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Interdisciplinary Project

Multi storey Building

Static Report

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1. Introduction

This static report was made for a multi-story building composed by two buildings and a winter-garden in between, connecting both of them. The purpose of this report is to show all the calculations, drawings, schemes and all necessary information to prove the building's loadbearing capacity and stability against lateral loads, in order to get the building permit from the local authorities. The substructure that supports the winter garden is independent from the one used in the building blocks. The report includes calculations for dimensioning a wide variety of structures: concrete, steel and wood elements.

Loadbearing system

For the foundations it was chosen concrete: ground beams (strip foundation) receive loads from basement concrete walls, while footings (pad foundations) receive loads from basement concrete columns. All the elements connected to the foundations are cantilevered. From the basement until the uppermost level there are steel columns that support the whole building. These columns are connected to each other through steel beams, in a frame like structure and it is based in a simply supported construction (the bending moment is higher, however it makes much easier its construction). Sometimes a non conventional steel profile was chosen for beams, for example delta beams. These beams are an alternative to normal beams and can enable the usage of shallow element structures, the connection between slab elements, via steel bars, thanks to its holes, all over its length. Besides all this, delta beam are proved to have a great fire resistance compared to other beams.

In what regards the winter-garden structures, the huge glass façades, as well the glazed roof are supported by a steel frame, that includes steel columns and steel beams. These columns are made of hollow sections, so they can be filled with reinforced concrete, increasing even more the loadbearing capacity.

Wind bracing system

The building is also provided with elements, able to brace wind loads (lateral deflections) so that it can ensure the stability of the whole construction. The layout and materials for the building were discussed and chosen in previous phases, also according to statical principles, applied here. The lateral loads that hit the buildings all over its height are received through diaphragm elements (horizontal flat elements - flooring construction) such as hollowcore slabs (in the ground floor level), SL-slabs (1st floor level) and wooden cassettes (2nd floor level). These elements (lightweight elements, what means less dead weight) transfer the loads to shear walls, which location is crucial to ensure this system. Shear walls lead then the loads to foundations walls, and these to ground beams. The building resist wind forces in bending by cantilever supports at foundations level. In this project shear walls are not conventional reinforced concrete walls, instead of that, rammed earth walls and composite wooden walls were used to create this effect. Rammed earth is a material that has been studied during last years, coming to the conclusion that this material, when reinforced with steel bars, can act like concrete against lateral deformation. In other hand, wooden walls follow the principles of typical timber shear wall, to create braced panels in the wall, using structural plywood sheathing nailed at the edges to a supporting timber frame. Although this was a conventional system, the wooden walls chosen for this project represent the last developments in this field, making this component a perfect choice to carry lateral loads. Because the building has a considerable size, some interior walls had to be braced as well. Besides these walls, the shafts, made with reinforced concrete walls, also constitute a shear core providing torsional or twisting resistance as well lateral bracing.

In the winter garden, the steel columns that are turned to the glass façades are connected by steel beams. To provide the necessary wind bracing to these glass façades, it was chosen a truss system. The steel frame is connected to vertical trusses, in which compression elements (horizontal) are made of steel struts, while tension elements (vertical and diagonal) can be just a prestressed steel cable connecting the steel frame and the steel struts. This steel frame is simply supported at its base allowing some bending movements.

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2. Calculation Assumptions

2.1 Standards.

EN 1990 Basis of structural design.
EN 1991-1-1 Densities, self-weight, imposed loads for buildings
EN 1991-1-2 Actions on structures exposed to fire
EN 1991-1-3 Snow loads.
EN 1991-1-4 Wind loads.
EN 1993-1-1 Design of steel structures, General rules and rules for buildings
EN 1993-1-2 Design of steel structures, Structural fire design
EN 1993-1-8 Design of steel structures , Design of joints
EN 1995-1-1 Design of timber structures, General rules and rules for buildings
EN 1995-1-2 Design of timber structures, Structural fire design

2.2 Literature

Compedium Load and safety
Compendium for Load bearing constructions 2. Semester Timber and steel beams
Compendium Wind action

3. Materials

The building blocks are subjected to consequence class 2 (CC2) since these buildings include a commercial area in the ground floor and a residential area in the second and third floor. The winter garden structure is placed under consequence class 3 (CC3) due to its large span, height above 12 meters and public occupancy. All material data are placed under Section 8 of the specific material.

4. Loads

4.1 Permanent action: Self-weight

4.1.1) Self-weight sloped green roof construction, v (bottom-top):

- Acoustic pine wood panel: 14 mm

  0,014m * 6kN/m^3 = 0,08 kN/m^2

- 2 layers of plasterboard: 39mm

  0,039m * 9kN/m^3 = 0,35 kN/m^2

- 1 layer of wood board (pine wood): 26.5mm

  0,0265m* 4,6kN/m^3 = 0,12 kN/m^2

- Rips of pine wood (in 1 m^2 there are 7*70mm= 490mm): 210mm

  0,21m * 4,6kN/m^3 /2 = 0,48 kN/m^2

- Finishing wooden cross layer/ top chord (pine wood): 160mm

  0,160m * 4,6kN/m^3 = 0,74 kN/m^2

- 1 layer of plasterboard:10mm

  0,01m * 9kN/m^3 = 0,09 kN/m^2

- Insulation layer- mineral wool: 100mm

  0,1m * 0,6kN/m^3 = 0,06 kN/m^2

- Bitumen layer: 5mm

  0,005m * 14kN/m^3 = 0,07 kN/m^2

- Drainage layer: 30mm

  $0{,}03m * 0{,}3kN/m^3 \quad = \quad 0{,}003\ kN/m^2$

- Soil: 80mm

  $0{,}08m * 15kN/m^3 \quad = \quad 12\ kN/m^2$

Total self-weight v: **$14\ kN/m^2$**

4.1.2) Self-weight flat green roof construction, f (bottom-top):

- Acoustic pine wood panel: 40 mm

  $0{,}04m * 6kN/m^3 \quad = \quad 0{,}24\ kN/m^2$

- 2 layers of plasterboard: 25mm

  $0{,}025m * 9kN/m^3 \quad = \quad 0{,}23\ kN/m^2$

- 1 layer of wood board (pine wood): 26.5mm

  $0{,}0265m * 4{,}6kN/m^3 \quad = \quad 0{,}12\ kN/m^2$

- 212 x 75 Joist floor

  $(0{,}212m * 0{,}075m) * 4{,}6kN/m^3 / 0{,}154m$

- Finishing wooden cross layer/ top chord (pine wood): 160mm

  $0{,}160m * 4{,}6kN/m^3 \quad = \quad 0{,}11\ kN/m^2$

- 1 layer of plasterboard:10mm

  $0{,}01m * 9kN/m^3 \quad = \quad 0{,}09\ kN/m^2$

- Bitumen layer: 5mm

  $0{,}005m * 14kN/m^3 \quad = \quad 0{,}07\ kN/m^2$

- Insulation layer- mineral wool: 100mm

  $0{,}1m * 0{,}6kN/m^3 \quad = \quad 0{,}06\ kN/m^2$

- Bitumen layer: 5mm

  $0{,}005m * 14kN/m^3 \quad = \quad 0{,}07\ kN/m^2$

- Drainage layer: 30mm

$0,03m * 0,3kN/m^3$ = $0,003\ kN/m^2$

- Soil: 80mm

$0,08m * 15kN/m^3$ = $12\ kN/m^2$

Total self-weight f: **$13.053\ kN/m^2$**

4.1.3) Self-weight wooden partition walls, j:

- Finishing layer of plasterboard: 26 mm

$0,026m * 9kN/m^3$ = $0,234\ kN/m^2$

- Air layer: 25mm
- Insulation layer - fiber wooden insulation: 51 mm

$0,051m * 8\ kN/m^3$ = $0,408\ kN/m^2$

- Wood construction (pine wood): 110 mm

$0,11 * 4,6\ kN/m^3$ = $0,506\ kN/m^2$

- Finishing layer of plasterboard: 26 mm

$0,026m * 9kN/m^3$ = $0,234\ kN/m^2$

Total self-weight j: **$1,38\ kN/m^2$**

4.1.4) Self-weight wooden storey partition (bottom-top), w:

- Acoustic pine wood panel: 40 mm

$0,04m * 6kN/m^3$ = $0,24\ kN/m^2$

- 2 layers of plasterboard: 25mm

$0,025m * 9kN/m^3$ = $0,23\ kN/m^2$

- 1 layer of wood board (pine wood): 26.5mm

$0,0265m * 4,6kN/m^3$ = $0,12\ kN/m^2$

- 212 x 75 Joist floor

(0,212m * 0,075m) * 4,6kN/m^3 / 0,154m = 0,47 kN/m^2

- Finishing layer of pine wood: 24mm

  0,024m * 4,6kN/m^3 = 0,11 kN/m^2

- 1 layer of hardwood: 15mm

  0,015m * 10kN/m^3 = 0,15 kN/m^2

- 1 layer of impact sound insulation (hard fiberboard): 30mm

  0,03m * 10kN/m^3 = 0,3 kN/m^2

- Cement layer: 50mm

  0,05m * 16 kN/m^3 = 0,8 kN/m^2

- Finishing layer of oak parquet: 15mm = 0,09 kN/m^2

Total self-weight w: 2.51 kN/m^2

4.1.4) Self-weight wooden exterior walls construction, P (inside-outside):

- Plasterboard: 26mm

  0,026m * 9kN/m^3 = 0,23 kN/m^2

- Load bearing inner wall (pine wood): 110mm

  0,11m * 4,6kN/m^3 = 0,51 kN/m^2

- Wooden fiber insulation: 300mm

  0,30m * 8kN/m^3 = 2,4 kN/m^2

- Fire wooden insulation (high density): 40mm

  0,04m * 9kN/m^3 = 0,4 kN/m^2

- Outer finishing pine wood layer: 62mm

  0,062m * 4,6kN/m^3 = 0,29 kN/m^2

Total self-weight P: 3,79 kN/m^2

4.1.4) Self-weight flooring construction, h (bottom-top):

- Concrete:50mm

  0,05m * 25kN/m^3 = 1,25 kN/m^2

- Rigid insulation:100mm

  0,1m * 0,3kN/m^3 = 0,03 kN/m^2

- Concrete: 50mm

  0,05m * 25kN/m^3 = 1,25 kN/m^2

- Finishing layer (ceramic tiles and oak wood flooring): 50mm

  0,05m * 21kN/m^3 * 0,3 = 0,315 kN/m^2

  0,3kN/m^2 * 0,7 = 0,21 kN/m^2

Total self-weight h: 3,49 kN/m^2

4.1.4) Self-weight rammed earth wall construction, c (inside - outside):

- Reinforced concrete:150mm

  0,15m * 25kN/m^3 = 3,75 kN/m^2

- Rigid insulation:150mm

  0,15m * 0,3kN/m^3 = 0,045 kN/m^2

- Rammed earth: 200mm

  0,2m * 25kN/m^3 = 5 kN/m^2

Total self-weight c: 8,8 kN/m^2

4.1.5) Self-weight sloped glazing roof, r (inside-outside):

- Steel profile

  2m * 0,0776 kN/m = 0,155 kN/m^2

- Double glazing insulation unit

  0,88kN/m^2

- Aluminum cover section

  2m * 0,004 kN/m = 0,0086 kN/m^2

Total self-weight r: **1,04 kN/m^2**

4.1.6) Self-weight winter garden flooring, t (bottom-top):

- Concrete: 50mm

  0,05m * 25kN/m^3 = 1,25 kN/m^2

- Rigid insulation: 100mm

  0,1m * 0,3kN/m^3 = 0,03 kN/m^2

- Drainage layer (gravel): 200mm

  0,2m * 15kN/m^3 = 3 kN/m^2

- Soil: 600mm

  0,6m * 17kN/m^3 = 10,2kN/m^2

- Grass: 40mm

  0,04m * 7kN/m^3 = 0,28kN/m^2

Total self-weight t: **14,76 kN/m^2**

4.2 Variable action: Environmental actions

4.2.1. Snow loads

g: the roof pitch is 14,30°.

f: the roof pitch is 2,25º.

r: the roof pitch is 10º.

Table 5.2: Snow load shape coefficients

Angle of pitch of roof α	$0° \le \alpha \le 30°$	$30° < \alpha < 60°$	$\alpha \ge 60°$
μ_1	0,8	$0,8(60 - \alpha)/30$	0,0
μ_2	$0,8 + 0,8\ \alpha/30$	1,6	--

If the roof is smaller or the same as 30° then the snow load coefficient is 0.8 kN/m^2.

