ART GALLERY
_LLOYDKADE

design_exercise_group7

CTB3340-15_building_structures_1

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GROUP 7

1. Description of location and Building

The building which is to be developed is located at Lloydkade Rotterdam which fronts towards the Maas river. It is a one storey building which is proposed to be used for a commercial purpose consisting of an art gallery together an office (with storage). For the building structure, insitu, precast concrete and timber will be used, while a glass structure will be used to connect the office and the art gallery.

The art gallery is available for the public, while the office and storage will only be for the employees.

- Art gallery on both floors: area is (10*15) = 150m^2
- Storage on ground floor : area is (4*10)+(6*10) = 100m^2
- Office on first floor : area is (12*8) = 96m^2

As earlier stated, the building is a one storey made up of the ground and first floor which will both be connected with a staircase and an elevator. On the ground floor, the entrance will be located with an area of 100m^2 including the art gallery and with transparent facades towards the orientation to the south and north having an area of 150m^2 and 96m^2 respectively. At the entrance of the building a small glass structure will be designed, with glass facades, and a glass floor. The glass structure will be a glass box which will be designed with an elevator in the corner, in this way the elevator in the corner of the room will support the first floor enabling to use slimmer structural components.

As earlier stated, the ground floor and first floor will be connected with a staircase and an elevator, the staircase will be slightly off center in the room starting 3 meters from the side with the glass floor and the elevator will be located in the lobby next to the staircase in order to provide easy accessibility for workers and visitors.

Since the building is one storey, a standard elevator will be installed with a capacity of 320kg, with shank size 1.3m (B) X 1.45m (D) with a cabin size of 0.9m (B) x 1m (D). On the first floor, there is an office and art gallery which will be separated by a glass floor. The art gallery has an area of 150m^2, while the office has an area of about 96m^2 respectively. The length of the cantilevering roof over the office and gallery are 3 meters and 2 meters respectively. Since the cantilevering roof spans over the terrace as well, this length would also be taken into account for the terrace on the both sides.

Having stated the dimensions of the ground and first floor, other basic parts which will be included in the building are namely the strip window, roof, façade, insulation etc. For the roof, steel cladding will be used because it protects against corrosion, which makes the wear protection reliable over a long period, thus making the roof cost-effective on the long run. More so, a steel cladding will be used because it also increases the mechanical strength of the building, hence making it more resistant to cracks caused by temperature, sunlight etc.

On the east and west side of the building, the cladding will be made using the masonry/concrete at the bottom and timber for the first floor.

For the façade cladding, double glass will be used on the side of the office and art gallery in order to increase the energy efficiency of the building, this is because the temperature of the inner glass is kept much lower, hence less heat is transferred from the building's exterior to its interior thus creating a comfortable indoor environment.

1.1 Windows for Office and Art gallery

One of the main design concepts of the architect was to have the building as transparent as possible i.e. a north and south facing façade. This is why in the construction design, this concept will be held in mind while designing parts such as roofs and shading.

Starting with the windows on the ground floor, the storage facing south does not necessarily need to be completely transparent as giving it a transparent look would not be entirely beneficial for the general looks of the building. Its main function is storage. This is why a small strip of windows will be placed on the top of the façade so the space will be able to benefit from natural light.

The art gallery part of the ground and first floor will have a consistent façade, this being one fully made out of glass. The detailing of these façades and choice of glass type will be further elaborated on in future chapters. As the north side of the building won't be exposed.

For the office space, the design idea of a transparent façade will also be used just like the art gallery floors. However, because the sun directly shines onto the south facing the office space, a cantilever roof will be constructed to block most of the summertime sunrays and allow the winter sun to provide natural lighting and passive solar heating in the colder periods. The figures below illustrates the facades towards the south and north.

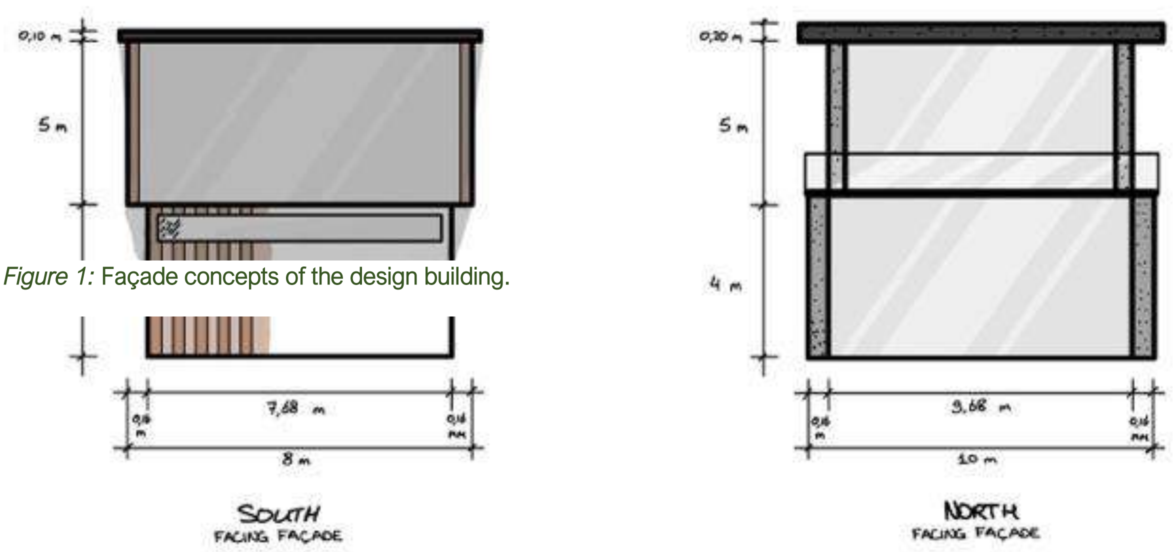


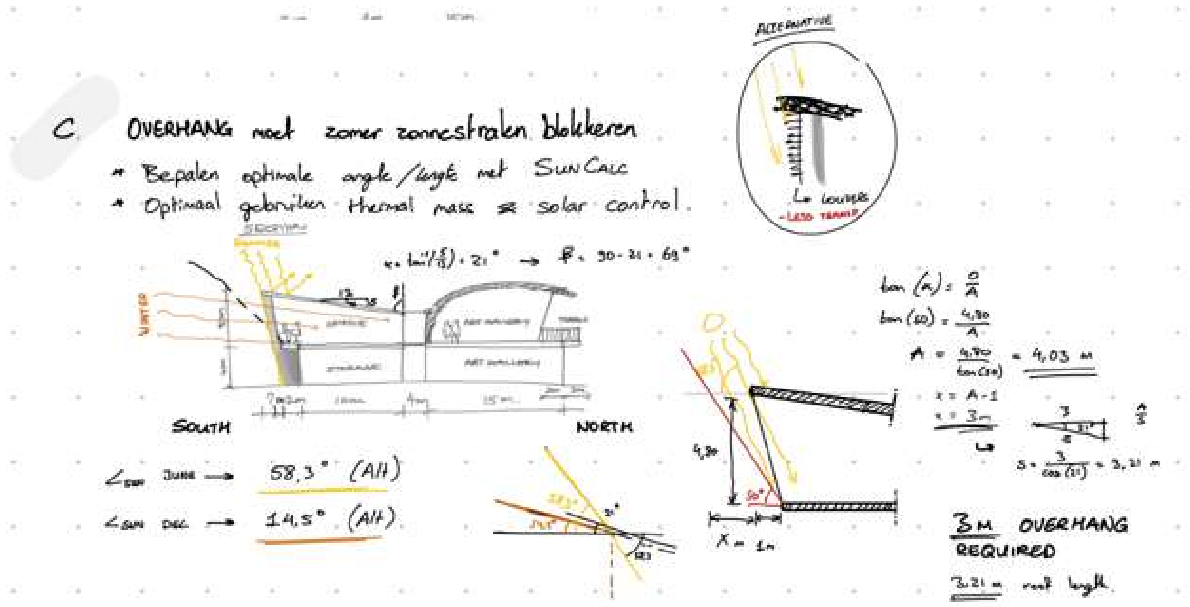
Figure 1: Façade concepts of the design building.

1.2 Length of the cantilevering roof of the gallery

Under the cantilevering roof, facing northwards is a balcony on top at the ground floor of the art gallery. To create a pleasant environment where the balcony is mostly covered from rain and downpour, the roof is extended to the edge of the balcony. This is an overhang of around 2 meters.

1.3 Length of the cantilevering roof of the office

The calculations around the optimal length to block most of the summertime sunrays is found in figure 2. The total horizontal length of the overhang comes down to 3 meters, just like the art gallery overhang. The actual roof length is 3,31 meters.



The material still needs to be chosen. This will be further elaborated in the following questions of alternatives.

1.4 Positioning of staircase and elevator

Elevator

We proposed to have the elevator in the middle part of the entire structure, the glass bridge, in order to connect both buildings and save space and cost of a possible double elevator setup or odd pathing for the end users. The placement will be on the east side of the structure, and as far south as possible, being connected to the office building.

Staircase

We propose to have two staircases, thus one in each building. On the ground floor, a spiral glass staircase will be located in the middle of the art gallery keeping the transparency theme consistent in that space and for the storage a simple staircase will be located next to the loadbearing wall on the west side of the building.

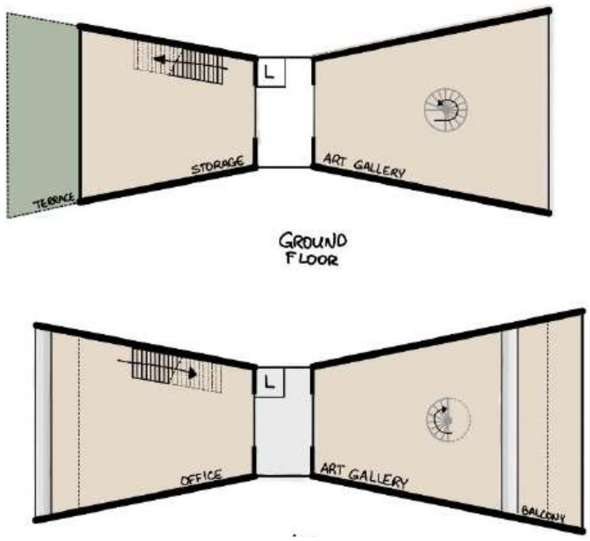


Figure 3: Floorplan and stair lay out of the buildings.

Insulation

For the east and west side of the building, two parallel walls will be used with insulation between the two walls. The outer wall will be masonry and the inside wall will be made of concrete. Towards the north and south facade double glass is used.

<u>Façade cladding</u>

We are going to consider different types of materials. Using the wood cladding, steel or the concrete. If we are going to use a wood cladding, then we are going to make use of the longitudinal strips as seen in the figure 4 below.

Wood

Steel

Concrete

Figure 4: Wooden, steel and concrete façade cladding

Roof

For the roof steel cladding will be used due to the fact that this type of roof helps to increase the mechanical stability of the overall structure. More so, its ability to fight off the harsh elements, thus protecting the structure from changes in temperature, wind, water absorption, sunlight and which could damage the structural integrity of a building.

1.5 Description of material qualities and loads

For the building structure, the materials to be used are namely:

- Concrete
- Steel
- Timber
- Glass

On the choice of materials, some quality checks are done are namely:

- Strength
- Density
- Elasticity

Concrete

The compressive strength of concrete determines the load bearing capacity of a concrete construction. For the building structure, the concrete class C30/37 was chosen based on the mechanical strength, durability etc. More so, because of its durability to withstand or resist abrasion while maintaining its desired engineering properties. For the building, the concrete with a high compressive strength will be used because due to the provided reinforcement, it can withstand a good amount of tensile strength of the building structure.

Steel

For the steel reinforcement, the qualities put into consideration for the choice of the steel includes the tensile and yield strength. The S460 was chosen because of its high yield point and ultimate tensile strength thus its ability to carry dynamically loaded structures such as: columns, platforms etc.

Timber

For the choice of timber, the GL24h will be used because this type of timber is suitable for application in storey buildings. More so, the GL24h has dimensional stability i.e. it remains stable for years and also retains its aesthetic appeal thus serving the purpose for the building structure. It also has high strength, high load bearing capacity with low density. This type of timber can also be used for columns and main beams with large spans. The other qualities which were considered for the GL24h includes the bending strength, tensile strength, modulus of elasticity, etc.

Glass

For the part of the building to constructed with glass, the thermically reinforced float glass will be used imploring the use of the DSF system to increase the energy efficiency of the building structure. In addition, because this system consists of three components namely the exterior wall, interior wall and ventilated cavity, it helps to protect the building against the weather. For the building structure, the glass part will have an outer layer of about 6mm, the void and also the inner layer which will serve for the actual load bearing. The float glass will be used because of the hardness and also because it is a great insulator and has a low expansion. More so, other qualities considered for the glass is also the scratch resistance i.e. its ability to withstand scratches, which is why the float glass will be used because it is robust against scratches.

2. Description of load bearing behavior

To provide stability by means of braced and unbraced structures in the North-South and East-West direction of the building, different possibilities were analyzed in order to see how load can be transferred horizontally and vertically.
For the horizontal and vertical load transfer, the loads which are considered are namely the floor load and wind load. The former is the vertical load which are forces that are applied perpendicular to the floor so as previously stated, while the latter is the horizontal load i.e. loads that are applied parallel to the ground with horizontal forces acting on the building structure.
Looking at the horizontal load transfer, on the ground floor, concrete walls will be used to handle the load from the first floor (i.e. from
the office and gallery) together with columns in the room itself, which will be made out of concrete.
For the vertical load transfer i.e. the wind load, it is analyzed how the wind forces will be transferred from the roof to the foundation in both the East-West and North-South directions. For all three alternatives the same bracing method is used in the East-West direction, as this made the most sense for a symmetric façade and to leave space for a doorway in the center, for the connecting glass structure.

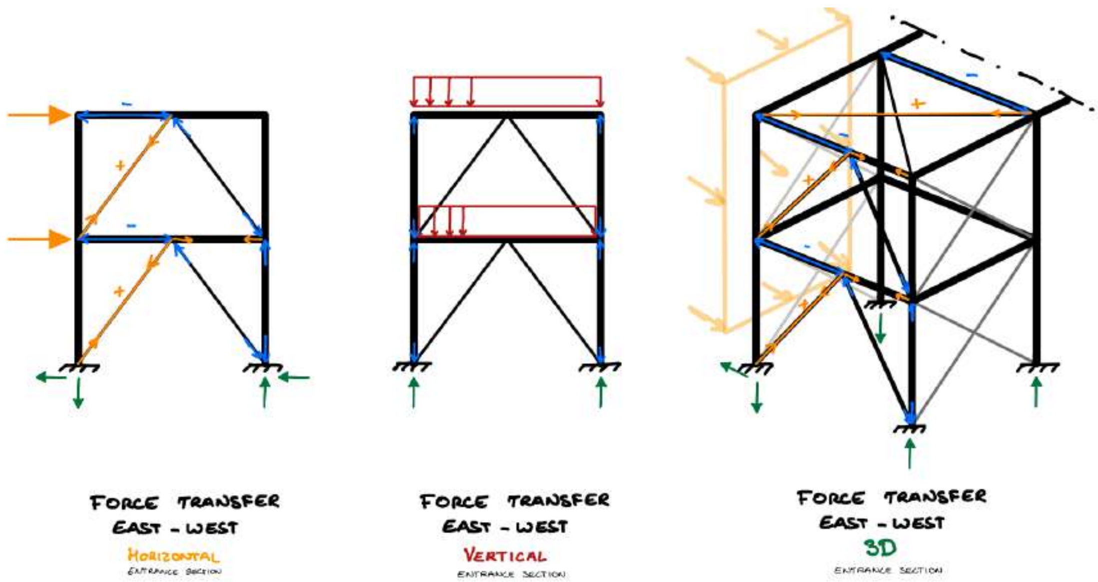


Figure 5: Force transfer and load bearing behavior

Braced structures have different possibilities for the location of the bracing but it is quite important that there is room left for the doors, entrances to the buildings. Unbraced structures can be supported by in-situ casted concrete, connecting columns and beams. Below are sketches of the East-West & North-South façade(see figure 6&7).

East – West façade

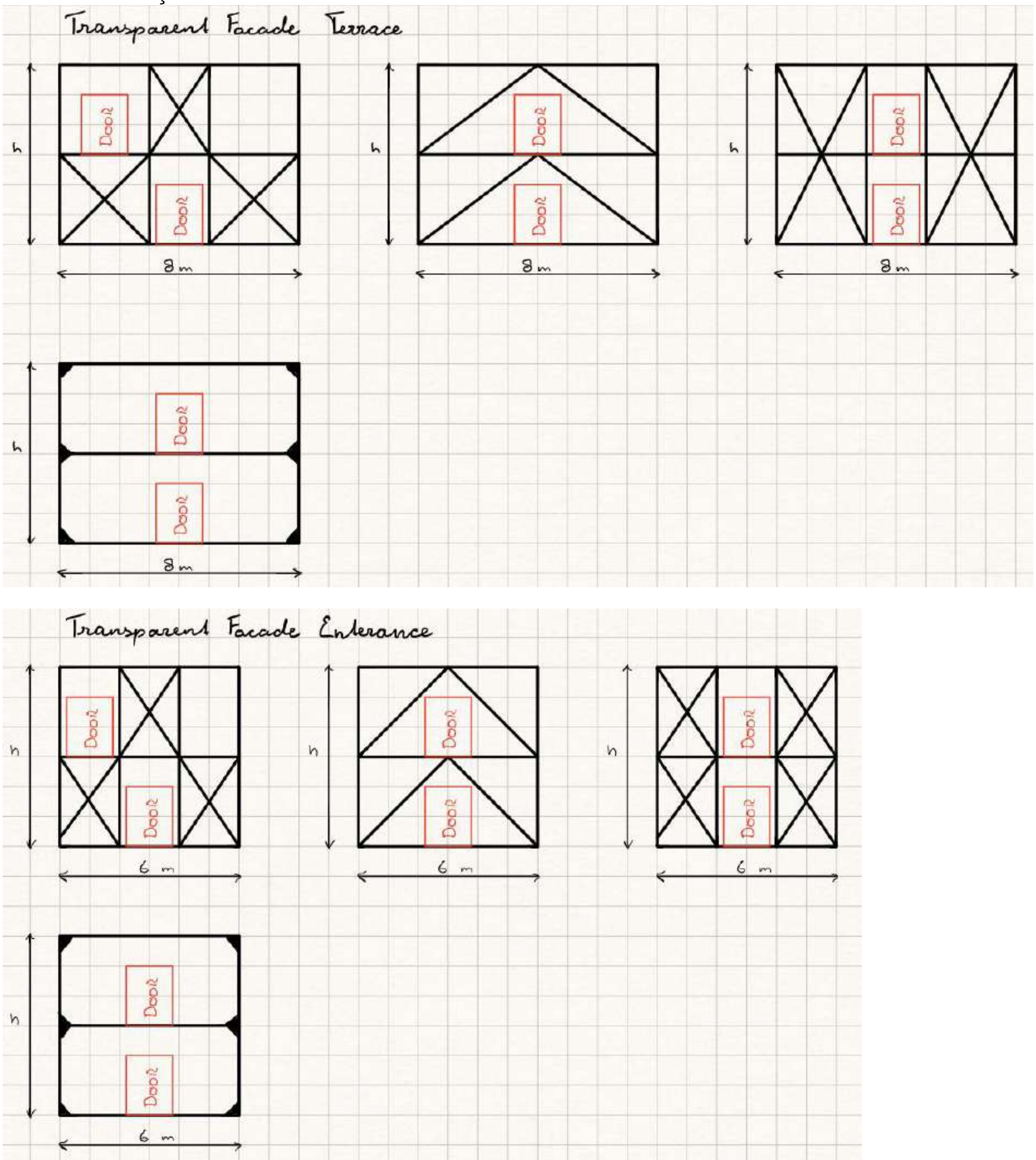


Figure 6: East-West façade

Figure 7 : North-South façade

2.1 Positioning of vertical load bearing elements of the office and art gallery (walls and columns) for both ground floor and first floor

In the building the main public attraction and experience will be in the art gallery. In order to keep the transparency concept and design in mind with the construction. The art gallery space will be clear of any columns. This will be also necessary in order to create the glass spiral staircase in the middle of the art gallery to stand out when looking at it.

As for the vertical load bearing elements, the main load bearing elements will be the east and west facing walls. These walls will consist of multiple reinforced columns with corbels in order to hold the supporting beams. On top of these beams the floorplates will be placed.

This results in the floor plans in figure 8 with all necessary spans presented.

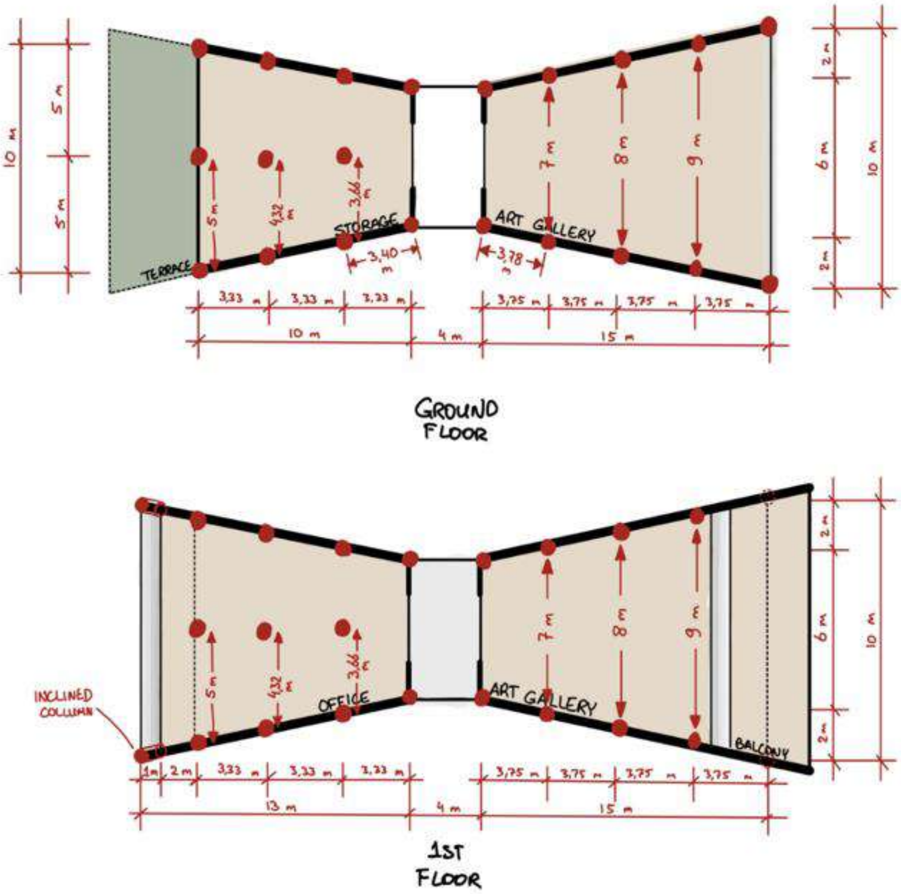


Figure 8: Column plan of both ground and first floor of the structure.

2.2 Wind loads

<u>Transfer of Wind forces from roof to the foundation (North-South)</u>

The building transfers the wind loads on the north and south façade through the floors into the beams and over to the bracings. Depending on the direction of the wind, the main bracing elements that are loaded are the ones of the building of the wind load direction. While both buildings are transferring loads to the main building.

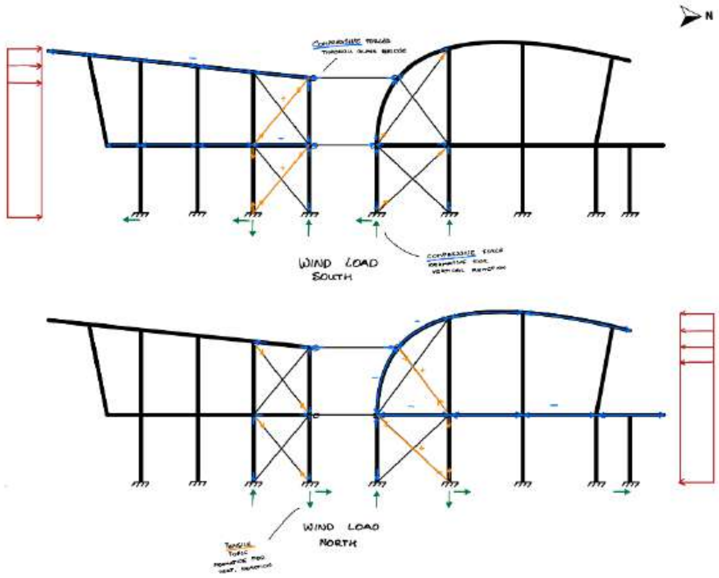


Figure 9: Horizontal wind load transfer to foundation in the North-South direction.

<u>Transfer of Wind forces from roof to the foundation East-West</u>

For all three alternatives the same bracing method is used in the East-West direction, as this made the most sense for a symmetric façade and to leave space for a doorway in the center, for the connecting glass structure.

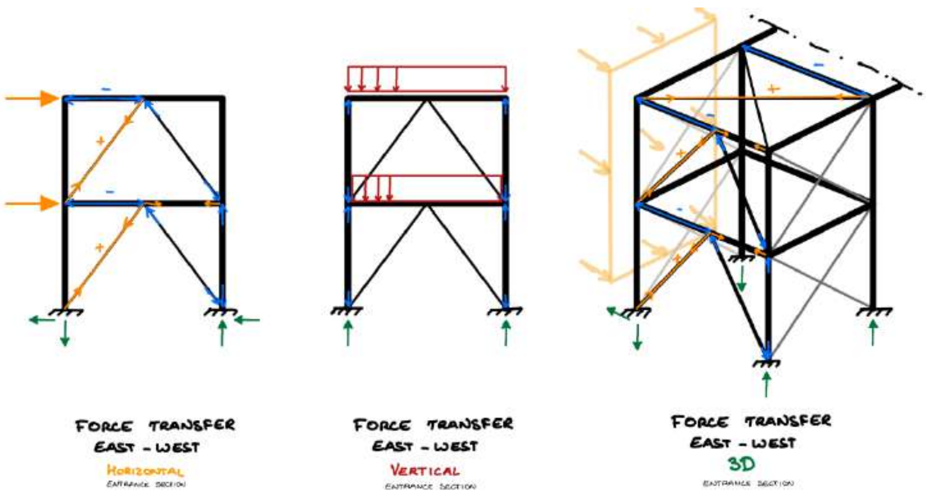


Figure 10: Horizontal wind load transfer to foundation – East-West direction.

2.2.1 Calculations by hand

With the calculations for the stability of the load bearing structure with wind loads, the actual structure is simplified to a single rectangular box.

The height will be taken from the highest point in this project, which is 9m. In order to create easier calculations and subsequently create a safer design, this height is increased to 10m.
The width, in crosswind direction, is the combined length of the buildings. Finally, the depth, in wind direction, is taken as the maximum span from east to west, which is 10m.

The wind coefficients are given from the external compression coefficients. Depending on the height/width ratio, this value will be different. The actual wind force on the structure is also dependent on the peak velocity pressure at reference height ($q_p(z_e)$). This is found in table gl 12 of the Quick Reference manual 2014.

The building needs to be placed within a certain environment class to be able to read the table. The museum on the Lloydkade in Rotterdam lies within the wind zone II and even though it is located in an urban city, the structure itself is facing the wide Maas river. This results in higher wind loads than a standard urban environment. This is why the higher value class of 'rural' is chosen for the calculations.

On the assumed reference height of 10 meters, in a 'rural' area in wind area II, this value is 0,85 kN/m^2.

The added coefficients for the actual wind force can be found in the figures 11 to 13. Here a simple unity check is done to confirm that the actual deformations on the structure because of wind loads do not exceed the maximum allowed deformation.

In order to further simplify the calculations done, the highest wind load on the façade will be chosen as the governing wind load. This

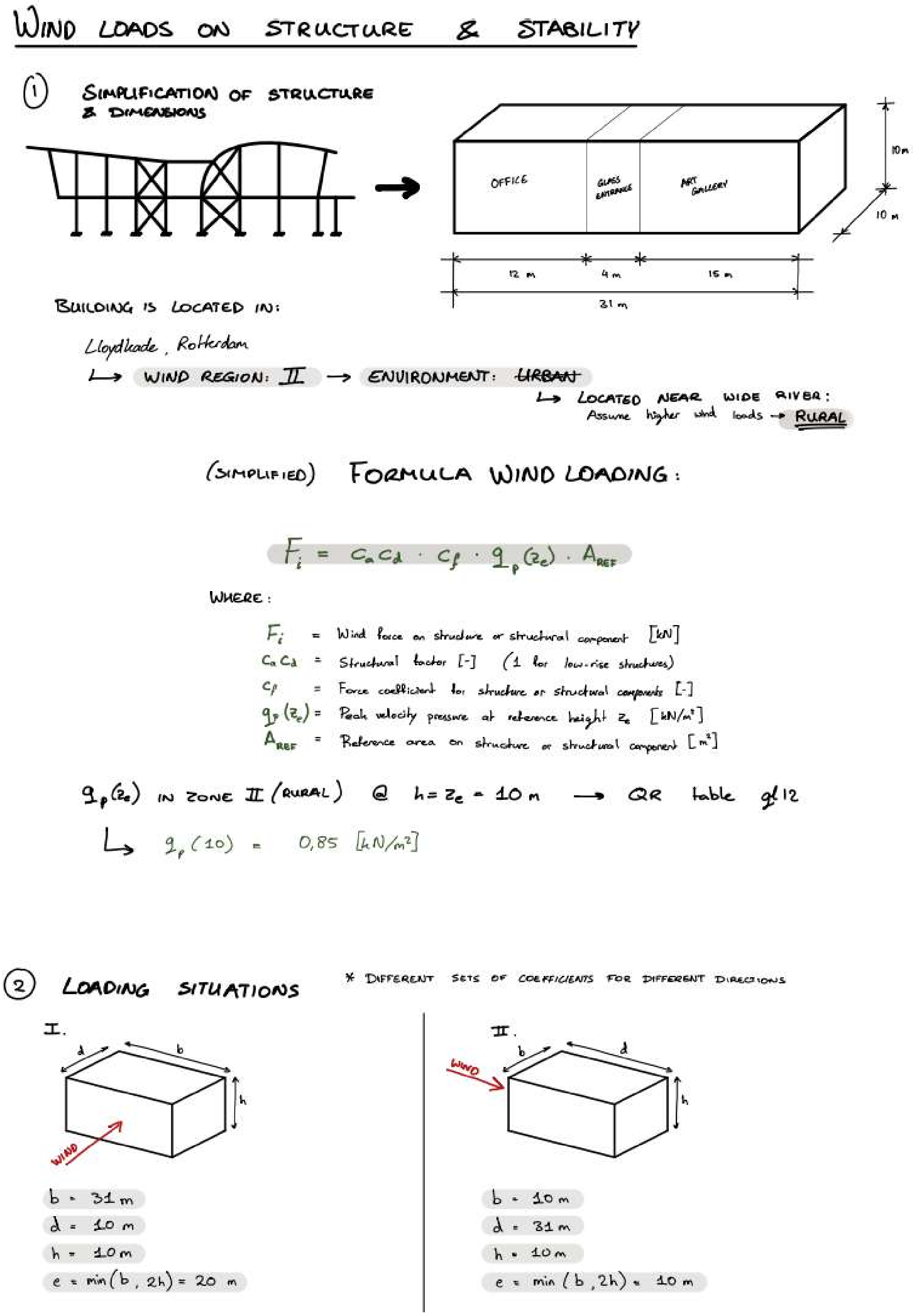


Figure 11: Wind load calculations 1/3

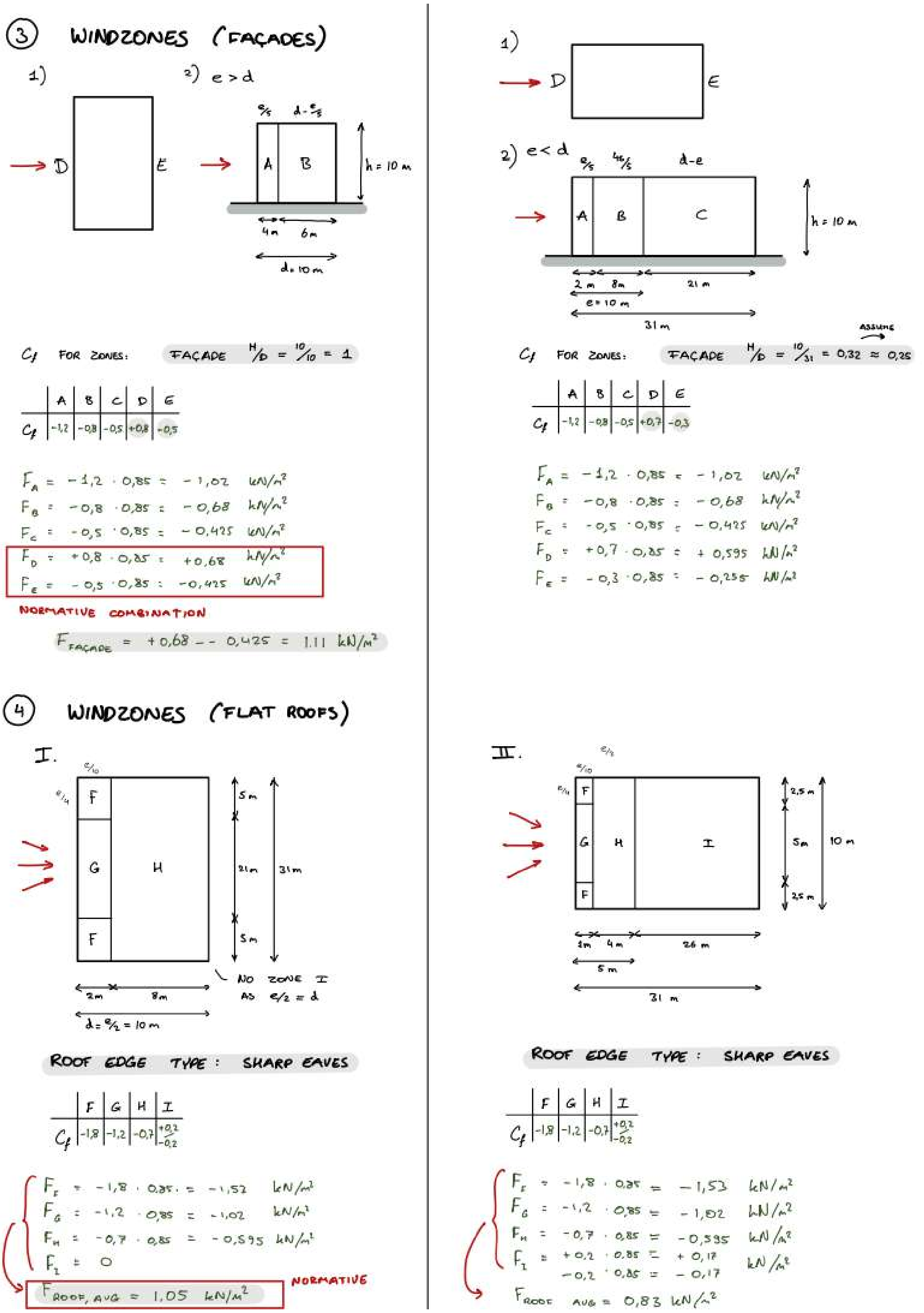


Figure 12: Wind load calculations 2/3

(5) FRICTION (DISMISSABLE)

