

Comparative analysis of different construction solutions for industrial halls

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Notations

Acronyms

<i>CTBUH</i>	Council in Tall Buildings and Urban Habitat
<i>CTICM</i>	Centre Technique Industriel de la Construction Métallique
<i>EE</i>	Embodied Energy
<i>EN</i>	European Norm
<i>EQU</i>	Equilibrium
<i>glulam</i>	Glue Laminated Timber
<i>GWP</i>	Global Warming Potential
<i>ISSD</i>	Interactive Software for Structural Design
<i>LCA</i>	Life Cycle Assessment
<i>LTS</i>	Load Transferring System
<i>SCIA</i> ®	SCIentific Application
<i>SLS</i>	Service Limit State
<i>ULS</i>	Ultimate Limit State
<i>VIP</i>	Variable Inertia Profile

Roman symbols

A	[m ²]	Area
B	[m]	Base dimension
C_0	[—]	Horographic coefficient
$C\#$	[MPa]	Timber grade
C_d	[—]	Deflection limit
C_{dir}	[—]	Direction coefficient
C_e	[—]	Wind exposition coefficient
C_{pe}	[—]	External pressure coefficient
C_{pi}	[—]	Internal pressure coefficient
c_r	[—]	Ground roughness coefficient
C_{season}	[—]	Temporary construction coefficient
C_t	[—]	Thermal coefficient
e	[m]	Thickness
E	[Pa]	Young's modulus
EE	[MJ]	Embodied Energy
f	[MPa]	Material Strength
F	[N]	Force
f_y	[MPa]	Elastic limit
G	[N]	Permanent action

GWP	[kgCO ₂ /kg]	Global Warming Potential
h	[m]	Height
I	[mm ⁴]	Moment of inertia
I_v	[—]	Turbulence intensity
k_{def}	[—]	Creep coefficient
k_{dis}	[—]	Stress dispersion coefficient
k_h	[—]	Height coefficient
k_m, α	[—]	Coefficient of decrease for the height
k_{mod}	[—]	Load duration and service class depending coefficient
k_p	[—]	Load-bearing capacity factor
k_p	[—]	Shape coefficient
k_r	[—]	Wood strips bending strength factor
k_r	[—]	Terrain factor
k_{sys}, k_h	[—]	Wood calculation geometry coefficient
k_{vol}	[—]	Influence of the volume coefficient
L	[m]	Span
L_f	[m]	Buckling length
M	[N m]	Bending moment
N	[N]	Normal force
p_l	[N/m ²]	Limit pressure at the level of the foundation
q	[N/m ²]	Linear Load
q_0	[N/m ²]	Total vertical pressure in the soil
Q	[N]	Variable action
q_p	[N/m ²]	Dynamic peak pressure
r	[m]	Radius of curvature
R_d	[—]	Design resistance
$S\#$	[MPa]	Steel grade
s	[m]	Web thickness
s_k	[N/m ²]	Snow load
S	[kN/m ²]	Snow action
S_d	[—]	Design solicitation
t	[m]	Flange thickness
U	[W/m ² K]	Overall heat transfer coefficient
V	[N]	Shear force
V	[m ³]	Volume
v_b	[m/s]	Wind speed
W	[m]	Deflection
W	[N/m ²]	Wind action
W_{axis}	[mm ³]	Section modulus
z	[m]	Elevation
z_0	[m]	Roughness equivalent length

Subscripts

0	Reference value
ap	Ridge
b	Bottom
c	Compression
cr	Critical
d	Design
el	Elastic
f	Flange of the beam
k	Characteristic value
m	Bending
M	Mean value
p	Pressure
pl	Plastic
s	Suction
t	Top
v	Shear
w	Web of the beam

Greek symbols

α	[°]	Slope
α	[−]	Imperfection factor
Γ	[−]	Ratio of solicitation and resistance
γ_G	[−]	Partial coefficient on permanent action
γ_M	[−]	Material coefficient
γ_Q	[−]	Partial coefficient on variable action
γ_s	[N/m ³]	Soil unit weight
λ	[−]	Slenderness
μ_i	[−]	Form coefficient
σ	[MPa]	Stress
τ	[MPa]	Shear stress
Ψ	[−]	Statistical factor

Introduction

In Belgium, the construction sector is on a roll since the end of the last economic crisis. It is nowadays the sixth field to contribute to the gross domestic product in the country and the number of buildings increased by 12 % over the last twenty years [19]. Non-residential constructions increased by 9 % between 2010 and 2015. More than 17 % of developed land surfaces are devoted to industrial purpose, especially as industrial halls. In the light of the growth of the industrial halls market, it appears relevant to perform a comparative study of the most common technologies. Hence, the aim of our dissertation is to design a large panel of building solutions in accordance with the Eurocodes and to make a detailed comparison between them in order to extract their assets and drawbacks. This is intended to allow people involved in such a project to choose the most appropriate solution to meet their needs and priorities.

Since the Industrial Revolution and the associated boom in steel construction, the building business has seen a democratization of precast concrete and, more recently, a revival of building timber. Nevertheless, as far as industrial halls are concerned, steel holds the largest market share. This can be explained by many historical and practical reasons. It is well known that steel has a great resistance-on-weight factor, which guarantees attractive prices. Even though it suffers from a bad reputation regarding fire (especially in Belgium where the 1967 Innovation store fire received a lot of public attention), engineers keep choosing steel since it seems to be embedded in the civil engineering culture. Moreover, Belgians have an embedded tie with this material as it represents a large part of their industry. Still, other materials start to gain in importance: precast pre-stressed concrete is relatively easy to design and does not require much research time for engineers. It also has a very good reputation as a fire-resistant component. Besides, building timber has the advantage of lightness and makes the most of today's ecological movement. Its association with concrete columns makes the solution even more attractive. Indeed, concrete columns appear to be cheaper and assemblies with foundations are simpler to execute.

For industrialists, productivity remains the number one priority. In an economical system based on growth, anything that might improve yield is worth considering. Higher yields translate into higher incomes, more job opportunities and improved living standards. The construction sector is no exception. Contractors as well as techniques are chosen for their price, their reliability and rate of implementation.

However, as signs of excessive pollution have become more and more pressing these last decades, new preoccupations have come to surface. Environmental impact is being closely monitored for an increasing number of new projects, from the raw materials production to the heat system consumption.

The present thesis emphasizes both aspects: productivity and environmental impact.

This document is divided into six chapters:

First and foremost, we present and design the various basic cases that we have chosen to depict the Belgian market as precisely as possible. The aim of these first two chapters is threefold: to fix the framework of the study, to allow the comparison between different building solutions by accurately dimensioning each constituent of the structures, and lastly, to propose a complete method to dimension such an industrial building from A to Z. This could then provide students or junior engineers a collection of all the essential information.

Then, throughout the three following chapters we compare the halls that were designed in Chapter 2 and vary some parameters like hall size or building envelopes to investigate the impact on performances. In order to confront the different results objectively, precise criteria like weight, environmental impact indicators or time to build up the construction are required.

Finally, we present the limitations of our method and conclude the study by giving some recommendations addressed to professionals; both engineers that are responsible for an industrial hall project and customers or contractors. The aim is not to provide a perfect solution but to propose an impartial comparative analysis of the most common and efficient building technologies used for industrial halls.

Framework of the studies

1.2 Cases description

Some cases were chosen from the beginning of the study. The goal is to cover a wide panel of situations in order to describe the situation of the current industrial hall market as precisely as possible. With this aim in mind, we start with our basic case: The 42x16m hall, comparable to many agricultural halls, which represents a significant part of the demand. The distance between two successive frames is 6 m because of the standardization of many building elements. Indeed, most of the materials we need are available in this size. This characteristic is constant for every situation in this report. The dimensions of all our cases are fixed in agreement with the previously mentioned study Hall-in-Kit [2]. It is designed with four different materials (or material associations), that we chose to be most relevant with the current demand:

- Hot-rolled steel profiles: These elements are produced by rolling a large piece of steel through a rolling machine. Roll after roll, the metal is deformed until obtaining the required section. It can be manufactured at a very high rate and many standard sizes are available. This is the most common steel construction material nowadays.
- Welded plate profiles: These are assembled from steel plates previously cut. The pieces are welded together. This manufacturing process allows for a varying height along the element. It enables to optimize the design with variable inertia that fits the variable stresses. Nevertheless, adding complexity to the production process leads to an increase in the price.
- Precast concrete: Manufacturers like Ronveaux or Ergon sell pre-stressed and precast concrete on catalogue. These elements have standard dimensions and admissible loads, making projects using them very easy to design.
- Precast concrete columns and glue laminated timber girders: This solution combines precast concrete columns, as previously described, with glue laminated timber beams. Glulam is produced out of thin wood slices glued together. It generates a material with very few defects and thus, with much more consistent mechanical properties. Furthermore, the gluing process enables to create many shapes. Hence variable inertia is possible.

The first two situations are the opportunity to compare most common steel grades:

- S235
- S355
- S460

Furthermore, different building envelope technologies are compared regarding heating system consumption during usage phase of the building. Finally, we observe the influence of the aforementioned parameters on two additional cases:

- The 24x42 hall dedicated to industrial tasks
- The 36x42 hall that brings us to push the materials and the structural systems to the limits of feasibility.

Only the frames are calculated as the secondary structure doesn't change from one case to another.

For some aspects of the comparison process (like the wind action calculation or the evaluation of the environmental impact due to transportation) it is required to know the location of the building. We chose the industrial estate of Nivelles for its central position in the country (see figure 1.1 ¹).



Figure 1.1: Chosen location for the construction site

1.3 Comparison criteria

So as to objectively compare these various situations we need precise criteria:

- The weight: this is a key parameters on which many of the others depend.
- The environmental impact: it depends on many parameters (material, building envelope, transport ...) and takes all the phases into account; from raw material production and usage phase to recycling.
- The assembly duration: this criterion can be determinant for financial viability of some projects. It depends mostly on the number of elements, their weight and their ability to be handled by crane.

1.4 Comparative study : methodology and objective

In chapter 2, all the solution are designed and their mass are compared. Three building dimensions, two structural models, two foundation systems, four material types and three steel grades are considered. From these quantities, it is possible to go further into the analysis.

In the chapter 4 and 5 the different halls will be compared one to each other in terms of their environmental impact, on the one hand and of their assembly time on the other hand. The

¹Source of the picture: Google Maps

present section introduces this analyze by presenting the method, the tools and some basic elements required for the reader to fully apprehend the results which come as an output of the study.

1.4.1 Life cycle assessment

In the past ten years, ecological considerations have become more and more important. With the observations of climate change, new issues emerge and the ecology is a factor we can no longer ignore. In this context, and knowing that both construction industry and residential heating are among the largest CO₂ emitting sectors, it seems relevant to integrate the environmental impact in the comparison.

life cycle phase

In this analysis, all phases of the life of the building are considered. From the materials production to the dismantling and the recycling. These steps are often separated into five distinct stages (see table 4.1) :

- Stage A: raw material is extracted, it is transported to the places of processing and all the required industrial transformations are performed to obtain the usable material. Then, the operable material is transported again to its place of use and is placed by on-site teams.
- Stage B: once the material is placed, it is used, requires some consumables like water or combustibles, must be replaced, its maintenance must be performed. . .
- Stage C: at the end of its life, material must be dismantled, transported to the waste processing centre and transformed to be stored.
- Stage D: finally, the waste can be reused or recycled.

The question of including the stage D to the life cycle assessment is controversial. The answer depends on convictions and economic interest of specialists; defenders of poorly recyclable materials want to neglect the recycling phase while defenders of recyclable materials advocate taking it into account. The legal framework governing this issue vary from one country to another. In Belgium, taking the D phase into account is mandatory. In this study, both solutions are analyzed and compared in order to maintain our neutrality and to keep our results consistent no matter the country.

Indicators choice

Many indicators and coefficients exist to translate the data and represent the environmental impact of a project. We chose the two most used ones: GWP and EE. Hence, our results and conclusions can be compared with data from other studies.

- GWP is the Global Warming Potential and describes the influence all the used materials on the global warming effect throughout their life stages.
- EE is the Embodied Energy and record all the energy required to produce the final product.

Tools and data

In order to successfully complete this analyze, we use the software Ameco developed by Arcelor-Mittal. It is basically an accounting system which takes the mass of the different materials as an input and multiply them by suitable coefficients. This coefficients database is the heart of the method and is presented in more detail in the section 4.2. Microsoft Excel was then used to obtain the relevant graphs from the data.

General remark

Reader must keep in mind that regarding the life cycle assessment, all results depend highly on the selected parameters and indicators. We chose them among available information to be as most representative of the reality as possible. However, parameters and indicators always depend on interpretations and so, some results must be put into context. For example, the embodied energy calculation takes either renewable and non-renewable energy into account. Yet, the renewable energy required to produce wood is high in comparison with the total energy. Thus, wooden structures will automatically appear less environmentally friendly based on EE. It shows that conclusions must be done using critical judgment and that different evaluation techniques must be used.

Building envelope and stage B

Furthermore, environmental performances of the building envelope were also studied. The aim is to compare highly and poorly insulated materials that constitute the facades and roof in terms of heating consumption. Indeed, maintaining a constant temperature in the industrial building incurs energy consumption and it is useful to know whether or not it is energetically effective to invest in high performance coverings.

1.4.2 Assembly time

As previously explained, the time between the first day of the construction and the beginning of the exploitation is a key factor to determine the financial profitability of an industrial project. Then, the assembly time is integrated as a comparison criterion. In order to evaluate the production duration of each solution, a database was provided by ArcelorMittal and adapted to suit every situation of this study (see section 5.1). Knowing the unit time for all the products, we can compute the duration required to install the elements of complete solutions.

1.4.3 Objectives of the analysis

As said in the introduction of this report, the aim is not to provide a perfect solution that is optimal for every criterion but to make some recommendations towards project owners, engineers and contractors according to their needs and priorities.

Chapter 2

Basic cases design

This chapter contains the design of the first case with the four constructive methods. The first point defines the dimensions of both main structure and secondary structure. Then, the different loads and combinations of loads, taken into account in agreement with the Eurocodes, will be explained.

Once the hypotheses considered in the study are set, the rest of the chapter presents the design of secondary and main structures. For the design of the portal frames, two structural systems are considered. The first one is a pinned base frame for the hot-rolled steel solution and for the welded built-up one. The second system is composed of rigid base columns and a pinned girder for precast concrete and precast concrete columns combined with glue laminated timber girders.

At the end of the chapter, reader will find a table summarizing the different designs.

2.1 Structure dimensions

The first industrial hall case is 42 m long and 16 m wide. Frames are 6 m apart from each other. Columns are 4 m high and the roof has a slope of 2 % which corresponds to a 1.15° angle. Purlins are placed every 1.6 m. Girts are spaced out by 1 m and end-wall columns are 2 m apart from each other. This geometry is described by the figure 2.1.

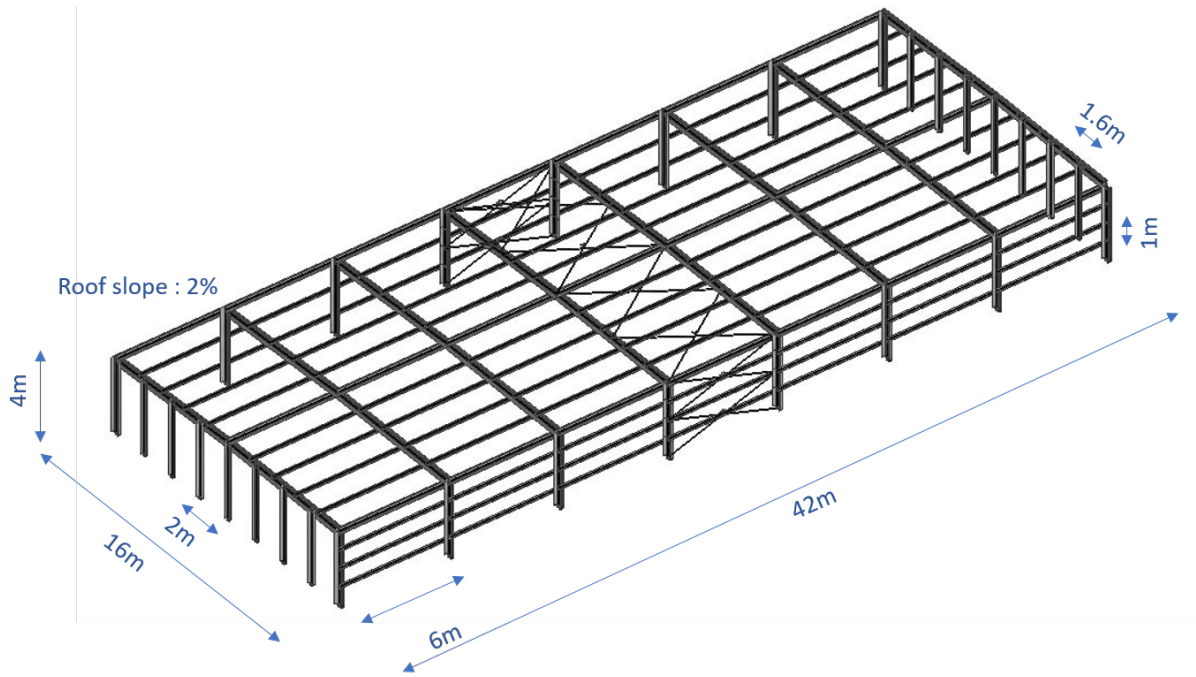


Figure 2.1: First case hall geometry

2.2 Loads

2.2.1 Permanent loads

In the first place, let's notice that the weight of the steel was considered in accordance with the Eurocode 1 [7]. The weight of the covers corresponds to the heaviest solution among the two studied sandwich panels in section 4.2. The different permanent loads used for the study are the following:

- Steel self-weight: $78 \text{ [KN/m}^3\text{]}$ (Eurocode 1-1-1, Table A.4 [7])
- Other materials self-weight: different from one element to another.
- Roof and walls covering: $0,15 \text{ [KN/m}^2\text{]}$ (constructalia.com [3])

2.2.2 variable loads

Eurocode 1 defines loads that must be taken into account when designing a structure. Its first part (EN 1991-1) is itself divided into different sub-parts, where loads due to different types of actions (wind, snow, imposed loads ...) are defined. This section explains how these variable actions are computed for an industrial hall.

Imposed loads

To determine the value of the imposed loads, the assumption is made that roofs are inaccessible except for routine maintenance and repairs. The Eurocode defines the following load:

Category H: Inaccessible roof 0,7 [kN/m²] (EN1991-1-1-1 [7])

Snow loads

Snow action can be calculated by the following formula (following EN1991-1-1-3[8]):

$$S = \mu_i \cdot C_e \cdot C_t \cdot s_k = 0.4 \text{ [kN/m}^2\text{]} \quad (2.1)$$

With :

- μ_i : Form coefficient, it depends on the roof inclination. For an angle of 1.15°, $\mu_i = 0.8$
- s_k : Characteristic value of snow load on the roof. We take $s_k = 0.5 \text{ [kN/m}^2\text{]}$ as the altitude is lower than 100 m.
- C_e, C_t : Wind exposition coefficient and thermal coefficient (both are equal to 1 in Belgium).

Wind actions

The following assumptions were made:

- The hall is located in the Brabant Wallon. It gives $v_{b,0} = 25 \text{ [m/s]}$
- Ground roughness of category IIIb (industrial zone)
- Flat ground (horographic coefficient $C_0(z) = 1$)

Let's follow the Eurocode method [9] to evaluate the winds actions. First, the reference speed is calculated with the following equation:

$$v_b = C_{dir} \cdot C_{season} \cdot v_{(b,0)} = 25 \text{ [m/s]} \quad (2.2)$$

with :

- $v_{b,0}$: The reference speed of the wind = 25 [m/s]
- C_{dir} : The direction coefficient = 1
- C_{season} : The temporary construction coefficient = 1

Secondly, at an elevation z above the ground, the mean speed is computed as follows:

$$v_m(z) = C_o \cdot c_r \cdot v_b \quad (2.3)$$

Where:

- $C_o(z)$: The horographic coefficient $C_o(z) = 1$
- $c_r(z)$: The ground roughness coefficient calculate with equation (2.4).

The ground roughness coefficient $c_r(z)$ depends on the height of the structure z exposed to the wind and the roughness of the ground around the building. The values of z_0 and z_{min} depend on the terrain category and are found in the table 2.1. The hall is built in a industrial park. It corresponds to a III terrain category. Hence, $z_0 = 0.3 \text{ m}$ and $z_{min} = 5 \text{ m}$.

Catégorie de terrain	z_0 [m]	z_{min} [m]
0 Mer ou zone côtière exposée aux vents de mer	0,003	1
I Lacs ou zone plate et horizontale à végétation négligeable et libre de tous obstacles	0,01	1
II Zone à végétation basse telle que de l'herbe, avec ou non quelques obstacles isolés (arbres, bâtiments) séparés les uns des autres d'au moins 20 fois leur hauteur	0,05	2
III Zone avec une couverture végétale régulière ou des bâtiments, ou avec des obstacles isolés séparés d'au plus 20 fois leur hauteur (par exemple des villages, des zones suburbaines, des forêts permanentes)	0,3	5
IV Zone dont au moins 15 % de la surface sont recouverts de bâtiments dont la hauteur moyenne est supérieure à 15 m	1,0	10
NOTE Les catégories de terrain sont illustrées en A.1.		

Table 2.1: Terrain category and parameters

For $z_{min} \leq z \leq z_{max} = 200 \text{ m}$:

$$c_r(z) = k_r \cdot \ln \left(\frac{z}{z_0} \right) \quad (2.4)$$

For $z \leq z_{min}$:

$$c_r(z) = c_r(z_{min}) \quad (2.5)$$

With :

- k_r : The terrain factor $= 0.19 \cdot \left(\frac{z_0}{z_{0,II}} \right)^{0.07}$
- z_0 : Roughness equivalent length
- z_{min} : Minimal height depending on the terrain category

Now, the reference height z of the building needs to be determined. As the height of the hall is smaller than its width, the Eurocode imposes to take the height of the ridge as the reference height equal to 4.16 m.

Then dynamic peak pressure is found:

$$q_p(z) = 0.625(1 + 7 \cdot I_v(z)) \cdot v_m^2(z) = 483 \text{ [N/m}^2\text{]} \quad (2.6)$$

Where:

$$v_m(z) = 0.19 \cdot v_{b,0} \cdot \left(\frac{z_0}{0.05}\right)^{0.07} \cdot \ln\left(\frac{z}{z_0}\right) \cdot c_0(z) \cdot C_{prob} \cdot C_{dir} \cdot C_{season} = 15.15 \text{ [m/s]}$$

$$I_v(z) = \frac{1 - 2 \cdot 10^{-4}(\log_{10}(z_0) + 3)^6}{\ln\left(\frac{z}{z_0}\right)} = 0.34$$

Further expressions will require the C_{pe} external pressure coefficient. It depends on the exposed area A and can be found for $A = 1 \text{ m}^2$ or $A = 10 \text{ m}^2$ and for different building configurations in the figure 2.2. Local coefficients are called $C_{pe,1}$ and global ones $C_{pe,10}$.

EN 1991-1-4:2005 (F)

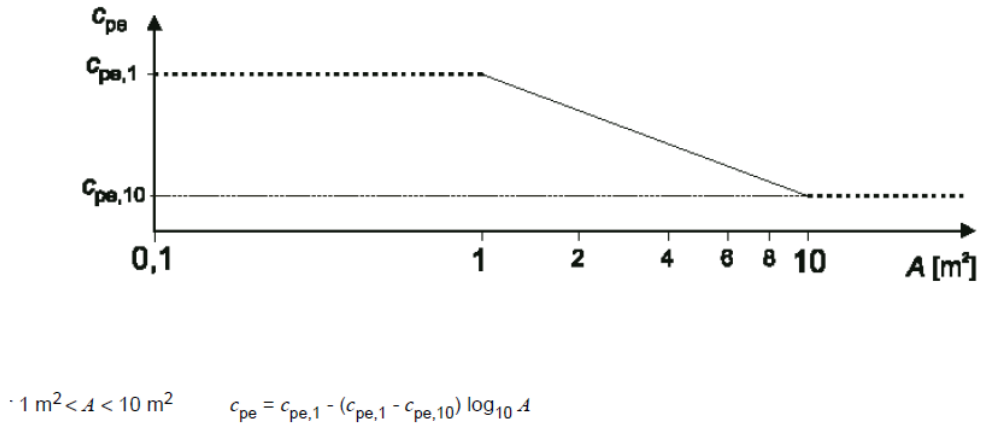


Figure 2.2: Global and local external pressure coefficients

Wind actions on the roof

Thereafter, readers will find a figure that describes influence zones of the wind and a table gathering the coefficients values. According to the Eurocode, since the slope of the roof is under 5° the roof can be considered as flat.

	Windward	Leeward
Case 1	Suction	Suction
Case 2	Suction	Pressure
Case 3	Pressure	Suction
Case 4	Pressure	Pressure

If the wind is perpendicular to the frame:

Zone	F	G	H	I
	$C_{pe,10}$	$C_{pe,10}$	$C_{pe,10}$	$C_{pe,10}$
Suction	-2.34	-1.20	-0.70	-0.20
Pressure	0	0	0	0.20

Once again, four cases can be identified for this wind direction:

	Windward	Leeward
Case 1	Suction	Suction
Case 2	Suction	Pressure
Case 3	Pressure	Suction
Case 4	Pressure	Pressure

As the building is closed, internal pressure coefficient C_{pi} can be taken equal to 0.2 or -0.3.

Wind action on the structure is computed like so:

$$W = q_p(z) \cdot (C_{pe} - C_{pi}) \quad (2.7)$$

Results (with maximal values in red):

Wind parallel to the frame				
Zone	F	G	H	I
	$C_{pe,1.7}$	$C_{pe,10}$	$C_{pe,10}$	$C_{pe,10}$
Suction	-2.34	-1.20	-0.70	-0.20
$W(C_{pi} = -0.3) [N/m^2]$	-724	-434	-193	48.3
$W(C_{pi} = 0.2) [N/m^2]$	-965	-675	-434	-193
Pressure	-2.34	-1.20	-0.70	0.20
$W(C_{pi} = -0.3) [N/m^2]$	-2.34	-1.20	-0.70	241
$W(C_{pi} = 0.2) [N/m^2]$	-965	-675	-434	0
Wind perpendicular to the frame				
Zone	F	G	H	I
	$C_{pe,1.7}$	$C_{pe,10}$	$C_{pe,10}$	$C_{pe,10}$
Suction	-2.34	-1.20	-0.70	-0.20
$W(C_{pi} = -0.3) [N/m^2]$	-724	-434	-193	48.3
$W(C_{pi} = 0.2) [N/m^2]$	-965	-675	-434	-193
Pressure	0	0	0	0.20
$W(C_{pi} = -0.3) [N/m^2]$	-724	-434	-193	241
$W(C_{pi} = 0.2) [N/m^2]$	-965	-675	-434	0

Wind actions on the side walls

The dynamic peak pressure at 4 m high is taken as calculated before. The external pressure coefficient corresponding to the vertical walls is computed with the following tables.

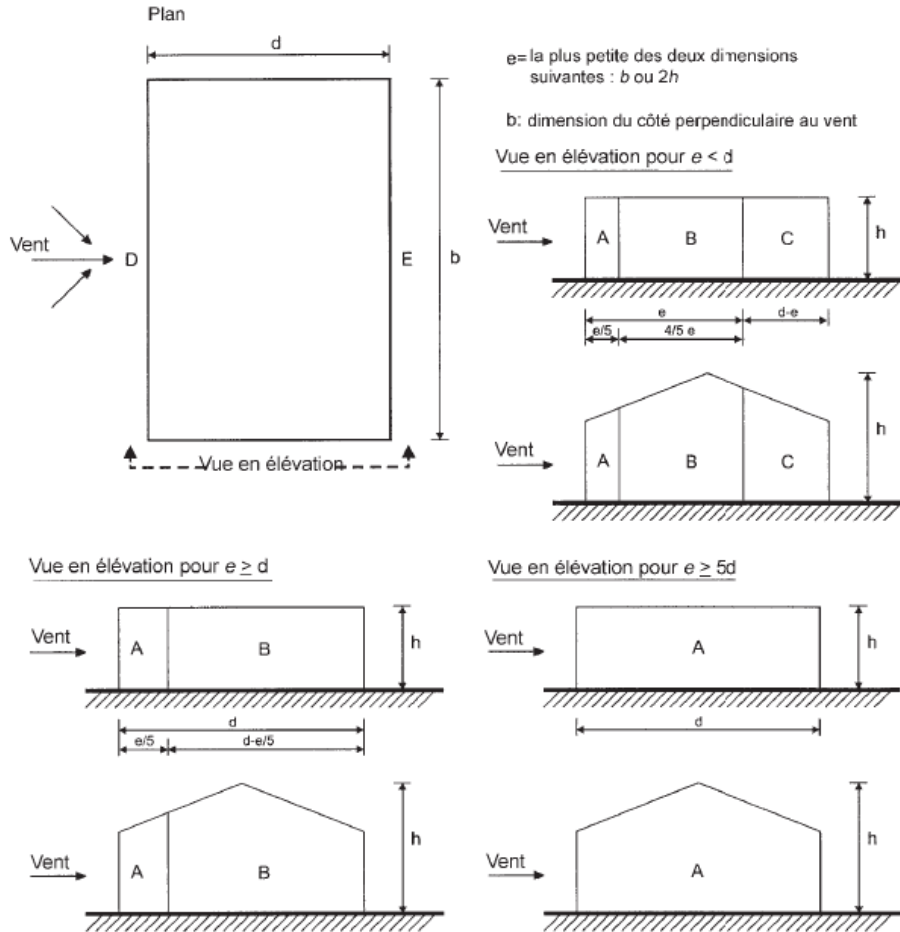


Figure 2.4: Influence zones of the wind on vertical walls

Tableau 7.1 — Valeurs recommandées des coefficients de pression extérieure pour les murs verticaux des bâtiments à plan rectangulaire

Zone	A		B		C		D		E	
h/d	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$
5	-1,2	-1,4	-0,8	-1,1	-0,5		+0,8	+1,0	-0,7	
1	-1,2	-1,4	-0,8	-1,1	-0,5		+0,8	+1,0	-0,5	
$\leq 0,25$	-1,2	-1,4	-0,8	-1,1	-0,5		+0,7	+1,0	-0,3	

Table 2.3: External pressure coefficients for side walls

If the wind is parallel to the frame:

Zone	Surface [m ²]	Height [m]	q_p	$C_{pe,10}$ ≥ 0	$C_{pe,1}$ ≥ 0	$C_{pe,10}$ ≤ 0	$C_{pe,1}$ ≤ 0	C_{pi} ≥ 0	C_{pi} ≤ 0	Pressure [N/m ²]	Depression [N/m ²]
A	13.3	7.23	567			-1.20	-1.40		0.20		-794
B	70	7.23	567			-0.80	-1.10		0.20		-567
C	6.6	7.23	567			0.50	-0.50		0.20		-397
D	168.0	4.0	482	0.73	1.0			-0.30		495	
E	168.0	7.23	567			-0.35	0.35		0.20		-314

If the wind is perpendicular to the frame:

Zone	Surface [m ²]	Height [m]	q_p	$C_{pe,10}$ ≥ 0	$C_{pe,1}$ ≥ 0	$C_{pe,10}$ ≤ 0	$C_{pe,1}$ ≤ 0	C_{pi} ≥ 0	C_{pi} ≤ 0	Pressure [N/m ²]	Depression [N/m ²]
A	11.6	7.23	567			-1.20	-1.40		0.20		-794
B	46.3	7.23	567			-0.80	-1.10		0.20		-567
C	110.1	7.23	567			0.50	-0.50		0.20		-397
D	89.9	7.23	567	0.70	1.0			-0.30		567	
E	89.9	7.23	567			-0.30	0.30		0.20		-284

2.3 Combination of actions

Structural reliability must be checked for a limit state under the simultaneous effect of different actions [12]. A combination corresponds to a load calculated by performing the sum of the actions weighted by different coefficients. The combination depends on whether the ultimate limit state or the service limit state is checked.

