \ \rightarrow \ a_{s,prov} = 468 \ mm^2, s = 190 \ mm$

Check:

$$x = \frac{a_{s,prov} \times f_{yd}}{0.8 \times b \times f_{cd}} = \frac{468 \times 435}{0.8 \times 1100 \times 20} = 11.6mm$$

$$z = d - 0.4 \times x = 205 - 0.4 \times 11.6 = 200.4mm$$

$$m_{Rd} = a_{s,prov} \times f_{yd} \times z = 468 \times 435 \times 200.4 = 40.8kNm$$

$m_{Rd} = 40.8Nm \geq m_{Ed} = 24.17kNm$ ✓

- ***Flight (in midspan)***

$\emptyset = 10mm$ (assumption)

h_s = 230mm

$$d = 230 - 20 - \frac{10}{2} = 205mm$$

$$\mu = \frac{M_{Ed}}{b \times d^2 \times f_{cd}} = \frac{19.35 \times 10^6}{1100 \times 205^2 \times 20} = 0.02 \ \rightarrow \ \zeta = 0.99$$

$$a_{s,req} = \frac{M_{Ed}}{\zeta \times d \times f_{yd}} = \frac{19.35 \times 10^6}{0.99 \times 205 \times 435} = 219.2mm^2$$

DESIGN: $6 \times \emptyset 10mm \ \rightarrow \ a_{s,prov} = 468 \ mm^2$

Detailing rules:

- $a_{s,prov} \geq a_{s,min,1} = max.\left(0.26 \times \frac{f_{ctm}}{f_{yk}} \times b \times d; 0.0013 \times b \times d\right) = max.(0.26 \times \frac{2.9}{500} \times 1100 \times 205; 0.0013 \times 1100 \times 205)$

$a_{s,prov} \geq a_{s,min,1} = max.(340.1mm^2; 293.2mm^2) = 340.1mm^2$

$a_{s,prov} = 468mm^2 \geq a_{s,min.1} = 340.1mm^2$ ✓

- $a_{s,prov} \ \geq a_{s,min,2} \ = \ \frac{k_c \times k \times f_{ct,eff} \times A_{ct}}{\sigma_s} = \ \frac{0.4 \times 1 \times 2.9 \times 0.5 \times 1100 \times 205}{500} = 261.6mm^2$

$a_{s,prov} = 468mm^2 \geq a_{s,min,2} = 261.6mm^2$ ✓

- Spacing:
  $s_{max} \leq min.(2 \times h; 250) = 250mm$
  $$s_a = \frac{b - 2 \times c - 5 \times \emptyset}{4} = \frac{1100 - 2 \times 20 - 6 \times 10}{5} = 200mm$$
  $s = 190 \ mm$

Check:

$$x = \frac{a_{s,prov} \times f_{yd}}{0.8 \times b \times f_{cd}} = \frac{468 \times 435}{0.8 \times 1100 \times 20} = 11.6mm$$

$$z = d - 0.4 \times x = 205 - 0.4 \times 11.6 = 200.4mm$$

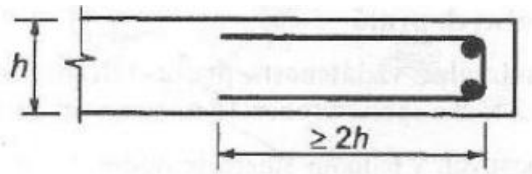
$$m_{Rd} = a_{s,prov} \times f_{yd} \times z = 468 \times 435 \times 200.4 = 40.8kNm$$

$m_{Rd} = 40.8Nm \geq m_{Ed} = 19.35kNm$ ✓.

In the landing moment in midspan is very small, so I will design the same reinforcement as for the flight in midspan.

DESIGN: $6 \times \emptyset 10mm \rightarrow a_{s,prov} = 468\ mm^2, s = 190\ mm$

- ***Edge reinforcement:***



$7 \times \emptyset 10mm$ for the landing

$6\times \emptyset 10mm$ for the flight

- ***Transverse reinforcement:***

$a_{s,tr} \geq 0.25 \times a_{s,main} = 0.25 \times 550 = 137.5mm^2$

$s_{tr} \leq min. (3 \times h; 400) = 400mm$

$7 \times \emptyset 8mm \rightarrow a_{s,prov} = 350mm^2$ for the landing

$6 \times \emptyset 8mm \rightarrow a_{s,prov} = 300mm^2$ for the flight

$s_{tr} = 190\ mm$ for the landing (the same as the main reinforcement)

$s_{tr} = 200\ mm$ for the flight (the same as the main reinforcement)

- ***Secondary reinforcement of the upper surface:***

The same as the transverse reinforcement.

$7 \times \emptyset 8mm \rightarrow a_{s,prov} = 350mm^2$ for the landing

$6 \times \emptyset 8mm \rightarrow a_{s,prov} = 300mm^2$ for the flight

- ***End stirrups***:

Design is according to the manufacturer of sound insulation elements, i.e. 2x Ø8

8. FOUNDATION

This part of the thesis was made under supervision of ***Ing. Jan Salak, CSc.*** *in Department of Geotechnics.*

Before designing the foundation of the building, different surveying procedures must be done to inspect the soil properties of the area that were used for design.