I. MATERIAL FAÇADES PARALLEL TO WIND DIRECTION : GLASS / STEEL

"SMOOTH"

$C_{fr} = 0,01$

* DISMISSABLE IF:

$$A_{tot.\,//\,wind} < 4 \cdot A_{tot\,ext.}$$

$$(2 \cdot 10 \cdot 10) < 4 \cdot (2 \cdot 31 \cdot 10)$$

$$200\ m^2 < 2480\ m^2$$

II. MATERIAL FAÇADES PARALLEL TO WIND DIRECTION : TIMBER / MASONRY

"ROUGH"

$C_{fr} = 0,02$

$$(2 \cdot 31 \cdot 10) < 4 \cdot (2 \cdot 10 \cdot 10)$$

$$620\ m^2 < 800\ m^2$$

(6) WIND FORCES CALCULATION

* ENTIRE STRUCTURE

$$F_{wind_{tot}} = F_{normative} \cdot A_{ref}$$

$\hookrightarrow 310\ m^2$

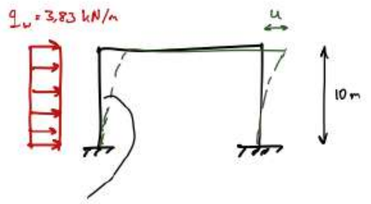
$$F_{wind_{tot}} = 1,11 \cdot 310 = 344,1\ kN$$

$$F_{wind-foundation} = 344,1 / 9 = \pm 38,23\ kN \quad \text{per stramien}$$

(7) UNITY CHECK

$$u < \frac{1}{300}\ h$$

$$u < \frac{1}{300} \cdot 10 = 0,033\ m = 33\ mm$$

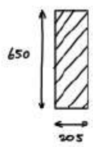


$q_w = F_{normative} \cdot \text{width} \cdot \#\text{column lines}$

$q_w = 1,11 \cdot 31 / 9 = 3,83\ kN/m$

$$u = \frac{q \cdot l^4}{8EI}$$

$$u = \frac{3,83 \cdot 10^4}{8 \cdot 5,45 \cdot 10^{10}} = 8,78 \cdot 10^{-8}\ m = 8,78 \cdot 10^{-5}\ mm$$



GL 24 h

$E = 11600\ N/mm^2$

$I_{yy} = 4,7 \cdot 10^9\ mm^4$

$\Big\}\ EI = 5,45 \cdot 10^{13}\ Nmm^2$

$5,45 \cdot 10^{10}\ kNm^2$

STRUCTURE WILL BE ABLE TO RESIST THE WIND LOADS AND TRANSFER THESE TO THE FOUNDATION

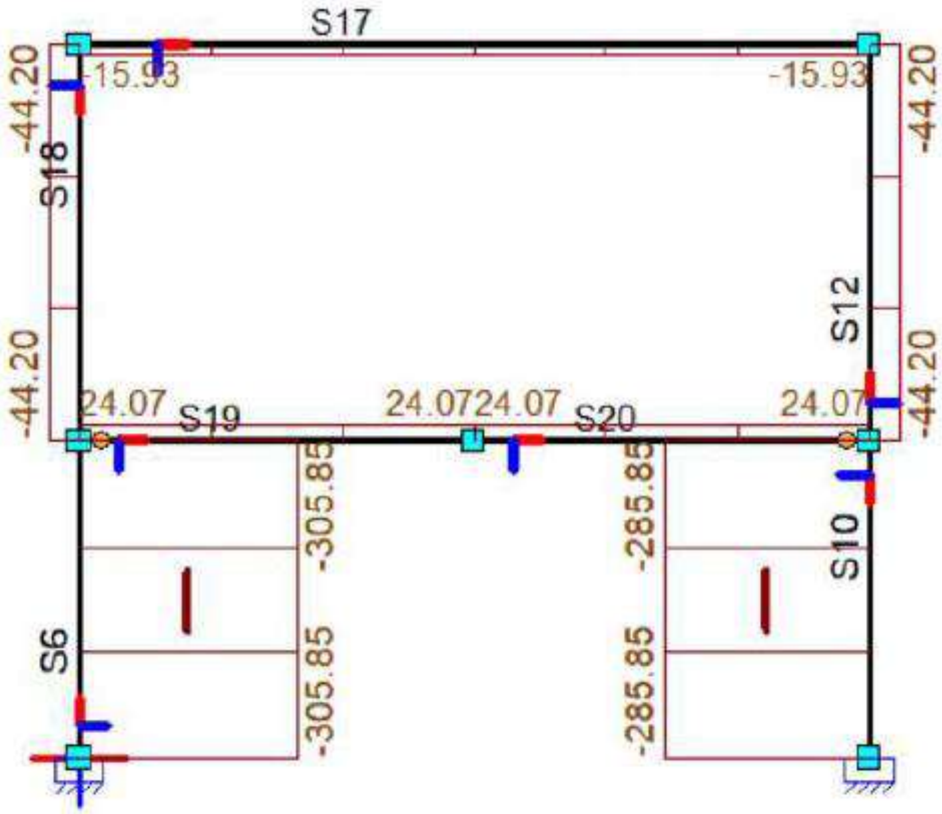
* Wind loads are relatively much smaller than the other governing vertical loads. This wont be normative in the determination of dimensions of the elements

Figure 13: Wind load calculations 3/3

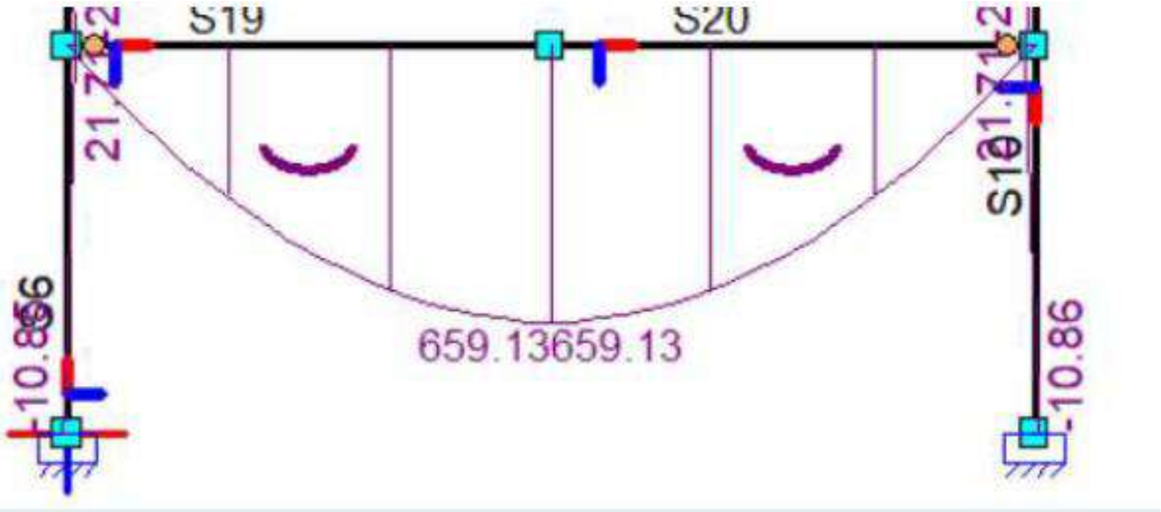
2.2.2 Calculation by Matrixframe

After the calculations were done manually/by hand, a computer program was used to check the forces in the columns and beams.

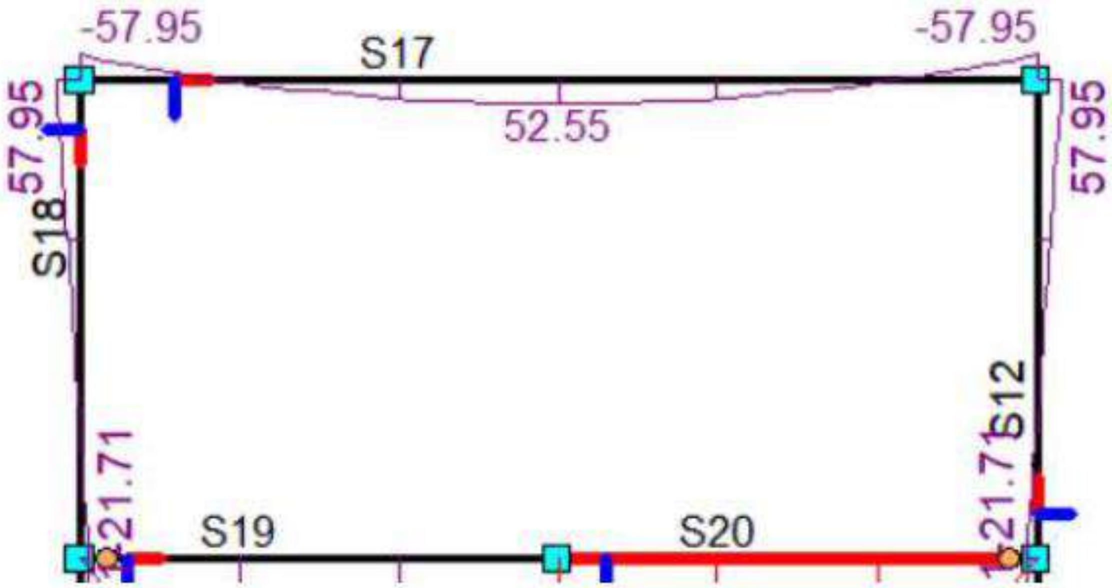
Normal forces in the column



Bending moment of the steel beam



Bending moment of the wooden beam



3. Design alternatives & determination of sizes of structural elements by rules of thumb

3.1 Design alternatives

3.1.1. Alternative 1: “Woody” - Concrete-timber lightweight

On the ground floor we will have precast concrete walls to handle the load from the first floor together with precast concrete columns in the room itself. The ground floor will be made out of concrete, essentially forming a heavy bottom layer for the structure and the first floor is designed with timber elements, to reduce the loads on the columns and foundation even more, because of its lightweight properties. Using timber on the first floor will also be beneficial as the overhangs on the office side are over three meters.

Columns:	Laminated timber
Main beams:	Laminated timber
Secondary beams:	Laminated timber
Ground floor/foundation:	In-situ concrete
Walls:	Timber
Purlins:	Laminated timber
Roof:	Laminated timber
Bracing East – West:	Steel
Bracing North– South:	Steel

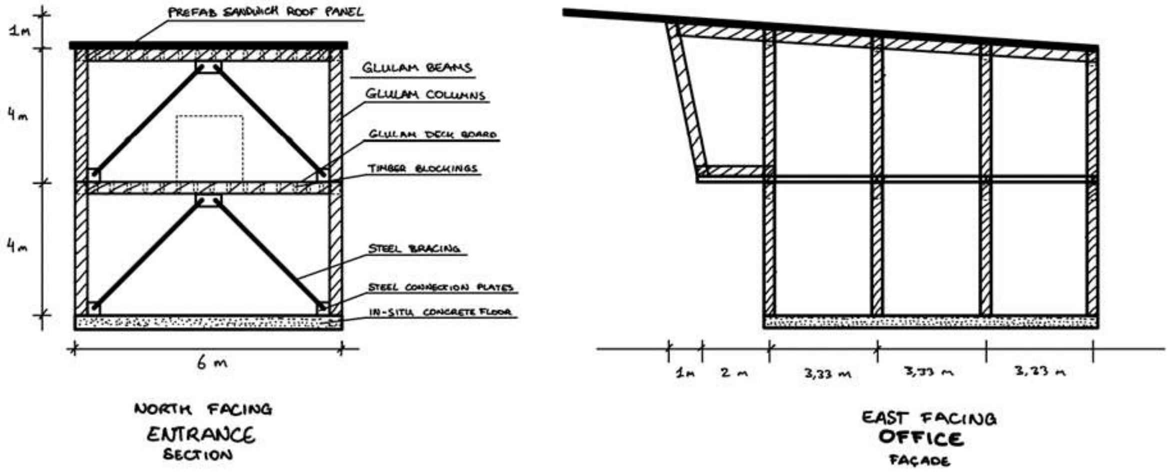


Figure 14: Cross sectional design sketches of alternative 1.

3.1.2 Alternative 2: “Bulk” - Concrete

On the ground floor we will have in-situ concrete walls to handle the load from the first floor together with columns in the room itself, which will also be made out of concrete. All floors will be made out of precast concrete. Whereas, in this case the beams will be I-beams, the rest of the building will remain the same as design alternative 1. For all the options above we will also look at removing a row of columns from the art gallery and consider adding an extra row of columns to the office.

Columns:	Precast concrete
Main beams:	Precast concrete
Secondary beams:	Precast hollow core slab
Ground floor/foundation:	in-situ concrete
Walls:	Precast concrete
Purlins:	Laminated timber
Roof:	Steel
Bracing East – West:	Steel
Bracing North – South:	Steel

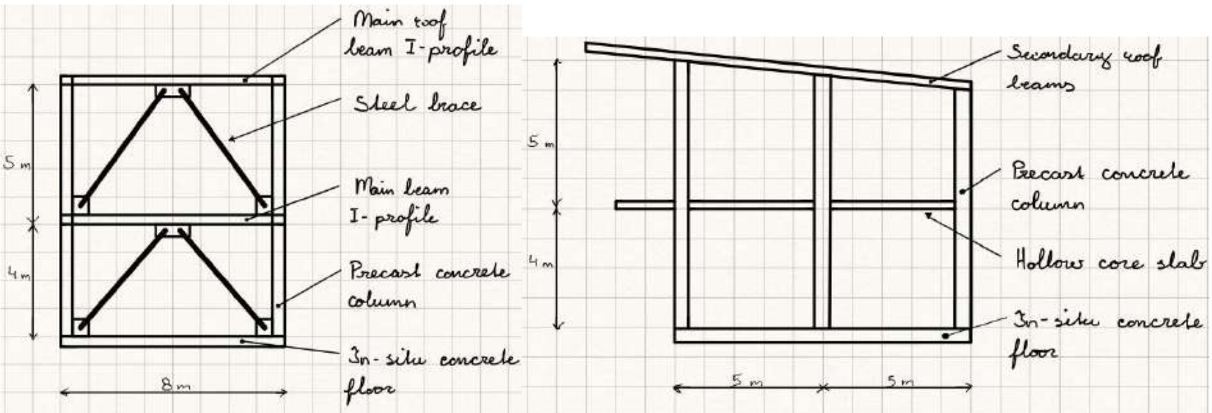


Figure 15: Cross sectional design sketches of alternative 2.

3.1.3 Alternative 3: "Two-Type" – Concrete-timber separate

The two sides of the buildings will have a different façade and a different building material. The design office side will have a structure and finish built out of mostly timber and glass. To contrast this, the art gallery side will consist of mostly precast concrete pieces and columns with corbels to create a grid with beams to support the floorplates. Both the office and art gallery will have bracing elements on three sides, with 2 points of intersection. This will provide the lateral stability to the individual parts.

Columns:	Laminated timber / Precast concrete
Main beams:	Laminated timber / Precast concrete
Secondary beams:	Timber / Precast concrete
Ground floor/foundation:	In-situ concrete
Walls:	Timber / Precast concrete
Purlins:	Timber
Roof:	Prefab panels / In-situ concrete

Bracing East – West: Steel

Bracing North-South: Steel

The sketch below only visualizes the office side of the structure. The art gallery will have the same column layout with concrete as the chosen material.

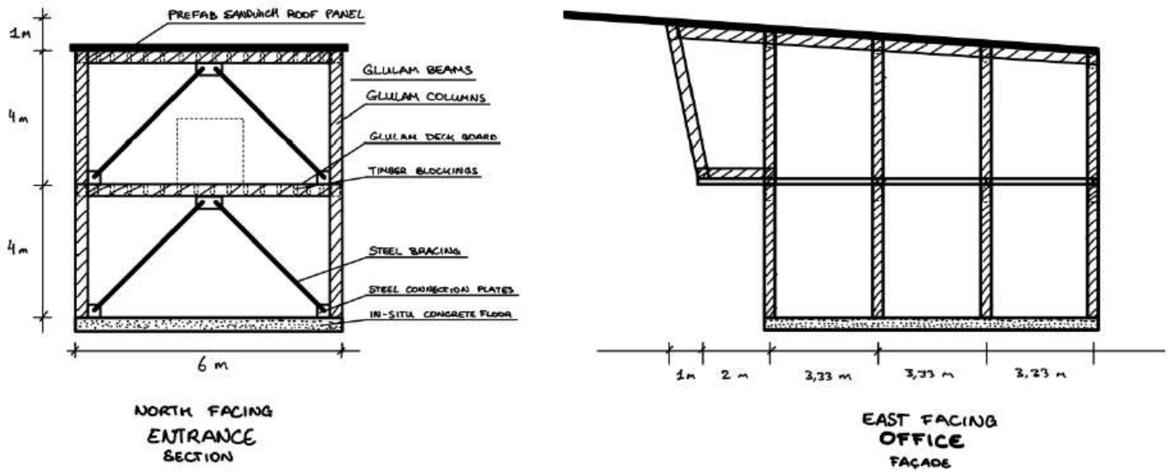


Figure 16: Cross sectional design sketches of alternative 3.

3.2 Ground floor and foundation beams.

A pavement floor will not be used in either parts of the structure, because the settlements of this floor will be disruptive when they have to be fixed. We assume the floor will have settlements because the building will be next to the River Maas and it is a Dutch soil.
Foundation depends a lot on the positioning of vertical load bearing elements of the office and art gallery (walls and columns) for both ground floor and first floor because the foundation needs to be placed under the bearing columns. The foundation should reach a sand layer in the ground, in order to find the depth of a sand layer a soil probe analysis is done. Multiple soil probe measurements have been taken from DINOloket (see Appendix A). Sand layer is recognizable with a conus resistance of > 5 MPa and friction coefficient of 0,6 – 1,0 %.

Table 1: Soil probe data for sand layer, (see Appendix A) for graphs.

BRO-ID	Date measurement	Depth sand layer [m]
CPT000000145178	26-08-2013	12 and 17
CPT000000141539	22-08-2013	13 and 19
CPT000000142242	22-08-2013	14 and 18
CPT000000141654	22-08-2013	12 and 19

Foundation should reach 19 meters below the surface.

Table 2: Load bearing capacities various prefab foundation piles. Values presented are compressive forces. Capacity for tensile forces is 20% of the capacity of compressive forces.

	220*220 mm^2	250*250 mm^2	320*320 mm^2
L=18 m	$F_{R;d}$=290 kN	$F_{R;d}$ =380 kN	$F_{R;d}$ = 620 kN
L=20 m	too slender	$F_{R;d}$ =560 kN	$F_{R;d}$ = 920 kN
L=24 m	too slender	$F_{R;d}$ =750 kN	$F_{R;d}$ = 1228 kN

3.3 Structural calculation of Design Office and Art Gallery

3.3.1 Euro code 1 (NEN-EN 1991): Actions on structures

2.3.2 Eurocode 1 (NEN-EN 1991): Actions on structures

Eurocode 1 defines all the conceivable (and inconceivable) loads that may arise during the construction and use of a structure. This standard is divided up into a number of sub-standards specifying the different loads acting on building structures:

- Part 1-1 - Volumetric weights, selfweight and imposed loads for buildings
- Part 1-2 - Fire load
- Part 1-3 - Snow load
- Part 1-4 - Wind load
- Part 1-5 - Thermal load
- Part 1-6 - Site load
- Part 1-7 - Accidental actions: Impact loads and explosions

NEN-EN 1991-1-1 gives an overview of the vertical variable actions to be taken into account for floor structures. A distinction is made between uniformly distributed loads, linear loads and point loads acting on floors, balconies and stairs in buildings and on access roads. The magnitude of the loads is, as evident from Table 2.10, dependent on the specific service function.

3.3.1.1 Live loads

The table below takes into account, variable actions working on the floors of the construction. As client requested the desired load are take into account, which are higher than the standard loads following NEN-EN 1991-1. Client missing load were increased by the percent increase given by the client for the corresponding space (Pasterkamp & van Es, 2014; Terwel, 2018).

Table 3: Live loads on building construction

Space	*Use class*	*Liveload on floors NEN-EN 1991-1*		*Client request*	
		q_k [kN/m^2]	*Q_k [kN]*	*q_k [kN/m^2]*	*Q_k [kN]*
Office	B: offices	2,5	3	5	6
Gallery	C3: people walking around	5	7	7,5	10
Storage	E2: industrial use	≥ 3	≥ 7	7	15

3.1.1.2 Snowloads (NEN-EN 1991-1-3)

Snow load and rainwater load (NEN-EN 1991-1-3)

A snow load is calculated by multiplying the thickness of the snow pack with the density of the snow. Here, the problem is that there is no such thing as the 'density of snow'. Snow does not have a specific gravity: the longer the snow lies on the ground, the greater its bulk density (NEN-EN 1991-1-3, Table E.1).

- Freshly fallen snow: $\rho \approx 1{,}0\ kN/m^3$
- Compact snow (several hours or days after snowfall): $\rho \approx 2\ kN/m^3$
- Old snow (several weeks or months after snowfall) $\rho \approx 2{,}5 - 3{,}5\ kN/m^3$
- Wet: $\rho \approx 4\ kN/m^3$

This is why snow load is defined in a different manner in the Eurocode.

For design situations, the snow load is defined as

$$s = \mu_i \cdot C_e \cdot C_t \cdot s_k$$

Where

μ_i is the snow load form factor: this depends on the geometry of the roof surface; the Eurocode provides values to be taken into account for a variety of situations; for flat roofs without any significant projections, the value is $\mu_i = 0{,}8$; in case of obstacles or rising façades connecting to the roof the resulting value of μ_i can, in extreme cases, be as high as 4.8.

C_e is the exposure ratio (set to 1.0 in the Netherlands).

C_t is the thermal coefficient (set to 1.0 in the Netherlands).

s_k is the characteristic value of the snow load on the ground ($s_k = 0{,}7\ {}^{kN}/_{m^2}$)

Hence, under 'normal' design situations and in case of a flat roof, there is a snow load of

$$s = \mu_i \cdot C_e \cdot C_t \cdot s_k = 0{,}8 \cdot 1{,}0 \cdot 1{,}0 \cdot 0{,}7 = 0{,}56\ kN/m^2$$

In other situations, the load may - whether or not locally - increase to:

$$s = \mu_i \cdot C_e \cdot C_t \cdot s_k = (4{,}0 + 0{,}8) \cdot 1{,}0 \cdot 1{,}0 \cdot 0{,}7 = 3{,}36\ kN/m^2$$

In practice we see that, particularly in the case of lightweight roof structures, the locally increased loads can lead to significant design changes; in the case of heavier structures, the relative increase in the load is more limited.

Water accumulation follows the calculations for snow load. We will however look further into water drainage system to be sure no water accumulation takes place on the roof.

3.1.1.3 Windloads (NEN-EN 1991-1-4)

$$F_w = c_s c_d \cdot c_f \cdot q_p(z_e) \cdot A_{ref}$$

Where

$c_s c_d$ is the structural factor: two facets are combined in the structural factor, i.e. the non-simultaneous occurrence of the extreme wind pressure on the total surface of the structure (c_s) and the effect of vibrations of the structure induced by turbulence (c_d); the value of $c_s c_d$ follows from:

$$c_s c_d = \max\left(\frac{1 + 2 \cdot k_p \cdot I_v(z_s) \cdot \sqrt{B^2 + R^2}}{1 + 7 \cdot I_v(z_s)}; 0{,}85\right)$$

Where:

- $k_p, I_v(z_s)$, B and R are variables that are dependent on the width and the height of the structure, the natural frequency of the structure, the distribution of the mass over the height of the structure, the damping in the structure and the terrain category[8,9]. For the sake of simplicity, for buildings whose height is less than four times the building depth in the direction of the wind and not higher than 100 metres, it can be assumed that $c_s c_d = 1$.

c_f:	1,1 – 1,3	[-]
$q_p(z_e)$:	1,5	[kN/m^2]

Wind area II. The building will be constructed in Rotterdam being a built up area, but since it is located next to the Nieuwe Maas, an open waterway, an underdeveloped area will be assumed. From the graph the desired wind load can be determined, with a construction height of around 10 meters.

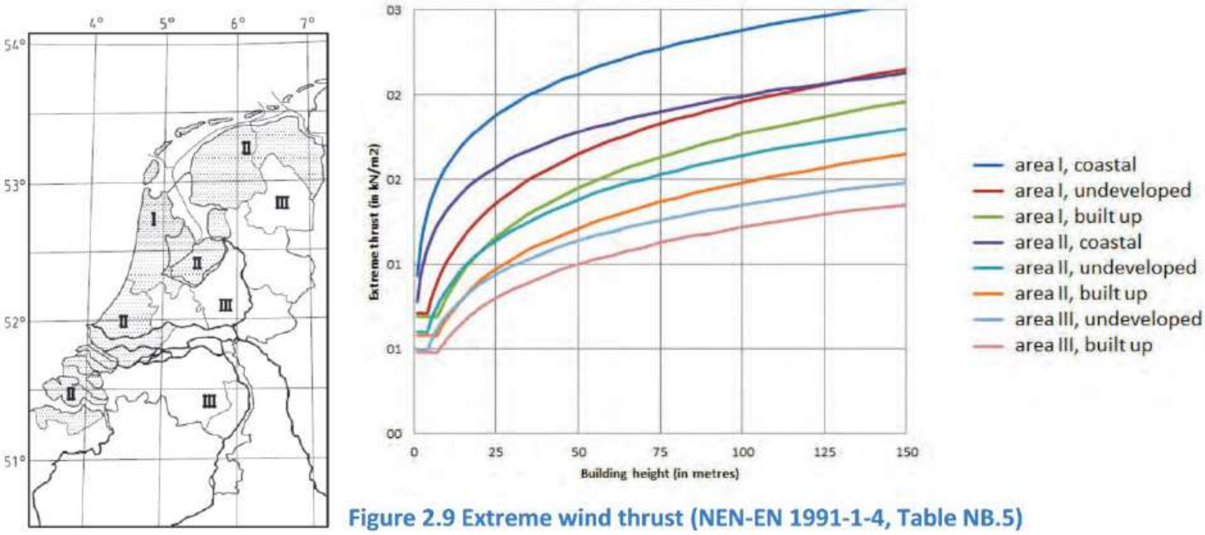


Figure 2.9 Extreme wind thrust (NEN-EN 1991-1-4, Table NB.5)

A_{ref}:	$A_{North\text{-}South}$	[m^2]
	$A_{East\text{-}West}$	[m^2]

Need to take into account that wind load in practice are almost always higher than in calculations.

3.1.1.4 Accidental actions (NEN-EN 1991-1-7)

Traffic category	F_{dx} $(in\ kN)$	F_{dy} $(in\ kN)$	d_b $(in\ m)$
Roads in urban areas	1000	500	10

F_{dx} Force in the normal direction of travel
F_{dy} Force perpendicular to the normal direction of travel
d_b Reference distance in metres

Table 2.12 Vehicle collision loads against supporting structural elements located above or adjacent to roads (equivalent static design values, NEN-EN 1991-1-7, Table NB.1-4.1)

The magnitude of the acting force (see, Figure 2.10) may be reduced as the distance d from the middle of the road to the point of collision increases (NEN-EN 1991-1-7, Section 4.3.1(1)). The factor used for this is:

$$\sqrt{(1 - d/d_b}$$

Construction will be located in the vicinity of roads in urban areas.

Distance to the construction from the most nearby road (d) is about 10 meters. Reduction factor then becomes $\sqrt{\left(1 - \frac{5}{10}\right)} = 0{,}71$

Traffic category	Reduction factor [-]	F_{dx} [kN]	F_{dy} [kN]
Roads in urban areas	0,71	1408	704

3.3.2 Eurocode 0 (NEN-EN 1990): Basis of structural design

3.3.2.1 Consequence classes

Consequence class 2(a)

Class is described as: moderate consequences in terms of loss of human life and/or substantial economic, social or environmental consequences.

And subclass 2a specifies single family residences with 4 or more floors, residential buildings, hotels and **office buildings with maximum 4 floors**, educational buildings with 1 floor, shops with maximum 2 floors, **public buildings with a floor area < 2000 m2 per floor**, industrial buildings with maximum 2 floors, car parks with maximum 2 floors.

$K_{FI} = 1{,}0\ [-]$ corresponding factor has to be multiplied by the applicable load factors γ_G & $\gamma_{Q;1}$

(source: table 2.4)

Table 4: ψ factors for the use of load combinations (Quick Reference – general loads 3)

Load type:	ψ_0	ψ_1	ψ_2
Category B: office Areas	0,5	0,5	0,3
Category C3: area without obstacles for people (museum)	0,4	0,7	0,6
Category E: storage areas	1,0	0,9	0,8
Category H: roofs	0	0	0

Snow loads of buildings	0	0,2	0
Wind loads on buildings	0	0,2	0
Temperature of buildings (non-fire)	0	0,5	0

6.10a	$\sum_{j\geq 1} \gamma_{G,j} \cdot G_{k,j} \text{ "} + \text{"} \gamma_P \cdot P \text{"} + \text{"} \gamma_{Q,1} \cdot \psi_{0,1} \cdot Q_{k,1} \text{"} + \text{"} \sum_{i>1} \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_{k,i}$
6.10b	$\sum_{j\geq 1} \xi_j \cdot \gamma_{G,j} \cdot G_{k,j} \text{ "} + \text{"} \gamma_P \cdot P \text{"} + \text{"} \gamma_{Q,1} \cdot \psi_{0,1} \cdot Q_{k,1} \text{"} + \text{"} \sum_{i>1} \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_{k,i}$
G	permanent loads
P	loads due to prestressing
Q	live loads
$\psi_{0,1}$	combination value for live loads
'+'	means 'to be combined with'
ξ	Reduction factor for unfavourable, permanent actions

Table 2.5 Load combinations for persistent or transient design situations (fundamental combinations)

	Cf	Permanent actions		Predominantly live action	Live actions occurring simultaneously with predominantly live action	
		Unfavourable	Favourable		Most important (if necessary)	Other
CC2	6.10a	1.35 $G_{kj,\ sup}$	0.9 $G_{kj,\ inf}$		1.50 $\psi_{0,1}$ $Q_{k,1}$	1.50 $\psi_{0,i}$ $Q_{k,i}$ for $i > 1$
CC2	6.10b	1.2 $G_{kj,\ sup}$	0.9 $G_{kj,\ inf}$	1.50 $Q_{k,1}$		1.50 $\psi_{0,i}$ $Q_{k,i}$ for $i > 1$

Where

$G_{kj,\ sup}$ Upper limit of the characteristic value of the permanent action

$G_{kj,\ inf}$ Lower limit of the characteristic value of the permanent action

$Q_{k,i}$ Characteristic value of the variable action i

$\psi_{0,i}$ Combination value of the variable action (to be derived from NEN-EN 1990, Table NB2-A.1.1)

Table 2.6 Load factors for determining the strength of the structure for persistent design situations (NEN-EN 1990, equations 6.10a and 6.10b)

Fire safety : Based on the building decree 2012 a fire resistance of > 90 minutes is required. (source: table 2.2)

3.4 Determination of sizes of structural elements by rules of thumb

3.4.1 Design Alternative 1: "Woody"

Element	Material	Type	Profile	L x h x b [mm]
Roof	Steel	cold formed corrugated sheet	h = 106 mm t = 1,00 mm	6000 x 106 x 2000
Purlins	Laminated timber	GL24h	350 mm x 85 mm	6000 x 350 x 85
Roof beams	Laminated timber	GL24h	600 mm x 85 mm	10.000 x 600 x 85
First floor	Precast concrete	Hollow slab floor	Dycore type T255	6000 x 255 x 1197
Floor beams	Laminated timber	GL24h	600 mm x 85 mm	10.000 x 600 x 85
Walls	Laminated timber	GL24h	d = 200 mm	3750 x 5000 x 200
Columns	Laminated timber	GL24h	650 mm x 205 mm	9300 x 650 x 205
Bracing East – West	Steel	S460 – double bracing	M16	5.200 – 6.000
Bracing North – South	Steel	S460 – double bracing	M16	5.200 – 6.000

Roof:	Steel	cold formed corrugated sheet
Length of cold formed corrugated sheet	$l = 4$	[m]
Height range	$h = \frac{1}{40} * l\,; \frac{1}{70} * l = \frac{1}{40} * 4\,; \frac{1}{70} * 4$	
	$h = 0{,}4\,; 0{,}06$	[m]

Profile:

h = 106 mm

L = 2000 mm

B = 6000 mm

t = 1,00 mm

shape	dimensions		mass	section properties	
	h	t		I_y x 10^4	$W_{y;el}$ x 10^3
	[mm]	[mm]	[N/m²]	[mm⁴]	[mm³]
	40	0.88	94	30	9.1
		1.00	107	33	10.4
	70	0.88	107	96	22.7
		1.00	122	110	26.6
	96	0.88	121	195	21.7
		1.00	138	223	24.8
	106	0.88	115	213	26.9
		1.00	130	243	31.8
	120	0.88	136	320	43.0
		1.00	154	365	55.0

Note: the sections have variable width. Section moduli and moments of inertia are given per meter width.

