Politecnico di Milano – Scuola di Architettura e Società

BUILDING TECHNOLOGY STUDIO B

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Structural report

Academic year 2014-2015

Group 11

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Content:

5. Foundations

I. Introduction

Project Name : Smart Home Milano / San Cristoforo /
Project location : Milano / Italy /
Latitude : 45.4667° N, 9.1833° E
Time zone from Greenwich + 1
Altitude: 121 m.

1. Description of the work

1.1. Concept design

The study process in Building Technology Studio (B) helped and lead us to design fundamental Structural report in which we integrate the design, the static performance, the calculations for different elements, structural cheeks and produce technical drawings.

The study of the functional program resulted in several considerations to be implemented in the design: the temporary functions require the right volume, good accessibility, permeability and capacity to host an efficient number of people and activities during all year; the residential and public functions require the specific area and depend from many aspects. Giving priority to the design and on the other had technologies functions/shapes/volumes/circulations have been explicitly designed having in mind many factors. Relationship to the ground is made by perfectly integrated slope which serve as connection tool and base for our building. Giving respect to the peoples flow and their free movement in South / North direction is made by creating a public space which is in the base of our structure and at the same time is divided and can function independently. An underground floor is added to host conference rooms, exhibitions spaces , shops, cafe etc. the demand for parking lots have not been considered according to the instructions given from the BTS.

The elements of the site that influenced the design the most are the main entrance, the orientation of the building, the railway lines, the channel, the whole master-plan. Special attention is paid to the central space of the entrance to which is given the importance of a main volume, connecting the project area and green zones in Milano, in such a way that we don't have interruption of pedestrian passage.
The orientation, position and type of the living apartments is made in such way that we can take advantage of the natural light, natural heating, big open space that gives the feeling of more space(two level apartments), efficiency (corridors which serve as buffer from the noise coming from the train) and not least the view.
All apartments can be easily reshaped and organized. In different levels and zones we can design living spaces that can be orientated for different category people (good marketing positioning).
The materials we used are different, but blended in the appropriate way giving respect to the culture, history, traditions and new technologies .
As a basic interior material we used wood which is giving natural appearance and feeling, the glass façade is playing different roles (which will be described more in details in the other reports).Type of structure - steel with reinforced concrete cores.

1.2. Structural design

The parameters of the building are 63m by 24m. The tower is 110 m high and the bearing structure is steel. An additional core blocks are formed in the sides which serves as a main resistance for horizontal actions. Technical rooms are concentrated on the underground floor.

The blocks are following basically the same structural scheme, leaving a 3-meter cantilever on both sides in longitudinal direction. Horizontal bracing is not design because the floor (corrugated slabs + mesh and concrete) generate the necessity for resistance in horizontal direction.

The whole structure is divided in 2 parts in order to follow EURO codes. The size is to big to create on discontinuity body and that's why the volume is divided in two. Every one of them is working independently, they are connected and work as one organism but they are self-sustaining. All calculations are design separately but the functions follow same idea . Our aim is to reach and design the structure in such a way that quality, flexibility and durability will become our main terms.

Our team work hand-in-hand with the professors and tutors to optimize the use of our materials, technologies, techniques and methods.

2. Building structure:

- Steel frame structure
- Concrete cores
- Cantilevers

3. Codes adopted:

EN 1990 Eurocode : Basis of Structural Design2.
EN 1991 Eurocode 1: Actions on structures3.
EN 1992 Eurocode 2: Design of concrete structures4.
EN 1993 Eurocode 3: Design of steel structures5.
EN 1994 Eurocode 4: Design of composite steel and concrete

4. Material properties:

- Steel: S 275 (Fe430)

- Concrete: class C25/30; reinforced

II. Load analysis

1. General considerations:

1.1. Classification of actions according to EN 1990- 4.1.1, p.30:

*G- permanent actions: self-weight of structures, fixed equipment

* Q- variable actions: imposed loads on building floors, beams and roofs, wind action and snow

1.2 Basic values of partial safety factors- Class B (failure of structure or structural elements) according to EC3- Structural Steelwork Euro codes:

	Ultimate Limit States		Serviceability Limit States	
	Q	G	Q	G
Unfavourable effect	1.5 (1.35)*	1.35	1.0 (0.9)*	1
Favourable effect	0	1	-	-

1.3 Combinations of actions

*Recommended values of ψ factors for buildings according to EN 1990- Table A1.1

Action	Ψ0	Ψ1	Ψ2
Imposed loads on buildings, category			
Category A: domestic, residential areas	0.7	0.5	0.3
Category B: office areas	0.7	0.5	0.3
Category C: congregation areas	0.7	0.7	0.6

*ULS: Combinations of actions for persistent or transient design situations (fundamental combinations) according to EC 1990:

$$\sum_{j\geq 1} \gamma_{G,j} G_{k,j} "+" \gamma_P P "+" \gamma_{Q,1} Q_{k,1} "+" \sum_{i>1} \gamma_{Q,i} \psi_{0,i} Q_{k,i} \qquad (6.10)$$

*SLS: Combination of actions a) Characteristic combination

$$\sum_{j\geq 1} G_{k,j} "+" P "+" Q_{k,1} "+" \sum_{i>1} \psi_{0,i} Q_{k,i} \qquad (6.14b)$$

b) Frequent combination

$$\sum_{j\geq 1} G_{k,j} \text{"+"} P \text{"+"} \psi_{1,1} Q_{k,1} \text{"+"} \sum_{i>1} \psi_{2,i} Q_{k,i} \qquad (6.15b)$$

c) Quasi-permanent combination

$$\sum_{j\geq 1} G_{k,j} \text{"+"} P \text{"+"} \sum_{i\geq 1} \psi_{2,i} Q_{k,i} \qquad (6.16b)$$

*Combination values of variable actions:

- Ψ0 Qk - used for verification of ULS and irreversible SLS; the value is chosen so that the probability that the effect caused by the combination is approximately the same as by the characteristic value of an individual action.

- Ψ1 Qk - used for verification of ULS involving accidental actions and reversible SLS; the value is determined in such a way that either the total time, within the reference period, during which it is exceeded is only a small given part of the reference period, or the frequency of it being exceeded is limited to a given value.

- Ψ2 Qk - used for verification of ULS involving accidental actions and for verification of reversible SLS; used for the calculation of long-term effects; the value is determined so that the total period of time for which it will be exceeded is a large fraction of the reference period.

1.5 Recommended limiting values for vertical deflections at Serviceability Limit States

Conditions	Limits	
	δ max	δ2
Roofs generally	L/200	L/250
Roofs frequently carrying personnel other than for maintenance	L/250	L/300
Floors generally	L/250	L/300
Floors and roofs supporting plaster or other brittle finish of non flexible partition	L/250	L/300
Floors supporting columns (unless the deflection has been included in the global state analysis for ULS)	L/400	L/500
Where δ max can impair the appearance of the building	L/250	

2. Floor

Dead load calculation

N	Material	Thickness (mm)	Density (kN/m3)	Weight (kg/m2)	Weight (kN/m^2)
1	Covering wood	20	16	32	0.320
2	Screed (mortar/leveling)	20	20	40	0.400
3	Insulation plaster	50	0.60	0.3	0.030
4	water proofing film	-	-	-	0.010
5	ComFlor® 46 Composite slab				2.380
6	Suspended ceiling	-	-	-	0.100
7	floor heating system				0.150
	TOTAL				3.39 KN/m^2

Smart facade / perimeter wall /		2.1KN/m^2
Internal partition		0.65KN/m^2

Live load calculation

Loaded areas	qk (kN/m2) uniformly distributed load
Category A	2.00

3.Roof

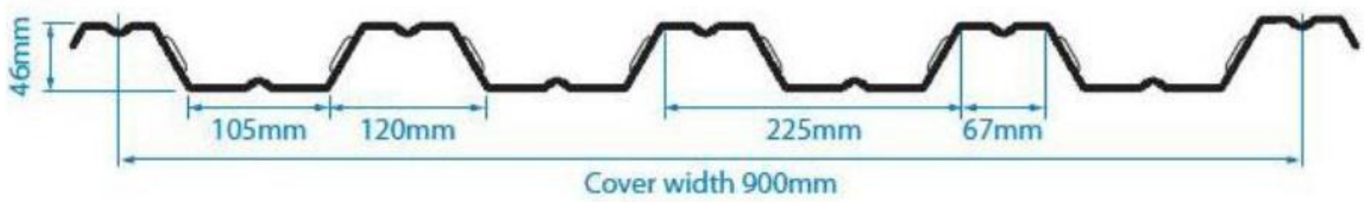
Dead load calculation

N	Material	Thickness (mm)	Density (kN/m3)	Weight (kg/m2)	Weight (kN/m^2)
1	Land of culture	170	17.0	289	2.89
2	Sand/stone	20	20	40	0.40
3	Thermal insulation XPS - 150	150	0.3	5	0.05
4	water proofing film	-	-	1	0.01
5	Loor slab	-	-	-	2.38
6	service/ suspended celling	-	-	-	0.10
	TOTAL				5.833

Live load calculation

live load roof	Q (kN/m2)	Q x safety factor 1.5
Account for snow load in Milan	1.2	1.8

Comflor 46 Composite slab



ComFlor® 46 Composite slab – volume and weight

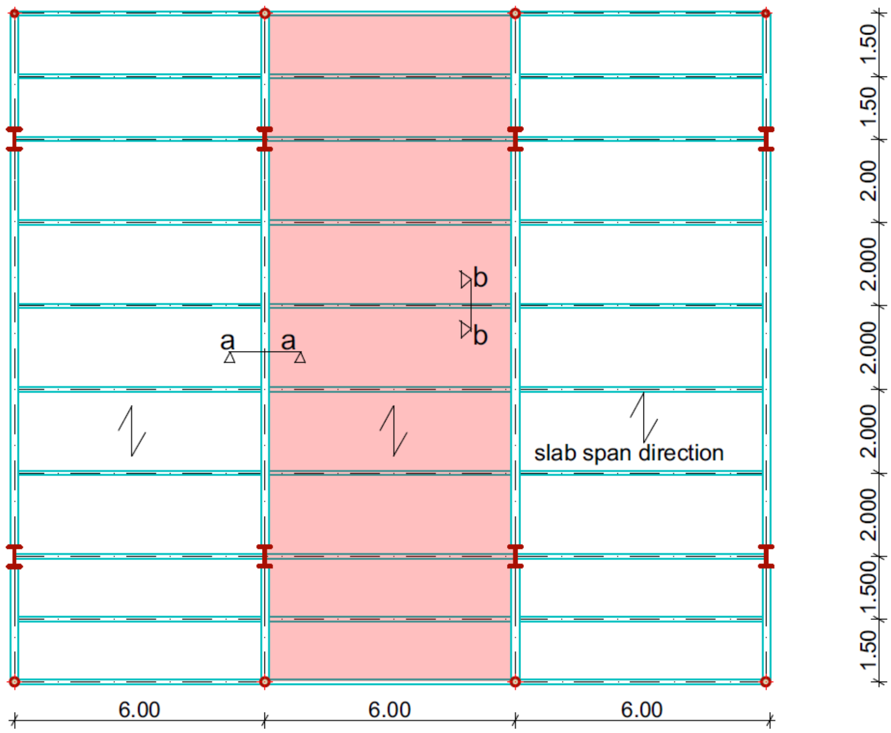
Slab depth (mm)	Concrete volume (m^3/m^2)	Weight of concrete (kN/m^2) Normal weight Concrete Wet	Normal weight Concrete Dry	Lightweight Concrete Wet	Lightweight Concrete Dry
110	0.091	2.14	2.10	1.69	1.60
115	0.096	2.26	2.21	1.79	1.69
120	0.101	2.38	2.33	1.88	1.78
130	0.111	2.61	2.56	2.07	1.96
140	0.121	2.85	2.79	2.25	2.13
145	0.126	2.96	2.90	2.35	2.22
150	0.131	3.08	3.02	2.44	2.31
180	0.161	3.79	3.71	3.00	2.84
200	0.181	4.26	4.17	3.37	3.19
240	0.221	5.20	5.09	4.12	3.90

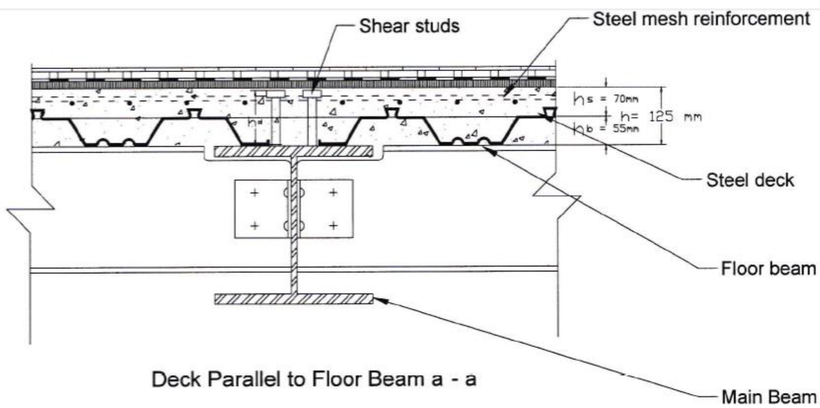
Simple trapezoidal composite deck with strong and reliable shear bond performance.
The profile is economic and nest able, reducing transport and handling costs.

Low concrete usage
The trapezoidal shape profile of ComFlor® 46 reduces the volume of concrete used, with resultant savings in structural and foundation costs.

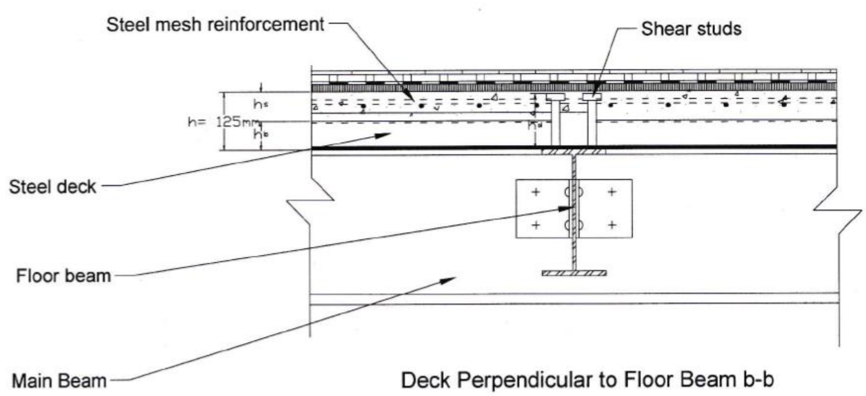
Fire performance
Comflor shallow and deep composite floor decks offering different spanning and floor depth options have undergone extensive fire testing and fire engineering work to derive maximum benefit to the customer.

Slab detail



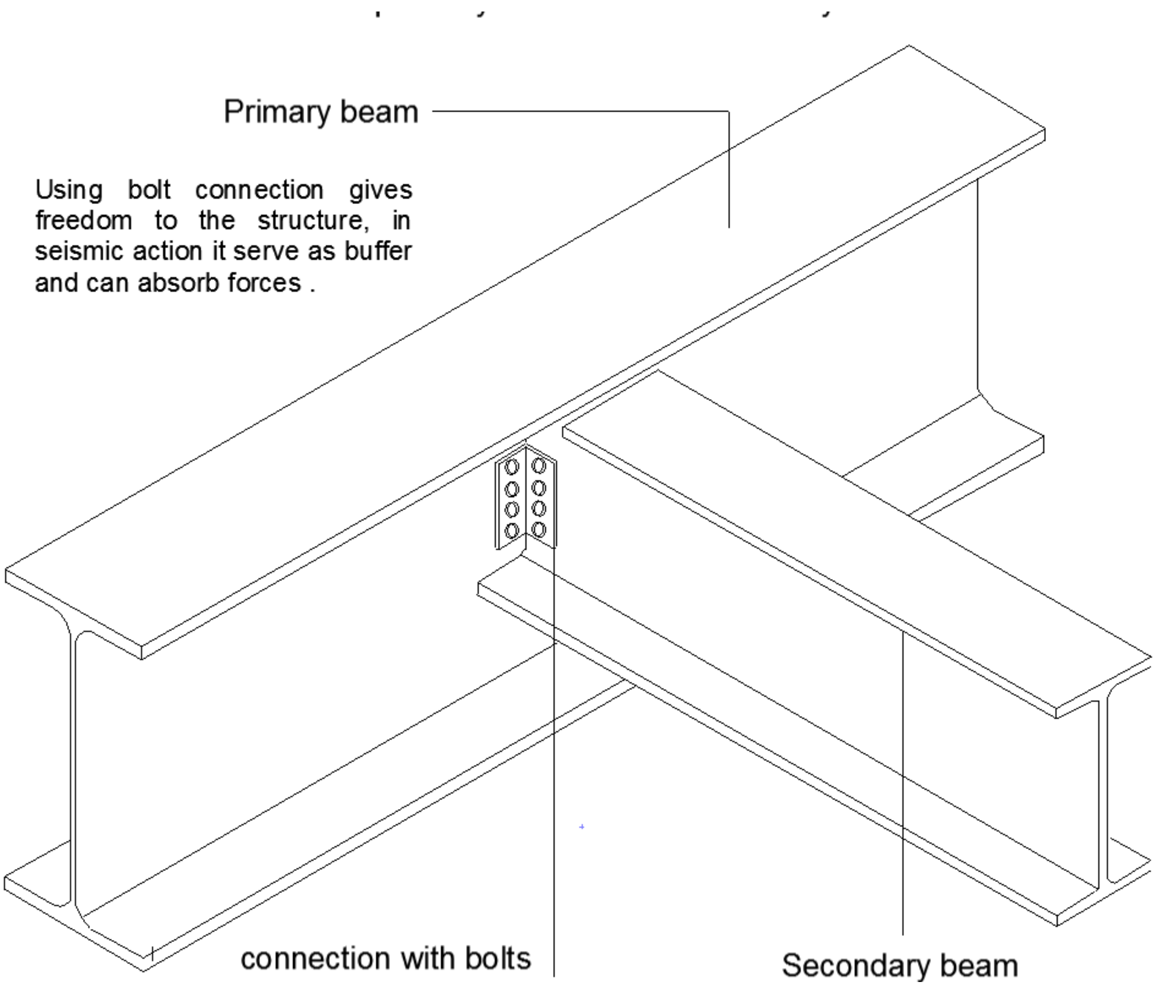


Deck Parallel to Floor Beam a - a

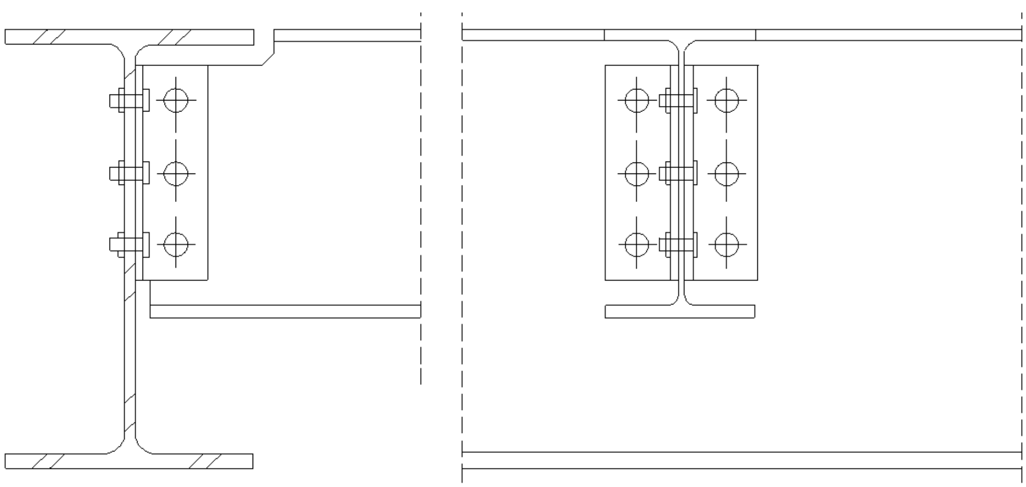


Deck Perpendicular to Floor Beam b-b

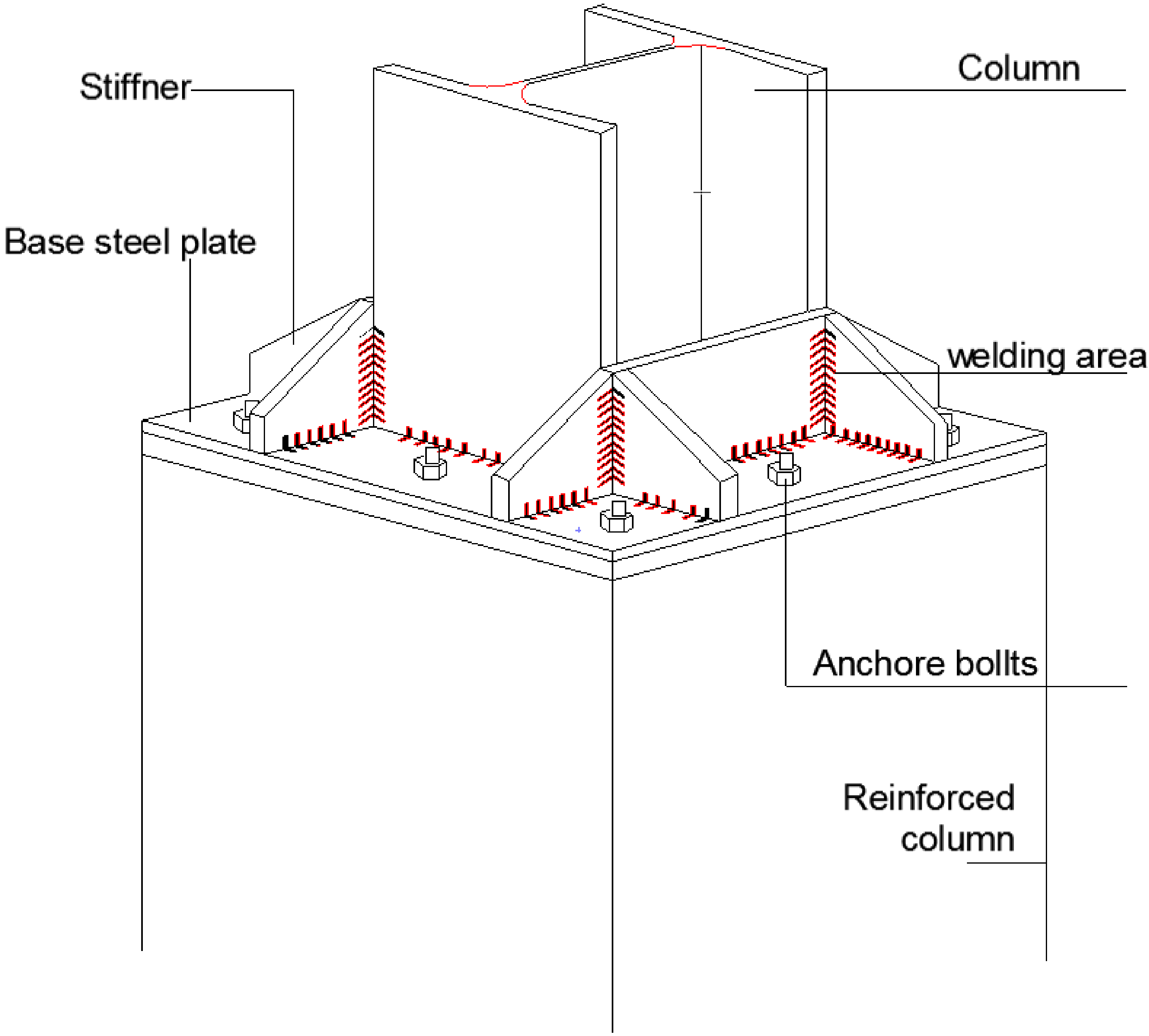
Connection between primary beam and secondary beam



