1918
TALLINNA TEHNIKAÜLIKOOL
TALLINN UNIVERSITY OF TECHNOLOGY

STRUCTURAL DESIGN OF AN ELECTRICAL ENERGY PLANT OF ESTONIA

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1. Resume

This master thesis is going to talk about a calculate and structural design of a factory that will be build for heating and electricity production. The name of this company is Eesti Energia AS Iru Elektrijaam. This factory have some buildings and we have choosen one of them because the procediment for the others it is the same. In the edificie choosen is used like flue gas treatment hall.

The factory is localized in Maardu, Tallinn, (Estonia) and during this semester will be started the works for build it. The results of the thesis will be compared with the results of differents kind of software and with the real results of the engineer who designed this project.

For this study, it will use the Eurocode 1(Actions on Estructures) and Eurocode 3(Design of steel structures). The software, that we will check our final results, will be applicated the eurocode parametres too.

We should mention that in preparing the thesis has always had the idea that this project will respresent the reality as possible. The project that was released and it may well be rated against other with similar characteristics and, therefore, has had as primary objective to comply with the regulations on projects of this nature and have a competitive cost.

2. Index

3. Introduction

3.1 Thesis objective

The purpose of the thesis presented is the calculation and design of steel structure designed for a factory of energy production, located in an industrial area in Estonia.

3.2 Thesis scope

Presented in this thesis is intended to perform the calculations necessary to determine what should be the metallic structure and the foundations for the realization of an industrial warehouse.

3.3 Current regulation

For the project will take into account the following regulations:

- EN 1991 Eurocode 1
- EN 1993 Eurocode 3

4. Body

4.1 Urban Location and justification

4.1.1 Location

The proposed building will be located in the municipality of Maardu, in the county of Harjumaa in Estonia. The location is shown in the map below. The UTM coordinates of the plot are X381695,69 and Y6591254,62 (zone 35V).

The plot delimits in the east and in the north with Peterburi tee street and in the north, north-west with a highway. It haven't got neighboring parcels.

4.1.2 Urban conditions

In this thesis, It will talk about one building of the factory because it is too big and we would need more time than one semester to do the full study.

In this section, It will be studied the urban conditions of whole the industry, because the plot information has been calculated with all the components and edifices.

For the exterior design of the geometry of the ship have been used the Eurocode 3 and the National Ordinances of Tallinn.

4.2 Charges and support services

4.2.1 Durability

The structure must be calculated so that the deterioration over the life of calculation does not impede the performance of the structure below expectations, taking into account the environment in which it is and the level of planned maintenance.

In order to obtain a properly structured long-term, it should consider the following:

-Intended and foreseeable use of the structure;

-The calculation criteria required;

-The expected environmental conditions;

-The composition, properties and performance of materials and products;

- Soil properties;

-The choice of the structural system;

-The shape of the elements and construction details of the structure;

-The quality of workmanship and level of control;

-Specific protective measures;

-Scheduled maintenance for the lifetime calculation.

Category lifespan calculation	Indicative estimate life in years	Exemples
1	10	Temporary structures(1)
2	10-50	Replaceable structural parts, for example, beams of the running, support equipment
3	15-30	Agricultural and similar structures
4	50	Structures of buildings and other structures common
5	100	Monumental building structures, bridges and other civil engineering structures.
(1) Structures or parts of structures that can be removed with the intention of re-used should not be regarded as temporary.		

We will use the category number 4 because our construction is a building. It is designed for a service period of 50 years. In this period, our factory has to bear the last limit states and states limit service without the structure suffered damage.

4.2.2 Current regulation

In preparing the project has followed the applicable regulations currently in the European Union in the field of industrial construction:

- Eurocode 1 (EN 1991): Actions and structures
- Eurocode 3 (EN 1993): Design of steel structure

4.2.3 States limits last

The last limit states are those that if it can't be fulfilled, can be a dangerous situation for the people, either because the building is out of service or because of the collapse all or part of it. You can consider the following types of limit states last:

- Loss of balance of the building or part structurally independent considered a rigid body.
- Failure due to excessive deformation, transformation of the structure or part of it into a mechanism, rupture of its structural elements (including supports and foundations) or their unions, or instability elements including those caused by structural effects depend on the time (fatigue, corrosion).