8.1. Characteristic of soil

- Type of soil: sandy loam and loamy sand F3-S4
- Design load-bearing capacity from table: R_{dt} = 225kPa
- Volume density: 1800kg/m^3
- Effective cohesion: c_{ef} = 10kPa
- Effective angle of internal friction: $\Phi_{ef} = 25^o$
- Suitability for fillings: suitable to very suitable

8.2. Calculation of foundation dimensions

Preliminary design of foundations for each vertical load-bearing element is provided in following figures below. Loads acting on each structure were taken from SCIA Engineer.

8.3.Foundation dimensions

- Foundation strip dimensions for perimeter walls: B=1.6m, L=3.5m, H=0.86m.
- Foundation strip dimensions for walls around staircase and elevator: B=1m, L=1m, H=0.86m.
- Foundation pad dimensions for columns: B=2.4m, L=2.5m, H=1m.
- Foundation pad dimensions for pillar: B=1.6m, L=4.8m, H=1m.

Type of soil Sandy loam and loamy sand F3-S4

Preliminary load bearing capacity Rdt = 225 kPa

Character of soil		
cef	10	effective cohession
φef	25	effective angle of friction
cd	8	c'/Yc
φd	20.5	arctg(tg(φ')/1.25)
Y	18	
Yd	18	

Partial safety factors	
cd=ce/Yc	
Yc	1.25
Yφ	1.25
YY	1

Load on the foundation pad

Vdn	negative effect		3931.97

Vd = V + Vf

V = 3276.64 kN

Vf = 0.2*V = 655.33 kN

Preliminary design of foundation pad

A = 17.48 m2 A=Vdn/Rdt

side of foundation pa 4.18 m

Coefficient calculation

Foundation pad dime	
b	2.4
l	2.5
h	1
d	0

load bearing coefficient		
Nc	52.95	φd>0
Nd	20.79	
Nb	11.09	
shape of foundation pad coefficient		
sc	1.192	
sd	1.03	
sb	0.712	
depth of foundation coefficient		
dc	1.00	
dd	1.00	
db	1	
coefficient of slope of force		
ic	1	
id	1	
ib	1	

for φd=0 Nc = 5,14 else Nc = (Nd-1)*cotg(φd)

Nd=tg^2(45+φd/2)*e^(pi*tgφd)

Nb=1,5*(Nd-1)*tg(φd)

S_c = 1+0,2*b/l

S_d = 1+0,1*b/l*sinφd

S_b = 1-0,3*b/l

d = depth of foundation

d_c = 1+0,1*√(d/b)

d_d = 1+0,1*√(d/b*sin(2φd))

d_b = 1

$i_c = i_d = i_b = (1\text{-}tg\delta)^2 = (1\text{-}H/V)^2$

R/A 675.51 kPa load bearing capacity of soil

Stress below foundation pad

σd 655 kPa

σd<R/A 655 < 676 **ok**

Figure 8. Calculation of column foundation

Type of soil	Sandy loam and loamy sand	Preliminary load breaing capacity	
	F3-S4	Rdt =	225 kPa

Character of soil		
cef	10	effective cohession
φef	25	effective angle of friction
cd	8	c'/Yc
φd	20.5	arctg(tg(φ')/1.25)
Y	18	
Yd	18	

Partial safety factors	
cd=ce/Yc	
Yc	1.25
Yφ	1.25
YY	1

Load on the foundation pad

Vdn	negative effect		4271.6

Vd = V + Vf

V = 3559.67 kN

Vf = 0.2*V = 711.934 kN

Preliminary design of foundation pad

A = 18.98 m2 A=Vdn/Rdt

side of foundation pa 4.36 m

Coefficient calculation

Foundation pad dime	
b	1.6
l	4.8
h	1
d	0

load bearing coefficient		
Nc	52.95	φd>0
Nd	20.79	
Nb	11.09	
shape of foundation pad coefficient		
sc	1.06667	
sd	1.01	
sb	0.9	
depth of foundation coefficient		
dc	1.00	
dd	1.00	
db	1	
coefficient of slope of force		
ic	1	
id	1	
ib	1	

for φd=0 Nc = 5,14 else Nc = (Nd-1)*cotg(φd)

Nd=tg^2(45+φd/2)*e^(pi*tgφd)

Nb=1,5*(Nd-1)*tg(φd)

S_c = 1+0,2*b/l

S_d = 1+0,1*b/l*sinφd

S_b = 1-0,3*b/l

d = depth of foundation

$d_c = 1+0,1*\sqrt{(d/b)}$

$d_d = 1+0,1*\sqrt{(d/b*\sin(2\varphi d))}$

$d_b = 1$

$i_c = i_d = i_b = (1-tg\delta)^2 = (1-H/V)^2$

R/A 595.59 kPa load bearing capacity of soil

Stress below foundation pad

σd 556 kPa

σd<R/A 556 < 596 **ok**

Figure 9. Calculation of pillar foundation

Type of soil Sandy loam and loamy sand F3-S4

Preliminary load breaing capacity
Rdt = 225 kPa

Character of soil		
cef	10	effective cohession
φef	25	effective angle of friction
cd	8	c'/Yc
φd	20.5	arctg(tg(φ')/1.25)
Y	18	
Yd	18	