Quick reference steel design 4: roofing plates

Quick reference steel basis 16: Corrugated sheet-sections

Purlins: Laminated timber Glulam beams

Length of purlins $l = 5\,;6$ [m]

Height range $h = \frac{1}{17} * l = \frac{1}{17} * 6$ for s < 5 m

$h = 0{,}35$ [m]

Quick reference timber design 2: straight beam (possibly with precamber)

Glulam 350 mm x 85 mm (h x b) profile

Profile properties:

Area (face) $A = 29{,}7 * 10^3$ [mm²]

Density gl24h $\rho_{mean} = 420$ [kg/m³]

Weight $G = A * \rho_{mean} = 29{,}7 * 10^3 * 10^{-6} * 420 * 10 = 0{,}125$ [kN/m]

Primary roof beams: Laminated timber Glulam beams

Length of primary roof beams $l = 6; 10$ [m]

Height range $h = \frac{1}{17} * l = \frac{1}{17} * 6;\ \frac{1}{17} * 10$ for s < 5 m

$h = 0{,}35 - 0{,}59$ [m]

Quick reference timber design 2: straight beam (possibly with precamber)

Glulam 600 mm x 85 mm (h x b) profile

Profile properties:

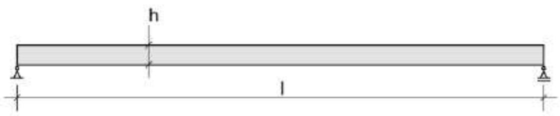
Area (face) $A = 51 * 10^3$ [mm²]

Density gl24h $\rho_{mean} = 420$ [kg/m³]

Weight $G = A * \rho_{mean} = 51 * 10^3 * 10^{-6} * 420 * 10 = 0{,}214$ [kN/m]

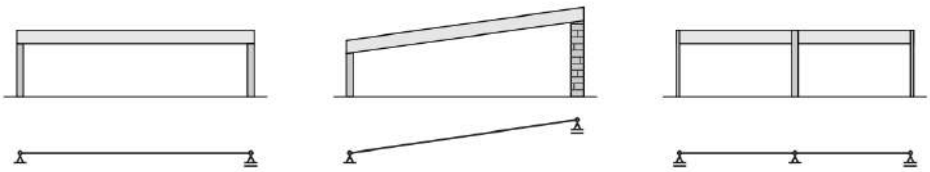
h	width 55 mm			width 85 mm			width 110 mm		
	A 10^3 [mm²]	W_y 10^6 [mm³]	I_y 10^6 [mm⁴]	A 10^3 [mm²]	W_y 10^6 [mm³]	I_y 10^6 [mm⁴]	A 10^3 [mm²]	W_y 10^6 [mm³]	I_y 10^6 [mm⁴]
200	11,0	0,36	36,6	17,0	0,56	56,6	22,0	0,73	73,3
250	13,7	0,57	71,6	21,2	0,88	110,6	27,5	1,14	143,2
300	16,5	0,82	123,7	25,5	1,27	191,2	33,0	1,65	247,5
350	19,2	1,12	196,5	29,7	1,73	303,6	38,5	2,24	393,0
400	22,0	1,46	293,3	34,0	2,26	453,3	44,0	2,93	586,6
450	24,7	1,85	417,6	38,2	2,86	645,4	49,5	3,71	835,3
500	27,5	2,29	572,9	42,5	3,54	885,4	55,0	4,58	1145,8
550	30,2	2,77	762,5	46,7	4,28	1178,4	60,5	5,54	1525,1
600				51,0	5,10	1530,0	66,0	6,60	1980,0
650				55,2	5,98	1945,2	71,5	7,74	2517,3

>> straight beam (possibly with precamber)

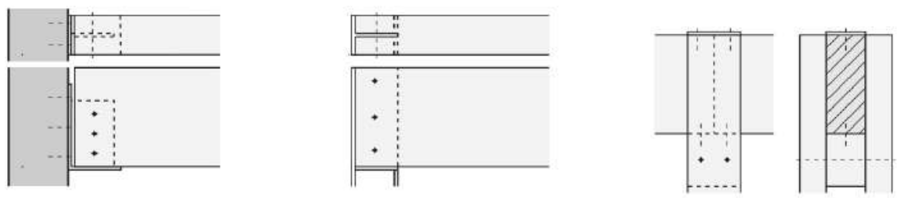


solid timber beams
l = 2 - 6 m
h = l/15 - l/20
glulam beams
l = 6-30 m
s < 5 m h = l/17
s = 5 - 8 m h = l/16

possible shapes



details



support on continuous column

support on concrete wall

support on double timber column (by means of support block)

Floor: Precast concrete	hollow core slab		
Length of hollow core slab	$l = 5\,;6$		[m]
Height range	$h = \frac{1}{25} * l\,;\frac{1}{35} * l = \frac{1}{25} * 6\,;\frac{1}{35} * 6$		

$h = 0{,}24\,;0{,}17$ [m]

Dycore hollow core slab profile type T: 255 mm height

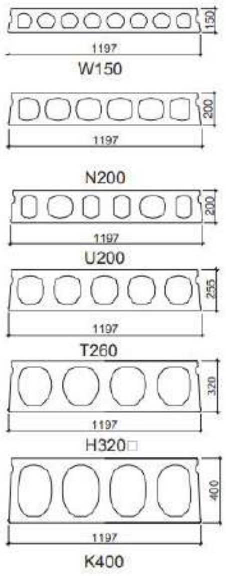
Quick reference concrete design 14: product specifications Dycore hollow core slabs

Leverancier: Dycore

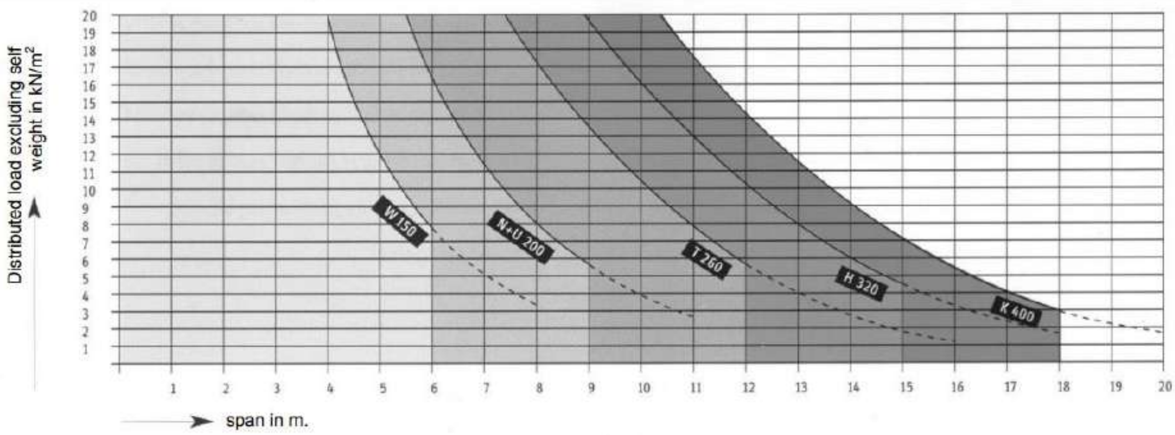
product specifications Dycore hollow core slabs

fire resistance 60 - 90 minutes
density 2400 kg/m^3
prestressing steel FeP 1860

type	height h [mm]	weight G [kN/m^2]	joints - [l/m]	cross section A [mm^2]	I_{zz} x 10^6 [mm^4]	M_{cr} [kNm]	filler slabs min. width [m]	max. length [m]
W	150	2,4	4,4	0,11	297	83	6	5,0
N	200	2,7	5,7	0,13	643	119	9	7,2
U	200	3,1	5,7	0,15	666	164	9	7,2
T	255	3,7	7,2	0,17	1364	290	12	9,3
H	320	4,3	8,7	0,20	2517	436	15	11,5
K	400	5,0	10,9	0,23	4625	593	18	14,4



spans



the graph indicates the maximum distributed load (excluding self weight) for different spans for slabs on two supports

Primary beams first floor:	Laminated timber	Glulam beams
Length of primary floor beams [m]	$l = 6; 10$	
Height range	$h = \frac{1}{17} * l = \frac{1}{17} * 6; \frac{1}{17} * 10$	for s < 5 m

$h = 0,35 - 0,59$ [m]

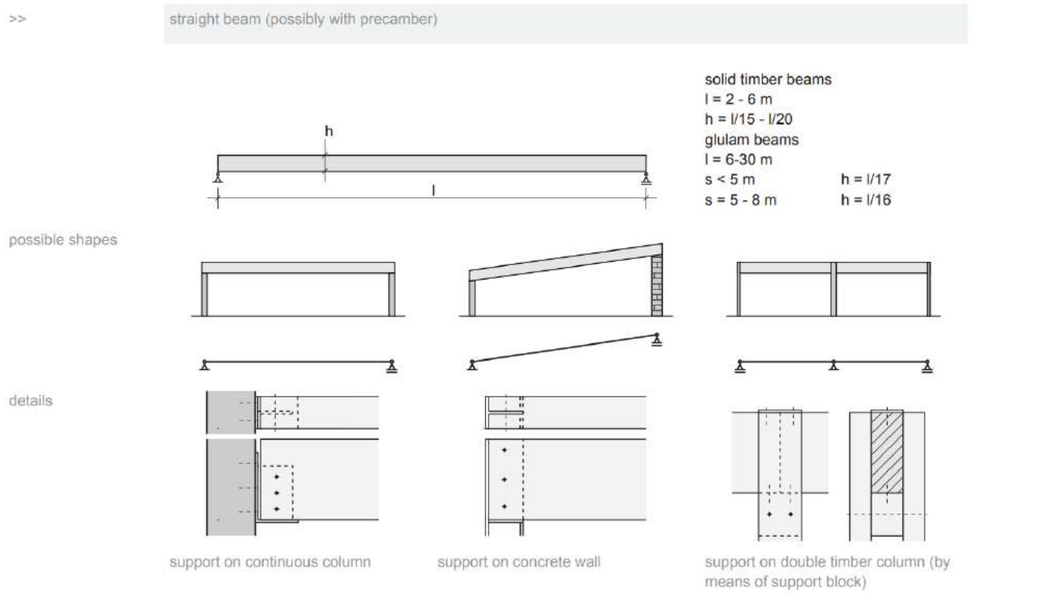
Quick reference timber design 2: straight beam (possibly with precamber)

Glulam 600 mm x 85 mm (h x b) profile

Profile properties:

Area (face)	$A = 51 * 10^3$	[mm^2]
Density gl24h	$\rho_{mean} = 420$	[kg/m^3]
Weight	$G = A * \rho_{mean} = 51 * 10^3 * 10^{-6} * 420 * 10 = 0{,}214$ [kN/m]	

	width 55 mm			width 85 mm			width 110 mm		
h	A 10^3 [mm^2]	W_y 10^6 [mm^3]	I_y 10^6 [mm^4]	A 10^3 [mm^2]	W_y 10^6 [mm^3]	I_y 10^6 [mm^4]	A 10^3 [mm^2]	W_y 10^6 [mm^3]	I_y 10^6 [mm^4]
200	11,0	0,36	36,6	17,0	0,56	56,6	22,0	0,73	73,3
250	13,7	0,57	71,6	21,2	0,88	110,6	27,5	1,14	143,2
300	16,5	0,82	123,7	25,5	1,27	191,2	33,0	1,65	247,5
350	19,2	1,12	196,5	29,7	1,73	303,6	38,5	2,24	393,0
400	22,0	1,46	293,3	34,0	2,26	453,3	44,0	2,93	586,6
450	24,7	1,85	417,6	38,2	2,86	645,4	49,5	3,71	835,3
500	27,5	2,29	572,9	42,5	3,54	885,4	55,0	4,58	1145,8
550	30,2	2,77	762,5	46,7	4,28	1178,4	60,5	5,54	1525,1
600				51,0	5,10	1530,0	66,0	6,60	1980,0
650				55,2	5,98	1945,2	71,5	7,74	2517,3



Walls:		
Primary beams first floor:	Laminated timber	Glulam beams
Length of walls	$l = 3750\ [mm]$	
Height range	$h = 5000\ [mm]$	
Thickness	$d = 200\ [mm]$	

.

https://www.stommel-haus.co.uk/product-service/timber-wall-construction/

columns: Laminated timber	Glulam beams	
Gl24h profile 650 mm x 205 mm		
Length		$l = 9300\ [mm]$

Quick reference timber design 2: straight beam (possibly with precamber)

Ground floor/foundation:	in-situ concrete	
Length of in-situ concrete	$l = 10$	[m]
Height range	$h = \frac{1}{25} * l\,; \frac{1}{35} * l = \frac{1}{25} * 10\,; \frac{1}{35} * 10$	
	$h = 0{,}4\,; 0{,}29$	[m]

In-situ concrete ground floor with a thickness of 0,35 meter.

Bracing East – West and North – South:

Macallory s460 M16

Table 1 - Material Properties

Product name	Material	Minimum Yield Stress N/mm²	Min. Breaking Stress N/mm²	Min. Elongation %	Min. Charpy Impact Value J@ -20°C	Youngs Modulus kN/mm²
Macalloy 460	Carbon Steel	460	610	19	27	205
Macalloy S460	Stainless Steel	460	610	15	27	205
Macalloy 520	Carbon Steel	520	660	19	27	205
Macalloy S520	Stainless Steel	520	660	15	27	205

Table 2 - Tendon Capacities for Carbon and Stainless Macalloy 460

Thread	Units	M10	M12	M16	M20	M24	M30	M36	M42	M48	M56	M64	M76	M85	M90	M100
Nominal Bar Dia.	mm	10	11	15	19	22	28	34	39	45	52	60	72	82	87	97
Min. Yield Load	kN	25	36	69	108	156	249	364	501	660	912	1204	1756	2239	2533	3172
Min. Break Load	kN	33	48	91	143	207	330	483	665	875	1209	1596	2329	2969	3358	4206
Design Resistance to EC3	kN	24	35	66	103	149	238	348	479	630	870	1149	1677	2138	2418	3029
Nominal Bar Weight	(kg/m)	0.5	0.75	1.4	2.2	3.0	4.8	7.1	9.4	12.5	16.7	22.2	32	41.5	46.7	58

Table 3 - Tendon Capacities for Carbon and Stainless Macalloy 520

Thread	Units	M10	M12	M16	M20	M24	M30	M36	M42	M48	M56	M64	M76	M85	M90	M100
Nominal Bar Dia.	mm	10	11	15	19	22	28	34	39	45	52	60	72	82	87	97
Min. Yield Load	kN	28	41	77	122	176	284	411	566	746	1030	1360	1985	2531	2862	3585
Min. Break Load	kN	35	52	98	155	223	360	522	719	946	1308	1727	2520	3212	3633	4551
Design Resistance to EC3	kN	26	38	71	112	161	257	376	518	682	942	1244	1814	2313	2616	3277
Nominal Bar Weight	(kg/m)	0.5	0.75	1.4	2.2	3	4.8	7.1	9.4	12.5	16.7	22.2	32	41.5	46.7	58

M85 to M100 in stainless is not covered by ETA but is available by special request.

Table 4 - Maximum Length of Individual Bar Lengths

Diameter	Stainless Steel	Carbon	Galvanised
M10 - M16	6.0m	11.95m	6.0m
M20 - M30	6.0m	11.95m	8.0m
M36 - M100	6.0m	11.95m	11.95m

Longer lengths can be supplied as made to order if required

Figuur 32: Tabellen van de leverancier Macalloy.

3.4.2 Design Alternative 2: "Bulk"

Element	Material	Type	Profile	h [mm]
Columns	Precast concrete	C30/37	200 x 200	9300
Main beams	Precast concrete	TT-plate	IC32,5/70	700
Floor	Precast concrete	Hollow core slab	Type T	255
Ground floor	In-situ concrete	C30/37	-	350
Walls	Precast concrete	C30/37	200 mm	4000-5000
Main roof beams	Laminated timber	GL24h	600 x 85	10000
Purlins	Laminated timber	GL24h	350 x 85	350
Roof	Steel	Cold formed plates	-	200
Bracing East – West	Steel	S460 – double bracing	M16	5.200 – 6.000
Bracing North – South	Steel	S460 – double bracing	M16	5.200 – 6.000

Columns: The columns will be made of Precast Concrete.

A concrete type of C30/37 with column profile 200 mm x 200 mm F60 will be used

Main beams: Precast concrete I-beam

Length of main beam $l = 6\,;10$ [m]

Height range $h = \frac{1}{10} * l\,; \frac{1}{15} * l = \frac{1}{10} * 10\,; \frac{1}{15} * 10$

$h = 1\,; 0{,}66$ [m]

Profile I – beam: IC32,5/70

Profile properties:

Weight $G = \frac{W_{24m} - W_{8m}}{L_{24m} - L_{8m}} * L * g = \frac{9002 - 3213}{24 - 8} * 10 * 10$

$G = 36{,}18$ [kN/10 m beam]

$G = 3{,}618$ [kN/m]

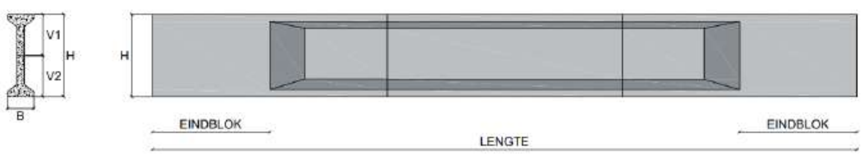
Quick reference concrete basis 8: prefab prestressed beams (non-residential buildings)

Leverancier: https://prefabsystems.be/nl/producten/balken-in-voorgespannen-beton/

I-LIGGERS
IN VOORGESPANNEN BETON

Liggers met I-vormige sectie en constante hoogte, IC-liggers of IK-liggers, worden gebruikt om over langere afstand hogere lijnlasten op te vangen (IC-liggers) van bijvoorbeeld vloerelementen. Ze kunnen ook toegepast worden als moerbalk (IK-liggers) en uitgevoerd worden met of zonder consoles. Door het toepassen van IK-liggers (moerbalken) kan het aantal vrijstaande kolommen sterk gereduceerd worden.

IC-LIGGERS – LIGGERS MET EEN CONSTANTE HOOGTE



KARAKTERISTIEKEN

Ligger IC(b)/(h)	A ($x10^2 mm^2$)	I ($x10^4 mm^4$)	v_1 (mm)	v_2 (mm)	minimum L_{min} (m)*	minimum EG_{min} (kg)	maximum L_{max} (m)	maximum EG_{max} (kg)
IC30/55	1050	368648	280	270	8	2371	20	5585
IC32,5/55	1187,5	403345	280	270	8	2652	20	6299
IC30/70	1240	722909	350	350	8	2856	24	7931
IC32,5/70	1415	794368	350	350	8	3213	24	9002
IC35/80	1525	1211707	408	392	8	3621	28	11399

Floor:	Precast concrete	hollow core slab	
Length of hollow core slab		$l = 5\ ;6$	[m]
Height range		$h = \frac{1}{25} * l\ ; \frac{1}{35} * l = \frac{1}{25} * 6\ ; \frac{1}{35} * 6$	
		$h = 0{,}24\ ; 0{,}17$ [m]	

Dycore hollow core slab profile type T: 255 mm height

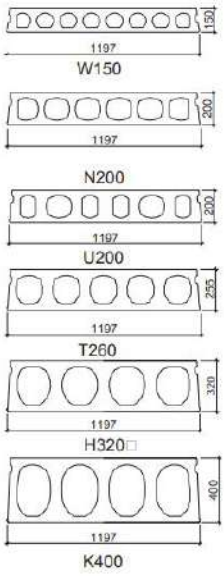
Quick reference concrete design 14: product specifications Dycore hollow core slabs

Leverancier: Dycore

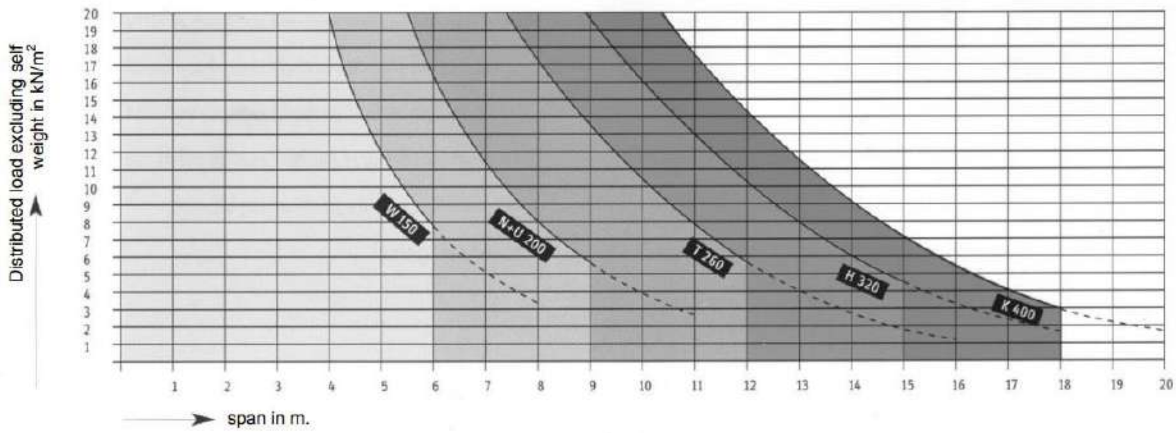
product specifications Dycore hollow core slabs

fire resistance	60 - 90 minutes
density	2400 kg/m³
prestressing steel	FeP 1860

type	height h [mm]	weight G [kN/m²]	joints - [l/m]	cross section A [mm²]	 I_{zz} x 10^6 [mm⁴]	 M_{cr} [kNm]	filler slabs min. width [m]	max. length [m]
W	150	2,4	4,4	0,11	297	83	6	5,0
N	200	2,7	5,7	0,13	643	119	9	7,2
U	200	3,1	5,7	0,15	666	164	9	7,2
T	255	3,7	7,2	0,17	1364	290	12	9,3
H	320	4,3	8,7	0,20	2517	436	15	11,5
K	400	5,0	10,9	0,23	4625	593	18	14,4



spans



the graph indicates the maximum distributed load (excluding self weight) for different spans for slabs on two supports

Ground floor/foundation:	in-situ concrete	
Length of in-situ concrete	$l = 10$	[m]
Height range	$h = \frac{1}{25} * l\,; \frac{1}{35} * l = \frac{1}{25} * 10\,; \frac{1}{35} * 10$	
	$h = 0{,}4\,; 0{,}29$	[m]
In-situ concrete ground floor with a thickness of 0,35 meter.		

Walls:	Precast concrete
4000 mm x 5000 mm x 200 mm	
(BuildingSupply, n.d.)	

Primary roof beams: Laminated timber	Glulam beams	
Length of primary roof beams	$l = 6; 10$	[m]
Height range	$h = \frac{1}{17} * l = \frac{1}{17} * 6;\ \frac{1}{17} * 10$ for s < 5 m	
	$h = 0{,}35 - 0{,}59$	[m]

Quick reference timber design 2: straight beam (possibly with precamber)

Glulam 600 mm x 85 mm (h x b) profile

Profile properties:

Area (face)	$A = 51 * 10^3$	[mm^2]
Density gl24h	$\rho_{mean} = 420$	[kg/m^3]
Weight	$G = A * \rho_{mean} = 51 * 10^3 * 10^{-6} * 420 * 10$	
	$G = 0{,}214$	[kN/m]

Purlins:	Laminated timber	Glulam beams	
Length of purlins	$l = 5\,;6$		[m]
Height range	$h = \frac{1}{17} * l = \frac{1}{17} * 6$	for s < 5 m	
	$h = 0{,}35$		[m]

Quick reference timber design 2: straight beam (possibly with precamber)

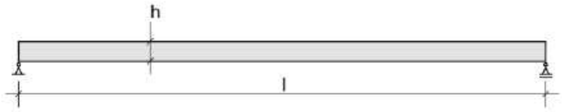
Glulam 350 mm x 85 mm (h x b) profile

Profile properties:

Area (face)	$A = 29{,}7 * 10^3$	[mm²]
Density gl24h	$\rho_{mean} = 420$	[kg/m³]
Weight	$G = A * \rho_{mean} = 29{,}7 * 10^3 * 10^{-6} * 420 * 10$	
	$G = 0{,}125$	[kN/m]

h	width 55 mm A 10^3 [mm²]	W_y 10^6 [mm³]	I_y 10^6 [mm⁴]	width 85 mm A 10^3 [mm²]	W_y 10^6 [mm³]	I_y 10^6 [mm⁴]	width 110 mm A 10^3 [mm²]	W_y 10^6 [mm³]	I_y 10^6 [mm⁴]
200	11,0	0,36	36,6	17,0	0,56	56,6	22,0	0,73	73,3
250	13,7	0,57	71,6	21,2	0,88	110,6	27,5	1,14	143,2
300	16,5	0,82	123,7	25,5	1,27	191,2	33,0	1,65	247,5
350	19,2	1,12	196,5	29,7	1,73	303,6	38,5	2,24	393,0
400	22,0	1,46	293,3	34,0	2,26	453,3	44,0	2,93	586,6
450	24,7	1,85	417,6	38,2	2,86	645,4	49,5	3,71	835,3
500	27,5	2,29	572,9	42,5	3,54	885,4	55,0	4,58	1145,8
550	30,2	2,77	762,5	46,7	4,28	1178,4	60,5	5,54	1525,1
600				51,0	5,10	1530,0	66,0	6,60	1980,0
650				55,2	5,98	1945,2	71,5	7,74	2517,3

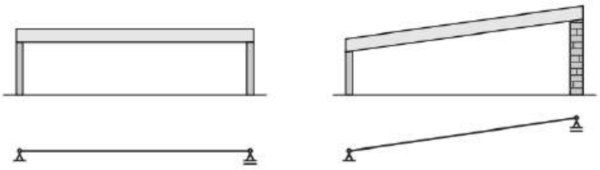
>> straight beam (possibly with precamber)



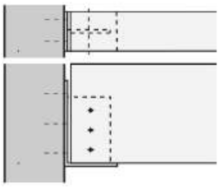
solid timber beams
l = 2 - 6 m
h = l/15 - l/20
glulam beams
l = 6-30 m

s < 5 m	h = l/17
s = 5 - 8 m	h = l/16

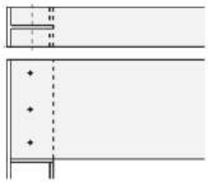
possible shapes



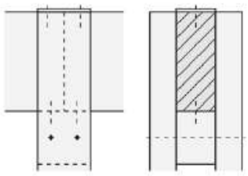
details



support on continuous column



support on concrete wall



support on double timber column (by means of support block)

Roof:	Steel	Cold formed plates
Length of hollow core slab	$l = 4$	[m]
Height range	$h = \frac{1}{40} * l\, ; \frac{1}{70} * l = \frac{1}{40} * 4\, ; \frac{1}{70} * 4$	
	$h = 0{,}4\, ; 0{,}06$	[m]

Quick reference steel design 4: roofing plates

Bracing East – West and South – North

Macallory s460 M16

Table 1 - Material Properties

Product name	Material	Minimum Yield Stress N/mm²	Min. Breaking Stress N/mm²	Min. Elongation %	Min. Charpy Impact Value J@ -20°C	Youngs Modulus kN/mm²
Macalloy 460	Carbon Steel	460	610	19	27	205
Macalloy S460	Stainless Steel	460	610	15	27	205
Macalloy 520	Carbon Steel	520	660	19	27	205
Macalloy S520	Stainless Steel	520	660	15	27	205

Table 2 - Tendon Capacities for Carbon and Stainless Macalloy 460

Thread	Units	M10	M12	M16	M20	M24	M30	M36	M42	M48	M56	M64	M76	M85	M90	M100
Nominal Bar Dia.	mm	10	11	15	19	22	28	34	39	45	52	60	72	82	87	97
Min. Yield Load	kN	25	36	69	108	156	249	364	501	660	912	1204	1756	2239	2533	3172
Min. Break Load	kN	33	48	91	143	207	330	483	665	875	1209	1596	2329	2969	3358	4206
Design Resistance to EC3	kN	24	35	66	103	149	238	348	479	630	870	1149	1677	2138	2418	3029
Nominal Bar Weight	(kg/m)	0.5	0.75	1.4	2.2	3.0	4.8	7.1	9.4	12.5	16.7	22.2	32	41.5	46.7	58

Table 3 - Tendon Capacities for Carbon and Stainless Macalloy 520

Thread	Units	M10	M12	M16	M20	M24	M30	M36	M42	M48	M56	M64	M76	M85	M90	M100
Nominal Bar Dia.	mm	10	11	15	19	22	28	34	39	45	52	60	72	82	87	97
Min. Yield Load	kN	28	41	77	122	176	284	411	566	746	1030	1360	1985	2531	2862	3585
Min. Break Load	kN	35	52	98	155	223	360	522	719	946	1308	1727	2520	3212	3633	4551
Design Resistance to EC3	kN	26	38	71	112	161	257	376	518	682	942	1244	1814	2313	2616	3277
Nominal Bar Weight	(kg/m)	0.5	0.75	1.4	2.2	3	4.8	7.1	9.4	12.5	16.7	22.2	32	41.5	46.7	58

M85 to M100 in stainless is not covered by ETA but is available by special request.

Table 4 - Maximum Length of Individual Bar Lengths

Diameter	Stainless Steel	Carbon	Galvanised
M10 - M16	6.0m	11.95m	6.0m
M20 - M30	6.0m	11.95m	8.0m
M36 - M100	6.0m	11.95m	11.95m

Longer lengths can be supplied as made to order if required

Figuur 32: Tabellen van de leverancier Macalloy.

3.4.3 Design Alternative 3: “Two-Type”

Element	Material	Type	Profile [mm]	h [mm]	L [m]
Columns ART	Precast concrete	C50/60	450 x 450	4000-9000	-
Main beams ART	Precast concrete	Rectangular C50/60 (prestressed)	300 x 600	600	6-10
Floor ART	Precast concrete	Hollow core slab	1197 x 255	150	4-5
Columns OFFICE	Laminated timber	GL24h	650 x 205	9000	-
Main beams OFFICE	Laminated timber	GL24h	600 x 85	600	6-10
Floor OFFICE	Laminated timber	GL24h	2500 x 300	300	5-6,5
Ground floor	In-situ concrete	C30/37	-	350	-
Walls ART	Precast concrete	C30/37	200	4000	3-4
Walls OFFICE	Laminated timber	GL24h	100x400	400	4-5
Purlins OFFICE	Laminated timber	GL24h	350 x 85	350	-
Roof ART	Precast arched concrete	C30/37	-	150	-
Roof OFFICE	Cold formed corrugated sheet	t = 1,00	-	106	-
Bracing East – West	Steel	S460 – double bracing	M16	-	5,2 - 6
Bracing North – South	Steel	S460 – double bracing	M16	-	5,2 - 6

The same rule of thumbs have been used as for the calculations of design 1 and design 2.

4. Structural Verification of Sizes

4.1 Design Alternative 1: “Woody”

Element	*Material*	*Profile*	*New profile*
Roof	Cold formed corrugated sheet	h = 106 mm t = 1,00 mm	- -
Purlin	Timber GL24h	350 mm x 85 mm	-
Main Roof Beam	Timber GL24h	600 mm x 85 mm	600 mm x 185 mm
Hollow slab floor	Prefab concrete	Dycore type T255	Dycore type W150
Main beam first floor	Timber GL24h	600 mm x 85 mm	600 mm x 205 mm
Walls	Timber GL24h	b = 200 mm	-
Columns	Timber GL24h	650 mm x 205 mm	-

Roof

Cold formed corrugated sheet profile:

Material: steel S235

$h = 106\ [mm]$

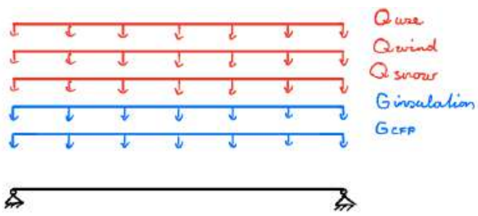
$L = 2000\ [mm]$

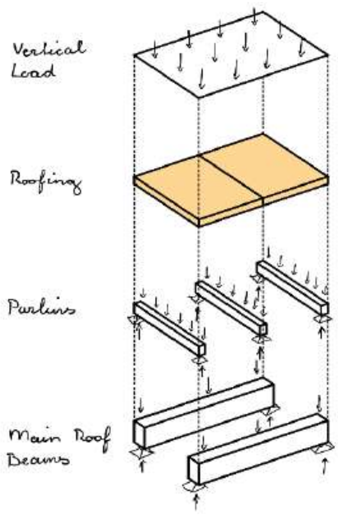
$b = 6000\ [mm]$

$t = 1{,}00\ [mm]$

$W = 31{,}8 * 10^3\ [mm^3]$

$f_{y;d} = 235\ \left[\frac{N}{mm^2}\right]$





Weight table for the roof.

Element	Detail	Permanent load	Variable load	Source:
Cold formed corrugated sheet		0,130 [kN/m^2]	-	Quick reference – sb 16
Insulation		0,20 [kN/m^2]	-	Quick reference – gl 4
Use		-	1,00 [kN/m^2]	Quick reference – gl 5
Wind		-	0,46 [kN/m^2]	Chapter 3
Snow		-	0,56 [kN/m^2]	Chapter 3
Total		**0,33 [kN/m2]**		

$\boxed{\text{ULT}}$

FC1: Permanent Load

$$= \sum \gamma_{G,j} \cdot G_j + \sum \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_i$$

$$= \gamma_G \cdot G_{TOT} + \gamma_Q \cdot \psi_0 \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{wind} + \gamma_Q \cdot \psi_0 \cdot Q_{snow}$$

$$= 1{,}35 \cdot 0{,}33 + 0 + 0 + 0$$

$$Q_{per} = 0{,}446 \quad [kN/m^2]$$

FC2: Variable load

$$= \sum \gamma_{G,j} \cdot G_j + \gamma_Q \cdot Q_1 + \sum_{i>1} \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_i$$

$$= \gamma_G \cdot G_{TOT} + \gamma_Q \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{snow} + \gamma_Q \cdot \psi_0 \cdot Q_{wind}$$

$$= 1{,}2 \cdot 0{,}33 + 1{,}5 \cdot 1{,}0 + 0 + 0$$

$$Q_{var} = 1{,}896 \quad [kN/m^2]$$

$\boxed{BGT}$

$= \sum \gamma_{G,j} \cdot G_j + \gamma_Q \cdot Q_1 + \sum_{i>1} \gamma_{Qi} \cdot \psi_0 \cdot Q_i$

$= \gamma_G \cdot G_{TOT} + \gamma_Q \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{snow} + \gamma_Q \cdot \psi_0 \cdot Q_{wind}$

$= 1,0 \cdot 0,33 + 1,0 \cdot 1,0 + 0 + 0$

$Q_{Bend} = 1,33$ [kN/m²]

$q = L_{purlins} \cdot Q_{var} = 3,75 \cdot 1,896$

$q = 7,11$ [kN/m]

$M_{max} = 1/8 \cdot q \cdot L^2 = 1/8 \cdot 7,11 \cdot 2^2$

$M_{max} = 3,555$ [kNm] $= 3,555 \cdot 10^6$ [Nmm]

$V_{max} = 1/2 \cdot q \cdot L = 1/2 \cdot 7,11 \cdot 2$

$V_{max} = 7,11$ [kN]

Bending $\frac{M_d}{W*\omega_{latbuc}} \leq f_{y;d}$ for $\omega_{buc} = 1,0$

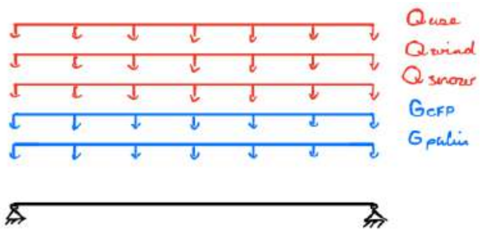
$\frac{3,555*10^6}{31,8*10^3*1,0} \leq 235 = 111,79 \leq 235$ profile satisfies

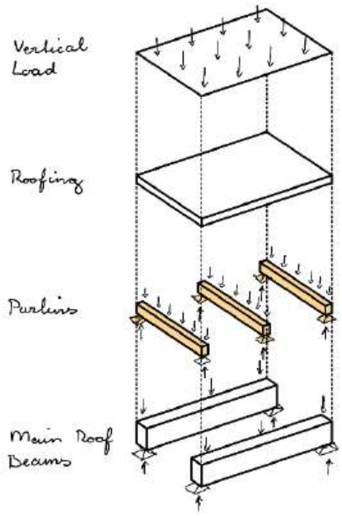
Shear $\tau_d = \frac{V_d}{A_{web}} \leq \frac{f_{y;d}}{\sqrt{3}}$

$\tau_d = \frac{7,11*10^3}{1,00*6000} \leq \frac{235}{\sqrt{3}} = 1,185 \leq 135,68$ profile satisfies

Purlins

Purlins profile:





Material: laminated timber GL24h

$h = 350\ [mm]$

$L = 6000\ [mm]$

$b = 85\ [mm]$

$W = 1{,}73 * 10^6\ [mm^3]$

$f_{m,0,d} = 24 * 0{,}7 = 16{,}8\ \left[\frac{N}{mm^2}\right]$

$f_{v,d} = 2{,}7\ \left[\frac{N}{mm^2}\right]$

$A = 29{,}7 * 10^3\ [mm^2]$

Each purlin will be loaded with the area roof load of 2000 mm wide. For the permanent weight in the table below the load over area has been multiplied by 2 meter.