Snow action: $S = \mu_1 * S_k$

S = 0.8 * 1.0 kN/m^2 = 0.8 kN/m^2

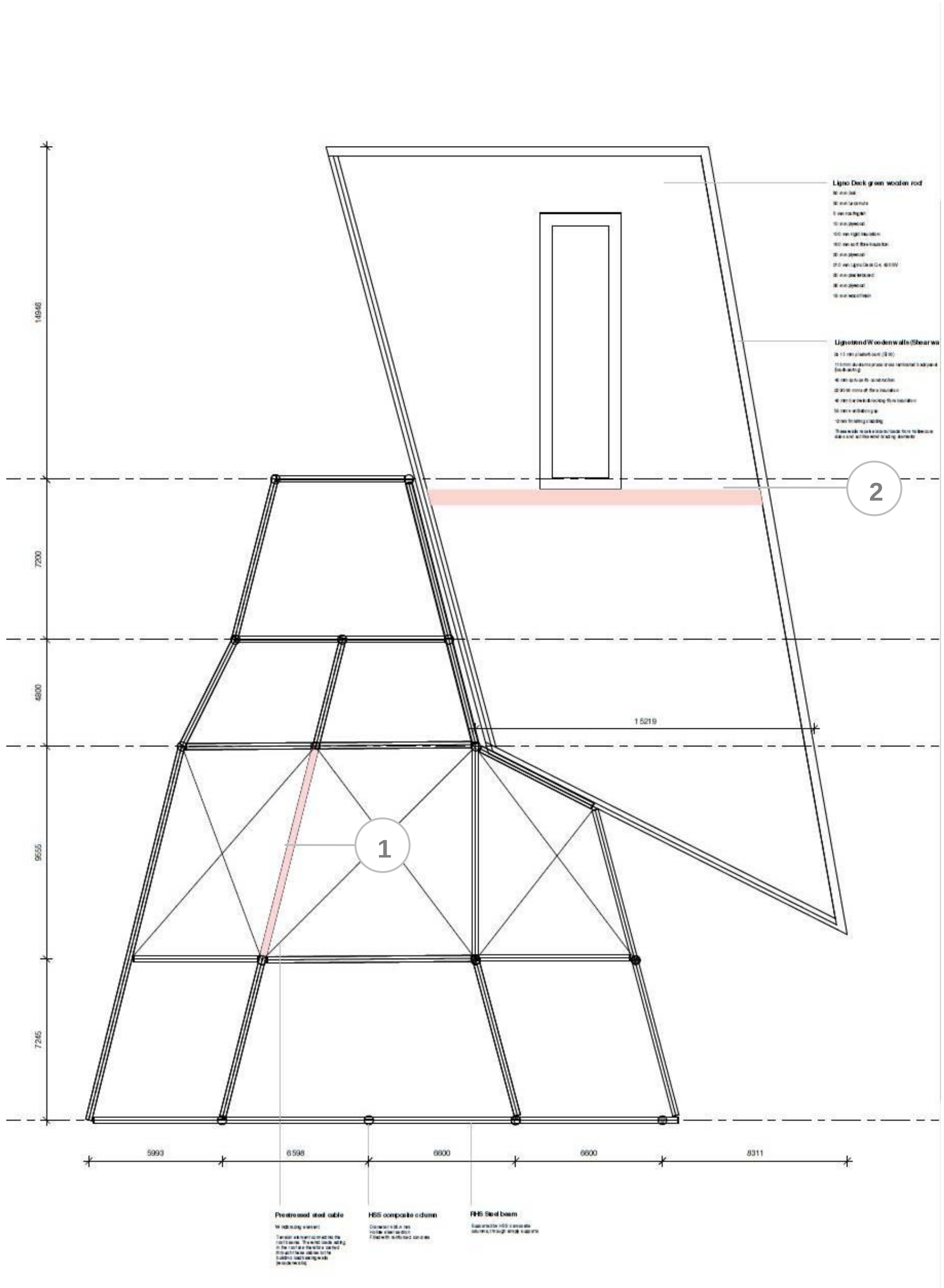
5. Load combinations

$$E_d = \gamma_G \cdot G_k + \gamma_{Q1} \cdot Q + \gamma_{Qi} \cdot \Psi_{0,i} \cdot Q_i$$

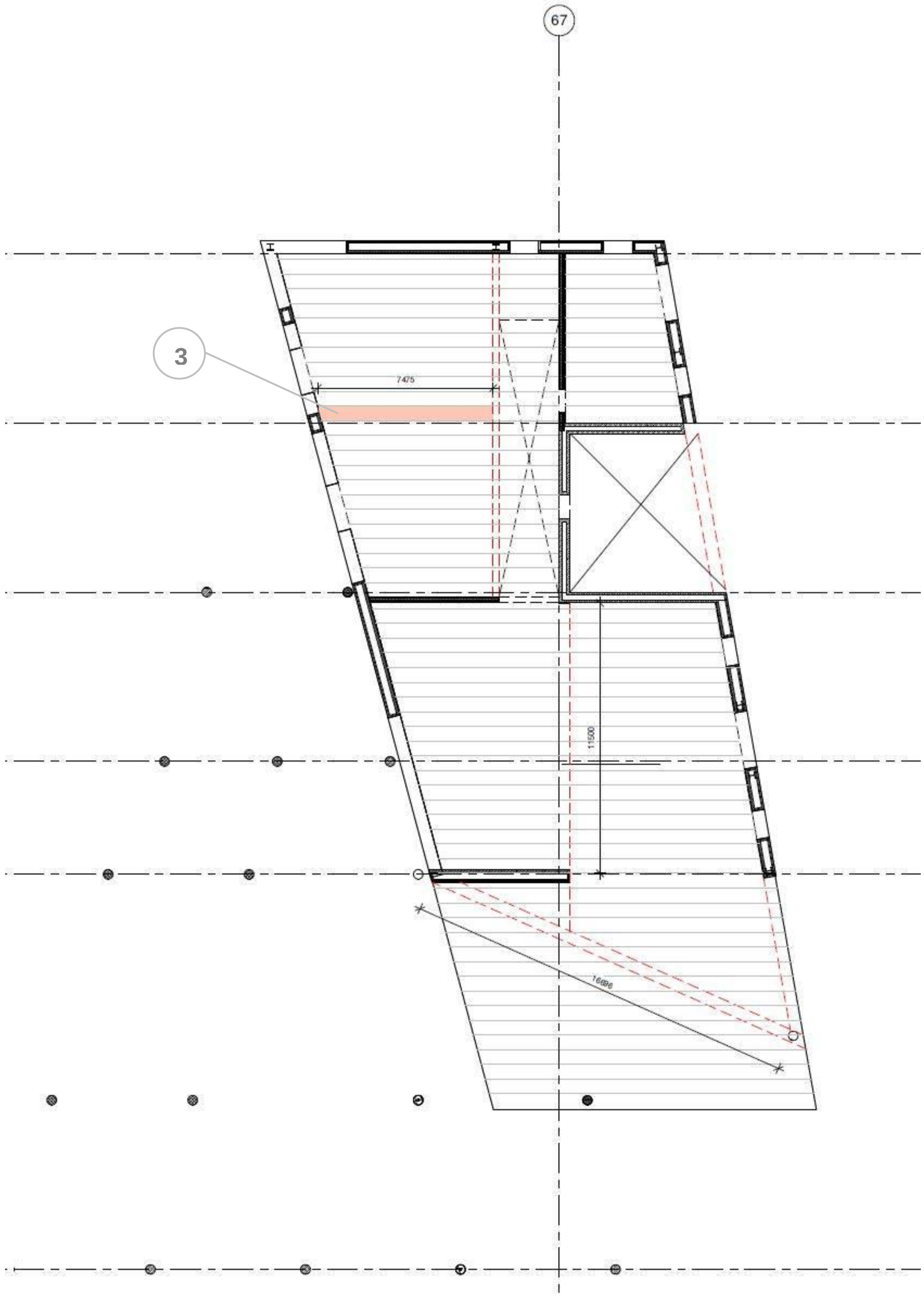
Symbols:
G: Permanent action [kN]
g: Permanent action [kN/m]
Q: Variable action [kN]
q: Variable action [kN/m]
γ (gamma): Partial safety factor (DS/EN 1990 table A 1.2(B) or National annex)
ψ(psi): Load combination factor (DS/EN 1990 table A 1.1 or National annex)

6. Static calculations

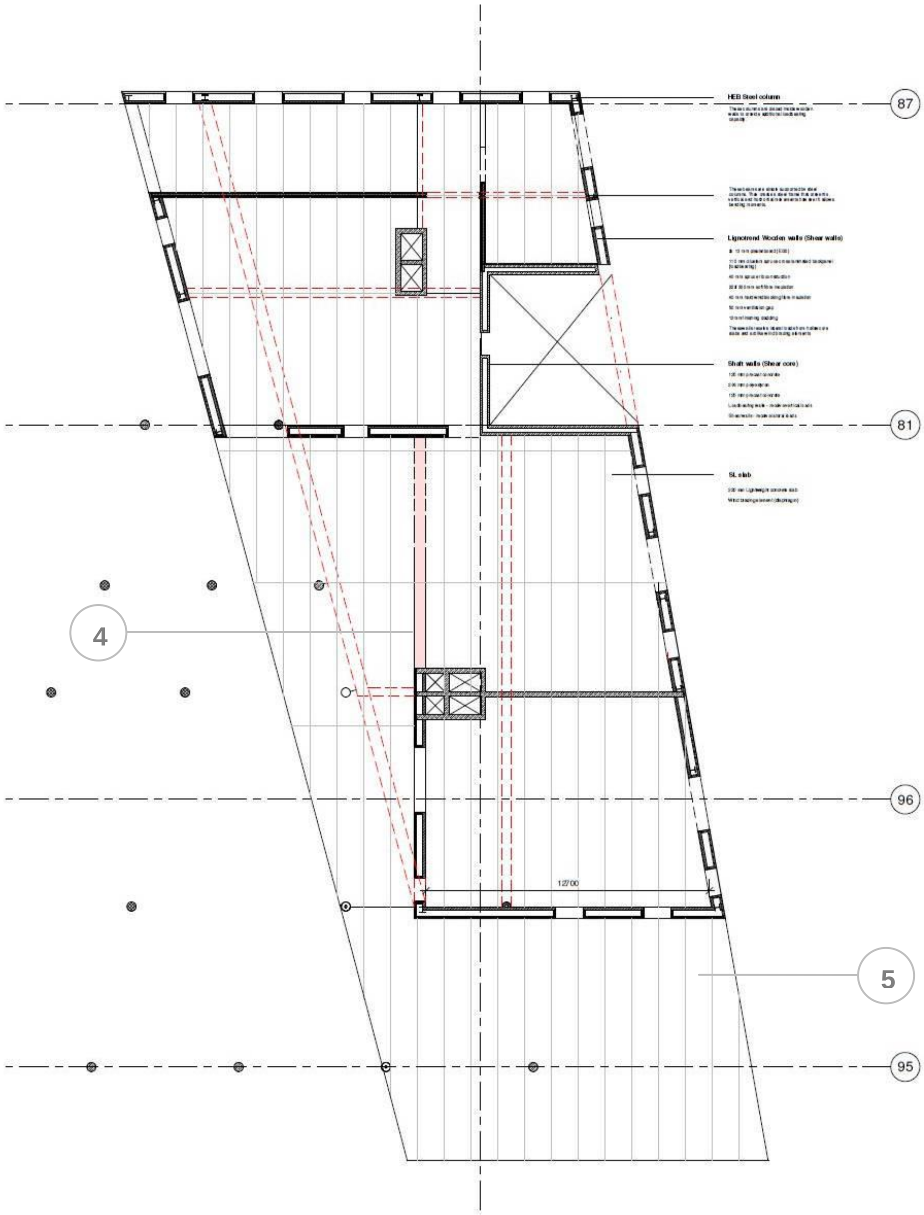
Location of sized elements:



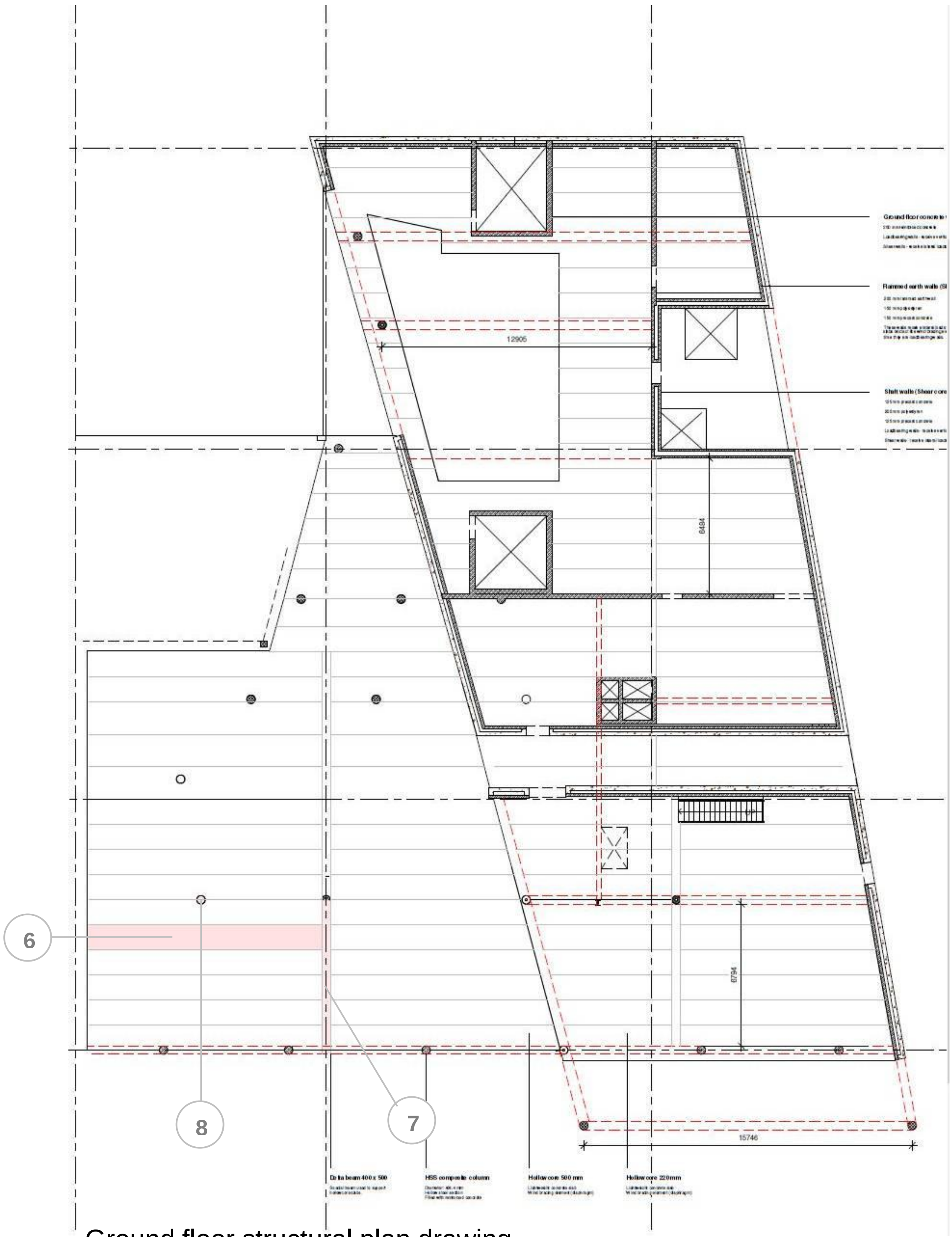
Roof structural plan drawing



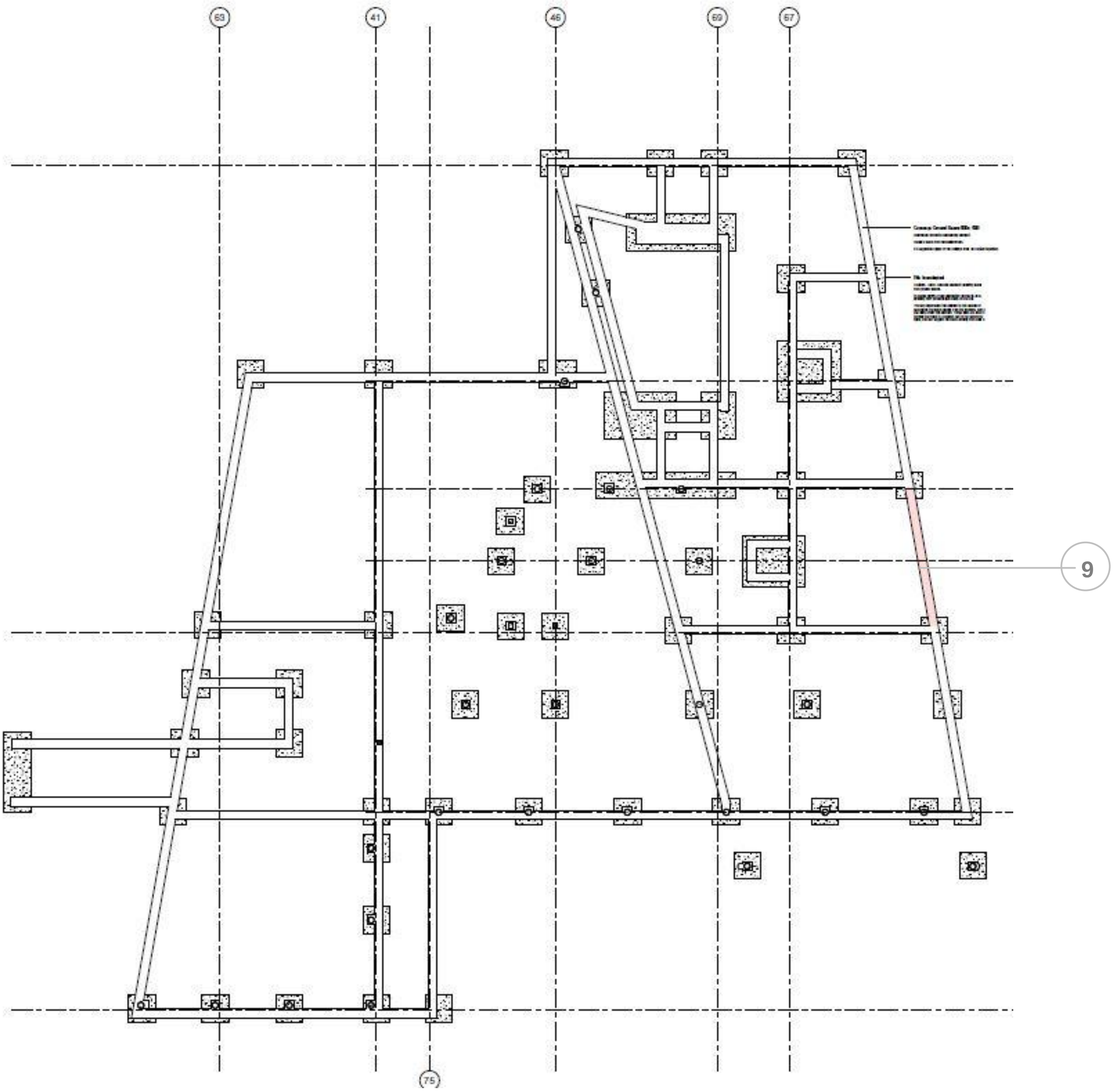
Second floor structural plan drawing



First floor structural plan drawing

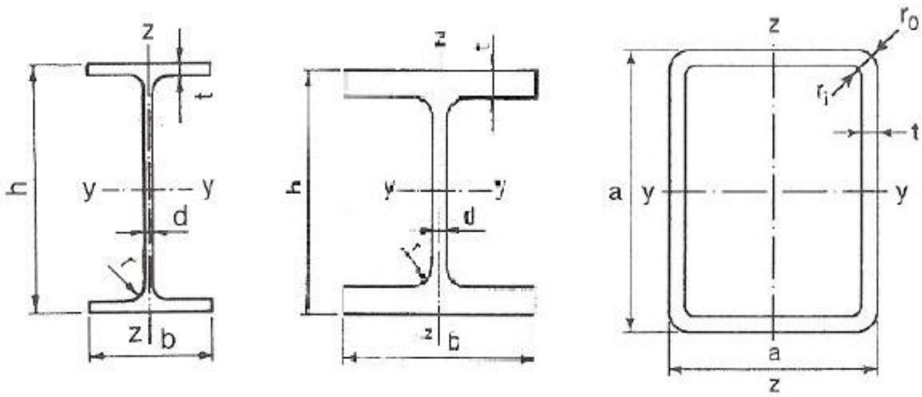


Ground floor structural plan drawing



Foundations plan drawing

6.1 Steel Beam



6.1.1 Material Data

Consequences classes:	CC2 Medium
Cross section class:	CS1
Normal inspection level:	$\gamma_3 = 1.0$
Steel grade: S235	$f_y = 235\ N/mm^2$
Safe factor $\gamma_{M0} = 1.1 * \gamma_3$	$\gamma_{M0} = 1.1 * 1.0 = 1.1$
Modulus of elasticity (E):	$0{,}21 * 10^6\ N/mm^2$
Beam length (l):	10 m

6.1.2 Line loads

Load span:		7,8 m
Self-weight (g): r * 7,8m = 1,04kN/m^2 * 7,8m	=	8,11 kN/m
Snow load (s): s * 7,8m = 0,8 kN/m^2 * 7,8m	=	6,24 kN/m

6.1.3 Design Load

$E_d = \gamma g * g + \gamma s * s$

E_d = 1,0 * 8,11kN/m + 1,5 * 6,24kN/m	=	17,47 kN/m

6.1.4 Internal Forces

6.1.4.1) Shear forces

$V_{d,max} = 0.5 * E_d * l$

$V_{d,max} = 0.5 * 17{,}47kN/m * 10m$ = 87,35 kN

6.1.4.2) Bending forces

$M_{max} = 1/8 * E_d * l^2$

$M_{max} = 1/8 * 17{,}47kN/m * 10^2 m$ = 218,38 kN.m

6.1.5 Minimum section modulus (W_{min})

$$W_{min} = \frac{M_{max} \cdot \gamma_m}{f_y}$$

$W_{min} = 218{,}38 * 10^6 N.mm * 1{,}1 / 235 N/mm^2$ = $1022{,}2 * 10^3 mm^3$

Looking to the value of Minimum section modulus it is possible to take a conclusion. In this case, we choose a rectangular hollow section with the following characteristics:

Rectangular Hollow section properties chart (*Compendium for Load bearing constructions 2. Semester Timber and steel beams*)

$a \times b$ mm	t mm	A mm^2	u m^2/m	g kg/m	I_y mm^4	$W_{el,y}$ mm^3	i_y mm	I_z mm^4	$W_{el,z}$ mm^3	i_z mm	$W_{pl,y}$ mm^3	$W_{pl,z}$ mm^3	I_v mm^4	W_v mm^3
faktor	1	10^3	1	1	10^6	10^3	1	10^6	10^3	1	10^3	10^3	10^6	10^3
450×250	10,0	13,5	1,37	106	369	1640	165	148	1185	105	2000	1331	333	1986
450×250	12,0	16,1	1,37	126	434	1930	164	174	1389	104	2367	1572	393	2324
450×250	16,0	21,1	1,36	166	557	2476	162	220	1763	102	3070	2029	505	2947

Now the calculations must be adjusted, since the self weight of the beam was not taken into account before.

6.1.2) Self-weight (g) = r + beam weight

Self-weight (g) = 8,11kN/m + 1,04kN/m = 9,15 kN/m

6.1.3) $E_d = \gamma g * g + \gamma s * s$

E_d = 1,0 * (8,11kN/m + 1,04kN/m) + 1,5 * 6,24kN/m = 18,51 kN/m

6.1.4.1) $V_{d,max}$ = 0.5 * E_d * l

$V_{d,max}$ = 0.5 * 18,51kN/m * 10m = 92,55 kN

6.1.4.2) M_{max} = 1/8 * E_d * l²

M_{max} = 1/8 * 18,51kN/m * 10^2m = 231,38 kN.m

6.1.5) $W_{min} = \frac{M_{max} \cdot \gamma_m}{f_y}$

W_{min} = 231,38 * 10^6 N.mm * 1,1 / 235 N/mm^2 = 1083,1 * 10^3 mm^3

6.1.6 Deflection

The limitation value of deflection for a steel beam in a roof structure is:

$u_{max} = \frac{l}{200}$

u_{max} = 10000mm / 200 = 50 mm

6.1.6.1) Self-weight (G)

$$U_{inst,G} = \frac{5 \cdot q \cdot l^4}{384 \cdot I_y \cdot E}$$

$U_{inst,G}$ = (5 * 9,15N/mm *10000^4) / (384 * 369 * 10^6 mm^4 * 0,21 * 10^6 N/mm^2)
= 15,37 mm

15,37mm ≤ 50 mm **OK**

6.1.6.2) Snow load (S)

$$U_{inst,S} = \frac{5 \cdot q \cdot l^4}{384 \cdot I_y \cdot E}$$

$U_{inst,S}$ = (5 * 6,24N/mm *10000^4) / (384 * 369 * 10^6 mm^4 * 0,21 * 10^6 N/mm^2)
= 10,49 mm

10,49mm ≤ 50 mm **OK**

Total final deflection

U_{max} = $U_{inst,G}$ + $U_{inst,S}$

U_{max} = 15,37mm + 10,49mm = 25,86 mm

25,86 mm ≤ 50 mm **OK**

At this point , it is obvious that the beam was over dimensioned, or in other words, it was chosen a section whose properties are much better than the ones required (special attention for the deflection value obtained, that is two times smaller than the maximum allowed). Even if this means a safety increase for the loadbearing capacity of the beam, in other hand it is a waste of material. To counter this situation, it is required to check the section chart again and select a new one.