4.2.4 States limits service

The last limit states are those that were not complete, do not pose a danger to people but affect the comfort and welfare of the users or third parties to proper functioning of the building or construction appearance. The service limit states may be reversible or irreversible. As states service limits must be considered relative to the:

- Deformations (arrows, settlements) that affect the structure appearance, the comfort of users, or the operation of equipment and facilities.
- Vibrations affecting the comfort of users, or the operation of equipment and facilities.
- Damage or deterioration that may adversely affect the durability or functionality of the work.

4.2.5 Loads applied

4.2.5.1 Own weight

This section considers the weight of the girders and pillars that form the structure and elements of reinforcement.

4.2.5.2 Permanent actions

This section considers the weight represent roof straps, side straps as well as closures formed plate base with polyurethane insulation, in cover and in the side walls. It is also considered in this address the elements of the fixing elements included in the section.

4.2.5.3 Use overload

The "use overload" is the overhead due to the weight of all objects that can gravitate through use, even during execution. In general, the effects of live loading can be simulated by applying a uniformly distributed load.

These values include both the effects of normal use, people, furniture, household goods, common goods, content channels, where machinery and vehicles, as well as from the use unusual, as the accumulation of people, or furniture during a transfer.

Also, for local checks bearing capacity must be held to a concentrated load acting at any point in the area. This load is considered acting simultaneously with uniformly distributed load in the areas of traffic and parking use of light vehicles, and independently and not simultaneously with it in the other cases.

In our case, the snow action is more important that this overload and normally, when the roof has snow, we don't do any tasks or works in the winter when is completely snow-covered. So, we will not consider this action, but normally is between 2 and 3 kN/m^2.

4.2.5.4 Wind Action

The action of wind is represented by a simplified set of pressures or forces whose effects are equivalent to the extreme effects of turbulent wind. The wind actions should be considered as fixed variables actions. The effect of wind on the structure depends on the size, shape and dynamic properties of the structure. This standard covers the dynamic response to turbulence in the wind direction in resonance with the vibrations in wind direction in a fundamental way of bending of constant sign.

The wind pressure on exterior surfaces (w_e) should be obtained from the expression:

$$w_e = q_p(z_e) \cdot c_{pe}$$

$q_p(z_e)$ is the pressure at peak speed (21 m/s).

c_{pe} is the ratio of pressure to the external pressure.

z_e is the height of reference for external pressure.

And:

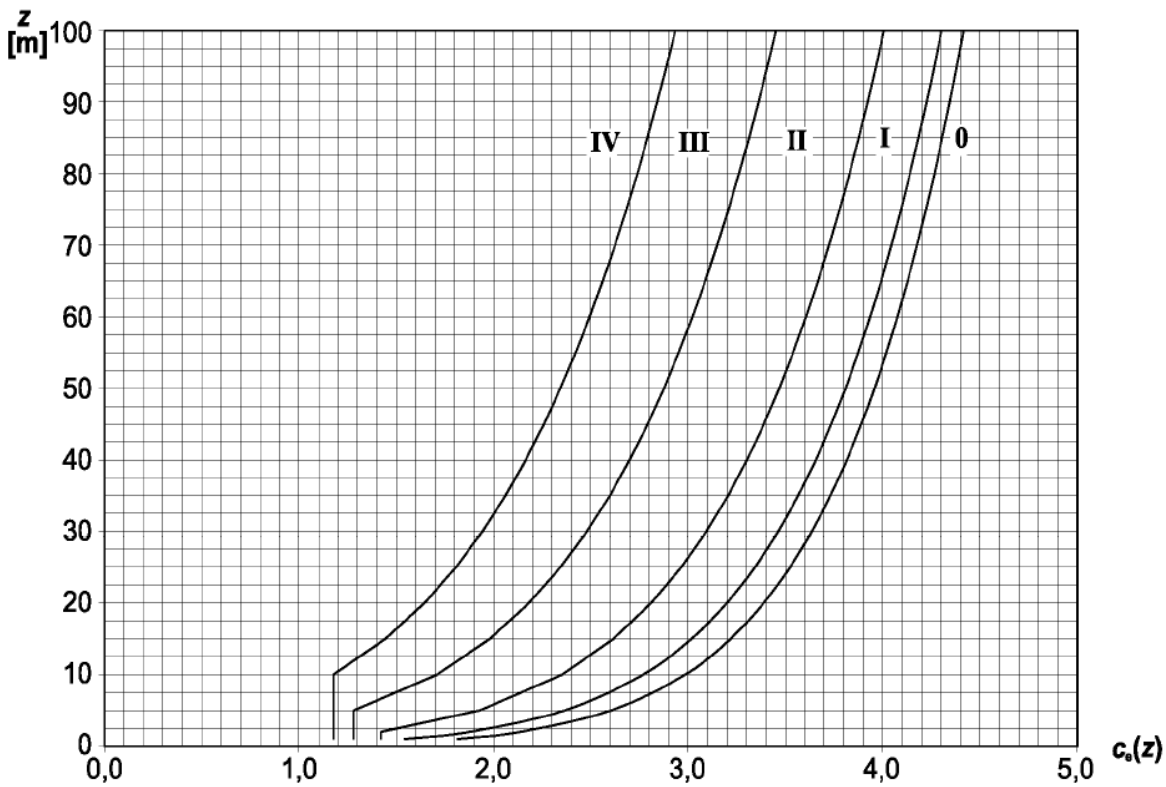
$$q_p(z_e) = c_e(z) \cdot q_b$$

$$q_b = \frac{1}{2} \cdot \rho \cdot v_b^2$$

ρ is the air density, which depends on altitude, temperature and barometric pressure expected in the region during wind storms(1,25 kg/m^3).

v_b^2 is the basic wind speed, defined in terms of wind direction and time of the year to 10 m in height of a plot of category II

$c_e(z)$ is the exposure factor of the graph

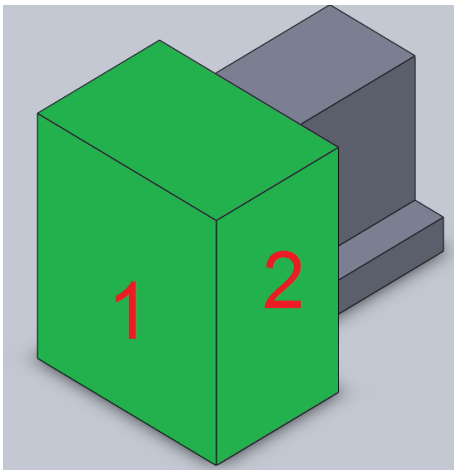


Our buildind is situated in an areea that is in the II category:

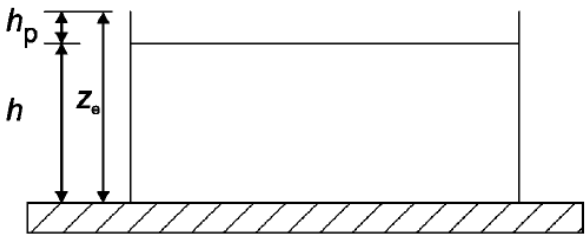
Terrain category	
0	Open sea or coastal area exposed to the open sea
I	Lakes or horizontal flat areas with negligible vegetation and unhindered
II	Areas with low vegetation such as grass and isolated obstacles (trees, buildings) with separations of at least 20 times the height of obstacles.
III	Areas with uniform vegetation cover or buildings or isolated obstacles with a maximum spacing of 20 times the height of obstacles (villas, land Suburban permanent forest)
IV	Areas in which at least 15% of the surface is covered by buildings which average height exceeds 15 m

The net pressure on the wall, roof or element is the difference between the pressures in each of its opposite faces, taking into account the sign. The pressure directed toward the surface is taken as positive, while sucking, directed outward from the surface is taken as negative.