The verification of an ultimate limit state is expressed by the following inequation:

$$S_d \leq R_d \quad (2.8)$$

- R_d is the design resistance of the structure. The resistance calculation values are obtained by applying the appropriate partial safety coefficients of the materials on the characteristic resistances.
- S_d represents internal stresses (calculation values) in a critical section under the effect of the most unfavorable combination of actions.

The solicitation is obtained by combining the different load case multiplied by appropriate coefficients as expressed in the equation

$$S_d = \overbrace{\gamma_G G}^{\text{Permanent action}} + \overbrace{\gamma_{Q1} Q1}^{\text{Basic variable action}} + \overbrace{\sum_{n=2}^n \Psi_{0,i} \gamma_{Q,i} Q_i}^{\text{Secondary variable action}} \quad (2.9)$$

- γ_G is the partial coefficient on permanent action
- γ_{Q1} is the partial coefficient on variable action

- Ψ is a statistical factor, it reflects the probability that the different actions occur simultaneously

For the deflection verification, we use the SLS combination. The general expression of the verification of a service limit state takes the form:

$$S_d \leq W_d \quad (2.10)$$

- W_d is the deflection limit value which may not be exceeded
- S_d represents the calculated value of deflection

It exists three types of SLS combinations:

- Rare combination

$$G + Q_1 + \sum_{n=2}^{n=i} \Psi_{0,i} Q_i \quad (2.11)$$

- Frequent combination

$$G + \Psi_{1,1} Q_1 + \sum_{n=2}^{n=i} \Psi_{2,i} Q_i \quad (2.12)$$

- Practically permanent combination

$$G + \sum_{n=1}^{n=i} \Psi_{2,i} Q_i \quad (2.13)$$

The EN1990 recommends to use $\gamma_G = 1.35$ and $\gamma_Q = 1.5$ and all these Ψ coefficients can be found in the table 2.4 below:

Variable action	Ψ_0	Ψ_1	Ψ_2
Imposed loads in buildings, category			
Category A : domestic, residential areas	0.7	0.5	0.3
Category B : office areas	0.7	0.5	0.3
Category C : congregation areas	0.7	0.7	0.6
Category D : shopping areas	0.7	0.7	0.6
Category E : storage areas	1	0.9	0.8
Category H : roofs	0	0	0
Snow loads	0.7	0.5	0.2
Altitude $\geq$ 1000 m	0.7	0.5	0.2
Altitude $\leq$ 1000 m	0.5	0.3	0
Wind loads	0.6	0.2	0

Table 2.4: Recommended values of Ψ factors for buildings

As a lot of Ψ coefficients are equal to zero, only the rare combination 2.11 will be used in the rest of the study. Indeed, this combination leads to the most important stresses in the structure.

2.4 Secondary structural system design

Once our loads are properly set, the secondary structure can be computed. This term gathers four main elements:

- The purlins perform two functions: They act as a support for the roofing and transmit the lateral load from one frame to another.
- The girts hold the lateral cladding.
- The lateral stability system ensures the sustainability of the structure when it is subjected to loads perpendicular to the plan of the frames.
- The end-wall columns hold the lateral cladding and transmit the wind load to the first and the last frames.

Later in this report, some other hall cases with different dimensions will be designed. Nevertheless, the space between two successive frames remains constant and equal to 6 m. Therefore, we consider that the secondary structure is very similar from one situation to another and it will not be re-computed.

2.4.1 Purlins

In this section, the purlins are designed with steel and wood. The aim is to check that the actions never result in stresses greater than the resistance of the material and that the deflection does not exceed a limit value. Both materials are commonly used for this purpose and it gives us the opportunity to compare their performances and attractiveness.

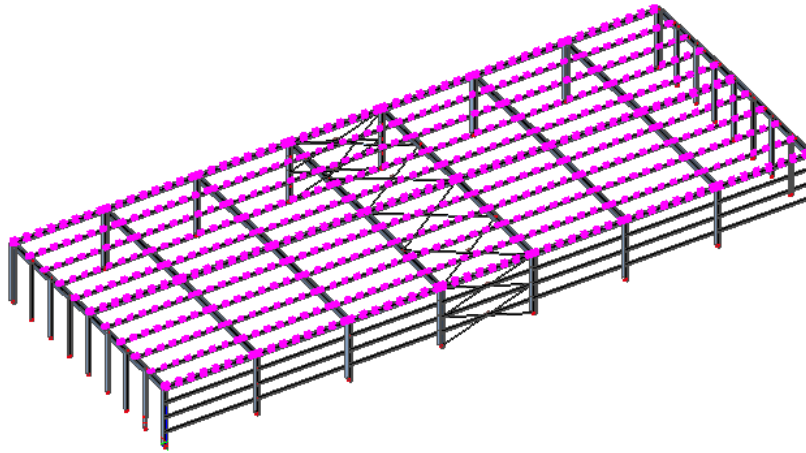


Figure 2.5: The purlins

Let's remind that:

- Roof slope: 2 %
- Span: 6 [m]
- Distance between two purlins: 1.6 m

Steel

The roof is tilted with an angle of 1.15° . So it can be considered as flat. Therefore, we don't have to compute biaxial bending. We need to check the resistance of each element under the effect of different actions applied simultaneously. Thereafter, readers can find a table that sums up the combinations used to check the stability of the purlins. We distinguish two types of combinations:

- Resistance of the structure (ULS)
- Check at the service limit state (SLS)

We obtain:

Combination	Permanent action	Primary variable action	Accompanying variable actions	Numerical result [kN/m^2]
ULS 1	$1.35 \cdot G$	$1.5 \cdot Q$	$1.5 \cdot 0.6 \cdot W_{0^{\circ},p} + 1.5 \cdot 0.5 \cdot S$	1.97
ULS 2	$1.35 \cdot G$	$1.5 \cdot Q$	$1.5 \cdot 0.6 \cdot W_{0^{\circ},s} + 1.5 \cdot 0.5 \cdot S$	0.88
ULS 3	$1.35 \cdot G$	$1.5 \cdot W_{0^{\circ},p}$	$1.5 \cdot 0.5 \cdot S$	1.06
ULS 4	$1.35 \cdot G$	$1.5 \cdot W_{0^{\circ},s}$	$1.5 \cdot 0.5 \cdot S$	-0.74
SLS 1	G	Q	$0.6 \cdot W_{0^{\circ},p}$	1.14
SLS 2	G	Q	$0.6 \cdot W_{0^{\circ},s}$	0.421
EQU	G	$1.5 \cdot W_{0^{\circ},s}$	-	-0.168

Where:

- Q is the exploitation charge
- S is the snow load
- W is the wind
- Index s is for suction
- Index p is for pressure

One may notice that following the Eurocode [12], $\psi_0 = 0$ for exploitation loads on roof.

Resistance condition

The purlins are spaced 1.6 m apart. We can compute the linear charge for the most critical combination (in red here-above). We obtain $3.15 kN/m$. The bending moments M_y is calculated thanks to SCIA engineer to take the continuity between purlins into account:



Figure 2.6: Maximal bending moment

The following relation is used to check the resistance [10].

$$\left| \frac{M_{y,Sd}}{W_y f_y / \gamma_{M0}} \right| \leq 1 \quad (2.14)$$

The minimal section modulus to resist the bending moment:

$$W_{y,min} = \frac{M_{y,Sd}}{f_y} = \frac{11.95 \cdot 10^3}{350 \cdot 10^6 \text{ or } 390 \cdot 10^6} = 34.14 \text{ cm}^3 \text{ or } 30.64 \text{ cm}^3 \quad (2.15)$$

It corresponds to a A170200 Multibeam profile for S350 steel[1].

Deflection condition

The over-determinate beam on 8 supports is calculated by hand with Clapeyron method [22] and verified thanks to Macquart abacus. It gives a factor between the real maximal deflection and the deflection computed with an isostatic similar beam. The load used here must be the most critical SLS case ($= 1.14 \cdot 1.6 = 1.824 \text{ kN/m}$)

$$W_{real} = 0.49 \cdot \left(\frac{5ql^4}{384EI} \right) < \frac{L}{200} = 3 \text{ cm} \quad (2.16)$$

As the section isn't strong enough to respect the deflection criterion, dimensions must be increased until obtaining a under 3 cm deflection.

Profile	$f_y [Mpa]$	Deflection [cm]
A170200	350	2.4
A170200	390	2.4

For both steel grades, the result is the same. The A170200 profile in S350 steel is selected. It has a mass of 5.39 kg/m.

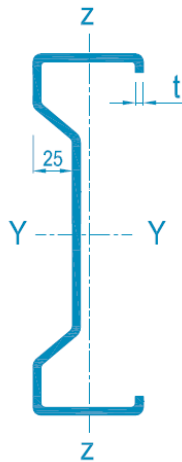


Figure 2.7: Selected profile

Wood

The following assumptions were made for the verification of the wood purlins:

- Service class 1: structures protected from the bad weather
- Purlins in solid wood: C24
- Thickness: 100 [mm]
- Height: 300 [mm]

Verification of the resistance (ULS)

The bending moment produces a stress in the direction of the axis of the beam. It is equal to zero on the average line and maximum in the upper part of the beam. It must remain below the bending resistance.

$$\frac{\sigma_{m,d}}{f_{m,d}} \leq 1 \quad (2.17)$$

- $\sigma_{m,d}$: Bending stress induced by the combination of actions
- $f_{m,d}$: Bending resistance

Calculation of stress $\sigma_{m,y,d}$

$$\sigma_{m,d} = \frac{M_{f,y}}{\frac{I_{G,y}}{V}} = 9.46 \text{ [MPa]} \quad (2.18)$$

- $M_{f,y} = \frac{q_z \cdot L^2}{8}$: Bending moment, for a beam on two hinged supports with an uniform load
- $\frac{I_{G,y}}{V}$: Modulus of inertia for a rectangular section

Calculation of the bending resistance $f_{m,d}$ in MPa

$$f_{m,y,d} = f_{m,z,d} = f_{m,k} \cdot \frac{k_{mod}}{\gamma_M} \cdot k_{sys} \cdot k_h = 22.34 \text{ [MPa]} \quad (2.19)$$

- $f_{m,k} = 24 \text{ MPa}$: Characteristic stress resistance to bending
- $k_{mod} = 1.1$: Reduction coefficient as a function of the shorter duration load and the class of service.
- $\gamma_M = 1.3$: Security coefficient.
- k_{sys} : Equal to 1 because $L \geq 1.2 \text{ m}$.
- k_h : Equal to 1 because $h \geq 150 \text{ mm}$.

Verification for the ULS

$$\frac{\sigma_{m,d}}{f_{m,d}} = 0.42 \leq 1 \quad (2.20)$$

Verification of the deflection (SLS)

The second check concerns deformation. For most of wooden beams working in flexion, this is the most critical condition. The service limit state is reached when the permissible deformations are exceeded. Their values are $W_{limited,inst} = L/300 = 13.3 [mm]$ and $W_{limited,net,fin} = L/200 = 20 [mm]$

The first deflection is the instant one induced by all the actions without considering the effect of the time.

$$W_{inst(Q)} = \frac{5 \cdot q_{inst} \cdot L^4}{384 \cdot E_{o,mean} \cdot I} = 12.44 [mm] \quad (2.21)$$

With:

- W : Deflection in $[mm]$
- q_{inst} : The linear load (SLS)
- $E_{o,mean}$: Mean Young's modulus of the wood = 11000 [MPa]
- I : Moment of inertia in $[mm^4]$

The second deflection is the final one which is the sum of the instant deflection and the creep deflection affected by the time of the loading and the humidity of the wood. The instant deflection being proportional to the load, it is much simpler to calculate the final net deflection from the instantaneous one.

$$W_{net,fin} = W_{inst} + W_{creep} = 22.38 [mm] \quad (2.22)$$

Where:

- $W_{creep} = W_{inst} \cdot k_{def} = 9.9 [mm]$
- k_{def} : Creep coefficient of 0.8 (solid wood and unheated zone)

Finally the deflection criterion can be validated thanks to the following expression:

$$maximum \left\{ \begin{array}{l} \frac{W_{inst}}{W_{limit}} = \frac{12.43}{20} = 0,62 \\ \frac{W_{net,fin}}{W_{limit,net,fin}} = \frac{22.38}{30} = 0,74 \end{array} \right. \leq 1 \quad (2.23)$$

2.4.2 Girts

The girts provide a support for lateral cladding and transmit lateral loads to the frames.

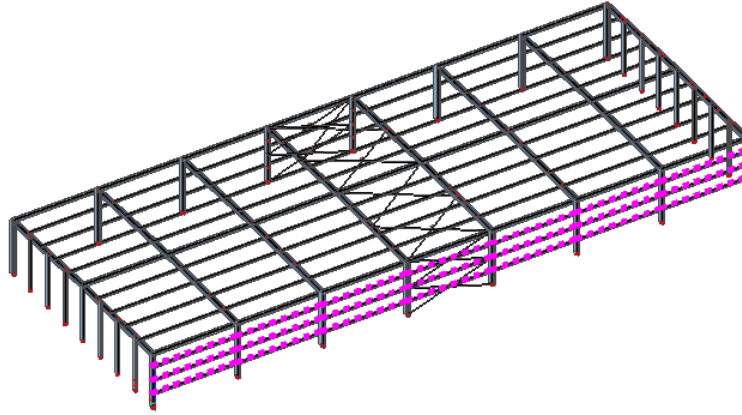


Figure 2.8: The girts

Loads

The loads applied on each girt are the following ones:

- Self weight of the girt: 0.1 [kN/m]
- Siding panels weight: $0.15 \text{ [kN/m}^2\text{]}$
- The wind pressure (or suction): $0.483 \text{ [kN/m}^2\text{]}$ at the maximum.

This case correspond to a bi-axial bending.

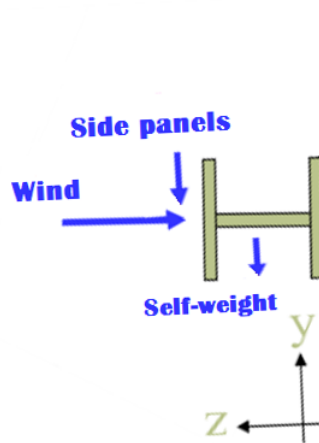


Figure 2.9: Force on the girts

We consider that continuity is ensured along the all length of the hall and that girts are installed every one meter, which suits for a large panel of cladding technology.

Multibeams Arval of type C are chosen for their lightness and their ability to constitute continuous beams.

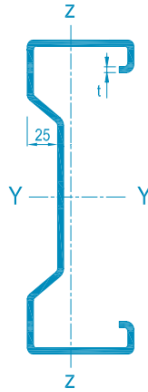


Figure 2.10: Chosen section for girts

The girt is simulated on SCIA as a statically undetermined beam and maximal bending moment in both axis are found to determine the right profile:

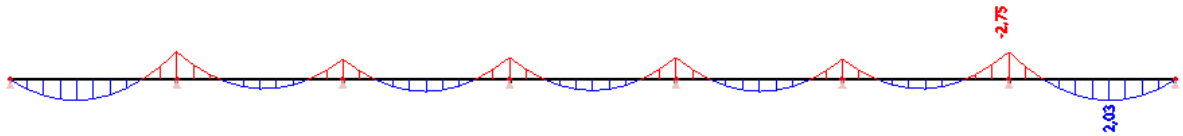


Figure 2.11: Model of the girt with maximal bending moment due to the wind

The minimal section modulus is found:

$$M_{Ed} \leq \frac{W_{el,z} \cdot f_y}{(\gamma = 1)} \rightarrow W_{el,z} \geq \frac{M_{Ed}}{f_y} \quad (2.24)$$

Steel grade	$M_{max,z}$ [kN.m]	$W_{min,z}$ [cm ³]	$M_{max,y}$ [kN.m]	$W_{min,y}$ [cm ³]	Required profile	Mass [kg/m]
S350	2.75	7.86	1.27	3.63	C260150	6.08
S390	2.75	7.05	1.27	3.26	C260150	6.08

With the same method than in section 2.4.1, the deflection criterion is checked:

Profile	f_y [Mpa]	Deflection y [cm]	Deflection z [cm] < 3 cm
C260150	350 or 390	0.15	4.4
C260250	350 or 390	0.1	2.77

As the deflection is the most restrictive factor, the result is similar from one steel grade to another. In order to reduce the price, C260250 profile in S350 steel is finally selected. It has a linear mass of 9.93 kg/m.

Members must sustain the following normal forces:

- In traction: $N = 6.42 \text{ kN}$
- In compression: $N = 2.55 \text{ kN}$

Let's compute the $N_{Rd, \text{traction}}$ and $N_{Rd, \text{compression}}$ of the A170200 profile (in S350 steel) that was previously chosen for purlins.

$$N_{Rd, \text{traction}} = \frac{A \cdot f_y}{\gamma_{M0} = 1} = 6.75 \cdot 10^{-4} \cdot 350 \cdot 10^6 = 236.25 \text{ kN} \gg 6.42 \text{ kN} \quad (2.25)$$

The following method is used to take the buckling into account [14]:

$$L_f \rightarrow N_{cr} \rightarrow \lambda \rightarrow \alpha \rightarrow \phi \rightarrow \chi \rightarrow N_{b, Rd} \quad (2.26)$$

Further explanations are given in the section 2.6.1.

Steel grade	L_f [m]	N_{crit} [kN]	$\bar{\lambda}$	bending curve	α	ϕ	χ	$N_{b, Rd}$ [kN]
S350	6	17.51	3.67	c	0.49	8.1	0.065	15.43 > 11.09

The lateral load transfer is thus ensured by profiles that were previously chosen as purlins.

Bracing

For computations, we consider that bracing elements only take traction.

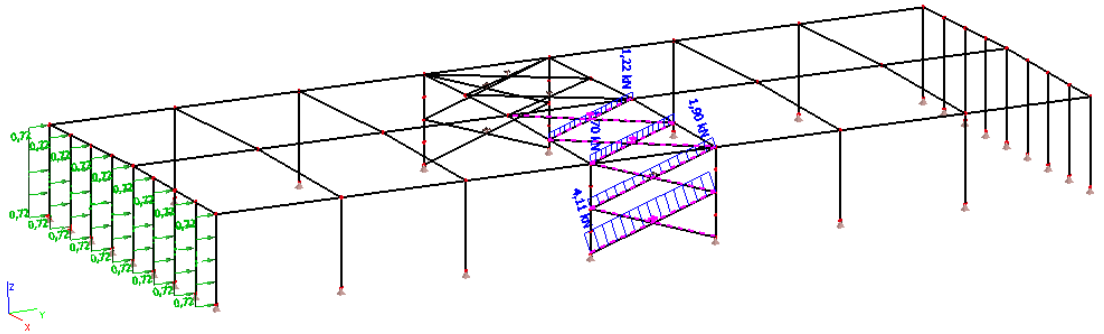


Figure 2.14: Normal forces on bracing system under wind loads

Members must sustain a traction force of 4.11 kN. We choose L shape profiles from ArcelorMittal catalogue [4]. It is now possible to find the required area:

$$\begin{aligned}
 N_{Sd} &\leq N_{pl, Rd} = \frac{A f_y}{\gamma_{m0} = 1} \\
 A &= \frac{N_{Sd}}{f_y} = \frac{4.11 \cdot 1000}{355 \cdot 10^6} \\
 &= 18 \text{ (S235) or } 12 \text{ (S355) } mm^2
 \end{aligned} \quad (2.27)$$

The smallest L profile is adequate for both steel grades offered by manufacturer: L20x20x3 with an area of 112 mm^2 and a linear mass of 0.879 kg/m .

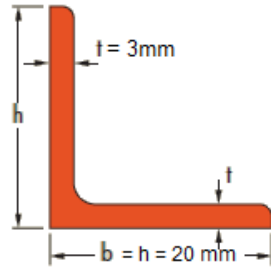


Figure 2.15: Chosen profile

2.4.4 End-wall columns

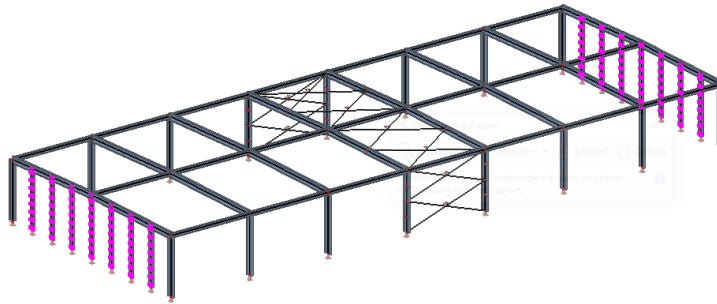


Figure 2.16: End-wall columns

End-wall columns must sustain the following loads (which have to be weighted by suitable coefficients):

- Their own self-weight
- The weight of the siding panels: 0.15 kN/m^2
- The wind perpendicular to the plane of the frames: 0.675 kN/m^2

In order to suit every siding panels types, they are positioned every 2 meters.

Deflection condition

The column is considered as a beam on two pinned supports so it has a maximal span L of 4.16 m and an equal buckling length L_f . Deflection in the middle of the beam equal to:

$$\begin{aligned} W &= \frac{5 q L_f^4}{384 EI} \leq \frac{L}{200} \\ I &\geq \frac{5 \cdot 200 q L^3}{384 E} \\ &= 180.78 \cdot 10^4 \text{ mm}^4 \end{aligned} \quad (2.28)$$

It corresponds to a IPE 120 AA profile (which has a mass of 8.4 kg/m). Catalogue [4] learns us that, in bending, the section belongs to class 1. Thus it can be calculated plastically.

Bending moment

$$M_{Ed} = \frac{qL^2}{8} = 4.38 \text{ kNm} \leq \frac{W_{pl,y} \cdot f_y}{(\gamma = 1)} \quad (2.29)$$

Steel grade	$M_{Rd}[\text{kNm}]$	Result
S235	9.80	$M_{Rd} \geq M_{Ed}$
S355	14.80	$M_{Rd} \geq M_{Ed}$

Normal force

$$N = \left(\frac{8.4 \cdot 9.81}{1000} + 0.15 \right) \cdot 4.16 \cdot (\gamma_G = 1.35) \cdot 2 = 2.61 \text{ kN} \quad (2.30)$$

Simple compression

$$N_{Sd} = 2.61 \text{ kN} \ll N_{Pl}, R_d = A f_y / \gamma_{M0} = (10.7 \cdot 10^{-4} \cdot 235 \cdot 10^6) / 1 = 251.45 \text{ kN} \quad (2.31)$$

Buckling

$$N_{b,Rd} = \chi \frac{A \cdot f_y}{\gamma_{M1} = 1} \quad (2.32)$$

$N_{crit} = \frac{\pi^2 \cdot EI_z}{L_f^2}$ is computed. It appears that:

$$\frac{N_{Ed}}{N_{crit}} = 0.10 > 0.04 \quad (2.33)$$

Hence, the buckling effect may not be ignored.

Steel grade	N_{crit} [kN]	$\bar{\lambda}$	bending curve	α	ϕ	χ	$N_{b,Rd}$ [kN]
S235	25.27	3.15	c	0.49	6.19	0.087	$21.8 \gg 2.61$
S355	25.27	3.88	c	0.49	8.92	0.059	$22.42 \gg 2.61$

These last conditions gives the IPE 120 AA profile the green light no matter the steel grade. For the sake of price reduction, S235 steel is selected.

2.5 Combinations of actions applied to the frame

The determination of the stresses affecting the frame requires to define the combinations of the following actions: permanent loads, imposed loads, snow, wind on the long-pan, wind on gables. Then, the most unfavorable combinations must be determined in order to properly choose the sections of the profiles constituting the frame.

Permanent, imposed and snow loads are calculated as above. Furthermore, different wind directions must be taken into account in order to compute wind actions. Hence, we have 4 wind cases:

- Wind 1: Parallel wind with positive internal coefficient
- Wind 2: Parallel wind with negative internal coefficient
- Wind 3: Perpendicular wind with positive internal coefficient
- Wind 4: Perpendicular wind with negative internal coefficient

The four diagrams below show the pressures on each part of the building :

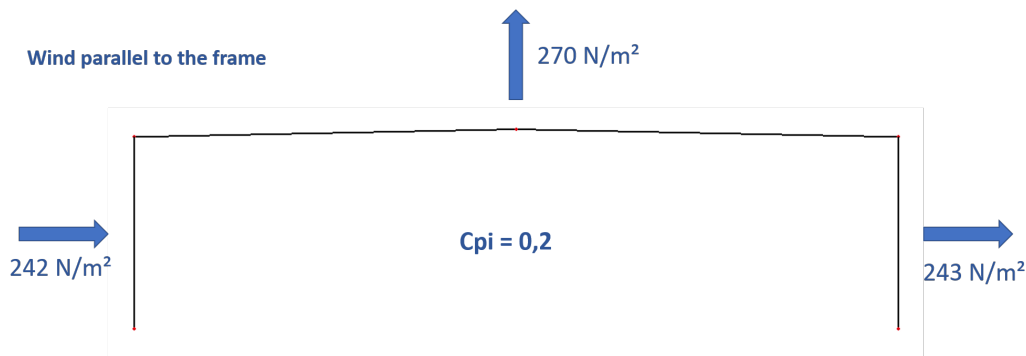


Figure 2.17: Parallel wind with positive internal coefficient

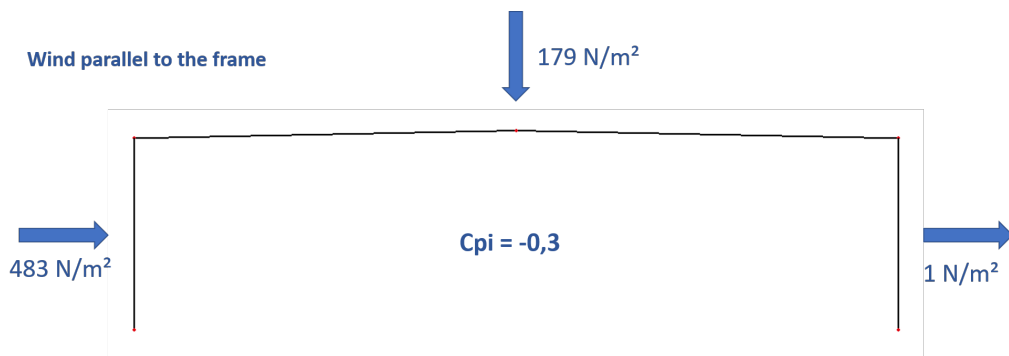


Figure 2.18: Parallel wind with negative internal coefficient

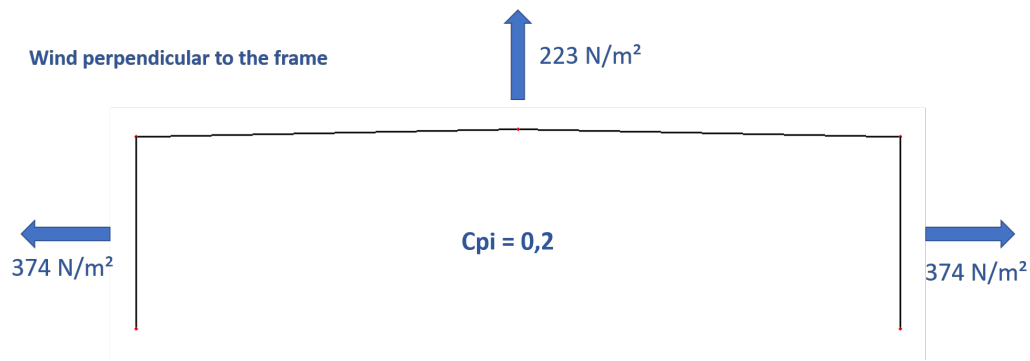


Figure 2.19: Perpendicular wind with positive internal coefficient

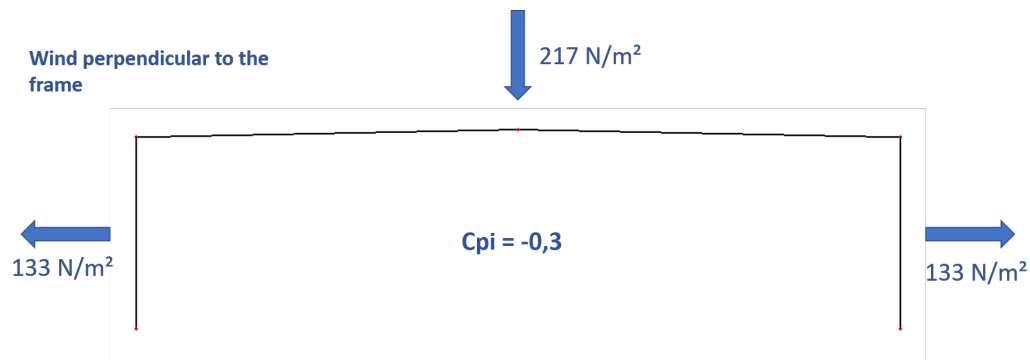


Figure 2.20: Perpendicular wind with negative internal coefficient

The combinations are the same as in section 2.3. The summary table 2.5 gives the pressures applied on each part of the frame.