Rectangular Hollow section properties chart (*Compendium for Load bearing constructions 2. Semester Timber and steel beams*)

$a \times b$ mm	t mm	A mm^2	u m^2/m	g kg/m	I_y mm^4	$W_{el,y}$ mm^3	i_y mm	I_z mm^4	$W_{el,z}$ mm^3	i_z mm	$W_{pl,y}$ mm^3	$W_{pl,z}$ mm^3	I_v mm^4	W_v mm^3
faktor	1	10^3	1	1	10^6	10^3	1	10^6	10^3	1	10^3	10^3	10^6	10^3
300×200	6,3	6,10	0,984	47,9	78,3	522	113	41,9	419	82,9	624	472	84,8	681
300×200	8,0	7,68	0,979	60,3	97,2	648	113	51,8	518	82,2	779	589	106	840
300×200	10,0	9,49	0,974	74,5	118	788	112	62,8	628	81,3	956	721	129	1015
300×200	12,0	11,3	0,969	88,5	138	920	111	72,9	729	80,5	1124	847	151	1178
400×200	10,0	11,5	1,17	90,2	239	1196	144	80,8	808	83,9	1480	911	193	1376
400×200	12,0	13,7	1,17	107	281	1403	143	94,2	942	83,0	1748	1072	226	1602
400×200	16,0	17,9	1,16	141	357	1787	141	118	1182	81,3	2256	1374	289	2010
450×250	10,0	13,5	1,37	106	369	1640	165	148	1185	105	2000	1331	333	1986
450×250	12,0	16,1	1,37	126	434	1930	164	174	1389	104	2367	1572	393	2324
450×250	16,0	21,1	1,36	166	557	2476	162	220	1763	102	3070	2029	505	2947

Using the formulas on 6.1.6.1 and 6.1.6.2, makes possible to test different sections by just changing the beam self-weight and the I_y

Section 400x200 (t=10,0)

$U_{inst,G}$ = (5 * (8,11+0,88)kN/mm *10000^4) / (384 * 239 * 10^6 mm^4 * 0,21 * 10^6 N/mm^2)
= 23,32 mm
$U_{inst,S}$ = (5 * 6,24N/mm *10000^4) / (384 * 239 * 10^6 mm^4 * 0,21 * 10^6 N/mm^2)
= 16,19 mm

U_{max} = 23,32mm + 16,19mm = 39,5 mm

39,5 mm ≤ 50 mm **OK**

Section 300x200 (t=12,0)

$U_{inst,G}$ = (5 * (8,11+0,87)/mm *10000^4) / (384 * 138 * 10^6 mm^4 * 0,21 * 10^6 N/mm^2)
= 40,35 mm
$U_{inst,S}$ = (5 * 6,24N/mm *10000^4) / (384 * 138 * 10^6 mm^4 * 0,21 * 10^6 N/mm^2)
= 28,03 mm

U_{max} = 40,35mm + 28,03mm = 39,5 mm

68,38 mm ≤ 50 mm **X**

All the sections among the rectangular hollow sections table have deflection values that are far from the maximum required, so that, it is wise to take a look into other profiles. Once again, the obtained value for the minimum section modulus will be taken into consideration.

HEB

U_{max} = (5 * (8,11 + 1,01)kN/mm *10000^4) / (384 * 192,7 * 10^6 mm^4 * 0,21 * 10^6 N/mm^2)
+ (5 * 6,24N/mm *10000^4) / (384 * 192,7 * 10^{6}mm^4 * 0,21 * 10^6 N/mm^2)
= 49,42 mm

The HEB profile number 280* has deflection value of 49,42mm what makes it the perfect choice regarding all the requirements. Now, some of the calculations made before need to be done again with the updated values.

HEB profiles properties chart (*Compendium for Load bearing constructions 2. Semester Timber and steel beams*)

profil nr.	h mm	b mm	d mm	t mm	r mm	A mm^2	u m^2/m	g kg/m	I_y mm^4	$W_{el,y}$ mm^3	i_y mm	I_z mm^4	$W_{el,z}$ mm^3	i_z mm	I_v mm^4	I_w mm^6	W_{pl} mm^3
faktor	1	1	1	1	1	10^3	1	1	10^6	10^3	1	10^6	10^3	1	10^3	10^9	10^3
220*	220	220	9,5	16	18	9,10	1,27	71,5	80,9	736	94,3	28,4	258	55,9	768	295	828
240*	240	240	10	17	21	10,6	1,38	83,2	112,6	938	103	39,2	327	60,8	1030	487	1054
260*	260	260	10	17,5	24	11,8	1,50	93,0	149,2	1150	112	51,3	395	65,8	1240	754	1282
280*	280	280	10,5	18	24	13,1	1,62	103	192,7	1380	121	65,9	471	70,9	1440	1130	1534
300*	300	300	11	19	27	14,9	1,73	117	251,7	1680	130	85,6	571	75,8	1860	1690	1868

6.1.2) Self-weight (g) = r + beam weight

Self-weight (g) = 8,11kN/m + 1,01kN/m = 9,12 kN/m

6.1.3) E_d = ɤg * g + ɤs * s

E_d = 1,0 * (8,11kN/m + 1,01kN/m) + 1,5 * 6,24kN/m = 18,48 kN/m

6.1.4.1) $V_{d,max}$ = 0.5 * E_d * l

$V_{d,max}$ = 0.5 * 18,48kN/m * 10m = 92,4 kN

6.1.4.2) M_{max} = 1/8 * E_d * l²

M_{max} = 1/8 * 18,48kN/m * 10^2m = 231 kN.m

6.1.5) $W_{min} = \dfrac{M_{max} \cdot \gamma_m}{f_y}$

W_{min} = 231 * 10^6 N.mm * 1,1 / 235 N/mm^2 = 1081,3 * 10^3 mm^3

6.1.7 The final bending stress

$\sigma_b = \dfrac{M_{Ed}}{W}$ = 231 * 10^6N.mm / 1081,3 * 10^3 mm^3 = 212,63 N/mm^2

$\dfrac{f_y}{\gamma_{M0}}$ = 235N/mm^2 / 1.1 = 213 N/mm^2

$$\sigma_b = \frac{M_{Ed}}{W} \leq \frac{f_y}{\gamma_{M0}}$$

212,63 N/mm^2 ≤ 213 N/mm^2 **OK**

6.1.8 The final shear stress

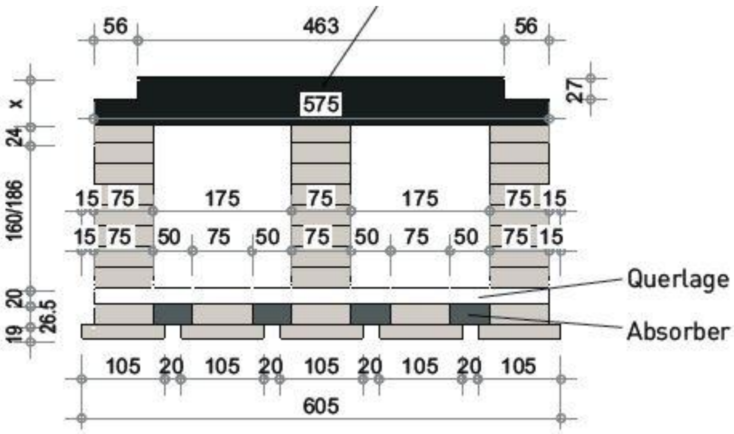
$\tau_d = \frac{V_r \sqrt{3}}{A_v}$ = 92,4kN * 10^3 * $\sqrt{3}$ / 6,55 * 10^3 mm^2 = 24,43 N/mm^2

A_v = A * h/ (b + h)
A_v = 13,1 * 10^3 mm^2 * 280mm / (280mm + 280mm)
= 6,55 * 10^3 N/mm^2

$$\tau_d = \frac{V_r \sqrt{3}}{A_v} \leq \frac{f_y}{\gamma_{M0}}$$

24,43 N/mm^2 ≤ 213 N/mm^2 **OK**

6.2 Load-bearing wooden roof (pos. 2)



The wooden roof used in the top of the building is made of glued laminated timber, however it cannot be sized according to the rules for the glued laminated beams. The reason behind is that, the component shown above acts like a unique loadbearing element, so calculating it in this way, would lead into a wrong conclusion. The only way to find out the right element, with the required dimensions, is to take a look at the tables, provided by the manufacturer. The only needed parameters for this are the span and the design load value.

6.2.1 Material Data

Consequences classes: CC2 Medium

Span (the biggest): 15 m

6.2.2 Loads

Self-weight (g): f = 13,053 kN/m^2

Snow load (s): s = 0,8kN/m^2

6.2.3 Design Load

$S_d = \gamma_g * g + \gamma s * s$

$S_d = 1{,}0 * 13{,}053 kN/m^2 + 1{,}5 * 0{,}8 kN/m^2$ = 14,253 kN/m^2

Table of loadbearing capacity values for the wooden storey partition

	Höhe	309 BV	335 BV	355 BV	375 BV	395 BV	415 BV	435 BV	mm
L_{ef} [m]	Untersicht	Akustik klassik	Akustik klassik	Akustik klassik	Akustik klassik	Akustik klassik	Akustik klassik	Akustik klassik	
5,00	EI_{ef}	9450	11854	14462	17237	20227	23471	27006	kNm²
	$M_{R,k}$	100,0	115,1	128,3	141,6	155,4	169,8	185,0	kNm
	$-M_{R,k}$	146,1	170,3	192,0	212,4	233,0	254,6	277,4	kNm
	$V_{R,k}$	44,4	51,1	57,0	62,9	69,0	75,4	82,1	kN
10,00	EI_{ef}	10307	12873	15688	18671	21871	25328	29081	kNm²
	$M_{R,k}$	93,0	106,4	118,0	129,7	141,7	154,2	167,4	kNm
	$-M_{R,k}$	138,9	159,6	177,0	194,5	212,6	231,4	251,1	kNm
	$V_{R,k}$	41,3	47,2	52,4	57,6	62,9	68,5	74,3	kN
15,00	EI_{ef}	10499	13102	15965	18996	22245	25752	29555	kNm²
	$M_{R,k}$	91,7	104,8	116,1	127,5	139,2	151,4	164,2	kNm
	$-M_{R,k}$	137,5	157,2	174,2	191,2	208,8	227,0	246,2	kNm
	$V_{R,k}$	40,7	46,5	51,5	56,6	61,8	67,2	72,9	kN

Loadbearing capacity: 29555 kN/m^2 > 14,253 kN/m^2 **OK**

6.2.4 Design load deflection

$S_d = g + s$

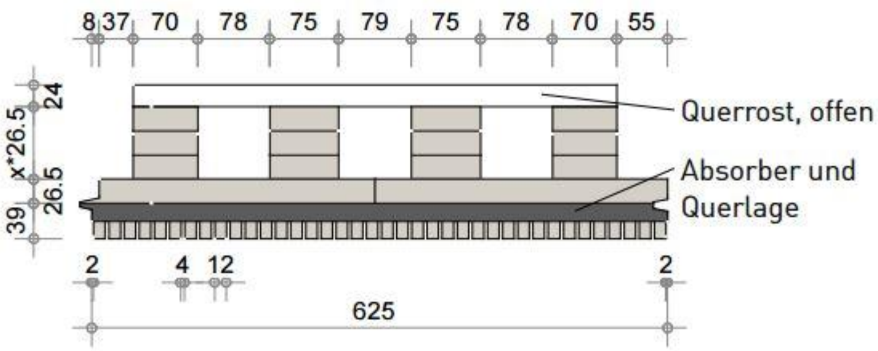
$S_d = 13{,}053 kN/m^2 + 0{,}8 kN/m^2$ = $13{,}853 kN/m^2$

Table of deflection values for wooden storey partition

		Elementhöhe	196	222	249	275	302	309	335	355	375	395	415	435	mm
LIGNO Akustik Q3 klassik	Schub	$V_{R,k,y}$	37,9					29,1	29,1	42,9	57,5	72,8	88,2	102,7	kN
		GA_{ef}	3265					15732	15732	23184	31050	39330	47610	55476	kN
	Biegung in Plattenebene	I_z	181	205	231	255	281	299	324	361	398	435	472	508	$10^3 cm^4$
		$M_{R,k,z}$	101,1	115,0	129,3	143,1	157,2	167,7	181,5	202,2	222,8	243,5	264,2	284,9	kNm

Deflection: $284{,}9 kN/m^2 > 13{,}853 kN/m^2$ **OK**

6.3 Load-bearing wooden storey-partition (pos. 3)



The wooden storey partition used in the second floor follows the same principles as the wooden roof, however, and just to prove that the joists in between the layers of glued laminated timber are not isolated loadbearing members, they will be dimensioned as normal glued laminated timber beams.

6.3.1 Material Data

Consequences classes: CC2 Medium

Service class: 1

Strength Class: GL 24h

Load duration class: P and M

Span (l): 7,5 m

Chosen size for the glue laminated beams: 75 x 212 mm

Table with the strength classes: design values for homogeneous glued laminated timber (MPa).