Weight table for the purlins.

		Permanent load	Variable load	Source:
Roof		0,33 [kN/m^2] 0,66 [kN/m]	-	-
Purlins	GL24h 350 mm x 85 mm	0,125 [kN/m]	-	Quick reference – gl 4
Installation and piping		0,50 [kN/m^2] 1,00 [kN/m]	-	-
Category H: roofs	Roof angle $0 \leq \alpha \leq 15$	-	1,0 [kN/m^2] 2,0 [kN/m]	Quick reference – gl 5
Wind		-	[kN/m^2]	Chapter 3
Snow		-	0,56 [kN/m^2]	Chapter 3
Total		**1,785 [kN/m]**		

$\boxed{ULT}$

FC1: Permanent Load

$$Q_{per} = \sum \gamma_{G,j} \cdot G_j + \sum \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_i$$

$$= \gamma_G \cdot G_{TOT} + \gamma_Q \cdot \psi_0 \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{wind} + \gamma_Q \cdot \psi_0 \cdot Q_{snow}$$

$$= 1{,}35 \cdot 1{,}785 + 0 + 0 + 0$$

$$Q_{per} = 2{,}410 \quad [kN/m]$$

FC2: Variable load

$$Q_{var} = \sum \gamma_{G,j} \cdot G_j + \gamma_Q \cdot Q_1 + \sum_{i>1} \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_i$$

$$= \gamma_G \cdot G_{TOT} + \gamma_Q \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{snow} + \gamma_Q \cdot \psi_0 \cdot Q_{wind}$$

$$= 1{,}2 \cdot 1{,}785 + 1{,}5 \cdot 2{,}0 + 0 + 0$$

$$Q_{var} = 5{,}142 \quad [kN/m]$$

$\boxed{BGT}$

$$Q_{Bend} = \sum \gamma_{G,j} \cdot G_j + \gamma_Q \cdot Q_1 + \sum_{i>1} \gamma_{Q i} \cdot \psi_0 \cdot Q_i$$

$$= \gamma_G \cdot G_{TOT} + \gamma_Q \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{snow} + \gamma_Q \cdot \psi_0 \cdot Q_{wind}$$

$$= 1{,}0 \cdot 1{,}785 + 1{,}0 \cdot 2{,}0 + 0 + 0$$

$$Q_{Bend} = 3{,}785 \quad [kN/m^2]$$

$$M_{max} = \tfrac{1}{8} \cdot q \cdot L^2 = \tfrac{1}{8} \cdot 5{,}142 \cdot 6^2$$

$$M_{max} = 23{,}139 \quad [kNm] = 23{,}139 \cdot 10^6 \quad [Nmm]$$

$$V_{max} = \tfrac{1}{2} \cdot q \cdot L = \tfrac{1}{2} \cdot 5{,}142 \cdot 6$$

$$V_{max} = 15{,}426 \quad [kN]$$

Bending

$\frac{M_d}{W*\omega_{latbuc}} \leq f_{m,0,d}$ where $\omega_{latbuc} = 1{,}0$

$\frac{23{,}139*10^6}{1{,}73*10^6*1{,}0} \leq 16{,}8 = 13{,}38 \leq 16{,}8$ profile satisfies

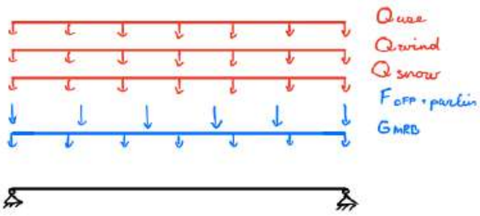
Shear

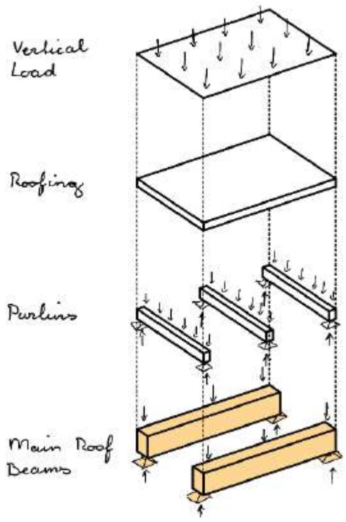
$\tau_d = \frac{3*V_d}{2*A_{web}} \leq f_{v;d}$

$\tau_d = \frac{3*15{,}426*10^3}{2*29{,}7*10^3} \leq 2{,}7 = 0{,}78 \leq 2{,}7$ profile satisfies

Main roof beams

Main roof beams profile:





Material: laminated timber GL24h

$h = 600\ [mm]$

$L = 10.000\ [mm]$

$b = 85\ [mm]$

$W = 5{,}10 * 10^6\ [mm^3]$

$f_{m,0,d} = 24 * 0{,}7 = 16{,}8\ \left[\frac{N}{mm^2}\right]$

$f_{v,d} = 2{,}7\ \left[\frac{N}{mm^2}\right]$

$A = 51{,}0 * 10^3\ [mm^2]$

Weight table for the main roof beams.

		Permanent load	Variable load	Source:
Roof Purlins	Point load	10,926 [kN]	-	-
Main roof beams	GL24h 600 mm x 85 mm	0,214 [kN/m]	-	Quick reference – gl 4
Category H: roofs	Roof angle $0 \leq \alpha \leq 15$	-	1,0 [kN/m^2]	Quick reference – gl 5
Wind		-	[kN/m^2]	Chapter 3
Snow		-	0,56 [kN/m^2]	Chapter 3
Total		**10,926 [kN]** **0,214 [kN/m]**		

$\boxed{ULT}$

FC 1: Permanent Load

$$Q_{per} = \Sigma\, \gamma_{G,j} \cdot G_j + \Sigma\, \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_i$$

$$= \gamma_G \cdot G_{MRB} + \gamma_Q \cdot \psi_0 \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{wind} + \gamma_Q \cdot \psi_0 \cdot Q_{snow}$$

$$= 1{,}35 \cdot 0{,}214 + 0 + 0 + 0$$

$$Q_{per} = 0{,}289 \quad [kN/m]$$

$$F_{per} = \gamma_G \cdot G_{CFP+purlins} + \gamma_Q \cdot \psi_0 \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{wind} + \gamma_Q \cdot \psi_0 \cdot Q_{snow}$$

$$= 1{,}35 \cdot 10{,}926 + 0 + 0 + 0$$

$$F_{per} = 14{,}75 \quad [kN]$$

FC2: Variable load

$$Q_{var} = \sum \gamma_{G,j} \cdot G_j + \gamma_Q \cdot Q_1 + \sum_{i>1} \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_i$$

$$= \gamma_G \cdot G_{MRB} + \gamma_Q \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{snow} + \gamma_Q \cdot \psi_0 \cdot Q_{wind}$$

$$= 1{,}2 \cdot 0{,}214 + 1{,}5 \cdot 1{,}0 + 0 + 0$$

$$Q_{var} = 1{,}757 \quad [kN/m]$$

$$F_{var} = \gamma_G \cdot G_{CFP+purlins} + \gamma_Q \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{snow} + \gamma_Q \cdot \psi_0 \cdot Q_{wind}$$

$$= 1{,}2 \cdot 10{,}926 + 1{,}5 \cdot 1{,}0 + 0 + 0$$

$$F_{per} = 14{,}611 \quad [kN]$$

$\boxed{BGT}$

$$Q_{Bend} = \sum \gamma_{G,j} \cdot G_j + \gamma_Q \cdot Q_1 + \sum_{i>1} \gamma_{Q,i} \cdot \psi_0 \cdot Q_i$$

$$= \gamma_G \cdot G_{MRB} + \gamma_Q \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{snow} + \gamma_Q \cdot \psi_0 \cdot Q_{wind}$$

$$= 1{,}0 \cdot 0{,}214 + 1{,}0 \cdot 1{,}0 + 0 + 0$$

$$Q_{Bend} = 1{,}214 \quad [kN/m^2]$$

$$F_{Bend} = \gamma_G \cdot G_{CFP+purl} + \gamma_Q \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{snow} + \gamma_Q \cdot \psi_0 \cdot Q_{wind}$$

$$= 1{,}0 \cdot 10{,}926 + 1{,}0 \cdot 1{,}0 + 0 + 0$$

$$F_{Bend} = 11{,}926 \quad [kN]$$

$$M_{max} = 1/8 \cdot q \cdot L^2 + F(L/2) + F(L/2 - 2) + F(L/2 - 4)$$

$$= 1/8 \cdot 1{,}757 \cdot 10^2 + 14{,}611 \cdot (10/2) + 14{,}611 (10/2 - 2) + 14{,}611 (10/2 - 4)$$

$$M_{max} = 153{,}46 \quad [kNm] = 153{,}46 \cdot 10^6 \quad [Nmm]$$

$$V_{max} = 1/2 (q \cdot L + 6 \cdot F)$$

$$V_{max} = 1/2 (1{,}757 \cdot 10 + 6 \cdot 14{,}611) = 52{,}62 \quad [kN]$$

Bending

$\frac{M_d}{W*\omega_{latbuc}} \leq f_{m,0,d}$ where $\omega_{latbuc} = 1{,}0$

$\frac{153{,}46*10^6}{5{,}10*10^6*1{,}0} = \leq 16{,}8 = 30{,}09 > 16{,}8$ larger profile needed

Profile GL24h 600 mm x 185 mm

$\frac{153{,}46*10^6}{11{,}10*10^6*1{,}0} = \leq 16{,}8 = 13{,}83 \leq 16{,}8$ profile satisfies

Shear

$\tau_d = \frac{3*V_d}{2*A_{web}} \leq f_{v;d}$

$\tau_d = \frac{3*152{,}62*10^3}{2*111{,}0*10^3} \leq 2{,}06 = \leq 2{,}7$ profile satisfies

Quick reference timber design 2: straight beam (possibly with precamber)

Glulam 600 mm x 185 mm (h x b) profile

Profile properties:

Area (face) $A = 111{,}0 * 10^3 \; [\mathrm{mm}^2]$

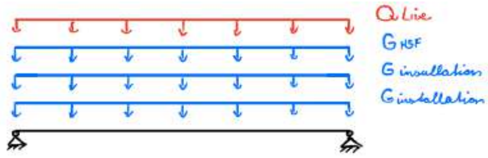
Density gl24h $\rho_{mean} = 420 \left[\frac{\mathrm{kg}}{\mathrm{m}^3}\right]$

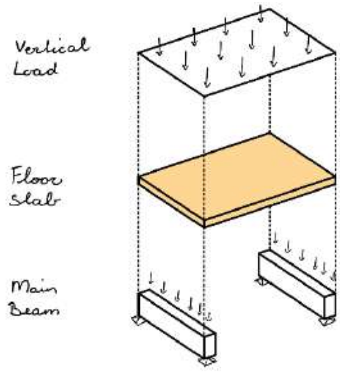
Weight $G = A * \rho_{mean} = 111{,}0 * 10^3 * 10^{-6} * 420 * 10$

$G = 0{,}466 \left[\frac{\mathrm{kN}}{\mathrm{m}}\right]$

Floor slab first floor

Hollow slab floor:





Material: Dycore type T

$L = 3750\ [mm]$

$d = 255\ [mm]$

$b = 1197\ [mm]$

$f_s = 235\ \left[\frac{N}{mm^2}\right]$

Weight table for the floor slabs on the first floor.

		Permanent load	Variable load	Source:
Hollow slab floor	Dycore type T: 255 mm	3,7 [kN/m^2]	-	Quick reference – gl 4
Insulation		0,20 [kN/m^2]	-	Quick reference – gl 4
Installation and piping		0,50 [kN/m^2]	-	-
Liveload (client)	Gallery C3: people walking around	-	7,5 [kN/m^2]	Chapter 3
Total		**4,4 [kN/m2]**		

$\boxed{ULT}$

FC1: Permanent Load

$$Q_{per} = \sum \gamma_{G,j} \cdot G_j + \sum \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_i$$

$$= \gamma_G \cdot G_{TOT} + \gamma_Q \cdot \psi_0 \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{wind} + \gamma_Q \cdot \psi_0 \cdot Q_{snow}$$

$$= 1{,}35 \cdot 4{,}4 + 0 + 0 + 0$$

$$Q_{per} = 5{,}94 \quad [kN/m]$$

FC2: Variable load

$$Q_{var} = \sum \gamma_{G,j} \cdot G_j + \gamma_Q \cdot Q_1 + \sum_{i>1} \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_i$$

$$= \gamma_G \cdot G_{TOT} + \gamma_Q \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{snow} + \gamma_Q \cdot \psi_0 \cdot Q_{wind}$$

$$= 1{,}2 \cdot 4{,}4 + 1{,}5 \cdot 7{,}5 \cdot 1{,}197 + 0 + 0$$

$$Q_{var} = 18{,}75 \quad [kN/m]$$

$\boxed{BGT}$

$$Q_{Bend} = \Sigma \gamma_{G,j} \cdot G_j + \gamma_Q \cdot Q_1 + \sum_{i>1} \gamma_{Qi} \cdot \psi_0 \cdot Q_i$$

$$= \gamma_G \cdot G_{TOT} + \gamma_Q \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{snow} + \gamma_Q \cdot \psi_0 \cdot Q_{wind}$$

$$= 1{,}0 \cdot 4{,}4 + 1{,}0 \cdot 7{,}5 \cdot + 0 + 0$$

$$Q_{Bend} = 11{,}9 \quad [kN/m^2]$$

$$M_{max} = 1/8 \cdot q \cdot L^2 = 1/8 \cdot 18{,}75 \cdot 3{,}75^2$$

$$M_{max} = 32{,}95 \quad [kNm] = 32{,}95 \cdot 10^6 \quad [Nmm]$$

$$V_{max} = 1/2 \cdot q \cdot L = 1/2 \cdot 18{,}75 \cdot 3{,}75$$

$$V_{max} = 35{,}16 \quad [kN]$$

Concrete

Prefab concrete

Reinforcement

$N_s = \frac{M_d}{0{,}75*h} = \frac{29{,}06*10^6}{0{,}75*255} = 151{,}95 * 10^3 \ [N]$

$A_s = \frac{N_s}{f_s} = \frac{151{,}95*10^3}{235} = 646{,}57 \ [mm^2]$

$\rho = \frac{A_s}{b*d} * 100 = \frac{646{,}57}{1197*255} * 100 = 0{,}2$ % profile satisfies

for beams 0,8% to 1,2% economic value

Profile Dycore Type W150

Reinforcement

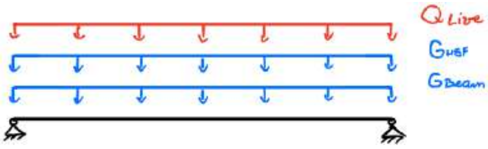
$N_s = \frac{M_d}{0{,}75*h} = \frac{29{,}06*10^6}{0{,}75*150} = 258.311 \ [N]$

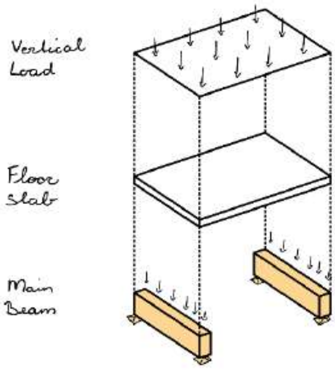
$A_s = \frac{N_s}{f_s} = \frac{258.311}{235} = 1099{,}20 \ [mm^2]$

$\rho = \frac{A_s}{b*d} * 100 = \frac{1099{,}20}{1197*150} * 100 = 0{,}612$ % profile satisfies

Main beams first floor

Material: laminated timber GL24h





$h = 600\ [mm]$

$L = 10.000\ [mm]$

$b = 85\ [mm]$

$W = 5{,}10 * 10^6\ [mm^3]$

$f_{m,0,d} = 24 * 0{,}7 = 16{,}8\ \left[\frac{N}{mm^2}\right]$

$f_{v,d} = 2{,}7\ \left[\frac{N}{mm^2}\right]$

$A = 51{,}0 * 10^3\ [mm^2]$

Weight table for the main beams on the first floor.

		Permanent load	Variable load	Source:
Hollow slab floor		4,4 [kN/m^2] 16,5 [kN/m]	-	-
Main roof beams	GL24h 600 mm x 85 mm	0,214 [kN/m]	-	Quick reference – gl 4
Liveload (client)	Gallery C3: people walking around	-	7,5 [kN/m^2]	Chapter 3
Total		16,714 [kN/m]		

$\boxed{\text{UGT}}$

FC1: Permanent Load

$$Q_{per} = \sum \gamma_{G,j} \cdot G_j + \sum \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_i$$

$$= \gamma_G \cdot G_{TOT} + \gamma_Q \cdot \psi_0 \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{wind} + \gamma_Q \cdot \psi_0 \cdot Q_{snow}$$

$$= 1{,}35 \cdot 16{,}714 + 0 + 0 + 0$$

$$Q_{per} = 22{,}56\ [kN/m]$$

FC2: Variable load

$$Q_{var} = \sum \gamma_{G,j} \cdot G_j + \gamma_Q \cdot Q_1 + \sum_{i>1} \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_i$$

$$= \gamma_G \cdot G_{TOT} + \gamma_Q \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{snow} + \gamma_Q \cdot \psi_0 \cdot Q_{wind}$$

$$= 1{,}2 \cdot 16{,}714 + 1{,}5 \cdot 7{,}5 \cdot 3{,}75 + 0 + 0$$

$$Q_{var} = 62{,}24\ [kN/m]$$

$\boxed{BGT}$

$Q_{Bend} = \Sigma \gamma_{G;j} \cdot G_j + \gamma_Q \cdot Q_1 + \sum_{i>1} \gamma_{Qi} \cdot \psi_0 \cdot Q_i$

$= \gamma_G \cdot G_{TOT} + \gamma_Q \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{snow} + \gamma_Q \cdot \psi_0 \cdot Q_{wind}$

$= 1{,}0 \cdot 16{,}714 + 1{,}0 \cdot 7{,}5 \cdot 3{,}75 + 0 + 0$

$Q_{Bend} = 44{,}84$ [kN/m²]

$M_{max} = 1/8 \cdot q \cdot L^2 = 1/8 \cdot 62{,}24 \cdot 5^2$

$M_{max} = 194{,}5$ [kNm] $= 194{,}5 \cdot 10^6$ [Nmm]

$V_{max} = 1/2 \cdot q \cdot L = 1/2 \cdot 62{,}24 \cdot 5$

$V_{max} = 155{,}6$ [kN]

Bending

$\frac{M_d}{W*\omega_{latbuc}} \leq f_{m,0,d}$ where $\omega_{latbuc} = 1{,}0$

$\frac{194{,}5*10^6}{5{,}10*10^6*1{,}0} = \leq 16{,}8 = 38{,}14 > 16{,}8$ larger profile needed

Profile GL24h 600 mm x 205 mm

$\frac{194{,}5*10^6}{12{,}30*10^6*1{,}0} = \leq 16{,}8 = 15{,}81 \leq 16{,}8$ profile satisfies

Shear

$\tau_d = \frac{3*V_d}{2*A_{web}} \leq f_{v;d}$

$\tau_d = \frac{3*155{,}6*10^3}{2*123{,}0*10^3} \leq 1{,}90 =\leq 2{,}7$ profile satisfies

Quick reference timber design 2: straight beam (possibly with precamber)

Glulam 600 mm x 205 mm (h x b) profile

Profile properties:

Area (face)	$A = 123{,}0 * 10^3$	[mm²]
Density gl24h	$\rho_{mean} = 420$	[kg/m³]
Weight	$G = A * \rho_{mean} = 123{,}0 * 10^3 * 10^{-6} * 420 * 10$	
	$G = 0{,}517$	[kN/m]

Walls

Material: laminated timber GL24h

$h = 5000\ [mm]$

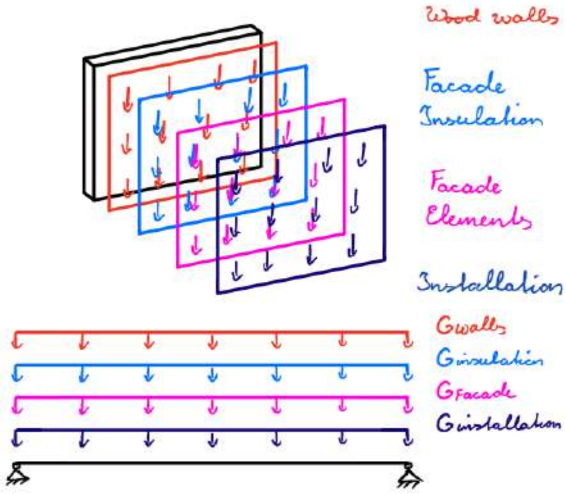
$L = 3750\ [mm]$

$b = 200\ [mm]$

$f_{m,0,d} = 24 * 0{,}7 = 16{,}8\ \left[\frac{N}{mm^2}\right]$

$f_{v,d} = 2{,}7\ \left[\frac{N}{mm^2}\right]$

$A = 375.000 * 10^3\ [mm^2]$



Weight table for the walls.

		Permanent load	Variable load	Source:
Insulation		0,05 [kN/m²]	-	Quick reference – gl 4
Façade elements		0,13 [kN/m²]		Quick reference – gl 4
Installation		0,50 [kN/m²]		-
Walls timber	***420*5*10/1000	21 [kN/m²]	-	Chapter 3
Total		**21,68 [kN/m²]**		

$\boxed{\text{ULT}}$

FC1: Permanent Load

$$Q_{per} = \Sigma\, \gamma_{G,j} \cdot G_j + \Sigma\, \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_i$$

$$= \gamma_G \cdot G_{TOT} + \gamma_Q \cdot \psi_0 \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{wind} + \gamma_Q \cdot \psi_0 \cdot Q_{snow}$$

$$= 1{,}35 \cdot 21{,}68 + 0 + 0 + 0$$

$$Q_{per} = 29{,}27 \quad [kN/m^2]$$

FC2: Variable load

$$Q_{var} = \Sigma\, \gamma_{G,j} \cdot G_j + \gamma_Q \cdot Q_1 + \sum_{i>1} \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_i$$

$$= \gamma_G \cdot G_{TOT} + \gamma_Q \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{snow} + \gamma_Q \cdot \psi_0 \cdot Q_{wind}$$

$$= 1{,}2 \cdot 21{,}86 + 1{,}5 \cdot 0 + 0 + 0$$

$$Q_{var} = 26{,}02 \quad [kN/m^2]$$

$\boxed{\text{BGT}}$

$$Q_{Bend} = \sum \gamma_{G,j} \cdot G_j + \gamma_Q \cdot Q_1 + \sum_{i>1} \gamma_{Q_i} \cdot \psi_0 \cdot Q_i$$

$$= \gamma_G \cdot G_{TOT} + \gamma_Q \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{snow} + \gamma_Q \cdot \psi_0 \cdot Q_{wind}$$

$$= 1{,}0 \cdot 21{,}68 + 1{,}0 \cdot 0 + 0 + 0$$

$$Q_{Bend} = 21{,}68 \quad [kN/m^2]$$

$$V_{max} = Q \cdot A = 29{,}27 \cdot (3{,}75 \cdot 0{,}20)$$

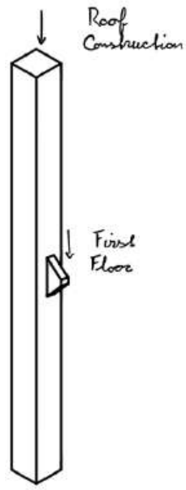
$$V_{max} = 21{,}95 \quad [kN]$$

Axial compressive force $\sigma_{c;0;d} = \frac{N_{Ed}}{A} \leq f_{c,0,d} * \omega_{buc}$

$\sigma_{c;0;d} = \frac{21{,}95*10^3}{750.000} \leq 16{,}8 * 1{,}0 = 0{,}03 \leq 16{,}8$ profile satisfies

Columns

Material: laminated timber GL24h



$h = 9300\ [mm]$

$L = 650\ [mm]$

$b = 205\ [mm]$

$W = 14{,}43 * 10^6\ [mm^3]$

$f_{m,0,d} = 24 * 0{,}7 = 16{,}8\ \left[\frac{N}{mm^2}\right]$

$f_{v,d} = 2{,}7\ \left[\frac{N}{mm^2}\right]$

$A = 133.200\ [mm^2]$

Weight table for the columns.

		Permanent load	Variable load	Source:
Roof construction		52,62 [kN]	-	-
First floor		155,6 [kN]	-	-
Walls		21,95 [kN]		-
Total		**230,17 [kN]**		

Axial compressive force $\sigma_{c;0;d} = \frac{N_{Ed}}{A} \leq f_{c,0,d} * \omega_{buc}$

$\sigma_{c;0;d} = \frac{230{,}17*10^3}{133.200} \leq 16{,}8 * 1{,}0 = 1{,}73 \leq 16{,}8$ profile satisfies

Bending

$\frac{M_d}{W*\omega_{latbuc}} \leq f_{m,0,d}$ where $\omega_{latbuc} = 1{,}0$

$\frac{230{,}17*10^6}{14{,}43*10^6*1{,}0} = \leq 16{,}8 = 15{,}95 \leq 16{,}8$ profile satisfies

4.2 Design Alternative 2: “Bulk”

Element	*Material*	*Profile*	*New profile*
Roof	Cold formed corrugated sheet	h = 106 mm t = 1,00 mm	- -
Purlin	Timber GL24h	350 mm x 85 mm	-
Main Roof Beam	Timber GL24h	600 mm x 85 mm	600 mm x 185 mm
Hollow slab floor	Prefab concrete	Dycore type T255	Dycore type W150
Main beam first floor	Precast concrete I – beam	IC32/70	-
Walls	Precast concrete C30/37	b = 200 mm	-
Columns	Precast concrete C30/37	200 mm x 200 mm	400 mm x 400 mm

Roof

Cold formed corrugated sheet profile:

Material: steel S235

$h = 106\ [mm]$

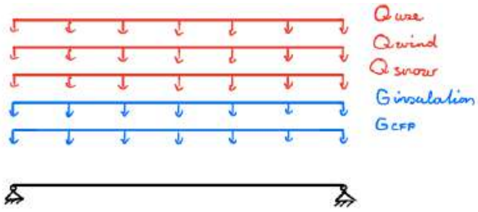
$L = 2000\ [mm]$

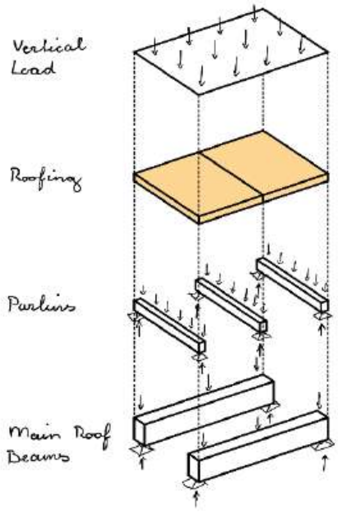
$b = 6000\ [mm]$

$t = 1{,}00\ [mm]$

$W = 31{,}8 * 10^3\ [mm^3]$

$f_{y;d} = 235\ \left[\frac{N}{mm^2}\right]$





Weight table for the roof.

Element	Detail	Permanent load	Variable load	Source:
Cold formed corrugated sheet		0,130 [kN/m^2]	-	Quick reference – sb 16
Insulation		0,20 [kN/m^2]	-	Quick reference – gl 4
Use		-	1,00 [kN/m^2]	Quick reference – gl 5
Wind		-	0,46 [kN/m^2]	Chapter 3
Snow		-	0,56 [kN/m^2]	Chapter 3
Total		**0,33 [kN/m2]**		

$\boxed{ULT}$

FC1: Permanent Load

$= \sum \gamma_{G,j} \cdot G_j + \sum \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_i$

$= \gamma_G \cdot G_{TOT} + \gamma_Q \cdot \psi_0 \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{wind} + \gamma_Q \cdot \psi_0 \cdot Q_{snow}$

$= 1{,}35 \cdot 0{,}33 + 0 + 0 + 0$

$Q_{per} = 0{,}446 \quad [kN/m^2]$

FC2: Variable load

$= \sum \gamma_{G,j} \cdot G_j + \gamma_Q \cdot Q_1 + \sum_{i>1} \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_i$

$= \gamma_G \cdot G_{TOT} + \gamma_Q \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{snow} + \gamma_Q \cdot \psi_0 \cdot Q_{wind}$

$= 1{,}2 \cdot 0{,}33 + 1{,}5 \cdot 1{,}0 + 0 + 0$

$Q_{var} = 1{,}896 \quad [kN/m^2]$

$\boxed{BGT}$

$= \sum \gamma_{G,j} \cdot G_j + \gamma_Q \cdot Q_1 + \sum_{i>1} \gamma_{Qi} \cdot \psi_0 \cdot Q_i$

$= \gamma_G \cdot G_{TOT} + \gamma_Q \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{snow} + \gamma_Q \cdot \psi_0 \cdot Q_{wind}$

$= 1{,}0 \cdot 0{,}33 + 1{,}0 \cdot 1{,}0 + 0 + 0$

$Q_{Bend} = 1{,}33$ [kN/m²]

$q = L_{purlins} \cdot Q_{var} = 3{,}75 \cdot 1{,}896$

$q = 7{,}11$ [kN/m]

$M_{max} = 1/8 \cdot q \cdot L^2 = 1/8 \cdot 7{,}11 \cdot 2^2$

$M_{max} = 3{,}555$ [kNm] $= 3{,}555 \cdot 10^6$ [Nmm]

$V_{max} = 1/2 \cdot q \cdot L = 1/2 \cdot 7{,}11 \cdot 2$

$V_{max} = 7{,}11$ [kN]

Steel tests

Bending $\frac{M_d}{W*\omega_{latbuc}} \leq f_{y;d}$ for $\omega_{buc} = 1{,}0$

$\frac{3{,}555*10^6}{31{,}8*10^3*1{,}0} \leq 235 = 111{,}79 \leq 235$ profile satisfies

Shear $\tau_d = \frac{V_d}{A_{web}} \leq \frac{f_{y;d}}{\sqrt{3}}$

$\tau_d = \frac{7{,}11*10^3}{1{,}00*6000} \leq \frac{235}{\sqrt{3}} = 1{,}185 \leq 135{,}68$ profile satisfies

Purlins

Purlins profile:

Material: laminated timber GL24h

$h = 350\ [mm]$

$L = 6000\ [mm]$

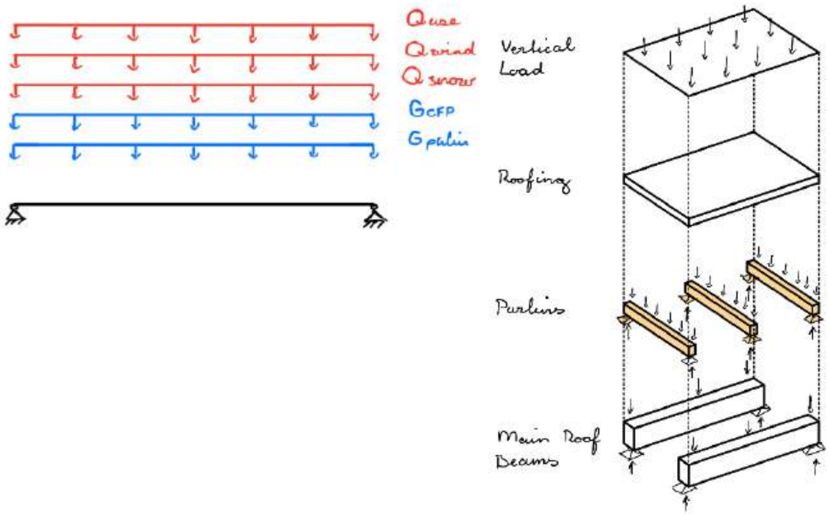
$b = 85\ [mm]$

$W = 1{,}73 * 10^6\ [mm^3]$

$f_{m,0,d} = 24 * 0{,}7 = 16{,}8\ \left[\frac{N}{mm^2}\right]$

$f_{v,d} = 2{,}7\ \left[\frac{N}{mm^2}\right]$

$A = 29{,}7 * 10^3\ [mm^2]$



Each purlin will be loaded with the area roof load of 2000 mm wide. For the permanent weight in the table below the load over area has been multiplied by 2 meter.

Weight table for the purlins.