TALLEST BUILDING



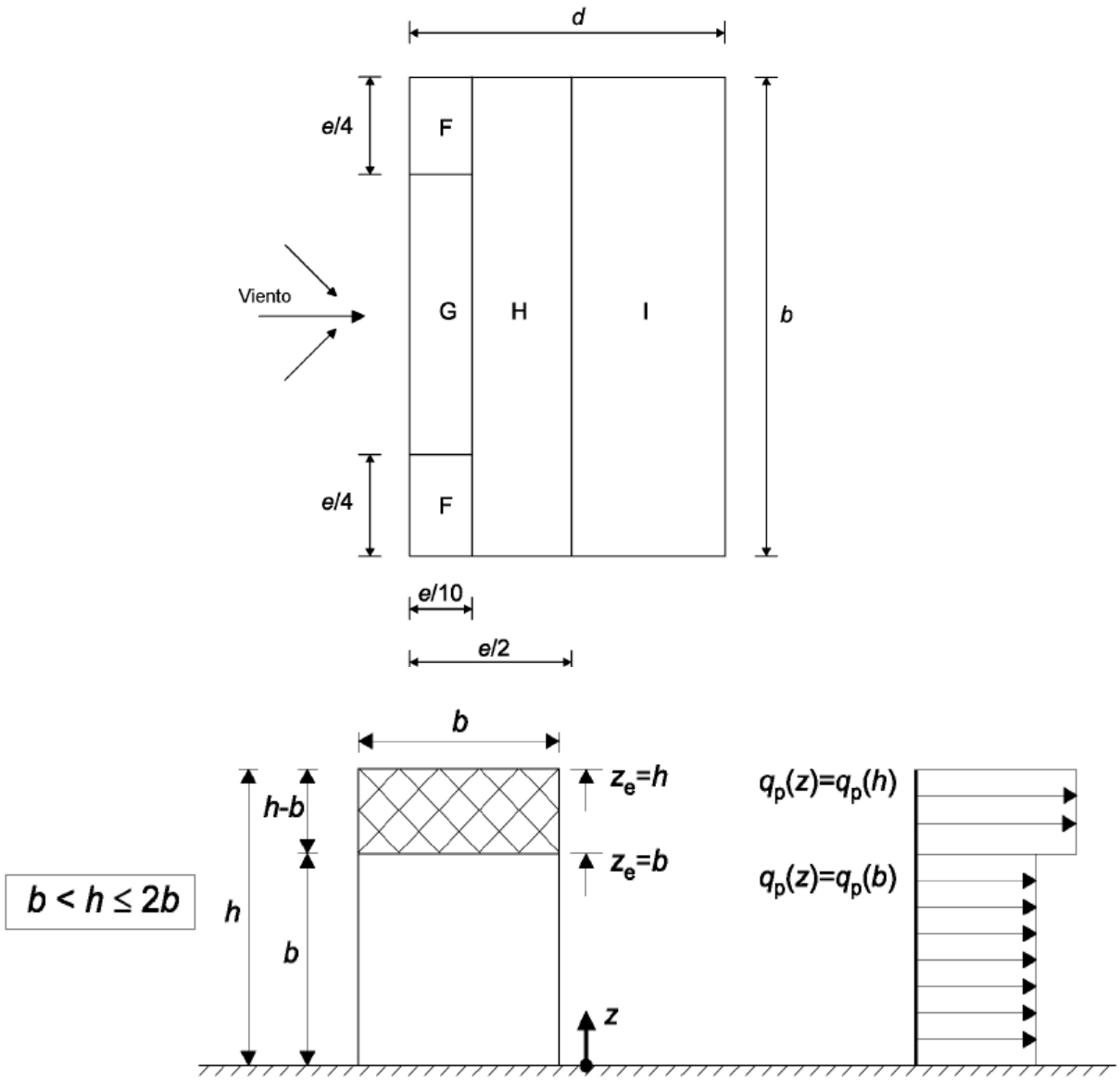
A flat roof is defined as one with a slope of -5 º to 5 º:



In our building:

$$h = 34\ m$$
$$h_p = 2\ m$$
$$z_e = 36\ m$$

The wind distribution is like this:



- <u>Side 1:</u>

  $e = \min(b_1; 2h) = \min(31{,}5; 2 \cdot 34) = 31{,}5$

  $d = \text{dimension transverse to the wind} = 21{,}5m$

$$\frac{h_p}{h} = \frac{2m}{34m} = 0{,}0588 \approx\ 0{,}05$$

*SEE ANNEX TABLE 1

F		G		H		I	
C_{pe1}	C_{pe10}	C_{pe1}	C_{pe10}	C_{pe1}	C_{pe10}	C_{pe1}	C_{pe10}
-1,4	-2	-0,9	-1,6	-0,7	-1,2	0,2	
						-0,2	

$$z_e = h = 36m \gg c_e(z) = 3{,}2 \gg q_p(z_e) = 968\,{}^{kg}/_{m \cdot s^2} = 968\,{}^{N}/_{m^2}$$

$$w_e = q_p(z_e) \cdot c_{pe}$$

$$w_{eF} = q_p(z_e) \cdot c_{peF} = -1.936\,{}^{N}/_{m^2}$$

$$w_{eG} = q_p(z_e) \cdot c_{peG} = -1.548{,}8\,{}^{N}/_{m^2}$$

$$w_{eH} = q_p(z_e) \cdot c_{peH} = -1.161{,}6\,{}^{N}/_{m^2}$$

$$w_{eI} = q_p(z_e) \cdot c_{peI} = \pm 193{,}6\,{}^{N}/_{m^2}$$

- <u>Side 2:</u>

  $\mathrm{e} = \min(\mathrm{b}_2; 2\mathrm{h}) = \min(21{,}5; 2 \cdot 34) = 21{,}5$