Strength classes : design values for Homogeneous glued laminated timber MPa																
Service class 1 and 2																
		GL32h					GL28h					GL24h				
		P	L	M	K	Ø	P	L	M	K	Ø	P	L	M	K	Ø
Bending	$f_{m,d}$	14.8	17.2	19.7	22.2	27.1	12.9	15.1	17.2	19.4	23.7	11.1	12.9	14.8	16.6	20.3
Tension	$f_{t,0,d}$	11.8	13.8	15.8	17.7	21.7	10.3	12.0	13.7	15.4	18.9	8.9	10.3	11.8	13.3	16.2
	$f_{t,90,d}$	0.23	0.27	0.31	0.35	0.42	0.23	0.27	0.31	0.35	0.42	0.23	0.27	0.31	0.35	0.42
Comp.	$f_{c,0d}$	14.8	17.2	19.7	22.2	27.1	12.9	15.1	17.2	19.4	23.7	11.1	12.9	14.8	16.6	20.3
	$f_{c,90,d}$	1.15	1.35	1.54	1.73	2.12	1.15	1.35	1.54	1.73	2.12	1.15	1.35	1.54	1.73	2.12
Shear	$f_{v,d}$	1.62	1.88	2.15	2.42	2.96	1.62	1.88	2.15	2.42	2.96	1.62	1.88	2.15	2.42	2.96

Table with the characteristic values of the rigidity factor.

Strength clas		C30	C24	C18	C14	GL32h	GL32c	GL28h	GL28c	GL24c	GL24h
Elastic modulus	E_0 MPa	12000	11000	9000	7000	14200	13500	12600	12500	11500	11000

Table with values of k_{def} for timber and wood-based materials.

Material	Standard	Service class		
		1	2	3
Solid timber	EN 14081-1	0,60	0,80	2,00
Glued Laminated timber	EN 14080	0,60	0,80	2,00

6.3.2 Line loads

Load span: 0,154 m

Self-weight (G): w * 0,154

= 2,51kN/m^2 * 0,154m = 0,39 kN/m

Imposed load (Q_k): A1 dwellings qk * 0,154m

$= 1{,}5\ kN/m^2 * 0{,}154m \quad = \quad 0{,}23\ kN/m$

6.3.3 Design Load

$E_d = \gamma_G * G + \gamma_{Qk} * Q_k$

$E_d = 1{,}0 * 0{,}39kN/m + 1{,}5 * 0{,}23kN/m \quad = \quad 0{,}74\ kN/m$

6.3.4 Internal Forces

6.3.4.1) Shear forces

$V_{d,max} = 0.5 * E_d * l$

$V_{d,max} = 0.5 * 0{,}74kN/m * 7{,}5m \quad = \quad 2{,}76\ kN$

6.3.4.2) Bending forces

$M_{max} = 1/8 * E_d * l^2$

$M_{max} = 1/8 * 0{,}735kN/m * 7{,}5^2m \quad = \quad 5{,}17\ kN.m$

6.3.5 Minimum section modulus (W_{min})

The table with the section properties for glue laminated timber doesn't provide the dimensions used here, so it requires the calculation of W_y and I_y, before finding the minimum section modulus.

$W_y = 1/6 * b * h^2$

$= 1/6 * 75mm * 212mm^2 \quad = \quad 561{,}8 * 10^3\ mm^3$

$I_y = 1/12 * b * h^3$

$= 1/12 * 75mm * 212^3mm \quad = \quad 595{,}5 * 10^6\ mm^4$

$W_{min} = M_{,max} / f_{md}$

$= 5{,}17\ kN.m * 10^6 / 11{,}1\ MPa \quad = \quad 465{,}77 * 10^3\ mm^3$

6.3.6 Deflection

The limitation value of deflection for a floor joist in a residential multi-story building is L/600 for a 1.5 kN/m^2 load.

U_{max} = l/600

= 7500mm / 600 = 14,5 mm

6.3.6.1 Imposed load

$$U_{inst,Q} = \frac{5 \cdot q \cdot l^4}{384 \cdot I_y \cdot E}$$

= 5 * 0,23 N/mm * 7500^4mm / 384 * 11000mm^4 * 595,5 * 10^6 = 11,9 mm

Imposed load including the safety factors

$U_{fin,Q} = U_{inst,Q} (1+ \psi_2 * K_{def})$

= 11,9mm * (1 + 0.2 * 0.6) = 12,4 mm

- ψ_2 = 0.2
- k_{def} = 0.6 (service class 1)

12,4mm < 14,5mm **X**

Table with recommended values of ψ factors for buildings.

Action	ψ_0	ψ_1	ψ_2
Imposed loads in buildings, see EN 1991-1-1			
Category A: domestic, residential areas	0,5	0,3	0,2
Category B: office areas	0,6	0,4	0,2
Category C: congregation areas	0,6	0,6	0,5
Category D: shopping areas	0,6	0,6	0,5
Category E: storage areas	0,8	0,8	0,7
Category F: traffic area, vehicle weight ≤ 30 kN	0,6	0,6	0,5
Category G: traffic area, 30 kN < vehicle weight ≤ 160 kN	0,6	0,4	0,2
Category H: roofs	0	0	0
Snow loads			
For combinations with leading imposed loads of category E	0,6	0,2	0
For combinations with leading wind actions	0	0	0
Otherwise	0,3	0,2	0
Wind actions			
For combinations with leading imposed loads of category E	0,6	0,2	0
For combinations with fire in an accidental design situation	-	-	0,2
Otherwise	0,3	0,2	0
Temperature	0,6	0,5	0

6.3.6.2 Selfweight

$$U_{inst,Q} = \frac{5 \cdot q \cdot l^4}{384 \cdot I_y \cdot E}$$

$= 5 * 0{,}39\text{N/mm} * 7500^4\text{mm} \; / \; 384 * 11000\text{mm}^4 * 595{,}5 * 10^6 =$ 1,45 mm

<u>Self weight load including the safety factors</u>

$U_{finG} = U_{inst,G} (1 + k_{def})$

$= 1{,}45\text{mm} * (1+ 0.6) = 2{,}32\text{mm}$

6.3.6.3 Total deflection

$U_{fin} = U_{fin,G} + U_{fin,Q}$

$= 12{,}4 \text{ mm} + 2{,}05 \text{ mm}$ = 14,45 mm

14,45 < 14,5 **X**

6.3.7 The minimum size of the bearing area

$$L_{min} = \frac{V_d}{f_{c90,d} * w}$$

- V_d = 2,76 kN
- $f_{c,90,d}$ = 0,23 N/mm^2

L_{min} = 2,76 * 10^{3}N / 0,23 N/mm^2 * 100 mm = 98mm

98 mm < 100 mm **X**

6.3.8 The final bending stress

σ_b = $M_d / W_y \leq f_{md}$

= 5,17 * 10^6 N/mm^2 / 561,8 * 10 ^{3}mm^3 = 9,2 MPa

9,2 MPa < 11,1 MPa **X**

6.3.9 The final shear stress

τ_d = 1,5 * V_d / A $\leq f_{vd}$

= 1,5 * 2,76 * 10^3 / 75mm * 212mm = 0,26 N/mm^2

0.26 MPa < 1,62 MPa **X**

The values obtained until here do not respect the reference levels, so that, a remark has to be done. The loadbearing capacity of this storey partition is not just determined by the joists in between the wood layers. Actually the whole component acts like a single element with its own loadbearing values. To find these values, it would be necessary to use a special calculation programme created by the same company that develops this product. In this case it would be impractible, so the best way to find the suitable dimensions for the component, according to the length and loadspan, is to look at the tables provided with the component.

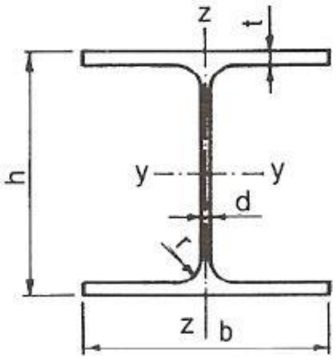
Table of loadbearing capacity values for the wooden storey partition.

Höhe		143		169		196		222		249		275		mm
L_{ef} [m]	Oberfläche	Akustik Fichte	Akustik Weisstanne	Akustik Fichte	Akustik Weisstanne	Akustik Fichte	Akustik Weisstanne	Akustik Fichte	Akustik Weisstanne	Akustik Fichte	Akustik Weisstanne	Akustik Fichte	Akustik Weisstanne	
2,50	EI_{ef}	189	465	466	874	893	1462	1512	2272	2354	3337	3451	4688	kNm²
	$M_{R,k}$	8,7	8,7	15,6	15,6	24,3	24,3	34,7	34,7	46,8	46,8	60,7	60,7	kNm
	$-M_{R,k}$	7,8	7,8	14,4	14,4	22,9	22,9	33,1	33,1	45,1	45,1	58,7	58,7	kNm
	$V_{R,k}$	30,9	30,9	52,4	52,4	66,3	66,3	80,1	80,1	93,8	93,8	107,4	107,4	kN
5,00	EI_{ef}	189	494	466	920	893	1527	1512	2362	2354	3455	3451	4839	kNm²
	$M_{R,k}$	8,7	8,7	15,6	15,6	24,3	24,3	34,7	34,7	46,8	46,8	60,7	60,7	kNm
	$-M_{R,k}$	7,8	7,8	14,4	14,4	22,9	22,9	33,1	33,1	45,1	45,1	58,7	58,7	kNm
	$V_{R,k}$	30,9	30,9	52,4	52,4	66,3	66,3	80,1	80,1	93,8	93,8	107,4	107,4	kN
7,50	EI_{ef}	189	500	466	929	893	1541	1512	2381	2354	3480	3451	4871	kNm²
	$M_{R,k}$	8,7	8,7	15,6	15,6	24,3	24,3	34,7	34,7	46,8	46,8	60,7	60,7	kNm
	$-M_{R,k}$	7,8	7,8	14,4	14,4	22,9	22,9	33,1	33,1	45,1	45,1	58,7	58,7	kNm
	$V_{R,k}$	30,9	30,9	52,4	52,4	66,3	66,3	80,1	80,1	93,8	93,8	107,4	107,4	kN

Table of deflection values for wooden storey partition.

		Elementhöhe	143	169	196	222	249	275	302	mm
LIGNO Decke Q3	Schub	$V_{R,k,y}$	21,5							kN
		GA_{ef}	1986							kN
	Biegung in Plattenebene	I_z	115	134	154	173	192	212		$10^3 cm^4$
		$M_{R,k,z}$	70,6	82,6	94,5	106,5	118,5	130,5		kNm
LIGNO Decke Q4	Schub	$V_{R,k,y}$	22,1							kN
		GA_{ef}	2339							kN
	Biegung in Plattenebene	I_z	120	141	163	185	207	229		$10^3 cm^4$
		$M_{R,k,z}$	73,6	87,1	100,6	114,1	127,5	141,0		kNm

6.4 Steel Beam (pos.4)



6.4.1 Material Data

Consequences classes: CC2 Medium

Cross section class: CS1

Normal inspection level: $\gamma_3 = 1.0$

Steel grade: S235 $f_y = 235$ N/mm^2

Safe factor $\gamma_{M0} = 1.1 * \gamma_3$ $\gamma_{M0} = 1.1 * 1.0 = 1.1$

Modulus of elasticity (E): $0{,}21 * 10^6$ N/mm^2

Beam length (l): 10,8 m

6.4.2 Line loads

Load span: 7,34 m

Imposed load (q_k): A1 dwellings qk

1,5kN/m^2 * 7,34 m = 11,01 kN/m

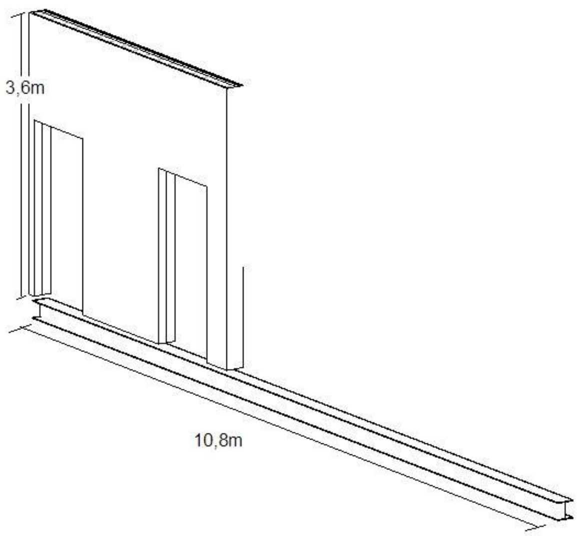
Self-weight (g): w * 7,34 m + 14,61kN / 10,8m

= 2,1kN/m^2 * 7,34 m + 14,61kN / 10,8m = 16,76 kN/m

Partition wall weight

Wall dimensions: 3,6m x 10,8m

Area partition wall: = 10,59 m^2



Weight partition wall (j): 1,38 kN/m^2

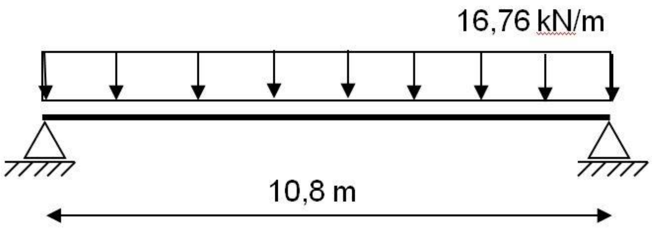
In 1 m^2 of partition wall there are 1,38kN/m, so in 10,59m^2 there are 14,61 kN