Combination	Permanent action	Primary variable action	Accompanying variable actions	Numerical result [kN/m^2]		
				Windward	Girder	Leeward
ULS 1	$1.35 \cdot G$	$1.5 \cdot Q$	$1.5 \cdot 0.6 \cdot W_{0^\circ,p} + 1.5 \cdot 0.5 \cdot S$	0.435	1.92	$\simeq 0$
ULS 2	$1.35 \cdot G$	$1.5 \cdot Q$	$1.5 \cdot 0.6 \cdot W_{0^\circ,s} + 1.5 \cdot 0.5 \cdot S$	0.22	1.512	-0.22
ULS 3	$1.35 \cdot G$	$1.5 \cdot Q$	$1.5 \cdot 0.6 \cdot W_{90^\circ,p} + 1.5 \cdot 0.5 \cdot S$	-0.12	1.95	-0.12
ULS 4	$1.35 \cdot G$	$1.5 \cdot Q$	$1.5 \cdot 0.6 \cdot W_{90^\circ,s} + 1.5 \cdot 0.5 \cdot S$	-0.34	1.53	-0.34
ULS 5	$1.35 \cdot G$	$1.5 \cdot W_{0^\circ,p}$	$1.5 \cdot 0.5 \cdot S$	0.72	0.97	$\simeq 0$
ULS 6	$1.35 \cdot G$	$1.5 \cdot W_{0^\circ,s}$	$1.5 \cdot 0.5 \cdot S$	0.36	0.3	-0.36
ULS 7	$1.35 \cdot G$	$1.5 \cdot W_{90^\circ,p}$	$1.5 \cdot 0.5 \cdot S$	-0.2	1.03	-0.2
ULS 8	$1.35 \cdot G$	$1.5 \cdot W_{90^\circ,s}$	$1.5 \cdot 0.5 \cdot S$	-0.56	0.37	-0.56
SLS 1	G	Q	$0.6 \cdot W_{0^\circ,p}$	0.29	1.1	$\simeq 0$
SLS 2	G	Q	$0.6 \cdot W_{0^\circ,s}$	0.14	0.84	-0.15
SLS 3	G	Q	$0.6 \cdot W_{90^\circ,p}$	-0.08	1.13	-0.08
SLS 4	G	Q	$0.6 \cdot W_{90^\circ,s}$	-0.22	0.87	-0.22

Table 2.5: Combination of actions applied to the portal frame.

2.6 Structural system: pinned base portal frame

The portal-frame structural system is composed with columns and inclined beams, connected to one another by assemblies that transmits the bending moment. Columns are connected to the foundations by a joint which can be considered as a pinned connection. Thus, this rigid portal frame structure is stable in its plane. This type of structure is convenient for hot-rolled steel and welded built-up profiles. This scheme is represented in the figure 2.21

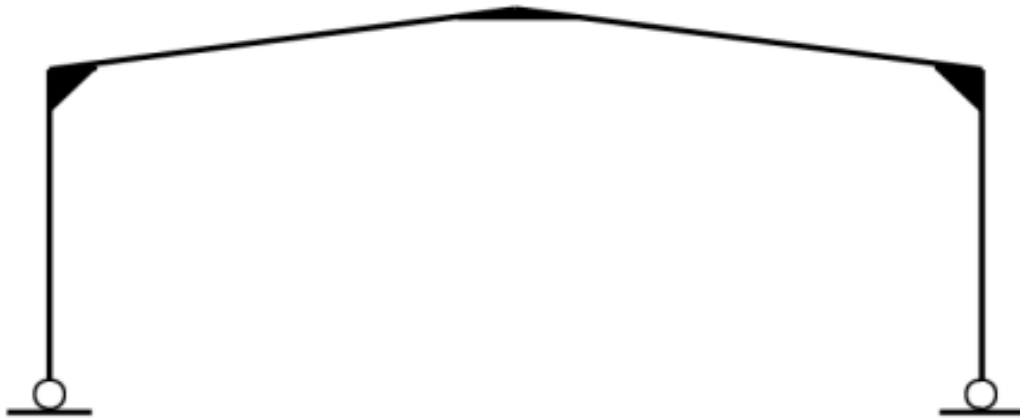


Figure 2.21: Structural system: pinned-base portal frame

2.6.1 Hot-rolled steel

In this section, the weight of the girder is estimated at $0.8 [kN/m]$. The different combinations of loads on the portal-frame are the same as previously where G has been increased by the weight of the rafter.

The profiles chosen in this section will be only of class 1 to allow an elastic-plastic design method. This method is applicable for cross-sections which can reach their plastic strength, without the risk of local buckling, and having a limited (Class 2) or large (Class 1) rotation capacity. Thus, the ruin of a structural element is considered as soon as the stress induced by the actions on the building exceeds the plastic resistance. This corresponds in the reality to the appearance of a plastic hinge at the point of maximal stress.

The elastic-plastic method was used instead of the plastic-plastic design method which is only significant for portal frames with no deflection criterion. In the case of industrial halls, this assumption is not relevant since portal frames must have limited deflections, in particular to ensure a good dynamic behavior under, among others, wind loads.[20]

Since the distance between the portals is 6 m, the following values in $[kN/m]$ are found for each portal frame:

Combination	Numerical application $[kN/m^2]$		
	Windward column	Girder	Leeward column
ULS 1	2.61	12.32	$\simeq 0$
ULS 2	1.32	9.872	-1.32
ULS 3	-0.72	12.5	-0.72
ULS 4	-2.04	9.98	-2.04
ULS 5	4.32	6.62	$\simeq 0$
ULS 6	2.16	2.6	-2.16
ULS 7	-1.2	6.98	-1.2
ULS 8	-3.36	3.02	-3.36
SLS 1	1.74	7.76	$\simeq 0$
SLS 2	0.84	7.58	-0.9
SLS 3	-0.48	7.58	-0.48
SLS 4	-1.32	6.02	-1.32

Internal forces in the portal frame

To locate the internal forces in the different parts of the portal frame it was decided to call them as in the following figure 2.22



Figure 2.22: Localization of the internal forces

Now, the following values need to be evaluated:

- Support reactions: HA, HE, VA, VE.
- The maximum bending moments: MB, MC, MD.

These solicitations are determined from the different combinations of loads that were just computed and thanks to ISSD, which is a structural calculation software. They are found in the table 2.6

Combination	Support reactions [kN]				Moments [kN.m]		
	Ha	Va	He	Ve	Mb	Mc	Md
ULS 1	48	97	58	100	213	161	234
ULS 2	39	78	50	80	168	130	189
ULS 3	58	100	58	100	226	165	226
ULS 4	49	80	49	80	180	132	180
ULS 5	17	51	34	55	103	86	137
ULS 6	3	19	20	23	30	34	64
ULS 7	34	56	34	56	126	92	126
ULS 8	20	24	20	24	54	41	54

Table 2.6: Values of the internal forces

Verification of the bending stresses

Using the table 2.6, we find the following maximum bending moments:

- At the joint between the girder and the column: 226 [kNm]
- At the ridge: 165 [kNm]

Then the bending resistance is checked:

$$\sigma \leq \sigma_r \quad (2.34)$$

$$\frac{M}{W_{pl}} \leq \frac{M_{pl,rd}}{W_{pl}} \quad (2.35)$$

$$\text{With } M_{pl,rd} = \frac{W_{pl} \cdot f_y}{\gamma}$$

$$\frac{M}{f_y} \leq W_{pl} \quad (2.36)$$

The value found can be used to select a profile which resists to the bending moment based on information found in the product catalogue "Sections and Merchant Bars Profil" of ArcelorMittal[4].

$f_y [MPa]$	$W_{pl} [\cdot 10^3 mm^3]$	Chosen profile
235	961	IPE 360
355	637	IPE 330
460	491	IPE 300

The shear stress is also computed and compared with the shear strength of the selected sections:

$$V_{rd} = \frac{f_y \cdot A_{vz}}{\sqrt{3}} = 99 [KN] \quad (2.37)$$

Chosen profile	$f_y [MPa]$	$V_{ed} [KN]$
IPE 360	235	476
IPE 330	355	526
IPE 300	460	587

Non lateral buckling condition for the girder

The upper flange of the rafter, on the one hand, cannot buckle under positive bending moment (due to downward forces) because it is immobilized laterally, blocked by the purlins. On the other hand, the lower flange could be subject to buckling, under negative bending moment. But, as indicated in the table 2.5 which shows the different combinations of actions, under no circumstances the wind forces are sufficient to cause an uplifting.

Column resistance verification

To check the resistance of the columns the same profiles than for the girders are adopted as a first assumption. They will be checked taking buckling into account. As a reminder, the profiles were the following:

$f_y [MPa]$	Chosen profil
235	IPE 360
355	IPE 330
460	IPE 300

$$\frac{N_{Ed}}{N_{b,Rd}} + \frac{M_{Ed}}{M_{Rd}} \leq 1 \quad (2.38)$$

All the columns have been calculated using the same following calculation method:

$$L_f \rightarrow N_{cr} \rightarrow \lambda \rightarrow \alpha \rightarrow \phi \rightarrow \chi \rightarrow N_{b,Rd} \quad (2.39)$$

1. Calculation of the buckling length L_f according to support conditions. In the lowest inertia rotary axis, both ends are pinned. The foot of the column is held by the assembly of the foundation and the head of the column by the eave strut.

$$L_f = 4 [m]$$

2. Calculation of the Eulerian critical load.

$$N_{cr} = \frac{\pi^2 \cdot E \cdot I}{L_f^2}$$

3. Calculation of the reduced slenderness.

$$\lambda_{LT} = \sqrt{\frac{A \cdot f_y}{N_{cr}}}$$

4. Calculation of the appropriate imperfection factor α . The buckling curve needs to be determined according to the geometry of the profile (see figure 2.23). Then, α is evaluated with the following table 2.7.

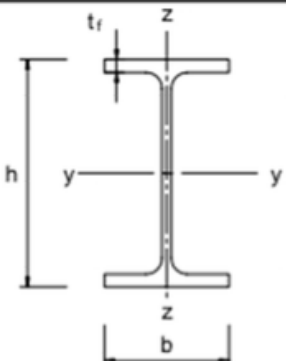
Cross section		Limits	Buckling about axis	Buckling curve	
				S 235 S 275 S 355 S 420	S 460
	$h/b > 1,2$	$t_f \leq 40 \text{ mm}$	y-y z-z	a b	a_0 a_0
		$40 \text{ mm} < t_f \leq 100$	y-y z-z	b c	a a
	$h/b \leq 1,2$	$t_f \leq 100 \text{ mm}$	y-y z-z	b c	a a
		$t_f > 100 \text{ mm}$	y-y z-z	d d	c c

Figure 2.23: Buckling curve regarding geometry of the profile

Buckling curve	a_0	a	b	c	d
Imperfection factor α	0.13	0.21	0.34	0.49	0.76

Table 2.7: Imperfection factor in function of buckling curve

5. Calculation of ϕ

$$\phi_{LT} = 0.5(1 + \alpha_{LT}(\lambda_{LT} - 0.2) + \lambda_{LT}^2)$$

6. Calculation of χ

$$\chi_{LT} = \frac{1}{\phi_{LT} + \sqrt{\phi_{LT}^2 - \lambda_{LT}^2}}$$

7. Calculation of the design buckling strength.

$$N_{b,Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}}$$

Where $\gamma_{M1} = 1$

A summary of the results for the selected profiles can be found in the table 2.8

$f_y[MPa]$	Profile	N_{cr}	λ_{LT}	α_{LT}	ϕ_{LT}	χ_{LT}	$N_{b,Rd}[KN]$
235	IPE 360	1351	2,12	0,34	1.29	0,52	1708.45
355	IPE 330	1020	1.47	0,34	1.8	0,35	2074
460	IPE 300	782	1.77	0,13	2.18	0,29	2365

Table 2.8: Results for the chosen profiles

With the calculated values of $N_{b,Rd}$ and M_{Rd} the condition (2.38) can be verified. The table 2.9 summarizes the checks for the three steel grades.

$f_y[MPa]$	Profile	$\frac{N_{Ed}}{N_{b,Rd}}$	$\frac{M_{Ed}}{M_{Rd}}$	$\frac{N_{Ed}}{N_{b,Rd}} + \frac{M_{Ed}}{M_{Rd}} \leq 1$
235	IPE 360	0.058	0.94	$\simeq 1$
355	IPE 330	0,048	0.79	0.84
460	IPE 300	0.042	0.78	0.83

Table 2.9: Condition 2.38 verification

Comparison of analytic results with Portal+ numerical calculation

In order to verify the results calculated by hand, Portal+ was used. It is a software for the pre-sizing of portal frames in accordance with Eurocode 3 [10]. It was developed by ArcelorMittal in partnership with CTICM. The aim of this comparison is to validate the use of the software for later calculations. Indeed, different hall sizes must be studied as explain in the chapter 3. Hence, the use of a numerical method for hot-rolled steel design will save lots of time. First, the internal forces will be compared under the most critical combination which is the ULS 4 combination. Secondly, the sectional strength criteria $\Gamma_M = \frac{M_{Ed}}{M_{Rd}}$ and $\Gamma_N = \frac{N_{Ed}}{N_{b,Rd}}$ are verified for the IPE 300 and S355 steel.

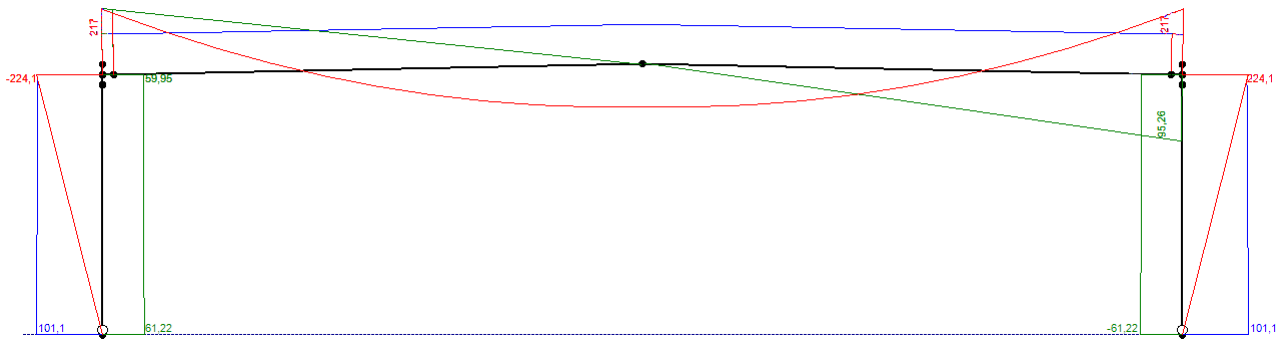


Figure 2.24: Internal forces diagram obtained with Portal+

	Hand calculation	Portal+	Relative error
Ha	58	61.2	5%
Va	100	101.1	1%
He	58	61.2	5%
Ve	100	101.1	1%
Mb	226	224.1	$\leq 1\%$
Mc	165	151.4	8%
Md	226	224.1	$\leq 1\%$

Table 2.10: Comparison between internal forces calculated by hand and those obtained by Portal+

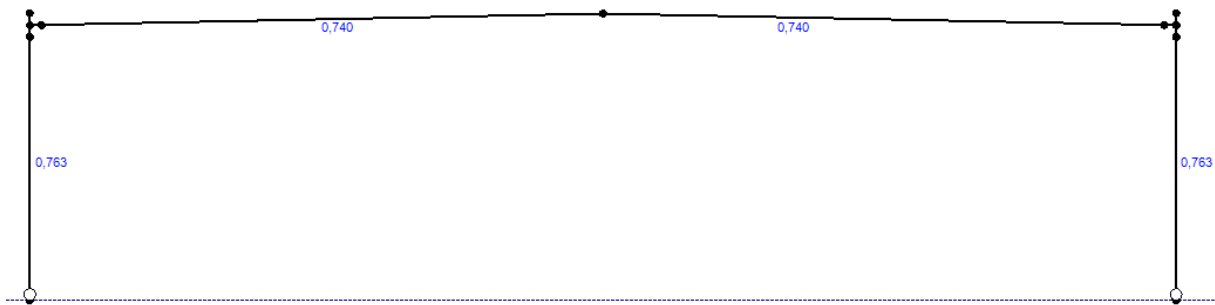
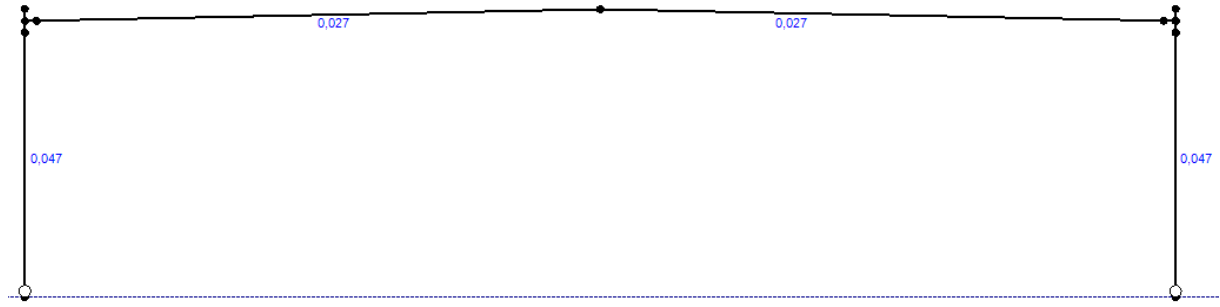


Figure 2.25: Γ_M for IPE 300 S355

Figure 2.26: Γ_N for IPE 300 S355

	Hand calculation	Portal+	Relative error
Γ_M	0.79	0.74	6%
Γ_N	0.048	0.047	2%

Table 2.11: Comparison between sectional strength criteria calculated by hand and those obtained by Portal+

We do not observe relative error greater than 8 % for the internal forces and 6 % for the resistance criteria, as shown in the tables 2.10 and 2.11 . We may now consider the results from Portal+ as reliable. In the next section, the software is used for calculation of the maximum deflection.

Deflection verification

For the deflection verification, Portal+ is also used. The deflection condition $L/200$ is not respected with the current profiles, so the section have to be increased. The following three figures show the results of deformation calculation with the adapted profiles and for every steel grades. Now, the deflection condition is fully respected.

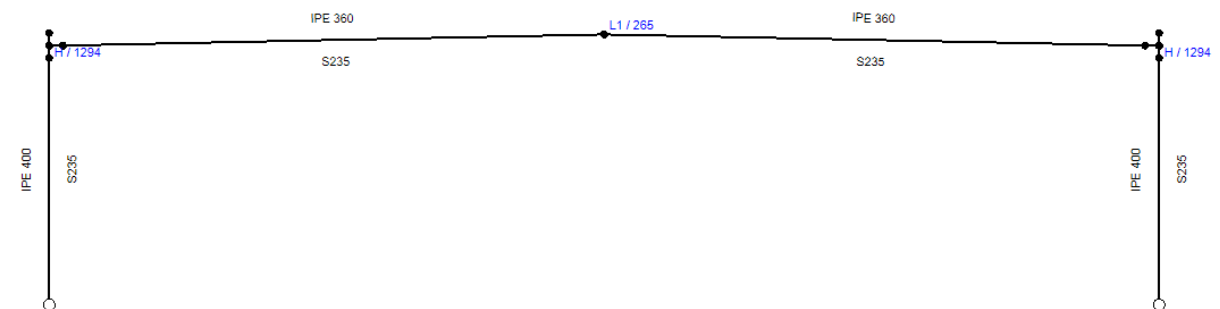


Figure 2.27: Deflection and selected profiles for steel grade S235

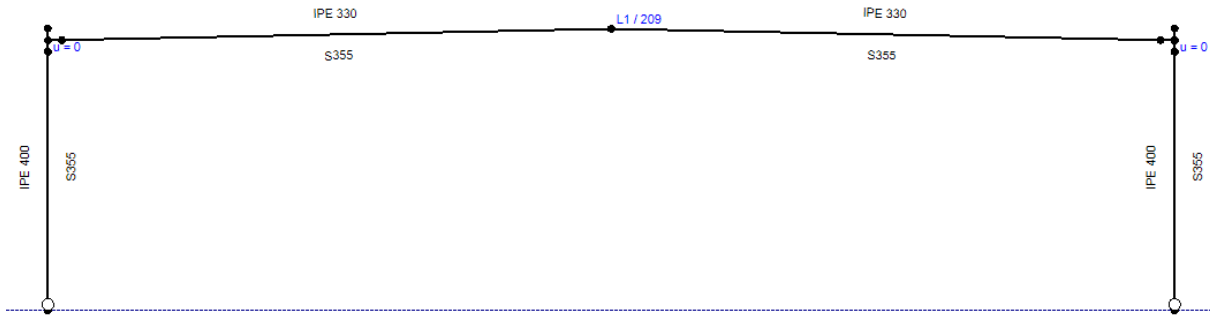


Figure 2.28: Deflection and selected profiles for steel grade S355

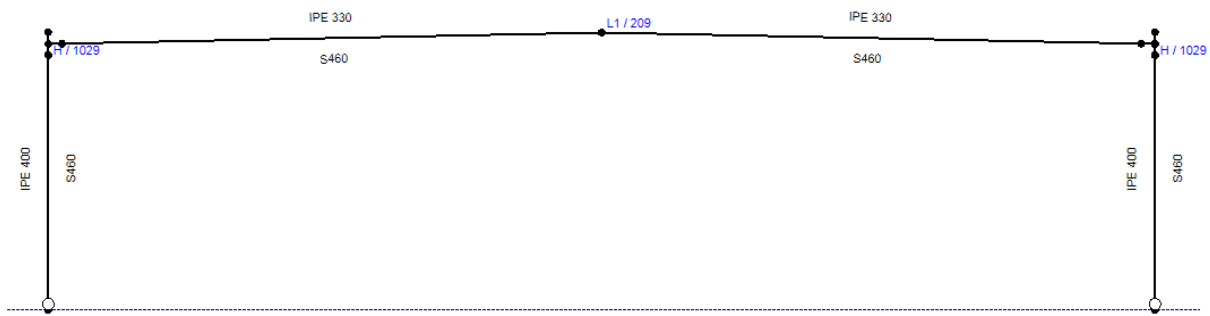


Figure 2.29: Deflection and selected profiles for steel grade S460

The following table 2.12 summarizes the different chosen hot-rolled profiles for all the steel grades and for the first hall dimension.

Steel	Element	Profile	[kg/m]	[kg]	[t]
S235	Columns	IPE400	66.3	530.4	1.44384
	Girders	IPE360	57.09	913.44	
S355	Columns	IPE400	66.3	530.4	1.3168
	Girders	IPE330	49.15	786.4	
S460	Columns	IPE400	66.3	530.4	1.3168
	Girders	IPE330	49.15	786.4	

Table 2.12: Summary of the profiles and masses for one frame

2.6.2 Welded built-up frame

Variable section elements allows the project engineer to optimize the structure. Indeed the inertia can be variable along the members to suit the varying stress rather than designing to the extreme solicitations which irremediably leads to weight, price and environmental impact supplement. In order to ensure that components can actually be built, some restrictions comes naturally:

- The flange dimensions and the thickness of the web must remain constant along the element to reduce the price of manufacturing.
- The height of the section must vary linearly for the same reason.
- The web must be of class 3 in order to ensure the optimization of the design.

As a first step we calculate the envelope of internal forces under the same loads than before. SCIA software allows us to automatically take a large amount of load cases into account. Thus, we find the three envelopes (bending moment, normal force and shear force):

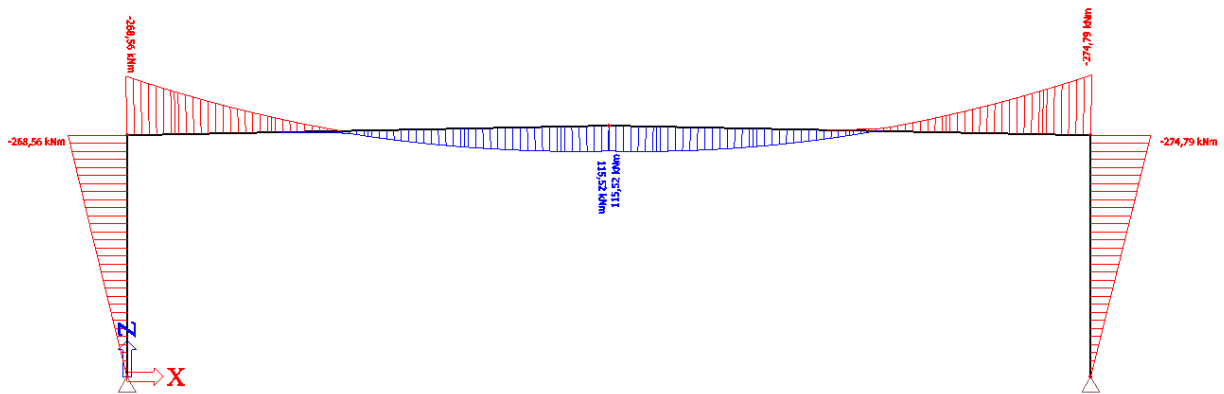


Figure 2.30: Bending moment envelope

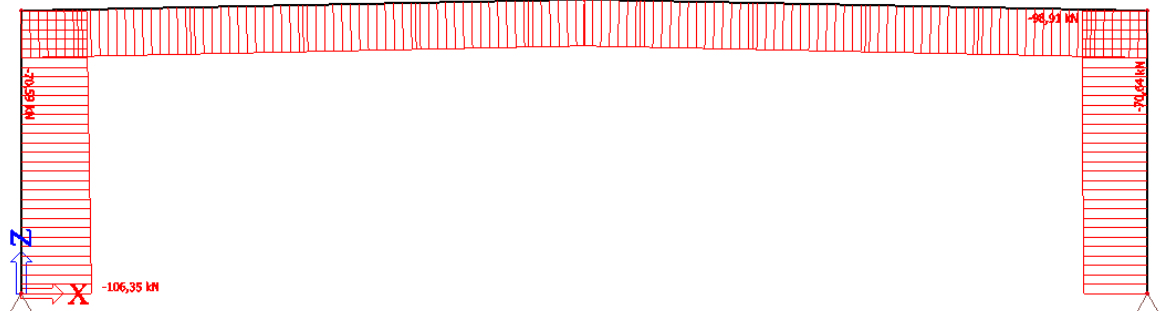


Figure 2.31: Normal force envelope

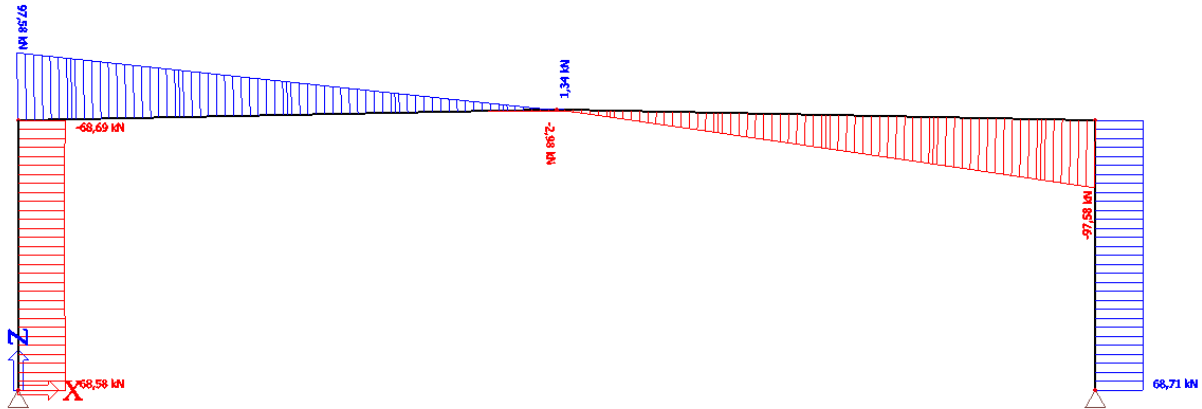


Figure 2.32: Shear force envelope

In order to design the frame, a few steps are required:

- Some hypothesis about the geometry of the section must be taken.
- The bottom of the column isn't subjected to any bending moment as it is actually pinned. Thus, it must be designed to the normal force and shear force.
- Next, the structure is calculated according to the bending moment solicitation.
- The obtained design is then checked regarding the normal force, shear force deflection and (lateral) buckling.

Let's notice that welded built-up sections are designed elastically since geometries are chosen to remain in class 3.

Geometry hypothesis

On the figure 2.33, reader will find hypothesis that were made. These flanges dimensions are based on a IPE 240 profile that is necessary to withstand the minimal internal forces on the structure while ensuring a class 3 web.

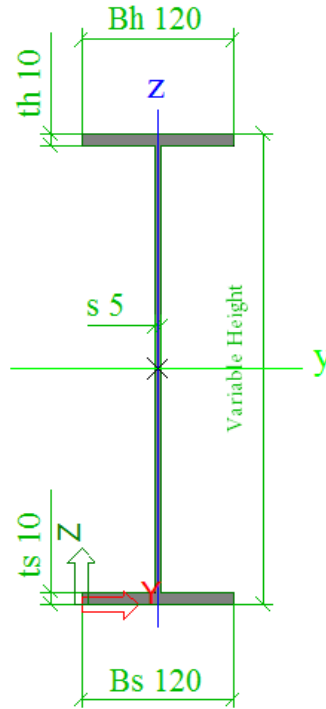


Figure 2.33: Hypothesis on the geometry of the welded built-up section

Bottom end design

The table here under presents the maximal forces applied on the bottom of the column.