		Permanent load	Variable load	Source:
Roof		0,33 [kN/m^2] 0,66 [kN/m]	-	-
Purlins	GL24h 350 mm x 85 mm	0,125 [kN/m]	-	Quick reference – gl 4
Installation and piping		0,50 [kN/m^2] 1,00 [kN/m]	-	-
Category H: roofs	Roof angle $0 \leq \alpha \leq 15$	-	1,0 [kN/m^2] 2,0 [kN/m]	Quick reference – gl 5
Wind		-	[kN/m^2]	Chapter 3
Snow		-	0,56 [kN/m^2]	Chapter 3
Total		**1,785 [kN/m]**		

$\boxed{\text{UGT}}$

FC1: Permanent Load

$$Q_{per} = \Sigma\, \gamma_{G,j} \cdot G_j + \Sigma\, \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_i$$

$$= \gamma_G \cdot G_{TOT} + \gamma_Q \cdot \psi_0 \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{wind} + \gamma_Q \cdot \psi_0 \cdot Q_{snow}$$

$$= 1{,}35 \cdot 1{,}785 + 0 + 0 + 0$$

$$Q_{per} = 2{,}410\ [kN/m]$$

FC2: Variable load

$Q_{var} = \sum \gamma_{G,j} \cdot G_j + \gamma_Q \cdot Q_1 + \sum_{i>1} \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_i$

$= \gamma_G \cdot G_{TOT} + \gamma_Q \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{snow} + \gamma_Q \cdot \psi_0 \cdot Q_{wind}$

$= 1{,}2 \cdot 1{,}785 + 1{,}5 \cdot 2{,}0 + 0 + 0$

$Q_{var} = 5{,}142$ [kN/m]

$\boxed{BGT}$

$Q_{Bend} = \sum \gamma_{G,j} \cdot G_j + \gamma_Q \cdot Q_1 + \sum_{i>1} \gamma_{Q,i} \cdot \psi_0 \cdot Q_i$

$= \gamma_G \cdot G_{TOT} + \gamma_Q \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{snow} + \gamma_Q \cdot \psi_0 \cdot Q_{wind}$

$= 1{,}0 \cdot 1{,}785 + 1{,}0 \cdot 2{,}0 + 0 + 0$

$Q_{Bend} = 3{,}785$ [kN/m²]

$M_{max} = 1/8 \cdot q \cdot L^2 = 1/8 \cdot 5{,}142 \cdot 6^2$

$M_{max} = 23{,}139$ [kNm] $= 23{,}139 \cdot 10^6$ [Nmm]

$V_{max} = 1/2 \cdot q \cdot L = 1/2 \cdot 5{,}142 \cdot 6$

$V_{max} = 15{,}426$ [kN]

Bending

$\frac{M_d}{W*\omega_{latbuc}} \leq f_{m,0,d}$ where $\omega_{latbuc} = 1{,}0$

$\frac{23{,}139*10^6}{1{,}73*10^6*1{,}0} \leq 16{,}8 = 13{,}38 \leq 16{,}8$ profile satisfies

Shear

$\tau_d = \frac{3*V_d}{2*A_{web}} \leq f_{v;d}$

$\tau_d = \frac{3*15{,}426*10^3}{2*29{,}7*10^3} \leq 2{,}7 = 0{,}78 \leq 2{,}7$ profile satisfies

Main roof beams

Main roof beams profile:

Material: laminated timber GL24h

$h = 600\ [mm]$

$L = 10.000\ [mm]$

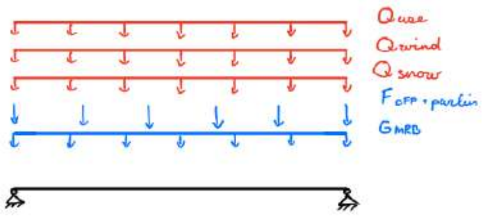
$b = 85\ [mm]$

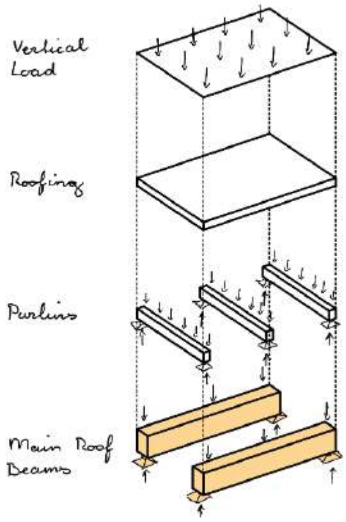
$W = 5{,}10 * 10^6\ [mm^3]$

$f_{m,0,d} = 24 * 0{,}7 = 16{,}8\ \left[\frac{N}{mm^2}\right]$

$f_{v,d} = 2{,}7\ \left[\frac{N}{mm^2}\right]$

$A = 51{,}0 * 10^3\ [mm^2]$





Weight table for the main roof beams.

		Permanent load	Variable load	Source:
Roof Purlins	Point load	10,926 [kN]	-	-
Main roof beams	GL24h 600 mm x 85 mm	0,214 [kN/m]	-	Quick reference – gl 4
Category H: roofs	Roof angle $0 \leq \alpha \leq 15$	-	1,0 [kN/m²]	Quick reference – gl 5
Wind		-	[kN/m²]	Chapter 3
Snow		-	0,56 [kN/m²]	Chapter 3
Total		**10,926 [kN]** **0,214 [kN/m]**		

$\boxed{\text{UGT}}$

FC1: Permanent Load

$$Q_{per} = \Sigma\, \gamma_{G,j} \cdot G_j + \Sigma\, \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_i$$

$$= \gamma_G \cdot G_{MRB} + \gamma_Q \cdot \psi_0 \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{wind} + \gamma_Q \cdot \psi_0 \cdot Q_{snow}$$

$$= 1{,}35 \cdot 0{,}214 + 0 + 0 + 0$$

$$Q_{per} = 0{,}289\ [kN/m]$$

$$F_{per} = \gamma_G \cdot G_{CFP+purlins} + \gamma_Q \cdot \psi_0 \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{wind} + \gamma_Q \cdot \psi_0 \cdot Q_{snow}$$

$$= 1{,}35 \cdot 10{,}926 + 0 + 0 + 0$$

$$F_{per} = 14{,}75 \quad [kN]$$

FC2: Variable load

$$Q_{var} = \sum \gamma_{G,j} \cdot G_j + \gamma_Q \cdot Q_1 + \sum_{i>1} \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_i$$

$$= \gamma_G \cdot G_{MRB} + \gamma_Q \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{snow} + \gamma_Q \cdot \psi_0 \cdot Q_{wind}$$

$$= 1{,}2 \cdot 0{,}214 + 1{,}5 \cdot 1{,}0 + 0 + 0$$

$$Q_{var} = 1{,}757 \quad [kN/m]$$

$$F_{var} = \gamma_G \cdot G_{CFP+purlins} + \gamma_Q \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{snow} + \gamma_Q \cdot \psi_0 \cdot Q_{wind}$$

$$= 1{,}2 \cdot 10{,}926 + 1{,}5 \cdot 1{,}0 + 0 + 0$$

$$F_{per} = 14{,}611 \quad [kN]$$

$\boxed{BGT}$

$$Q_{Bend} = \sum \gamma_{G,j} \cdot G_j + \gamma_Q \cdot Q_1 + \sum_{i>1} \gamma_{Q,i} \cdot \psi_0 \cdot Q_i$$

$$= \gamma_G \cdot G_{MRB} + \gamma_Q \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{snow} + \gamma_Q \cdot \psi_0 \cdot Q_{wind}$$

$$= 1{,}0 \cdot 0{,}214 + 1{,}0 \cdot 1{,}0 + 0 + 0$$

$$Q_{Bend} = 1{,}214 \quad [kN/m^2]$$

$$F_{Bend} = \gamma_G \cdot G_{CFP+purl} + \gamma_Q \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{snow} + \gamma_Q \cdot \psi_0 \cdot Q_{wind}$$

$$= 1{,}0 \cdot 10{,}926 + 1{,}0 \cdot 1{,}0 + 0 + 0$$

$$F_{Bend} = 11{,}926 \quad [kN]$$

$$M_{max} = 1/8 \cdot q \cdot L^2 + F(L/2) + F(L/2 - 2) + F(L/2 - 4)$$

$$= 1/8 \cdot 1{,}757 \cdot 10^2 + 14{,}611 \cdot (10/2) + 14{,}611\,(10/2 - 2) + 14{,}611\,(10/2 - 4)$$

$$M_{max} = 153{,}46 \; [kNm] = 153{,}46 \cdot 10^6 \; [Nmm]$$

$$V_{max} = 1/2\,(q \cdot L + 6 \cdot F)$$

$$V_{max} = 1/2\,(1{,}757 \cdot 10 + 6 \cdot 14{,}611) = 52{,}62 \; [kN]$$

Bending

$\frac{M_d}{W*\omega_{latbuc}} \leq f_{m,0,d}$ where $\omega_{latbuc} = 1{,}0$

$\frac{153{,}46*10^6}{5{,}10*10^6*1{,}0} = \leq 16{,}8 = 30{,}09 > 16{,}8$ larger profile needed

Profile GL24h 600 mm x 185 mm

$\frac{153{,}46*10^6}{11{,}10*10^6*1{,}0} = \leq 16{,}8 = 13{,}83 \leq 16{,}8$ profile satisfies

Shear

$\tau_d = \frac{3*V_d}{2*A_{web}} \leq f_{v;d}$

$\tau_d = \frac{3*152{,}62*10^3}{2*111{,}0*10^3} \leq 2{,}06 = \leq 2{,}7$ profile satisfies

Quick reference timber design 2: straight beam (possibly with precamber)

Glulam 600 mm x 185 mm (h x b) profile

Profile properties:

Area (face)	$A = 111{,}0 * 10^3$	[mm^2]
Density gl24h	$\rho_{mean} = 420$	[kg/m^3]
Weight	$G = A * \rho_{mean} = 111{,}0 * 10^3 * 10^{-6} * 420 * 10$	
	$G = 0{,}466$	[kN/m]

Floor slab first floor

Hollow slab floor:

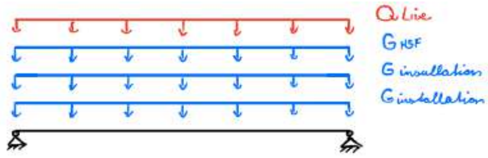
Material: Dycore type T

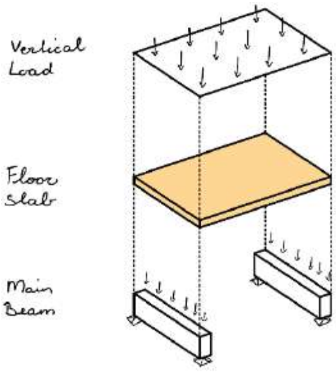
$L = 3750\ [mm]$

$d = 255\ [mm]$

$b = 1197\ [mm]$

$f_s = 235\ \left[\frac{N}{mm^2}\right]$





Weight table for the floor slabs on the first floor.

		Permanent load	Variable load	Source:
Hollow slab floor	Dycore type T: 255 mm	3,7 [kN/m^2]	-	Quick reference – gl 4
Insulation		0,20 [kN/m^2]	-	Quick reference – gl 4
Installation and piping		0,50 [kN/m^2]	-	-
Liveload (client)	Gallery C3: people walking around	-	7,5 [kN/m^2]	Chapter 3
Total		4,4 [kN/m2]		

ULT

FC1: Permanent Load

$$Q_{per} = \sum \gamma_{G,j} \cdot G_j + \sum \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_i$$

$$= \gamma_G \cdot G_{TOT} + \gamma_Q \cdot \psi_0 \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{wind} + \gamma_Q \cdot \psi_0 \cdot Q_{snow}$$

$$= 1{,}35 \cdot 4{,}4 + 0 + 0 + 0$$

$$Q_{per} = 5{,}94 \quad [kN/m]$$

FC2: Variable load

$$Q_{var} = \sum \gamma_{G,j} \cdot G_j + \gamma_Q \cdot Q_1 + \sum_{i>1} \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_i$$

$$= \gamma_G \cdot G_{TOT} + \gamma_Q \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{snow} + \gamma_Q \cdot \psi_0 \cdot Q_{wind}$$

$$= 1{,}2 \cdot 4{,}4 + 1{,}5 \cdot 7{,}5 \cdot 1{,}197 + 0 + 0$$

$$Q_{var} = 18{,}75 \quad [kN/m]$$

BGT

$$Q_{Bend} = \Sigma \gamma_{G,j} \cdot G_j + \gamma_Q \cdot Q_1 + \sum_{i>1} \gamma_{Qi} \cdot \psi_0 \cdot Q_i$$

$$= \gamma_G \cdot G_{TOT} + \gamma_Q \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{snow} + \gamma_Q \cdot \psi_0 \cdot Q_{wind}$$

$$= 1{,}0 \cdot 4{,}4 + 1{,}0 \cdot 7{,}5 + 0 + 0$$

$$Q_{Bend} = 11{,}9 \quad [kN/m^2]$$

$$M_{max} = 1/8 \cdot q \cdot L^2 = 1/8 \cdot 18{,}75 \cdot 3{,}75^2$$

$$M_{max} = 32{,}95 \quad [kNm] = 32{,}95 \cdot 10^6 \quad [Nmm]$$

$$V_{max} = 1/2 \cdot q \cdot L = 1/2 \cdot 18{,}75 \cdot 3{,}75$$

$$V_{max} = 35{,}16 \quad [kN]$$

Concrete

Prefab concrete

Reinforcement $N_s = \frac{M_d}{0{,}75*h} = \frac{29{,}06*10^6}{0{,}75*255} = 151{,}95*10^3\ [N]$

$A_s = \frac{N_s}{f_s} = \frac{151{,}95*10^3}{235} = 646{,}57\ [mm^2]$

$\rho = \frac{A_s}{b*d} = \frac{646{,}57}{1197*255} = 0{,}002\ [-]$

for beams 0,8% to 1,2% economic value

Profile Dycore Type W150

Reinforcement $N_s = \frac{M_d}{0{,}75*h} = \frac{29{,}06*10^6}{0{,}75*150} = 258.311\ [N]$

$A_s = \frac{N_s}{f_s} = \frac{258.311}{235} = 1099{,}20\ [mm^2]$

$\rho = \frac{A_s}{b*d} * 100 = \frac{1099{,}20}{1197*150} * 100 = 0{,}612\ \%$

Shear force $v_{ed} = \frac{V_{Ed}}{b*d}$

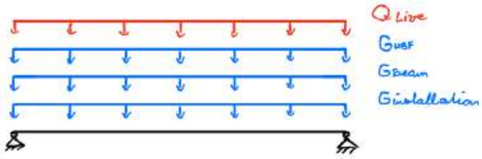
Deformation $E_{c,eff} = 0{,}33 * E_{cm}$

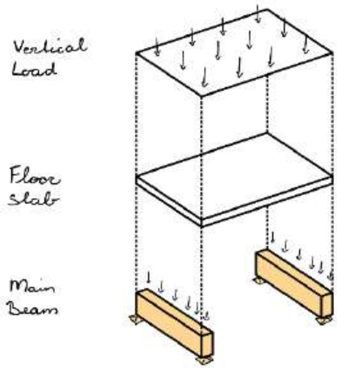
Main beams first floor

Main beams first floor:

Material: I – beam

$L = 10000\ [mm]$





Each beam will be loaded by a width of 3750 mm of hollow slab floor. Permanent distributed load in the table below have been accounted for this.

Weight table for the main beams on the first floor.

		Permanent load	Variable load	Source:
Hollow slab floor		4,4 [kN/m^2] 16,5 [kN/m]	-	-
I – beam		3,618 [kN/m]	-	Quick reference – gl 4
Liveload (client)	Gallery C3: people walking around	-	7,5 [kN/m^2]	Chapter 3
Total		**20,118 [kN/m]**		

$\boxed{UGT}$

FC1: Permanent Load

$$Q_{per} = \sum \gamma_{G,j} \cdot G_j + \sum \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_i$$

$$= \gamma_G \cdot G_{TOT} + \gamma_Q \cdot \psi_0 \cdot Q_{live} + \gamma_Q \cdot \psi_0 \cdot Q_{wind} + \gamma_Q \cdot \psi_0 \cdot Q_{snow}$$

$$= 1{,}35 \cdot 20{,}118 + 0 + 0 + 0$$

$$Q_{per} = 27{,}159 \quad [kN/m]$$

FC2: Variable load

$$Q_{var} = \sum \gamma_{G,j} \cdot G_j + \gamma_Q \cdot Q_1 + \sum_{i>1} \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_i$$

$$= \gamma_G \cdot G_{TOT} + \gamma_Q \cdot Q_{live} + \gamma_Q \cdot \psi_0 \cdot Q_{snow} + \gamma_Q \cdot \psi_0 \cdot Q_{wind}$$

$$= 1{,}2 \cdot 20{,}118 + 1{,}5 \cdot 7{,}5 \cdot 3{,}75 + 0 + 0$$

$$Q_{var} = 66{,}33 \quad [kN/m]$$

$\boxed{BGT}$

$$Q_{Bend} = \sum \gamma_{G,j} \cdot G_j + \gamma_Q \cdot Q_1 + \sum_{i>1} \gamma_{Q,i} \cdot \psi_0 \cdot Q_i$$

$$= \gamma_G \cdot G_{TOT} + \gamma_Q \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{snow} + \gamma_Q \cdot \psi_0 \cdot Q_{wind}$$

$$= 1{,}0 \cdot 20{,}118 + 1{,}0 \cdot 7{,}5 + 0 + 0$$

$$Q_{Bend} = 27.618 \quad [kN/m^2]$$

$$M_{max} = 1/8 \cdot q \cdot L^2 = 1/8 \cdot 66{,}33 \cdot 10^2$$

$$M_{max} = 829{,}11 \quad [kNm] = 829{,}11 \cdot 10^6 \quad [Nmm]$$

$$V_{max} = 1/2 \cdot q \cdot L = 1/2 \cdot 66{,}33 \cdot 10$$

$$V_{max} = 331{,}65 \quad [kN]$$

Concrete

Reinforcement $N_s = \frac{M_d}{0{,}75*h} = \frac{829{,}11*10^6}{0{,}75*700} = 1{,}58 * 10^6\ [kN]$

$A_s = \frac{N_s}{f_s} = \frac{1{,}58*10^6}{435} = 3632\ [mm^2]$

$\rho = \frac{A_s}{b*d} * 100 = \frac{3632}{1415*10^2} * 100 = 2{,}57\ \%$

For beams 0,8% to 1,2% is an economic value, but in this case extra reinforcement is needed because of the length of the beam.

Bending moment $\frac{M_{ed}}{b*d^2} \leq 4000 = \frac{829{,}11*10^6}{1240*10^2*700} = 9{,}55 \leq 4000$

Shear force $v_{ed} = \frac{331{,}65*10^3}{1240*10^2} = 2{,}67\ [N]$

Deformation $E_{c,eff} = 0{,}33 * E_{cm}$

Walls

Precast concrete

Material: C30/37

$h = 5.000\ [mm]$

$L = 3750\ [mm]$

$b = 200\ [mm]$

$f_{cm} = 38\ \left[\frac{N}{mm^2}\right]$

$E_{cm} = 33.000\ \left[\frac{N}{mm^2}\right]$

$\rho_{rep} = 2.500\ \left[\frac{kg}{m^3}\right] = 25\ \left[\frac{kN}{m^3}\right]$

Weight table for the main beams on the first floor.

		Permanent load	Variable load	Source:
Insulation	Area = 5 m x 3.75 m	0,20 [kN/m^2] 3,75 [kN]	-	Quick reference – gl 4
Installation and piping	Area = 5 m x 3.75 m	0,50 [kN/m^2] 9,38 [kN]	-	-
Façade elements	Area = 5 m x 3.75 m	0,13 [kN/m^2] 2,44 [kN]	-	Quick reference – gl 4
Walls	Precast I = 5 m x 3,75 m x 0,2 m	25 [kN/m^3] 93,75 [kN]	-	-
Total		165,01 [kN]		

$\boxed{ULGT}$

FC1: Permanent Load

$$= \sum \gamma_{G,j} \cdot G_j + \sum \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_i$$

$$= \gamma_G \cdot G_{TOT} + \gamma_Q \cdot \psi_0 \cdot Q_{wze} + \gamma_Q \cdot \psi_0 \cdot Q_{wind} + \gamma_Q \cdot \psi_0 \cdot Q_{snow}$$

$$= 1{,}35 \cdot 165{,}01 + 0 + 0 + 0$$

$$Q_{per} = 222{,}76\ [kN]$$

$\boxed{BGT}$

$= \Sigma \gamma_{G,j} \cdot G_j + \gamma_Q \cdot Q_1 + \sum_{i>1} \gamma_{Qi} \cdot \psi_0 \cdot Q_i$

$= \gamma_G \cdot G_{TOT} + \gamma_Q \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{snow} + \gamma_Q \cdot \psi_0 \cdot Q_{wind}$

$= 1{,}0 \cdot 165{,}01 + 1{,}0 \cdot 1{,}0 \cdot 0{,}2 \cdot 3{,}75 + 0 + 0$

$Q_{Bend} = 165{,}76$ [kN]

$V_{max} = 222{,}76$ [kN]

$M_{max} = e \cdot V_{max} = 0{,}1 \cdot 222{,}76$

$M_{max} = 22{,}28$ [kNm]

Prefab concrete

Axial compressive force $A_c \geq \frac{N_{ed}}{f_{cd}} * 1{,}5 = 200 * 3750 \geq \frac{222{,}76*10^3}{38}$

$750.000 \geq 5860$ profile satisfies

Reinforcement $N_s = \frac{M_d}{0{,}75*h} = \frac{22{,}28*10^6}{0{,}75*200} = 148.533\ [N]$

$A_s = \frac{N_s}{f_s} = \frac{148.533}{435} = 341\ [mm^2]$

$\rho = \frac{A_s}{b*d} * 100 = \frac{341}{200*3750} * 100 = 0{,}05$ % profile satisfies

for beams 0,8% to 1,2%

Bending moment $\frac{22{,}28*10^6}{3750*200^2} \leq 4000 = 0{,}15 \leq 4000$ profile satisfies

Deformation $E_{c,eff} = 0{,}33 * E_{cm} = 0{,}33 * 33.000 = 10.890\ \left[\frac{N}{mm^2}\right]$

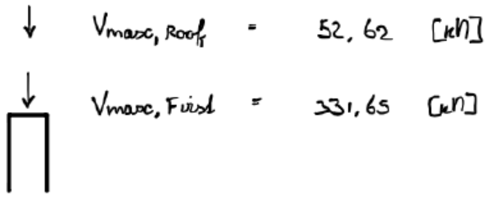
Column

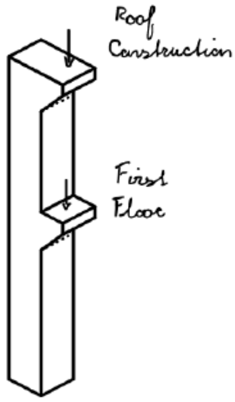
Concrete type: C30/37 F60

Profile 200 mm x 200 mm

H = 9300 mm

$E_{cm} = 33.000 \left[\frac{N}{mm^2}\right]$





Weight table for the columns

		Permanent load	Variable load	Source:
Roof construction		52,62 [kN]	-	-
First floor		331,65 [kN]	-	-
Walls		165,01 [kN]	-	-
Total		**549,28 [kN]**		

$M_{max} = e * F_{max} = 0{,}3 * 549{,}28$

$M_{max} = 109{,}86\ [kNm]$

Concrete

Prefab concrete

Axial compressive force $A_c \geq \frac{N_{ed}}{f_{cd}} * (1{,}0\ until\ 1{,}5)$ for prefab 1,5

$200^2 \geq \frac{549{,}85*10^3}{37} * 1{,}5$

$40 * 10^3 \geq 14{,}85 * 10^3$ profile satisfies

Reinforcement $N_s = \frac{M_d}{0{,}75*h} = \frac{109{,}86*10^6}{0{,}75*200} = 732.373\ [N]$

$A_s = \frac{N_s}{f_s} = \frac{732.3730}{435} = 1684\ [mm^2]$

$\rho = \frac{A_s}{b*d} * 100 = \frac{1684}{200^2} * 100 = 4{,}21\ \%$

For beams 0,8% to 1,2%, profile does not satisfy. Larger column needed.

Column profile: 400 mm x 400 mm

$M_{max} = e * F_{max} = 0{,}4 * 549{,}28$

$M_{max} = 219.71\ [kNm]$

Reinforcement $N_s = \frac{M_d}{0,75*h} = \frac{219,71*10^6}{0,75*400} = 732.373\ [N]$

$A_s = \frac{N_s}{f_s} = \frac{732.3730}{435} = 1684\ [mm^2]$

$\rho = \frac{A_s}{b*d} * 100 = \frac{1684}{400^2} * 100 = 1,05\ \%$ profile satisfies

Bending moment $\frac{M_{ed}}{b*d^2} \leq 4000$

$\frac{219,71*10^6}{400*400^2} = 3,42 \leq 4000$ profile satisfies

Shear force $v_{ed} = \frac{V_{Ed}}{b*d} = \frac{549,28*10^3}{400*400} = 3,43$

Deformation $E_{c,eff} = 0,33 * E_{cm} = 0,33 * 33.000$

$$E_{c,eff} = 10890\ \left[\frac{N}{mm^2}\right]$$

4.3 Design Alternative 3: “Two-Type”

Element	*Material*	*Profile*	*New profile*
		Art gallery (concrete)	
Roof	Precast arched concrete C30/37	h = 150 mm	-
Columns	Precast concrete C50/60	b = 450 mm h = 450 mm	-
Hollow slab floor	Dycore type T255	h = 255 mm	Dycore W150 h = 150 mm
Main beam first floor	Precast concrete I – beam	IC32/70	-
Walls	Precast concrete C30/37	b = 200 mm	-
		Office (timber)	
Roof	Cold formed corrugated sheet	h = 106 mm t = 1,00 mm	- -
Purlin	Timber GL24h	350 mm x 85 mm	-
Main Roof Beam	Timber GL24h	600 mm x 85 mm	600 mm x 185 mm
Hollow slab floor	Prefab concrete	Dycore type T255	Dycore type W150
Main beam first floor	Timber GL24h	600 mm x 85 mm	600 mm x 205 mm
Walls	Timber GL24h	b = 200 mm	-
Columns	Timber GL24h	650 mm x 205 mm	-

Roof

Concrete curved roof

Material: C30/37

$h = 150\ [mm]$

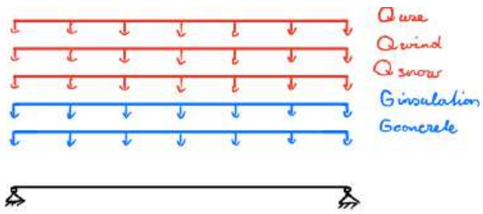
$L = 10.000\ [mm]$

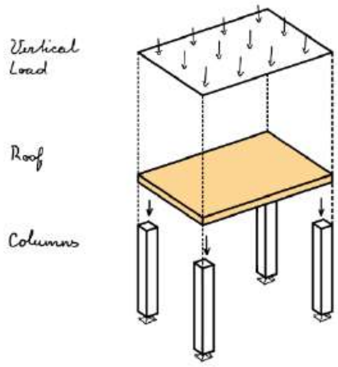
$b = 3.750\ [mm]$

$f_{cm} = 38\ \left[\frac{N}{mm^2}\right]$

$E_{cm} = 33.000\ \left[\frac{N}{mm^2}\right]$

$\rho_{rep} = 2.500\ \left[\frac{kg}{m^3}\right] = 25\ \left[\frac{kN}{m^3}\right]$





Weight table for the roof.

Element	Detail	Permanent load	Variable load	Source:
Concrete	In – situ 150 mm	3,75 [kN/m^2]	-	Quick reference – cb 2
Insulation		0,20 [kN/m^2]	-	Quick reference – gl 4
Use		-	1,00 [kN/m^2]	Quick reference – gl 5
Wind		-	0,46 [kN/m^2]	Chapter 3
Snow		-	0,56 [kN/m^2]	Chapter 3
Total		**3,95 [kN/m2]**		

$\boxed{UGT}$

FC1: Permanent Load

$$= \sum \gamma_{G,j} \cdot G_j + \sum \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_i$$

$$= \gamma_G \cdot G_{TOT} + \gamma_Q \cdot \psi_0 \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{wind} + \gamma_Q \cdot \psi_0 \cdot Q_{snow}$$

$$= 1{,}35 \cdot 3{,}95 + 0 + 0 + 0$$

$$Q_{per} = 5{,}33 \quad [kN/m^2]$$

FC2: Variable load

$$= \sum \gamma_{G,j} \cdot G_j + \gamma_Q \cdot Q_1 + \sum_{i>1} \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_i$$

$$= \gamma_G \cdot G_{TOT} + \gamma_Q \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{snow} + \gamma_Q \cdot \psi_0 \cdot Q_{wind}$$

$$= 1{,}2 \cdot 3{,}95 + 1{,}5 \cdot 1{,}0 + 0 + 0$$

$$Q_{var} = 6{,}24 \quad [kN/m^2]$$

$\boxed{BGT}$

$= \Sigma \gamma_{G,j} \cdot G_j + \gamma_Q \cdot Q_1 + \sum_{i>1} \gamma_{Qi} \cdot \psi_0 \cdot Q_i$

$= \gamma_G \cdot G_{TOT} + \gamma_Q \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{snow} + \gamma_Q \cdot \psi_0 \cdot Q_{wind}$

$= 1{,}0 \cdot 3{,}95 + 1{,}0 \cdot 1{,}0 + 0 + 0$

$Q_{Bend} = 4{,}95$ [kN/m²]

$q = a_{columns} \cdot Q_{var} = 3{,}75 \cdot 6{,}24$

$q = 23{,}46$ [kN/m]

$M_{max} = 1/8 \cdot q \cdot L^2 = 1/8 \cdot 23{,}46 \cdot 10^2$

$M_{max} = 293{,}28$ [kNm] $= 293{,}28 \cdot 10^6$ [Nmm]

$V_{max} = 1/2 \cdot q \cdot L = 1/2 \cdot 23{,}46 \cdot 10$

$V_{max} = 117{,}3$ [kN]

Reinforcement

$$N_s = \frac{M_d}{0{,}75*h} = \frac{293{,}28*10^6}{0{,}75*150} = 2{,}61*10^6\ [N]$$

$$A_s = \frac{N_s}{f_s} = \frac{2{,}61*10^6}{435} = 11.093\ [mm^2]$$

$$\rho = \frac{A_s}{b*d}*100 = \frac{11.093}{3750*150}*100 = 1{,}97\ \%$$

for beams 0,8% to 1,2%

Bending moment

$$\frac{293{,}28*10^6}{3750*150^2} \leq 4000 = 3{,}48 \leq 4000$$

Deformation

$$E_{c,eff} = 0{,}33*E_{cm} = 0{,}33*33.000 = 10.890\ \left[\frac{N}{mm^2}\right]$$

Columns

Concrete columns

Material: C50/60

$h = 9.000\ [mm]$

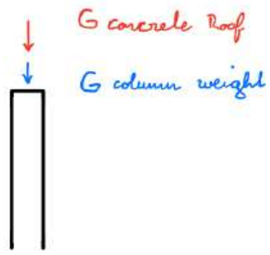
$L = 450\ [mm]$

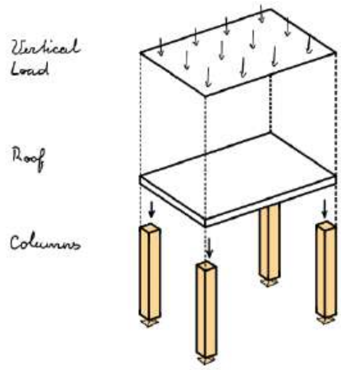
$b = 450\ [mm]$

$f_{cm} = 58\ \left[\frac{N}{mm^2}\right]$

$E_{cm} = 37.000\ \left[\frac{N}{mm^2}\right]$

$\rho_{rep} = 2.500\ \left[\frac{kg}{m^3}\right] = 25\ \left[\frac{kN}{m^3}\right]$





Weight table for the roof.

Element	Detail	Permanent load	Variable load	Source:
Concrete roof		117,3 [kN]	-	
Concrete	Precast	25 [kN/m³]	-	Quick reference – cb 2
Use		-	1,00 [kN/m²]	Quick reference – gl 5
Wind		-	0,46 [kN/m²]	Chapter 3
Snow		-	0,56 [kN/m²]	Chapter 3
Total		117,3 [kN] 25 [kN/m³]		

$I_{column} = L \times b \times h = 0{,}450 \times 0{,}450 \times 9{,}000$

$I_{column} = 1{,}8225\ [m^3]$

$F_{column} = I_{column} \cdot f_{concrete} = 1{,}8225 \cdot 25$

$F_{column} = 45{,}56\ [kN]$

$\boxed{UGT}$

FC1: Permanent Load

$= \Sigma\, \gamma_{G,j} \cdot G_j + \Sigma\, \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_i$

$= \gamma_G \cdot G_{TOT} + \gamma_Q \cdot \psi_0 \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{wind} + \gamma_Q \cdot \psi_0 \cdot Q_{snow}$

$= 1{,}35 \cdot\ , + 0 + 0 + 0$

$Q_{per} = 45{,}56\ [kN/m^2]$

BGT

$= \Sigma \gamma_{G,j} \cdot G_j + \gamma_Q \cdot Q_1 + \sum_{i>1} \gamma_{Q,i} \cdot \psi_0 \cdot Q_i$

$= \gamma_G \cdot G_{TOT} + \gamma_Q \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{snow} + \gamma_Q \cdot \psi_0 \cdot Q_{wind}$

$= 1{,}0 \cdot 45{,}56 + 1{,}0 \cdot 1{,}0 \cdot 0{,}45 \cdot 0{,}45 + 0 + 0$

$Q_{Bend} = 45{,}76 \quad [kN/m^2]$

$M_{max} = 1/8 \cdot q \cdot L^2 = 1/8 \cdot 23{,}46 \cdot 10^2$

$M_{max} = 293{,}28 \quad [kNm] = 293{,}28 \cdot 10^6 \quad [Nmm]$

$V_{max} = F_{Roof} + F_{column} = 117{,}3 + 45{,}56$

$V_{max} = 163{,}86 \quad [kN]$

Prefab concrete

Axial compressive force $A_c \geq \frac{N_{ed}}{f_{cd}} * 1{,}5 = 450 * 450 \geq \frac{163{,}86*10^3}{58}$

202.500 ≥ 2825 profile satisfies

Reinforcement $N_s = \frac{M_d}{0{,}75*h}$

$A_s = \frac{N_s}{f_s}$

$\rho = \frac{A_s}{b*d}$ for beams 0,8% to 1,2%

Bending moment $\frac{M_{ed}}{b*d^2} \leq 4000$

Deformation $E_{c,eff} = 0{,}33 * E_{cm} = 0{,}33 * 37.000 = 12.210 \left[\frac{N}{mm^2}\right]$

Floor slab first floor

Hollow slab floor:

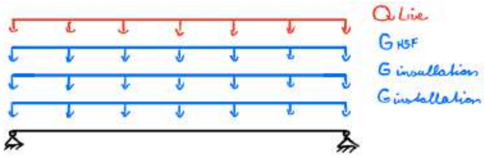
Material: Dycore type T

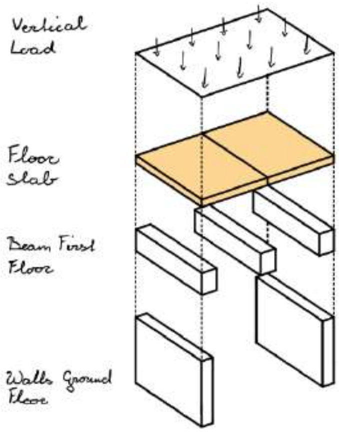
$L = 3750\ [mm]$

$d = 255\ [mm]$

$b = 1197\ [mm]$

$f_s = 235\ \left[\frac{N}{mm^2}\right]$





Weight table for the floor slabs on the first floor.