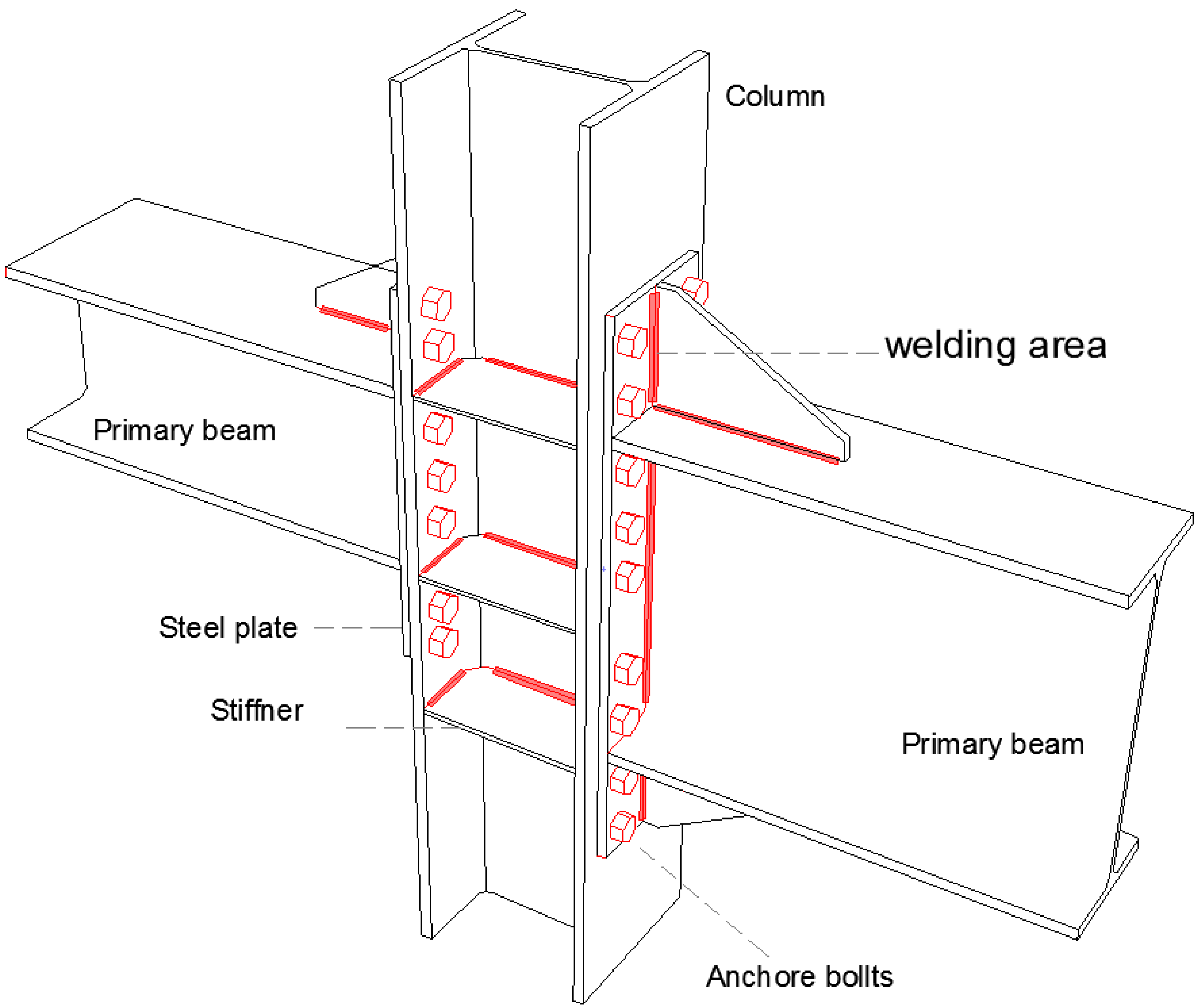
To escape deformation during welding process , the connections should be done step by step. Very big importance have to be paid to the welding material, quality (electrode) .



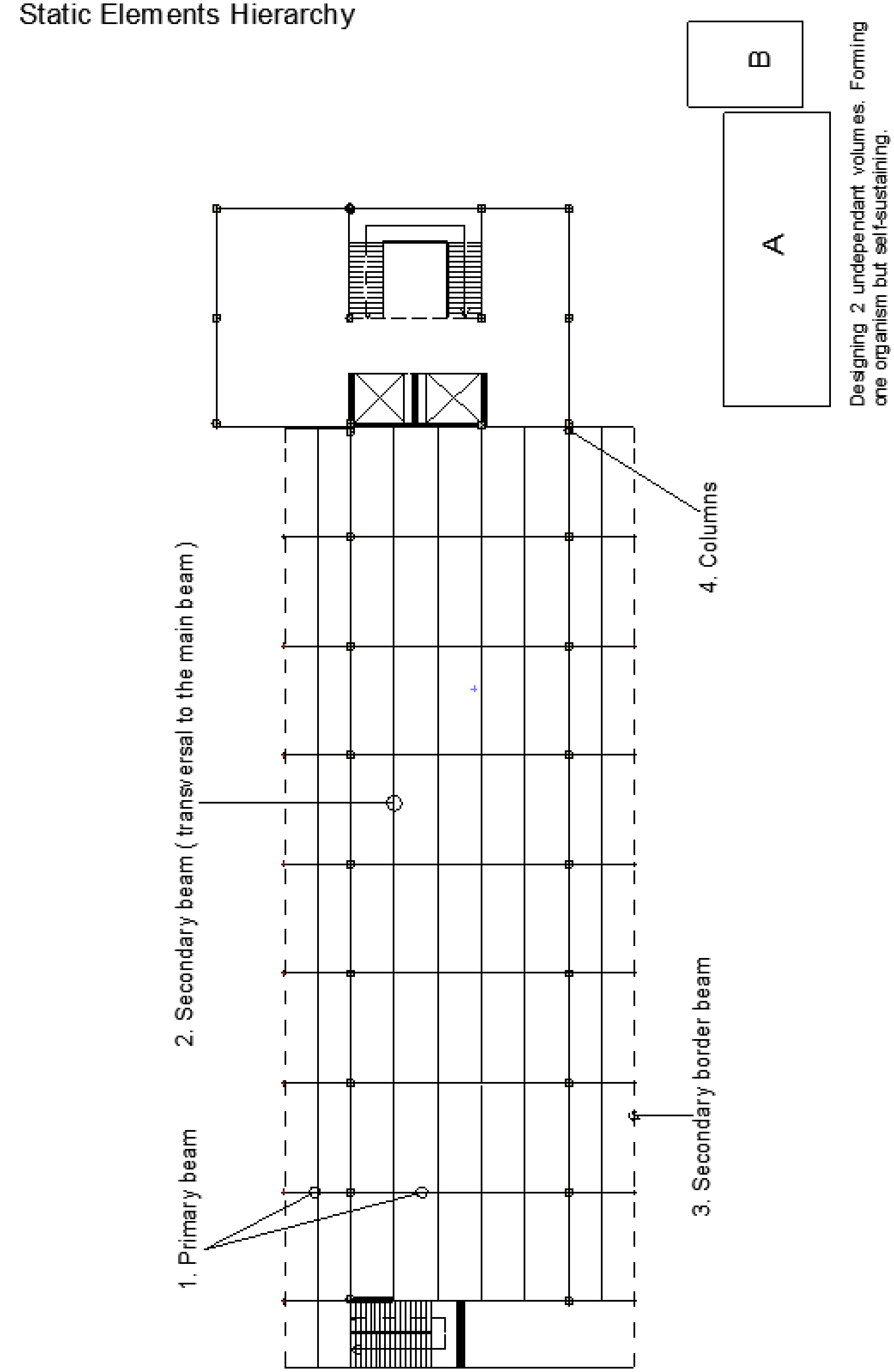
Foundations and column



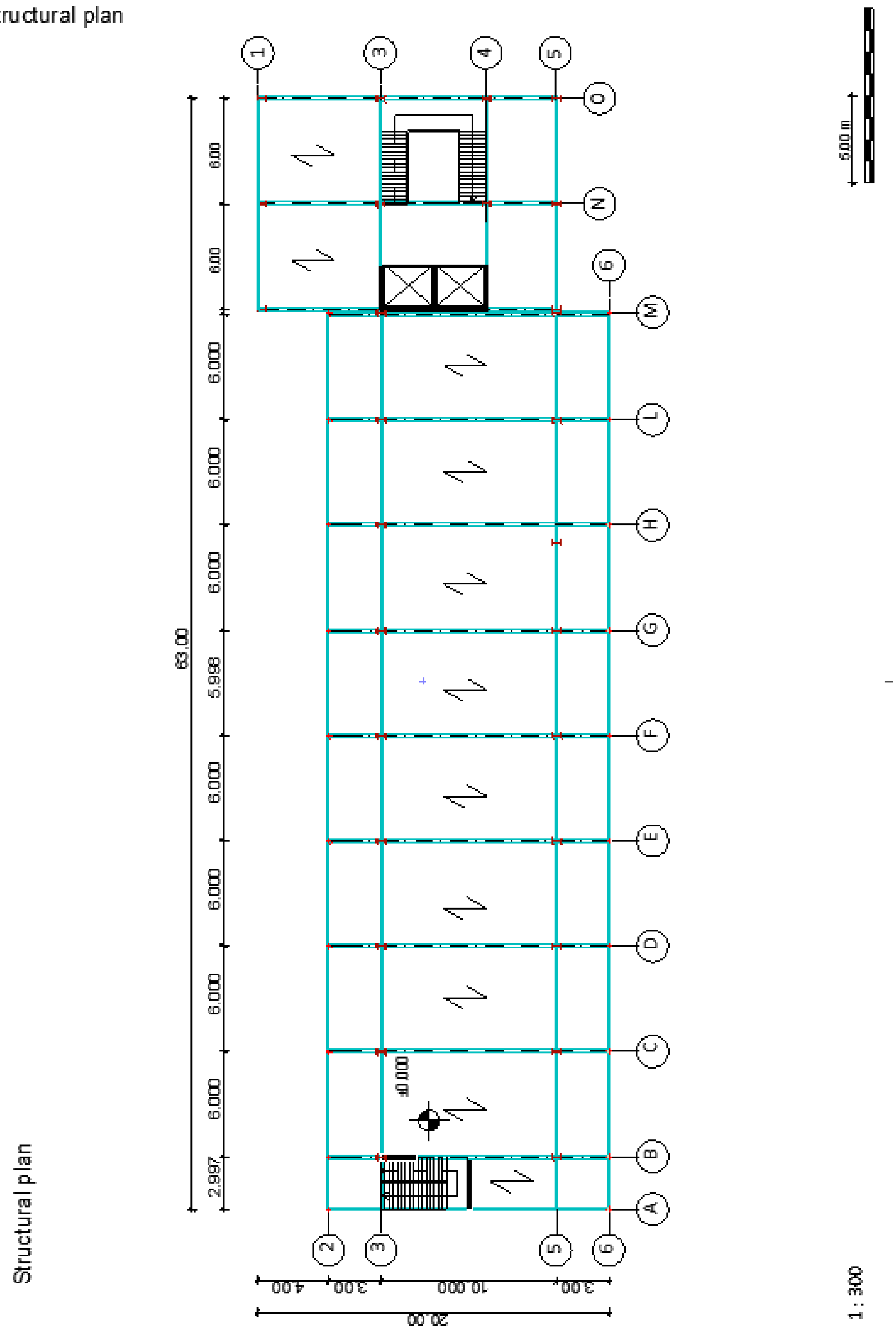
Connection between column and primary middle and cantilever beam.



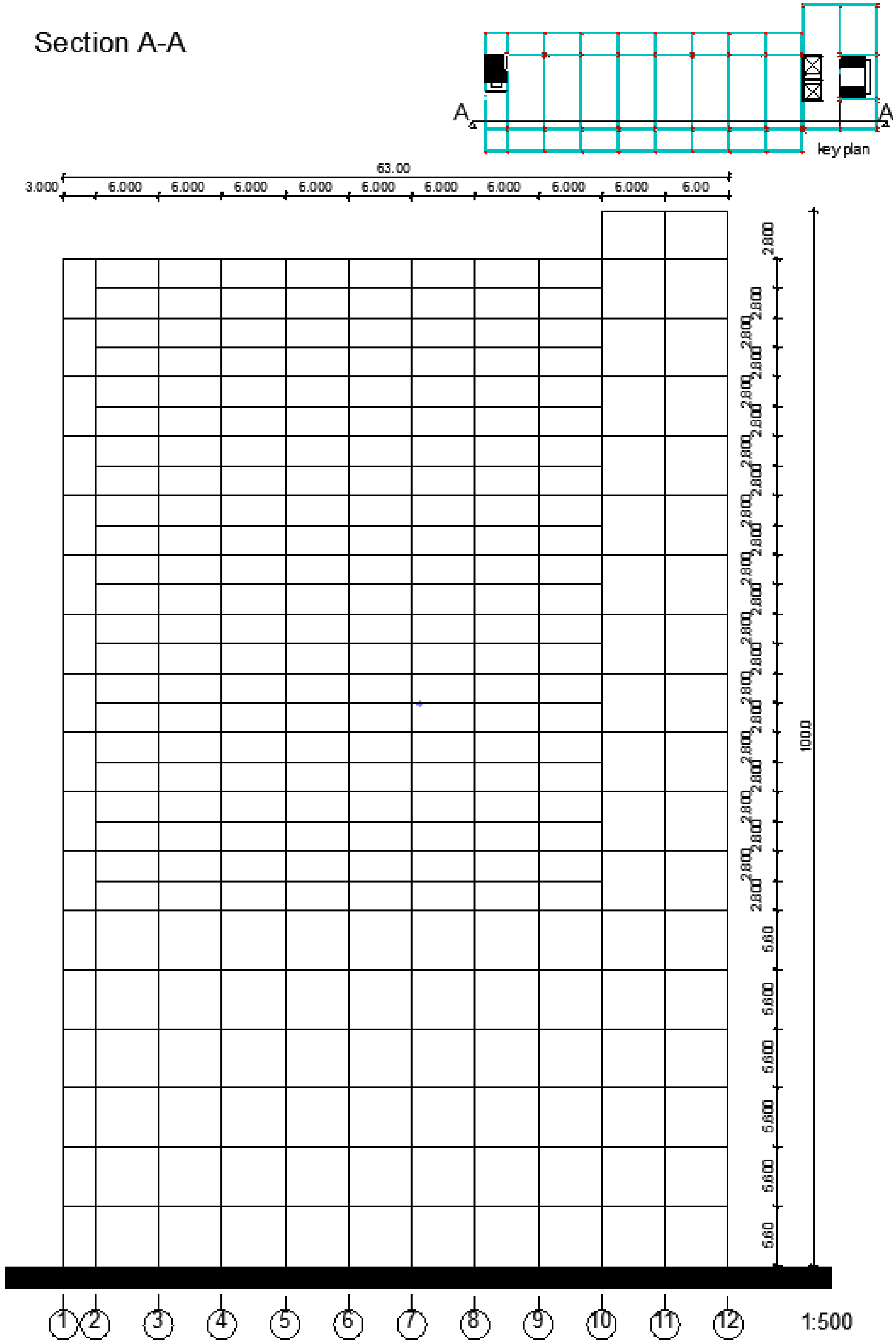
Static Elements Hierarchy



Structural plan



Section A-A


A
A
key plan
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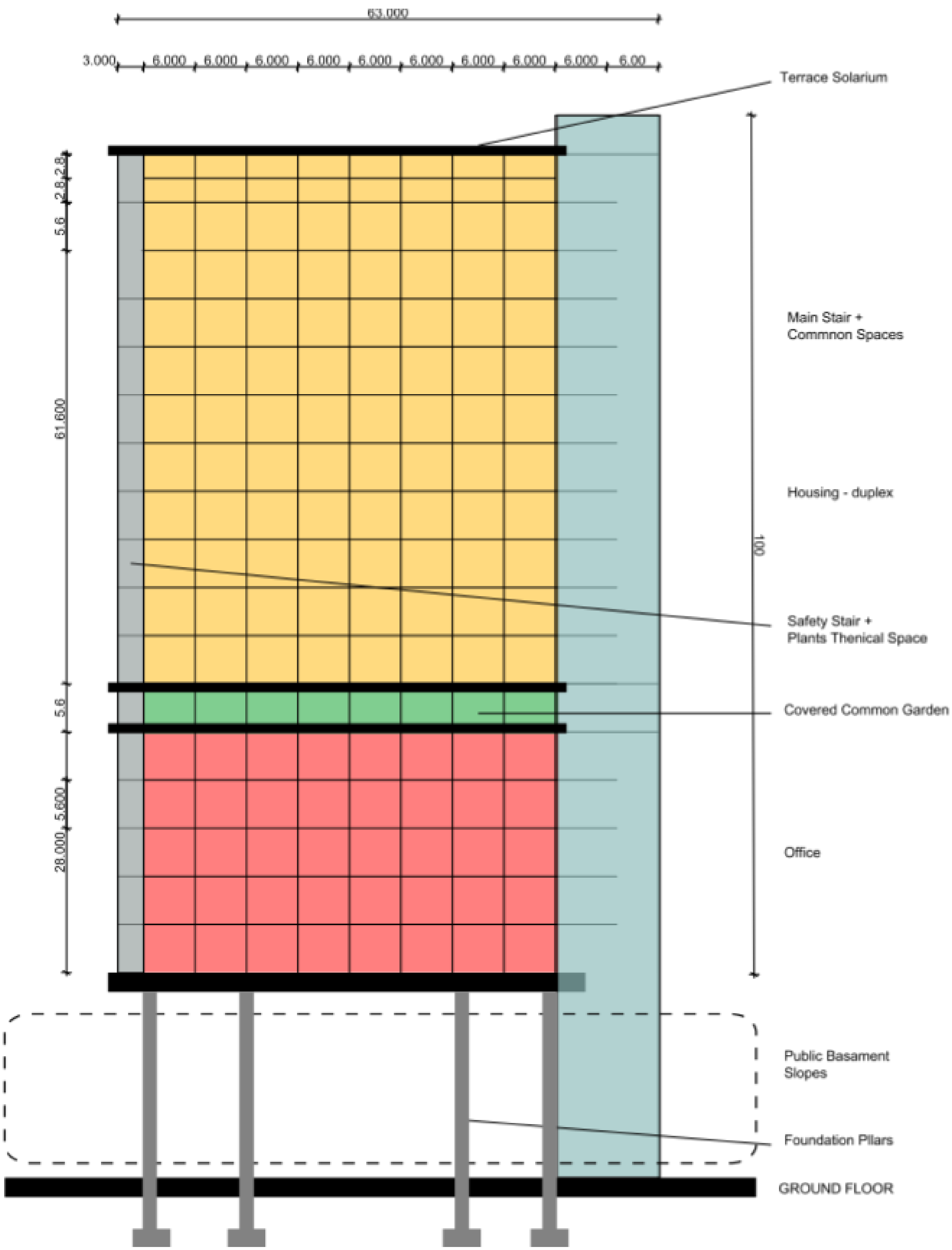
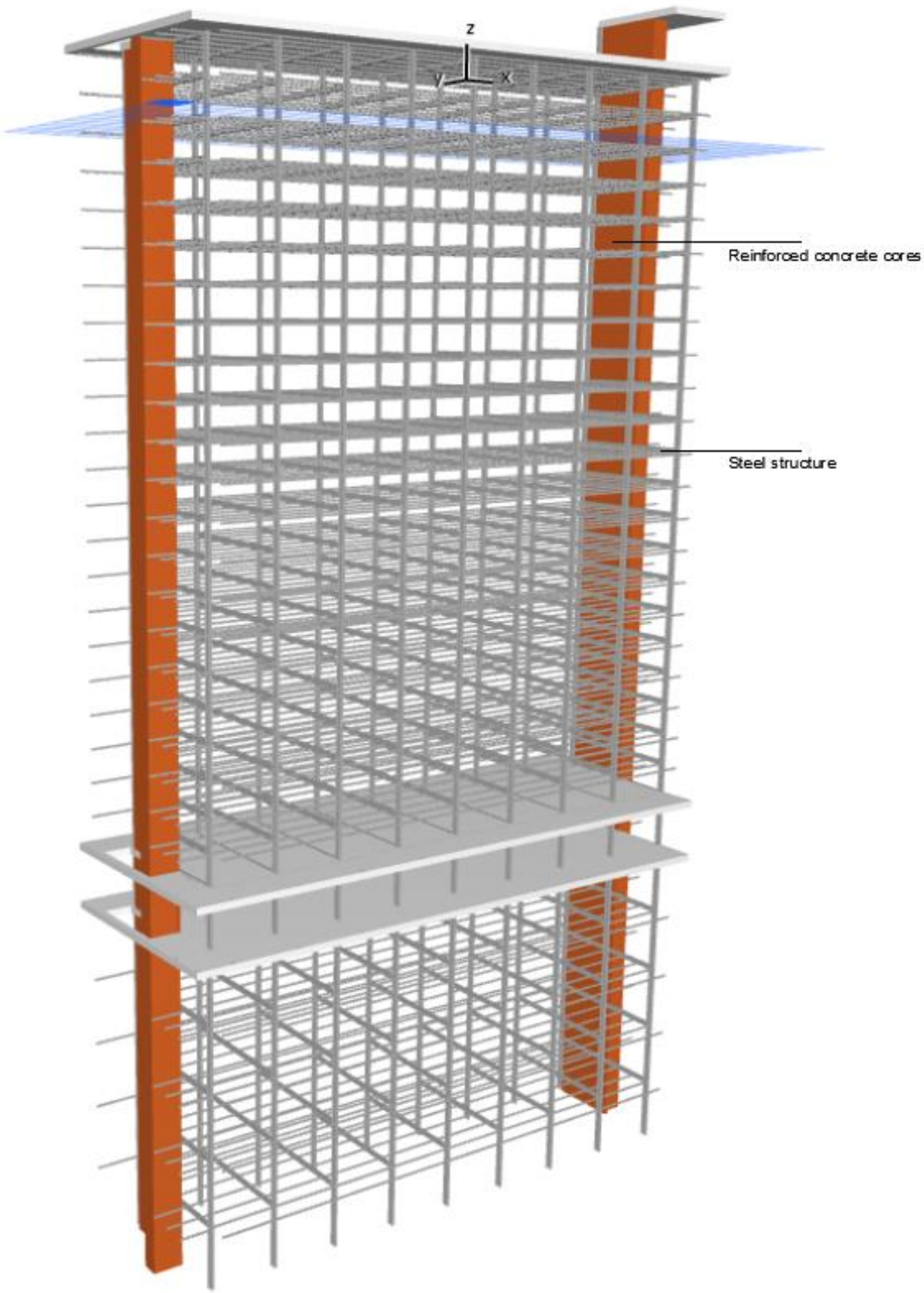
Vertical program distribution / concept /