Maximal internal stress	Normal force [kN]	shear force [kN]
Positive	-	68.58
Negative	-106.35 (compression)	-68.71

We are looking for the height of the profile at the bottom end of the column:

Solicitation	Equations	Required height [mm]		
		S235	S355	S460
Compression:	$N_{b,Rd} = \chi \frac{A \cdot f_y}{\gamma_{M1}}$	110		
Shear force:	$\frac{\tau_{Ed}}{f_y / (\sqrt{3} \gamma_{Mo})} \leq 1.0$ and $\tau_{Ed} = \frac{V_{Ed}}{A_w}$ ¹	120	90	70

We see that a very thin section is required at the base of the structure. In order to ensure a class 3 web, we must take the following minimal height:

Steel grade	Minimal Height [mm]
S235	452
S355	372
S460	329

¹Following Eurocode 3 for elastic calculation as $\frac{A_f}{A_w} \geq 0.6$

bending moment design

$$M_{Ed} \leq M_{el,Rd} = \frac{W_{el} \cdot f_y}{\gamma_{M0}} \quad (2.40)$$

For each Steel grade (S235,S355 and S460) we compute the minimal height of the profile required to withstand the bending solicitation described by the bending moment envelope. Then, taking account of the chosen minimal dimensions, the real height of the member is found.

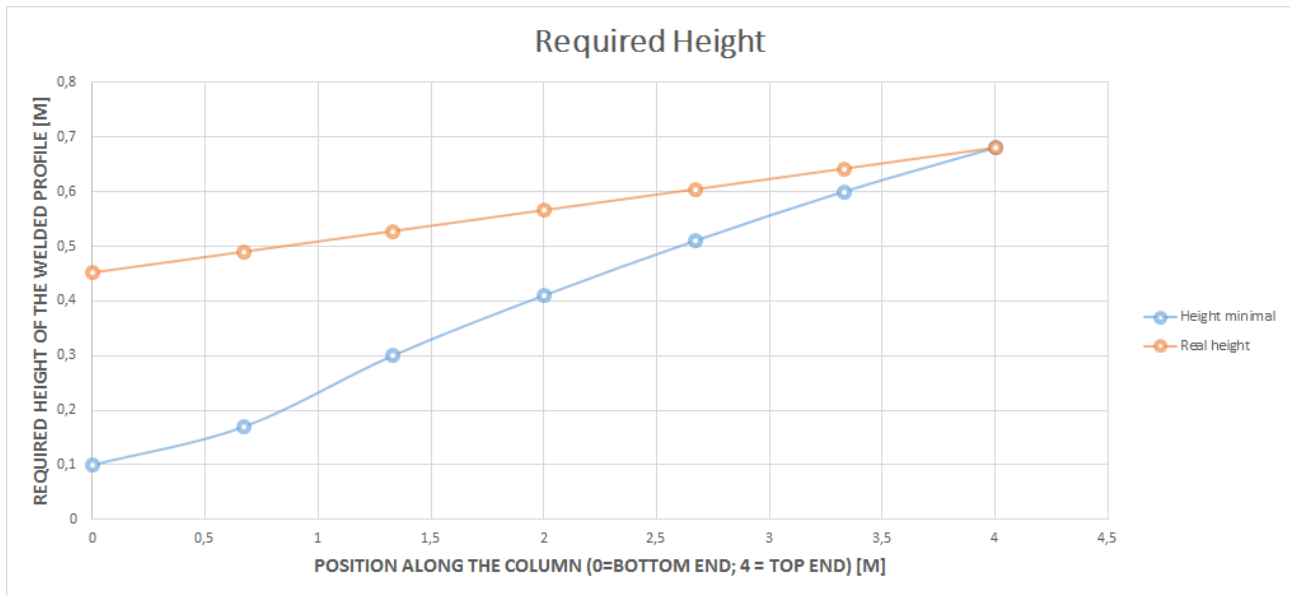


Figure 2.34: Minimal and final height required for the S235 column profile

The process is repeated for the cross beam and sections can finally be characterized.

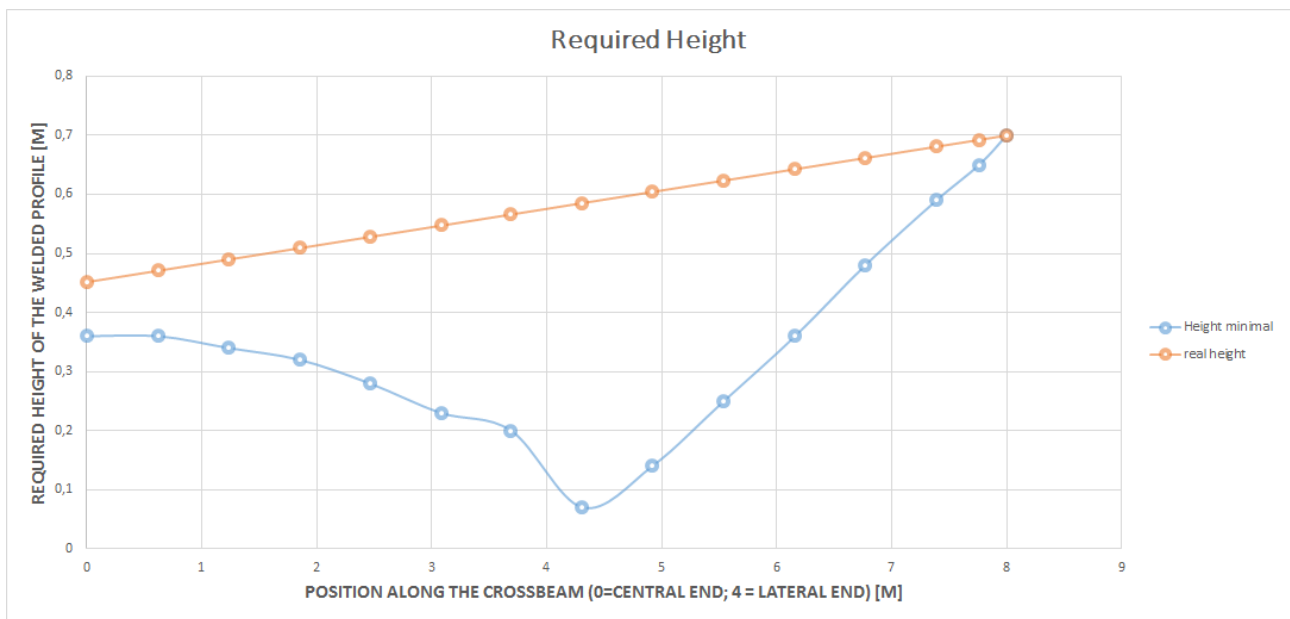


Figure 2.35: Minimal and final height required for the S235 cross-beam profile

Section height [mm]	Column		Cross-beam	
	Bottom end	Top end	central end	lateral end
S235	452	680	452	700
S355	372	500	372	510
S460	329	410	329	420

Class of the web verifications

Web class	Column		Cross-beam	
	Bottom end	Top end	central end	lateral end
S235	3	>3	3	>3
S355	3	3	3	3
S460	3	3	3	3

With S235 steel, ensuring a class 3 web requires a too thin web design. It doesn't provide any advantage in comparison with hot-rolled steel. Yet, the welded built-up profiles are more expensive to produce. Then, the S235 welded built-up solution will not be studied for this size of building (16x42m).

Further verifications

As the used sections are complex, hand calculation of buckling and other verifications would be time consuming and imprecise. Thus, SCIA engineer software is used with this aim in mind.

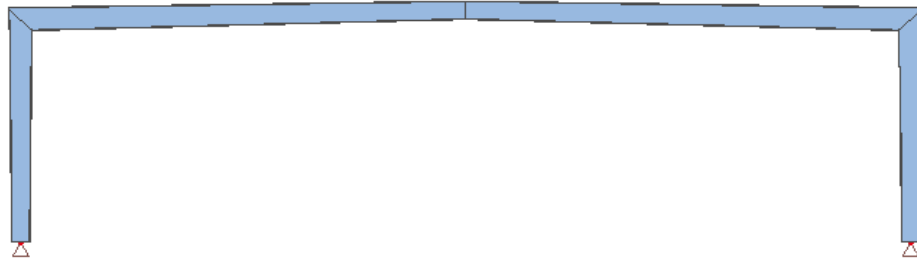


Figure 2.36: Scia design

We must check that our deflection remains under a value required by the norms [12]:

$$\frac{f}{L} \leq \frac{1}{200} \Rightarrow f_{max} < 80 \text{ mm} \quad (2.41)$$

where:

- f is the deformation
- L is the span [m]

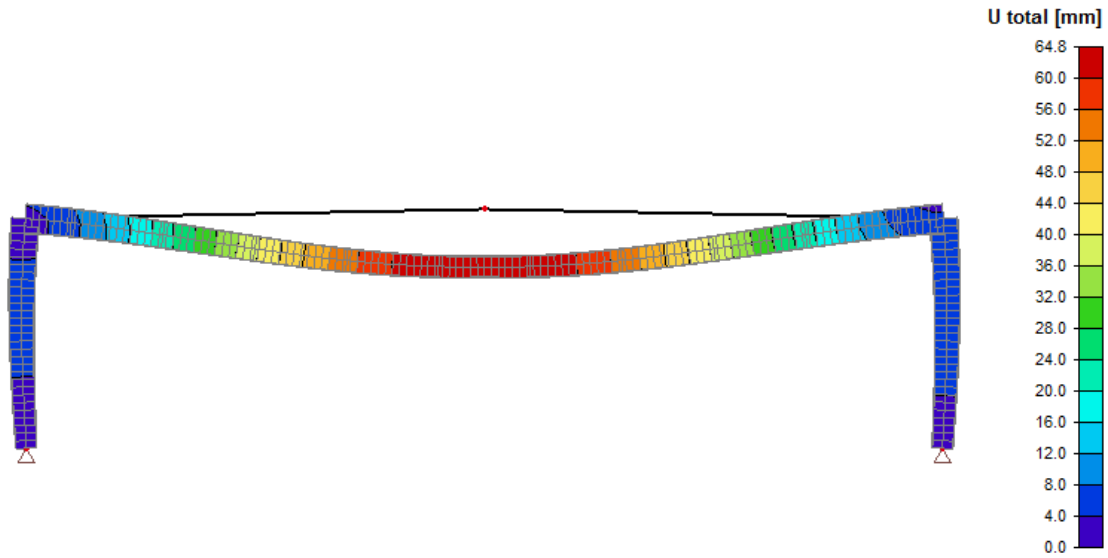


Figure 2.37: Numerically calculated 3D deformations on the S355 structure

In the figure 2.37 reader may notice we find acceptable deformations for S355 steel structure. However, S460 design leads to excessive deformations. We have to gradually increase the dimensions till deformation becomes small enough:

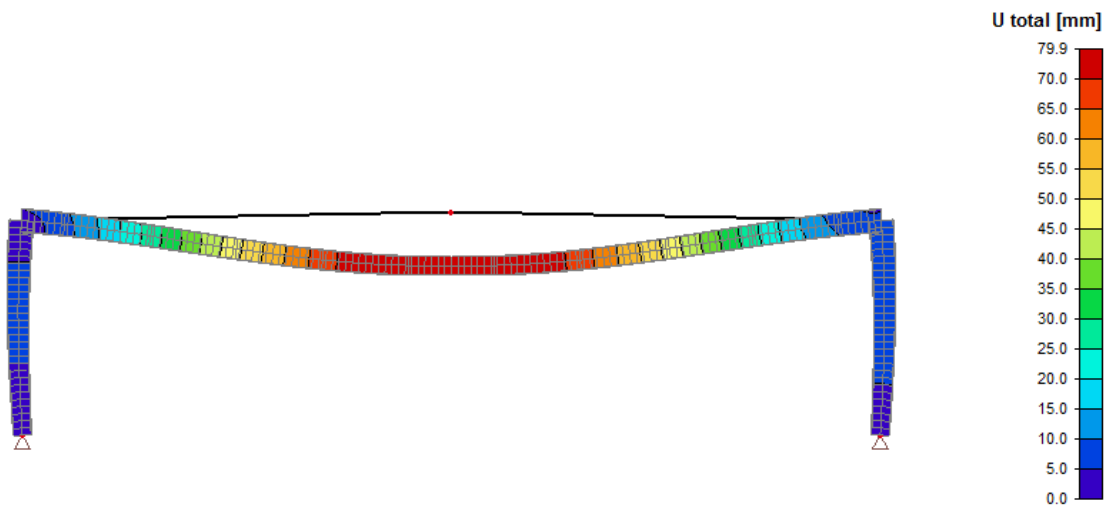


Figure 2.38: Numerically calculated deformations on the S460 structure

Stresses on members are also computed to check that the elastic limit isn't exceeded anywhere on the section taking account of the final self-weight.

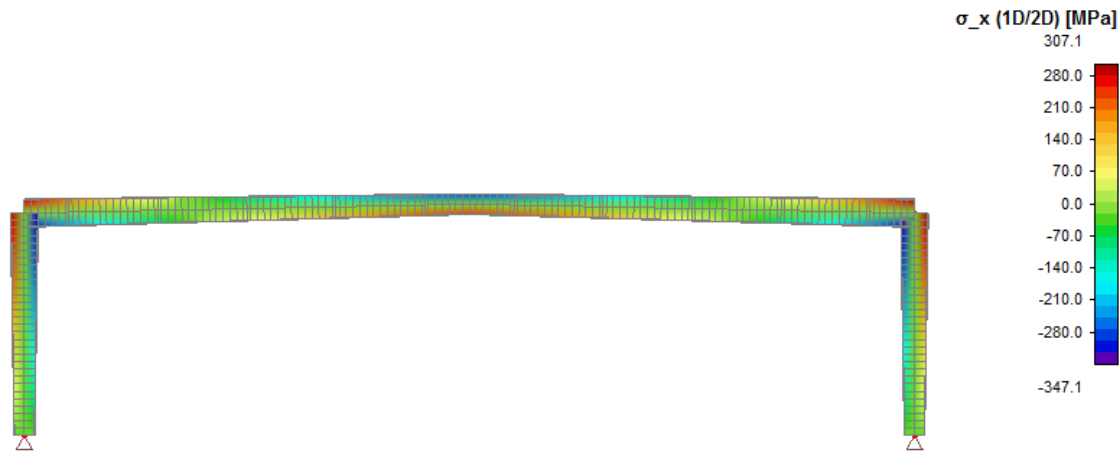


Figure 2.39: Stress analysis of S355 structure

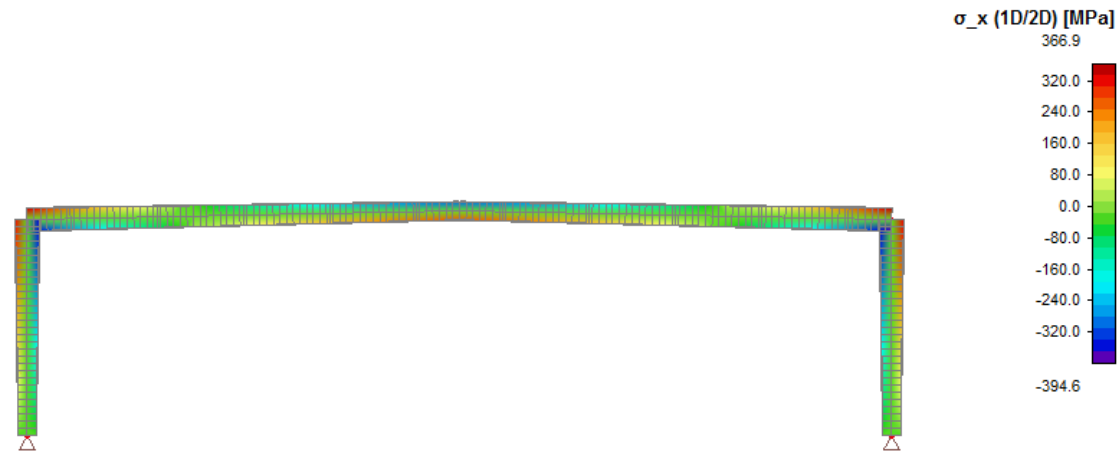


Figure 2.40: Stress analysis of S460 structure

Finally, the results are summarized:

Section height [mm]	Column			Cross-beam			B_h [mm]	t [mm]	s [mm]
	Bottom end	Top end	mass [T]	central end	lateral end	mass [T]			
S355	372	500	0.15	372	510	0.29	120	9.8	5.2
S460	345	460	0.14	350	440	0.28	120	9.8	5.2

2.7 Structural system: rigid base columns, pinned girder

For the precast concrete on one hand, and precast concrete columns associated with glue laminated timber girder on the other hand, a rigid base columns system is used. This scheme is represented in the figure 2.41

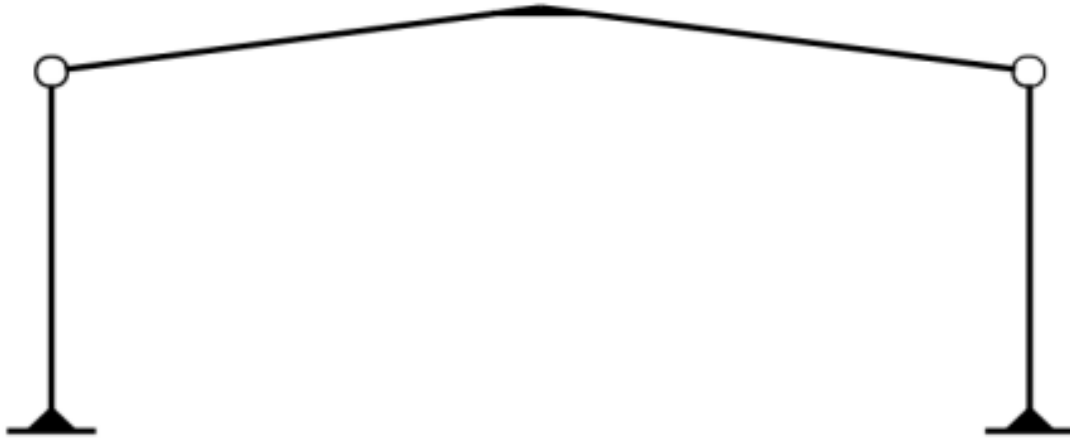


Figure 2.41: Structural system: rigid-based columns, pinned girder

2.7.1 Precast concrete

This solution is entirely designed thanks to manufacturer catalogue. All available sizes and admissible loads are exhaustively provided by concrete companies. Ergon company was chosen because their online commercial documents were more complete.

Cross-beam design

In order to choose the right profile among the wide offering, it is required to evaluate the applied vertical load. However, the catalogue [17] allows us to choose a profile knowing the span and the live load. That's why we don't need to know a priori the self-weight of the beam.

- The ULS 3 loads combination is used since it gives the worst load on the beam:
 $Q = 1.95 \text{ kN/m}^2 \cdot 6 \text{ m} = 11.7 \text{ kN/m}$
- The span is 16 m long.

The concrete girder is designed with different models of profile from the catalogue[17]. The aim is to consider every available possibilities for this span and loads situation in order to compare their mass.

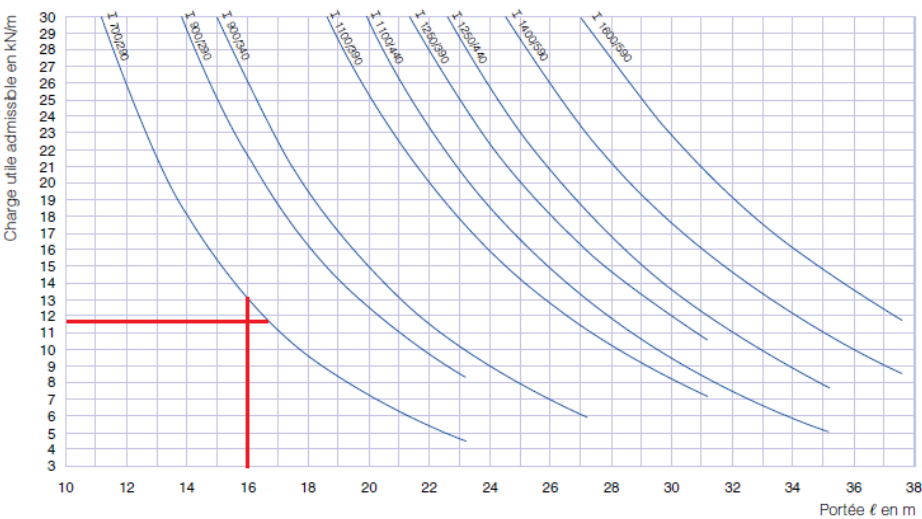


Figure 2.42: I type beams design table

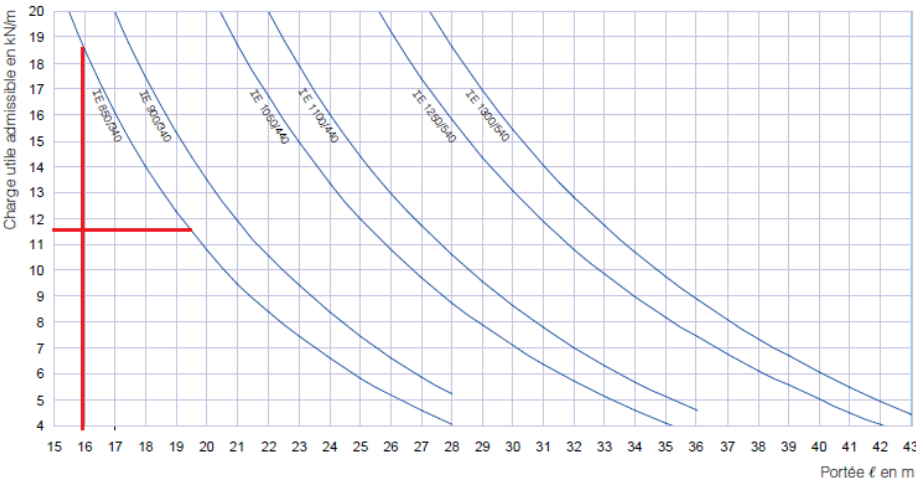


Figure 2.43: IE type beams design table

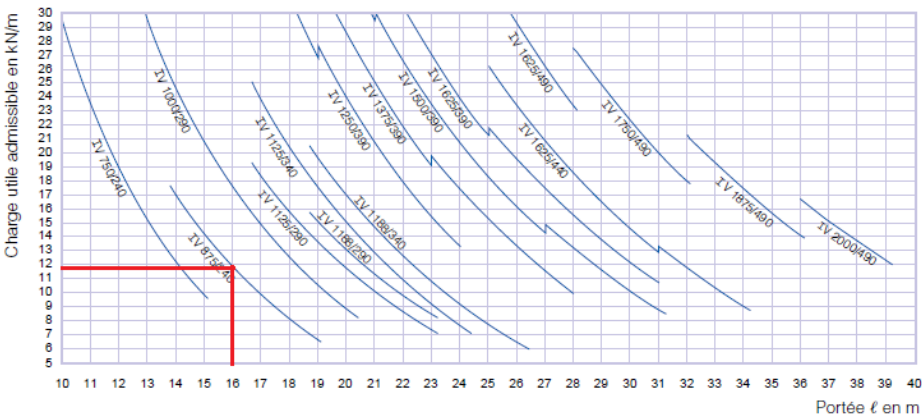


Figure 2.44: IV type beams design table

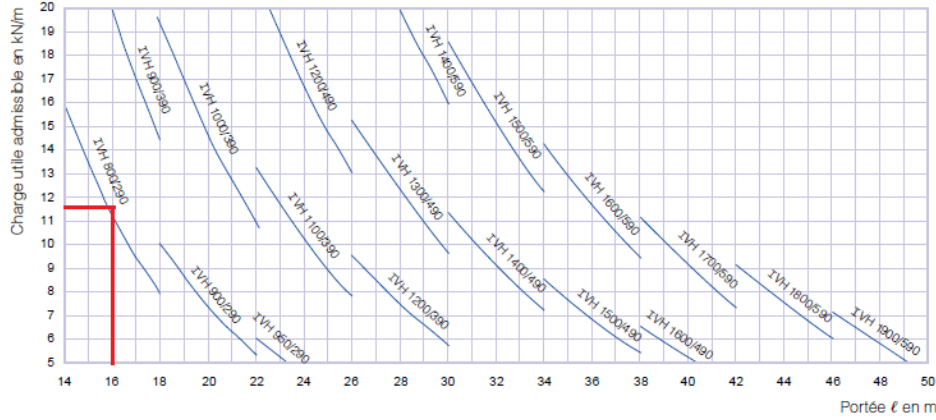


Figure 2.45: IVH type beams design table

Beam profile	Max available length	mass [T]
I 700/290	23.2 > 16	5.68
IE 850/340	27.2 > 16	5.34
IV 875/240	19.1 > 16	3.98
IVH 800/290	18 > 16	4.28

For the sake of mass reduction, which usually leads to environmental impact improvement and makes the on-site handling easier, the IV 875/240 is selected.

Column design

Each column must sustain a normal force and a bending moment. This couple of solicitations will allows us to design the column.

$$\begin{aligned}
 N &= \text{load on the cross beam} + \text{beam self weight} + \text{walls cover} \\
 &= 11.7 \cdot \frac{16}{2} + 1.35 \cdot \frac{3.98 \cdot 9.81}{2} + 1.35 \cdot 0.15 \cdot 6 \cdot 4 \\
 &= 125kN
 \end{aligned} \tag{2.42}$$

$$\begin{aligned}
 M &= \text{wind on the vertical walls} \\
 &= 1.5 \cdot (0.483 \cdot 6) \cdot \frac{4^2}{2} \\
 &= 34.78kNm
 \end{aligned}$$

To find the buckling length, we follow the recommendations from Ergon catalogue [17]:

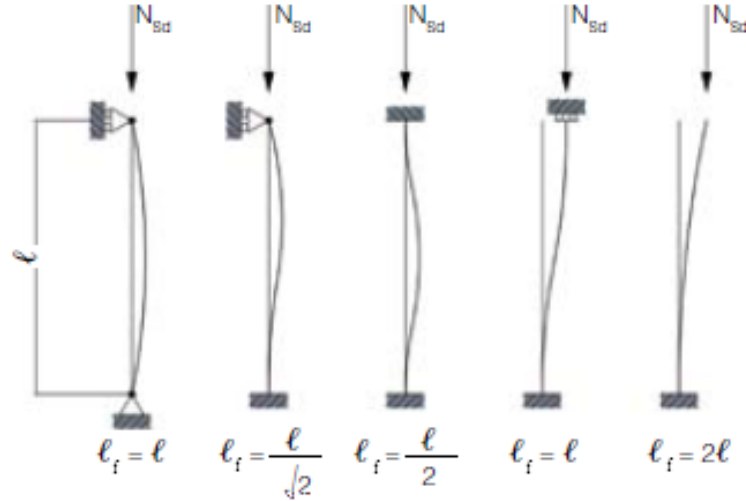


Figure 2.46: Buckling length

As the column is encased into the soil, we must take a buckling length equal to $h/\sqrt{2} = 2.9m$. In the catalogue, the beams are classified according to their buckling lengths. We take the length that is the closest to our situation:

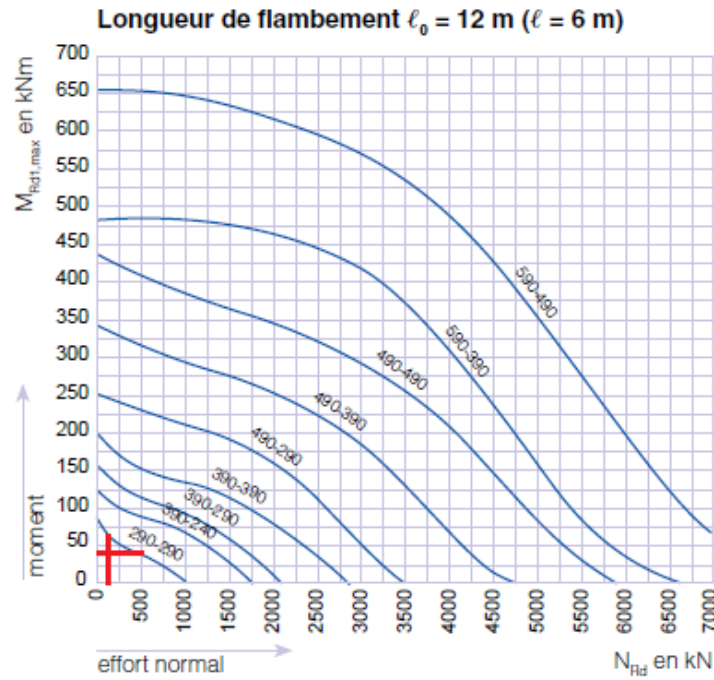


Figure 2.47: CR-RK type columns design table

We select a CR-RK 290-290 profile which has a weight of 2.06 kN/m.

2.7.2 Precast concrete column and glue laminated timber girders

In order to keep a roof slope of 1.15 degrees. It was decided to use a girder with variable inertia. The element is flat on its bottom base and has two slopes at the top side; the center of the rafter being the highest point of the element. Its dimensions can be seen in figure 2.48. For this type of beam four criteria must be checked, all in accordance with the Eurocode 5[23][11]:

- The first one is the verification of the stresses due to the bending in each single decrease zone.
- The second one is also the verification of the stresses due to the bending but in the ridge area.
- The third one is the verification of the stress due to traction perpendicular to the fiber orientation of the wood in ridge area
- A final check will be made for the compression of the fibers at the supports

The following assumptions were made for the verification of the girder:

- Service Class 1: structures protected from the bad weather
- h_{ap} : Height of the ridge = 960 [mm]
- h_s : Height of the end = 800 [mm]
- Thickness: 200 [mm]
- Material: GL28h. All its properties can be found in the table 2.13 here under [5].