		Permanent load	Variable load	Source:
Hollow slab floor	Dycore type T: 255 mm	3,7 [kN/m^2]	-	Quick reference – gl 4
Insulation		0,20 [kN/m^2]	-	Quick reference – gl 4
Installation and piping		0,50 [kN/m^2]	-	-
Liveload (client)	Gallery C3: people walking around	-	7,5 [kN/m^2]	Chapter 3
Total		4,4 [kN/m2]		

$\boxed{UGT}$

FC1: Permanent Load

$$Q_{per} = \sum \gamma_{G,j} \cdot G_j + \sum \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_i$$

$$= \gamma_G \cdot G_{TOT} + \gamma_Q \cdot \psi_0 \cdot Q_{live} + \gamma_Q \cdot \psi_0 \cdot Q_{wind} + \gamma_Q \cdot \psi_0 \cdot Q_{snow}$$

$$= 1{,}35 \cdot 4{,}4 + 0 + 0 + 0$$

$$Q_{per} = 5{,}94 \quad [kN/m]$$

FC2: Variable load

$$Q_{var} = \sum \gamma_{G,j} \cdot G_j + \gamma_Q \cdot Q_1 + \sum_{i>1} \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_i$$

$$= \gamma_G \cdot G_{TOT} + \gamma_Q \cdot Q_{live} + \gamma_Q \cdot \psi_0 \cdot Q_{snow} + \gamma_Q \cdot \psi_0 \cdot Q_{wind}$$

$$= 1{,}2 \cdot 4{,}4 + 1{,}5 \cdot 7{,}5 \cdot 1{,}197 + 0 + 0$$

$$Q_{var} = 18{,}75 \quad [kN/m]$$

$\boxed{BGT}$

$$Q_{Bend} = \Sigma \gamma_{G,j} \cdot G_j + \gamma_Q \cdot Q_1 + \sum_{i>1} \gamma_{Q,i} \cdot \psi_0 \cdot Q_i$$

$$= \gamma_G \cdot G_{TOT} + \gamma_Q \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{snow} + \gamma_Q \cdot \psi_0 \cdot Q_{wind}$$

$$= 1{,}0 \cdot 4{,}4 + 1{,}0 \cdot 7{,}5 + 0 + 0$$

$$Q_{Bend} = 11{,}9 \quad [kN/m^2]$$

$$M_{max} = 1/8 \cdot q \cdot L^2 = 1/8 \cdot 18{,}75 \cdot 3{,}75^2$$

$$M_{max} = 32{,}95 \quad [kNm] = 32{,}95 \cdot 10^6 \quad [Nmm]$$

$$V_{max} = 1/2 \cdot q \cdot L = 1/2 \cdot 18{,}75 \cdot 3{,}75$$

$$V_{max} = 35{,}16 \quad [kN]$$

Concrete

Prefab concrete

Reinforcement

$$N_s = \frac{M_d}{0{,}75*h} = \frac{29{,}06*10^6}{0{,}75*255} = 151{,}95*10^3 \ [N]$$

$$A_s = \frac{N_s}{f_s} = \frac{151{,}95*10^3}{235} = 646{,}57 \ [mm^2]$$

$$\rho = \frac{A_s}{b*d} = \frac{646{,}57}{1197*255} = 0{,}002 \ [-]$$

for beams 0,8% to 1,2% economic value

Profile Dycore Type W150

Reinforcement

$$N_s = \frac{M_d}{0{,}75*h} = \frac{29{,}06*10^6}{0{,}75*150} = 258.311 \ [N]$$

$$A_s = \frac{N_s}{f_s} = \frac{258.311}{235} = 1099{,}20 \ [mm^2]$$

$$\rho = \frac{A_s}{b*d}*100 = \frac{1099{,}20}{1197*150}*100 = 0{,}612\ \%$$

Shear force

$$v_{ed} = \frac{V_{Ed}}{b*d}$$

Deformation

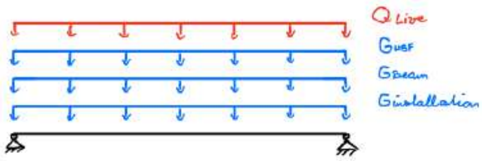
$$E_{c,eff} = 0{,}33*E_{cm}$$

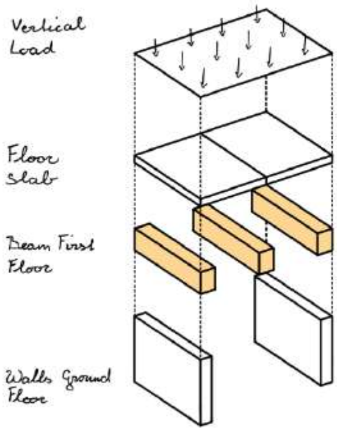
Main beams first floor

Main beams first floor:

Material: I – beam

$L = 10000\ [mm]$





Each beam will be loaded by a width of 3750 mm of hollow slab floor. Permanent distributed load in the table below have been accounted for this.

Weight table for the main beams on the first floor.

		Permanent load	Variable load	Source:
Hollow slab floor		4,4 [kN/m^2] 16,5 [kN/m]	-	-
I – beam		3,618 [kN/m]	-	Quick reference – gl 4
Liveload (client)	Gallery C3: people walking around	-	7,5 [kN/m^2]	Chapter 3
Total		**20,118 [kN/m]**		

$\boxed{\text{ULT}}$

FC1: Permanent Load

$$Q_{per} = \sum \gamma_{G,j} \cdot G_j + \sum \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_i$$

$$= \gamma_G \cdot G_{TOT} + \gamma_Q \cdot \psi_0 \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{wind} + \gamma_Q \cdot \psi_0 \cdot Q_{snow}$$

$$= 1{,}35 \cdot 20{,}118 + 0 + 0 + 0$$

$$Q_{per} = 27{,}159 \quad [kN/m]$$

FC2: Variable load

$$Q_{var} = \sum \gamma_{G,j} \cdot G_j + \gamma_Q \cdot Q_1 + \sum_{i>1} \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_i$$

$$= \gamma_G \cdot G_{TOT} + \gamma_Q \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{snow} + \gamma_Q \cdot \psi_0 \cdot Q_{wind}$$

$$= 1{,}2 \cdot 20{,}118 + 1{,}5 \cdot 7{,}5 \cdot 3{,}75 + 0 + 0$$

$$Q_{var} = 66{,}33 \quad [kN/m]$$

$\boxed{BGT}$

$$Q_{Bend} = \sum \gamma_{G,j} \cdot G_j + \gamma_Q \cdot Q_1 + \sum_{i>1} \gamma_{Q_i} \cdot \psi_0 \cdot Q_i$$

$$= \gamma_G \cdot G_{TOT} + \gamma_Q \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{snow} + \gamma_Q \cdot \psi_0 \cdot Q_{wind}$$

$$= 1{,}0 \cdot 20{,}118 + 1{,}0 \cdot 7{,}5 + 0 + 0$$

$$Q_{Bend} = 27.618 \quad [kN/m^2]$$

$$M_{max} = 1/8 \cdot q \cdot L^2 = 1/8 \cdot 66{,}33 \cdot 10^2$$

$$M_{max} = 829{,}11 \quad [kNm] = 829{,}11 \cdot 10^6 \quad [Nmm]$$

$$V_{max} = 1/2 \cdot q \cdot L = 1/2 \cdot 66{,}33 \cdot 10$$

$$V_{max} = 331{,}65 \quad [kN]$$

Concrete

Reinforcement $N_s = \frac{M_d}{0{,}75*h} = \frac{829{,}11*10^6}{0{,}75*700} = 1{,}58*10^6\ [kN]$

$A_s = \frac{N_s}{f_s} = \frac{1{,}58*10^6}{435} = 3632\ [mm^2]$

$\rho = \frac{A_s}{b*d}*100 = \frac{3632}{1415*10^2}*100 = 2{,}57\ \%$

For beams 0,8% to 1,2% is an economic value, but in this case extra reinforcement is needed because of the length of the beam.

Bending moment $\frac{M_{ed}}{b*d^2} \leq 4000 = \frac{829{,}11*10^6}{1240*10^2*700} = 9{,}55 \leq 4000$

Shear force $v_{ed} = \frac{331{,}65*10^3}{1240*10^2} = 2{,}67\ [N]$

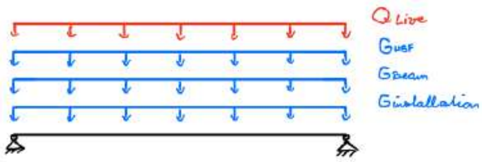
Deformation $E_{c,eff} = 0{,}33*E_{cm}$

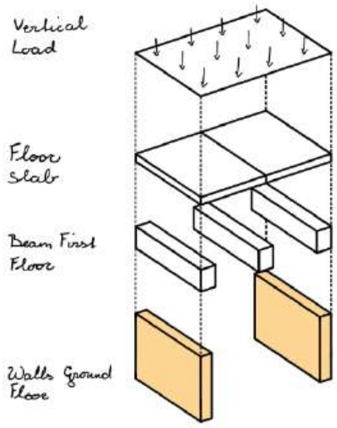
Walls

Precast concrete

Material: C30/37

$h = 5.000\ [mm]$





$L = 3750\ [mm]$

$b = 200\ [mm]$

$f_{cm} = 38\ \left[\frac{N}{mm^2}\right]$

$E_{cm} = 33.000\ \left[\frac{N}{mm^2}\right]$

$\rho_{rep} = 2.500\ \left[\frac{kg}{m^3}\right] = 25\ \left[\frac{kN}{m^3}\right]$

Weight table for the main beams on the first floor.

		Permanent load	Variable load	Source:
Insulation	Area = 5 m x 3.75 m	0,20 [kN/m^2] 3,75 [kN]	-	Quick reference – gl 4
Installation and piping	Area = 5 m x 3.75 m	0,50 [kN/m^2] 9,38 [kN]	-	-
Floor	Area = 5 m x 3,75 m	3,1 [kN/m^2] 58,13 [kN]	-	Quick reference – cd 14
Beam	I – beam C32,5/70	331,65 [kN]		-
Walls	Precast concrete Two stacked	25 [kN/m^3] 187,5 [kN]	-	Quick reference – cb 2
Total		**590,41 [kN]**		

$\boxed{\text{ULS}}$

FC1: Permanent Load

$$= \Sigma\, \gamma_{G,j} \cdot G_j + \Sigma\, \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_i$$

$$= \gamma_G \cdot G_{TOT} + \gamma_Q \cdot \psi_0 \cdot Q_{Live} + \gamma_Q \cdot \psi_0 \cdot Q_{wind} + \gamma_Q \cdot \psi_0 \cdot Q_{snow}$$

$$= 1{,}35 \cdot 590{,}41 + 0 + 0 + 0$$

$$Q_{per} = 797{,}05\ [kN]$$

BGT

$$= \Sigma \gamma_{G,j} \cdot G_j + \gamma_Q \cdot Q_1 + \sum_{i>1} \gamma_{Qi} \cdot \psi_0 \cdot Q_i$$

$$= \gamma_G \cdot G_{TOT} + \gamma_Q \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{snow} + \gamma_Q \cdot \psi_0 \cdot Q_{wind}$$

$$= 1,0 \cdot 390,41 + 1,0 \cdot 1,0 \cdot 0,2 \cdot 3,75 + 0 + 0$$

$$Q_{Bend} = 391,16 \quad [kN]$$

$$V_{max} = 797,05 \quad [kN]$$

$$M_{max} = e \cdot V_{max} = 0,1 \cdot 797,05$$

$$M_{max} = 79,71 \quad [kNm]$$

Prefab concrete

Axial compressive force $A_c \geq \frac{N_{ed}}{f_{cd}} * 1,5 = 200 * 3750 \geq \frac{797,01*10^3}{38}$

$750.000 \geq 20.975$ profile satisfies

Reinforcement $N_s = \frac{M_d}{0,75*h} = \frac{79,71*10^6}{0,75*200} = 531.400\ [N]$

$A_s = \frac{N_s}{f_s} = \frac{531.400}{435} = 1.222\ [mm^2]$

$\rho = \frac{A_s}{b*d} * 100 = \frac{1.222}{200*3750} * 100 = 0,16\ \%$ profile satisfies

for beams 0,8% to 1,2%

Bending moment $\frac{79,71*10^6}{3750*200^2} \leq 4000 = 0,53 \leq 4000$ profile satisfies

Deformation $E_{c,eff} = 0,33 * E_{cm} = 0,33 * 33.000 = 10.890\ \left[\frac{N}{mm^2}\right]$

Office

Roof

Cold formed corrugated sheet profile:

Material: steel S235

$h = 106\ [mm]$

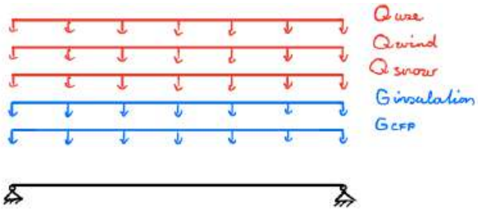
$L = 2000\ [mm]$

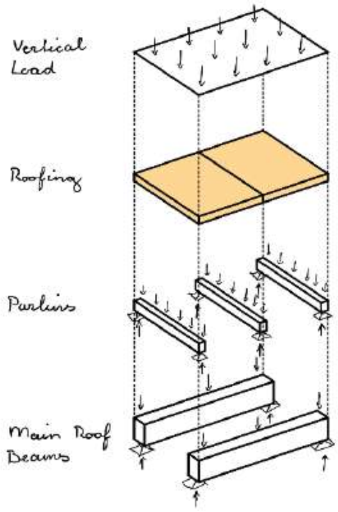
$b = 6000\ [mm]$

$t = 1{,}00\ [mm]$

$W = 31{,}8 * 10^3\ [mm^3]$

$f_{y;d} = 235\ \left[\frac{N}{mm^2}\right]$





Weight table for the roof.

Element	Detail	Permanent load	Variable load	Source:
Cold formed corrugated sheet		0,130 [kN/m^2]	-	Quick reference – sb 16
Insulation		0,20 [kN/m^2]	-	Quick reference – gl 4
Use		-	1,00 [kN/m^2]	Quick reference – gl 5
Wind		-	0,46 [kN/m^2]	Chapter 3
Snow		-	0,56 [kN/m^2]	Chapter 3
Total		**0,33 [kN/m2]**		

$\boxed{UGT}$

FC1: Permanent Load

$$= \Sigma\, \gamma_{G,j} \cdot G_j + \Sigma\, \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_i$$

$$= \gamma_G \cdot G_{TOT} + \gamma_Q \cdot \psi_0 \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{wind} + \gamma_Q \cdot \psi_0 \cdot Q_{snow}$$

$$= 1{,}35 \cdot 0{,}33 + 0 + 0 + 0$$

$$Q_{per} = 0{,}446 \quad [kN/m^2]$$

FC2: Variable load

$= \sum \gamma_{G,j} \cdot G_j + \gamma_Q \cdot Q_1 + \sum_{i>1} \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_i$

$= \gamma_G \cdot G_{TOT} + \gamma_Q \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{snow} + \gamma_Q \cdot \psi_0 \cdot Q_{wind}$

$= 1{,}2 \cdot 0{,}33 + 1{,}5 \cdot 1{,}0 + 0 + 0$

$Q_{var} = 1{,}896$ [kN/m²]

$\boxed{BGT}$

$= \sum \gamma_{G,j} \cdot G_j + \gamma_Q \cdot Q_1 + \sum_{i>1} \gamma_{Qi} \cdot \psi_0 \cdot Q_i$

$= \gamma_G \cdot G_{TOT} + \gamma_Q \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{snow} + \gamma_Q \cdot \psi_0 \cdot Q_{wind}$

$= 1{,}0 \cdot 0{,}33 + 1{,}0 \cdot 1{,}0 + 0 + 0$

$Q_{Bend} = 1{,}33$ [kN/m²]

$q = L_{purlins} \cdot Q_{var} = 3{,}75 \cdot 1{,}896$

$q = 7{,}11$ [kN/m]

$M_{max} = 1/8 \cdot q \cdot L^2 = 1/8 \cdot 7{,}11 \cdot 2^2$

$M_{max} = 3{,}555$ [kNm] $= 3{,}555 \cdot 10^6$ [Nmm]

$V_{max} = 1/2 \cdot q \cdot L = 1/2 \cdot 7{,}11 \cdot 2$

$V_{max} = 7{,}11$ [kN]

Bending $\frac{M_d}{W*\omega_{latbuc}} \leq f_{y;d}$ for $\omega_{buc} = 1{,}0$

$\frac{3{,}555*10^6}{31{,}8*10^3*1{,}0} \leq 235 = 111{,}79 \leq 235$ profile satisfies

Shear $\tau_d = \frac{V_d}{A_{web}} \leq \frac{f_{y;d}}{\sqrt{3}}$

$\tau_d = \frac{7{,}11*10^3}{1{,}00*6000} \leq \frac{235}{\sqrt{3}} = 1{,}185 \leq 135{,}68$ profile satisfies

Purlins

Purlins profile:

Material: laminated timber GL24h

$h = 350\ [mm]$

$L = 6000\ [mm]$

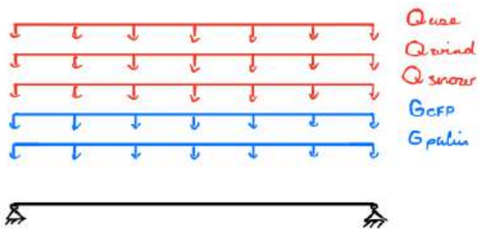
$b = 85\ [mm]$

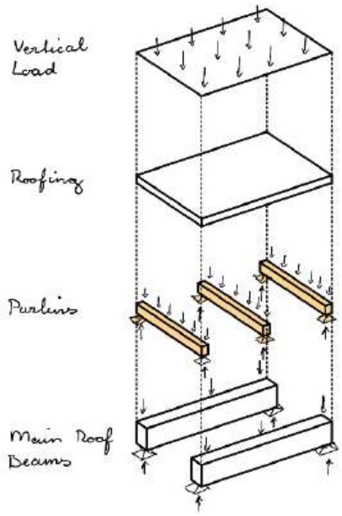
$W = 1{,}73 * 10^6\ [mm^3]$

$f_{m,0,d} = 24 * 0{,}7 = 16{,}8\ \left[\frac{N}{mm^2}\right]$

$f_{v,d} = 2{,}7\ \left[\frac{N}{mm^2}\right]$

$A = 29{,}7 * 10^3\ [mm^2]$





Each purlin will be loaded with the area roof load of 2000 mm wide. For the permanent weight in the table below the load over area has been multiplied by 2 meter.

Weight table for the purlins.

		Permanent load	Variable load	Source:
Roof		0,33 [kN/m^2] 0,66 [kN/m]	-	-
Purlins	GL24h 350 mm x 85 mm	0,125 [kN/m]	-	Quick reference – gl 4
Installation and piping		0,50 [kN/m^2] 1,00 [kN/m]	-	-
Category H: roofs	Roof angle $0 \leq \alpha \leq 15$	-	1,0 [kN/m^2] 2,0 [kN/m]	Quick reference – gl 5
Wind		-	[kN/m^2]	Chapter 3
Snow		-	0,56 [kN/m^2]	Chapter 3
Total		**1,785 [kN/m]**		

$\boxed{ULT}$

FC1: Permanent Load

$$Q_{per} = \Sigma \gamma_{G,j} \cdot G_j + \Sigma \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_i$$

$$= \gamma_G \cdot G_{TOT} + \gamma_Q \cdot \psi_0 \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{wind} + \gamma_Q \cdot \psi_0 \cdot Q_{snow}$$

$$= 1{,}35 \cdot 1{,}785 + 0 + 0 + 0$$

$$Q_{per} = 2{,}410 \quad [kN/m]$$

FC2: Variable load

$$Q_{var} = \sum \gamma_{G,j} \cdot G_j + \gamma_Q \cdot Q_1 + \sum_{i>1} \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_i$$

$$= \gamma_G \cdot G_{TOT} + \gamma_Q \cdot Q_{wze} + \gamma_Q \cdot \psi_0 \cdot Q_{snow} + \gamma_Q \cdot \psi_0 \cdot Q_{wind}$$

$$= 1{,}2 \cdot 1{,}785 + 1{,}5 \cdot 2{,}0 + 0 + 0$$

$$Q_{var} = 5{,}142 \quad [kN/m]$$

$\boxed{BGT}$

$$Q_{Bend} = \sum \gamma_{G,j} \cdot G_j + \gamma_Q \cdot Q_1 + \sum_{i>1} \gamma_{Q,i} \cdot \psi_0 \cdot Q_i$$

$$= \gamma_G \cdot G_{TOT} + \gamma_Q \cdot Q_{wze} + \gamma_Q \cdot \psi_0 \cdot Q_{snow} + \gamma_Q \cdot \psi_0 \cdot Q_{wind}$$

$$= 1{,}0 \cdot 1{,}785 + 1{,}0 \cdot 2{,}0 + 0 + 0$$

$$Q_{Bend} = 3{,}785 \quad [kN/m^2]$$

$$M_{max} = 1/8 \cdot q \cdot L^2 = 1/8 \cdot 5{,}142 \cdot 6^2$$

$$M_{max} = 23{,}139 \quad [kNm] = 23{,}139 \cdot 10^6 \quad [Nmm]$$

$$V_{max} = 1/2 \cdot q \cdot L = 1/2 \cdot 5{,}142 \cdot 6$$

$$V_{max} = 15{,}426 \quad [kN]$$

Bending

$\frac{M_d}{W*\omega_{latbuc}} \leq f_{m,0,d}$ where $\omega_{latbuc} = 1{,}0$

$\frac{23{,}139*10^6}{1{,}73*10^6*1{,}0} \leq 16{,}8 = 13{,}38 \leq 16{,}8$ profile satisfies

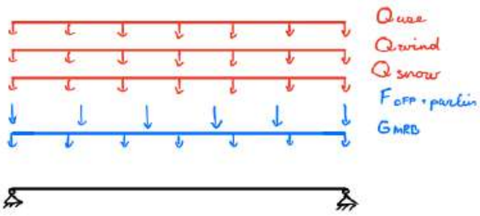
Shear

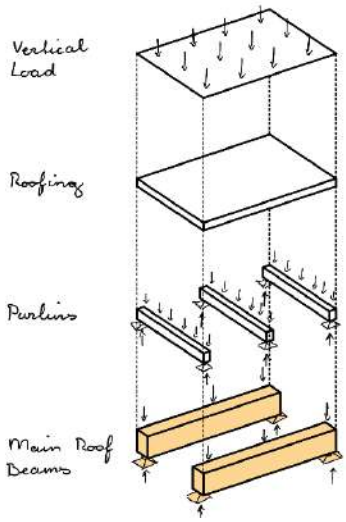
$\tau_d = \frac{3*V_d}{2*A_{web}} \leq f_{v;d}$

$\tau_d = \frac{3*15{,}426*10^3}{2*29{,}7*10^3} \leq 2{,}7 = 0{,}78 \leq 2{,}7$ profile satisfies

Main roof beams

Main roof beams profile:





Material: laminated timber GL24h

$h = 600\ [mm]$

$L = 10.000\ [mm]$

$b = 85\ [mm]$

$W = 5{,}10 * 10^6\ [mm^3]$

$f_{m,0,d} = 24 * 0{,}7 = 16{,}8\ \left[\frac{N}{mm^2}\right]$

$f_{v,d} = 2{,}7\ \left[\frac{N}{mm^2}\right]$

$A = 51{,}0 * 10^3\ [mm^2]$

Weight table for the main roof beams.

		Permanent load	Variable load	Source:
Roof Purlins	Point load	10,926 [kN]	-	-
Main roof beams	GL24h 600 mm x 85 mm	0,214 [kN/m]	-	Quick reference – gl 4
Category H: roofs	Roof angle $0 \le \alpha \le 15$	-	1,0 [kN/m^2]	Quick reference – gl 5
Wind		-	[kN/m^2]	Chapter 3
Snow		-	0,56 [kN/m^2]	Chapter 3
Total		**10,926 [kN] 0,214 [kN/m]**		

$\boxed{UGT}$

FC1: Permanent Load

$$Q_{per} = \sum \gamma_{G,j} \cdot G_j + \sum \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_i$$

$$= \gamma_G \cdot G_{MRB} + \gamma_Q \cdot \psi_0 \cdot Q_{wze} + \gamma_Q \cdot \psi_0 \cdot Q_{wind} + \gamma_Q \cdot \psi_0 \cdot Q_{snow}$$

$$= 1{,}35 \cdot 0{,}214 + 0 + 0 + 0$$

$$Q_{per} = 0{,}289\ [kN/m]$$

$$F_{per} = \gamma_G \cdot G_{CFP+purlins} + \gamma_Q \cdot \psi_0 \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{wind} + \gamma_Q \cdot \psi_0 \cdot Q_{snow}$$

$$= 1{,}35 \cdot 10{,}926 + 0 + 0 + 0$$

$$F_{per} = 14{,}75 \quad [kN]$$

FC2: Variable load

$$Q_{var} = \Sigma \gamma_{G,j} \cdot G_j + \gamma_Q \cdot Q_1 + \sum_{i>1} \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_i$$

$$= \gamma_G \cdot G_{MRB} + \gamma_Q \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{snow} + \gamma_Q \cdot \psi_0 \cdot Q_{wind}$$

$$= 1{,}2 \cdot 0{,}214 + 1{,}5 \cdot 1{,}0 + 0 + 0$$

$$Q_{var} = 1{,}757 \quad [kN/m]$$

$$F_{var} = \gamma_G \cdot G_{CFP+purlins} + \gamma_Q \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{snow} + \gamma_Q \cdot \psi_0 \cdot Q_{wind}$$

$$= 1{,}2 \cdot 10{,}926 + 1{,}5 \cdot 1{,}0 + 0 + 0$$

$$F_{per} = 14{,}611 \quad [kN]$$

$\boxed{BGT}$

$$Q_{Bend} = \Sigma \gamma_{G,j} \cdot G_j + \gamma_Q \cdot Q_1 + \sum_{i>1} \gamma_{Q i} \cdot \psi_0 \cdot Q_i$$

$$= \gamma_G \cdot G_{MRB} + \gamma_Q \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{snow} + \gamma_Q \cdot \psi_0 \cdot Q_{wind}$$

$$= 1{,}0 \cdot 0{,}214 + 1{,}0 \cdot 1{,}0 + 0 + 0$$

$$Q_{Bend} = 1{,}214 \quad [kN/m^2]$$

$$F_{Bend} = \gamma_G \cdot G_{CFP+purl} + \gamma_Q \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{snow} + \gamma_Q \cdot \psi_0 \cdot Q_{wind}$$

$$= 1{,}0 \cdot 10{,}926 + 1{,}0 \cdot 1{,}0 + 0 + 0$$

$$F_{Bend} = 11{,}926 \quad [kN]$$

$$M_{max} = \tfrac{1}{8} \cdot q \cdot L^2 + F\left(\tfrac{L}{2}\right) + F\left(\tfrac{L}{2} - 2\right) + F\left(\tfrac{L}{2} - 4\right)$$

$$= \tfrac{1}{8} \cdot 1{,}757 \cdot 10^2 + 14{,}611 \cdot \left(\tfrac{10}{2}\right) + 14{,}611 \left(\tfrac{10}{2} - 2\right) + 14{,}611 \left(\tfrac{10}{2} - 4\right)$$

$$M_{max} = 153{,}46 \quad [kNm] \quad = \quad 153{,}46 \cdot 10^6 \quad [Nmm]$$

$$V_{max} = \tfrac{1}{2} \left(q \cdot L + 6 \cdot F\right)$$

$$V_{max} = \tfrac{1}{2} \left(1{,}757 \cdot 10 + 6 \cdot 14{,}611\right) = 52{,}62 \quad [kN]$$

Bending

$\frac{M_d}{W*\omega_{latbuc}} \leq f_{m,0,d}$ where $\omega_{latbuc} = 1{,}0$

$\frac{153{,}46*10^6}{5{,}10*10^6*1{,}0} = \leq 16{,}8 = 30{,}09 > 16{,}8$ larger profile needed

Profile GL24h 600 mm x 185 mm

$\frac{153{,}46*10^6}{11{,}10*10^6*1{,}0} = \leq 16{,}8 = 13{,}83 \leq 16{,}8$ profile satisfies

Shear

$\tau_d = \frac{3*V_d}{2*A_{web}} \leq f_{v;d}$

$\tau_d = \frac{3*152{,}62*10^3}{2*111{,}0*10^3} \leq 2{,}06 =\leq 2{,}7$ profile satisfies

Quick reference timber design 2: straight beam (possibly with precamber)

Glulam 600 mm x 185 mm (h x b) profile

Profile properties:

Area (face)	$A = 111{,}0 * 10^3$	[mm^2]
Density gl24h	$\rho_{mean} = 420$	[kg/m^3]
Weight	$G = A * \rho_{mean} = 111{,}0 * 10^3 * 10^{-6} * 420 * 10$	
	$G = 0{,}466$	[kN/m]

Floor slab first floor

Hollow slab floor:

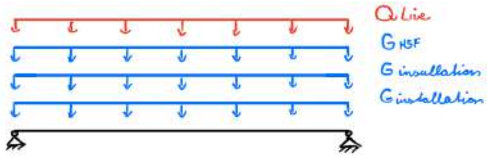
Material: Dycore type T

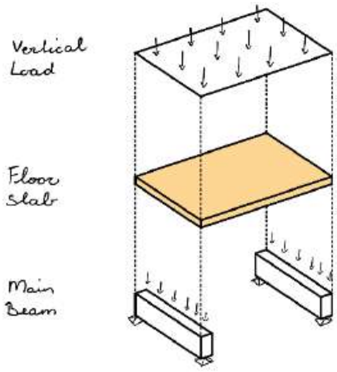
$L = 3750\ [mm]$

$d = 255\ [mm]$

$b = 1197\ [mm]$

$f_s = 235\ \left[\frac{N}{mm^2}\right]$





Weight table for the floor slabs on the first floor.

		Permanent load	Variable load	Source:
Hollow slab floor	Dycore type T: 255 mm	3,7 [kN/m^2]	-	Quick reference – gl 4
Insulation		0,20 [kN/m^2]	-	Quick reference – gl 4
Installation and piping		0,50 [kN/m^2]	-	-
Liveload (client)	Gallery C3: people walking around	-	7,5 [kN/m^2]	Chapter 3
Total		**4,4 [kN/m2]**		

$\boxed{UGT}$

FC1: Permanent Load

$$Q_{per} = \sum \gamma_{G,j} \cdot G_j + \sum \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_i$$

$$= \gamma_G \cdot G_{TOT} + \gamma_Q \cdot \psi_0 \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{wind} + \gamma_Q \cdot \psi_0 \cdot Q_{snow}$$

$$= 1{,}35 \cdot 4{,}4 + 0 + 0 + 0$$

$$Q_{per} = 5{,}94 \quad [kN/m]$$

FC2: Variable load

$$Q_{var} = \sum \gamma_{G,j} \cdot G_j + \gamma_Q \cdot Q_1 + \sum_{i>1} \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_i$$

$$= \gamma_G \cdot G_{TOT} + \gamma_Q \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{snow} + \gamma_Q \cdot \psi_0 \cdot Q_{wind}$$

$$= 1{,}2 \cdot 4{,}4 + 1{,}5 \cdot 7{,}5 \cdot 1{,}197 + 0 + 0$$

$$Q_{var} = 18{,}75 \quad [kN/m]$$

BGT

$$Q_{Bend} = \Sigma \gamma_{G,j} \cdot G_j + \gamma_Q \cdot Q_1 + \sum_{i>1} \gamma_{Qi} \cdot \psi_0 \cdot Q_i$$

$$= \gamma_G \cdot G_{TOT} + \gamma_Q \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{snow} + \gamma_Q \cdot \psi_0 \cdot Q_{wind}$$

$$= 1{,}0 \cdot 4{,}4 + 1{,}0 \cdot 7{,}5 \cdot + 0 + 0$$

$$Q_{Bend} = 11{,}9 \quad [kN/m^2]$$

$$M_{max} = 1/8 \cdot q \cdot L^2 = 1/8 \cdot 18{,}75 \cdot 3{,}75^2$$

$$M_{max} = 32{,}95 \quad [kNm] = 32{,}95 \cdot 10^6 \quad [Nmm]$$

$$V_{max} = 1/2 \cdot q \cdot L = 1/2 \cdot 18{,}75 \cdot 3{,}75$$

$$V_{max} = 35{,}16 \quad [kN]$$

Concrete

Prefab concrete

Reinforcement

$$N_s = \frac{M_d}{0{,}75*h} = \frac{29{,}06*10^6}{0{,}75*255} = 151{,}95*10^3\ [N]$$

$$A_s = \frac{N_s}{f_s} = \frac{151{,}95*10^3}{235} = 646{,}57\ [mm^2]$$

$$\rho = \frac{A_s}{b*d}*100 = \frac{646{,}57}{1197*255}*100 = 0{,}2\ \%$$

for beams 0,8% to 1,2% economic value

Profile Dycore Type W150

Reinforcement

$$N_s = \frac{M_d}{0{,}75*h} = \frac{29{,}06*10^6}{0{,}75*150} = 258.311\ [N]$$

$$A_s = \frac{N_s}{f_s} = \frac{258.311}{235} = 1099{,}20\ [mm^2]$$

$$\rho = \frac{A_s}{b*d}*100 = \frac{1099{,}20}{1197*150}*100 = 0{,}612\ \%$$

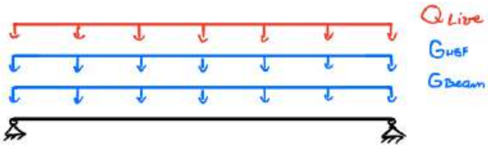
Shear force

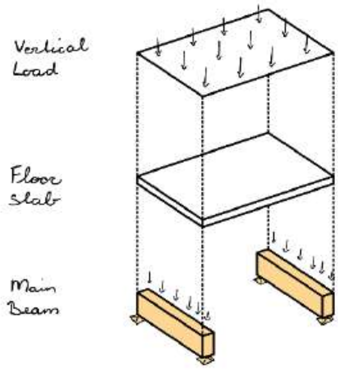
$$v_{ed} = \frac{V_{Ed}}{b*d}$$

Deformation

$$E_{c,eff} = 0{,}33*E_{cm}$$

Main beams first floor





Material: laminated timber GL24h

$h = 600\ [mm]$

$L = 10.000\ [mm]$

$b = 85\ [mm]$

$W = 5{,}10 * 10^6\ [mm^3]$

$f_{m,0,d} = 24 * 0{,}7 = 16{,}8\ \left[\frac{N}{mm^2}\right]$

$f_{v,d} = 2{,}7\ \left[\frac{N}{mm^2}\right]$

$A = 51{,}0 * 10^3\ [mm^2]$

Weight table for the main beams on the first floor.