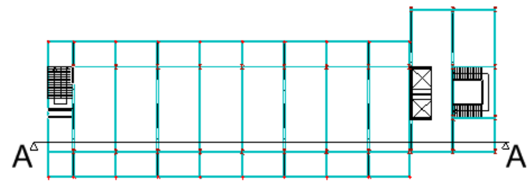
Structure elements of the building



STATIC LOADS

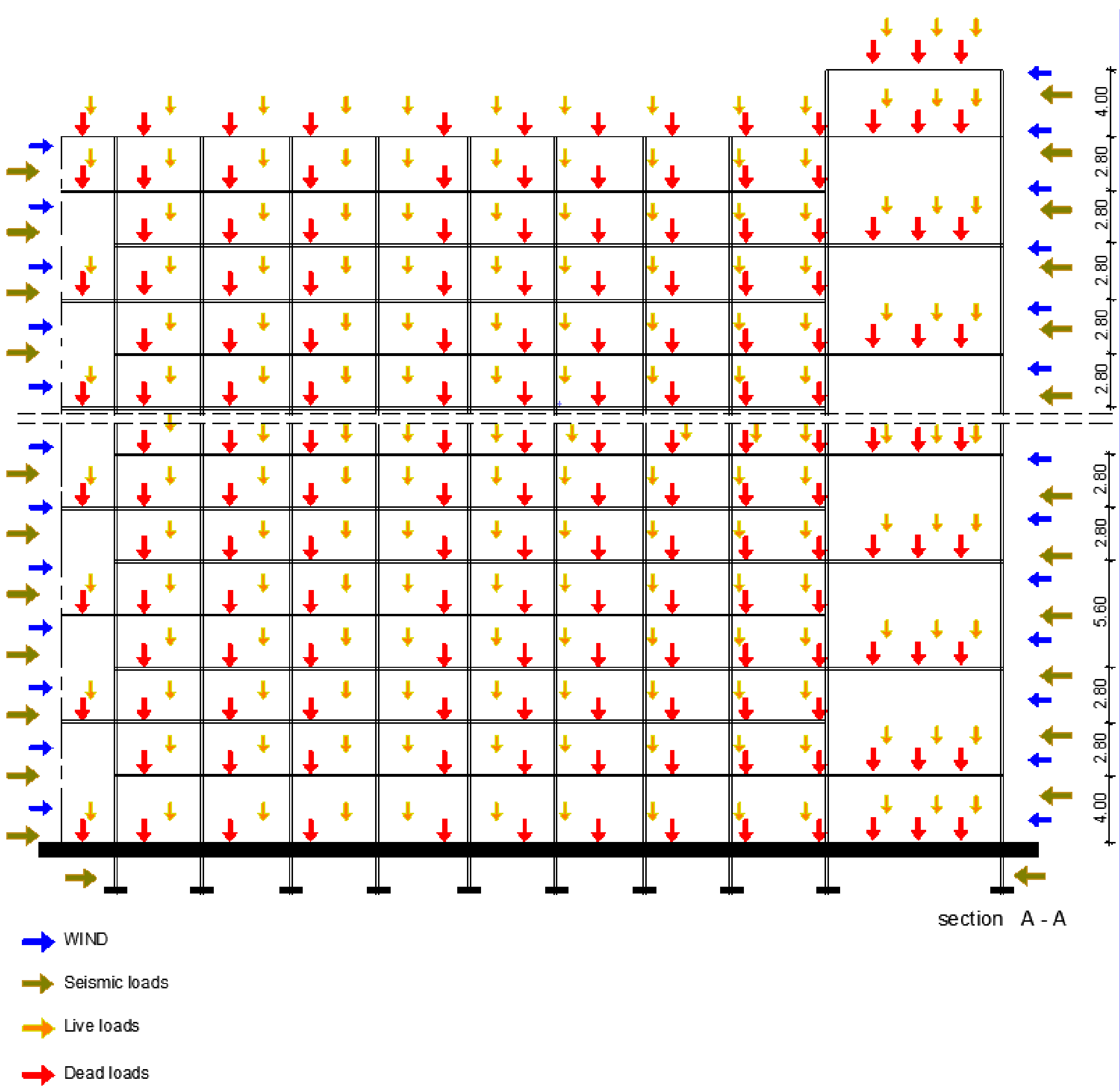
Load types

key plan



Dead loads/permanent
Live loads/variable (imposed, wind, snow)
Cyclic loads (Accidental loads) - earthquakes, thermal loads, fire

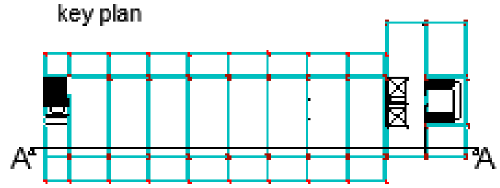
Load classification base on direction:
Vertical Loads: self weight of all components of the building, internal partition, people etc.
Horizontal loads : seismic vibration, wind.



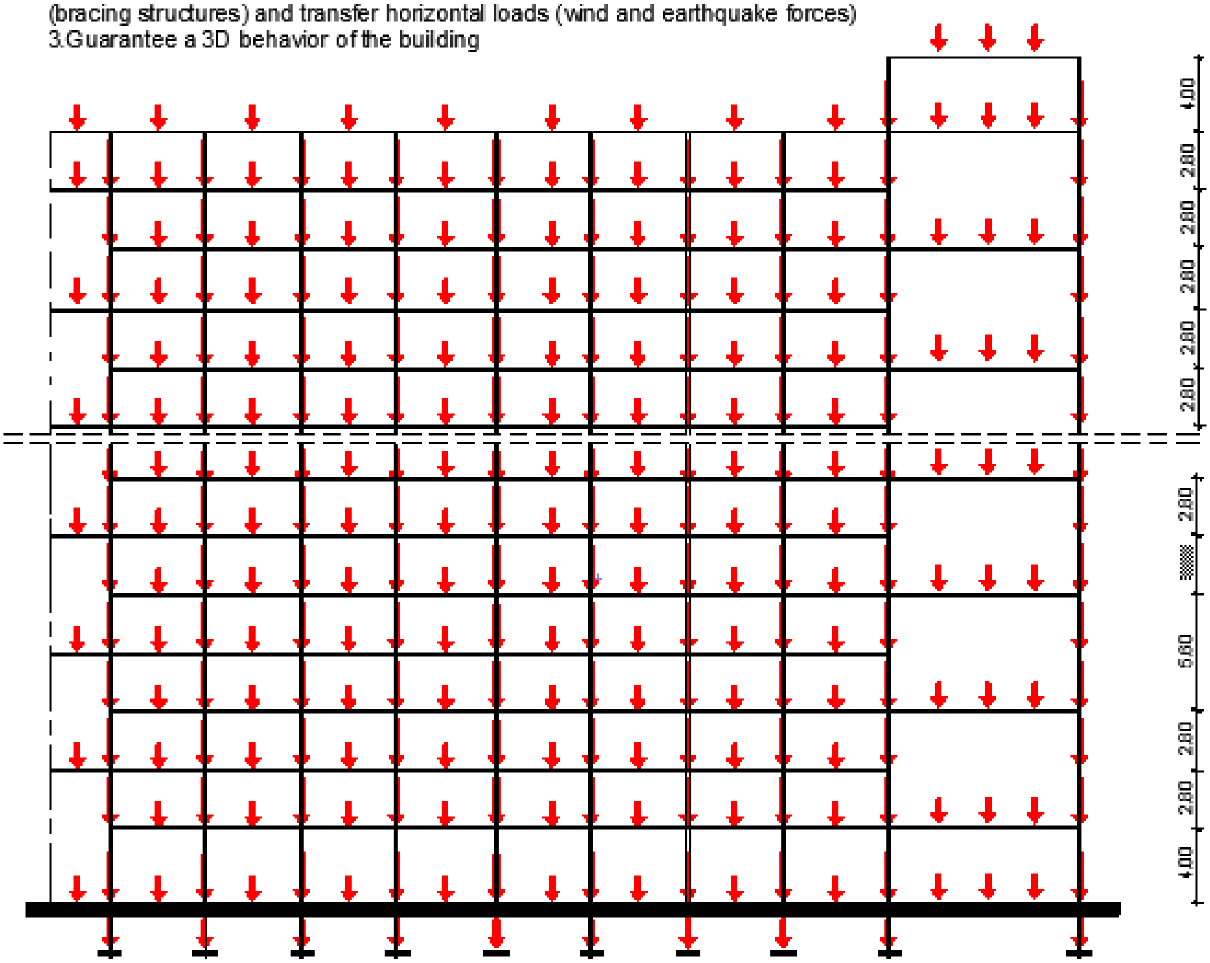
STATIC LOADS

Distribution of the loads on the Building

key plan



Purposes of the floor slab :
1.Collect and transfer the vertical loads to the beams
2.Connect the vertical structures resisting horizontal forces (bracing structures) and transfer horizontal loads (wind and earthquake forces)
3.Guarantee a 3D behavior of the building



section A- A

1. Distributed loads are collected by the floor system

2. Loads are tranferred by the floor system to the beams

3. Load are transferred by the beams to the columns

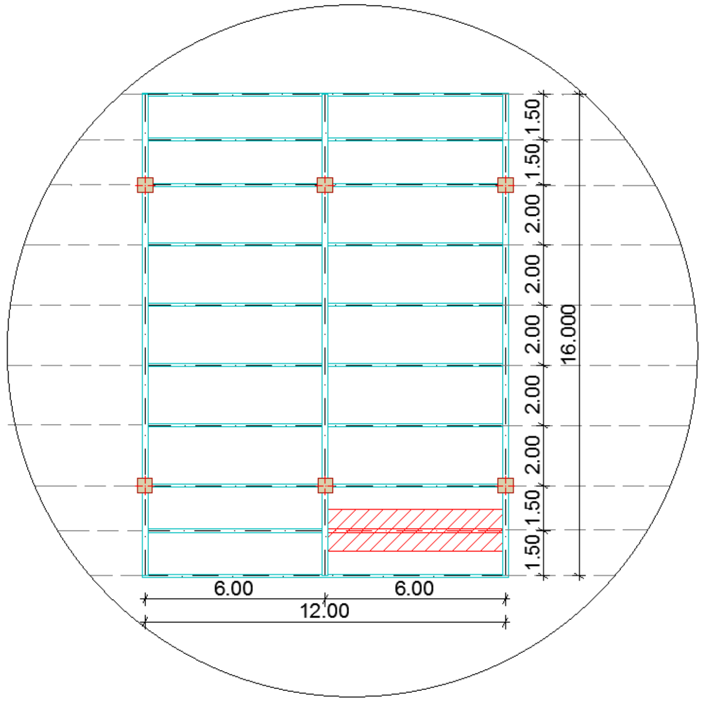
4. Loads are transferred by the columns to foundations

5. Loads are transferred by the foundations to ground

Calculations

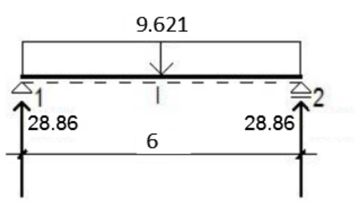
Floor. Secondary beam in cantiliver

Position on the floor plan

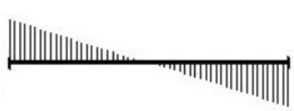


Length of the beam	L	6.00 m
Span of the tributary area	span	1.50 m
Tributary area	L x (span)	9m2
Permanent load: Due to floor weight Due to secondary beam weight	G: Gfl Gsb (IPE 200 = 22.4 kg/m)	 2.38 kN/m2 0.224 KN/m
Variable load	Qs	2 KN/m2
Total dead load	Gtot = span x Gfl + Gsb Gtot = 1.50 m x 2.38 kN/m2 + 0.224 KN/m	3.794 KN/m
Total variable load	Qtot = span x Qs Qtot = 1.50 m x 2.00 kN/m2	3KN/m
Design actions combination at SLS	Fd = 1.00 x Gtot + 1.00 x Qtot 1.00 x 3.794 kN/m + 1.00 x 3.00 kN/m	6.794KN/m
Design actions combination at ULS	Fd = 1.35 x Gtot + 1.50 x Qtot 1.35 x 3.794 kN/m + 1.50 x 3 kN/m	9.6219KN/m

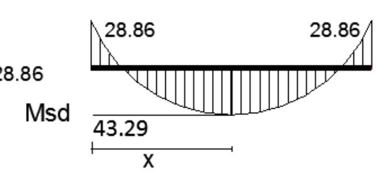
Diagrams / ULS



Shear
P x L / 2 = 28.86



Moment
P x L2 / 12= 28.86



Maximum bending moment

$M_{sd} = F_d \times L^2 / 8 = 9.621$ kN/m $\times$ (6 m)^2 / 8 = 43.29 kNm

$$W = \frac{M}{f_{yd}} = \frac{43.29 \cdot 10^6}{275} = 157.4 \text{ cm}^3$$ Designed profile IPE 200

Designed profile IPE 200 / 194.3 cm3

h mm	b mm	a mm	e mm	r mm	Peso kg/m	Sezione cm2	Momenti di inerzia Jx cm4	Jy cm4	Moduli di resistenza Wx cm3	Wy cm3
80	46	3,8	5,2	5	6,0	7,64	80,14	8,49	20,03	3,69
100	55	4,1	5,7	7	8,1	10,32	171,0	15,92	34,20	5,79
120	64	4,4	6,3	7	10,4	13,21	317,8	27,67	52,96	8,65
140	73	4.7	6,9	7	12,9	16,43	541,2	44,92	77,32	12,31
160	82	5,0	7,4	9	15,8	20,09	869,3	68,31	108,7	16,66
180	91	5,3	8,0	9	18,8	23,95	1.317	100,9	146,3	22,16
200	100	5,6	8,5	12	22,4	28,48	1.943	142,4	194,3	28,47
220	110	5,9	9,2	12	26,2	33,37	2.772	204,9	252,0	37,25
240	120	6,2	9,8	15	30,7	39,12	3.892	283,6	324,3	47,27
270	135	6,6	10,2	15	36,1	45,95	5.790	419,9	428,9	62,20
300	150	7,1	10,7	15	42,2	53,81	8.356	603,8	557,1	80,50
330	160	7,5	11,5	18	49,1	62,61	11.770	788,1	713,1	98,52
360	170	8,0	12,7	18	57,1	72,73	16.270	1.043	903,6	122,8
400	180	8,6	13,5	21	66,3	84,46	23.130	1.318	1.156	146,4
450	190	9,4	14,6	21	77,6	98,82	33.740	1.676	1.500	176,4
500	200	10,2	16,0	21	90,7	115,5	48.200	2.142	1.928	214,2

Deflection check

Deflection maximum allowed at SLS: floors generally

δmax = L/250 = 6000/250 = 24 mm

Deflection for uniformly distributed load

δ = 5/384 x (Fk x L3) / (E x ly)

Fk = Gk x L

E = elasticity modulus of steel = 210 MPa = 210 000 N/mm2

ly = 1943 cm4 for IPE 200

Deflection for dead load

Fk = Gk x L = 3.794 kN/m x 6.00 m = 22.76 kN

δ1= 5/384 x [22.76 x 103 N x (6000 mm)3] / (210 000 N/mm2 x 1943 x 104 mm4) = 12.28 mm

- Deflection for live load

Fk = Qk x L = 3 kN/m x 6.00 m = 18 kN

δ2 = 5/384 x [18 x 103 N x (60000 mm)3] / (210 000 N/mm2 x 1943 x 104mm4) = 8.67 mm

Maximum deflection of secondary beam

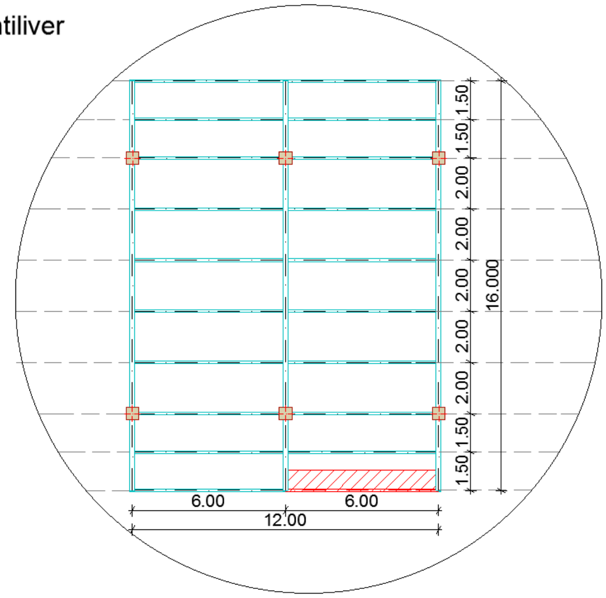
δmax = δ1 + δ2 = 12.28 mm + 8.67 mm= 20.95 mm

Deflection check

δmax = L/250 = 6000/250 = 24.00 mm > 20.95 mm - satisfied

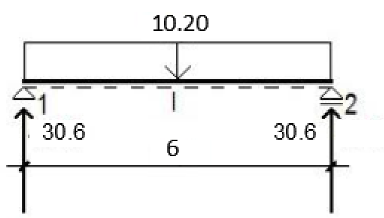
Floor. Secondary border beam in cantiliver

Position on the floor plan

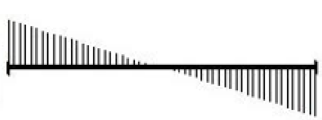


Length of the beam	L	6.00 m
Span of the tributary area	span	0.75 m
Tributary area	L x (span)	4.5m2
Permanent load:	G:	
Due to floor weight Due to secondary beam weight Due to smart facade weight	Gfl Gsb (IPE 200 = 22.4 kg/m) Gf	2.38 kN/m2 0.224 KN/m 2.10 KN/m
Variable load	Qs	2 KN/m2
Total dead load	Gtot = span x Gfl + Gsb +Gf Gtot = 0.75 m x 2.38 kN/m2 + 0.224 KN/m + 2.10KN/m	5.894 KN/m
Total variable load	Qtot = span x Qs Qtot = 0.75 m x 2.00 kN/m2	1.5KN/m
Design actions combination at SLS	Fd = 1.00 x Gtot + 1.00 x Qtot 1.00 x 5.894 kN/m + 1.00 x 1.5 kN/m	7.394KN/m
Design actions combination at ULS	Fd = 1.35 x Gtot + 1.50 x Qtot 1.35 x 5.894 kN/m + 1.50 x 1.5 kN/m	10.20Kn/m

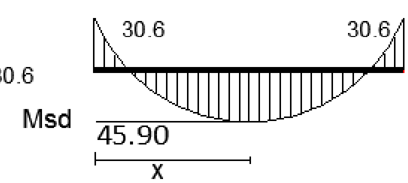
Diagrams / ULS



Shear
P x L / 2 = 30.6



Moment
P x L2 / 12= 30.6



Maximum bending moment

$M_{sd} = F_d \times L^2 / 8 = 10.20 \text{ kN/m} \times (6 \text{ m})^2 / 8 = 45.9 \text{ kNm}$

$$W = \frac{M}{f_{yd}} = \frac{45.90 \cdot 10^6}{275} = 166.9 \text{ cm}^3$$ Designed profile IPE 200