Properties	Symbol	[MPa]
Bending resistance	$f_{m,g,k}$	28
Tensile resistance	$f_{t,0,g,k}$	22.4
	$f_{t,90,g,k}$	0.5
Compression resistance	$f_{c,0,g,k}$	28
	$f_{c,90,g,k}$	2.5
Shear resistance	$f_{v,g,k}$	3.5
Elasticity modulus	E_{mean}	12600
Shear modulus	G_{mean}	650
Density	ρ_{mean}	460

Table 2.13: Properties of the glue laminated timber GL28H

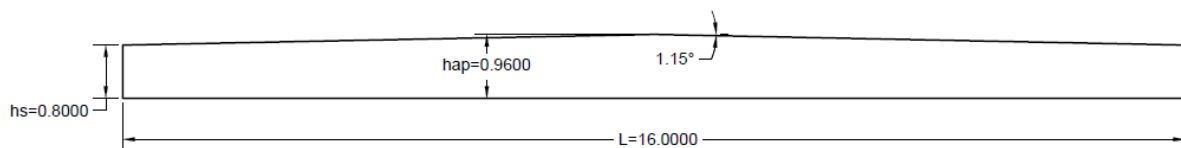


Figure 2.48: Characteristic of the girder

First criteria: verification of stresses (ULS), bending stress in each single decrease zone

$$\frac{\sigma_{m,\alpha,d}}{k_{m,\alpha} \cdot f_{m,d}} \leq 1 \quad (2.43)$$

$\sigma_{m,\alpha,d}$: bending stress induced by the ULS combination of actions

Calculation of the point x where the stress is maximum

$$x = 0.5 \cdot L \cdot \frac{h_s}{h_{ap}} = 6666 \text{ [mm]} \quad (2.44)$$

Calculation of the bending moment at point x

$$M_d(x) = \frac{q \cdot L \cdot x}{2} - \frac{q \cdot x^2}{2} = 3.88 \cdot 10^8 \text{ [KN.m]} \quad (2.45)$$

Calculation of the height of the section at point x

$$\begin{aligned} h(x) &= h_s + \tan(\alpha) \cdot x = 933 \text{ [mm]} \\ \tan(\alpha) &= \frac{h_{ap} - h_s}{0.5 \cdot L} = 0.02 \end{aligned} \quad (2.46)$$

Finally the highest stress is calculated

$$\sigma_{m,\alpha,d} = \frac{6 \cdot M_d}{b \cdot h^2} = 13.4 \text{ [MPa]} \quad (2.47)$$

$f_{m,d}$: bending resistance

$$f_{m,d} = f_{m,k} \cdot \frac{k_{mod}}{\gamma_M} \cdot k_{sys} \cdot k_h = 24.64 \text{ [MPa]} \quad (2.48)$$

- $f_{m,k} = 28$: Characteristic resistance to bending
- $k_{mod} = 1.1$: Reduction coefficient as a function of the shorter load (wind) duration and the service class
- $\gamma_M = 1.25$: Security coefficient
- k_{sys} : Equal to 1 because $L \geq 1.2$ m
- k_h : Equal to 1 because $h \geq 600$ mm

$k_{m,\alpha}$: coefficient of decrease for the height

$$k_{m,\alpha} = \frac{1}{\sqrt{1 + \left(\frac{f_{m,d}}{1.5 f_{v,d}} \tan(\alpha) \right)^2 + \left(\frac{f_{m,d}}{f_{c,90,d}} \tan^2(\alpha) \right)^2}} = 0.98 \quad (2.49)$$

- $f_{v,d}$: Shear strength = $f_{v,k} \cdot \frac{k_{mod}}{\gamma_M} = 3.08$
- $f_{c,90,d}$: Compression strength = $f_{c,90,k} \cdot \frac{k_{mod}}{\gamma_M} = 2.2$
- α : Slope

Verification of the first criterion

$$\frac{\sigma_{m,\alpha,d}}{k_{m,\alpha} \cdot f_{m,d}} = \frac{9.79}{0.85 \cdot 20.16} = 0.55 \leq 1 \quad (2.50)$$

Second criterion: verification of stresses (ULS), due to bending stress in ridge area

In this area, the resistance condition is:

$$\frac{\sigma_{m,d}}{k_r \cdot f_{m,d}} \leq 1 \quad (2.51)$$

The coefficient k_r decreases the bending strength of the strips when they are bent. Its value is a function of the bending radius and the thickness of the strips. In this case k_r is equal to 1 because the strips are straight.

 $\sigma_{m,d}$: bending stress in the ridge area induced by the ULS combination of actions

The induced stress is increased by the coefficient k_L obtained with the equation (2.53). It depends on the height at the ridge, the slope and the inner radius of curvature of the girder.

$$\sigma_{m,d} = k_L \frac{6 \cdot M_{ap,d}}{b \cdot h_{ap}^2} = 13.41 \text{ [Mpa]} \quad (2.52)$$

- $M_{ap,d}$: Bending moment at the ridge.
- b and h_{ap} : Thickness of the girder and height at the ridge.

$$k_L = k_1 + k_2 \cdot \left(\frac{h_{ap}}{r}\right) + k_3 \cdot \left(\frac{h_{ap}}{r}\right)^2 + k_4 \cdot \left(\frac{h_{ap}}{r}\right)^3 = 1.03 \quad (2.53)$$

$$k_1 = 1 + 1.4 \cdot \tan(\alpha) + 5.4 \cdot \tan^2(\alpha) = 1.03$$

It is not required to calculate $k_2; k_3; k_4$ because $r = \infty$ and so, $\frac{h_{ap}}{r} = 0$

Verification of the second criteria

$$\frac{\sigma_{m,d}}{k_r \cdot f_{m,d}} = \frac{13.41}{1 \cdot 24.64} = 0.54 \leq 1 \quad (2.54)$$

Third criterion: verification of stresses (ULS), traction stress perpendicular to the grain orientation of the wood in ridge area

The tensile stress is calculated from the bending stress and a shape coefficient k_p . Moreover the resistance condition is modified by the coefficient k_{dis} which reflects the dispersion of the stresses and by the coefficient k_{vol} which reflects the influence of the volume of the ridge area. So the resistance condition is:

$$\frac{\sigma_{t,90,d}}{k_{dis} \cdot k_{vol} \cdot f_{t,90,d}} \leq 1 \quad (2.55)$$

$k_{dis} = 1.4$ for girder with variable inertia.

$$k_{vol} = \left(\frac{V_0}{V} \right)^{0.2} = 0.56 \quad (2.56)$$

- V_0 : Reference volume = 0.01 m^3
- V : Volume of the ridge area calculated with equation (2.57) = 0.179 m^3

$$V = h_{ap}^2 \cdot b \left(1 - \frac{h_{ap}}{2L} \right) \quad (2.57)$$

$\sigma_{t,90,d}$: traction stress perpendicular to the grain orientation of the wood in ridge area induced by the combination of action of ultimate limit states

$$\sigma_{t,90,d} = k_p \frac{6 \cdot M_{ap,d}}{b \cdot h_{ap}^2} = 0.05 \text{ [MPa]} \quad (2.58)$$

- $M_{ap,d}$: Bending moment at the ridge
- b and h_{ap} : Thickness of the girder and height at the ridge
- k_p : Shape coefficient

$$k_p = k_5 + k_6 \cdot \left(\frac{h_{ap}}{r} \right) + k_7 \cdot \left(\frac{h_{ap}}{r} \right)^2 = 0.004 \quad (2.59)$$

$$k_5 = 0.2 \cdot \tan(\alpha) = 0.004$$

$$k_6 = 0$$

$$k_7 = 0$$

$f_{m,d}$: tensile resistance in MPa

$$f_{t,90,d} = f_{t,90,k} \cdot \frac{k_{mod}}{\gamma_M} = 0.44 \text{ [MPa]} \quad (2.60)$$

- $f_{t,90,k}$: Characteristic tensile resistance
- k_{mod} : Reduction coefficient as a function of the shorter duration load and the class of service
- γ_M : Security coefficient

Verification of the third criterion for the ULS

$$\frac{\sigma_{t,90,d}}{k_{dis} \cdot k_{vol} \cdot f_{t,90,d}} = \frac{0.05}{1.4 \cdot 0.56 \cdot 0.44} = 0.15 \leq 1 \quad (2.61)$$

Verification of the compression perpendicular to the fiber on girder supports

To avoid compression failure at the supports, a sufficiently large area is required. Since the width of the girder is known, only the laying length must be determined.

The ULS 3 loads combination is used since it gives the worst load on the beam: $Q = 1.95 \text{ kN/m}^2 \cdot 6 \text{ m} = 11.7 \text{ kN/m}$. As the beam is 16 m long, a force of 93.6 kN applies on each support.

Knowing that the resistance for compression applied perpendicular to the fiber for the glue laminated wood GL28H is 2.5 MPa and the width of the beam is 20 cm the required length can be calculated:

$$\begin{aligned} f_{c,90,d} &= k_{mod} \cdot \frac{f_{c,90,k}}{1.25} = 2.2 \left[\frac{N}{mm^2} \right] \\ 2.2 \left[\frac{N}{mm^2} \right] &= \frac{93600}{200 \cdot x} \left[\frac{N}{mm^2} \right] \\ x &= 187.2 [mm] \end{aligned} \quad (2.62)$$

As the chosen concrete column (CR-RK 290-290) has 290 mm sides, there is no problem for placing the girder on it.

Deflection verification (SLS)

For an uniform loading, it is possible to calculate the maximum instant deflection with the equation (2.63) [24]. Since the slenderness of this type of beam is important, the deflection caused by the shearing force could have been neglected. However, for sake of precision, it is calculated by the second part of the formula.

$$W_{inst} = k_m \frac{5 \cdot 12 \cdot q_{inst} \cdot L^4}{384 \cdot E_{mean} \cdot b \cdot h_s^3} + k_t \frac{6 \cdot q_{inst} \cdot L^2}{5 \cdot 8 \cdot G_{mean} \cdot b \cdot h_s} = 44.2 [mm] \quad (2.63)$$

With:

- W_{inst} : Deflection [mm]
- q_{inst} : Uniform loading [kN/m]
- L : Span [mm]
- E_{mean} : Elasticity modulus [Mpa]
- G_{mean} : Shear modulus [MPa]
- b : Thickness [mm]
- h_s : Height of the girder at the support [mm]
- k_m : Coefficient for the deflection caused by the bending moment defined by the figure 2.49
- k_t : Coefficient for the deflection caused by the shear force defined by the figure 2.50

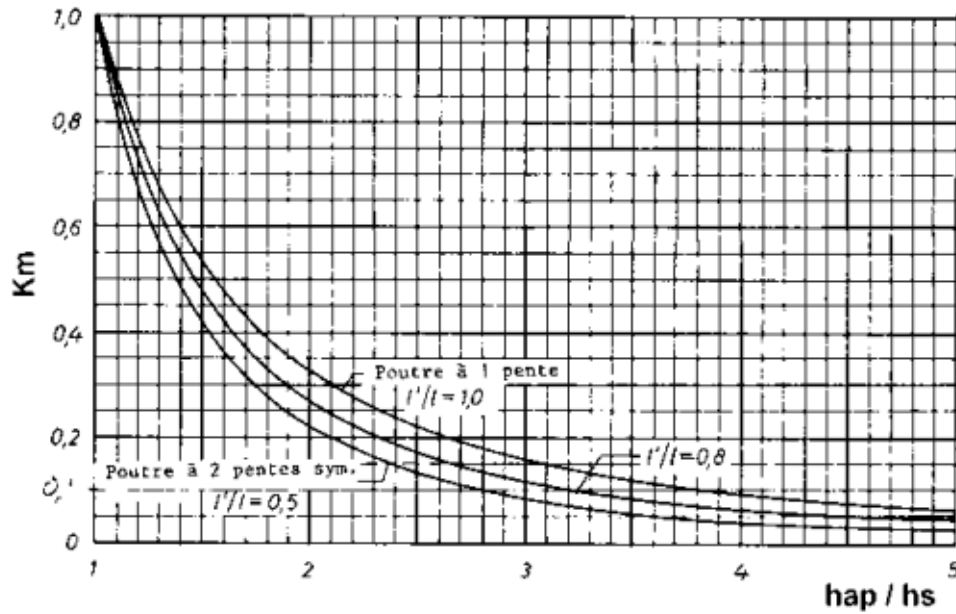


Figure 2.49: Coefficient related to the deflection caused by the bending moment

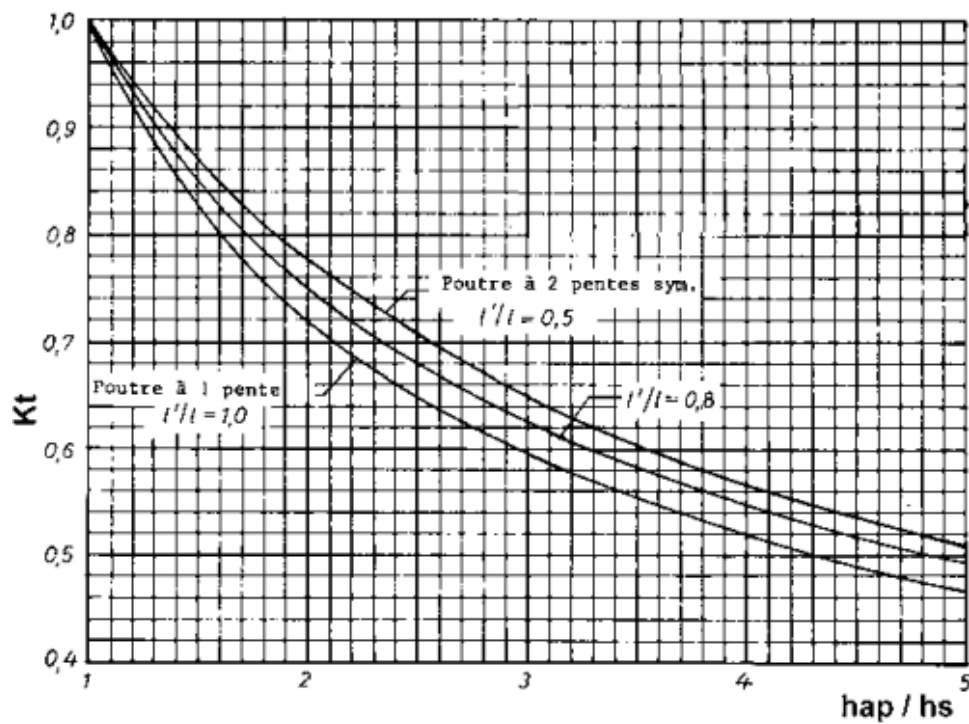


Figure 2.50: Coefficient related to the deflection caused by the shear force

The creep deflection, which is induced by the time of the loading and by the humidity of the wood, must be determined in order to calculate the final deflection which is the sum of the instant and the creep deflection. As the deflection is proportional to the load, it is simpler to calculate the creep deflection from the instantaneous one.

$$W_{net,fin} = W_{inst} + W_{creep} = 78.9 \text{ [mm]} \quad (2.64)$$

With:

- $W_{creep} = W_{inst} \cdot k_{def} = 35.1 [mm]$
- k_{def} : creep coefficient of 0.8 (glue laminated timber)

It needs to be checked if the deflection due to the actions on the structure stay under the limits ($L/300$ for W_{inst} and $L/200$ for $W_{net,fin}$):

$$maximum \begin{cases} \frac{W_{inst}}{W_{limit}} = \frac{43.8}{53.3} = 0.82 \\ \frac{W_{net,fin}}{W_{limit,net,fin}} = \frac{78.9}{80} = 0.98 \end{cases} \leq 1 \quad (2.65)$$

Column design

The same method than in section 2.7.1 is used.

$$\begin{aligned} N &= 11.7 \cdot \frac{16}{2} + 1.35 \cdot \frac{1.30 * 9.81}{2} + 1.35 \cdot 0.15 \cdot 6 \cdot 4 \\ &= 107kN \\ M &= 34.78kNm \end{aligned} \quad (2.66)$$

The smallest profile available on catalogue, CR-RK 290-290, is sufficient to withstand the solicitations. It has a weight of 2.06 kN/m.

2.8 Foundations pre-sizing

Like primary structure, foundations are divided into two categories: pinned columns and en-cased columns. These cases are analyzed separately. The aim of this section is to pre-size the foundations in order to take it into account in the following analysis (environmental impact, assembly time). Thus, a simplified method is used and only bearing capacity is considered.

2.8.1 Pinned columns

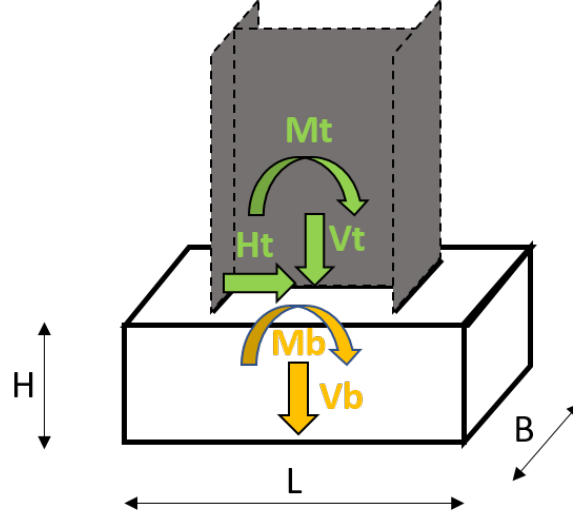


Figure 2.51: Foundation of the pinned column

This system is associated with the hot-rolled and the welded built-up technologies. As the hot-rolled frame is heavier, and to consider the worst case scenario, we use it to evaluate loads on the foundation.

Solicitations

$$\begin{aligned} V_t &= 100 \text{ kN} & (\text{see section 2.6}) \\ H_t &= 58 \text{ kN} \\ M_t &= 0 \text{ kNm} \end{aligned} \tag{2.67}$$

Now that loads are known on the top of the foundation, stress on the ground can be computed below the concrete block.

$$\begin{aligned} V_b &= V_t + Volume \cdot \rho_{concrete} = 100 + (H \cdot L \cdot B) \cdot 25 \propto Geometry \\ M_b &= M_t + H \cdot H_t \propto H \end{aligned} \tag{2.68}$$

Knowing the internal forces, we can find the maximal stress on the soil below the block [21]:

$$\sigma = \frac{N}{A} + \frac{My}{I} \tag{2.69}$$

This stress must be smaller than the soil bearing capacity.

Soil bearing capacity

We consider a firm clay with a unit weight γ_s of 18 kN/m³. The bearing capacity is estimated like so [16]:

$$q = q_0 + \frac{1}{3} \cdot (k_p \cdot p_l) \quad (2.70)$$

With:

- $q_0 = \text{depth} \cdot \gamma_s$
- $k_p = 0.8(1 + 0.25 \cdot (0.6 + 0.4 \cdot \frac{B}{L}) \frac{D_e}{B})$
- $p_l \in [1.2; 2] = 1.5 \text{ MPa}$ [16]

Solution

Both design stress and bearing capacity depend on our block geometry. First hypothesis is based on a life cycle assessment study conducted in collaboration with ArcelorMittal [25]. This ensures a prosaic design. Then, as the outcome of an iterative process, the final solution is found:

$$\begin{aligned} H &= 300 \text{ mm} \\ B &= 720 \text{ mm} \\ L &= 720 \text{ mm} \end{aligned} \quad (2.71)$$

We consider a steel reinforcement ratio of ten percent of the mass. It gives 77.76 kg of BSt 500 steel.

2.8.2 Encased columns

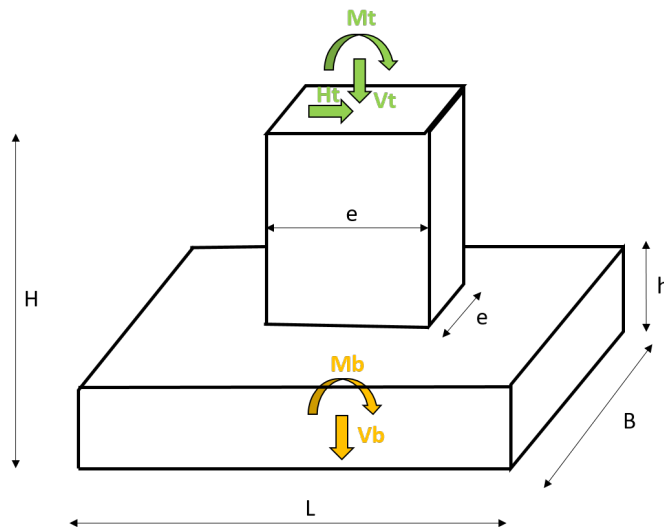


Figure 2.52: Foundation of the encased column

This system is associated with the precast concrete and the glue laminated timber/concrete technologies. Again, the heaviest solution, the all-concrete frame, is used to evaluate loads on

the foundation. The process is similar than in section 2.8.1.

$$\begin{aligned}
 V_t &= 125 \text{ kN} \quad (\text{see section 2.7.1}) \\
 H_t &= 58 \text{ kN} \\
 M_t &= 34.78 \text{ kNm} \\
 V_b &= V_t + Volume_{concrete} \cdot \rho_{concrete} + Volume_{soil} \cdot \rho_{soil} \propto Geometry \\
 M_b &= M_t + H \cdot H_t \propto H
 \end{aligned} \tag{2.72}$$

Solution

$$\begin{aligned}
 H &= 600 \text{ mm} \\
 B &= 1030 \text{ mm} \\
 L &= 1030 \text{ mm} \\
 e &= 340 \text{ mm} \\
 h &= 240 \text{ mm}
 \end{aligned} \tag{2.73}$$

We consider a steel reinforcement ratio of ten percent of the mass. It gives 148.12 kg of BSt 500 steel.

2.9 Partial conclusion: Designed solutions summary

On the table 2.14 reader will find a summary of the designed solutions for the first case with various materials. It contains the profile types and their mass as well as the number of elements required for each members of the primary structure. The graph 2.53 allows the comparison between the different materials regarding their mass. The written values comes from the sum of both columns and beam masses.

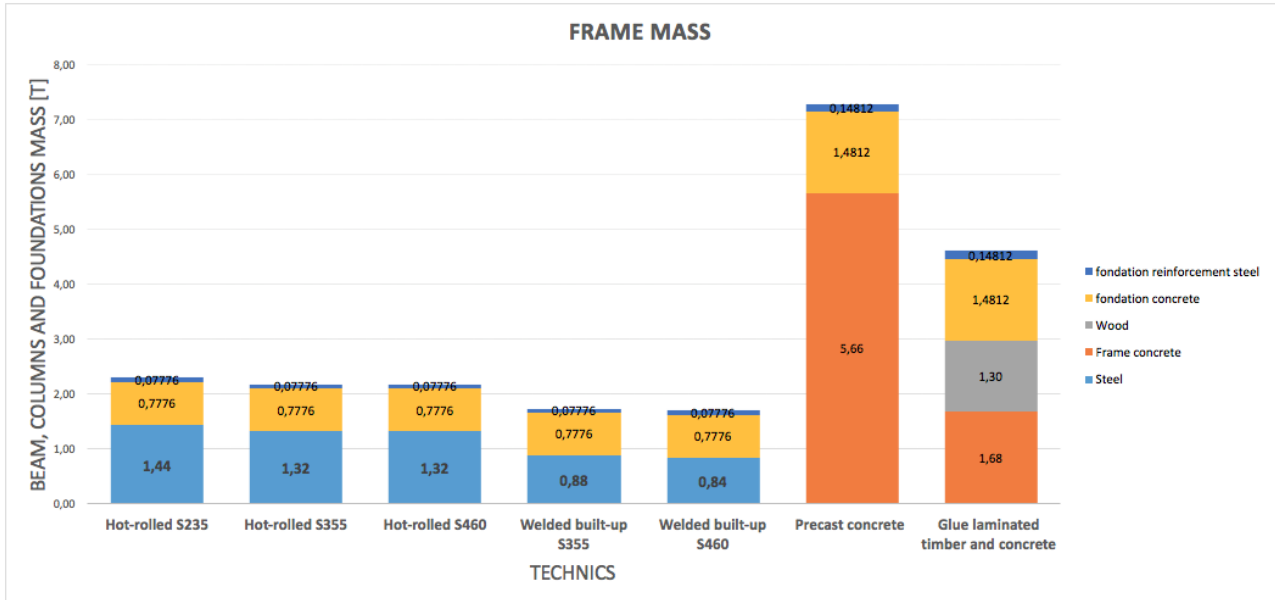


Figure 2.53: Comparison between the different materials regarding the mass

Several comments can be made based on the figure:

- The mass of a concrete frame is up to three times the mass of steel solution.
- The reinforced concrete column with a glue laminated timber is a medium weight design. The mass of the wood is almost identical to the mass of the steel, but the concrete columns remain much heavier than the steel.
- Foundations for the encased columns are twice as heavy than the solutions with pinned columns.
- The welded built-up solution is the lightest. It is due to the design process which leads to optimize the structure.

The table 2.15 is equivalent to the table 2.14 but concerns the secondary structure.

Mass [T]		Column					Beam				
		Profile	Number of elements	Unit length [m]	single	By frame	Profile	Number of elements	Unit length [m]	single	By frame
Hot-rolled profile	S235	IPE400	2	4	0,27	0,53	IPE360	2	8	0,46	0,91
	S355	IPE400	2	4	0,27	0,53	IPE330	2	8	0,39	0,79
	S460	IPE400	2	4	0,27	0,53	IPE330	2	8	0,39	0,79
Welded built-up	S355	I(372;120;9,8;5,2)- I(500;120;9,8;5,2)	2	4	0,15	0,30	I(372;120;9,8;5,2)- I(510;120;9,8;5,2)	2	8	0,29	0,58
	S460	I(345;120;9,8;5,2)- I(460;120;9,8;5,2)	2	4	0,14	0,28	I(350;120;9,8;5,2)- I(440;120;9,8;5,2)	2	8	0,28	0,56
Precast concrete	Ergon Catalog	CR-RK 290-290	2	4	0,84	1,68	IVH 875/240	1	16	3,98	
Glue laminated timber	GL28h	CR-RK 290/290	2	4	0,84	1,68	PIV 960-800-200	1	16	1,30	

Mass		Foundation				
		Dimensions [mm]	Materials	Number of elements	single [kg]	By frame [kg]
Hot-rolled profile	S235	720x720x300	C30-37 and BSt 500	2	concrete : 388,8 steel : 38,88	concrete : 777,6 steel : 77,76
	S355					
	S460					
Welded built-up	S355					
	S460					
Precast concrete	Ergon Catalog	1100x1100x600 Footing width 240 Column support width 340	C30-37 and BSt 500	2	concrete : 740,58 steel : 74,06	concrete : 1481,2 steel : 148,12
Glue laminated timber	GL28h					

Table 2.14: Summary of the different frame solutions

Element		Profile	# elements	Unit length [m]	Unit mass [kg]	Material	Mass [T]
Girts	By frame	C260250	8	6	59,58	S350	0,48
Bracings	Total	L20x20x3	16	6,33-7,21	5,56-6,34	S235	0,0952
Purlins	By frame	A170200	11	6	32,34	S350	0,36
		100x300	11	6	75,6	C24	0,83
End-wall columns	Total	IPE120AA	14	4,16-4,12-4,08-4,04	34,94-34,61-34,27-33,94	S235	0,48

Table 2.15: Summary of the different choices for secondary structure

Chapter 3

Hall size variation

Once the design method is described and validated, sizing of other hall size situations can be performed using exactly the same process. Besides the 16 m span basic case, two additional halls are considered: the 24 m span and the 36 m span.

3.1 Size 1: Height=4 [m];Span=16 [m]

Mass [T]		Column					Beam				
		Profile	Number of elements	Unit length [m]	single	By frame	Profile	Number of elements	Unit length [m]	single	By frame
Hot-rolled profile	S235	IPE400	2	4	0,27	0,53	IPE360	2	8	0,46	0,91
	S355	IPE400	2	4	0,27	0,53	IPE330	2	8	0,39	0,79
	S460	IPE400	2	4	0,27	0,53	IPE330	2	8	0,39	0,79
Welded built-up	S355	I(372;120;9,8;5,2)- I(500;120;9,8;5,2)	2	4	0,15	0,30	I(372;120;9,8;5,2)- I(510;120;9,8;5,2)	2	8	0,29	0,58
	S460	I(345;120;9,8;5,2)- I(460;120;9,8;5,2)	2	4	0,14	0,28	I(350;120;9,8;5,2)- I(440;120;9,8;5,2)	2	8	0,28	0,56
Precast concrete	Ergon Catalog	CR-RK 290-290	2	4	0,84	1,68	IVH 875/240	1	16	3,98	
Glue laminated timber	GL28h	CR-RK 290/290	2	4	0,84	1,68	PIV 960-800-200	1	16	1,30	

Table 3.1: Summary of the different choices for the superstructure of the 16m span frame

Mass		Foundation				
		Dimensions [mm]	Materials	Number of elements	single [kg]	By frame [kg]
Hot-rolled profile	S235	720x720x300	C30-37 and BSt 500	2	concrete : 388,8 steel : 38,88	concrete : 777,6 steel : 77,76
	S355					
	S460					
Welded built-up	S355					
	S460					
Precast concrete	Ergon Catalog	1100x1100x600 Footing width 240 Column support width 340	C30-37 and BSt 500	2	concrete : 740,58 steel : 74,06	concrete : 1481,2 steel : 148,12
Glue laminated timber	GL28h					

Table 3.2: Summary of the different choices for the foundations of the 18m span frame

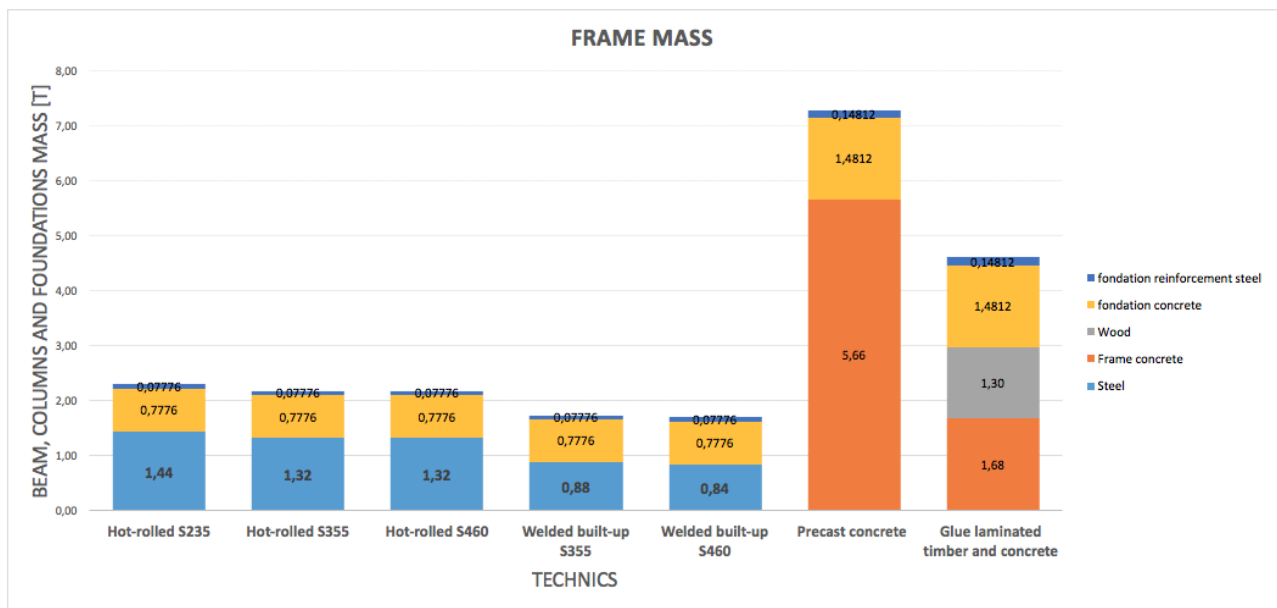


Figure 3.1: Mass comparison for the 16m span frame

3.2 Size 2: Height=12 [m];Span=24 [m]

These dimensions are meant to represent a typical industrial building. To suit its role for the industrialist, columns has been lengthened to 12 m to increase the usable volume. The tables below present the results of the design.