1 m^2 ------ 1,38 kN

10,59 m^2 ------------ y

(=) y = 10,59m^2 * 1,38kN/m / 1m^2

(=) y = 14,61 kN



Loads Diagram

6.4.3 Design Load

$E_d = \gamma g * g + \gamma q * q$

$E_d = 1{,}0 * 16{,}76kN/m + 1{,}5 * 11{,}01kN/m$ = 33,28 kN/m

6.4.4 Internal Forces

6.4.4.1) Shear forces

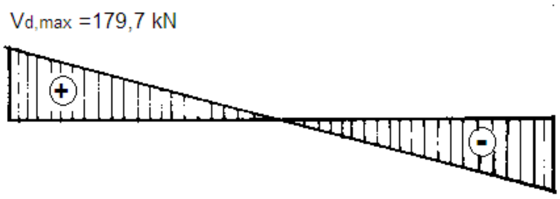
$V_{d,max} = 0.5 * E_d * l$

$V_{d,max} = 0.5 * 33{,}28kN/m * 10{,}8m$ = 179,7 kN

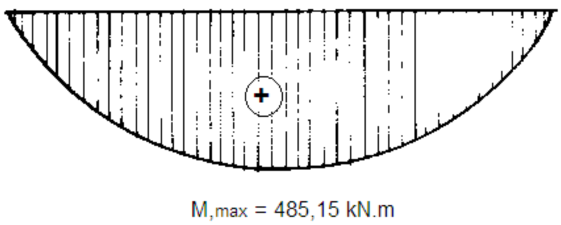
6.4.4.2) Bending forces

$M_{max} = 1/8 * E_d * l^2$

$M_{max} = 1/8 * 33{,}28kN/m * 10{,}8^2m$ = 485,15 kN.m



Shear Force diagram



Bending Moment diagram

6.4.5 Minimum section modulus (W_{min})

$$W_{min} = \frac{M_{max} \cdot \gamma_m}{f_y}$$

W_{min} = 485,15 * 10^6 N.mm * 1,1 / 235 N/mm^2 = 2270,9 * 10^3 mm^3

In this case we choose a HEB profile with these characteristics:

HEA profile section properties chart (*Compendium for Load bearing constructions 2. Semester Timber and steel beams)*

profil nr.	h mm	b mm	d mm	t mm	r mm	A mm^2	u m^2/m	g kg/m	I_y mm^4	$W_{el,y}$ mm^3	i_y mm	I_z mm^4	$W_{el,z}$ mm^3	i_z mm	I_v mm^4	I_w mm^6	W_{pl} [1)] mm^3
faktor	1	1	1	1	1	10^3	1	1	10^6	10^3	1	10^6	10^3	1	10^3	10^9	10^3
260*	250	260	7,5	12,5	24	8,68	1,48	68,2	104,5	836	110	36,7	282	65,0	526	516	920
280*	270	280	8	13	24	9,73	1,60	76,4	136,7	1010	119	47,6	340	70,0	624	785	1112
300*	290	300	8,5	14	27	11,2	1.72	88,3	182,6	1260	127	63,1	421	74,9	856	1200	1384

Now the calculations must be adjusted, since the self weight of the beam was not taken into account before.

6.4.2) Self-weight (G) = G + beam weight (g)

Self-weight (G) = 16,76kN/m + 0,87kN/m = 17,63 kN/m

6.4.3) E_d = ɤg * G + ɤq * Q

E_d = 1,0 * 17,63kN/m + 1,5 * 1,5kN/m = 19,88 kN/m

6.4.4.1) $V_{d,max}$ = 0.5 * E_d * l

$V_{d,max}$ = 0.5 * 19,88 kN/m * 10,8m = 107,35 kN

6.4.4.2) M_{max} = 1/8 * E_d * l²

M_{max} = 1/8 * 19,88 kN/m * $10,8^2$m = 289,85 kN.m

6.4.5) $W_{min} = \frac{M_{max} \cdot \gamma_m}{f_y}$

W_{min} = 289,85 * 10^6 N.mm * 1,1 / 235 N/mm^2 = 1356,75* 10^3 mm^3

6.4.6 Deflection

The limitation value of deflection for a steel beam in a roof structure is:

$$u_{max} = \frac{l}{400}$$

u_{max} = 10800mm / 400 = 27 mm

6.4.6.1) Self-weight (G)

$$U_{inst,G} = \frac{5 \cdot q \cdot l^4}{384 \cdot I_y \cdot E}$$

$U_{inst,G}$ = (5 * 17,63N/mm *10800^4) / (384 * 182,6 * 10^6 mm^4 * 0,21 * 10^6 N/mm^2)
= 81,45 mm

81,45 mm ≤ 27 mm **X**

At this point it is possible to claim that the deflection value for this beam (81,45mm) is above the maximum allowed (27mm), so that it is necessary to choose a bigger section with better properties.

HEA profile section properties chart (*Compendium for Load bearing constructions 2. Semester Timber and steel beams)*

profil nr.	h mm	b mm	d mm	t mm	r mm	A mm^2	u m^2/m	g kg/m	I_y mm^4	$W_{el,y}$ mm^3	i_y mm	I_z mm^4	$W_{el,z}$ mm^3	i_z mm	I_v mm^4	I_w mm^6	W_{pl} 1) mm^3
faktor	1	1	1	1	1	10^3	1	1	10^6	10^3	1	10^6	10^3	1	10^3	10^9	10^3
340*	330	300	9,5	16,5	27	13,3	1,79	105	276,9	1680	144	74,4	496	74,6	1280	1820	1850

Using the formula on 6.1.6.1 makes possible to verify different sections changing only the beam section self-weight and the I_y.

$U_{inst,G}$ = (5 * (16,76+ 1,03)N/mm *10800^4) / (384 * 276,9 * 10^6 mm^4 * 0,21 * 10^6 N/mm^2)
= 54,2 mm

54,2 mm ≤ 27 mm **X**

As the new chosen beam section does not satisfy the maximum value for deflection, it was decided to check other profiles, such as IPE, HEB and HEM. Keeping the same height of 330mm or close to it, when 330mm is not available, will allow a comparison between different profiles.

IPE

$U_{inst,G}$ = (5 * (16,76 + 0,48)N/mm * 10800^4) / (384 * 117,7 * 10^6 mm^4 * 0,21 * 10^6 N/mm^2)

= 123,56 mm

123,56 mm ≤ 27 mm **X**

HEB

$U_{inst,G}$ = (5 * (16,76 + 1,31)N/mm * 10800^4) / (384 * 366,6 * 10^6 mm^4 * 0,21 * 10^6 N/mm^2)

= 41,58 mm

41,58 mm ≤ 27 mm **X**

HEM

$U_{inst,G}$ = (5 * (16,76+ 2,33)N/mm * 10800^4) / (384 * 592 * 10^6 mm^4 * 0,21 * 10^6 N/mm^2)

= 54,2 mm

27mm ≤ 27 mm **OK**

Within the available section profiles with the height of 330mm/ 340mm, only the HEM complies with the maximum requirement for deflection. The difference between the values is lower than 1mm so this is the perfect section. In order to save some steel and even decrease the deflection, it is still possible to open some circular holes in the beam, along its length, as it is shown in the picture below.

The section implies a review in previous calculations, since the main parameters have changed.

HEM profile section properties chart (*Compendium for Load bearing constructions 2. Semester Timber and steel beams)*

profil nr.	h mm	b mm	d mm	t mm	r mm	A mm²	u m²/m	g kg/m	I_y mm⁴	$W_{el,y}$ mm³	i_y mm	I_z mm⁴	$W_{el,z}$ mm³	i_z mm	I_v mm⁴	I_w mm⁶	W_{pl} mm³
faktor	1	1	1	1	1	10^3	1	1	10^6	10^3	1	10^6	10^3	1	10^3	10^9	10^3
300	340	310	21	39	27	30,3	1,83	238	592,0	3480	140	194,0	1250	80,0	14100	4390	4080

6.4.2) Self-weight (G) = G + beam weight (g)

Self-weight (G) = 16,76kN/m + 2,33kN/m = 19,09 kN/m

6.4.3) $E_d = \gamma g * G + \gamma q * Q$

$E_d = 1{,}0 * 19{,}09kN/m + 1{,}5 * 1{,}5kN/m$ = 21,34 kN/m

6.4.4.1) $V_{d,max} = 0.5 * E_d * l$

$V_{d,max} = 0.5 * 21{,}34kN/m * 10{,}8m$ = 115,236 kN

6.4.4.2) $M_{max} = 1/8 * E_d * l^2$

$M_{max} = 1/8 * 21{,}34\ kN/m * 10{,}8^2 m$ = 311,14 kN.m

6.4.5) $W_{min} = \dfrac{M_{max} \cdot \gamma_m}{f_y}$

$W_{min} = 311{,}14 * 10^6\ N.mm * 1{,}1 / 235\ N/mm^2$ = $1456{,}4 * 10^3\ mm^3$

6.4.7 The final bending stress

$\sigma_b = \dfrac{M_{Ed}}{W}$ = $311{,}14 * 10^6 N.mm / 1456{,}4 * 10^3\ mm^3$ = $213{,}6\ N/mm^2$

$\dfrac{f_y}{\gamma_{M0}}$ = $235N/mm^2 / 1.1$ = $213\ N/mm^2$

$$\sigma_b = \frac{M_{Ed}}{W} \leq \frac{f_y}{\gamma_{M0}}$$

$213{,}6\ N/mm^2 \leq 213\ N/mm^2$ **OK**

6.4.8 The final shear stress

$$\tau_d = \frac{V_r\sqrt{3}}{A_v} = 115{,}236 \text{ kN} * 10^3 * \sqrt{3} \;/\; 9{,}045 * 10^3 \text{ mm}^2 \quad = \quad 22{,}06 \text{ N/mm}^2$$

$A_v = A - 2bt_f + (t_w + 2r)t_f$

$= 30{,}3 * 10^3 - 2 * 310 * 39 + (21 + 2 * 27) * 39$
$= 9{,}045 * 10^3 \text{ mm}^2$

$$\tau_d = \frac{V_r\sqrt{3}}{A_v} \leq \frac{f_y}{\gamma_{M0}}$$

$22{,}07 \text{ N/mm}^2 \leq 213 \text{ N/mm}^2$ **OK**

6.5 SL Slab (*pos. 5*)

6.5.1 Material Data

Consequences classes: CC2 Medium

Span: 18,25 m

6.5.2 Loads

Imposed load (q): A1 dwellings qk = $1{,}5 \text{ kN/m}^2$

Self-weight (g): = $6{,}49 \text{kN/m}^2$

6.5.3 Design Load

$S_d = \gamma_g * g + \gamma_q * q$

$S_d = 1,0 * 6,49 kN/m^2 + 1,5 * 1,5 kN/m^2$ = $8,74\ kN/m^2$

6.5.4 Design load deflection

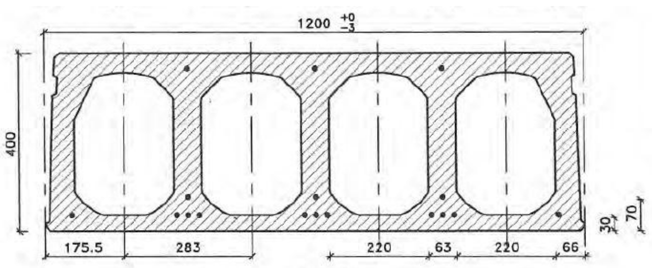
$S_d = g + 0,5 * q$

$S_d = 8,74 kN/m^2 + 0,5 * 1,5 kN/m^2$ = $9,49\ kN/m^2$

The values for the loadbearing capacity and design load deflection are below the maximum required for this component, according to the tables provided from the manufacturers, what means that its stability and safety are granted. Besides these factors, the quantity of material spent is also take into consideration, so that, it is possible to save material, while keeping the loadbearing and stability parameters.