Designed profile IPE 200 / 194.3 cm3

							Momenti di inerzia		Moduli di resistenza	
h mm	b mm	a mm	e mm	r mm	Peso kg/m	Sezione cm2	Jx cm4	Jy cm4	Wx cm3	Wy cm3
80	46	3,8	5,2	5	6,0	7,64	80,14	8,49	20,03	3,69
100	55	4,1	5,7	7	8,1	10,32	171,0	15,92	34,20	5,79
120	64	4,4	6,3	7	10,4	13,21	317,8	27,67	52,96	8,65
140	73	4.7	6,9	7	12,9	16,43	541,2	44,92	77,32	12,31
160	82	5,0	7,4	9	15,8	20,09	869,3	68,31	108,7	16,66
180	91	5,3	8,0	9	18,8	23,95	1.317	100,9	146,3	22,16
200	100	5,6	8,5	12	22,4	28,48	1.943	142,4	194,3	28,47
220	110	5,9	9,2	12	26,2	33,37	2.772	204,9	252,0	37,25
240	120	6,2	9,8	15	30,7	39,12	3.892	283,6	324,3	47,27
270	135	6,6	10,2	15	36,1	45,95	5.790	419,9	428,9	62,20
300	150	7,1	10,7	15	42,2	53,81	8.356	603,8	557,1	80,50
330	160	7,5	11,5	18	49,1	62,61	11.770	788,1	713,1	98,52
360	170	8,0	12,7	18	57,1	72,73	16.270	1.043	903,6	122,8
400	180	8,6	13,5	21	66,3	84,46	23.130	1.318	1.156	146,4
450	190	9,4	14,6	21	77,6	98,82	33.740	1.676	1.500	176,4
500	200	10,2	16,0	21	90,7	115,5	48.200	2.142	1.928	214,2

Deflection check

Deflection maximum allowed at SLS: floors generally

δmax = L/250 = 6000/250 = 24 mm

Deflection for uniformly distributed load

δ = 5/384 x (Fk x L3) / (E x ly)

Fk = Gk x L

E = elasticity modulus of steel = 210 MPa = 210 000 N/mm2

ly = 1943 cm4 for IPE 200

Deflection for dead load

Fk = Gk x L = 5.894 kN/m x 6.00 m = 35.36 kN

δ1= 5/384 x [35.36 x 103 N x (6000 mm)3] / (210 000 N/mm2 x 1943 x 104 mm4) = 17.12 mm

- Deflection for live load

Fk = Qk x L = 1.5 kN/m x 6.00 m = 9 kN

δ2 = 5/384 x [9 x 103 N x (60000 mm)3] / (210 000 N/mm2 x 1943 x 104mm4) = 4.33 mm

Maximum deflection of secondary beam

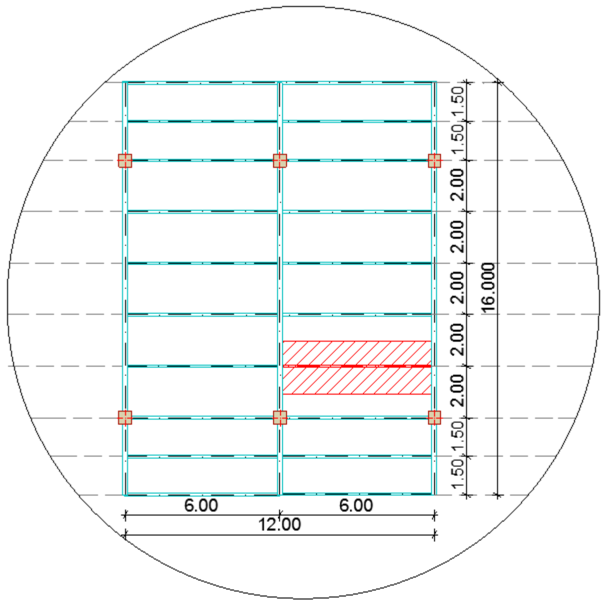
δmax = δ1 + δ2 = 17.12 mm + 4.33 mm= 21.45 mm

Deflection check

δmax = L/250 = 6000/250 = 24.00 mm > 21.45 mm - satisfied

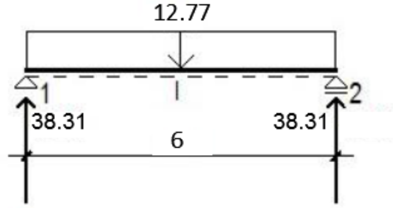
Floor.Secondary beam in middle part

Position on the floor plan

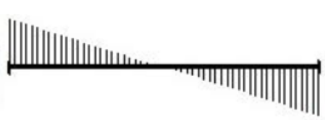


Length of the beam	L	6.00 m
Span of the tributary area	span	2.00m
Tributary area	L x (span)	12m2
Permanent load: Due to floor weight Due to secondary beam weight	G: Gfl Gsb (IPE 220 = 26.2 kg/m)	 2.38 kN/m2 0.262 KN/m
Variable load	Qs	2 KN/m2
Total dead load	Gtot = span x Gfl + Gsb Gtot = 2.00 m x 2.38 kN/m2 + 0.262 KN/m	5.022 KN/m
Total variable load	Qtot = span x Qs Qtot = 2.00 m x 2.00 kN/m2	4KN/m
Design actions combination at SLS	Fd = 1.00 x Gtot + 1.00 x Qtot 1.00 x 5.022 kN/m + 1.00 x 4.00 kN/m	9.022KN/m
Design actions combination at ULS	Fd = 1.35 x Gtot + 1.50 x Qtot 1.35 x 5.022 kN/m + 1.50 x 4 kN/m	12.77KN/m

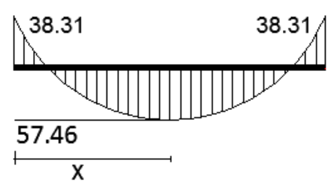
Diagrams / ULS



Shear
P x L / 2 = 38.31



Moment
P x L2 / 12= 38.31



Maximum bending moment

$M_{sd} = F_{d} \times L^{2} / 8 = 12.77 kN/m \times (6\ m)^2 / 8 = 57.46\ kNm$

$$W = \frac{M}{f_{yd}} = \frac{57.46 \cdot 10^{6}}{275} = 208.9.4\ cm^{3}$$ Designed profile IPE 220

Designed profile IPE 220 / 252.0 cm3

h mm	b mm	a mm	e mm	r mm	Peso kg/m	Sezione cm2	Momenti di inerzia Jx cm4	 Jy cm4	Moduli di resistenza Wx cm3	 Wy cm3
80	46	3,8	5,2	5	6,0	7,64	80,14	8,49	20,03	3,69
100	55	4,1	5,7	7	8,1	10,32	171,0	15,92	34,20	5,79
120	64	4,4	6,3	7	10,4	13,21	317,8	27,67	52,96	8,65
140	73	4.7	6,9	7	12,9	16,43	541,2	44,92	77,32	12,31
160	82	5,0	7,4	9	15,8	20,09	869,3	68,31	108,7	16,66
180	91	5,3	8,0	9	18,8	23,95	1.317	100,9	146,3	22,16
200	100	5,6	8,5	12	22,4	28,48	1.943	142,4	194,3	28,47
220	110	5,9	9,2	12	26,2	33,37	2.772	204,9	252,0	37,25
240	120	6,2	9,8	15	30,7	39,12	3.892	283,6	324,3	47,27
270	135	6,6	10,2	15	36,1	45,95	5.790	419,9	428,9	62,20
300	150	7,1	10,7	15	42,2	53,81	8.356	603,8	557,1	80,50
330	160	7,5	11,5	18	49,1	62,61	11.770	788,1	713,1	98,52
360	170	8,0	12,7	18	57,1	72,73	16.270	1.043	903,6	122,8
400	180	8,6	13,5	21	66,3	84,46	23.130	1.318	1.156	146,4
450	190	9,4	14,6	21	77,6	98,82	33.740	1.676	1.500	176,4
500	200	10,2	16,0	21	90,7	115,5	48.200	2.142	1.928	214,2

Deflection check

Deflection maximum allowed at SLS: floors generally

δmax = L/250 = 6000/250 = 24 mm

Deflection for uniformly distributed load

δ = 5/384 x (Fk x L3) / (E x ly)

Fk = Gk x L

E = elasticity modulus of steel = 210 MPa = 210 000 N/mm2

ly = 2.772 cm4 for IPE 220

Deflection for dead load

Fk = Gk x L = 5.022kN/m x 6.00 m = 30.132 kN

δ1= 5/384 x [35.36 x 103 N x (6000 mm)3] / (210 000 N/mm2 x 2772 x 104 mm4) = 10.42 mm

- Deflection for live load

Fk = Qk x L = 4 kN/m x 6.00 m = 24 kN

δ2 = 5/384 x [24 x 103 N x (60000 mm)3] / (210 000 N/mm2 x 2772 x 104mm4) = 12.53 mmMaximum deflection of secondary beam

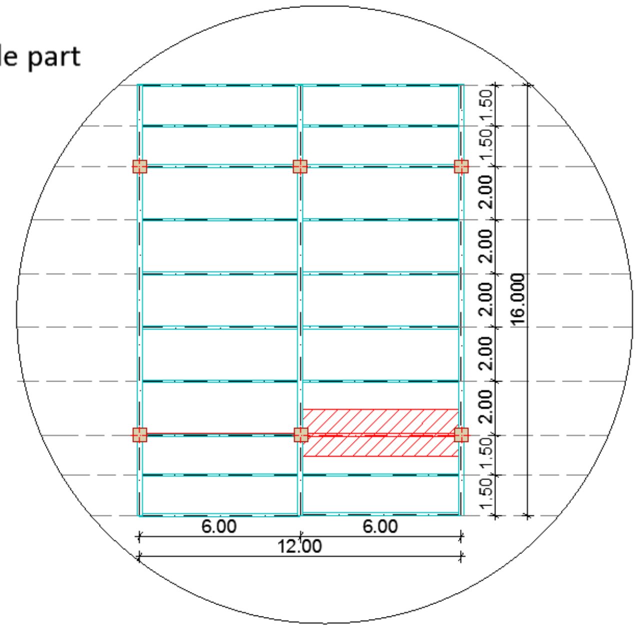
δmax = δ1 + δ2 = 10.42 mm + 12.53 mm= 22.95 mm

Deflection check

δmax = L/250 = 6000/250 = 24.00 mm > 22.95 mm - satisfied

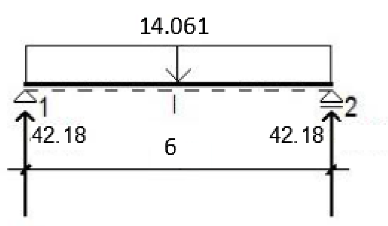
Floor.Secondary border beam in middle part

Position on the floor plan

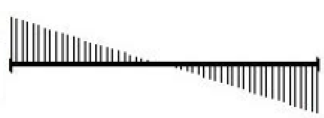


Length of the beam	L	6.00 m
Span of the tributary area	span	1.75 m
Tributary area	L x (span)	10.5m2
Permanent load:	G:	
Due to floor weight	Gfl	2.38 kN/m2
Due to secondary beam weight	Gsb (IPE 220 = 26.2 kg/m)	0.262 KN/m
Due to internal wall weight	Gf	2.10 KN/m
Variable load	Qs	2 KN/m2
Total dead load	Gtot = span x Gfl + Gsb +Gf Gtot =1.75 m x 2.38 kN/m2 + 0.262KN/m + 2.10KN/m	6.527 KN/m
Total variable load	Qtot = span x Qs Qtot = 1.75 m x 2.00 kN/m2	3.5KN/m
Design actions combination at SLS	Fd = 1.00 x Gtot + 1.00 x Qtot 1.00 x 6.527 kN/m + 1.00 x 3.5 kN/m	10.027KN/m
Design actions combination at ULS	Fd = 1.35 x Gtot + 1.50 x Qtot 1.35 x 6.527 kN/m + 1.50 x 3.5 kN/m	14.061Kn/m

Diagrams / ULS

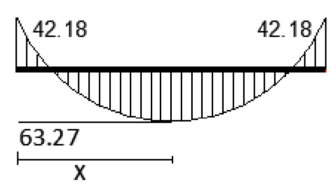


Shear
P x L / 2 = 42.18



Moment
P x L2 / 12= 42.18

Msd



Maximum bending moment

Msd = Fd x L²/ 8 = 14.061kN/m x (6 m)²/ 8 = 63.27 kNm

$$W = \frac{M}{f_{yd}} = \frac{63.27 \cdot 10^{6}}{275} = 230.0 \text{ cm}^{3}$$ Designed profile IPE 220

Designed profile IPE 220 / 252.0 cm3

h	b	a	e	r	Peso	Sezione	Momenti di inerzia		Moduli di resistenza	
							Jx	Jy	Wx	Wy
mm	mm	mm	mm	mm	kg/m	cm2	cm4	cm4	cm3	cm3
80	46	3,8	5,2	5	6,0	7,64	80,14	8,49	20,03	3,69
100	55	4,1	5,7	7	8,1	10,32	171,0	15,92	34,20	5,79
120	64	4,4	6,3	7	10,4	13,21	317,8	27,67	52,96	8,65
140	73	4.7	6,9	7	12,9	16,43	541,2	44,92	77,32	12,31
160	82	5,0	7,4	9	15,8	20,09	869,3	68,31	108,7	16,66
180	91	5,3	8,0	9	18,8	23,95	1.317	100,9	146,3	22,16
200	100	5,6	8,5	12	22,4	28,48	1.943	142,4	194,3	28,47
220	110	5,9	9,2	12	26,2	33,37	2.772	204,9	252,0	37,25
240	120	6,2	9,8	15	30,7	39,12	3.892	283,6	324,3	47,27
270	135	6,6	10,2	15	36,1	45,95	5.790	419,9	428,9	62,20
300	150	7,1	10,7	15	42,2	53,81	8.356	603,8	557,1	80,50
330	160	7,5	11,5	18	49,1	62,61	11.770	788,1	713,1	98,52
360	170	8,0	12,7	18	57,1	72,73	16.270	1.043	903,6	122,8
400	180	8,6	13,5	21	66,3	84,46	23.130	1.318	1.156	146,4
450	190	9,4	14,6	21	77,6	98,82	33.740	1.676	1.500	176,4
500	200	10,2	16,0	21	90,7	115,5	48.200	2.142	1.928	214,2

Deflection check

Deflection maximum allowed at SLS: floors generally

δmax = L/250 = 6000/250 = 24 mm

Deflection for uniformly distributed load

δ = 5/384 x (Fk x L3) / (E x ly)

Fk = Gk x L

E = elasticity modulus of steel = 210 MPa = 210 000 N/mm2

ly = 2.772 cm4 for IPE 220

Deflection for dead load

Fk = Gk x L = 6.527kN/m x 6.00 m = 39.16 kN

δ1= 5/384 x [39.16 x 103 N x (6000 mm)3] / (210 000 N/mm2 x 2772 x 104 mm4) = 13.96 mm

- Deflection for live load

Fk = Qk x L = 3.5 kN/m x 6.00 m = 21 kN

δ2 = 5/384 x [21x 103 N x (60000 mm)3] / (210 000 N/mm2 x 2772 x 104mm4) = 8.48 mm

Maximum deflection of secondary beam

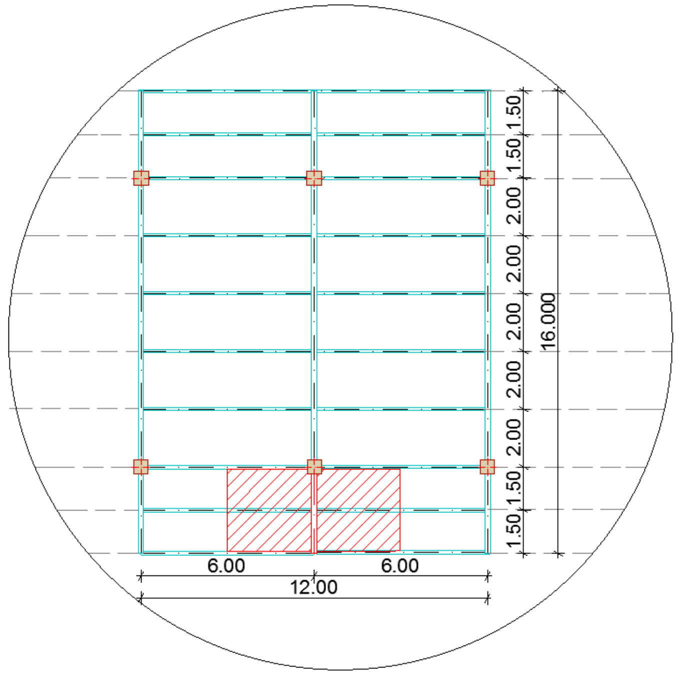
δmax = δ1 + δ2 = 13.96 mm + 8.48 mm= 22.44 mm

Deflection check

δmax = L/250 = 6000/250 = 24.00 mm > 22.44 mm - satisfied

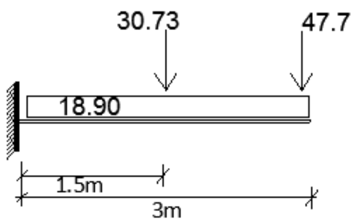
Floor. Primary beam in cantiliver

Position on the floor plan



Length of the beam	L	3.00 m
Span of the tributary area	span	6.00 m
Tributary area	L x (span)	18.00m^2
Due to primary beam weight	Gpl (IPE 400 = 66. kg/m)	0.663 KN/m
Due to secondary beam dead load	3.794 x 1.35 x 6 (point load at ULS)	30.73 KN
Due to secondary border beam load	5.89 x 1.35 x 6 (point load at ULS)	47.7 KN
Variable load	Qs	2 KN/m2
Total dead load	Gpl	0.663 KN/m
Total variable load	Qtot = span x Qs Qtot = 6 m x 2.00 kN/m2	12KN/m
Design actions combination at SLS	Fd = 1.00 x Gtot + 1.00 x Qtot 1.00 x 0.663kN/m + 1.00 x12 kN/m	12.663 KN/m
Design actions combination at ULS	Fd = 1.35 x Gtot + 1.50 x Qtot 1.35 x 0.663 kN/m + 1.50 x 12 kN/m	18.895KN/m

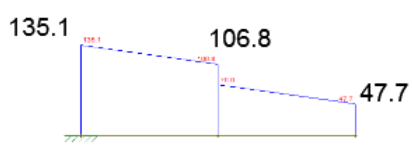
Diagrams / ULS



Maximum bending moment

Msd = 274.2

Shear



$$W = \frac{M}{f_{yd}} = \frac{274.2 \cdot 10^{6}}{275} = 997 \text{ cm}^{3}$$

Designed profile IPE 400

Designed profile IPE 330 / 713.1.0 cm3

							Momenti di inerzia		Moduli di resistenza	
h mm	b mm	a mm	e mm	r mm	Peso kg/m	Sezione cm2	Jx cm4	Jy cm4	Wx cm3	Wy cm3
80	46	3,8	5,2	5	6,0	7,64	80,14	8,49	20,03	3,69
100	55	4,1	5,7	7	8,1	10,32	171,0	15,92	34,20	5,79
120	64	4,4	6,3	7	10,4	13,21	317,8	27,67	52,96	8,65
140	73	4.7	6,9	7	12,9	16,43	541,2	44,92	77,32	12,31
160	82	5,0	7,4	9	15,8	20,09	869,3	68,31	108,7	16,66
180	91	5,3	8,0	9	18,8	23,95	1.317	100,9	146,3	22,16
200	100	5,6	8,5	12	22,4	28,48	1.943	142,4	194,3	28,47
220	110	5,9	9,2	12	26,2	33,37	2.772	204,9	252,0	37,25
240	120	6,2	9,8	15	30,7	39,12	3.892	283,6	324,3	47,27
270	135	6,6	10,2	15	36,1	45,95	5.790	419,9	428,9	62,20
300	150	7,1	10,7	15	42,2	53,81	8.356	603,8	557,1	80,50
330	160	7,5	11,5	18	49,1	62,61	11.770	788,1	713,1	98,52
360	170	8,0	12,7	18	57,1	72,73	16.270	1.043	903,6	122,8
400	180	8,6	13,5	21	66,3	84,46	23.130	1.318	1.156	146,4
450	190	9,4	14,6	21	77,6	98,82	33.740	1.676	1.500	176,4
500	200	10,2	16,0	21	90,7	115,5	48.200	2.142	1.928	214,2

Deflection check

Deflection maximum allowed at SLS: floors generally
δmax = L/250 = 3000/250 = 12 mm

Deflection for uniformly distributed load
δ = 5/384 x (Fk x L3) / (E x ly)
Fk = Gk x L
E = elasticity modulus of steel = 210 MPa = 210 000 N/mm2
ly = 23130 cm4 for IPE 400

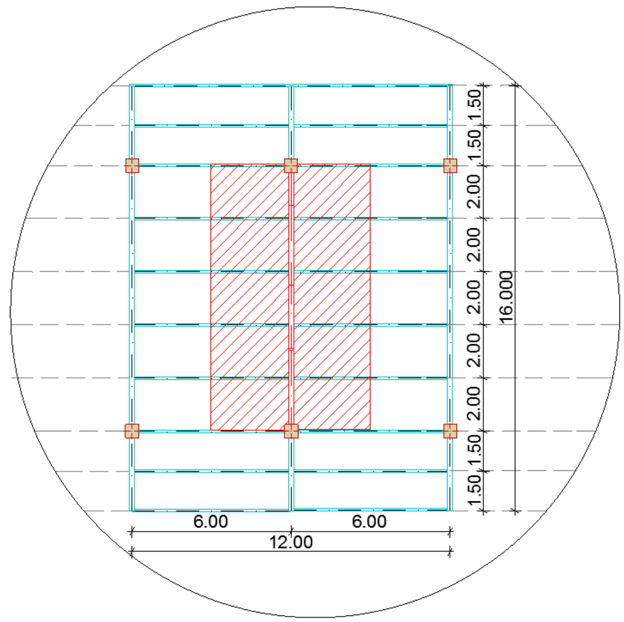
Deflection for dead load
Fk = Gk x L = 26.80kN/m x 3.00 m = 80.419kN
δ1= 5/384 x [80.419 x 103 N x (3000 mm)3] / (210 000 N/mm2 x 23130x 104 mm4) = 7.23 mm
- Deflection for live load
Fk = Qk x L = 12 kN/m x 3.00 m = 36 kN
δ2 = 5/384 x [36x 103 N x (30000 mm)3] / (210 000 N/mm2 x 23130 x 104mm4) = 2.724 mm