		Permanent load	Variable load	Source:
Hollow slab floor		4,4 [kN/m²] 16,5 [kN/m]	-	-
Main roof beams	GL24h 600 mm x 85 mm	0,214 [kN/m]	-	-
Liveload (client)	Gallery C3: people walking around	-	7,5 [kN/m²]	-
Total		16,714 [kN/m]		

$\boxed{UGT}$

FC1: Permanent Load

$$Q_{per} = \Sigma\, \gamma_{G,j} \cdot G_j + \Sigma\, \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_i$$

$$= \gamma_G \cdot G_{TOT} + \gamma_Q \cdot \psi_0 \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{wind} + \gamma_Q \cdot \psi_0 \cdot Q_{snow}$$

$$= 1{,}35 \cdot 16{,}714 + 0 + 0 + 0$$

$$Q_{per} = 22{,}56 \quad [kN/m]$$

FC2: Variable load

$$Q_{var} = \Sigma\, \gamma_{G,j} \cdot G_j + \gamma_Q \cdot Q_1 + \sum_{i>1} \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_i$$

$$= \gamma_G \cdot G_{TOT} + \gamma_Q \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{snow} + \gamma_Q \cdot \psi_0 \cdot Q_{wind}$$

$$= 1{,}2 \cdot 16{,}714 + 1{,}5 \cdot 7{,}5 \cdot 3{,}75 + 0 + 0$$

$$Q_{var} = 62{,}24 \quad [kN/m]$$

$\boxed{BGT}$

$$Q_{Bend} = \Sigma \gamma_{G;j} \cdot G_j + \gamma_Q \cdot Q_1 + \sum_{i>1} \gamma_{Qi} \cdot \psi_0 \cdot Q_i$$

$$= \gamma_G \cdot G_{TOT} + \gamma_Q \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{snow} + \gamma_Q \cdot \psi_0 \cdot Q_{wind}$$

$$= 1{,}0 \cdot 16{,}714 + 1{,}0 \cdot 7{,}5 \cdot 3{,}75 + 0 + 0$$

$$Q_{Bend} = 44{,}84 \quad [kN/m^2]$$

$$M_{max} = 1/8 \cdot q \cdot L^2 = 1/8 \cdot 62{,}24 \cdot 5^2$$

$$M_{max} = 194{,}5 \quad [kNm] = 194{,}5 \cdot 10^6 \quad [Nmm]$$

$$V_{max} = 1/2 \cdot q \cdot L = 1/2 \cdot 62{,}24 \cdot 5$$

$$V_{max} = 155{,}6 \quad [kN]$$

Bending

$\frac{M_d}{W * \omega_{latbuc}} \leq f_{m,0,d}$ where $\omega_{latbuc} = 1{,}0$

$\frac{194{,}5*10^6}{5{,}10*10^6*1{,}0} = \leq 16{,}8 = 38{,}14 > 16{,}8$ larger profile needed

Profile GL24h 600 mm x 205 mm

$\frac{194{,}5*10^6}{12{,}30*10^6*1{,}0} = \leq 16{,}8 = 15{,}81 \leq 16{,}8$ profile satisfies

Shear

$\tau_d = \frac{3*V_d}{2*A_{web}} \leq f_{v;d}$

$\tau_d = \frac{3*155{,}6*10^3}{2*123{,}0*10^3} \leq 1{,}90 = \leq 2{,}7$ profile satisfies

Quick reference timber design 2: straight beam (possibly with precamber)

Glulam 600 mm x 205 mm (h x b) profile

Profile properties:

Area (face)	$A = 123{,}0 * 10^3$	[mm²]
Density gl24h	$\rho_{mean} = 420$	[kg/m³]
Weight	$G = A * \rho_{mean} = 123{,}0 * 10^3 * 10^{-6} * 420 * 10$	
	$G = 0{,}517$	[kN/m]

Walls

Material: laminated timber GL24h

$h = 5000\ [mm]$

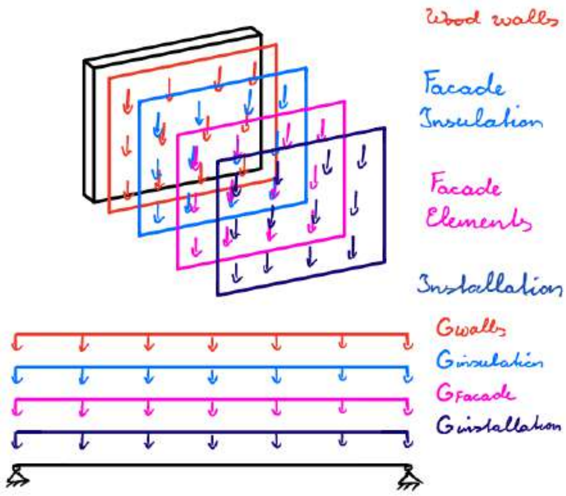
$L = 3750\ [mm]$

$b = 200\ [mm]$

$f_{m,0,d} = 24 * 0{,}7 = 16{,}8\ \left[\frac{N}{mm^2}\right]$

$f_{v,d} = 2{,}7\ \left[\frac{N}{mm^2}\right]$

$A = 375.000 * 10^3\ [mm^2]$



Weight table for the walls.

		Permanent load	Variable load	Source:
Insulation		0,05 [kN/m²]	-	Quick reference – gl 4
Façade elements		0,13 [kN/m²]	-	Quick reference – gl 4
Installation		0,50 [kN/m²]	-	-
Walls timber	***420*5*10/1000	21 [kN/m²]	-	-
Total		21,68 [kN/m²]		

$\boxed{UGT}$

FC1: Permanent Load

$$Q_{per} = \Sigma\, \gamma_{G,j} \cdot G_j + \Sigma\, \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_i$$

$$= \gamma_G \cdot G_{TOT} + \gamma_Q \cdot \psi_0 \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{wind} + \gamma_Q \cdot \psi_0 \cdot Q_{snow}$$

$$= 1{,}35 \cdot 21{,}68 + 0 + 0 + 0$$

$$Q_{per} = 29{,}27\ [kN/m^2]$$

FC2: Variable load

$$Q_{var} = \Sigma\, \gamma_{G,j} \cdot G_j + \gamma_Q \cdot Q_1 + \sum_{i>1} \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_i$$

$$= \gamma_G \cdot G_{TOT} + \gamma_Q \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{snow} + \gamma_Q \cdot \psi_0 \cdot Q_{wind}$$

$$= 1{,}2 \cdot 21{,}86 + 1{,}5 \cdot 0 + 0 + 0$$

$$Q_{var} = 26{,}02\ [kN/m^2]$$

$\boxed{BGT}$

$$Q_{Bend} = \sum \gamma_{G,j} \cdot G_j + \gamma_Q \cdot Q_1 + \sum_{i>1} \gamma_{Q_i} \cdot \psi_0 \cdot Q_i$$

$$= \gamma_G \cdot G_{TOT} + \gamma_Q \cdot Q_{use} + \gamma_Q \cdot \psi_0 \cdot Q_{snow} + \gamma_Q \cdot \psi_0 \cdot Q_{wind}$$

$$= 1{,}0 \cdot 21{,}68 + 1{,}0 \cdot 0 + 0 + 0$$

$$Q_{Bend} = 21{,}68 \quad [kN/m^2]$$

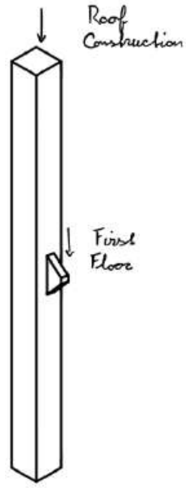
$$V_{max} = Q \cdot A = 29{,}27 \cdot (3{,}75 \cdot 0{,}20)$$

$$V_{max} = 21{,}95 \quad [kN]$$

Axial compressive force $\sigma_{c;0;d} = \frac{N_{Ed}}{A} \leq f_{c,0,d} * \omega_{buc}$

$\sigma_{c;0;d} = \frac{21{,}95*10^3}{750.000} \leq 16{,}8 * 1{,}0 = 0{,}03 \leq 16{,}8$ profile satisfies

Columns



Material: laminated timber GL24h

$h = 9300\ [mm]$

$L = 650\ [mm]$

$b = 205\ [mm]$

$W = 14{,}43 * 10^6\ [mm^3]$

$f_{m,0,d} = 24 * 0{,}7 = 16{,}8\ \left[\frac{N}{mm^2}\right]$

$f_{v,d} = 2{,}7\ \left[\frac{N}{mm^2}\right]$

$A = 133.200\ [mm^2]$

Weight table for the columns.

		Permanent load	Variable load	Source:
Roof construction		52,62 [kN]	-	-
First floor		155,6 [kN]	-	-
Walls		21,95 [kN]	-	-
Total		**230,17 [kN]**		

Axial compressive force $\sigma_{c;0;d} = \frac{N_{Ed}}{A} \leq f_{c,0,d} * \omega_{buc}$

$\sigma_{c;0;d} = \frac{230{,}17*10^3}{133.200} \leq 16{,}8 * 1{,}0 = 1{,}73 \leq 16{,}8$ profile satisfies

Bending

$\frac{M_d}{W*\omega_{latbuc}} \leq f_{m,0,d}$ where $\omega_{latbuc} = 1{,}0$

$\frac{230{,}17*10^6}{14{,}43*10^6*1{,}0} = \leq 16{,}8 = 15{,}95 \leq 16{,}8$ profile satisfies

4.4. Multi Criteria Analysis

To determine the best design we will make use of the multi criteria analysis, where different criteria get a weighted and scored. This way a conclusion can be made about the final design. Weighting of criteria is based on the reasoning provided further down the chapter.

Criteria that will be taken into consideration are:

- Cost
- Aesthetics
- Sustainability
- Manufacturability

Table 5: MCA of the three different designs, weighted by the criteria.

		Design 1 "Woody"		Design 2 "Bulk"		Design 3 "Two – Type"	
Criteria	*Weight*		*Score*		*Score*		*Score*
Cost	10	3	30	5	50	5	50
Manufacturability	8	4	24	5	24	1	8
Sustainability	7	5	35	2	14	3	21
Aesthetics	5	4	20	2	10	5	25
Total score			109		98		104

Cost

The main floor and foundation of the different design alternatives are considered to be the same to simplify calculations for material use. Material costs can be divided up into fabrication cost, transportation cost, assembling cost, maintenance & demolition cost. We will look at the material cost even though some of the material pricing may include labor cost.

Design alternative 1: "Woody"

Material	*Element*		*Amount*	*Cost*	*€*
Steel	Roof	h = 106 mm t = 1,00 mm	200 [m^2] 570 kg	3 [€/kg]	1.710
Timber	Purlin	350 mm x 85 mm x 3.750 mm	36	500 [€/m^3]	2.008
	Main roof beam	600 mm x 185 mm x 10.000 mm	9	500 [€/m^3]	4.995
	Main beam first floor	600 mm x 205 mm x 10.000 mm	9	500 [€/m^3]	5.535
	Walls	200 mm x 3.750 mm x 5.000 mm	28	500 [€/m^3]	52.500
	Columns	650 mm x 205 mm x 9.300 mm	18	500 [€/m^3]	11.153
Concrete	Hollow slab floor	Dycore type W150	200 [m^2]	45-55 [€/m^2] 10-15 [€/m^2] 40-60 [€/m^2]	26.000
Material cost (€)					103.901

Design alternative 2: “Bulk”

Material	*Element*	*Profile*	*Amount*	*Cost*	*€*
Steel	Roof	h = 106 mm t = 1,00 mm	200 [m^2] 570 kg	3 [€/kg]	1.710
Timber	Purlin	350 mm x 85 mm x 3.750 mm	36	70 [€/m^3]	2.008
	Main roof beam	600 mm x 185 mm x 10.000 mm	9	70 [€/m^3]	4.995
Concrete	Hollow slab floor	Dycore W150	200 [m^2]	45-55 [€/m^2] 10-15 [€/m^2] 40-60 [€/m^2]	26.000
	Main beam first floor	I – beam C32,5/70 141.500 mm^2 x 10.000 mm	9	300 [€/m^3] 120 [€/m^3]	5.349
	Walls	200 mm x 3.750 mm x 5.000 mm	28	200 [€/m^3] 120 [€/m^3]	33.600
	Columns	400 mm x 400 mm x 9.300 mm	18	300 [€/m^3] 120 [€/m^3]	11.249
Material cost (€)					84.911

Design alternative 2: “Two – Type”

Material	*Element*	*Profile*	*Amount*	*Cost*	*€*
		Art gallery (concrete)			
Steel					
Timber					
Concrete	Roof	150 mm	120*10^6 mm^2	300 [€/m^3] 120 [€/m^3]	7.560
	Columns	400 mm x 400 mm x 9.300 mm	10	300 [€/m^3] 120 [€/m^3]	6.250
	Hollow slab floor	Dycore W150	120 [m^2]	45-55 [€/m^2] 10-15 [€/m^2] 40-60 [€/m^2]	15.600
	Main beam first floor	I – beam C32,5/70 141.500 mm^2 x 10.000 mm	5	300 [€/m^3] 120 [€/m^3]	2.972
	Walls	200 mm x 3.750 mm x 5.000 mm	16	200 [€/m^3] 120 [€/m^3]	19.200
		Office (timber)			
Steel	Roof	h = 106 mm t = 1,00 mm	80 [m^2] 228 kg	3 [€/kg]	684
Timber	Purlin	350 mm x 85 mm x 3.750 mm	15	500 [€/m^3]	837
	Main roof beam	600 mm x 185 mm x 10.000 mm	4	500 [€/m^3]	2.220
Concrete	Hollow slab floor	Dycore W150	80 [m^2]	45-55 [€/m^2] 10-15 [€/m^2] 40-60 [€/m^2]	10.400
	Main beam first floor	I – beam C32,5/70	4	300 [€/m^3] 120 [€/m^3]	2.377

		141.500 mm^2 x 10.000 mm			
	Walls	200 mm x 3.750 mm x 5.000 mm	12	200 [€/m^3] 120 [€/m^3]	14.400
	Columns	400 mm x 400 mm x 9.300 mm	8	300 [€/m^3] 120 [€/m^3]	5.000
Material cost (€)					81.500

Sustainability

Timber has a smaller impact on the environment due to the fact that it is a sustainable material that can be grown again. Even though the shadow costs of laminated timber (€0,6495/kg) is actually higher than that of concrete (€0,0075/kg) with reinforcement (€0,24711/kg), timber is still more favorable when it comes to sustainability due to the fact that the timber elements can be reused and recycled more easily.

Manufacturability

The most difficult to manufacture is design 3 due to the many different types of materials and profiles used. This makes manufacturing more difficult because of the transportation/logistics and on site labor.

Design 2 and 3 mostly use prefab of laminated timber and concrete. This makes on site labor easier, since this is done at the manufacturing plant. The difference between design 1 and 2 will be in the detail, constructive connections between elements. For timber this will take more consideration into where to place connections and if the elements can resist the local load. There is also notably more experience in the industry working with prefab concrete instead of laminated timber for larger constructions.

Aesthetics

The construction will be located in a newly developed area around new and modern buildings. The aesthetic appeal of the building is of importance, especially because of the art gallery. Design 3 is the most interesting aesthetically due to the distinctive use of material between the office building and art gallery. Timber is aesthetically favored over concrete and steel because of the current importance of sustainability. By using timber the construction will have an sustainable appeal.

4.5. Detail of "Woody" construction

Details of the timber construction connections are seen below.

Connection between timber column and construction bracing. Steel plate connects the timber column and steel bracing by steel pins.

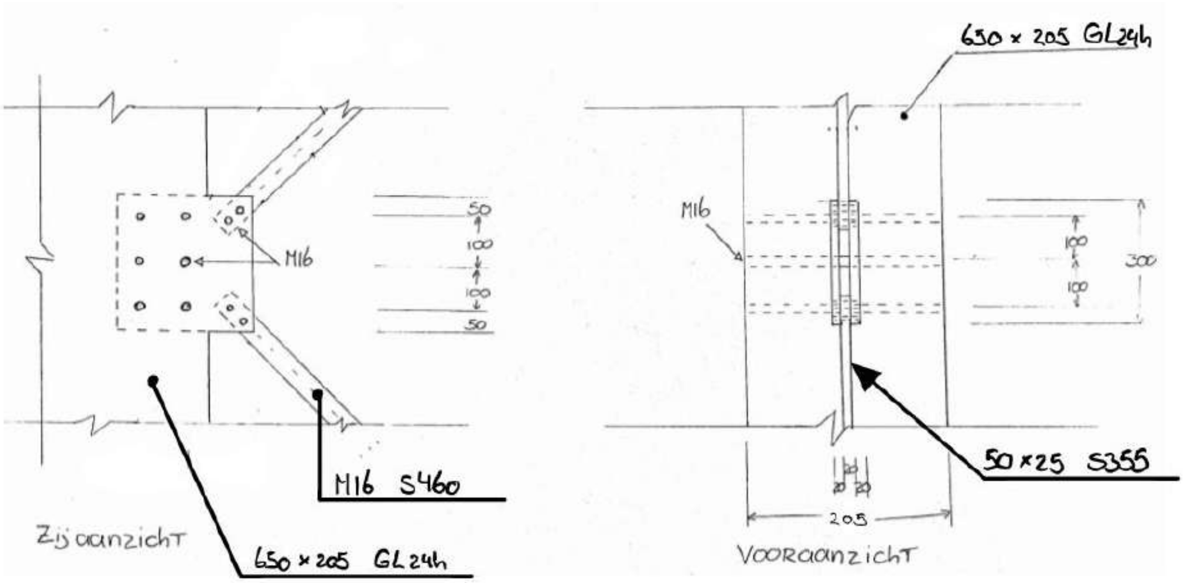


Figure 17: Connection between timber column and bracing

Connection between column and main beams.

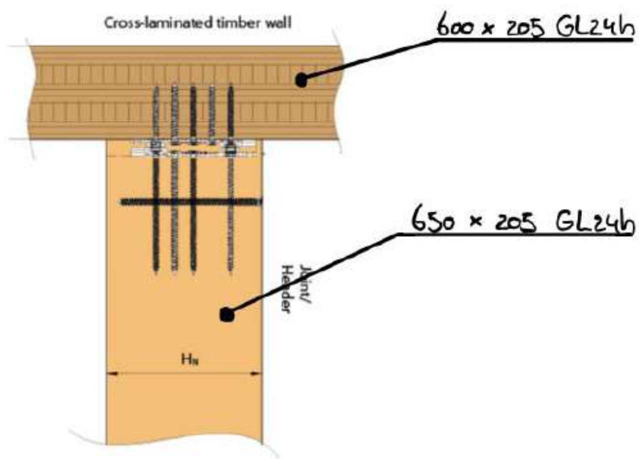


Figure 18: Connection between column and main beams

5. Glass Structure

For the part of the building to be constructed with glass, the thermally reinforced float glass will be used hereby imploring the use of a DSF system to increase the energy efficiency of the building structure. The float glass will be used because of its hardness and also due to the fact that it is a great insulator and also has low expansion. More so, this type of glass is also used because of its ability to withstand scratches. In addition, because this system consists of three components namely the exterior wall, interior wall and ventilated cavity it helps to protect the building against the weather. For the building structure, the glass part will have an outer layer of 6mm, the void and the inner layer which will also serve for the actual load bearing.

5.1 Calculation of the Glass Structure

For the determination of the sizes of the elements in the glass structure the loads, dimensions and materials are factors which needed to be put into consideration. The dimension of the glass structure is 4mx6m.e. the glass structure has a width of 4meters and a depth of 6meters. It is also important to know that the ground floor is about 4meters and the first floor 5meters. The type of glass to be used is the isolated thermally reinforced float glass. The loads which are present on the glass structure include the snow, wind, live and self-load.

Load type	kN/m2
Windload	0.46
Snowload	1.12
Liveload	1.5
Self load	where rho = 24.5kN/m3 (NEN 2608)

5.1.1 Factors for the variable loads:

We chose a factor of 1.5 for all variable loads except for the roof were have to use 1,27 since this has a lower consequence class.
The factors for determination of the glass strength (NEN 2608 8.3.1)

$$f_{mt;u;d} = \frac{k_a \times k_e \times k_{mod} \times k_{sp} \times f_{g;k}}{\gamma_{m;A}}$$

Ka = 1.0 (lineair calculation with a non-concentrated load NEN 2608 8.3.3 (5))
Ksp = 1.0 (floatglass NEN 2608 8.3.3)
Ke = 1.0 for wind load 0.8 for vertical load (thermically hardened floatglass NEN 2608 8.3.3(2))
Fg;k = 45 N/mm^2
Kmod = 0.44 for snow loads,
1 for wind load
0.29 for self-weight, floors and the variable load from people.
γm;A = 1.6 where the wind is the determining factor and 1.8 for all other situations.

5.1.2 Calculation of Glass Structure

A thickness of 0.14 m of the first floor

ULS Calculation (it is important to know that the width of the floor is 1m)

Qfloor =	Qlive*1.5 + 1.2*rho*hfloor;1.35*hfloor*rho)=11.6 kN/m
Md =	1/8 * Qfloor * Lfloor2 = 23.2 kNm
Wfloor =	1/6 * hfloor2 = 0.003 m^2
M/w =	7114.4 kN/m^2
f =	45000/1.8*0.29 = 7250

SLS Calculation

Qfloor =	Qlive + rho*hfloor = 8.4 kN/m
I =	1/12 * hfloor3 = 0.00023 m^4
w =	5/384 * Qfloor * Lfloor4/(EI) = 0.0018 m
umax =	sqrt(40002+60002)/65 = 110 mm

A thickness for the ground floor wall of 0.02 m(i.e. wall 1)

ULS Calculation

Calculation in the vertical direction	
F =	F = Qroof * 6 * 4 / 2 + dwall2 * Lwall2 * Bwall2 + dwall1 * Lwall1* Bwall1= 55.9 kN
N =	N = F / (dwall * Bwall) = 699.0 kN/m^2
f =	f = 45000 / 1.8 * 0.44 * 0.8 = 8800 kN/m^2

Calculation in the horizontal direction	
Md =	1/8 * Lwall * (Qwind*1.5)2 = 2.1 kNm
Wwall =	1/6 * Bwall * dwall^2 = 0.0002 m^3
M/w	10350 kN/m^2
f =	45000/1.6 = 28125 kN/m^2

SLS Calculation

Iwall =	1/12 * Bwall * dwall^3 = 2.7e-6 m4
wwall =	5/384 * Qwind * Lwall^4/(E*I) = 0.00821 m = 8.2 mm
umax =	Lwall / 200 = 0.02 m = 20 mm

A thickness of 0.02 meters of the first floor wall (i.e. wall 2)

ULS Calculation

Calculation in the vertical direction	
F =	Qroof * 6 * 4 / 2 + 0.5 * dwall2 * Lwall2 * Bwall2 = 43.2 kN
N =	F / (dwall2 * Bwall2) = 539.5 kN/m2
F =	45000 / 1.8 * 0.44 * 0.8 = 8800 kN/m2

Calculation in the horizontal direction	
Md =	1/8 * Lwall2 * (Qwind*1.5)2 = 3.2 kNm
Wwall1 =	1/6 * Bwall2 * dwall2 = 0.0002 m3
M/w =	16171.9 kN/m2
f =	45000/1.6 = 28125 kN/m2

SLS Calculation

Length of wall2 =	1/12 * Bwall2 * dwall1^3 = 2.7e-6 m4
width of wall2 =	5/384 * Qwind * Lwall1^4/(E*I) = 0.0201m = 20.1mm
umax =	Lwall2 / 200 = 0.025m = 25mm

Thickness of Roof : 60mm = 0.06m

ULS Calculation

Qroof	1.12*1.27+1.2*rho*h;1.35*rho*h= 3.2 kN/m
Md	1/8 * Qroof * Lspan2= 6.4 kNm (Lspan is 4 meters)
Weight of roof	Wroof = 1/6 * 1 * h2 = 0.0006 m3
f	45000/1.8 * 0.44 = 11000 kN/m2

SLS Calculation

Qroof	1,12+rho*h = 2.9 kN/m
length	1/12 * 1 * h3 = 0.000018 m4
w	5/384 * Qroof * Lspan4/(E * I) = 0.0076 m = 7.6 mm
umax	sqrt(40002+60002)/65 = 110 mm

6. Calculation of loads on foundation

6.1 Live loads

Apart from the loads on the building itself, the dead and live loads on the foundation were also determined considering the part of the office and art gallery;
Live loads

6.1.1 Art gallery

For the art gallery, in order to calculate the live load on the column, the snow loads and live loads are
considered. The live load on the art gallery was determined by taking the center to center distance for beams i.e. 5m, the overhanging length 2m, and the floor loads on the art gallery which is 10kN. The snow load was determined by taking into consideration the snow load of the art gallery and the central part as earlier stated i.e. 1.12 kN/m² and multiplying it with the distance in-between every column which is 3,75m.Thus, to determine the live load on the column in the art gallery the live load and the snow load is summed up and multiplied by 1.5 which is a safety factor taken into consideration.

Live load on art gallery	(5kN/m^2 * 5 * 3.75 *2) + 10kN = 103.75kN
Snowload	1.12kN/m^2 * 5m * 3.75m = 21kN
Live load on column	**103.75 + 21 * 1.5 = 187.125kN**

6.1.2 Office

In order to calculate the live load on the office column, the center to center distance for the beam is
considered i.e. 5m, the length of the outer beam as 10m and the overhanging length as 2m. Thence
giving a total live load on the office as 192.375kN.

Center to center distance for beams (office)	5m
Overhanging length (office)	2m
Length of outer beam (office)	10m
Load on the office roof	0.7 kN/m^2
Load on the office floor	5 kN/m^2
Load on the office column	**((2.5+2)*10*(1.5*0.7+1.5*5))/2 = 192.375kN**

6.2 Dead load

6.2.1 Art gallery

In order to determine the dead load on the art gallery, the weight of the roof, wooden beam and the
weight of the first floor are taken into consideration as that of the office. The weight of the beam and
column in the art gallery is calculated considering the weight per meter. Since the choice of the wooden beam made has a length of 5m, this is used to determine the weight of the beam, while on the other hand the weight of the column is determined by taking the height of both floors giving a total of 9m. Furthermore, the live load which is earlier calculated as 187.1kN is added to these weights in order to give the total dead load on the foundation of the art gallery:

Weight of roof	0.5kN/m^2
Wooden beam	0.49 * 5 = 2.45kN

Weight of first floor (floor thickness = 150mm)	2.7 kN/m^2
Weight of column	1.38 * 9 = 12.4kN
Weight of beam	2kN/m * 5m = 10kN
Weight of ground floor	2.7 kN/m^2
Total dead load	**187.1+(0.5+2.7)*5*3.75*1.2+12.4*1.2+10*1.1 = 285kN**

6.2.2 Office

Having determined the live loads on the office, it was also essential to determine the dead loads which includes the weight and density of timber and concrete to be used for the building structure and the roof weight taken as 0.5kN/m². Considering that columns and beams on the first floor will be made of timber because of its lightweight and esthetical properties, while all the floors will be made out of precast concrete, the dimensions of the wooden beam, column and precast concrete are determined
as follows and altogether used to calculate the column load on foundation which is 249.60kN, with a beam load of 48.384kN per meter. An explicit method of how the calculation is done is shown below:

Weight of roof	0.5kN/m^2
Dimensions of wooden beams	110*700mm
Density of wooden beams	650kg/m3
Weight of wooden beam per meter	650*9.81*0.11*0.7 = 0.490kN/m
Weight of wooden beam	10*0.490 = 4.9kN
Height of the office	5m
Weight of wooden column	5*0.490 = 2.45kN
Dimension of concrete	200*300
Density of concrete	235kN/m^3
Weight per meter	0.2*0.3*23 = 1.38kN/m
Weight concrete beam	1.38*10 = 13.8kN
Weight of concrete column	1.38 * 4 = 5.52kN
Self-weight of first floor	2.7kN/m^2
Load self-weight of first floor	(2.5+2)*10)*(2.7))/2 = 60.75kN
Load on beam per m	4.5*(5*1.5+2.71*1.2) = 48.384kN
Total column load on foundation	**(192.375)+(4.9*1.2*0.5)+(2.45*1.3)+(13.8*1.2/2)+(60.75*1.2/2) = 249.60kN**

6.3 Pile plan

With the given loads from the previous paragraph, a suitable pile profile can be chosen from the following table from chapter 3.2:

Table 2. Load bearing capacities various prefab foundation piles. Values presented are compressive forces. Capacity for tensile forces is 20% of the capacity of compressive forces.

	220*220 mm^2	250*250 mm^2	320*320 mm^2
L=18 m	$F_{R;d}$=290 kN	$F_{R;d}$ =380 kN	$F_{R;d}$ = 620 kN
L=20 m	too slender	$F_{R;d}$ =560 kN	$F_{R;d}$ = 920 kN
L=24 m	too slender	$F_{R;d}$ =750 kN	$F_{R;d}$ = 1228 kN

For the depth of the piles the results from the soil research can be used as guideline. The lower sand layers starts at 19 meters. The pile must therefor be at least 20 meters deep.

The total loads per column on the art gallery and office side are respectively 473 kN and 442 kN. For the foundation, the 20 meter long 250x250 mm^2 prefab piles are chosen as the design capacity can hold the given loads with a great margin.

The pile plan is provided in Appendix F: Sketch plan of foundation piles.

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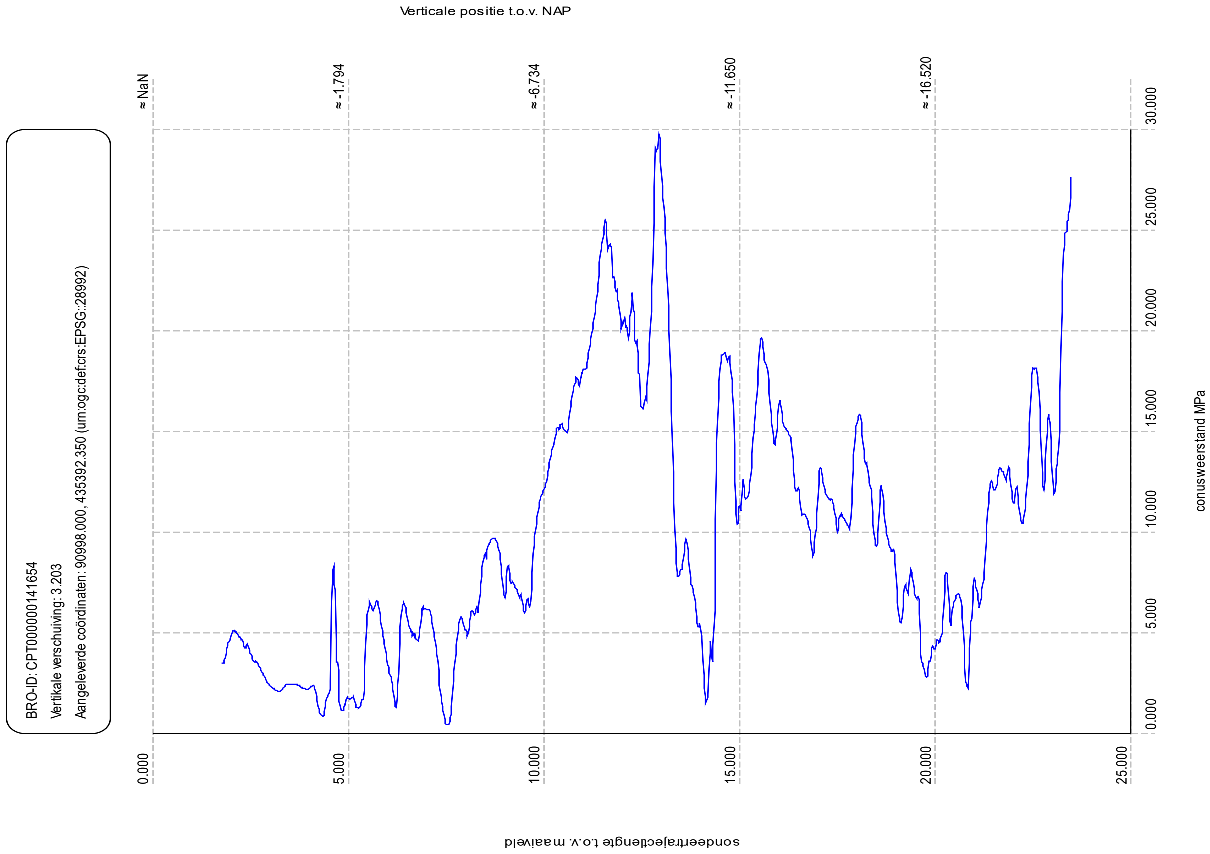
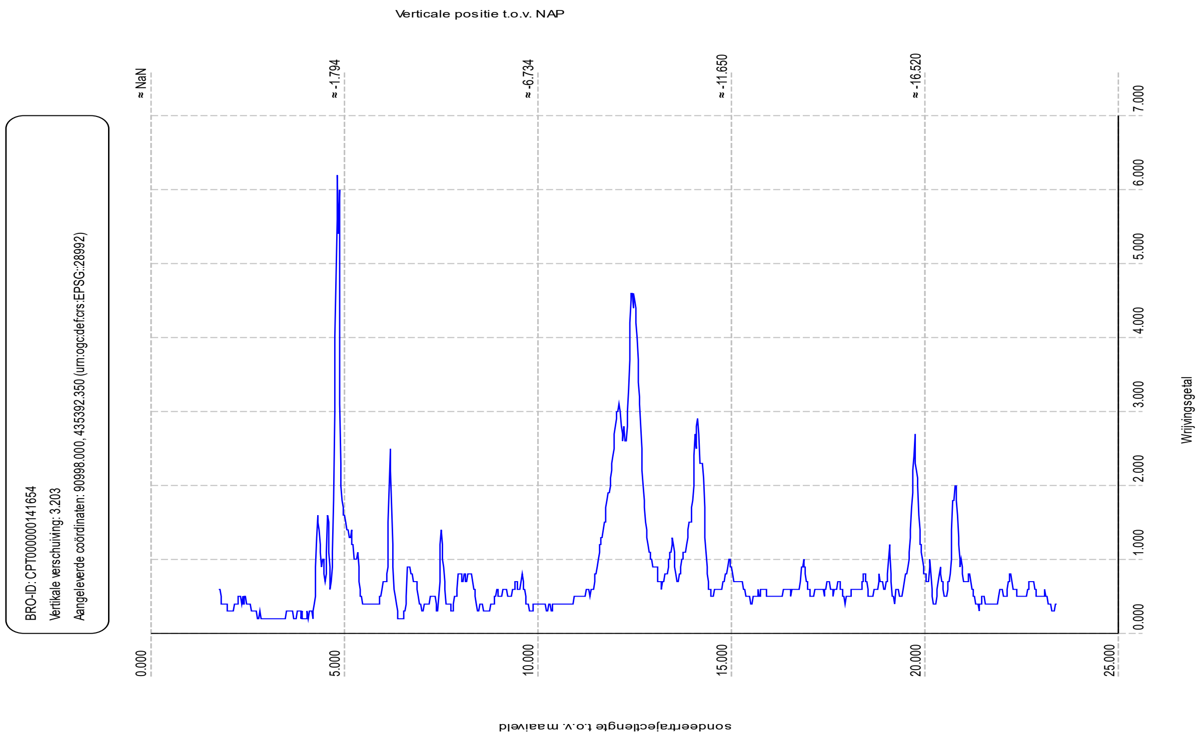
Appendices

Appendix A : Depth of soil layers (Dinoloket, n.d.).