Mass [T]		Column					Beam				
		Profile	Number of elements	Unit length [m]	single	By frame	Profile	Number of elements	Unit length [m]	single	By frame
Hot-rolled profile	S235	HE600 A	2	12	2,14	4,27	IPE 600	2	12	1,46	2,93
	S355	HE 650 A	2	12	2,28	4,56	IPE 550	2	12	1,27	2,54
	S460	HE 650A	2	12	2,28	4,56	IPE 550	2	12	1,27	2,54
Welded built-up	S235	I(720;150;15;8)- I(820;150;15;8)	2	12	1,00	2,01	I(670;140;15;7)- I(770;140;15;7)	2	12	0,87	1,74
	S355										
	S460										
Precast concrete	Ergon Catalog	CR-RK 590-390	2	12	6,90	13,80	I 1100/390	1	24	14,95	
							IE 1050/440	1	24	11,11	
							IV 1250/390	1	24	13,05	
							IVH 1200/490	1	24	12,74	
Glue laminated timber	GL28h	CR-RK 590-390	2	12	6,90	13,80	PIV 1340-1100-200	1	24	2,69	

Table 3.3: Summary of the different choices for the superstructure of the 24m span frame

Mass		Foundation				
		Dimensions [mm]	Materials	Number of elements	Single [kg]	By frame [kg]
Hot-rolled profile	S235	720x720x300	C30-37 and BSt 500	2	concrete : 774,4 steel : 77,44	concrete : 1548,8 steel : 154,88
	S355					
	S460					
Welded built-up	S235					
	S355					
	S460					
Precast concrete	Ergon Catalog	1100x1100x600 Footing width 240 Column support width 340	C30-37 and BSt 500	2	concrete : 3540 steel : 354	concrete : 7080 steel : 708
Glue laminated timber	GL28h					

Table 3.4: Summary of the different choices for the foundations of the 24m span frame

After summarizing all the elements required to build the structure, the mass is compared between the six considered solutions. The solution for welded built-up S355 and S460 is the same that the welded built-up S235 solution.

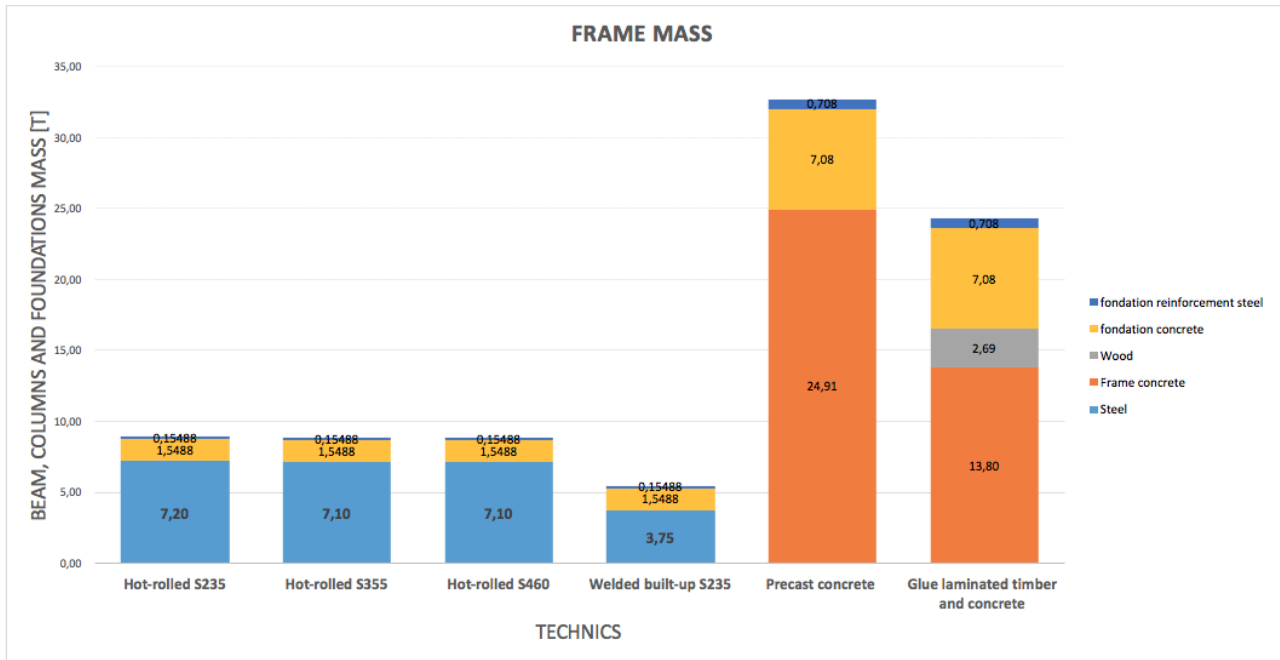


Figure 3.2: Mass comparison for the 24m span frame

3.3 Size 3: Height=12 [m];Span=36 [m]

This hall size allows us to consider a larger-scale building as well as pushing the materials and the structural mechanism to their limit. Once again, results are presented in both following tables.

Mass [T]		Column					Beam				
		Profile	Number of elements	Unit length [m]	single	By frame	Profile	Number of elements	Unit length [m]	single	By frame
Hot-rolled profile	S235	HE 800 M	2	12	3,80	7,61	IPE 750x196	2	18	2,35	4,70
	S355	HE 800 B	2	12	3,14	6,29	IPE 750x147	2	18	1,76	3,53
	S460	HE 800 A	2	12	2,69	5,38	IPE 750x147	2	18	1,76	3,53
Welded built-up	S235	I(699;160;17;8)- I(950;160;17;8)	2	12	1,13	2,27	I(850;170;20;8)- I(1000;170;20;8)	2	18	1,81	3,63
	S355										
	S460										
Precast concrete	Ergon Catalog	CR-RK 590-390	2	12	6,90	13,80	I 1600/590	1	36	31,78	
							IV 1875/490	1	36	29,90	
							IVH 1600/590	1	36	26,51	
Glue laminated timber	GL28h	CR-RK 590-390	2	12	6,90	13,80	PIV 2060-1700-200	1	36	6,23	

Table 3.5: Summary of the different choices for the superstructure of the 36m span frame

Mass		Foundation				
		Dimensions [mm]	Materials	Number of elements	Single [kg]	By frame [kg]
Hot-rolled profile	S235	720x720x300	C30-37 and BSt 500	2	concrete : 1232,1 steel : 123,21	concrete : 2464,2 steel : 246,42
	S355					
	S460					
Welded built-up	S235					
	S355					
	S460					
Precast concrete	Ergon Catalog	1100x1100x600 Footing width 240 Column support width 340	C30-37 and BSt 500	2	concrete : 4922,5 steel : 492,25	concrete : 9845 steel : 984,5
Glue laminated timber	GL28h					

Table 3.6: Summary of the different choices for the foundations of the 36m span frame

Then, masses of all possibilities are compared.

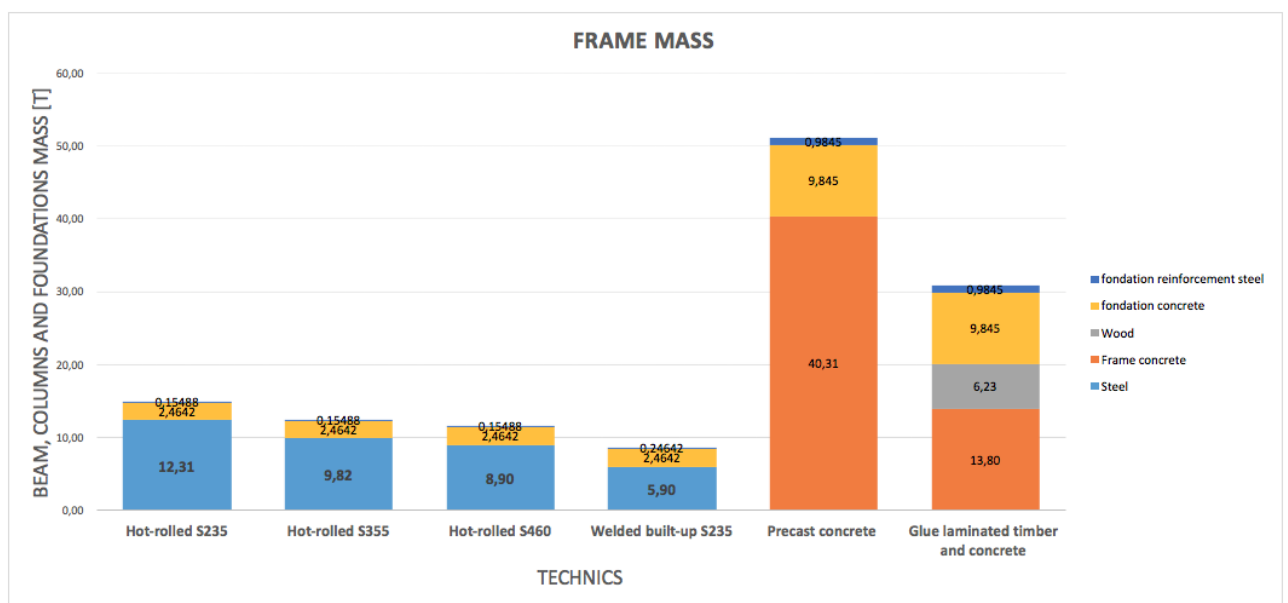


Figure 3.3: Mass comparison for the 36m span frame

3.4 Partial conclusion: Size variation

The histograms 3.4 and 3.5 allow the comparison between the mass of the different structural solutions regarding the hall size variation. The following comments can be made:

- The first histogram enables to analyze the mass reduction from one size to another for every constructive solution.

The decrease of the mass with the size is related to the chosen constructive solution. The heavier the used material, the more significant is the mass increase between the sizes: for solutions containing concrete (complete precast concrete frame and glue laminated timber girder with concrete columns), the mass decrease between the medium and the small size is around 80 %. The steel solutions have smaller masses and the mass variations between medium and small hall are between 68% and 75% depending on the constructive method and the steel grade.

From large to medium size the smallest decrease is for the solution involving wood with a 21 % drop. This size change leads to a smaller mass variation for two main reasons: On the one hand, the height of the column is the same for both medium and large halls. On the other hand, the columns supporting the 24 m girder was oversized because no smaller precast elements on the catalogue could sustain the required loads. Hence, in either case 590-390 elements were chosen. The concrete representing a large part of the total mass of the frame (see figure 3.3), concrete/wood solution are very similar in the 24 m and in the 36 m span cases regarding their mass. The mass of the full concrete solution decreases by 56 % mainly because of the bigger dimensions required to sustain the loads on such a large span. Indeed, a 36 m girder gets close to the maximal available length of many of the models. Steel solutions have highly variable decreasing percentages between 23 and 40 % depending on the steel grade.

- The second one shows the difference of mass between the various material choices for each of the three frame spans.

The first thing to notice is that for all three sizes the order of the constructive solutions ranked according to their weight is the same.

After that, it can be observed that the mass reductions between the 4 main families of solutions, all concrete solution, solution with a wooden beam, hot-rolled solution and finally optimized welded built-up solution, remain more or less constant regardless of the size. The concrete structures are 36 to 40% heavier than the glulam ones. These solutions are 50 to 63 heavier than steel solutions. Finally, the choice of a welded built-up solution allows a 20 to 38 % mass reduction.

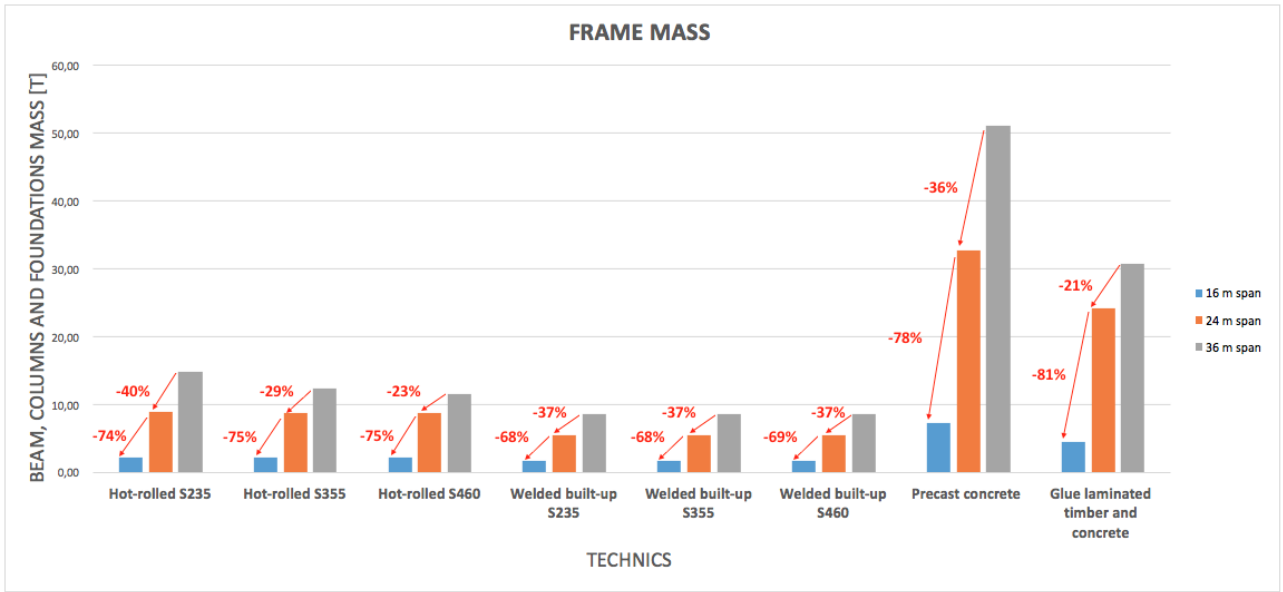


Figure 3.4: Mass comparison according to the size for every material

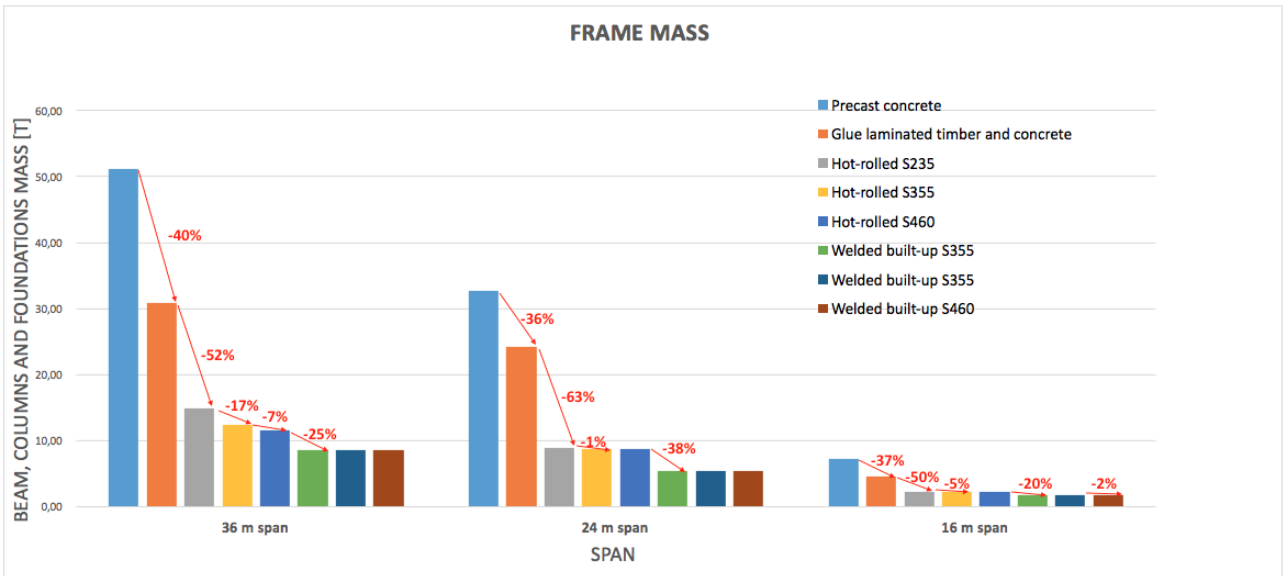


Figure 3.5: Mass comparison according to the material for every size

Chapter 4

Life cycle assessment

As previously said in the introduction, the construction sector is one of the most energy-consuming and greenhouse gas producer. As it seems essential to consider the environmental aspects in our comparison, this chapter will be devoted to them.

4.1 Life cycle assessment information

Developed during the 1990's, the LCA is a method that allows to quantify human actions on the environment and particularly regarding greenhouse gases. In a society where one feels more and more concerned with the environment, it was natural to carry out an LCA study for the industrial halls of this research.

The aim of this part of the dissertation is to compare the most common structural systems used in industrial halls (steel, concrete and wood) in order to select the best system for the environment. The LCA method contains a lot of indicators but only two impacts will be studied in the analysis: Global Warming potential and Embodied energy. They will be detailed below. We will also compare several envelopes to optimize the use phase in terms of energy.

For the comparison, the functional unit is a single frame module with its secondary structure and its foundation. Using it for the LCA allows us to easily compare different hall dimensions without taking account of the length of the building. Hence, as industrial halls are composed of a succession of modules regularly spaced, our results are easily usable no matter the number of modules required.

According to the European standard 15978, a LCA methodology includes several stages through which all construction materials must pass: going from the production of the material to the end of this life through transport and construction (see figure 4.1). Recycling of materials is not mandatory in the European standard EN 15978 but it has a high influence on the GWP and the EE. Let's notice that, in Belgium, taking the recycling into account is mandatory. This study will compare LCA with D phase considered (A to D) or ignored (A to C) to see the benefit given by the D stage. At first, the stage B will be assumed to be constant from one structural system to another. Secondly, it will be considered by comparing the impact of the choice of the covering on the heating consumption.

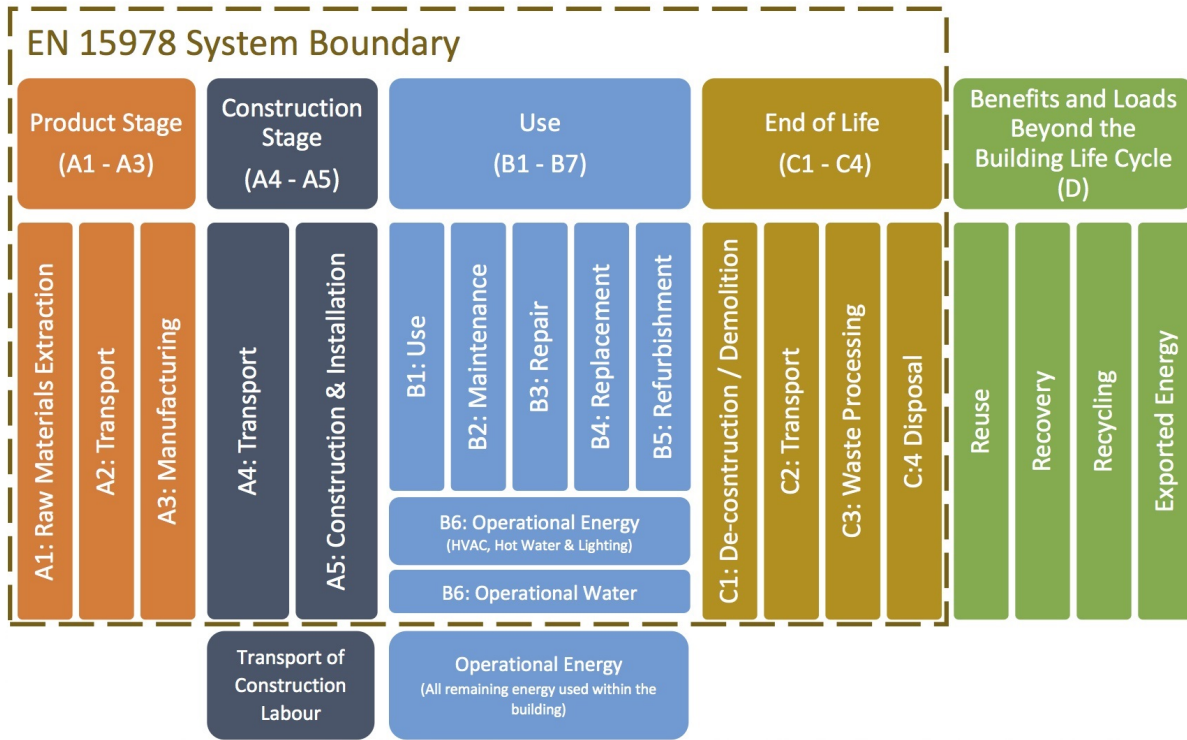


Figure 4.1: Life cycle stages for building products and buildings according to EN 15978. The stage *benefits and loads* is not indicated in the norm.[18]

4.2 Data for ecological information

In this part of the study, only two impact categories are considered: the Global Warming Potential (GWP) and the Embodied Energy (EE), which is all the energy required to produce each element (measured in *MJ*).

The Global Warming Potential describes the contribution of emissions to the greenhouse effect. It is measured in *kg CO₂-Equivalent*, which means that all the gases released during the life cycle of the materials are converted in terms of equivalent mass of CO₂, taking account of the strength of their global warming effect. These environmental data are issued from previous studies. The values for the structural steel, rebars and the concrete was calculated by the Council on Tall Buildings and Urban Habitat (CTBUH)[28] while the values for glue-laminated timber are found in a study of Bauforumstahl on the life cycle assessment comparison of a typical single storey building [25]. All these data are available in the table 4.1.

According to the same research from CTBUH [28], the various structural elements are considered with different levels of complexity: beams are fully fabricated products (through the addition of steel plates as described below); and columns are steel profiles fabricated without the addition of any plates.

Consequently, the total GWP and EE of columns and beams are calculated as follows:

Total GWP or EE of Steel Elements = GWP or EE of Steel Production + GWP or EE of Steel Elements Manufacture

ArcelorMittal indicated an electricity input of 0.15 MJ per kg for steel production [28]. This leads to an EE of 0.44 MJ and a GWP of 0.03 kg of CO₂ which need to be added to the accounting of the impact from every kilogram of beams and columns.

Hence, steel beams are considered as per the following expression:

$$1 \text{ kg steel beams} = 0.95 \text{ kg of steel profiles} + 0.05 \text{ kg of steel plates} + 0.15 \text{ [MJ] of electricity}$$

And columns are considered as:

$$1 \text{ kg steel columns} = 1 \text{ [kg] of steel profiles} + 0.15 \text{ [MJ] of electricity}$$

The following table 4.1 summarizes all the data chosen regarding the production phase. It may be noted that the GWP of wooden elements is negative. This is due to the fact that wood is considered as a reserve of carbon. This consideration also influences the stage D. Indeed, a certain amount of the CO₂ embed in the material during the tree growth is emitted during the recycling which is, most of the time, an incineration of waste.

Material	Global Warming Potential [Kg CO ₂ /Kg]	Embodied Energy [MJ/Kg]
Concrete		
C30/37	0.17	1.25
Structural steel		
Profiles	1.14	14.8
Plates	2.46	26.07
Beams	1.23	15.81
Columns	1.17	15.25
Rebars	1.24	16.4
Wood		
Glue-laminated timber	-1.19	27.32

Table 4.1: Environmental data for the product stage

The environmental impact related to the transportation of the structural elements is estimated by the AMECO software with a hypothetical distance of 100 km between the supplier and the construction site. This distance has been chosen because of the central location of the project in Belgium. We consider that any material can be found in the offer of a retailer located less than 100 km away.

4.3 Results and comparison

4.3.1 Comparison between frame materials

Here under reader will find some graphs which compare the environmental impact of the different solutions considered in this study. The three different dimensions of building are compared. Every result is given for one module (two columns, one beam and the foundations). One may notice that whether the recycling phase (D stage) is taken into account or not, the result varies a lot.

Indeed, the steel can be recycled in a large proportion and melted again to produce new material. The recycling rate for steel elements (sections and plates) is 99 %. This means that 99% will be reused during phase D. In particular, welded steel have a much larger environmental impact when recycling isn't taken into account because of the manufacturing process. Nevertheless, once the recycling is considered, the weight reduction enabled by the variable inertia elements allows an environmental impact reduction.

The wood has a lower impact during the first three stages of its usage but must be burned after de-construction to avoid landfill. When the wooden components are incinerated, they produce energy and release CO₂, so impact is very different when the recycling is considered. Let's notice that the energy released during the incineration can be used. That's why the questions about taking it into account in the environmental impact calculation divide specialists.

The concrete, for its part, cannot be recycled. This explains why A to C and A to D analyses give similar results.

The case of reinforcing steel is particular. It is made from 99% recycled steel, but the recycling rate of the rebar steel in precast concrete is only 70%. This is notably because extracting the rebars from the concrete is a very difficult process. Therefore, for solutions involving reinforcement steel the accounting of the recycling phase tend to increase both global warming potential and embodied energy.

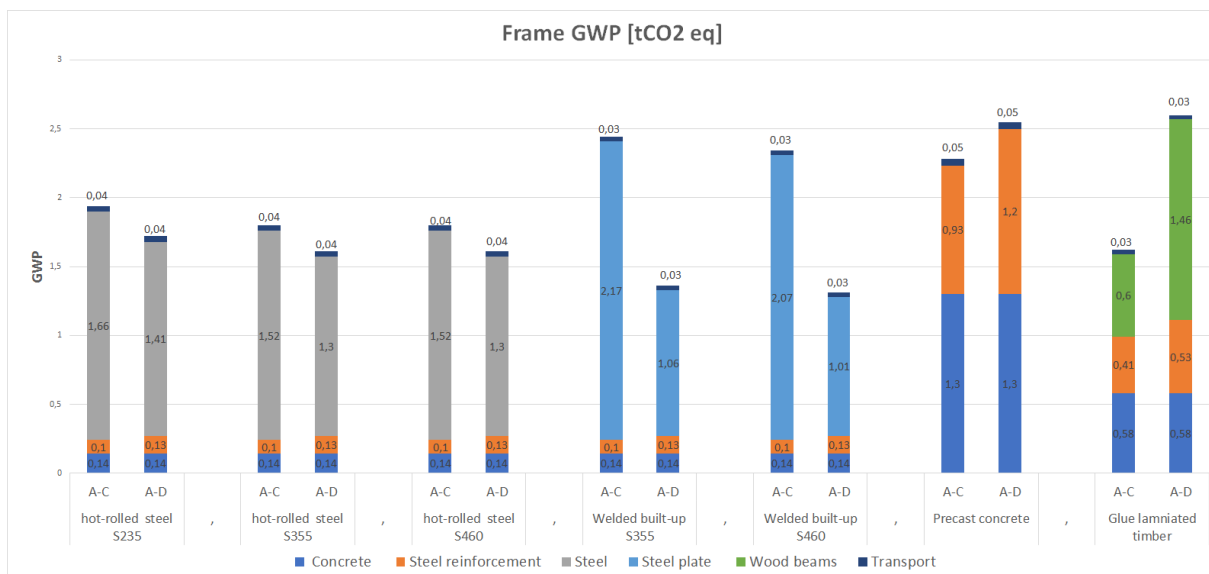


Figure 4.2: GWP for different frame materials from stage A to C and from A to D for 16 m span

The comparison of the Global Warming Potential for all the structural solutions shows that hot-rolled steel give close results from one grade to another. The quantity of GWP decreases with the mass of the hot-rolled frame so the lighter one is the most ecological.

The GWP of welded built-up solutions appears to be the most dependent on the consideration of the stage D. These are among the largest emitters of greenhouse gases if the analysis only takes into account stages A to C but they become the most efficient technology if the recycling is considered.

The conclusion for glue laminated girder associated with pre-stressed concrete columns is completely opposite. This difference is attributable to two already explained realities: the wood incineration after its use that releases CO₂ into the atmosphere and the high proportion of recycled steel into the steel reinforcement.

The prefabricated concrete solution comes out to be the most polluting whether the stage D is taken into account or not. The concrete cannot be recycled at all and again the reinforcement steel comes mostly from recycled steel but cannot be recycled at such a high proportion itself.

Transport represents only a very small part of the greenhouse gas emissions and is directly proportional to the mass of the transported elements. So, the heavier elements such as concrete parts have a slightly higher GWP.