Maximum deflection of secondary beam
δmax = δ1 + δ2 = 7.23 mm + 2.724 mm= 9.95 mm

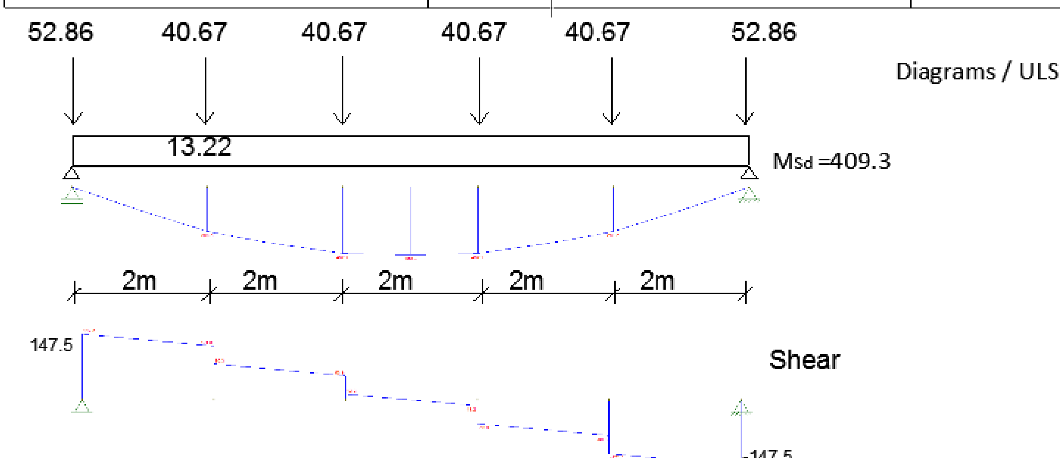
Deflection check
δmax = L/250 = 3000/250 = 12.00 mm > 9.95 mm - satisfied

Floor. Primary beam in midle part

Position on the floor plan



Length of the beam	L	10.00 m
Span of the tributary area	span	6.00 m
Tributary area	L x (span)	$60.00 m^2$
Permanent load:	G:	
Due to primary beam weight	Gpl (IPE 500 = 90.7 kg/m)	0.907 KN/m
Due to secondary middle load	5.022 x 1.35 x 6 (point load at ULS)	40.67 KN
Due to secondary border middle	6.527 x 1.35 x 6(point load at ULS)	52.86 KN
Internal partition	Gw	$0.65 KN/m^2$
Variable load	Qs	2 KN/m2
Total dead load		0.907 KN/m
Total variable load	Qtot = span x Qs Qtot = 6 m x 2.00 kN/m2	12KN/m
Design actions combination at SLS	Fd = 1.00 x Gtot + 1.00 x Qtot 1.00 x 0.907 kN/m + 1.00 x 12 kN/m	12.907 KN/m
Design actions combination at ULS	Fd = 1.35 x Gtot + 1.50 x Qtot 1.35 x 0.907 kN/m + 1.50 x 12 kN/m	13.224KN/m



Maximum bending moment

Msd = 409.3 kNm

$$W = \frac{M}{f_{yd}} = \frac{410 \cdot 10^{6}}{275} = 1500 \text{ cm}^{3}$$

Designed profile IPE 500

Designed profile IPE 500 /1.928 cm3

h mm	b mm	a mm	e mm	r mm	Peso kg/m	Sezione cm2	Momenti di inerzia		Moduli di resistenza	
							Jx cm4	Jy cm4	Wx cm3	Wy cm3
80	46	3,8	5,2	5	6,0	7,64	80,14	8,49	20,03	3,69
100	55	4,1	5,7	7	8,1	10,32	171,0	15,92	34,20	5,79
120	64	4,4	6,3	7	10,4	13,21	317,8	27,67	52,96	8,65
140	73	4.7	6,9	7	12,9	16,43	541,2	44,92	77,32	12,31
160	82	5,0	7,4	9	15,8	20,09	869,3	68,31	108,7	16,66
180	91	5,3	8,0	9	18,8	23,95	1.317	100,9	146,3	22,16
200	100	5,6	8,5	12	22,4	28,48	1.943	142,4	194,3	28,47
220	110	5,9	9,2	12	26,2	33,37	2.772	204,9	252,0	37,25
240	120	6,2	9,8	15	30,7	39,12	3.892	283,6	324,3	47,27
270	135	6,6	10,2	15	36,1	45,95	5.790	419,9	428,9	62,20
300	150	7,1	10,7	15	42,2	53,81	8.356	603,8	557,1	80,50
330	160	7,5	11,5	18	49,1	62,61	11.770	788,1	713,1	98,52
360	170	8,0	12,7	18	57,1	72,73	16.270	1.043	903,6	122,8
400	180	8,6	13,5	21	66,3	84,46	23.130	1.318	1.156	146,4
450	190	9,4	14,6	21	77,6	98,82	33.740	1.676	1.500	176,4
500	200	10,2	16,0	21	90,7	115,5	48.200	2.142	1.928	214,2

Deflection check

Deflection maximum allowed at SLS: floors generally

δmax = L/250 = 10000/250 = 40 mm

Deflection for uniformly distributed load

δ = 5/384 x (Fk x L3) / (E x ly)

Fk = Gk x L

E = elasticity modulus of steel = 210 MPa = 210 000 N/mm2

ly = 48200 cm4 for IPE 500

Deflection for dead load

Fk = Gk x L = 28kN/m x 10.00 m = 280kN

δ1= 5/384 x [280 x 103 N x (10000 mm)3] / (210 000 N/mm2 x 48200 x 104 mm4) = 18.23 mm

- Deflection for live load

Fk = Qk x L = 12 kN/m x 10.00 m = 120 kN

δ2 = 5/384 x [120x 103 N x (100000 mm)3] / (210 000 N/mm2 x 48200 x 104mm4) = 14.79 mm

Maximum deflection of secondary beam

δmax = δ1 + δ2 = 18.23 mm + 14.79 mm= 33.02 mm

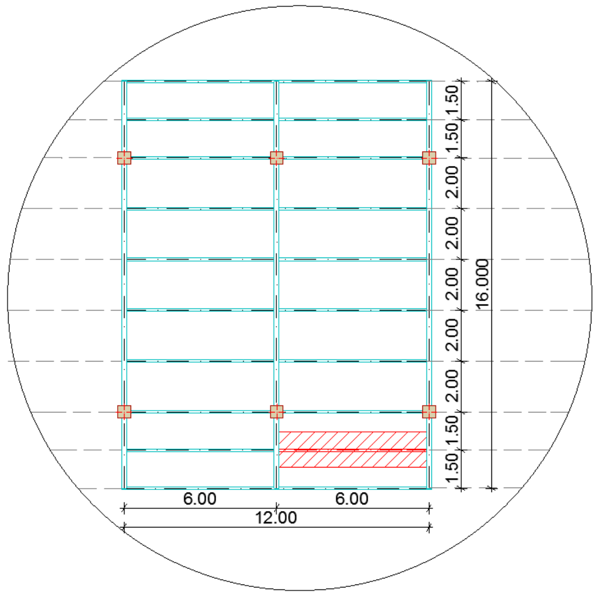
Deflection check

δmax = L/250 = 10000/250 = 40.00 mm > 33.02 mm - satisfied

2.Roof Calculations

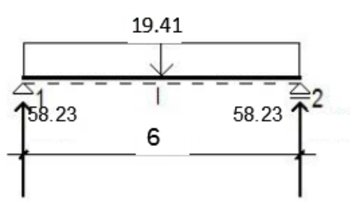
Secondary cantiliver middle beam

Position on the floor plan

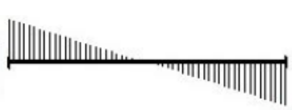


Length of the beam	L	6.00 m
Span of the tributary area	span	1.50 m
Tributary area	L x (span)	9.00m2
Permanent load:	G:	
Due to floor weight	Gfl	5.833 kN/m2
Due to secondary beam weight	Gsb (IPE 240 = 30.7 kg/m)	0.307 KN/m
Variable load Snow weight	Qp Qs	2 KN/m2 1.2KN/m2
Total dead load	Gtot = span x Gfl + Gsb Gtot =1.50 m x 5.833 kN/m2 + 0.307KN/m	9.05KN/m
Total variable load	Qtot = span x (Qs+Qp) Qtot = 1.50 m x (2.00 kN/m2 + 1.2KN/m)	4.8KN/m
Design actions combination at SLS	Fd = 1.00 x Gtot + 1.00 x Qtot 1.00 x 9.05kN/m + 1.00 x4.8 kN/m	13.85 KN/m
Design actions combination at ULS	Fd = 1.35 x Gtot + 1.50 x Qtot 1.35 x 9.05 kN/m + 1.50 x 4.8 kN/m	19.41KN/m

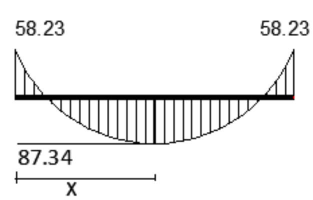
Diagrams / ULS



Shear
P x L / 2 = 58.23



Moment
P x L2 / 12= 58.23



Maximum bending moment

Msd = Fd x L²/ 8 = 19.41kN/m x (6m)^2 / 8 = 87.345 kNm

$$W = \frac{M}{f_{yd}} = \frac{87.345 \cdot 10^{6}}{275} = 317.61 \text{ cm}^3$$ Designed profile IPE 240

Designed profile IPE 240/324.3 cm^3

h	b	a	e	r	Peso	Sezione	Momenti di inerzia		Moduli di resistenza	
							Jx	Jy	Wx	Wy
mm	mm	mm	mm	mm	kg/m	cm2	cm4	cm4	cm3	cm3
80	46	3,8	5,2	5	6,0	7,64	80,14	8,49	20,03	3,69
100	55	4,1	5,7	7	8,1	10,32	171,0	15,92	34,20	5,79
120	64	4,4	6,3	7	10,4	13,21	317,8	27,67	52,96	8,65
140	73	4.7	6,9	7	12,9	16,43	541,2	44,92	77,32	12,31
160	82	5,0	7,4	9	15,8	20,09	869,3	68,31	108,7	16,66
180	91	5,3	8,0	9	18,8	23,95	1.317	100,9	146,3	22,16
200	100	5,6	8,5	12	22,4	28,48	1.943	142,4	194,3	28,47
220	110	5,9	9,2	12	26,2	33,37	2.772	204,9	252,0	37,25
240	120	6,2	9,8	15	30,7	39,12	3.892	283,6	324,3	47,27
270	135	6,6	10,2	15	36,1	45,95	5.790	419,9	428,9	62,20
300	150	7,1	10,7	15	42,2	53,81	8.356	603,8	557,1	80,50
330	160	7,5	11,5	18	49,1	62,61	11.770	788,1	713,1	98,52
360	170	8,0	12,7	18	57,1	72,73	16.270	1.043	903,6	122,8
400	180	8,6	13,5	21	66,3	84,46	23.130	1.318	1.156	146,4
450	190	9,4	14,6	21	77,6	98,82	33.740	1.676	1.500	176,4
500	200	10,2	16,0	21	90,7	115,5	48.200	2.142	1.928	214,2

Deflection check

Deflection maximum allowed at SLS: floors generally

δmax = L/250 = 6000/250 = 24 mm

Deflection for uniformly distributed load

δ = 5/384 x (Fk x L3) / (E x ly)

Fk = Gk x L

E = elasticity modulus of steel = 210 MPa = 210 000 N/mm2

ly = 3892 cm4 for IPE 240

Deflection for dead load

Fk = Gk x L =9.05kN/m x 6.00 m = 54.3kN

δ1= 5/384 x [54.3x 103 N x (6000 mm)3] / (210 000 N/mm2 x 3892 x 104 mm4) = 11.89 mm

- Deflection for live load

Fk = Qk x L = 4.8 kN/m x 6.00 m =28.8 kN

δ2 = 5/384 x [28.8x 103 N x (60000 mm)3] / (210 000 N/mm2 x 3892 x 104mm4) = 6.58 mm

Maximum deflection of secondary beam

δmax = δ1 + δ2 = 11.89 mm + 6.58 mm= 18.47 mm

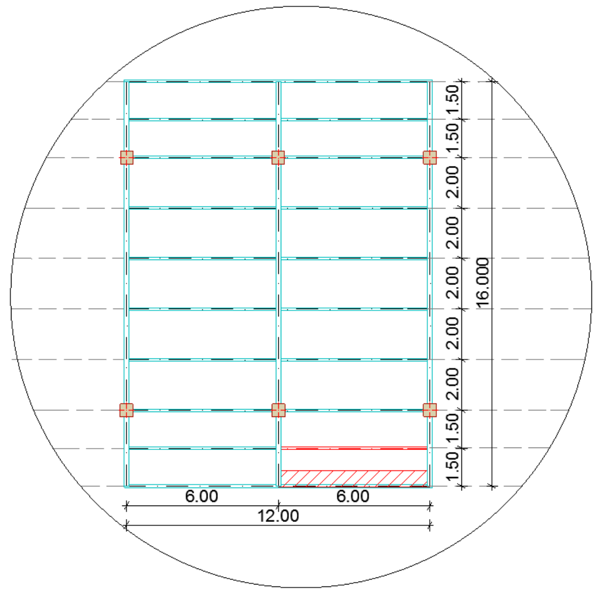
Deflection check

δmax = L/250 = 6000/250 =24.00 mm > 18.47 mm - satisfied

Roof Calculations

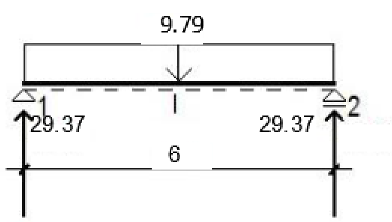
Secondary cantiliver border beam

Position on the floor plan

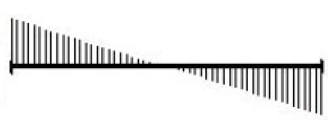


Length of the beam	L	6.00 m
Span of the tributary area	span	0.75 m
Tributary area	L x (span)	9.00m2
Permanent load:	G:	
Due to floor weight	Gfl	5.833 kN/m2
Due to secondary beam weight	Gsb (IPE 200 = 22.4 kg/m)	0.224 KN/m
Variable load Snow weight	Qp Qs	2 KN/m2 1.2 KN/m2
Total dead load	Gtot = span x Gfl + Gsb Gtot =0.75 m x 5.833 kN/m2 + 0.224KN/m	4.59KN/m
Total variable load	Qtot = span x (Qs+Qp) Qtot = 0.75 m x (2.00 kN/m2 + 1.2KN/m)	2.4 KN/m
Design actions combination at SLS	Fd = 1.00 x Gtot + 1.00 x Qtot 1.00 x 4.59kN/m + 1.00 x2.4 kN/m	6.99 KN/m
Design actions combination at ULS	Fd = 1.35 x Gtot + 1.50 x Qtot 1.35 x 4.59 kN/m + 1.50 x 2.4 kN/m	9.79 KN/m

Diagrams / ULS

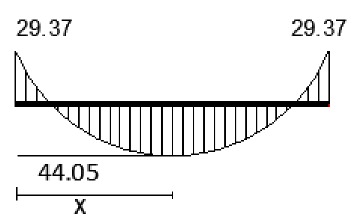


Shear
P x L / 2 = 29.37



Moment
P x L2 / 12= 29.37

Msd



Maximum bending moment

$M_{sd} = F_d \times L^2 / 8 = 9.79\ \text{kN/m} \times (6\text{m})^2 / 8 = 44.05\ \text{kNm}$

$$W = \frac{M}{f_{yd}} = \frac{44.05 \cdot 10^6}{275} = 160.2\ \text{cm}^3$$

Designed profile IPE 200

Designed profile IPE 200/194.3 cm3

h mm	b mm	a mm	e mm	r mm	Peso kg/m	Sezione cm2	Momenti di inerzia Jx cm4	Jy cm4	Moduli di resistenza Wx cm3	Wy cm3
80	46	3,8	5,2	5	6,0	7,64	80,14	8,49	20,03	3,69
100	55	4,1	5,7	7	8,1	10,32	171,0	15,92	34,20	5,79
120	64	4,4	6,3	7	10,4	13,21	317,8	27,67	52,96	8,65
140	73	4.7	6,9	7	12,9	16,43	541,2	44,92	77,32	12,31
160	82	5,0	7,4	9	15,8	20,09	869,3	68,31	108,7	16,66
180	91	5,3	8,0	9	18,8	23,95	1.317	100,9	146,3	22,16
200	100	5,6	8,5	12	22,4	28,48	1.943	142,4	194,3	28,47
220	110	5,9	9,2	12	26,2	33,37	2.772	204,9	252,0	37,25
240	120	6,2	9,8	15	30,7	39,12	3.892	283,6	324,3	47,27
270	135	6,6	10,2	15	36,1	45,95	5.790	419,9	428,9	62,20
300	150	7,1	10,7	15	42,2	53,81	8.356	603,8	557,1	80,50
330	160	7,5	11,5	18	49,1	62,61	11.770	788,1	713,1	98,52
360	170	8,0	12,7	18	57,1	72,73	16.270	1.043	903,6	122,8
400	180	8,6	13,5	21	66,3	84,46	23.130	1.318	1.156	146,4
450	190	9,4	14,6	21	77,6	98,82	33.740	1.676	1.500	176,4
500	200	10,2	16,0	21	90,7	115,5	48.200	2.142	1.928	214,2

Deflection check

Deflection maximum allowed at SLS: floors generally

δmax = L/250 = 6000/250 = 24 mm

Deflection for uniformly distributed load

δ = 5/384 x (Fk x L3) / (E x ly)

Fk = Gk x L

E = elasticity modulus of steel = 210 MPa = 210 000 N/mm2

ly = 1943 cm4 for IPE 200

Deflection for dead load

Fk = Gk x L =4.59kN/m x 6.00 m = 27.54kN

δ1= 5/384 x [27.54x 103 N x (6000 mm)3] / (210 000 N/mm2 x 1943 x 104 mm4) = 10.29 mm

- Deflection for live load

Fk = Qk x L = 2.4kN/m x 6.00 m =14.4kN

δ2 = 5/384 x [14.4x 103 N x (60000 mm)3] / (210 000 N/mm2 x 1943 x 104mm4) = 7.54mm

Maximum deflection of secondary beam

δmax = δ1 + δ2 = 10.29 mm + 7.54 mm= 17.83 mm

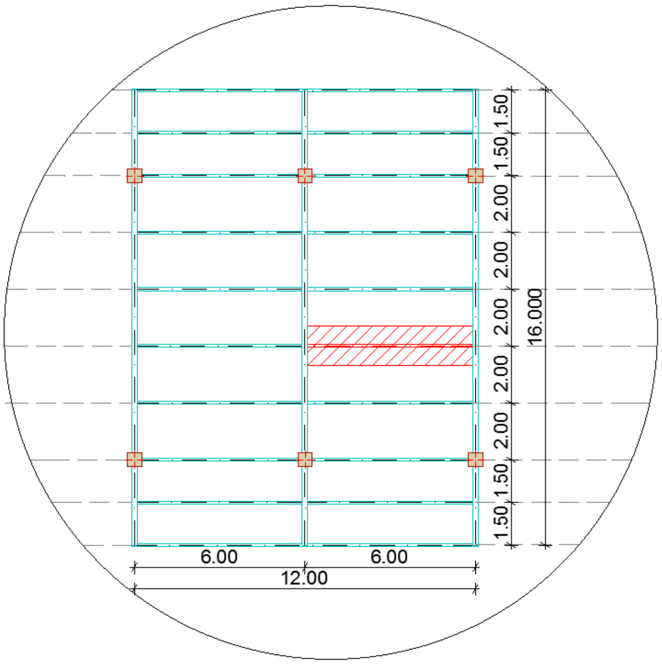
Deflection check

δmax = L/250 = 6000/250 =24.00 mm > 17.83 mm - satisfied

Roof

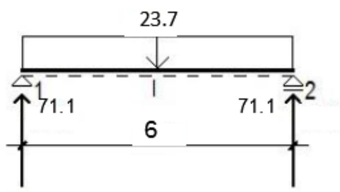
Secondary middle beam

Position on the floor plan

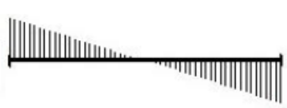


Length of the beam	L	6.00 m
Span of the tributary area	span	2.00 m
Tributary area	L x (span)	12.00m2
Permanent load:	G:	
Due to floor weight	Gfl	5.833 kN/m2
Due to secondary beam weight Due to solar pannels weight	Gsb (IPE 270 = 36.1 kg/m) Guv	0.361 KN/m 0.2 KN/m
Variable load Snow weight	Qp Qs	2 KN/m2 1.2KN/m2
Total dead load	Gtot = span x Gfl + Gsb +Gf Gtot =2.00 m x 5.833 kN/m2 + 0.361KN/m + 2.10KN/m	12.227KN/m
Total variable load	Qtot = span x (Qs+Qp) Qtot = 2.00m x (2.00 kN/m2 + 1.2KN/m)	6.4KN/m
Design actions combination at SLS	Fd = 1.00 x Gtot + 1.00 x Qtot 1.00 x 12.227 kN/m + 1.00 x6.4 kN/m	18.62KN/m
Design actions combination at ULS	Fd = 1.35 x Gtot + 1.50 x Qtot 1.35 x 12.227 kN/m + 1.50 x 4.8 kN/m	23.7Kn/m