(*still not available in the manufacturer website*)

<u>6.6 Hollowcore slab, d(*pos. 5*)</u>



6.6.1 Material Data

Consequences classes: CC3

Span: 11 m

6.6.2 Loads

Imposed load (q): C1 congregation room qk = $2,5\ kN/m^2$

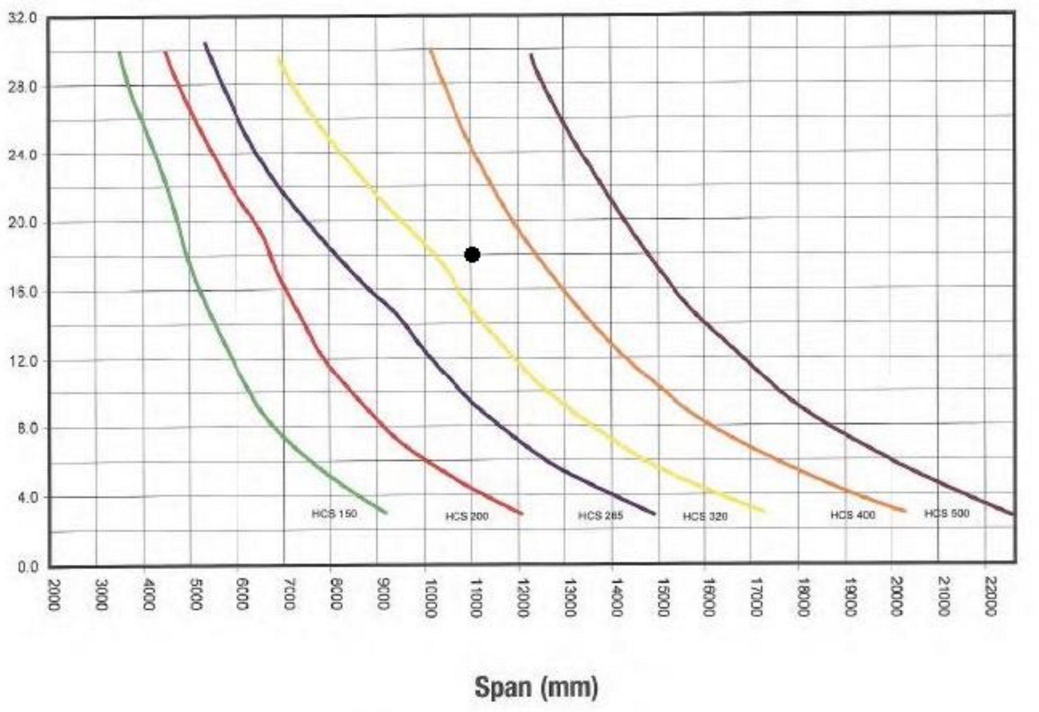
Self-weight (g): t = $14,76\ kN/m^2$

6.6.3 Design Load

$S_d = \gamma_g * g + \gamma_q * q$

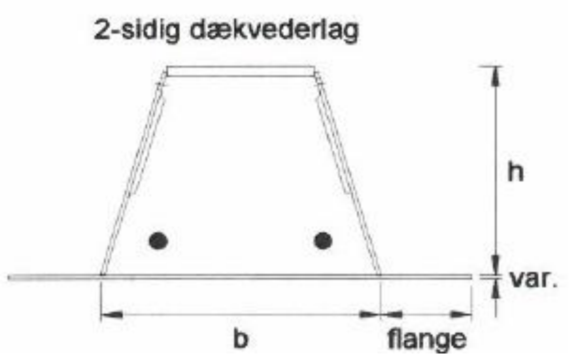
$S_d = 1{,}0 * 14{,}76\ kN/m^2 + 1{,}5 * 2{,}5 kN/m^2 \quad = \quad 18{,}51\ kN/m^2$

Graph showing the load curves for hollowcore slabs.



According to the chart provided by the manufacturer a hollowcore with 11m in the length and supporting a load of 18,51 kN/m^2, can be find in the orange curve - HCS 400, what means the thickness for this element is 400mm. The width doesn't change, it is always 1200mm. The weight for this hollowcore slab is 4,69 kN/m^2.

6.7 Delta Beam (pos.6)



6.7.1 Material Data

Consequences classes: CC3

Delta beam length (l): 7,5 m

6.7.2 Line loads

Load span: 11,7 m

Self-weight (g): r * 11,7m + d * 11,7

14,76 kN/m^2 * 11,7m + 4,69 kN/m^2 * 11,7 = 227,57 kN/m

Imposed Load (q): C1 Congregate room qk

2,5kN/m^2 * 11,7m = 29,25 kN/m

6.7.3 Design Load

$E_d = \gamma g * g + \gamma q * q$

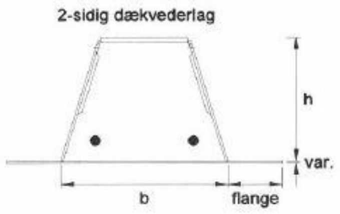
E_d = 1,0 * 227,57kN/m + 1,5 * 29,25kN/m = 241,45 kN/m

Once found the load span it is possible to look at the graph below and take the suitable dimensions for a delta beam with these values for the length and load span. It should be taken into consideration that the delta beam is going to be placed between hollowcores, so its height needs to fit the hollowcores (400mm). The following table shows the different dimensions available.

Table with the available sizes for delta beams.

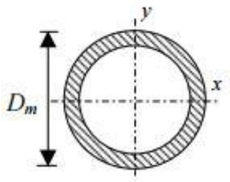
BÆREEVNETABEL - DELTABJÆLKER

	Positivt moment	Negativt moment	Tværsnitsdata		
			h	b	flange
Maksværdier:	M_{Rd}+ [kNm]	M_{Rd} - [kNm]	[mm]	[mm]	[mm]
D20-200	288.94	245.19	200	200	97.5
D20-300	427.08	383.81	200	300	97.5
D20-400	596.88	556.01	200	400	130
D22-300	465.93	411.90	220	300	97.5
D22-400	654.33	602.07	220	400	130
D25-300*	551.92	444.60	250	300	97.5
D25-400*	785.76	679.10	250	400	130
D26-300	537.37	432.78	265	300	97.5
D26-400	777.92	659.71	265	400	130
D30-300*	596.99	456.34	300	300	97.5
D30-400*	875.74	717.96	300	400	130
D32-300	605.42	440.29	320	300	97.5
D32-400	900.12	720.63	320	400	130
D37-400	1011.50	768.40	370	400	130
D37-500	1445.67	1200.73	370	500	130
D40-400	1120.51	854.02	400	400	130
D40-500	1449.79	1200.67	400	500	130
D50-500	1766.21	1387.50	500	500	130
D50-600	2157.34	1819.63	500	600	130



Through this table it is possible to conclude that the most suitable beam belongs to the category D-40 or D-50. Within this group of four beams, it will be chosen D40-500 because it has the same height as the hollowcore, already sized, and a bigger base.

6.8) HSS Composite column - steel column



6.8.1) Material data

Effective length of the column: 14 m

To size the column, first, is necessary to find the line loads carried by the beams that are connected to it (B1, B2). After finding the total amount of line loads, it will be converted into a point load.

6.8.2 Line loads

B1:

Effective length (b_1): 16,9 m

Weight: 1,04 kN/m

Load span: 7,75 m

Self-weight (g): r * 7,75m + 1,01kN/m = 1,04kN/m^2 * 7,75 + 1,01kN/m

= 9,07 kN/m

Snow load (s): s * 7,75m = 0,8 kN/m^2 * 7,75m = 6,2 kN/m

Design Load

E_d = ɤg * g + ɤs * s

E_d = 1,0 * 9,07kN/m + 1,5 * 6,2kN/m = 18,37 kN/m

B2:

Effective length (b_2): 15,5 m

Weight: 1,04 kN/m

Load span: 8,45 m

Self-weight (g): r * 8,45m + 1,01kN/m = 1,04kN/m^2 * 8,45m + 1,01kN/m

= 9,8 kN/m

Snow load (s): s * 8,45m = 0,8 kN/m^2 * 8,45m

= 6,76 kN/m

Design Load

E_d = ɤg * g + ɤs * s

E_d = 1,0 * 9,8kN/m + 1,5 * 6,76kN/m = 19,94 kN/m

6.8.3 Point Loads

B1

Load span: 8,45 m

Q (B_1) = E_d * 8,45m = 18,37kN/m * 8,45m

= 155,23 kN

B2

Load span: 7,75 m

Q (B_2) = Ed * 7,75m = 19,94kN/m * 7,75m

= 154,54 kN

Total: B1 + B2 = 155,23 + 154,54 = 309,77 kN

Table with loadbearing capacity for circular hollow section steel column.

	knæk-længde m	d_y (mm) 26.9	33.7	42.4	48.3	60.3	76.1	88.9	114.3	139.7	165.1
Middel svære	1,5	10,1	23,2	44,1	60,1	102	143	193	288	389	469
rør	2,0	5,91	13,9	28,1	40,8	79,4	126	178	276	378	458
	2,5		9,18	18,9	28,0	58,2	104	157	260	364	446
DIN 2440	3,0			13,5	20,1	42,9	81,5	131	238	346	432
S235	3,5				15,1	32,6	63,7	107	211	324	414
	4,0					25,5	50,6	86,4	182	297	393
	5,0					16,7	33,7	58,6	132	235	338
	6,0						24,0	42,0	96,6	181	276
	8,0							24,3	57,2	110	177
Svære	1,5	11,6	27,3	52,2	72,0	122	174	229	344	433	521
rør	2,0	6,77	16,3	33,1	48,5	94,7	153	211	329	420	509
	2,5		10,7	22,2	33,2	68,9	125	185	309	404	495
DIN 2441	3,0			15,8	23,8	50,7	97,9	154	282	384	479
S235	3,5				17,8	38,5	76,5	125	250	359	460
	4,0					30,1	60,7	101	215	329	436
	5,0						40,4	68,7	155	260	374
	6,0						28,7	49,1	114	199	305
	8,0								67,2	121	196

The chosen column is bigger than any other showed in the table above, so to find out the most suitable diameter for the steel column it is needed to use a special online tool provided by the manufacturer that helps sizing bigger sizes. So, with an effective length of 14m and a load of 310kN (approximately) the most suitable diameter is 405. This diameter is not included in the tables properties for this section, so the following one will be chosen - 406,4 mm.

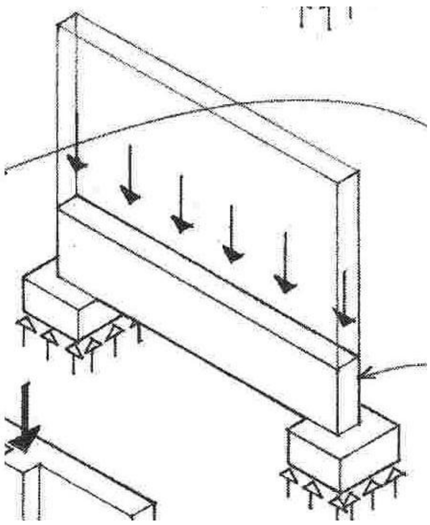
Table with dimensions and cross-sectional properties for circular hollow sections.

d_y mm	t mm	A mm^2	u m^2/m	g kg/m	I mm^4	W_{el} mm^3	i mm	I_v mm^4	W_v mm^3
faktor		10^3	1	1	10^6	10^3	1	10^6	10^3
355,6	8	8,74	1,12	68,6	132,0	742	123	264	1480
406,4	8,8	11,0	1,28	86,3	217,3	1069	141	435	2140
457	10	14,0	1,44	110	350,9	1536	158	702	3070

The column was sized as a steel column using a round hollow section, however it should be considered that the inner space can be filled with reinforced concrete what will increase the stability and loadbearing capacity of the whole column.

6.9) Concrete ground beam

The final element that needs to be sized are the ground beams made in concrete. The ground beams receive the loads from upper loadbearing walls (reinforced concrete foundation walls) and unload them into the footings, to which they are cantilevered (all movements restrained) by steel bars embedded in concrete. The footings will them spread these loads all over their area to the soil, which will offer resistance to support these loads.



6.9.1 Material Data

Consequences classes:	CC2 Medium
Inspection level:	Normal : γ_3 = 1.0
Environmental class	Passive
Min. cover	15 mm
Concrete strength class, min.	20 MPa

Safe factor $\gamma_{M0} = 1.1 * \gamma_3$ $\gamma_{M0} = 1.1 * 1.0 = 1.1$

f_{ck} = 20 MPa

f_{cd} = 13,8 MPa

ω_{min} = 0,035

f_{yd} = 458 MPa

ω_{bal} = 0,483

$f_{yk\ stirrups}$ = 410 MPa

Beam length (l): 9,3 m

6.9.2 Line loads

Load span: 4,4 m

Imposed load (q_k): A1 dwellings qk

1,5kN/m^2 * 4,4 m = 6,6 kN/m

Self-weight (g): g_{beam} + h + $g_{hollowcore}$ + $g_{earrh\ wall}$

= (b * h * $P_{concrete}$) + 3,06kN/m^2 * 4,4m + 3,3kN/m^2 * 4,4m +409,2/ 9,3m

= 0,65m * 0,65m * 24kN/m3 + 13,46kN/m + 14,5kN/m + 44kN/m

= 82,1 kN/m

Rammed earth wall weight

Wall dimensions: 5m x 9,3m

Area partition wall: = 46,5 m^2

Weight rammed earth wall (c): 8,8 kN/m^2

In 1 m^2 of partition wall there are 8,8 kN/m, so in 46,5 m^2 there are kN

1 m^2 ------ 8,8 kN

46,5 m^2 ------------ y

(=) $y = 46{,}5m^2 * 8{,}8kN/m / 1m^2$

(=) $y = 409{,}2$ kN

6.9.3 Design Load

$E_d = \gamma g * g + \gamma q * q$

$E_d = 1{,}0 * 82{,}1kN/m + 1{,}5 * 6{,}6kN/m$ = 92 kN/m

6.9.4 Internal Forces

6.9.4.1) Shear forces

$V_{d,max} = 0.5 * E_d * l$

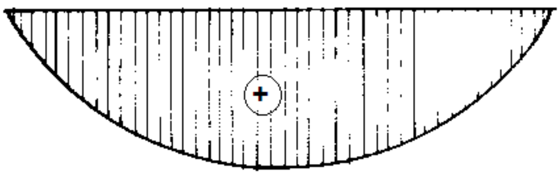
$V_{d,max} = 0.5 * 92kN/m * 9{,}3m$ = 427,8 kN

6.9.4.2) Bending forces

$M_{max} = 1/8 * E_d * l^2$

$M_{max} = 1/8 * 92kN/m * 9{,}3^2m$ = 994,6 kN.m

$V_{d,max} = 427{,}8$ kN



$M_{max} = 994{,}6$ kN.m

6.9.5 Reinforcement bars

6.9.5.1) Amount of reinforcement bars

$$A_{s\min} \cong \frac{M_d}{f_{yd} * 0{,}8 * h}$$

A_{smin} = 994,6kN.m * 10^6/ 458 * 0,8 * 650mm

A_{smin} = 4176,2 mm^2

6.9.5.2)Size and number of reinforcement bars

Cross sectional area in mm^2 (5.3.1.2 i TS)

d (mm)	number of bars 1	2	3	4	5	6	7	8	9	10
6	28	57	85	113	141	170	198	226	254	283
8	50	101	151	201	251	302	352	402	452	503
10	79	157	236	314	393	471	550	628	707	785
12	113	226	339	452	565	679	792	905	1018	1131
14	154	308	462	616	770	924	1078	1232	1385	1539
16	201	402	603	804	1005	1206	1407	1608	1810	2011
18	254	509	763	1018	1272	1527	1781	2036	2290	2545
20	314	628	942	1257	1571	1885	2199	2513	2827	3142
25	491	982	1473	1964	2454	2945	3436	3927	4418	4909
32	804	1608	2413	3217	4021	4825	5630	6434	7238	8042

8 pcs ø 25 are chosen because A_s = 4176,2 mm^2

The reinforcement bars are placed in two layers: 6 in the bottom and 3 in the top layer of the concrete beam.