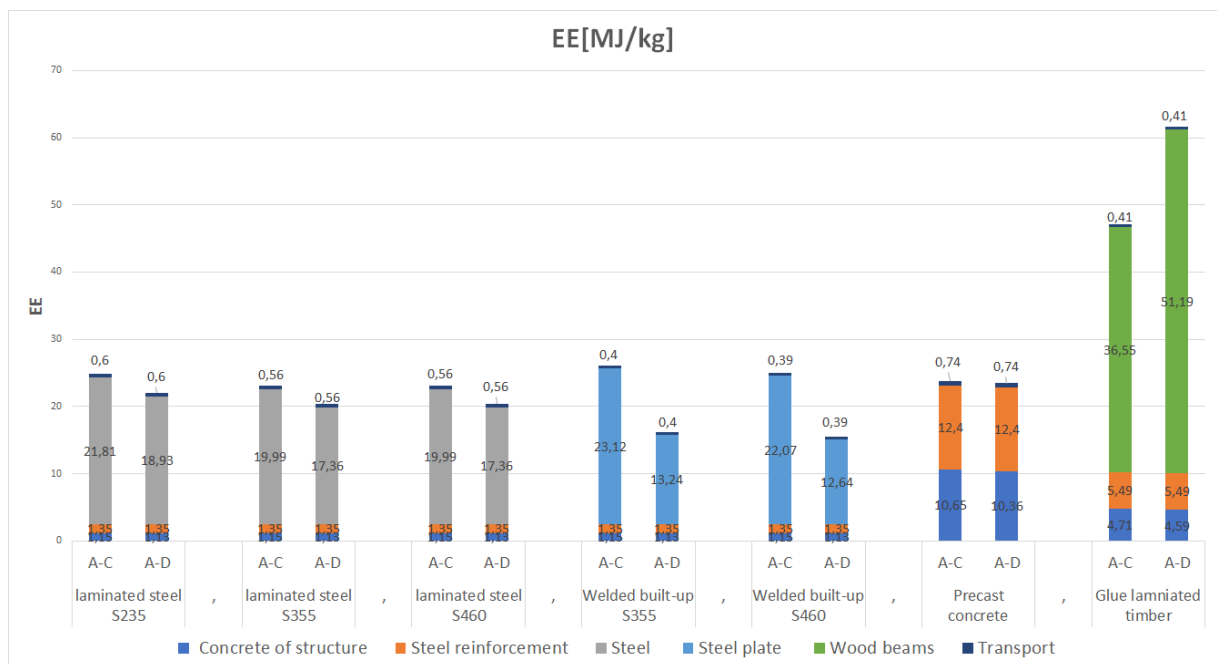


Figure 4.3: EE for different frame materials from stage A to C and from A to D for 16 m span

The comparison of Embodied Energy of the different options brings us to consider new reflections. Firstly, the precast concrete requires few energy to be produced. Most of its impact comes from the steel reinforcement. Secondly, wood requires a large amount of energy. However, this must be put into perspective as a large proportion of this value is taken by renewable energy (roughly half of it). This proportion is much larger for wood than for other material as reader may see in the graph 4.4.

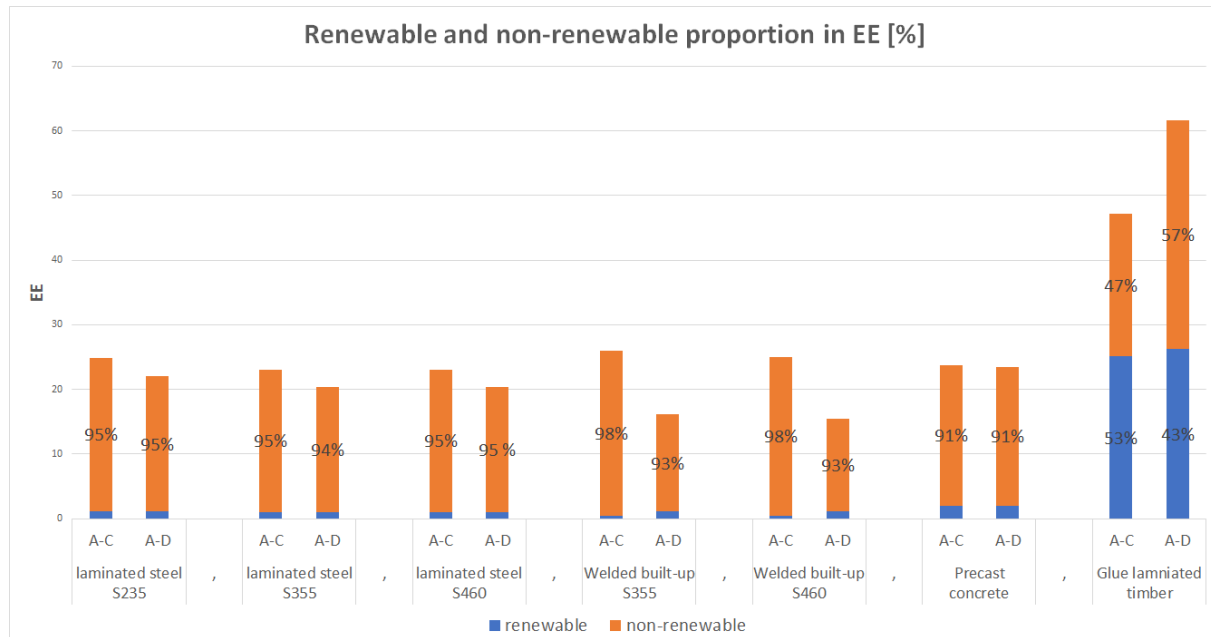


Figure 4.4: Proportion of renewable energy in the Embodied Energy

Then the GWP is observed for the two other dimensions; 24 m and 36 m span frames are analyzed. The aim is to study the influence of the size on the environmental impact. Will the tendencies, previously revealed, become clearer or not?

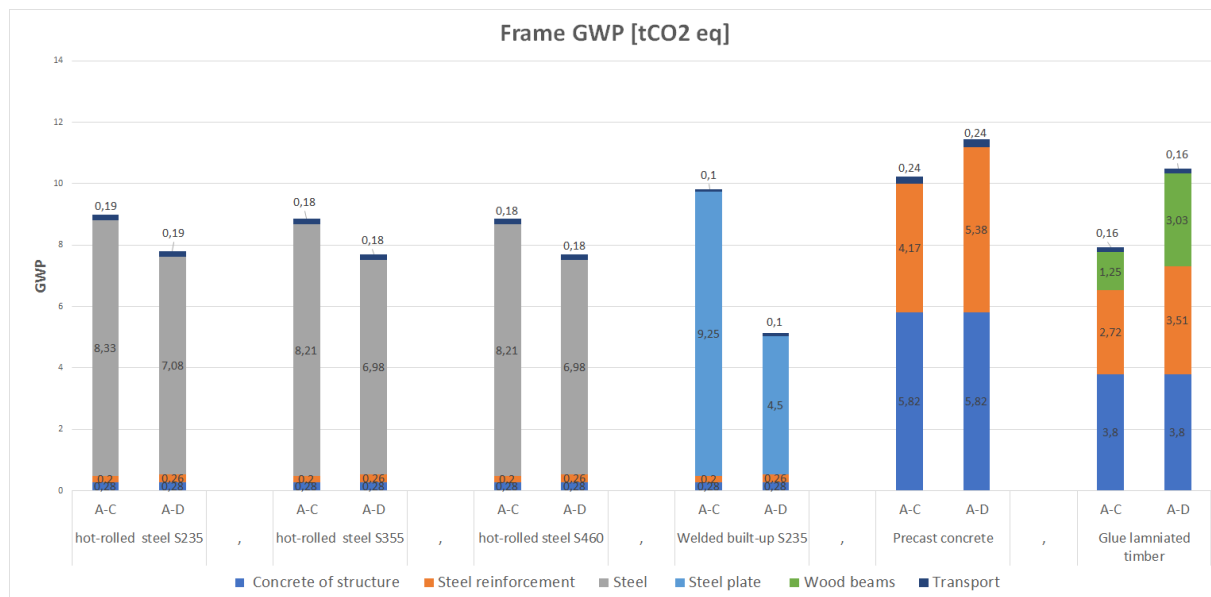


Figure 4.5: GWP for different frame materials from stage A to C and from A to D for 24 m span

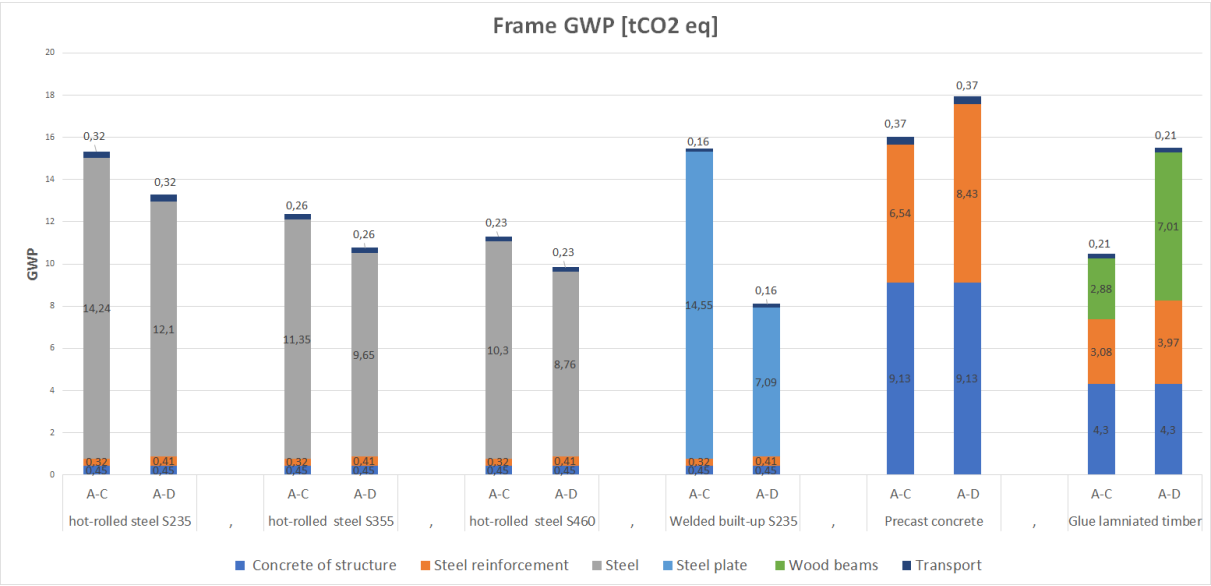


Figure 4.6: GWP for different frame materials from stage A to C and from A to D for 36 m span

In order to help the reader to easily apprehend the data, the three following graphs give the additional percentage of GWP for each span and in comparison with the most ecological option (if the recycling is taken into account): the welded built-up steel.

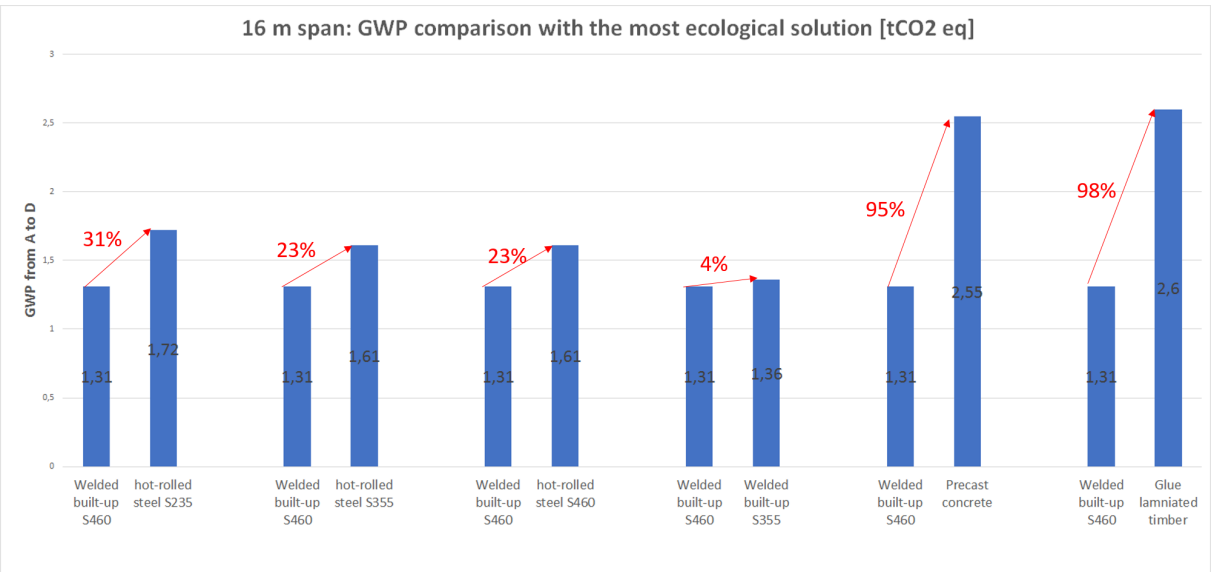


Figure 4.7: GWP from stage A to D comparison with the S460 welded built-up structure for 16 m span

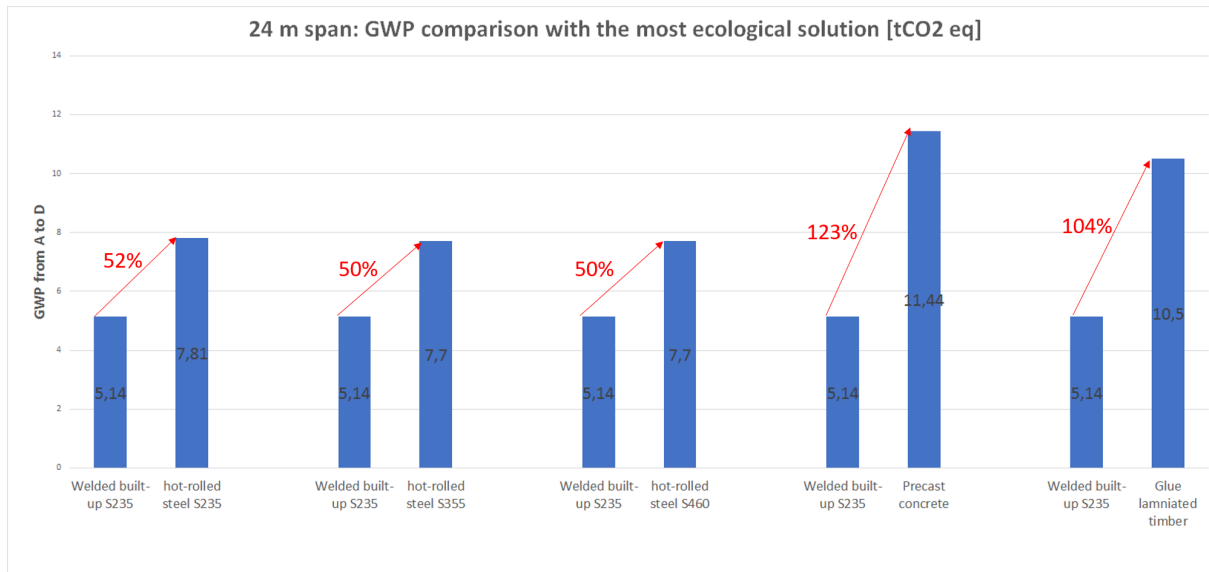


Figure 4.8: GWP from stage A to D comparison with the S235 welded built-up structure for 24 m span

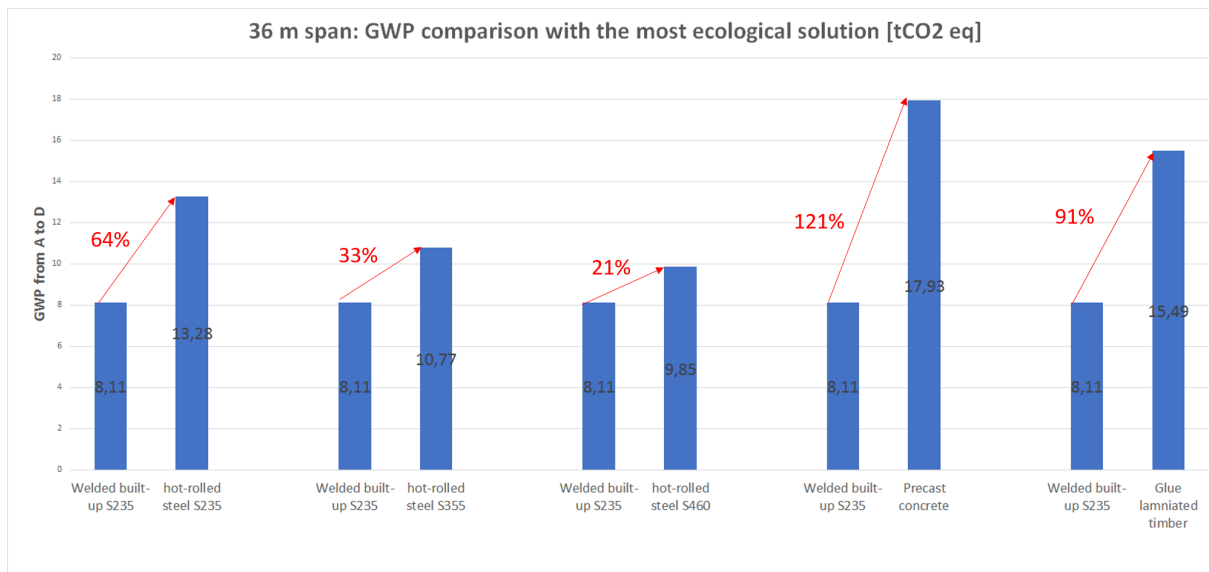


Figure 4.9: GWP from stage A to D comparison with the S235 welded built-up structure for 36 m span

The results show that the difference between materials doesn't systematically increase with the size:

- S235 hot-rolled steel and precast concrete: when the span rises, it is more and more attractive to build with welded built up instead of with these methods.
- S355 and S460 hot-rolled steel: for 16 and 24 m span structures the tendency is similar than with S235 steel, but as soon as the size increase up to 36 m, the propensity reverse and the difference with welded steel become smaller.

- Glue laminated timber and concrete: when the span is limited, the difference with welded steel decrease with the size. At higher span the effect invert and the difference becomes higher again.

4.3.2 Environmental impact of the secondary structure

Secondly, the secondary structure is observed. The graph below presents the environmental impact of the secondary structure in terms of greenhouse gas emission. Depending on whether the recycling phase is taken into account or not, the choice between steel and wood for purlins is different. If all stages are considered, steel comes out to be the most ecological choice as a material for purlins and wood is a better choice otherwise.

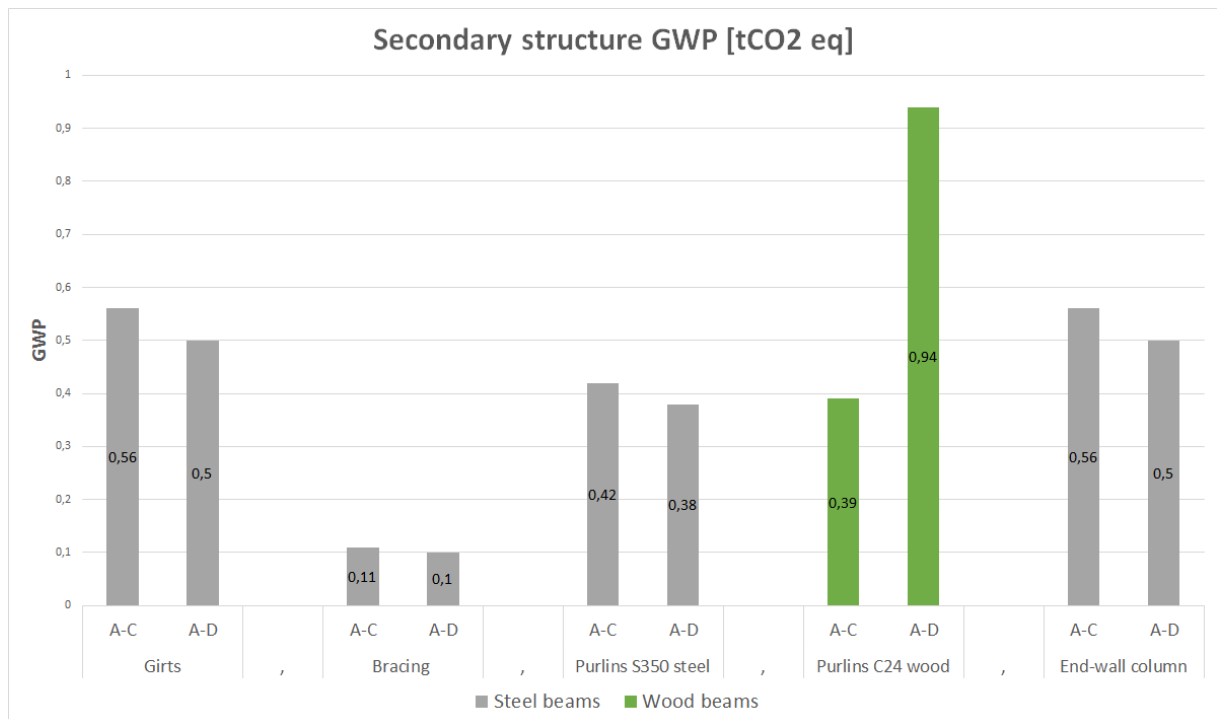


Figure 4.10: GWP of secondary structure from stage A to C and from A to D. For one module.

4.3.3 Analysis of primary and secondary structures

Once that all components were studied individually, it is now possible to combine the primary and the secondary structures to compare them in terms of eco-efficiency. Below, the reader may notably see the influence of the choice of the material for the purlins on the total environmental impact. All sizes and constructive methods are compared in the three graphs below. The girts and bracings are considered in every situation. Steel purlins were chosen for steel solutions and timber purlins for wood/concrete solution. In the case of the precast concrete halls, both materials are possible for the purlins so the two possibilities are analyzed.

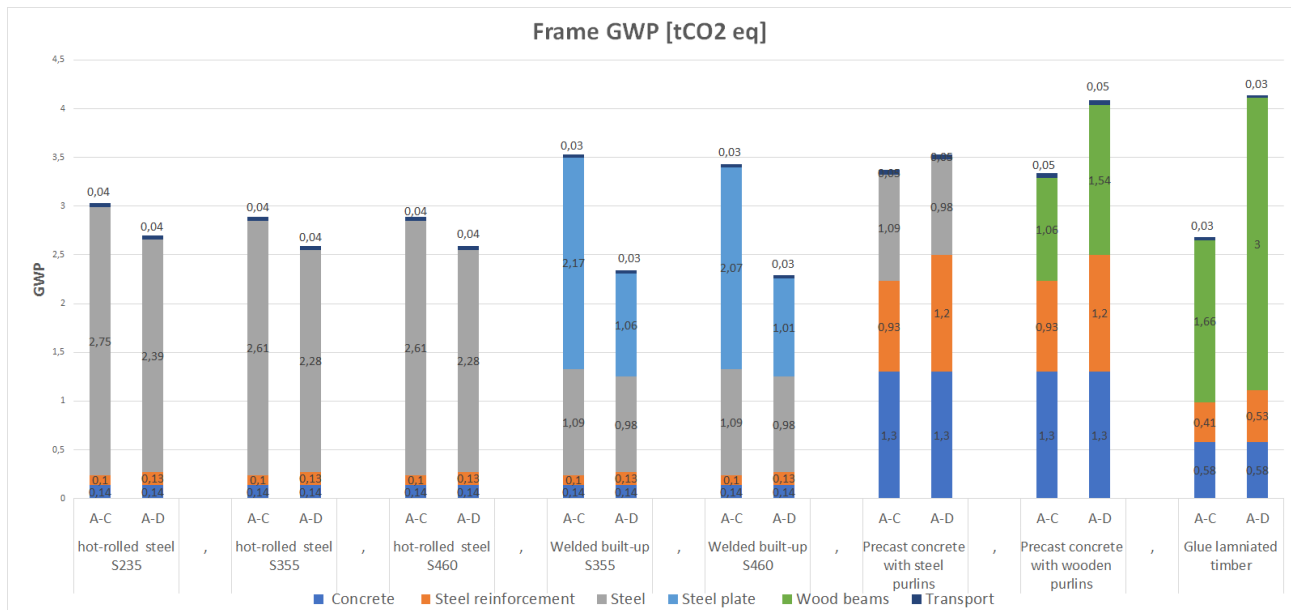


Figure 4.11: GWP for different frame and secondary structure materials from stage A to C and from A to D for 16 m span

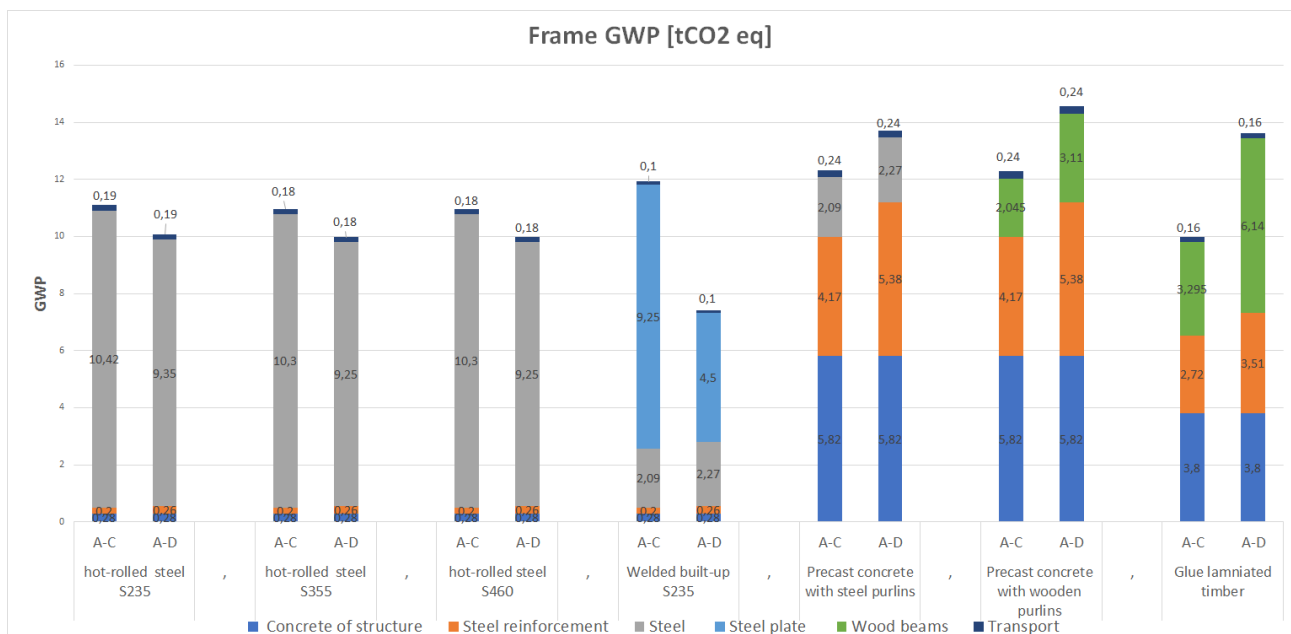


Figure 4.12: GWP for different frame and secondary structure materials from stage A to C and from A to D for 24 m span

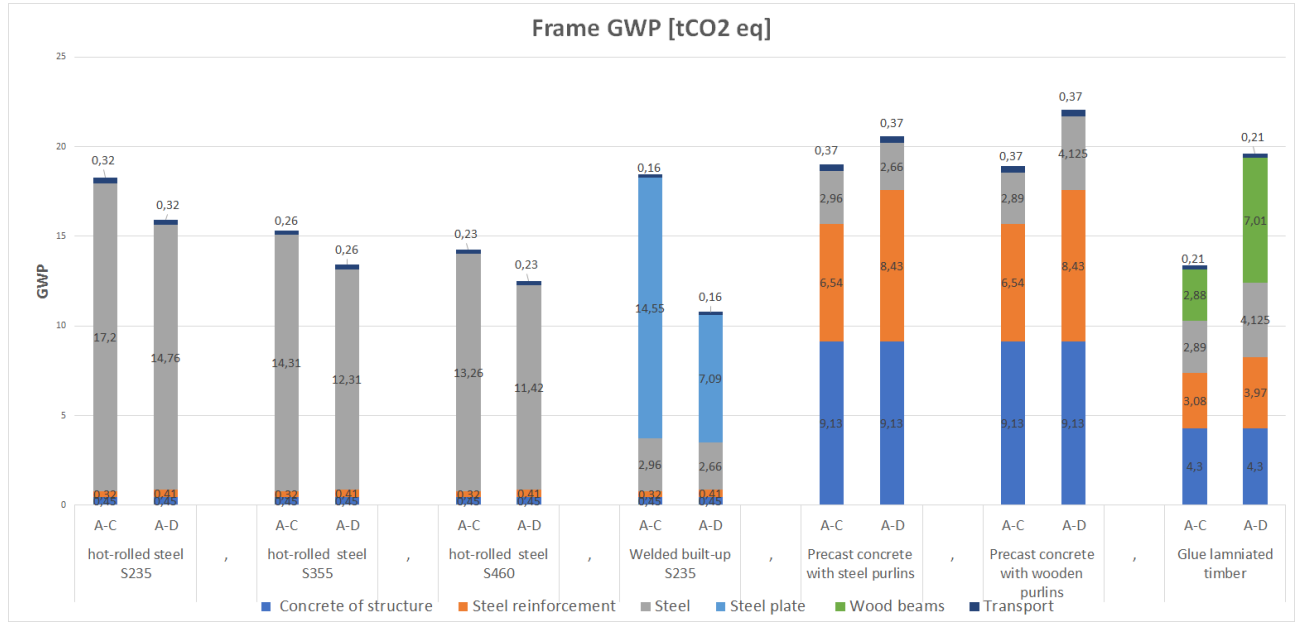


Figure 4.13: GWP for different frame and secondary structure materials from stage A to C and from A to D for 36 m span

The secondary structure has a significant consequence on the environmental impact no matter the building size. However, it does not change the previous observations and welded built-up remains the most "green" if recycling is taken into account while the concrete/timber solution is the best choice if the D phase is ignored.

4.4 Building envelope

In order to gain knowledge about the primary energy saving when the building is insulated, two types of envelope are considered in this section: a highly insulated (the warm one) made of sandwich panels and a poorly insulated (the cold one) made of aerated concrete for the walls and sandwich panels for the roof. These two variants are compared regarding the operational phase of all three studied halls. The various building envelopes and their physical properties are listed below in table 4.2.