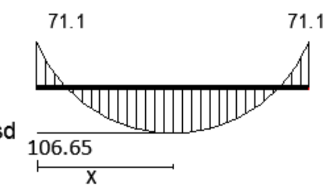
Diagrams / ULS



Shear
P x L / 2 = 71.1



Moment
P x L2 / 12= 71.1



Maximum bending moment

Msd = Fd x L²/ 8 = 23.7 kN/m x (6m)²/ 8 = 106.65 kNm

$$W = \frac{M}{f_{yd}} = \frac{106.65 \cdot 10^{6}}{275} = 387.8 \text{ cm}^{3}$$ Designed profile IPE 270

Designed profile IPE 270/428.9 cm3

h	b	a	e	r	Peso	Sezione	Momenti di inerzia		Moduli di resistenza	
h mm	b mm	a mm	e mm	r mm	Peso kg/m	Sezione cm2	Jx cm4	Jy cm4	Wx cm3	Wy cm3
80	46	3,8	5,2	5	6,0	7,64	80,14	8,49	20,03	3,69
100	55	4,1	5,7	7	8,1	10,32	171,0	15,92	34,20	5,79
120	64	4,4	6,3	7	10,4	13,21	317,8	27,67	52,96	8,65
140	73	4.7	6,9	7	12,9	16,43	541,2	44,92	77,32	12,31
160	82	5,0	7,4	9	15,8	20,09	869,3	68,31	108,7	16,66
180	91	5,3	8,0	9	18,8	23,95	1.317	100,9	146,3	22,16
200	100	5,6	8,5	12	22,4	28,48	1.943	142,4	194,3	28,47
220	110	5,9	9,2	12	26,2	33,37	2.772	204,9	252,0	37,25
240	120	6,2	9,8	15	30,7	39,12	3.892	283,6	324,3	47,27
270	135	6,6	10,2	15	36,1	45,95	5.790	419,9	428,9	62,20
300	150	7,1	10,7	15	42,2	53,81	8.356	603,8	557,1	80,50
330	160	7,5	11,5	18	49,1	62,61	11.770	788,1	713,1	98,52
360	170	8,0	12,7	18	57,1	72,73	16.270	1.043	903,6	122,8
400	180	8,6	13,5	21	66,3	84,46	23.130	1.318	1.156	146,4
450	190	9,4	14,6	21	77,6	98,82	33.740	1.676	1.500	176,4
500	200	10,2	16,0	21	90,7	115,5	48.200	2.142	1.928	214,2

Deflection check

Deflection maximum allowed at SLS: floors generally

$\delta max = L/250 = 6000/250 = 24$ mm

Deflection for uniformly distributed load

$\delta = 5/384 \times (Fk \times L3) / (E \times ly)$

Fk = Gk x L

E = elasticity modulus of steel = 210 MPa = 210 000 N/mm2

ly = 5790 cm4 for IPE 270

Deflection for dead load

Fk = Gk x L = 12.22kN/m x 6.00 m = 73.32kN

$\delta 1$= 5/384 x [73.32x 103 N x (6000 mm)3] / (210 000 N/mm2 x 5790 x 104 mm4) = 12.45 mm

- Deflection for live load

Fk = Qk x L = 6.4 kN/m x 6.00 m =38.4 kN

$\delta 2$ = 5/384 x [38.4x 103 N x (60000 mm)3] / (210 000 N/mm2 x 5790 x 104mm4) = 7.88 mm

Maximum deflection of secondary beam

$\delta max = \delta 1 + \delta 2$ = 12.45 mm + 7.88 mm= 20.33 mm

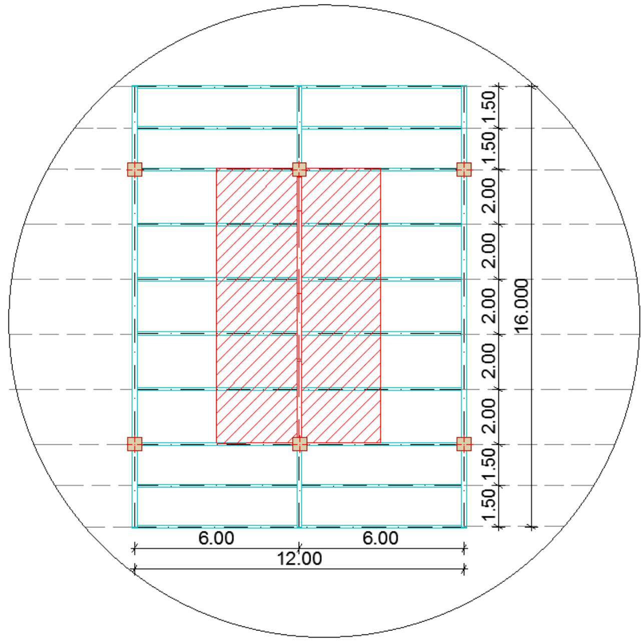
Deflection check

δmax = L/250 = 6000/250 =24.00 mm > 20.33 mm - satisfied

Roof

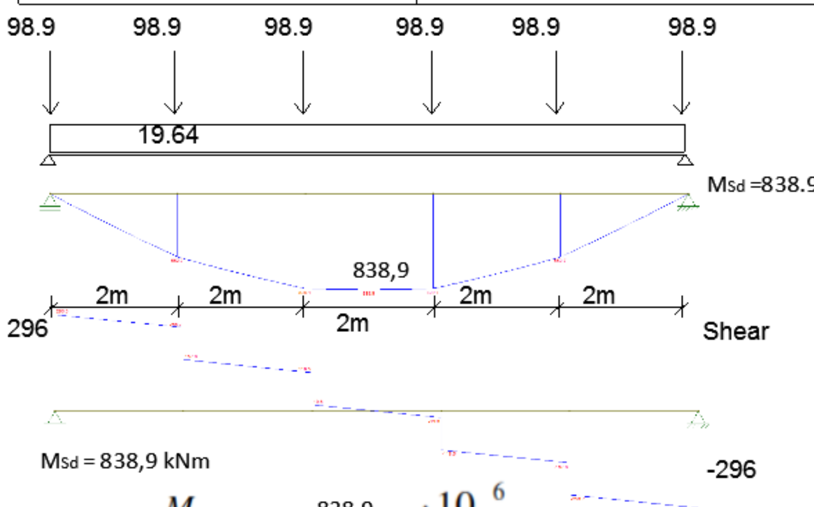
Primary middle beam

Position on the floor plan



Length of the beam	L	10.00 m
Span of the tributary area	span	6.00 m
Tributary area	L x (span)	$60.00m^2$
Permanent load:	G:	
Due to primary beam weight	Gpl (IPE 600 = 122 kg/m)	1.22 KN/m
Due to secondary beam load	12.22 x 1.35 x 6 (point load at ULS)	98.9 KN
Variable load	Qs	2 KN/m2
Total dead load		1.22 KN/m
Total variable load	Qtot = span x Qs Qtot = 6 m x 2.00 kN/m2	12KN/m
Design actions combination at SLS	Fd = 1.00 x Gtot + 1.00 x Qtot 1.00 x 1.22 kN/m + 1.00 x 12 kN/m	13.22 KN/m
Design actions combination at ULS	Fd = 1.35 x Gtot + 1.50 x Qtot 1.35 x 1.22 kN/m + 1.50 x 12 kN/m	19.64KN/m

Diagrams / ULS



Msd = 838,9 kNm

$$W = \frac{M}{f_{yd}} = \frac{838{,}9 \cdot 10^6}{275} = 3050 \text{ cm}^3$$

Designed profile IPE 600

Designed profile IPE 600/3070 cm3

Deflection check

Deflection maximum allowed at SLS: floors generally

δmax = L/250 = 6000/250 = 40 mm

Deflection for uniformly distributed load

δ = 5/384 x (Fk x L3) / (E x ly)

Fk = Gk x L

Identification	Section properties, Static data												
	strong axis x-x						weak axis y-y						
	Ix	Wel.x	Wpl.x	ix	Avy	Sx	Iy	Wel.y	Wpl.y	iy	Ss	It	Iw
	cm4	cm3	cm3	cm	cm2	cm3	cm4	cm3	cm3	cm	mm	cm4	
IPE 80	80,1	20,0	23,2	3,24	3,58	12	8,49	3,69	5,8	1,05	20,1	0,70	0,12
IPE 100	171	34,2	39,4	4,07	5,08	20	15,9	5,79	9,2	1,24	23,7	1,20	0,35
IPE 120	318	53,0	60,7	4,90	6,31	30	27,7	8,65	13,6	1,45	25,2	1,74	0,89
IPE 140	541	77,3	88,3	5,74	7,64	44,9	44,9	12,3	19,3	1,65	26,7	2,45	1,98
IPE 160	869	109,0	124,0	6,58	9,66	62	68,3	16,7	26,1	1,84	30,3	3,60	3,96
IPE 180	1317	146,0	166,0	7,42	11,30	83	101,0	22,2	34,6	2,05	31,8	4,79	7,43
IPE 200	1943	194,0	221,0	8,26	14,00	110	142,0	28,5	44,6	2,24	36,7	6,98	13,00
IPE 220	2772	252,0	285,0	9,11	15,90	143	205,0	37,3	58,1	2,48	38,4	9,07	22,70
IPE 240	3892	324,0	367,0	9,97	19,10	183	284,0	47,3	73,9	2,69	43,4	12,90	37,40
IPE 270	5790	429,0	484,0	11,20	22,10	242	420,0	62,2	97,0	3,02	44,6	15,90	70,60
IPE 300	8356	557,0	628,0	12,50	25,70	314	604,0	80,5	125,0	3,35	46,1	20,10	126,00
IPE 330	11770	713,0	804,0	13,70	30,80	402	788,0	98,5	154,0	3,55	51,6	28,20	199,00
IPE 360	16270	904,0	1019,0	15,00	35,10	510	1043,0	123,0	191,0	3,79	54,5	37,30	314,00
IPE 400	23130	1160,0	1307,0	16,60	42,70	654	1318,0	146,0	229,0	3,95	60,2	51,10	490,00
IPE 450	33740	1500,0	1702,0	18,48	50,90		1676,0	176,4	276,0	4,12	63,2	66,90	791,00
IPE 500	48200	1930,0	2194,0	20,43	59,90		2142,0	214,2	336,0	4,31	66,8	89,30	1249,00
IPE 550	67120	2440,0	2787,0	22,40	72,30		2668,0	254,1	401,0	4,45	73,6	123,00	1884,00
IPE 600	92080	3070,0	3512,0	24,30	83,8		3387,0	307,9	486,0	4,66	78,1	165,00	2846,00

E = elasticity modulus of steel = 210 MPa = 210 000 N/mm2

ly = 92080 cm4 for IPE 600

Deflection for dead load

Fk = Gk x L = 36.77kN/m x 10m = 367.7kN

δ1= 5/384 x [367.7x 103 N x (10000 mm)3] / (210 000 N/mm2 x 92080 x 104 mm4) = 26.55 mm

- Deflection for live load

Fk = Qk x L = 12 kN/m x 10.00 m =120 kN

δ2 = 5/384 x [120.4x 103 N x (100000 mm)3] / (210 000 N/mm2 x 92080 x 104mm4) = 13.22 mm

Maximum deflection of secondary beam

δmax = δ1 + δ2 = 26.55 mm + 13.22 mm= 39.77 mm

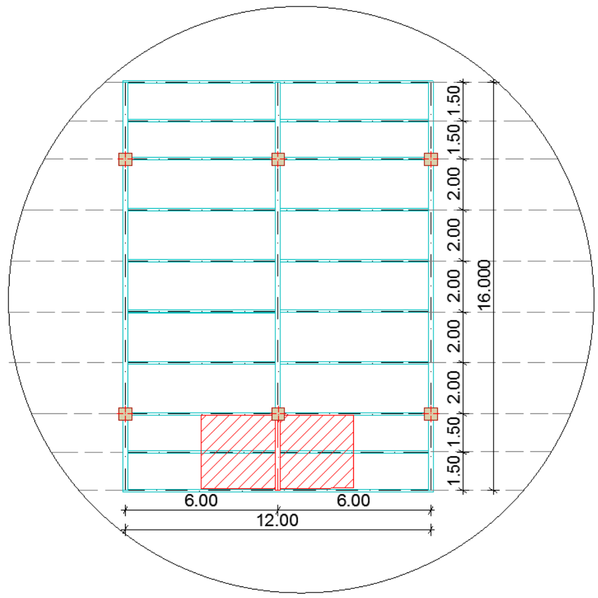
Deflection check

δmax = L/250 = 10000/250 =40.00 mm > 39.77 mm - satisfied

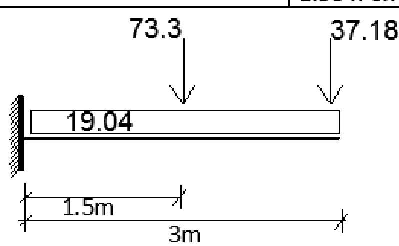
Roof

Primary cantilever beam

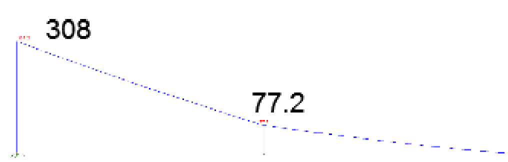
Position on the floor plan



Length of the beam	L	3.00 m
Span of the tributary area	span	6.00 m
Tributary area	L x (span)	18.00m^2
Due to primary beam weight	Gpl (IPE 450 = 77.6 kg/m)	0.776 KN/m
Due to secondary beam dead load	9,05 x 6 x 1.35 (point load at ULS)	73.3 KN
Due to secondary border beam load	4.59 x 6 x 1.35 (point load at ULS)	37.18KN
Variable load	Qs	2 KN/m2
Total dead load	Gpl	0.776 KN/m
Total variable load	Qtot = span x Qs Qtot = 6 m x 2.00 kN/m2	12KN/m
Design actions combination at SLS	Fd = 1.00 x Gtot + 1.00 x Qtot 1.00 x 0.776kN/m + 1.00 x12 kN/m	12.776 KN/m
Design actions combination at ULS	Fd = 1.35 x Gtot + 1.50 x Qtot 1.35 x 0.776 kN/m + 1.50 x 12 kN/m	19.04KN/m



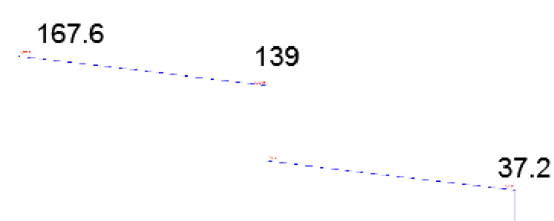
Diagrams / ULS



Maximum bending moment

Msd = 308

Shear



$$W = \frac{M}{f_{yd}} = \frac{308 \cdot 10^6}{275} = 1120 \text{ cm}^3$$

Designed profile IPE 450

Designed profile IPE 450/ 1500 cm3

h mm	b mm	a mm	e mm	r mm	Peso kg/m	Sezione cm2	Momenti di inerzia Jx cm4	Jy cm4	Moduli di resistenza Wx cm3	Wy cm3
80	46	3,8	5,2	5	6,0	7,64	80,14	8,49	20,03	3,69
100	55	4,1	5,7	7	8,1	10,32	171,0	15,92	34,20	5,79
120	64	4,4	6,3	7	10,4	13,21	317,8	27,67	52,96	8,65
140	73	4.7	6,9	7	12,9	16,43	541,2	44,92	77,32	12,31
160	82	5,0	7,4	9	15,8	20,09	869,3	68,31	108,7	16,66
180	91	5,3	8,0	9	18,8	23,95	1.317	100,9	146,3	22,16
200	100	5,6	8,5	12	22,4	28,48	1.943	142,4	194,3	28,47
220	110	5,9	9,2	12	26,2	33,37	2.772	204,9	252,0	37,25
240	120	6,2	9,8	15	30,7	39,12	3.892	283,6	324,3	47,27
270	135	6,6	10,2	15	36,1	45,95	5.790	419,9	428,9	62,20
300	150	7,1	10,7	15	42,2	53,81	8.356	603,8	557,1	80,50
330	160	7,5	11,5	18	49,1	62,61	11.770	788,1	713,1	98,52
360	170	8,0	12,7	18	57,1	72,73	16.270	1.043	903,6	122,8
400	180	8,6	13,5	21	66,3	84,46	23.130	1.318	1.156	146,4
450	190	9,4	14,6	21	77,6	98,82	33.740	1.676	1.500	176,4
500	200	10,2	16,0	21	90,7	115,5	48.200	2.142	1.928	214,2

Deflection check

Deflection maximum allowed at SLS: floors generally

δmax = L/250 = 3000/250 = 12 mm

Deflection for uniformly distributed load

δ = 5/384 x (Fk x L3) / (E x ly)

Fk = Gk x L

E = elasticity modulus of steel = 210 MPa = 210 000 N/mm2

ly = 33740 cm4 for IPE 450

Deflection for dead load

Fk = Gk x L =36.35kN/m x 3m = 109.05kN

δ1= 5/384 x [109.05x 103 N x (3000 mm)3] / (210 000 N/mm2 x 33740x 104 mm4) = 5.23 mm

- Deflection for live load

Fk = Qk x L = 12 kN/m x 3.00 m =36 kN

δ2 = 5/384 x [36x 103 N x (30000 mm)3] / (210 000 N/mm2 x 33740 x 104mm4) = 3.67 mm

Maximum deflection of secondary beam

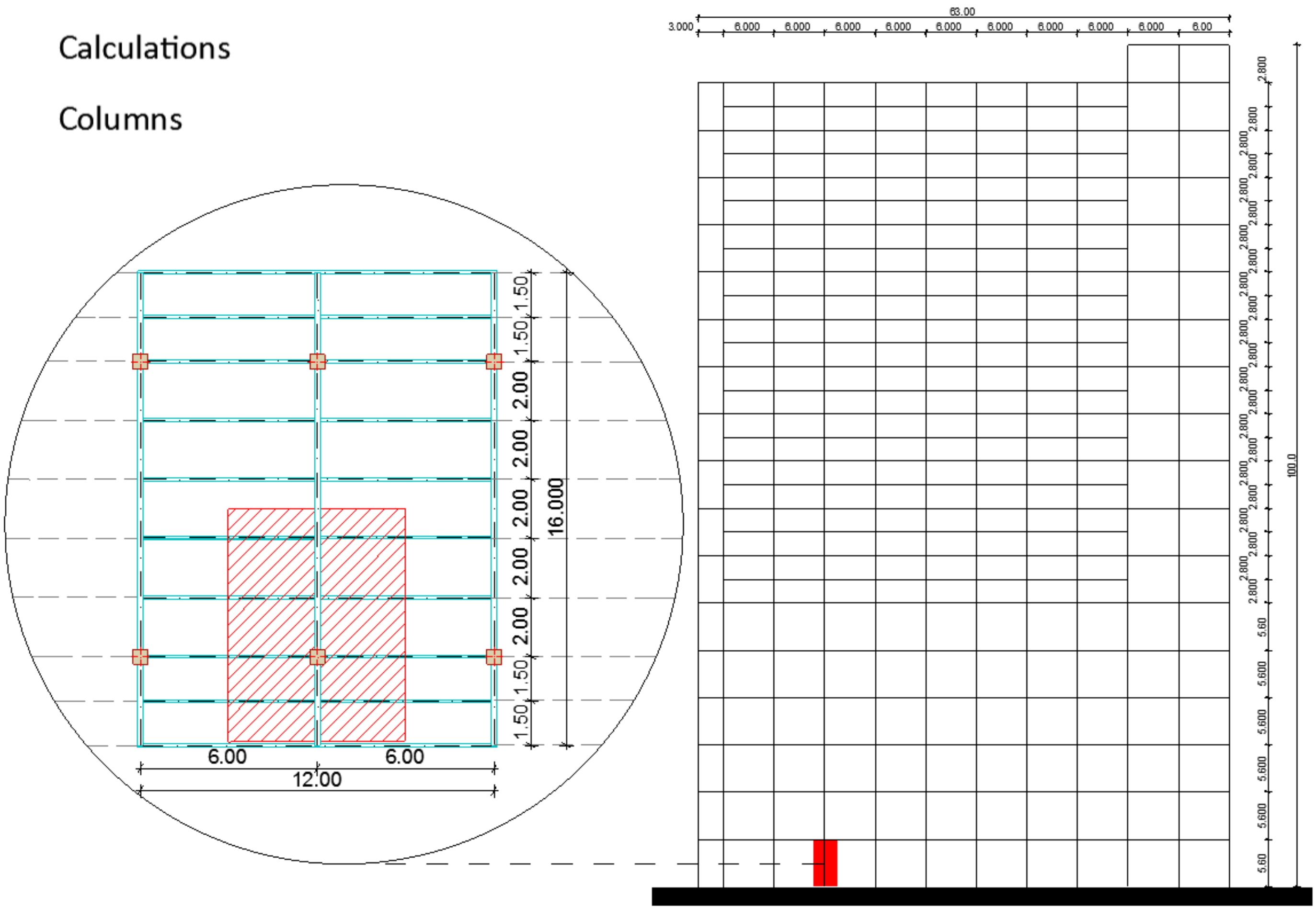
δmax = δ1 + δ2 = 5.23 mm + 2.67 mm= 8.9 mm

Deflection check

δmax = L/250 = 3000/250 =12.00 mm > 8.9 mm - satisfied

Calculations

Columns



Length of the column	L	5.60m
Flore	1	
Roof: beams' reactions at ULS: (Roof)	Primary middle / 2 + Primary cantiliver	462.5KN
Floor: beams' reactions at ULS: (Floor standards)	Primary middle /2 + Primary cantiliver	315.43 KN
Floor: beams' reactions at ULS: (duplex) floors on second level in dupelx floors	Primary middle /2	195.3 KN
Total compression force	Rroof (462.5) +16 x floor (315.43) +11 x duplex (195.3)	7657KN
Total compression force	NEd	7657KN

Combination at the ULS NEd=1.5*7657 kN=11485kN

$$A = \frac{P}{f_{yd}} = \frac{11485 \cdot 10^{3}}{261.9} = 438 \text{ cm}^{3}$$ Designed profile HEM 1000