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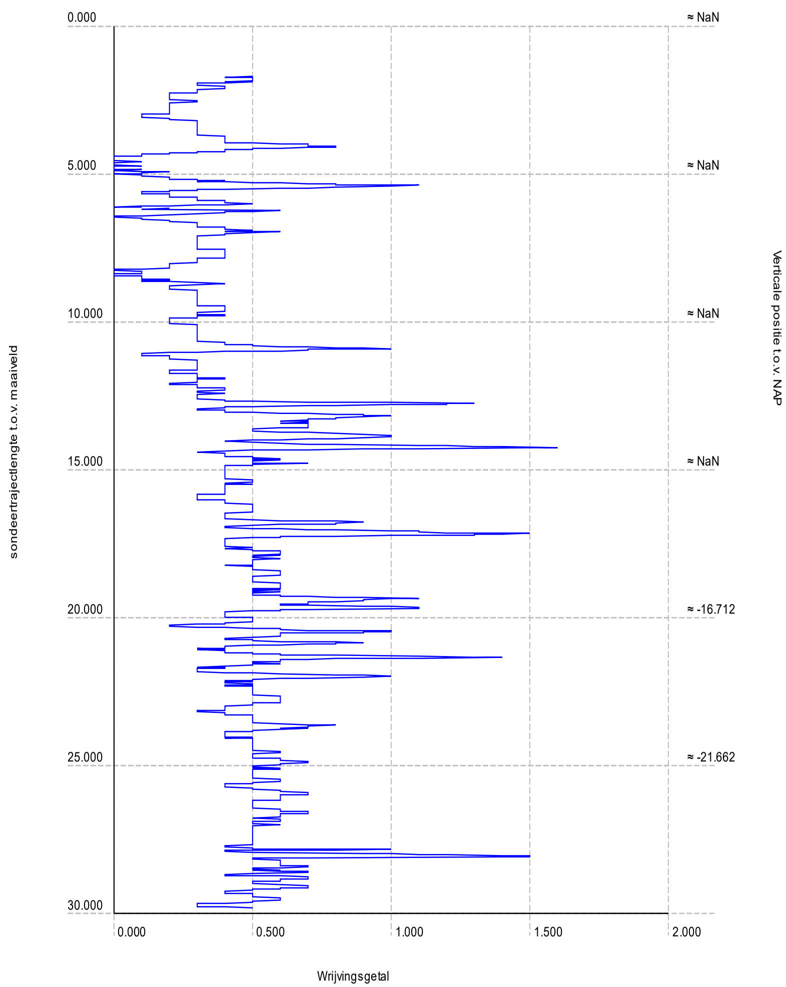

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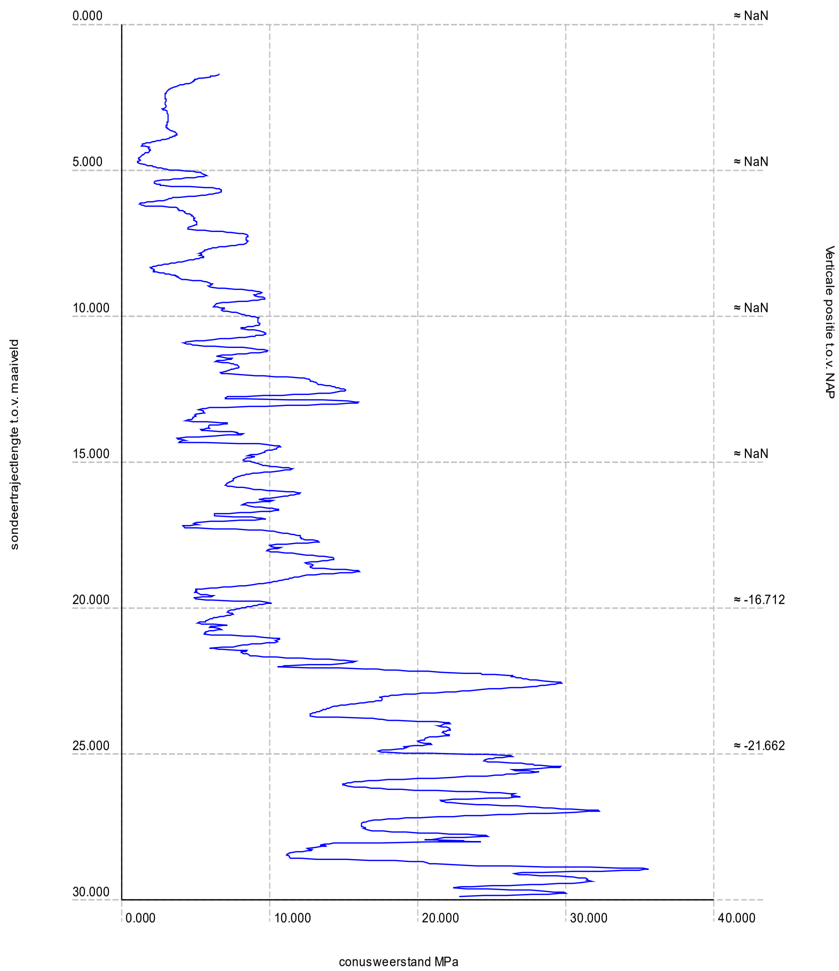

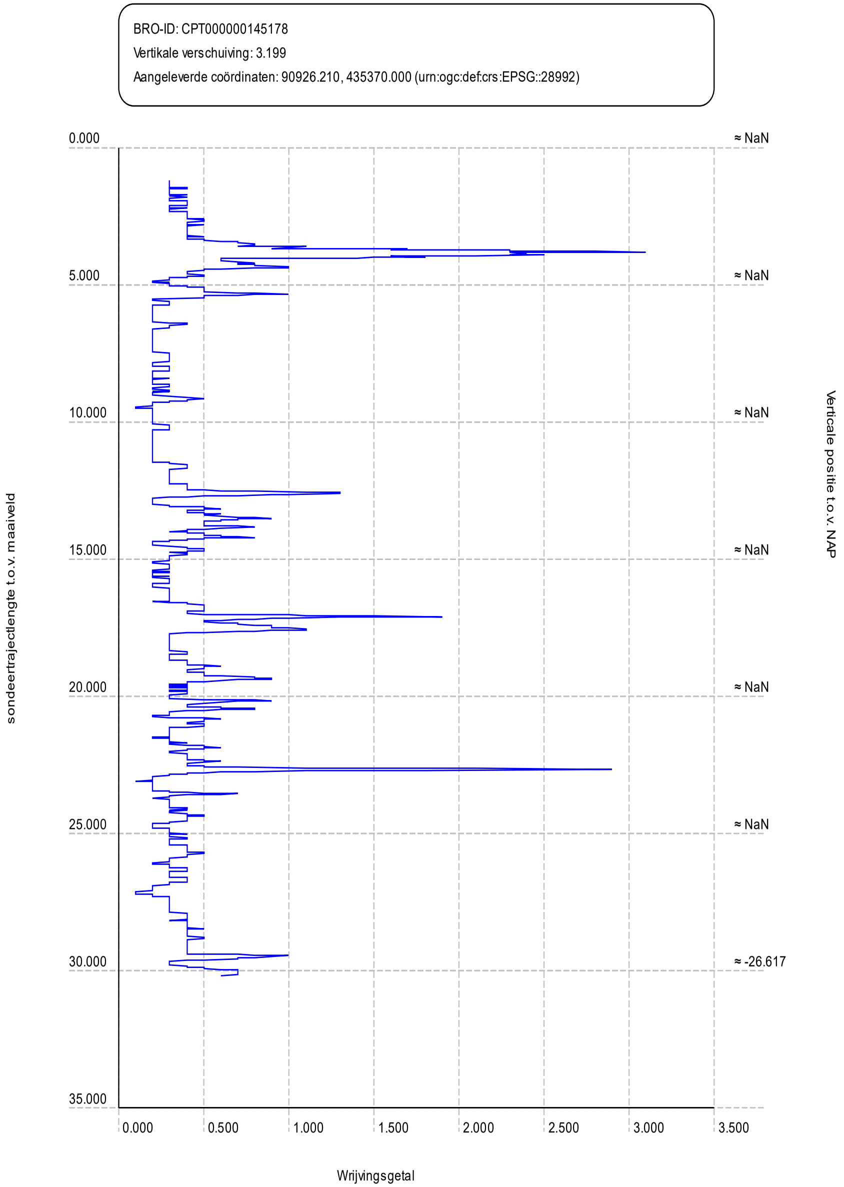
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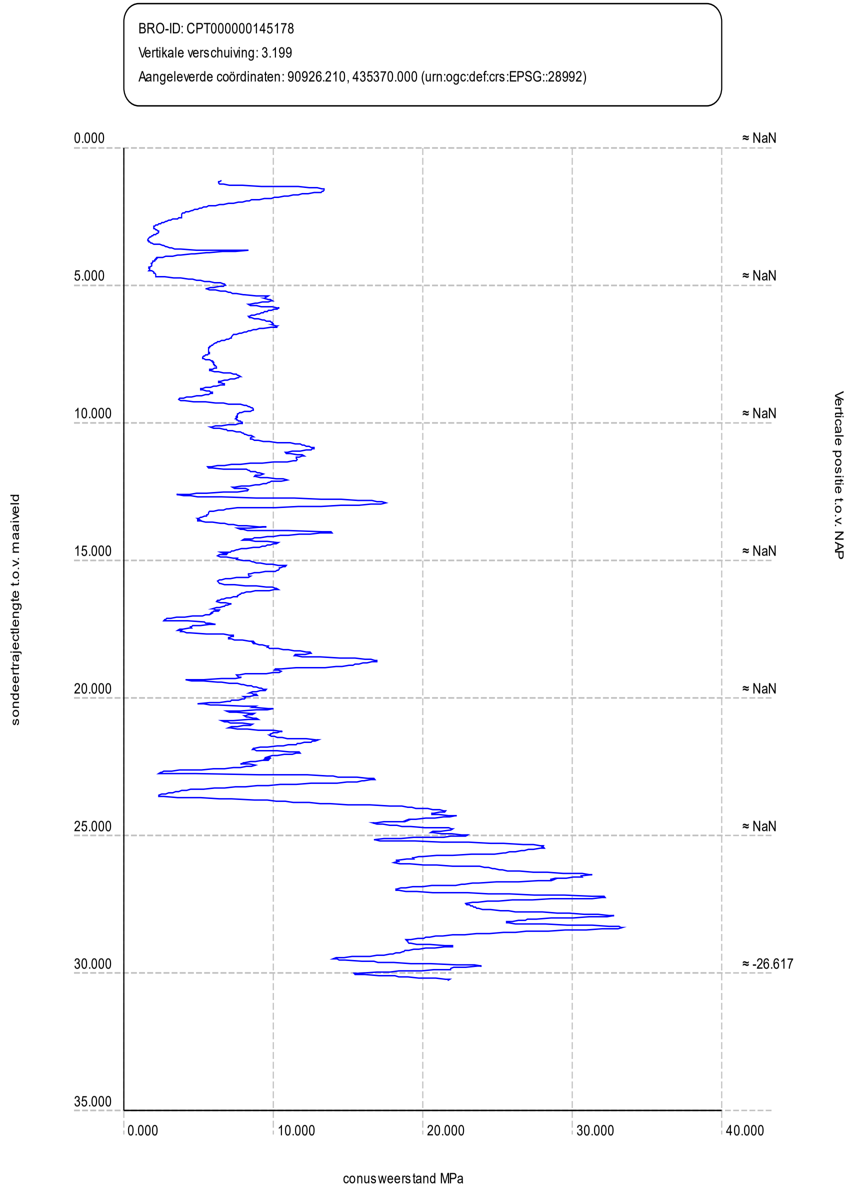
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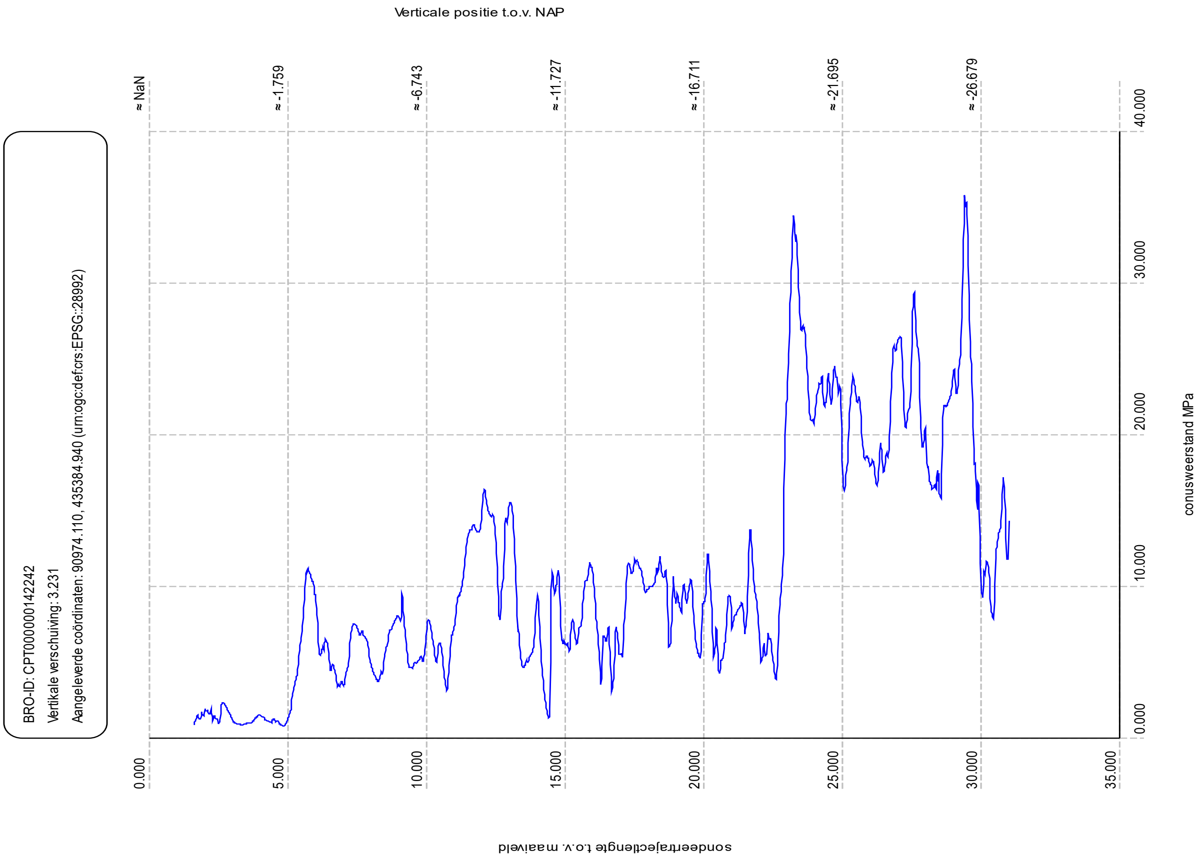
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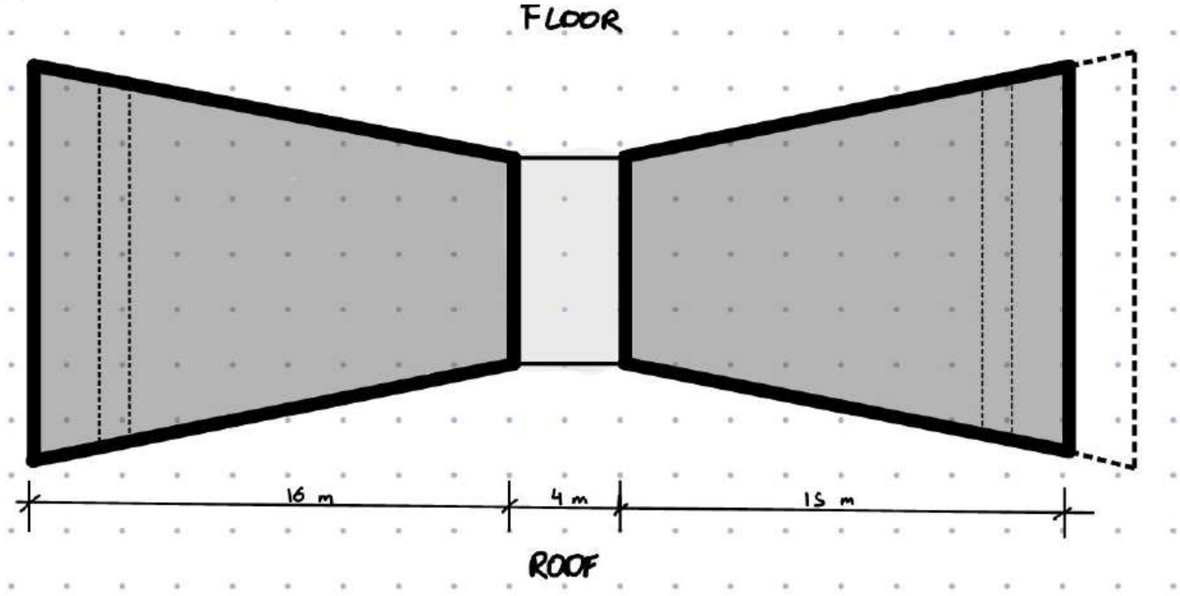
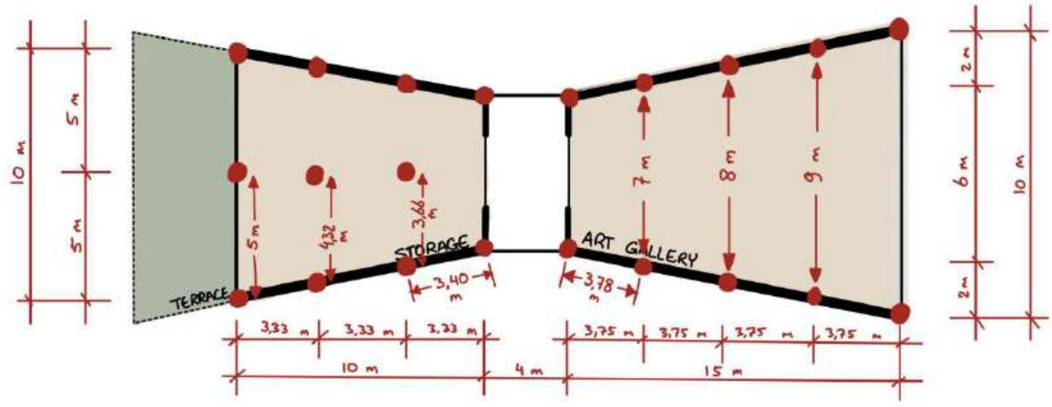
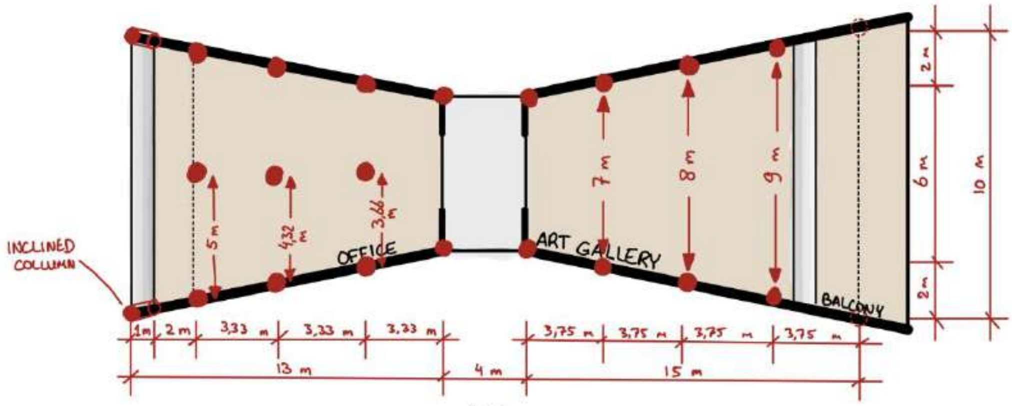
Appendix B : Roof plan of the Structure



Appendix C: First floor plan and ground floor plan of the Structure

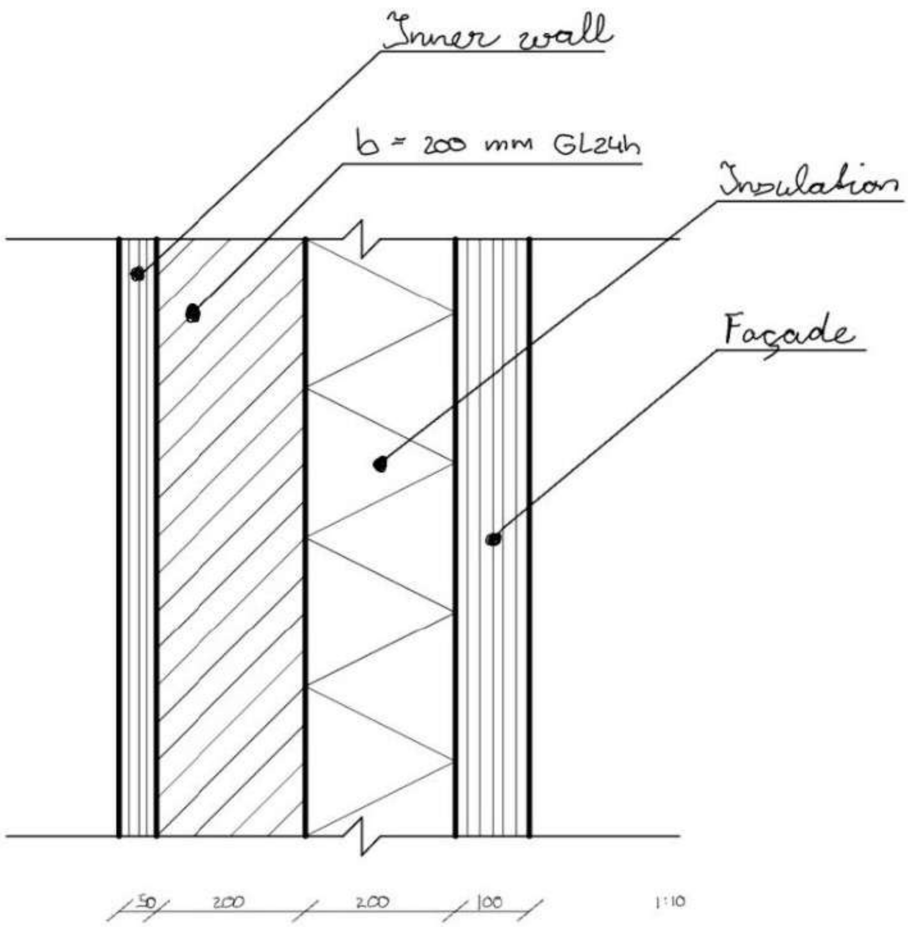


GROUND FLOOR

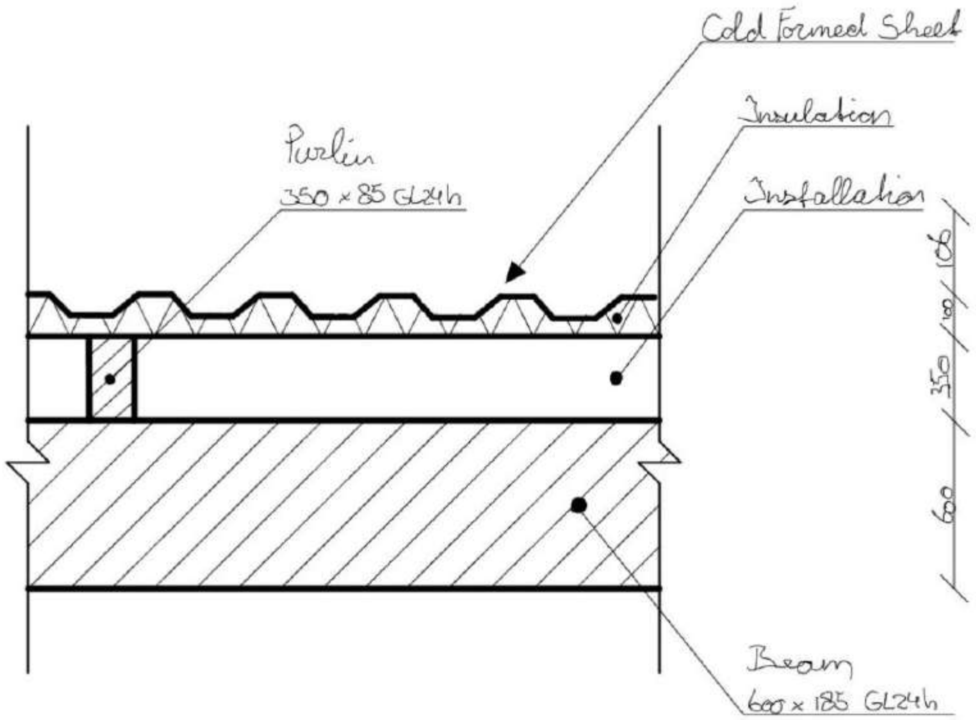


1ST FLOOR

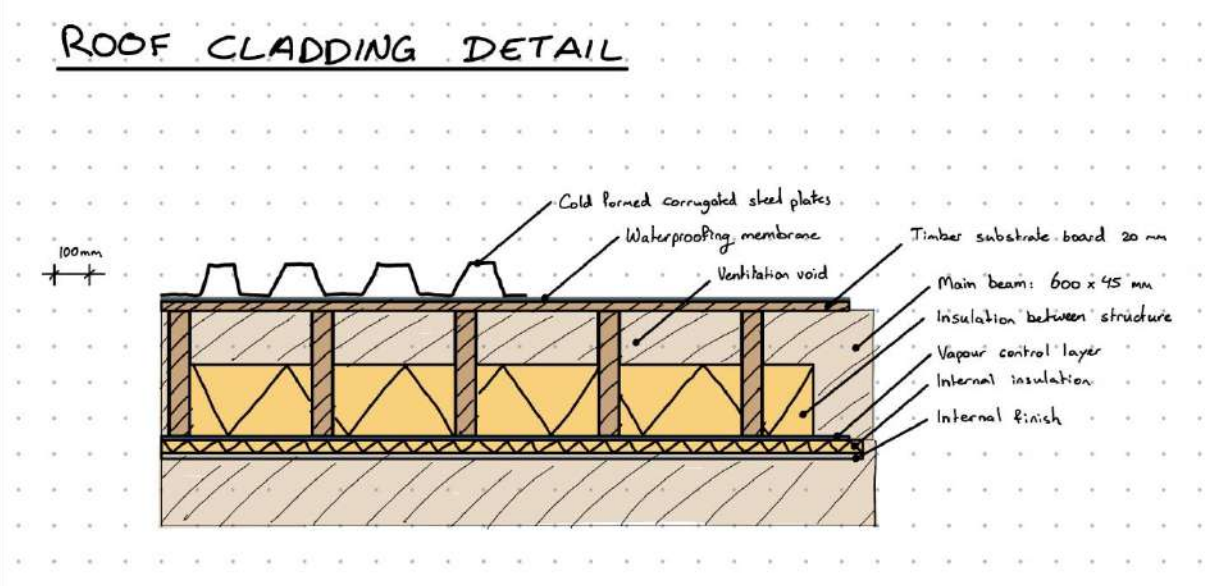
Appendix D: Cross section of the column



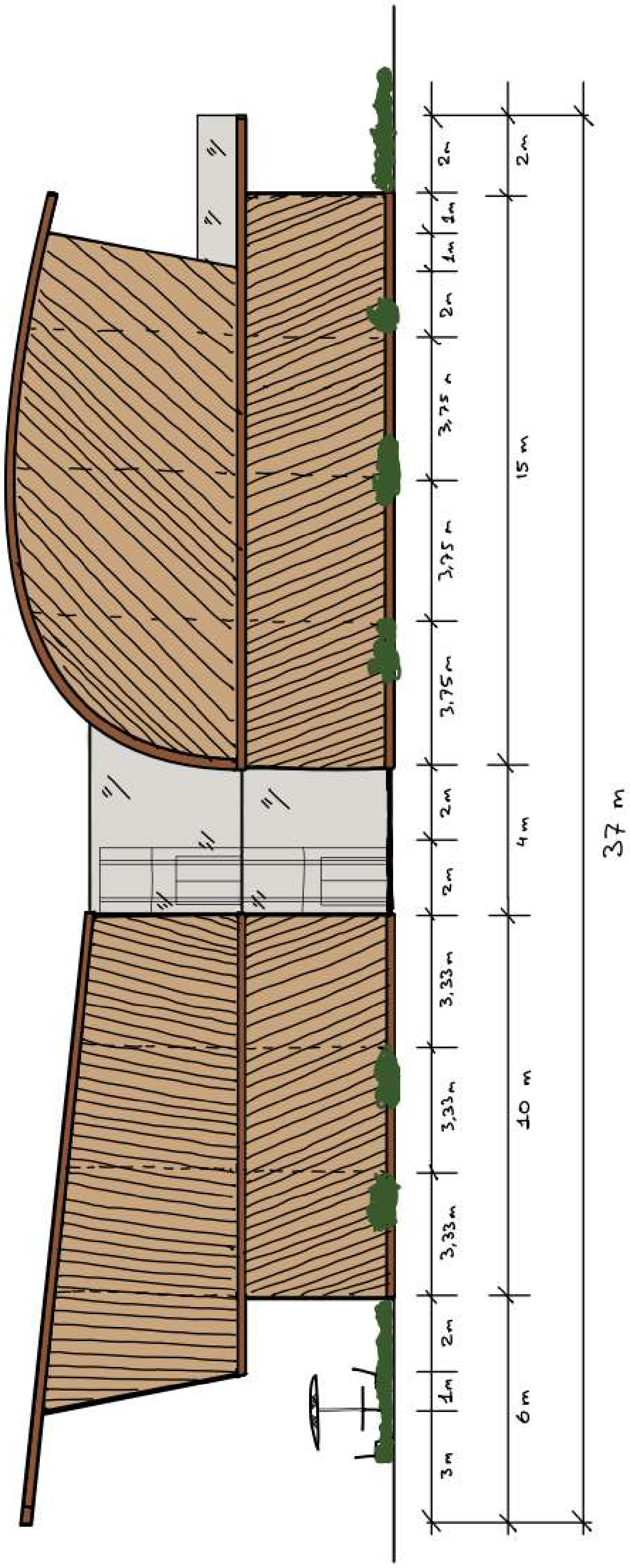
Appendix E: Cross section of the roof



Appendix F : Detail of roof cladding



Appendix G: Size view of building



Appendix H : Sketch plan of foundation piles

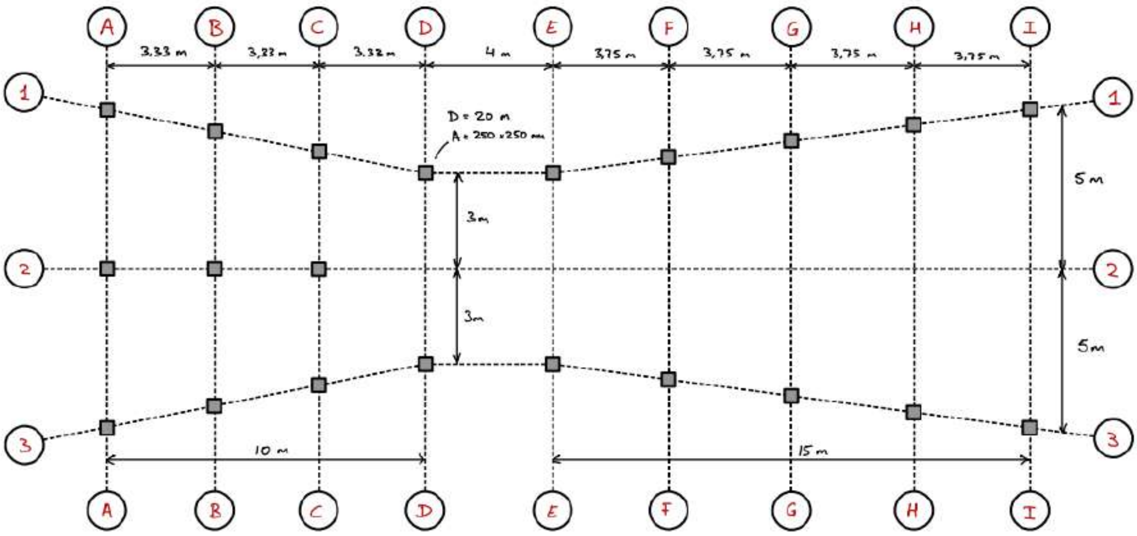
FOUNDATION PILE PLAN

PILE CHOICE:

	OFFICE	ART
Total dead loads =	249	285
Total live loads =	193	188
TOTAL LOAD =	442 kN	473 kN

> 380 kN L=10 m

< 560 kN L=20 m



Appendix I : Calculation of the Glass Structure

Calculation of Glass Structure								
Parameters								
rho	24,525							
E mod	70000000							
poisson	0,23							
h roof	0,06							
L span roof	4							
Qwind	0,46							
Qroof	3,1882							
Md	6,3764							
W roof	0,0006							
M/w	10627,23							
f	11000							
Deformation of roof								
Qroof	2,8858							
I	0,000018							
w	0,007634392							
	110,9400392							
First floor								
h floor	0,14		Qfloor	11,6				
florspan	4		Md	23,2				
			W floor	0				
			M/w	7114				
			f	7250				
Deformation of first floor								
Qfloor	8,4335							
I	0,000228667							
w	0,001756247							
umax	0,110940039							
Wall on first floor				**Vertical wall**			**Horizontal wall**	
depth wall	0,02			F	43,2		Qwall	3,234375
width wall	4			N	540		Wwall	0,0002
length wall	5			f	8800		M/w	16171,88
							f	28125
Deformation of wall on first floor								
Qwall	0,46							
I	0.00000276							
w	0,020054408							
umax	0,025							
Wall on ground floor				**Verical wall**			**Horizontal**	
depth wall	0,02			F	55,9		Qwall	2,07
width wall	4			N	699		Wwall	0,0002
length wall	4			f	8800		M/w	10350
							f	11000
Deformation ground floor								
Qwall	0,46							
I	2,6667E-06							
w	0,008214286							
umax	0,02							