	Cold envelope	Warm envelope
External walls	Siding slab "HEBEL" CC 3/500 $e = 365$ [mm], $U = 0.305$ [W/m^2K]	Ondatherm 1040 B Sandwich panels $e = 200$ [mm], $U = 0.126$ [W/m^2K]
Roof	Ondatherm 1040 TS $e = 60$ [mm], $U = 373$ [W/m^2K]	Ondatherm 1040 TS $e = 80$ [mm], $U = 0.31$ [W/m^2K]
Ground slab	Not insulated, $U = 0.599$ [W/m^2K]	

Table 4.2: Technical features of building envelopes

The first goal is to determine the number of years after which the energy saved thanks to a highly insulated envelope is greater than the extra energy required to produce it. Indeed, the more efficient is the insulation, the higher is the primary energy expenditure required to produce the envelope. This energy, which is obtained in GJ with Ameco is converted to MWh. The energy

demand as well is estimated using the Ameco software. The condition demanded to allow the software to compute the consumption of the heating system is a constant indoor temperature of 18°C. From this restriction, the program gives the energy needs in MWh for all three sizes. The output data are available in the table 4.3.

The comparison can now be made between the two envelopes. Energy consumption over the years are summed to obtain the accumulated consumption year after year for both technologies. The intersection between the obtained lines in the figures 4.14, 4.15 and 4.16 indicates the time in years that will be required for the investment in primary energy to become energetically profitable.

		Primary energy demand for production	Estimated annual primary energy demand for heating
Size 1: Height=4 [m] Span=16 [m]	Cold envelope	215 [MWh]	6.5 [MWh/a]
	Warm envelope	230 [MWh]	12.5 [MWh/a]
Size 1: Height=12 [m] Span=24 [m]	Cold envelope	403 [MWh]	91 [MWh/a]
	Warm envelope	550 [MWh]	36.5 [MWh/a]
Size 1: Height=12 [m] Span=36 [m]	Cold envelope	450 [MWh]	112 [MWh/a]
	Warm envelope	633 [MWh]	50.5 [MWh/a]

Table 4.3: Primary energy expenditure for production and annual primary energy demand for heating

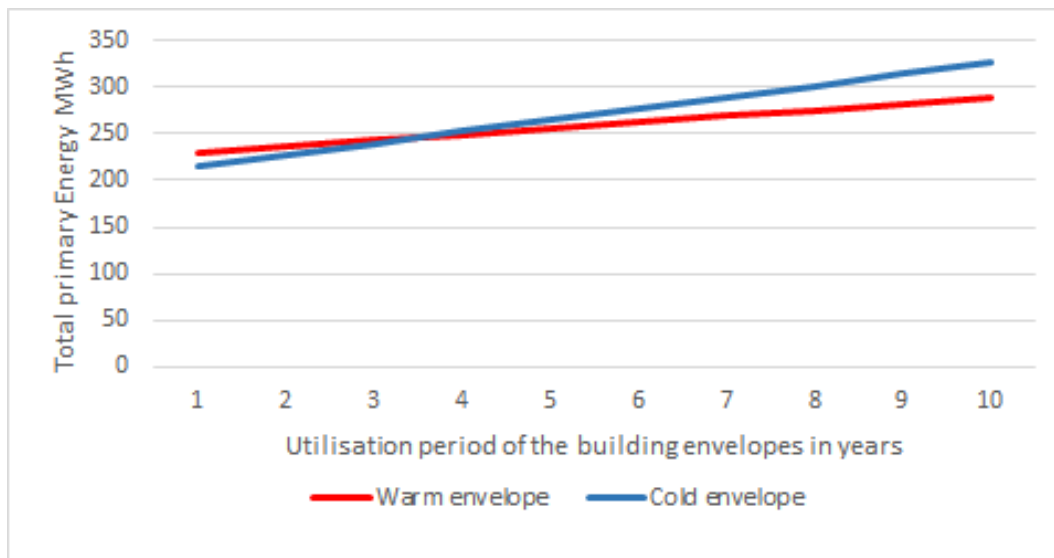


Figure 4.14: Comparison of primary energy demand for the warm and col envelopes over an utilization period of 10 years (for the smallest hall)

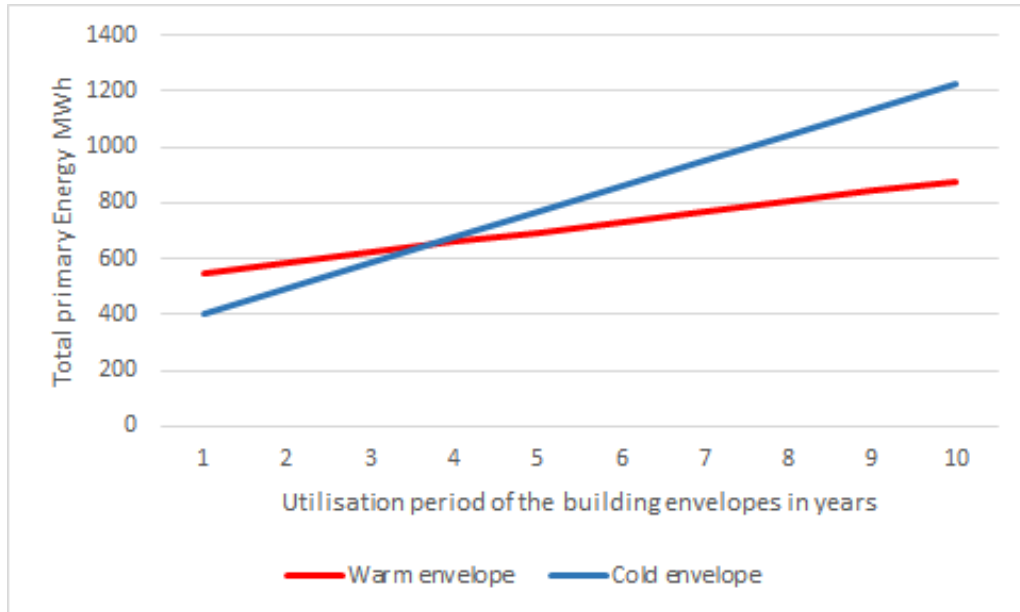


Figure 4.15: Comparison of primary energy demand for the warm and cold envelopes over an utilization period of 10 years (for the medium hall)

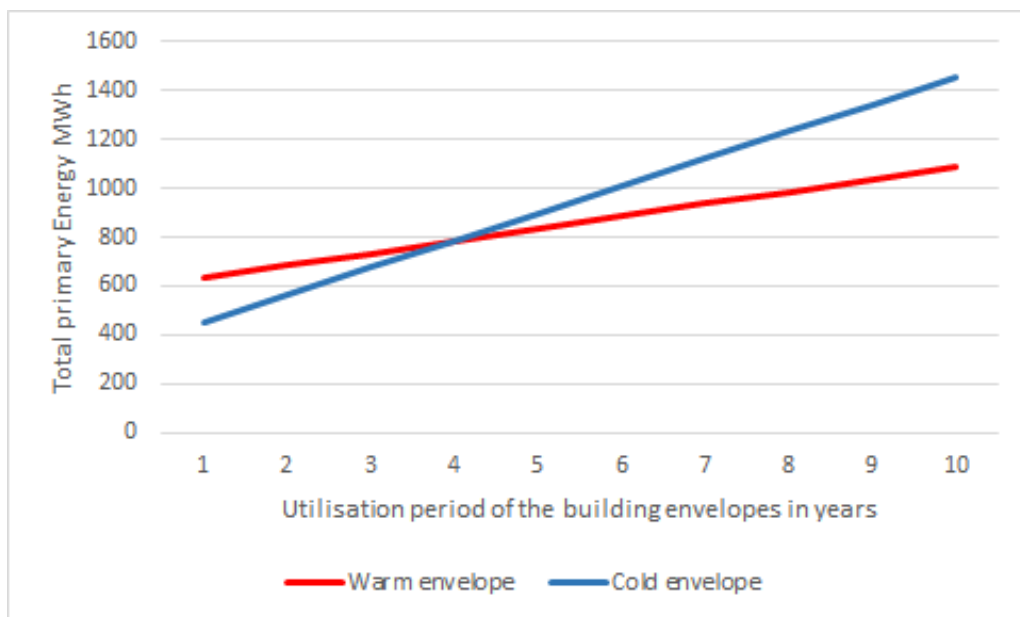


Figure 4.16: Comparison of primary energy demand for the warm and cold envelopes over an utilization period of 10 years (for the biggest hall)

In terms of primary energy demand, the choice of the warm building rather than the cold one pays off the energy investment after 4 years regardless of the size of the hall. The additional energy for production of the warm envelope compared to a poorly insulated building is compensated by the lower demand for operational energy to maintain the temperature at 18°C. Figure 4.17 illustrates this outcome and shows the real energy saving over 4 years. With this graph, we observe that the larger the building is the more energy is saved. A large building will therefore have more interest to be insulated.

This example of comparison shows the importance of considering the entire life cycle. As buildings are designed for a long period of use, the right decisions must be taken during the studies to make it the least expensive in the long run.

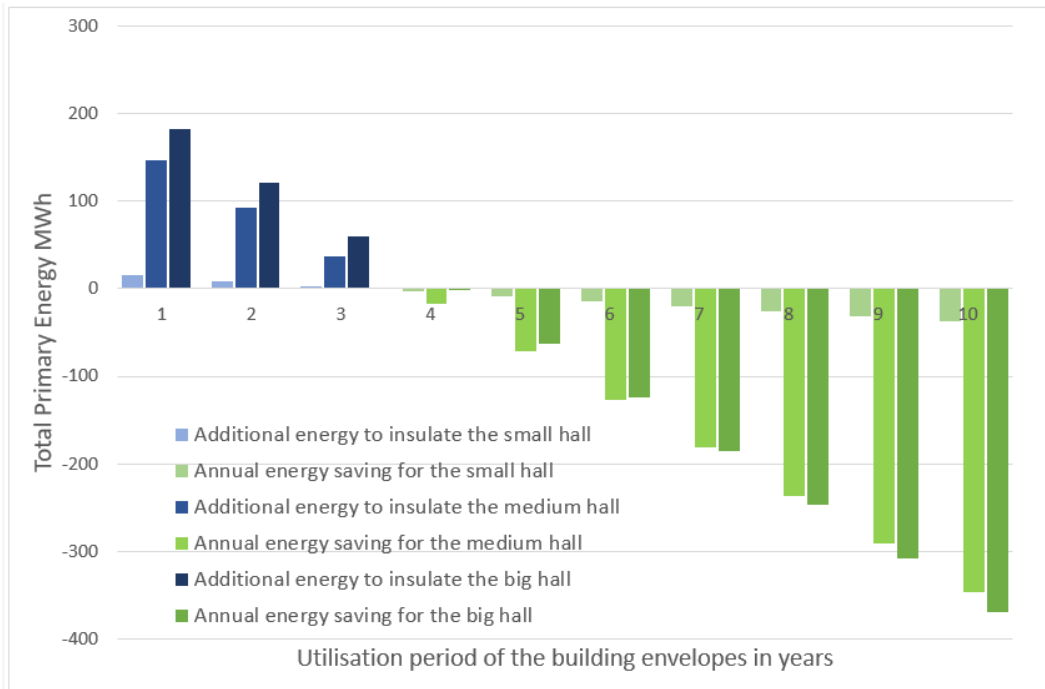


Figure 4.17: Energy saving over 10 years with insulation

4.5 Partial conclusion: Environmental impact

Throughout this chapter, the environmental effect was assessed using a LCA methodology that allows to quantify human actions on the planet. This analysis was carried out for all structural systems and also for all the sizes studied with a simple frame, its foundations and its secondary structure as the functional unit.

It was shown that steel was the most efficient material if the recycling stage was taken into account. It should also be noted that optimization of the profiles improves the eco-efficiency because of the mass reduction. Therefore, the welded built-up solution is the most ecological among steel solutions. If the recycling isn't considered, then, the wood is the most ecological in terms of GWP. Furthermore, the conclusion regarding the EE depends on two criteria: the inclusion of the D stage and the distinction between renewable and non-renewable energy. If the recycling is involved, then the wood appears to have poor eco-efficiency. However, if phase D is neglected and if only non-renewable energy is compared, the wood solution impact is similar to steel solutions. Precast concrete solutions, otherwise, have a medium impact if the end of life is neglected but are not "green" at all otherwise. This is because of their inability to be recycled.

The consideration of the secondary structure brought the same conclusions: wood is more attractive as a material for purlins if the recycling phase is neglected while steel is preferable as soon as all life stages are considered.

The two types of foundations were also considered. The foundations for the encased columns are much more massive than the foundations for the pinned columns. They also require more steel reinforcement to sustain the bending moment. This additional material consumption involves logically an increase of both the greenhouse gas emission and the embodied energy. To give some figures, the first type is about two times more polluting than the other. As a reminder the encased columns are used for the constructive solutions involving precast columns.

The change of the size does not affect the previous conclusions. The proportional gain (regarding GWP) done by choosing the welded built-up rather than other solutions doesn't systematically increase with the span. Depending on the materials, tendencies are divergent.

Transport represents only a very small part of the greenhouse gas emissions and is directly proportional to the mass of the elements transported. So the heavier elements like concrete parts have a slightly higher GWP.

The second part of the chapter covered the comparison of two building envelopes with different thermal properties. A "cold" one made of concrete block for the wall and sandwich panel for the roof and a "warm" one only made with sandwich panel with better thermal properties. This comparison shows that, regardless of the size of the halls, the primary energy investment becomes profitable after 4 years. It also brings out the importance of considering the entire life cycle of the building and not only the production and the end of life stage.

Chapter 5

Assembly Phase

After the design of the halls and the calculation of the environmental impact, this chapter focuses on the comparison between the assembly times of the different constructive solutions. The assembly time is proportional to the number of elements that need to be placed and their ability to be handled. Therefore, conclusions from the comparison between materials are similar from one building size to another. That is why only the basic case (16x42m) is considered.

The first part of this chapter concerns the different assumptions about the time required to carry out the necessary construction work.

After this, the execution time is compared considering three distinct construction stages:

- The construction of the foundations
- The assembly of the primary structure
- The installation of the roof and facades cladding

In order to estimate the assembly time of each stage, a database from ArcelorMittal was adapted to suit the conditions. Knowing the unit time for all the elements, the total duration can be quantified.

Finally, the last part is devoted to the establishment of a planning. This allows an easier understanding of the comparison between different combinations of structure solutions and envelope types.

The step of the construction in particular depends a lot on some external factors like weather, quality of the communication with the client, delivery delay, etc. Hence, even though we made all assumptions to reflect the reality as precisely as possible, reader must keep in mind that results can be very different depending on the conditions.

Let's notice that only installation time was considered. The displacements of the workers and the machines on the construction site, the time for the manufacturer to bring materials on site, the site installation, the concrete curing time, the concrete slab and other finishings were not considered. However, these elements are similar from one solution to another and has no impact on the comparison between them.

5.1 Assumptions

The first assumption is to consider only one team of workers and one crane. Hence all operations are successive and cannot be performed in parallel except for the concrete curing. This explains why these steps are not taken into account.

Regarding the working time of workers teams, the Belgian legislation is taken as a reference. Hence, a normal work day of 8 hours is considered.

Many tasks, like columns and girders mounting, assembly of facades and roofing, require the use of the crane. This last one should be used during 8 hours but unforeseen problems are unavoidable on a construction site. Therefore, one assumes that the effective crane time use is 6.4 hours per day.

To estimate the total mounting time, an unit time must be chosen for each task. The different assumptions for all these tasks are shown in the table 5.1. The reader should note that the assembly times for column and girder depend on the used materials. This is due to the difference between joining techniques from one material to another.

Hypothesis	Real Crane time use /days	8,00	h		
	Crane alea	0,20			
	Effective Crane time use /days for steel	6,40	h		
	Working time/days	8,00	h		
	Effective Working time/days	8,00	h		
Steel	Steel girder mounting (time per element)	0,33	h	20	min
	Steel Column (time for 1 columns)	0,25	h	15	min
	Steel frame (time for 1 frame)	0,58	h	50	min
Concrete	Concrete girder	0,40	h	24	min
	Concrete column (time for 1 column)	0,80	h	48	min
	Concrete frame (time for 1 frame)	2,00	h	120	min
Wood/concrete	Wood girder	0,33	h	20	min
	Concrete column (time for 1 column)	0,80	h	48	min
	Wooden frame	1,93	h	116	min
Envelope	Siding slab "HEBEL" (L=6 m, h=0,6 m)	0,16	h	9,6	min
	Purlins and girts	0,14	h	8,13	min
	Ondatherme 1040 B (L=12 m, h=1 m)	0,13	h	8	min
	Ondatherme 1040 TS	0,20	h/m ²	12	min/m ²
Foundation	Grading	1,00	h/m ³	60	min/m ³
	Placing of Rebars	5,88	h/tons	353	min/tons
	Concreting	0,13	h/m ³	7,5	min/m ³

Table 5.1: Assumptions on the mounting time of each elements

Let's notice that most of the unit mounting times remain very close from one hall size to another. However, the concrete beams are the exception to this rule because of their considerable mass. Indeed, the larger and the heavier a beam is, the most difficult and time-consuming it is to handle by crane. It can be considered that 24 m span girders take 1.5 times longer than 16 m girders to be placed and 36 m span girders take 2 times longer than 16 m girders to be placed.

5.2 Extensive comparison for each construction step

The various solutions studied in the previous chapters are now compared between them in terms of installation time.

The first part of the time comparison is devoted to the foundations. The type of foundation does not influence the number of tasks to be carried out and the following steps are involved in each foundation installation:

- Grading and excavating the soil
- Placement of the steel reinforcement
- Concreting

As shown in table 5.2 below, the foundations for the concrete encased columns require more time to complete. Indeed, the volume of this type of foundation is widely bigger than the volume of the pinned one (see chapter 2.8.1). This additional volume logically produced an increase in the amounts of soil to remove, of the steel to install and of the quantities of concrete to pour. Therefore, all these additional quantities lead finally to a longer execution time.

Foundations solution	Time for Grading [min]	Time for Placing of rebars [min]	Time for concreting [min]	Time required for one foundation [min]	Time required for the complete foundation system [h]
Pinned	9,33	13,72	1,17	24,2	6,5
Encased	67,20	26,14	2,48	95,8	25,6

Table 5.2: Work time required for the foundation systems¹

The second part examines the mounting time of the structural system. The number of elements installed with the crane is independent of the materials used. However, the joining techniques are different for each material.

Concrete columns will be assembled using waiting bars emerging from the foundation. These ones are inserted into sleeves in the column and overlap the main reinforcement once the sleeves have been filled with mortar and this one has cured. This way of assembling takes much longer than the simple bolting of the steel elements. This is why the solutions including concrete columns take longer to build up than those made only of steel.

¹The reader must keep in mind that these times represent the execution phase only. The displacements of workers and machines are not considered. This explains the very small values that appear here.

Hall structural solution	Time for one frame [min]	Time for the complete hall [min]	Time for the complete hall [h]	Days required for complete hall
Steel	50	400	6,67	1,0
Concrete	120	960	16,00	2,0
Concrete columns and glue laminated timber girders	116	928	15,47	1,9

Table 5.3: Work time required to mount the structures

The last element to compare is the envelope of the building. As a reminder, walls of the "cold envelope" are composed of siding slabs, 6 m in length and 0.6 m in height, directly laid on the slab (which is not considered here) and laterally fixed to the columns. Then, girts are unnecessary and their mounting time isn't considered. The roof is covered with sandwich panels and therefore, the installation time for purlins must be added.

The other envelope solution, the "warm one", is only composed of sandwich panels, 12 m in length and 1 m in height, and thus the time for girts and purlins installation is taken into account in the estimation.

It becomes clearly advantageous to use only sandwich panels for the envelope of the buildings. Indeed, in addition to offering a better thermal insulation, the assembly time of this solution is 27% faster to set up. The details of installation times are available below in figure 5.4.

Envelope solution	Time to install the walls [min]	Time for girts installation [min]	Time for purlins installation [min]	Time to install the roof [min]	Total time required [h]	Days required
Cold	618,67	0	626,01	1008	37,54	5,87
Warm	194,67	390,24	626,01	1008	36,98	4,62

Table 5.4: Comparison between envelopes regarding the mounting time

All the structure solutions studied can be combined with the different envelopes depending on the desired insulation. These combinations allow to determine the number of days required for mounting the complete chosen structure. This total time for each combination is found in table 5.5.

Solution	Total days required
Steel solution - Warm envelope	6,47
Steel solution - Cold envelope	7,72
Concrete - Warm envelope	9,82
Concrete - Cold envelope	11,06
Concrete columns and glue laminated timber girders - Warm envelope	9,75
Concrete columns and glue laminated timber girders - Cold envelope	10,99

Table 5.5: Comparison between different halls regarding the total time required

5.3 Construction planning

For a more visual comparison a schedule based on the previous sections in the form of a Gantt chart has been produced. To avoid any underestimations of the execution time, the days assigned to each task have always been rounded up. As explained above, draw up the planning of a project is not an exact science and depends on a lot of factors like work accidents, errors with purchases, bad weather or more simply human factor so the following chart must be considered as an approximation.

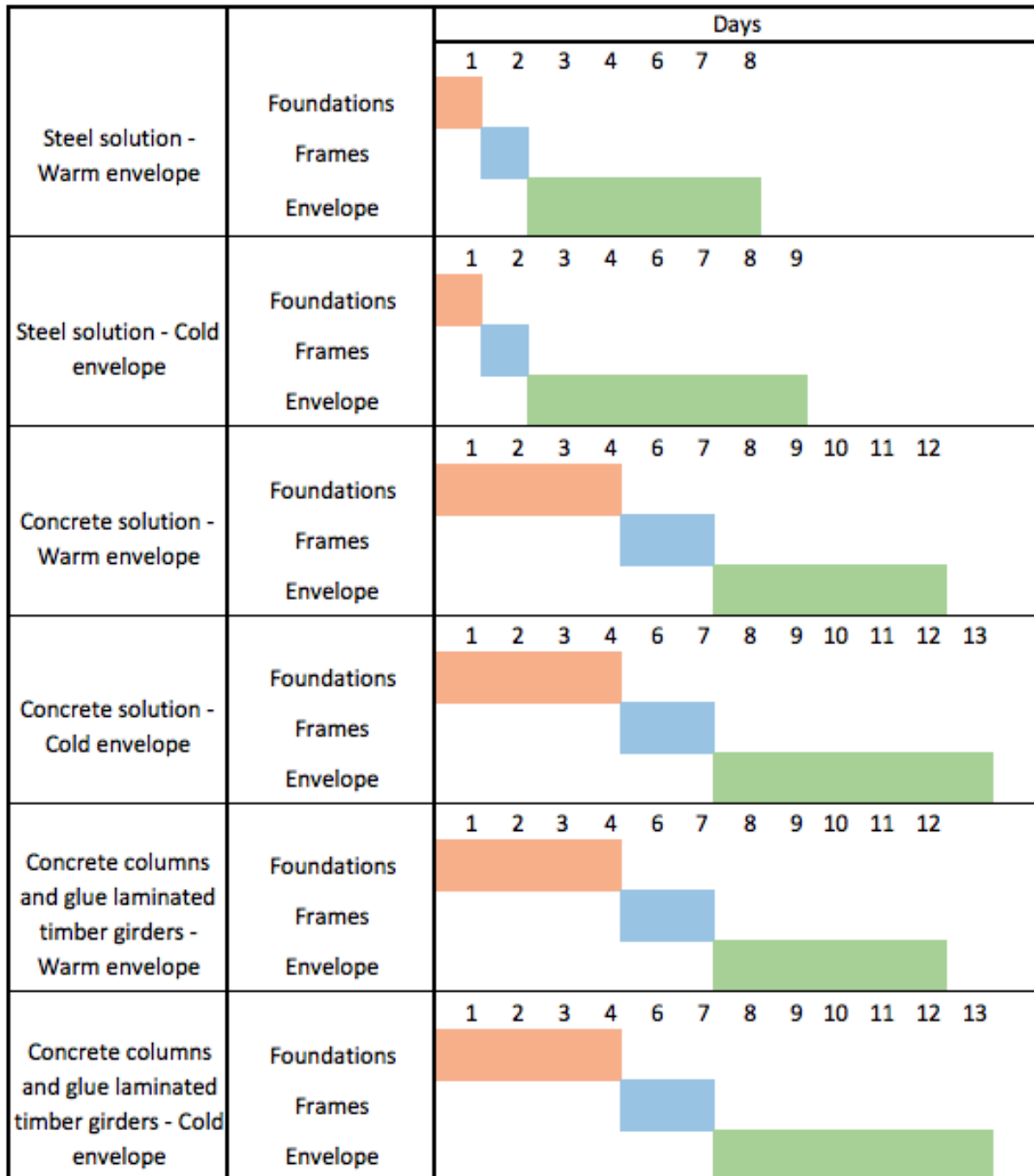


Figure 5.1: Complete halls assembly time comparison for different solutions

5.4 Partial conclusion: Assembly time

This chapter was dedicated to the comparison of assembly times for the different constitutive elements of industrial halls. Now, let's make some comments on the different observations based on the comparison:

- As seen previously, the foundations associated with the two solutions requiring concrete columns must be designed to absorb the bending moment transmitted by the columns. This transmission isn't an issue for steel solutions with pinned-base. Hence, the first type of structure needs larger foundations. It leads to additional manual labour for the workers and this type of foundation finally takes four times longer to complete.
- Due to joining techniques of concrete and wood solutions which are more time-consuming than the bolting of the steel structure, the assembly times for the non-metallic structures is more than thrice as long as the time allocated for steel ones. Indeed, for the 16x42 m hall, 3 days are required for the mounting of steel structures while 7 days are necessary for solutions involving concrete or wood. These differences are even more marked if greater sizes are considered. Indeed, the mass of the concrete girders increases much faster than the mass of the other beams leading to much longer crane handling process.
- The step which has the most impact on the total erection time is the installation of the envelope. This stage represents 70% of the total time for lightweight steel structures and 50% for solutions with concrete columns.

Chapter 6

Limits of the method and future improvements

The goal of the present study was to analyze and compare as much building possibilities regarding the most important factors. Hence we consequently had to make choices to set the framework of the research. This chapter reviews the present dissertation and proposes some improvements:

- The framework of the study does involve some of the most common structural systems. Furthermore, we only considered one span frames while it could be interesting to push the comparison further with intermediate column.
- The joinings were not designed because it is not determining as regards the analyzed criteria.
- The fire resistance was not part of our study because the aim was to fit the Belgian market where fire standards for industrial halls are not very restrictive at all. [15].
- For the same reason we didn't check the resistance to seismic solicitations. We designed our structure to sustain the wind which is, in Belgium, often more restrictive than loads due to earthquakes[15].
- We restricted our task to the structure and the covering. Therefore, additional finishing stages like the heating system, accesses or overhead crane are not addressed here.
- Only two indicators (GWP and EE) were considered in the life cycle assessment. We chose them since they are the most widely used and in order to make our data easily comparable. However, other indicators could have helped us to refine our results.

Some of these considerations are unnecessary to complete the aim of an exhaustive comparison of all building approaches. Yet, some should be developed in the future in order to improve and to refine the conclusions that we proposed as the outcome of this report.

Conclusion

In this dissertation, we performed a comparative analysis of different constructive solutions for industrial hall of various sizes. We wanted this study to be integrated into the present context. Between 2010 and 2015, the number of new non-residential projects in Belgium grew by 9 % [19] despite the economic crisis of 2007. On the other hand, the construction and industrial sectors emitted 19.4 % of the carbon dioxide released in 2005 in our country [6]. Hence, both technical performances and environmental impact are crucial factors to be considered.

In the first chapter, we defined the frame of the study and presented the methodology for the comparison. Next, the design process was implemented and performed. It allows further analyses and gathers all the essential indications required to design an industrial hall. This document can be a valuable assistance for students or junior engineers. Then, the designed solutions were compared according to their mass, eco-efficiency and installation time. As a conclusion, the comparative analysis enables us to provide recommendations to anyone involved in an industrial hall project, be it the project engineer, the contractors and subcontractors or even the project owners. These recommendations can be found in detail in the partial conclusions of each chapter, which are summarized hereafter.

Design process The stability study of a project often accounts for less than 5 % of the total budget[27]. However, it is crucial because it allows the design office to choose the structural systems that are easier to design. Regarding this criterion, the all-concrete solution is the best choice because catalogues are able to provide all the necessary data (available span, admissible loads, mass...) and engineers then only have to evaluate the loads. Hot-rolled steel profiles are also very easy to compute as softwares like Portal+ can speed up the process. On the contrary, welded built-up and wooden solutions require more complex calculation methods.

Mass The mass of the structure is a crucial parameter because numerous aspects of the project depend on it: the environmental impact, the ability to be handled, the transportation to the construction site... The use of concrete is a determinant factor. Indeed, solutions including this material tend to be the heaviest, no matter the dimensions of the building. Steel solutions, however, are much lighter. Using steel grade with higher yield strength and making use of variable inertia elements allows the engineer to optimize the structure and to reduce the required sections. Thus, the welded built-up solution made of the highest steel grade is the lightest solution regardless of the building size. Optimizing the sections by varying the inertia saves between 20 and 38% of the steel mass. However, foundations for clamped columns are 2 to 4 times heavier than foundations for pinned columns.

LCA In Belgium, the consideration of the D phase is compulsory. Therefore, the final recommendations take all the life phases into account, from the production phase (A) to the recycling

phase (D).

The welded built-up structure appears to be the most ecological solution.

Regarding the embodied energy, wood appears much less ecological but when isolating non-renewable energy, the eco-efficiency of wood becomes comparable to every other material.

Precast concrete solutions have poor results. This is because of its high mass and its inability to be recycled.

The secondary structure impact was also studied and the same conclusions have emerged: steel is a preferable material for purlins.

The study on the building envelopes showed that the choice of a highly insulated covering rather than a poorly insulated one becomes profitable (in terms of energy consumption) after 4 years of the building's use.

Assembly time Steel halls are installed much more rapidly than solutions involving concrete and/or wood. Indeed, the assembly times for the non-metallic structures is more than three times as long as the time needed for steel ones. The envelope also has a great importance regarding the mounting phase of a project. A cold envelope includes sliding slabs while a warm one involves sandwich panels. Yet, sliding slabs have smaller dimensions and more elements have to be handled to install the cladding. Thus, a cold envelope takes longer to set up.

This dissertation compared the most common construction solutions regarding various aspects. The analysis has shown that no universal solution exists, and that each of them presents different assets. The right choice will be the result of compromises and depends on the context and purposes of the project. The aim of this dissertation was to provide the information required to make such a choice knowingly

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