Cross-section of HEM 1000 - 444.2 cm2

Sigla HEM	b mm	h mm	a mm	e mm	r mm	Peso kg/m	Sezione cm2	Momenti di inerzia Jx cm4	Jy cm4	Moduli di resistenza Wx cm3	Wy cm3	Raggi di inerzia ix cm	iy cm
100	106	120	12,0	20,0	12	48,8	53,24	1.143	399,2	194,0	75,31	4,63	2,74
120	126	140	12,5	21,0	12	52,1	66,41	2.018	702,8	288,2	111,6	5,51	3,25
140	146	160	13,0	22,0	12	63,2	80,56	3.291	1.144	411,4	156,8	6,39	3,77
160	166	180	14,0	23,0	15	76,2	97,05	5.098	1.759	566,5	211,9	7,25	4,26
180	186	200	14,5	24,0	15	88,9	113,3	7.483	2.580	748,3	277,4	8,13	4,77
200	206	220	15,0	25,0	18	103	131,3	10.640	3.651	967,4	354,5	9,00	5,27
220	226	240	15,5	26,0	18	117	149,4	14.600	5.012	1.217	443,5	9,89	5,79
240	248	270	18,0	32,0	21	157	199,6	24.290	8.153	1.799	657,5	11,03	6,39
260	268	290	18,0	32,5	24	172	219,6	31.310	10.450	2.159	779,7	11,94	6,90
280	288	310	18,5	33,0	24	189	240,2	39.550	13.160	2.551	914,1	12,83	7,40
300	310	340	21,0	39,0	27	238	303,1	59.200	19.400	3.482	1.252	13,98	8,00
320	309	359	21,0	40,0	27	245	312,0	68.130	19.710	3.796	1.276	14,78	7,95
340	309	377	21,0	40,0	27	248	315,8	76.370	19.710	4.052	1.276	15,55	7,90
360	308	395	21,0	40,0	27	250	318,8	84.870	19.520	4.297	1.268	16,32	7,83
400	307	432	21,0	40,0	27	256	325,8	104.100	19.340	4.820	1.260	17,88	7,70
450	307	478	21,0	40,0	27	263	335,4	131.500	19.340	5.501	1.260	19,80	7,59
500	306	524	21,0	40,0	27	270	344,3	161.900	19.150	6.180	1.252	21,69	7,46
550	306	572	21,0	40,0	27	278	354,4	198.000	19.160	6.923	1.252	23.64	7,35
600	305	620	21,0	40,0	27	285	363,7	237.400	18.980	7.660	1.244	25,55	7,22
650	305	668	21,0	40,0	27	293	373,7	281.700	18.980	8.433	1.245	27,45	7,13
700	304	716	21,0	40,0	27	301	383,0	329.300	18.800	9.198	1.237	29,32	7.01
800	303	814	21,0	40,0	30	317	404,3	442.600	18.630	10.870	1.230	33,09	6,79
900	302	910	21,0	40,0	30	333	423,6	570.400	18.450	12.540	1.222	36,70	6,60
1000	302	1.008	21,0	40,0	30	349	444,2	722.300	18.460	14.330	1.222	40,32	6,45

Buckling check

$$N_{Ed} < N_{b,rd}$$

$$N_{b,rd} = \chi \cdot A \cdot f_{yd}$$

$$\chi = \frac{1}{\left(\varphi + (\varphi^2 - \bar{\lambda}^2)^{0,5}\right)} \leq 1$$

$$\varphi = 0,5 \cdot \left(1 + \alpha \cdot (\bar{\lambda} - 0,2) + \bar{\lambda}^2\right)$$

$$\bar{\lambda} = \sqrt{\frac{A \cdot f_{yk}}{N_{cr}}}$$

$$N_{cr} = \frac{\pi^2 \cdot E \cdot I_{min}}{L_0{}^2}$$

1. $N_{cr} = \dfrac{3.14 \times 21\,000 \times 1846}{5.60^2} = 388150$

2. $\bar{\lambda} = \sqrt{\dfrac{4442 \times 275}{388150}} = 1.77$

3. $\varphi = 0.5(\ 1 + 0.21(1.77-0.2) + 3.13) = 2.22$

4. $x = \dfrac{1}{2.22 + (\ 4.92 - 3.13)^{0.5} = 3.55} = 0.28$

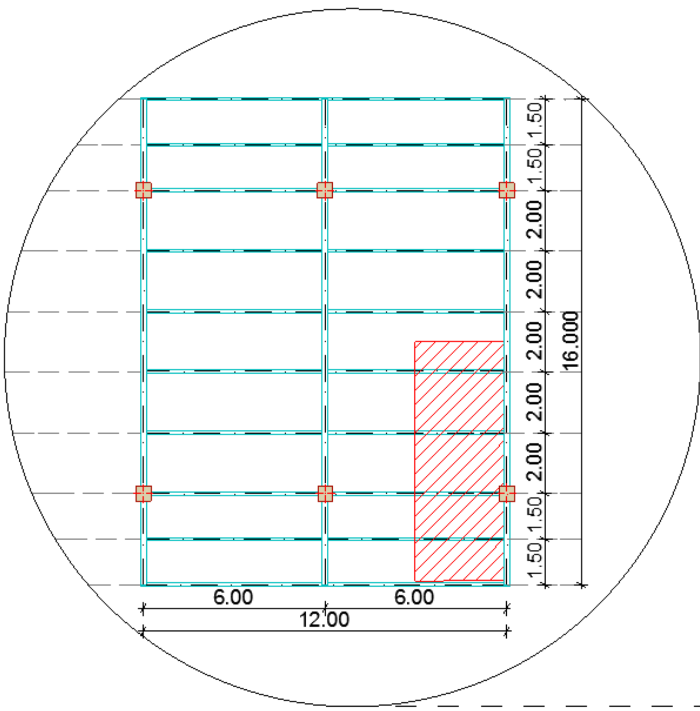
5. $N_{brd} = 0.28 \times 4442 \times 261.9 = 32574$

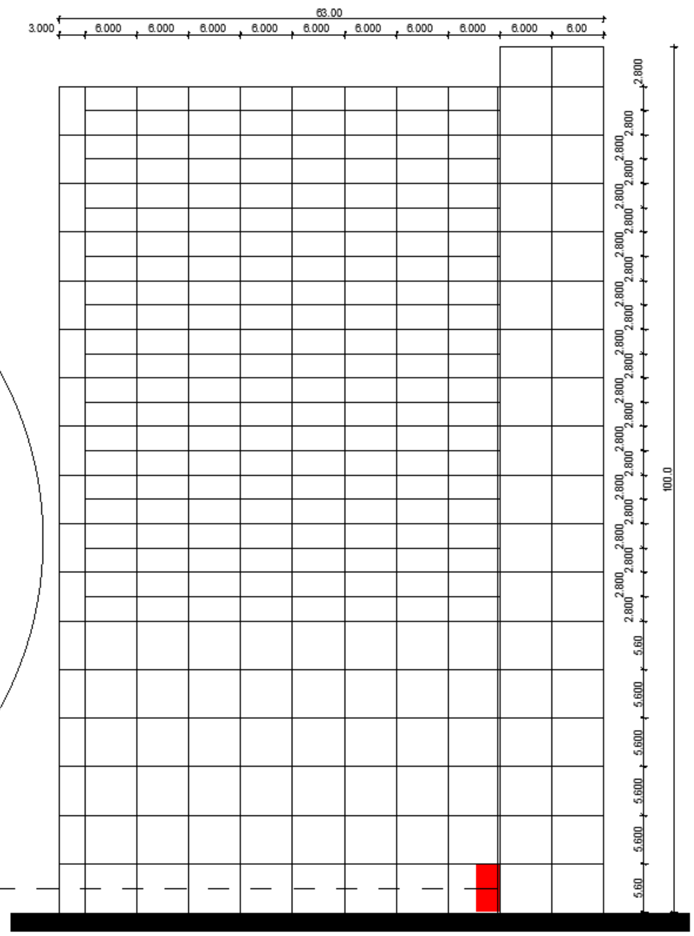
Buckling check

$N_{brd} = 32574$ KN > N_{ED} (11485 KN) - satisfied

Calculations

Border edge Column





Length of the column	L	5.60m
Flore	1	
Roof: beams' reactions at ULS: (Roof) /2	Primary middle / 2 + Primary cantiliver	231.25KN
Floor: beams' reactions at ULS:(Floor standards)/2	Primary middle /2 + Primary cantiliver	157.7 KN
Floor: beams' reactions at ULS: (duplex) /2 floors on second level in dupelx floors	Primary middle /2	97.6 KN
Total compression force	Rroof (231.25) +16 x floor (157.7) +11 x duplex (97.6)	3828KN
Total compression force	NEd	3828KN

Combination at the ULS NEd=1.5*3828 kN=5741kN

$$A = \frac{P}{f_{yd}} = \frac{5741 \cdot 10^{3}}{261.9} = 220 \text{ cm}^{3}$$ **Designed profile HEM 280**

Cross-section of HEM 280- 240.2 cm2

Sigla HEM	b mm	h mm	a mm	e mm	r mm	Peso kg/m	Sezione cm2	Momenti di inerzia Jx cm4	Jy cm4	Moduli di resistenza Wx cm3	Wy cm3	Raggi di inerzia ix cm	iy cm
100	106	120	12,0	20,0	12	48,8	53,24	1.143	399,2	194,0	75,31	4,63	2,74
120	126	140	12,5	21,0	12	52,1	66,41	2.018	702,8	288,2	111,6	5,51	3,25
140	146	160	13,0	22,0	12	63,2	80,56	3.291	1.144	411,4	156,8	6,39	3,77
160	166	180	14,0	23,0	15	76,2	97,05	5.098	1.759	566,5	211,9	7,25	4,26
180	186	200	14,5	24,0	15	88,9	113,3	7.483	2.580	748,3	277,4	8,13	4,77
200	206	220	15,0	25,0	18	103	131,3	10.640	3.651	967,4	354,5	9,00	5,27
220	226	240	15,5	26,0	18	117	149,4	14.600	5.012	1.217	443,5	9,89	5,79
240	248	270	18,0	32,0	21	157	199,6	24.290	8.153	1.799	657,5	11,03	6,39
260	268	290	18,0	32,5	24	172	219,6	31.310	10.450	2.159	779,7	11,94	6,90
280	288	310	18,5	33,0	24	189	240,2	39.550	13.160	2.551	914,1	12,83	7,40
300	310	340	21,0	39,0	27	238	303,1	59.200	19.400	3.482	1.252	13,98	8,00
320	309	359	21,0	40,0	27	245	312,0	68.130	19.710	3.796	1.276	14,78	7,95
340	309	377	21,0	40,0	27	248	315,8	76.370	19.710	4.052	1.276	15,55	7,90
360	308	395	21,0	40,0	27	250	318,8	84.870	19.520	4.297	1.268	16,32	7,83
400	307	432	21,0	40,0	27	256	325,8	104.100	19.340	4.820	1.260	17,88	7,70
450	307	478	21,0	40,0	27	263	335,4	131.500	19.340	5.501	1.260	19,80	7,59
500	306	524	21,0	40,0	27	270	344,3	161.900	19.150	6.180	1.252	21,69	7,46
550	306	572	21,0	40,0	27	278	354,4	198.000	19.160	6.923	1.252	23.64	7,35
600	305	620	21,0	40,0	27	285	363,7	237.400	18.980	7.660	1.244	25,55	7,22
650	305	668	21,0	40,0	27	293	373,7	281.700	18.980	8.433	1.245	27,45	7,13
700	304	716	21,0	40,0	27	301	383,0	329.300	18.800	9.198	1.237	29,32	7.01
800	303	814	21,0	40,0	30	317	404,3	442.600	18.630	10.870	1.230	33,09	6,79
900	302	910	21,0	40,0	30	333	423,6	570.400	18.450	12.540	1.222	36,70	6,60
1000	302	1.008	21,0	40,0	30	349	444,2	722.300	18.460	14.330	1.222	40,32	6,45

Buckling check

$$N_{Ed} < N_{b,rd}$$

$$N_{b,rd} = \chi \cdot A \cdot f_{yd}$$

$$\chi = \frac{1}{\left(\varphi + \left(\varphi^2 - \bar{\lambda}^2\right)^{0,5}\right)} \leq 1$$

$$\varphi = 0,5 \cdot \left(1 + \alpha \cdot \left(\bar{\lambda} - 0,2\right) + \bar{\lambda}^2\right)$$

$$\bar{\lambda} = \sqrt{\frac{A \cdot f_{yk}}{N_{cr}}}$$

$$N_{cr} = \frac{\pi^2 \cdot E \cdot I_{min}}{L_0{}^2}$$

1. $N_{cr} = \dfrac{3.14 \text{ x } 21\,000 \text{ x } 1316}{5.60^2} = 276712$

2. $\bar{\lambda} = \sqrt{\dfrac{2402 \text{ x } 275}{276712}} = 1.54$

3. $\varphi = 0.5(\ 1 + 0.34(1.54 - 0.2) + 2.37) = 1.91$

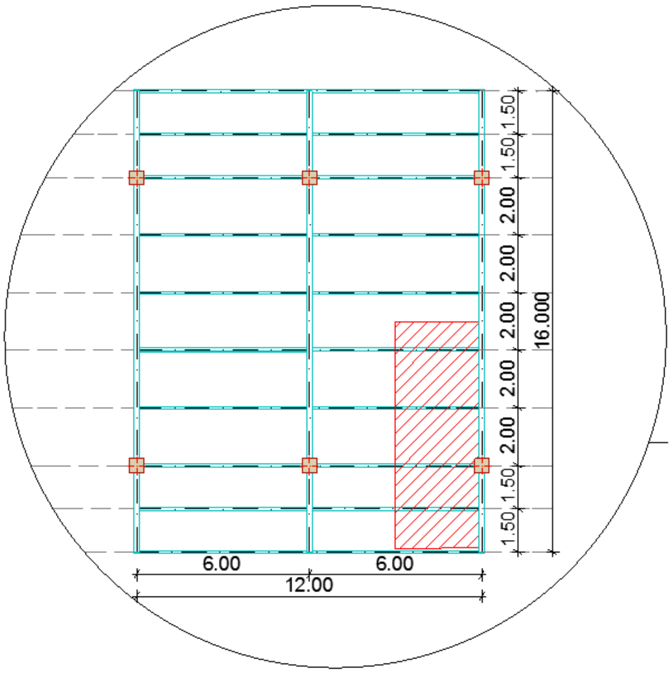
4. $\text{x} = \dfrac{1}{1.91 + (\ 3.65 - 2.37)^{0.5} = 3.04} = 0.32$

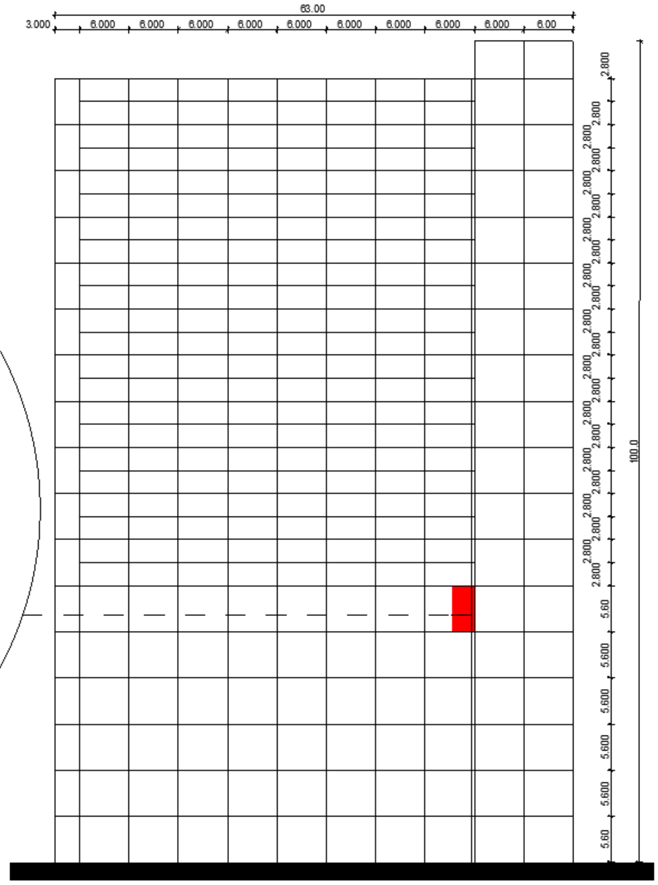
5. $N_{brd} = 0.32 \text{ x } 2402 \text{ x } 261.9 = 20130$

Buckling check
N_{brd} = 20130 KN > N_{ED} (5741KN) - satisfied

Calculations

Border edge Column





Length of the column	L	5.60m
Flore	1	
Roof: beams' reactions at ULS: (Roof) /2	Primary middle / 2 + Primary cantiliver	231.25KN
Floor: beams' reactions at ULS:(Floor standards)/2	Primary middle /2 + Primary cantiliver	157.7 KN
Floor: beams' reactions at ULS: (duplex) /2 floors on second level in dupelx floors	Primary middle /2	97.6 KN
Total compression force	Rroof (231.25) +11 x floor (157.7) +11 x duplex (97.6)	3828KN
Total compression force	NEd	3032KN

Combination at the ULS NEd=1.5*3032 kN=4548kN

$$A = \frac{P}{f_{yd}} = \frac{4548 \cdot 10^3}{261.9} = 173 \text{ cm}^3$$ Designed profile HEM 240

Cross-section of HEM 240- 199.6 cm2

Sigla HEM	b mm	h mm	a mm	e mm	r mm	Peso kg/m	Sezione cm2	Momenti di inerzia Jx cm4	Momenti di inerzia Jy cm4	Moduli di resistenza Wx cm3	Moduli di resistenza Wy cm3	Raggi di inerzia ix cm	Raggi di inerzia iy cm
100	106	120	12,0	20,0	12	48,8	53,24	1.143	399,2	194,0	75,31	4,63	2,74
120	126	140	12,5	21,0	12	52,1	66,41	2.018	702,8	288,2	111,6	5,51	3,25
140	146	160	13,0	22,0	12	63,2	80,56	3.291	1.144	411,4	156,8	6,39	3,77
160	166	180	14,0	23,0	15	76,2	97,05	5.098	1.759	566,5	211,9	7,25	4,26
180	186	200	14,5	24,0	15	88,9	113,3	7.483	2.580	748,3	277,4	8,13	4,77
200	206	220	15,0	25,0	18	103	131,3	10.640	3.651	967,4	354,5	9,00	5,27
220	226	240	15,5	26,0	18	117	149,4	14.600	5.012	1.217	443,5	9,89	5,79
240	248	270	18,0	32,0	21	157	199,6	24.290	8.153	1.799	657,5	11,03	6,39
260	268	290	18,0	32,5	24	172	219,6	31.310	10.450	2.159	779,7	11,94	6,90
280	288	310	18,5	33,0	24	189	240,2	39.550	13.160	2.551	914,1	12,83	7,40
300	310	340	21,0	39,0	27	238	303,1	59.200	19.400	3.482	1.252	13,98	8,00
320	309	359	21,0	40,0	27	245	312,0	68.130	19.710	3.796	1.276	14,78	7,95
340	309	377	21,0	40,0	27	248	315,8	76.370	19.710	4.052	1.276	15,55	7,90
360	308	395	21,0	40,0	27	250	318,8	84.870	19.520	4.297	1.268	16,32	7,83
400	307	432	21,0	40,0	27	256	325,8	104.100	19.340	4.820	1.260	17,88	7,70
450	307	478	21,0	40,0	27	263	335,4	131.500	19.340	5.501	1.260	19,80	7,59
500	306	524	21,0	40,0	27	270	344,3	161.900	19.150	6.180	1.252	21,69	7,46
550	306	572	21,0	40,0	27	278	354,4	198.000	19.160	6.923	1.252	23.64	7,35
600	305	620	21,0	40,0	27	285	363,7	237.400	18.980	7.660	1.244	25,55	7,22
650	305	668	21,0	40,0	27	293	373,7	281.700	18.980	8.433	1.245	27,45	7,13
700	304	716	21,0	40,0	27	301	383,0	329.300	18.800	9.198	1.237	29,32	7.01
800	303	814	21,0	40,0	30	317	404,3	442.600	18.630	10.870	1.230	33,09	6,79
900	302	910	21,0	40,0	30	333	423,6	570.400	18.450	12.540	1.222	36,70	6,60
1000	302	1.008	21,0	40,0	30	349	444,2	722.300	18.460	14.330	1.222	40,32	6,45

Buckling check

$$N_{Ed} < N_{b,rd}$$

$$N_{b,rd} = \chi \cdot A \cdot f_{yd}$$

$$\chi = \frac{1}{\left(\varphi + \left(\varphi^2 - \bar{\lambda}^2\right)^{0,5}\right)} \leq 1$$

$$\varphi = 0,5 \cdot \left(1 + \alpha \cdot (\bar{\lambda} - 0,2) + \bar{\lambda}^2\right)$$

$$\bar{\lambda} = \sqrt{\frac{A \cdot f_{yk}}{N_{cr}}}$$

$$N_{cr} = \frac{\pi^2 \cdot E \cdot I_{min}}{L_0{}^2}$$

1. $N_{cr} = \frac{3.14 \times 21\,000 \times 815}{5.60^2} = 171360$

2. $\bar{\lambda} = \sqrt{\frac{1996 \times 275}{171360}} = 1.78$

3. $\varphi = 0.5(\ 1 + 0.34(1.78-0.2) + 3.16) \ = 2.34$

4. $x = \frac{1}{2.34 + (\ 5.49 - 3.16)^{0.5} = 3.86} = 0.25$

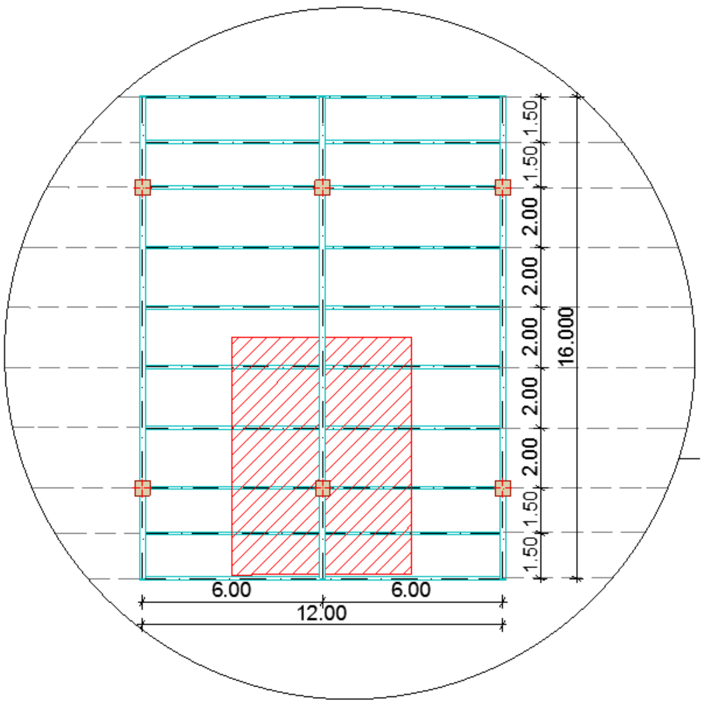
5. $N_{brd} = 0.25 \times 1996 \times 261.9 = 13542$