Partial safety factors	
cd=ce/Yc	
Yc	1.25
Yφ	1.25
YY	1

Load on the foundation strip

Vdn	negative effect		3333.11

Vd = V + Vf
V = 2777.59 kN
Vf = 0.2*V = 555.52 kN

Preliminary design of foundation strip

A 14.81 m2 A=Vdn/Rdt
side of foundation st 3.85 m

Coefficient calculation

Foundation strip dim	
b	1.6
l	3.5
h	0.86
d	0

load bearing coefficient		
Nc	52.95	φd>0
Nd	20.79	
Nb	11.09	
shape of foundation strip coefficient		
sc	1.09143	
sd	1.02	
sb	0.86286	
depth of foundation coefficient		
dc	1.00	
dd	1.00	
db	1	
coefficient of slope of force		
ic	1	
id	1	
ib	1	

for φd=0 Nc = 5,14 else Nc = (Nd-1)*cotg(φd)
Nd=tg^2(45+φd/2)*e^(pi*tgφd)
Nb=1,5*(Nd-1)*tg(φd)

S_c = 1+0,2*b/l
S_d = 1+0,1*b/l*sinφd
S_b = 1-0,3*b/l
d = depth of foundation

d_c = 1+0,1*√(d/b)
d_d = 1+0,1*√(d/b*sin(2φd))
d_b = 1

$i_c = i_d = i_b = (1-tg\delta)^2 = (1-H/V)^2$

R/A 600.15 kPa load bearing capacity of soil

stress **below foundation** strip
σd 595 kPa

σd<R/A 595 < 600 **ok**

Figure 10. Calculation of wall foundation

Type of soil	Sandy loam and loamy sand	Preliminary load breaing capacity	
	F3-S4	Rdt =	225 kPa

Character of soil		
cef	10	effective cohession
φef	25	effective angle of friction
cd	8	c'/Yc
φd	20.5	arctg(tg(φ')/1.25)
Y	18	
Yd	18	

Partial safety factors	
cd=ce/Yc	
Yc	1.25
Yφ	1.25
YY	1

Load on the foundation strip

Vdn	negative effect		437.99

Vd = V + Vf
V = 364.99 kN
Vf = 0.2*V = 73 kN

Preliminary design of foundation strip

A 1.95 m2 A=Vdn/Rdt
side of foundation st 1.40 m

Coefficient calculation

Foundation strip dim	
b	1
l	1
h	0.86
d	0

load bearing coefficient		
Nc	52.95	φd>0
Nd	20.79	
Nb	11.09	
shape of foundation strip coefficient		
sc	1.2	
sd	1.04	
sb	0.7	
depth of foundation coefficient		
dc	1.00	
dd	1.00	
db	1	
coefficient of slope of force		
ic	1	
id	1	
ib	1	

for φd=0 Nc = 5,14 else Nc = (Nd-1)*cotg(φd)
Nd=tg^2(45+φd/2)*e^(pi*tgφd)
Nb=1,5*(Nd-1)*tg(φd)

S_c = 1+0,2*b/l
S_d = 1+0,1*b/l*sinφd
S_b = 1-0,3*b/l
d = depth of foundation

d_c = 1+0,1*√(d/b)
d_d = 1+0,1*√(d/b*sin(2φd))
d_b = 1

$i_c = i_d = i_b = (1-tg\delta)^2 = (1-H/V)^2$

R/A 578.21 kPa load bearing capacity of soil

stress below foundation strip
σd 438 kPa

σd<R/A 438 < 578 ok

Figure 11. Calculation of foundation of communication area wall

9. STANDARDS

- **Eurocodes:**

- EN 1990 – Basis of structural design
- EN 1991 – Actions on structures
- EN 1992 – Design of concrete structures
- EN 1996 – Design of masonry structures
- EN 1997 – Geotechnical design

10. SOFTWARE

- AutoCAD 2018
- SCIA Engineer 18.1
- MS Office 2007

11. LIST OF DRAWINGS

- Structural systems (1:250)
- Ground floor plan (1:50)
- Typical floor plan (1:50)
- Section A-A’ (1:50)
- Section B-B’ (1:50)
- Detail A – Attic (1:10)
- Detail B – Staircase (1:10)
- Detail C – Window frame (1:5)
- Structural plan – Formwork (1:50)
- Flat slab upper reinforcement (1:50)
- Flat slab bottom reinforcement (1:50)
- Column reinforcement (1:25)
- Wall reinforcement (1:50)
- Staircase reinforcement (1:25)
- Foundation plan (1:50)

12. REFERENCES

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- "Concrete structures 1" Prof. Ing. Jaroslav Procházka, CSc., Ing. Petr Štemberk, Ph.D.
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