6.9.6 Minimum width of the beam

$$a \geq \begin{cases} ø \\ d_g + 5\ mm \\ 20\ mm \end{cases}$$

$$a \geq \begin{cases} ø = 25 \text{ mm} \\ d_g = 32 + 5 = 37 \text{ mm} \\ 20 \text{ mm} \end{cases}$$

$$c_1 \geq \begin{cases} ø + \text{deviation addition for dg} < 32 \text{ mm} \\ ø + \text{deviation addition} + 5 \text{ mm for dg} > 32 \text{ mm} \\ c + ø_t \end{cases}$$

$$c_1 \geq \begin{cases} 25 \text{ mm} + 5 \text{ mm} = 30 \text{ mm} \\ 35 \text{ mm} + 7 \text{ mm} = 42 \text{ mm} \end{cases}$$

With these values, the minimum base for the beam can be calculated:

b_{min} = 2 * 42 + 4 * 37 + 5 * 25

b_{min} = 357 mm

The chosen section has b = 600 mm > 357 mm **OK**

6.9.7 Reinforcement ratio

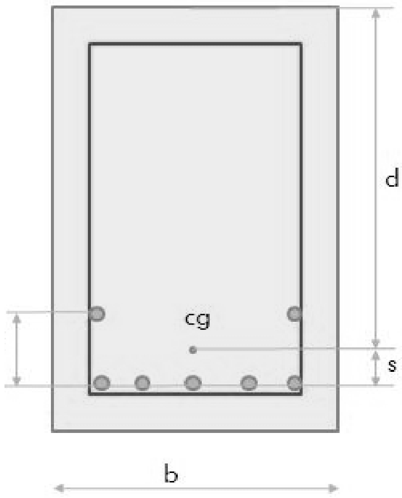
$$\omega = \frac{A_s * f_{yd}}{b * d * f_{cd}}$$

ω = 4176,2 * 458 / 600 * 527,79* 13,8

ω = 0,438

$\omega_{min} < \omega < \omega_{bal}$

0,035 < 0,438 < 0,483



Auxiliary calculations:

12,5 + 37 + 12,5 = 62 mm

S = 62 * 2 / 7 = 17,71 mm

d = 600 - (35 + 7 + 12,5 + 17,71)

d = 527,79 mm

6.9.8 Ultimate Moment of resistance M_{Rd}

$$M_{Rd} = A_s * f_{yd}(d - \frac{2}{5}x)$$

The only value that is still not known at this point is the "x". To find it the equilibrium condition ΣN = 0 is used:

$$\frac{4}{5} * b * x * f_{cd} - A_s * f_{yd} = 0$$

0,8 * 600 * x * 13,8 - 4176,2 * 458 = 0

x = 4176,2 * 458 / 0,8 * 600 * 13,8

x = 28,94 mm

MRd = 4176,2 * 458 * (527,79 - 0,4 * 28,94) * 10^{-6}

MRd = 987,4

M_{Rd} > MSd

987,4 kN.m > 994,6 kN.m **OK**

6.9.9 Maximum spacing between the Stirrup Reinforcement

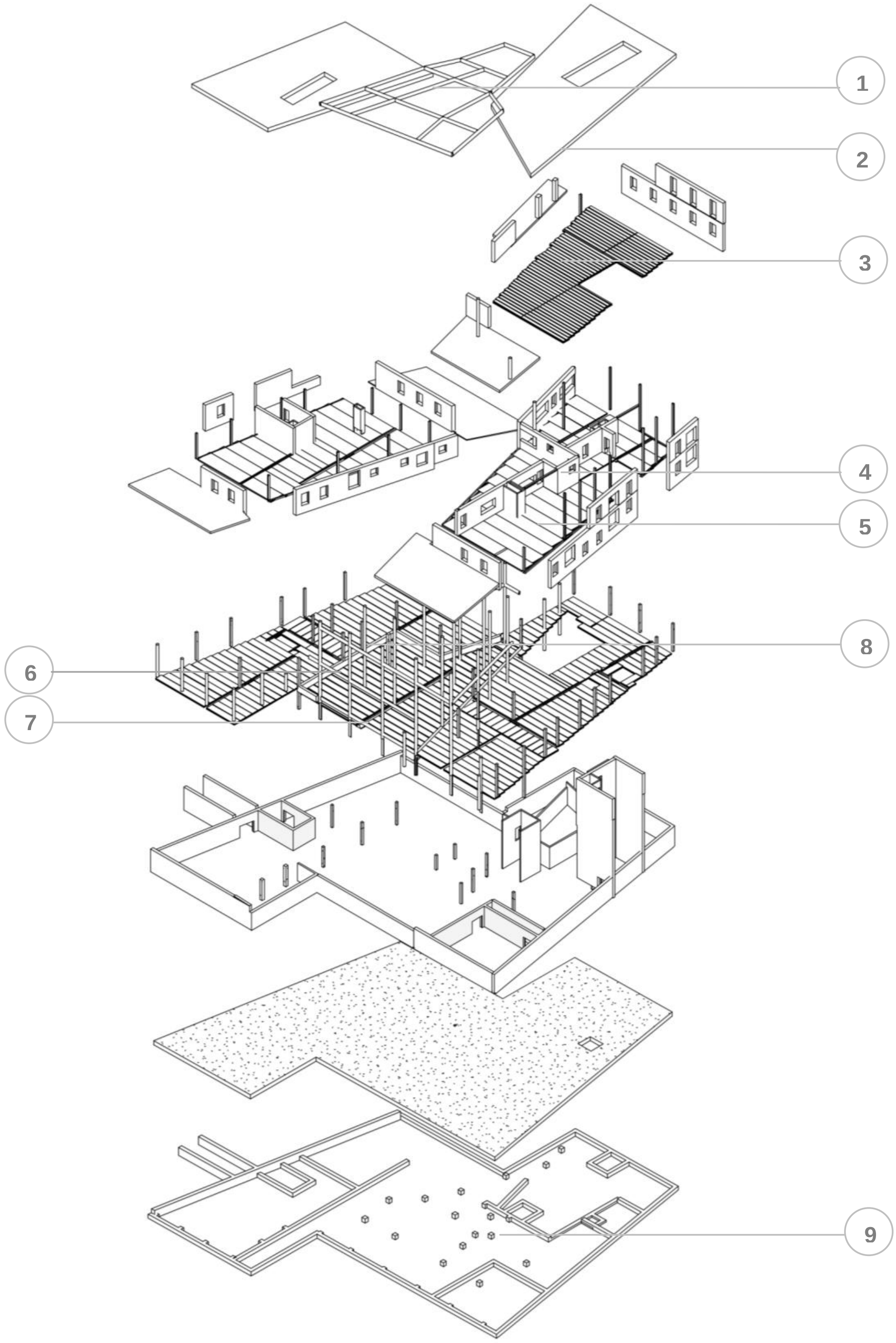
$$s \leq \begin{cases} 0{,}75d \\ 15.9 \dfrac{A_{sw}}{b_w} \dfrac{f_{yk}}{\sqrt{f_{ck}}} \end{cases}$$

$$s \leq \begin{cases} 0{,}75 * 527{,}79 = 395{,}8 \text{ mm} \\ 15{,}9 * 2 * (3{,}5^2 * \pi) * 410 / 600 * \sqrt{20} = 187 \text{ mm} \end{cases}$$

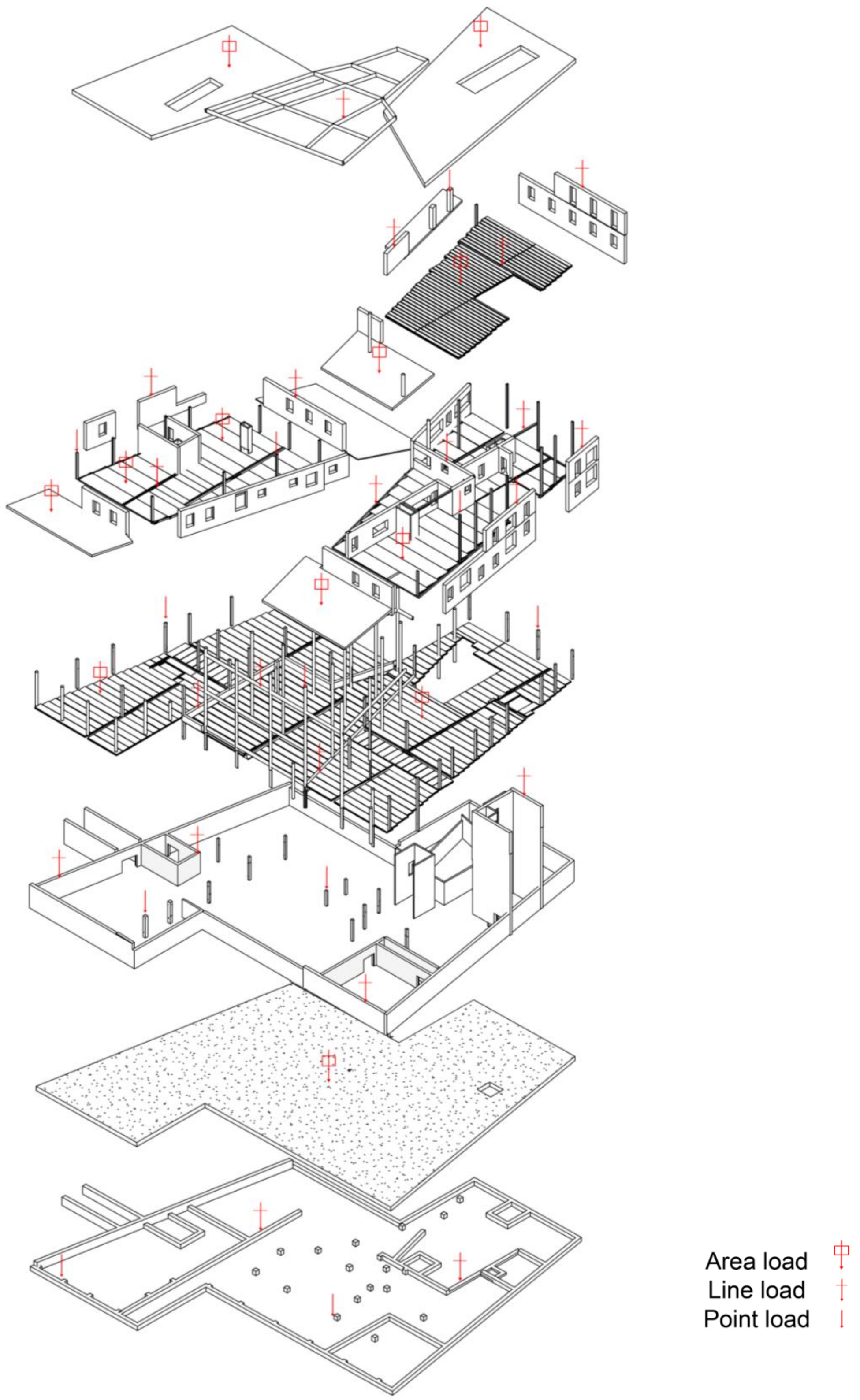
The distance between the stirrup must be less or equal to 187 mm. We chose 180 mm.

7. Exploded Drawing

7.1 Position level



7.1 Vertical load path diagram



7.2 Horizontal load path diagram