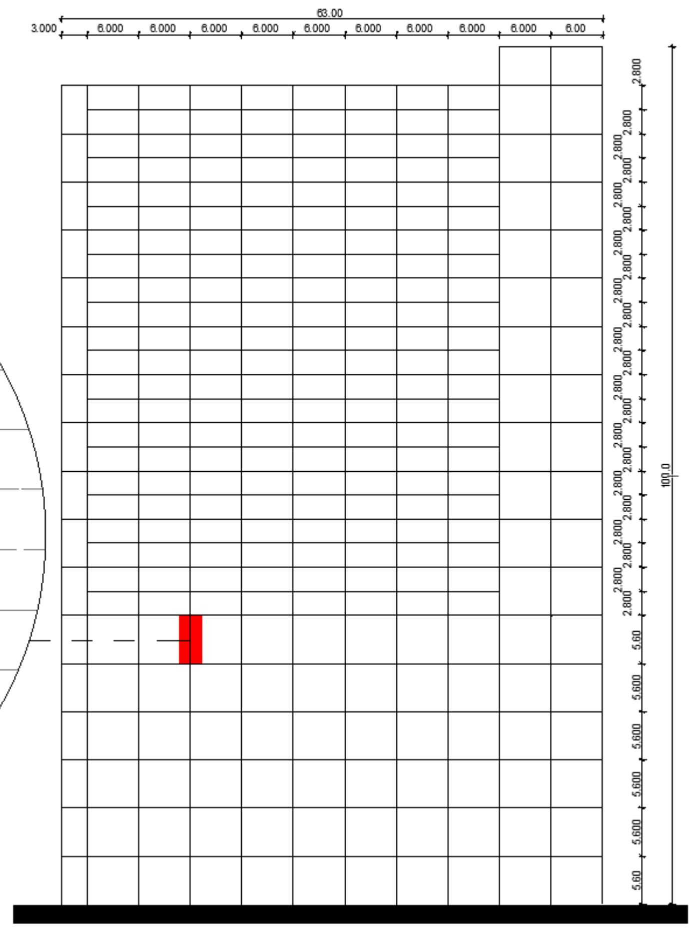
Buckling check

N_{brd} = 13542 KN > N_{ED} (4548KN) - satisfied

Calculations

Columns





Length of the column	L	5.60m
Flore	1	
Roof: beams' reactions at ULS: (Roof)	Primary middle / 2 + Primary cantiliver	462.5KN
Floor: beams' reactions at ULS: (Floor standards)	Primary middle /2 + Primary cantiliver	315.43 KN
Floor: beams' reactions at ULS: (duplex) floors on second level in dupelx floors	Primary middle /2	195.3 KN
Total compression force	Rroof (462.5) +11 x floor (315.43) +11 x duplex (195.3)	6080KN
Total compression force	NEd	6080KN

Combination at the ULS NEd=1.5*6080 kN=9120kN

$$A = \frac{P}{f_{yd}} = \frac{9120 \cdot 10^{3}}{261.9} = 348 \text{ cm}^{3}$$ Designed profile HEM 550

Cross-section of HEM 550 - 354.4 cm2

Sigla HEM	b mm	h mm	a mm	e mm	r mm	Peso kg/m	Sezione cm2	Momenti di inerzia Jx cm4	Jy cm4	Moduli di resistenza Wx cm3	Wy cm3	Raggi di inerzia ix cm	iy cm
100	106	120	12,0	20,0	12	48,8	53,24	1.143	399,2	194,0	75,31	4,63	2,74
120	126	140	12,5	21,0	12	52,1	66,41	2.018	702,8	288,2	111,6	5,51	3,25
140	146	160	13,0	22,0	12	63,2	80,56	3.291	1.144	411,4	156,8	6,39	3,77
160	166	180	14,0	23,0	15	76,2	97,05	5.098	1.759	566,5	211,9	7,25	4,26
180	186	200	14,5	24,0	15	88,9	113,3	7.483	2.580	748,3	277,4	8,13	4,77
200	206	220	15,0	25,0	18	103	131,3	10.640	3.651	967,4	354,5	9,00	5,27
220	226	240	15,5	26,0	18	117	149,4	14.600	5.012	1.217	443,5	9,89	5,79
240	248	270	18,0	32,0	21	157	199,6	24.290	8.153	1.799	657,5	11,03	6,39
260	268	290	18,0	32,5	24	172	219,6	31.310	10.450	2.159	779,7	11,94	6,90
280	288	310	18,5	33,0	24	189	240,2	39.550	13.160	2.551	914,1	12,83	7,40
300	310	340	21,0	39,0	27	238	303,1	59.200	19.400	3.482	1.252	13,98	8,00
320	309	359	21,0	40,0	27	245	312,0	68.130	19.710	3.796	1.276	14,78	7,95
340	309	377	21,0	40,0	27	248	315,8	76.370	19.710	4.052	1.276	15,55	7,90
360	308	395	21,0	40,0	27	250	318,8	84.870	19.520	4.297	1.268	16,32	7,83
400	307	432	21,0	40,0	27	256	325,8	104.100	19.340	4.820	1.260	17,88	7,70
450	307	478	21,0	40,0	27	263	335,4	131.500	19.340	5.501	1.260	19,80	7,59
500	306	524	21,0	40,0	27	270	344,3	161.900	19.150	6.180	1.252	21,69	7,46
550	306	572	21,0	40,0	27	278	354,4	198.000	19.160	6.923	1.252	23,64	7,35
600	305	620	21,0	40,0	27	285	363,7	237.400	18.980	7.660	1.244	25,55	7,22
650	305	668	21,0	40,0	27	293	373,7	281.700	18.980	8.433	1.245	27,45	7,13
700	304	716	21,0	40,0	27	301	383,0	329.300	18.800	9.198	1.237	29,32	7.01
800	303	814	21,0	40,0	30	317	404,3	442.600	18.630	10.870	1.230	33,09	6,79
900	302	910	21,0	40,0	30	333	423,6	570.400	18.450	12.540	1.222	36,70	6,60
1000	302	1.008	21,0	40,0	30	349	444,2	722.300	18.460	14.330	1.222	40,32	6,45

Buckling check

$$N_{Ed} < N_{b,rd}$$

$$N_{b,rd} = \chi \cdot A \cdot f_{yd}$$

$$\chi = \frac{1}{\left(\varphi + \left(\varphi^2 - \bar{\lambda}^2\right)^{0,5}\right)} \leq 1$$

$$\varphi = 0,5 \cdot \left(1 + \alpha \cdot \left(\bar{\lambda} - 0,2\right) + \bar{\lambda}^2\right)$$

$$\bar{\lambda} = \sqrt{\frac{A \cdot f_{yk}}{N_{cr}}}$$

$$N_{cr} = \frac{\pi^2 \cdot E \cdot I_{min}}{{L_0}^2}$$

1. $N_{cr} = \dfrac{3.14 \times 21\,000 \times 1916}{5.60^2} = 402873$

2. $\bar{\lambda} = \sqrt{\dfrac{3544 \times 275}{402873}} = 1.55$

3. $\varphi = 0.5(\ 1 + 0.21(1.55\text{-}0.2) + 2.41) = 1.84$

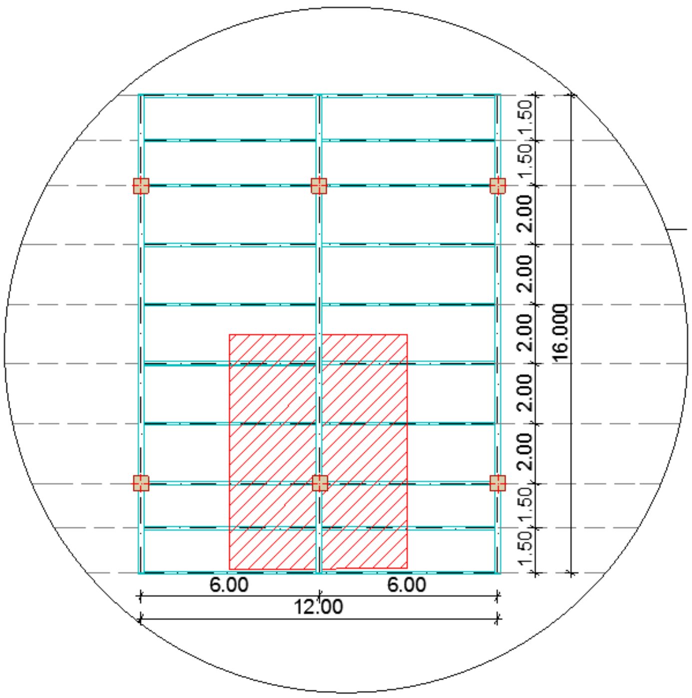
4. $x = \dfrac{1}{1.84 + (\ 3.38 - 2.40)^{0.5} = 2.82} = 0.35$

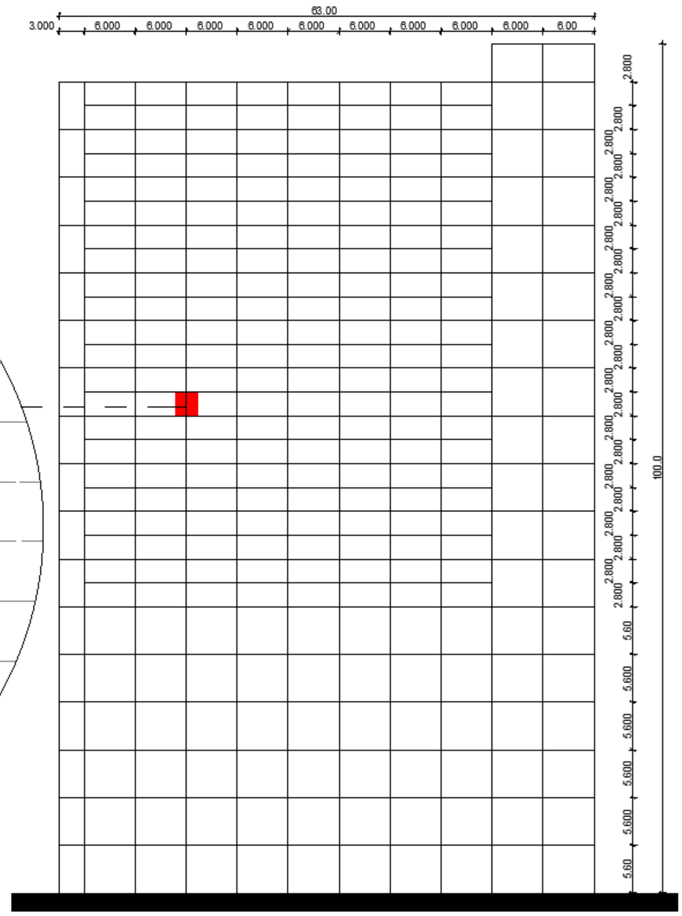
5. $N_{brd} = 0.35 \times 3544 \times 261.9 = 32486$

Buckling check
$N_{brd} = 32486$ KN > N_{ED} (9120 KN) - satisfied

Calculations

Columns





Length of the column	L	2.80m
Flore	1	
Roof: beams' reactions at ULS: (Roof)	Primary middle / 2 + Primary cantiliver	462.5KN
Floor: beams' reactions at ULS: (Floor standards)	Primary middle /2 + Primary cantiliver	315.43 KN
Floor: beams' reactions at ULS: (duplex) floors on second level in dupelx floors	Primary middle /2	195.3 KN
Total compression force	Rroof (462.5) +6 x floor (315.43) +7 x duplex (195.3)	3722KN
Total compression force	NEd	3722KN

Combination at the ULS NEd=1.5*3722 kN=5583.3kN

$$A = \frac{P}{f_{yd}} = \frac{5583.3 \cdot 10^{3}}{261.9} = 213 \text{ cm}^{3}$$ Designed profile HEM 260

Cross-section of HEM 260- 219.6 cm2

Sigla HEM	b mm	h mm	a mm	e mm	r mm	Peso kg/m	Sezione cm2	Momenti di inerzia Jx cm4	Jy cm4	Moduli di resistenza Wx cm3	Wy cm3	Raggi di inerzia ix cm	iy cm
100	106	120	12,0	20,0	12	48,8	53,24	1.143	399,2	194,0	75,31	4,63	2,74
120	126	140	12,5	21,0	12	52,1	66,41	2.018	702,8	288,2	111,6	5,51	3,25
140	146	160	13,0	22,0	12	63,2	80,56	3.291	1.144	411,4	156,8	6,39	3,77
160	166	180	14,0	23,0	15	76,2	97,05	5.098	1.759	566,5	211,9	7,25	4,26
180	186	200	14,5	24,0	15	88,9	113,3	7.483	2.580	748,3	277,4	8,13	4,77
200	206	220	15,0	25,0	18	103	131,3	10.640	3.651	967,4	354,5	9,00	5,27
220	226	240	15,5	26,0	18	117	149,4	14.600	5.012	1.217	443,5	9,89	5,79
240	248	270	18,0	32,0	21	157	199,6	24.290	8.153	1.799	657,5	11,03	6,39
260	268	290	18,0	32,5	24	172	219,6	31.310	10.450	2.159	779,7	11,94	6,90
280	288	310	18,5	33,0	24	189	240,2	39.550	13.160	2.551	914,1	12,83	7,40
300	310	340	21,0	39,0	27	238	303,1	59.200	19.400	3.482	1.252	13,98	8,00
320	309	359	21,0	40,0	27	245	312,0	68.130	19.710	3.796	1.276	14,78	7,95
340	309	377	21,0	40,0	27	248	315,8	76.370	19.710	4.052	1.276	15,55	7,90
360	308	395	21,0	40,0	27	250	318,8	84.870	19.520	4.297	1.268	16,32	7,83
400	307	432	21,0	40,0	27	256	325,8	104.100	19.340	4.820	1.260	17,88	7,70
450	307	478	21,0	40,0	27	263	335,4	131.500	19.340	5.501	1.260	19,80	7,59
500	306	524	21,0	40,0	27	270	344,3	161.900	19.150	6.180	1.252	21,69	7,46
550	306	572	21,0	40,0	27	278	354,4	198.000	19.160	6.923	1.252	23.64	7,35
600	305	620	21,0	40,0	27	285	363,7	237.400	18.980	7.660	1.244	25,55	7,22
650	305	668	21,0	40,0	27	293	373,7	281.700	18.980	8.433	1.245	27,45	7,13
700	304	716	21,0	40,0	27	301	383,0	329.300	18.800	9.198	1.237	29,32	7.01
800	303	814	21,0	40,0	30	317	404,3	442.600	18.630	10.870	1.230	33,09	6,79
900	302	910	21,0	40,0	30	333	423,6	570.400	18.450	12.540	1.222	36,70	6,60
1000	302	1.008	21,0	40,0	30	349	444,2	722.300	18.460	14.330	1.222	40,32	6,45

Buckling check

$$N_{Ed} < N_{b,rd}$$

$$N_{b,rd} = \chi \cdot A \cdot f_{yd}$$

$$\chi = \frac{1}{\left(\varphi + \left(\varphi^2 - \bar{\lambda}^2\right)^{0,5}\right)} \leq 1$$

$$\varphi = 0,5 \cdot \left(1 + \alpha \cdot (\bar{\lambda} - 0,2) + \bar{\lambda}^2\right)$$

$$\bar{\lambda} = \sqrt{\frac{A \cdot f_{yk}}{N_{cr}}}$$

$$N_{cr} = \frac{\pi^2 \cdot E \cdot I_{min}}{L_0{}^2}$$

1. $N_{cr} = \dfrac{3.14 \times 21\,000 \times 1045}{2.80^2} = 878919$

2. $\bar{\lambda} = \sqrt{\dfrac{2196 \times 275}{878919}} = 0.82$

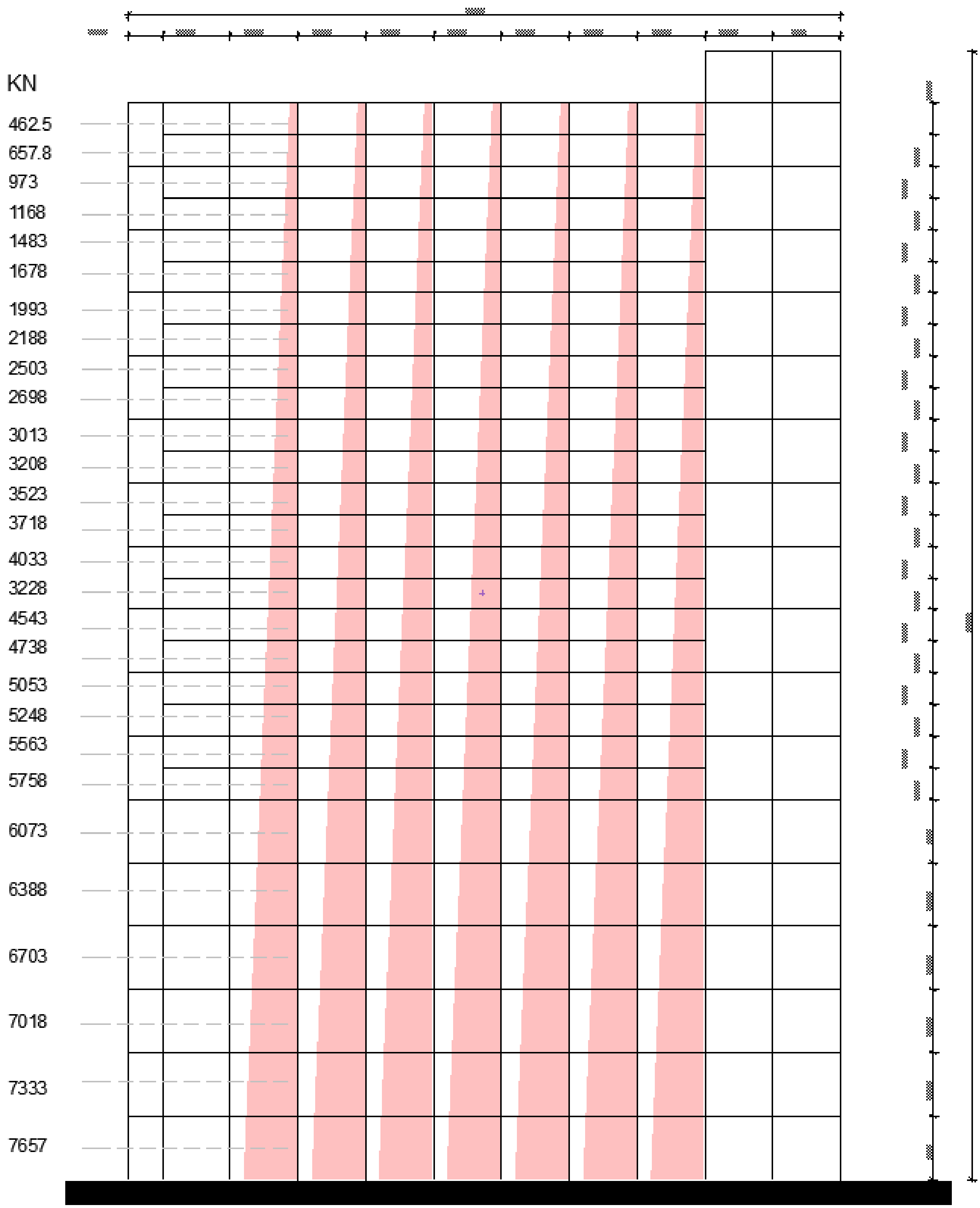
3. $\varphi = 0.5(\ 1 + 0.21(0.82\text{-}0.2) + 0.67) = 1.22$

4. $x = \dfrac{1}{1.22 + (\ 1.49 - 0.82)^{0.5} = 2.03} = 0.49$

5. $N_{brd} = 0.49 \times 2196 \times 261.9 = 28331$

Buckling check
$N_{brd} = 28331$ KN > N_{ED} (5584KN) - satisfied

Compression force variation on different levels /static diagram/



Foundations

N Design value of the compression force	Length of the column	Section	Self-weight	Self-load
1 $N_{Ed,r}$ =	39.2 m	HEM 260 M	172 kg/m	67.44 kN
2 $N_{Ed,r}$ =	28 m	HEM 550 M	278 kg/m	77.00 kN
3 $N_{Ed,r}$ =	28 m	HEM 1000 M	349 kg/m	97.72 KN
$N_{Ed,tot}$ = 11485kN + 67.4 kN + 77.00 kN + 97.72 KN			$N_{Ed,sw}$ = 11727 kN	

Estimation of the total load on the foundation including 10% for its self load
$F_{tot} = 1.10 \times N_{Ed,tot} = 1.10 \times 11727$ kN = 12899 kN

$A = F_{tot} / \sigma_{soil}$

A = 12899 kN / 3 kg/cm^2 = 429 966 cm^2
a= √ (429 66 cm^2) = 655 cm - square shape foundation