  $\mathrm{d} = \text{dimension transverse to the wind} = 31{,}5\mathrm{m}$

  $$\frac{h_p}{h} = \frac{2m}{34m} = 0{,}0588 \approx\ 0{,}05$$

*SEE ANNEX TABLE 1

F		G		H		I	
C_{pe1}	C_{pe10}	C_{pe1}	C_{pe10}	C_{pe1}	C_{pe10}	C_{pe1}	C_{pe10}
-1,4	-2	-0,9	-1,6	-0,7	-1,2	0,2	
						-0,2	

$$z_e = h = 36m \gg c_e(z) = 3{,}2 \gg q_p(z_e) = 968\,{}^{kg}/_{m \cdot s^2} = 968\,{}^{N}/_{m^2}$$

$$w_e = q_p(z_e) \cdot c_{pe}$$

$$w_{eF} = q_p(z_e) \cdot c_{peF} = -1.936\,{}^{N}/_{m^2}$$

$$w_{eG} = q_p(z_e) \cdot c_{peG} = -1.548{,}8\,{}^{N}/_{m^2}$$

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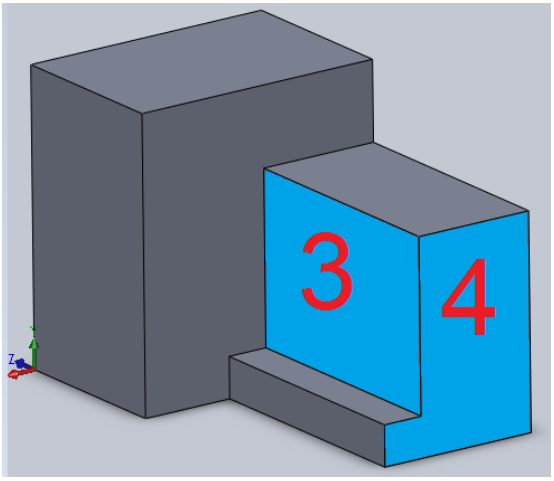
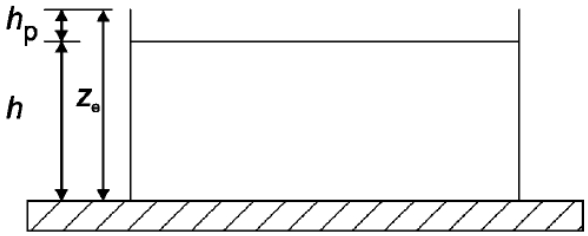
$$w_{eI} = q_p(z_e) \cdot c_{peI} = \pm 193{,}6\, {}^{N}/_{m^2}$$

SHORTEST BUILDING



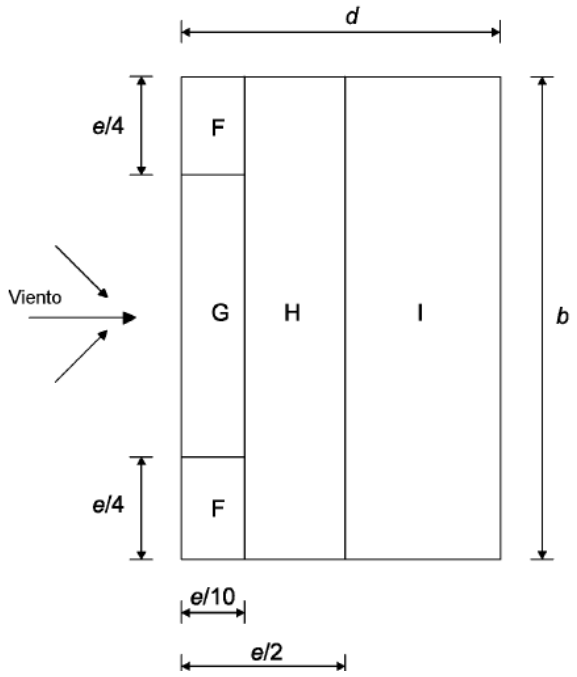
A flat roof is defined as one with a slope of -5 º to 5 º:



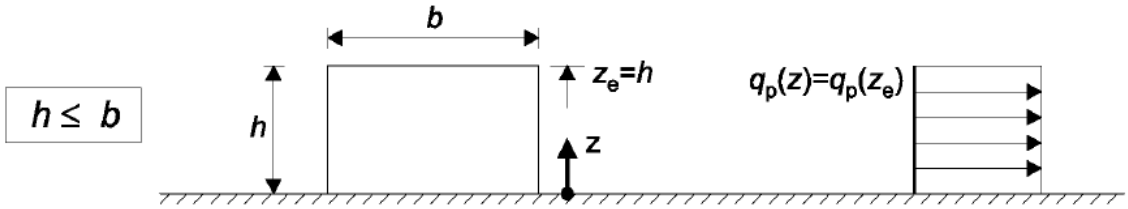
In our building:

$$\mathrm{h} = 23{,}5\ \mathrm{m}$$
$$\mathrm{h_p} = 2{,}7\ \mathrm{m}$$
$$\mathrm{z_e} = 26{,}2\ \mathrm{m}$$

The wind distribution is like this:



- <u>Side 3:</u>



$$e = \min(b_3; 2h) = \min(30; 2 \cdot 23{,}5) = 30$$

$$d = \text{dimension transverse to the wind} = 15\text{m}$$

$$\frac{h_p}{h} = \frac{2{,}7m}{30m} = 0{,}09 \approx 0{,}1$$

*SEE ANNEX TABLE 1

F		G		H		I	
C_{pe1}	C_{pe10}	C_{pe1}	C_{pe10}	C_{pe1}	C_{pe10}	C_{pe1}	C_{pe10}
-1,2	-1,8	-0,8	-1,4	-0,7	-1,2	0,2	
						-0,2	

$$z_e = h = 26{,}2m \gg c_e(z) = 3{,}0 \gg q_p(z_e) = 907{,}5\, {}^{kg}/_{m \cdot s^2} = 907{,}5\, {}^{N}/_{m^2}$$

$$w_e = q_p(z_e) \cdot c_{pe}$$

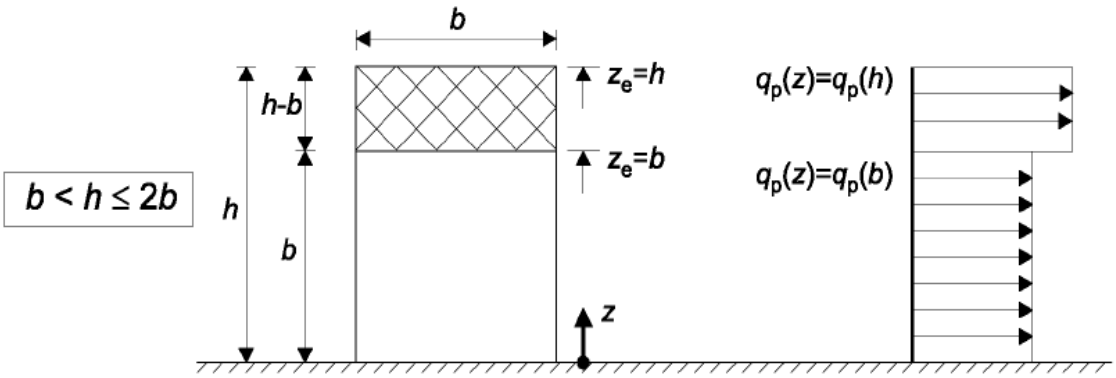
$$w_{eF} = q_p(z_e) \cdot c_{peF} = -1.633{,}5\, {}^{N}/_{m^2}$$

$$w_{eG} = q_p(z_e) \cdot c_{peG} = -1.270{,}5\, {}^{N}/_{m^2}$$

$w_{eH} = q_p(z_e) \cdot c_{peH} = -1.089\, {}^{N}/_{m^2}$

$w_{eI} = q_p(z_e) \cdot c_{peI} = \pm 181{,}5\, {}^{N}/_{m^2}$

- Side 4:



$\mathrm{e} = \min(\mathrm{b}_4; 2\mathrm{h}) = \min(15; 2 \cdot 23{,}5) = 15$

$\mathrm{d} = \text{dimension transverse to the wind} = 30\mathrm{m}$

$\frac{h_p}{h} = \frac{2{,}7m}{30m} = 0{,}09 \approx\ 0{,}1$

$$w_e = q_p(z_e) \cdot c_{pe}$$

$w_{eF} = q_p(z_e) \cdot c_{peF} = -1.633{,}5\, {}^{N}/_{m^2}$

$w_{eG} = q_p(z_e) \cdot c_{peG} = -1.270{,}5\, {}^{N}/_{m^2}$

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*SEE ANNEX TABLE 1

F		G		H		I	
C_{pe1}	C_{pe10}	C_{pe1}	C_{pe10}	C_{pe1}	C_{pe10}	C_{pe1}	C_{pe10}
-1,2	-1,8	-0,8	-1,4	-0,7	-1,2	0,2	
						-0,2	

4.2.5.5 Action stations

The structures in which the thermal depend mainly on its use are for example: towers cooling, silos, warehouses, cold storage facilities and heat, heating and cooling facilities, etc.

The elements of structures to withstand loads must be checked to ensure that thermal movements will not cause additional efforts to induce excessive stresses in the structure, either by available together or by considering these effects in the project. The magnitude of thermal effects depend on local climatic conditions, together with the orientation of the structure, its total mass, finishes and heating and ventilation systems and insulation in the case of building structures.

Due to the size and characteristics of the ship is not necessary to provide the planned expansions and retractions affect the safety of the structure, but should not forget this aspect.

4.2.5.6 Snow load

In locations where snow may occur off and which also may arise exceptional accumulations should be applied:

- Consideration should be given design situation persistent / transient load for the positions with or without accumulation
- Should be considered accidental design situation for loading positions with or without accumulation

The characteristic value of snow load at ground level (s_k) should be determined as described in paragraph 4.1.2 (7) P EN 1990:2002 and the definition of the characteristic value of snow load at ground level given in paragraph 1.6.1.

In specific locations where snow records displayed exceptional individual values can't be treated with standard statistical methods (like Estonia), should determine the characteristic values without regard these exceptional

values. Can be considered exceptional values outside the range of application of standard statistical methods as indicated in paragraph 4.3.

The representative values of snow load on a roof are as follows:

-Value combination ($\psi_0 s$)
-Frequent value ($\psi_1 s$)
-Quasi-permanent value ($\psi_2 s$)

Recommended values of the coefficients:

***Piirkond* (Region)**	**Ψ_0**	**Ψ_1**	**Ψ_2**
Kogu Eesti territoorium(territory of Estonia)	0,50	0,20	0,00

These values are not written in the Eurocode 3, because in this, Estonia hasn't been published and we have used the National Regulations of Estonia to get the most correct coefficients as much as possible.

The calculation for the loads of levels on the decks should consider that snow cover can be placed on providing several configurations. The snow load for persistent design situations is defined as:

$$\boxed{s = \mu_i \cdot C_e \cdot C_t \cdot s_k}$$

μ_i: Is the shape coefficient of the snow load.
s_k: Is the characteristic value of snow load at ground level.
C_e: Is the coefficient of exposure.
C_i: Is the temperature coefficient.

Recommended values for different topographies C_e

Maastik (Topography)	**C_e**
Tuulele avatud (Windage) [a]	1,0
Tavaline (Norma) I[b]	1,0
Varjatud (Protected) [c]	1,0

[a]flat areas with no obstacles, exposed on all fronts or less protected by terrain, for higher buildings or trees.
[b]areas is not expected redistribution of snow due to terrain, buildings or other trees.
[c]areas in which the work in question is considerably lower than the surrounding terrain and is surrounded by tall trees or buildings higher.

Last table is in the Estonian National Regulations and is not difference between the kind of topography that is situated our building.

In our building: C_e=1,0

The temperature coefficient C_t should be used to account for the reduction of snow load on roofs with high thermal conductivity (> 1 W/m2K), particularly in some glass-covered, due to melting snow due to thermal losses. The coefficient for the remaining cases:

$$C_t=1,0$$

The following expression and the figure shows the shape coefficients of the snow load should be used for ends of adjoining and nearby covered tallest buildings.

μ_1=0,8 (assuming the lower deck is flat)

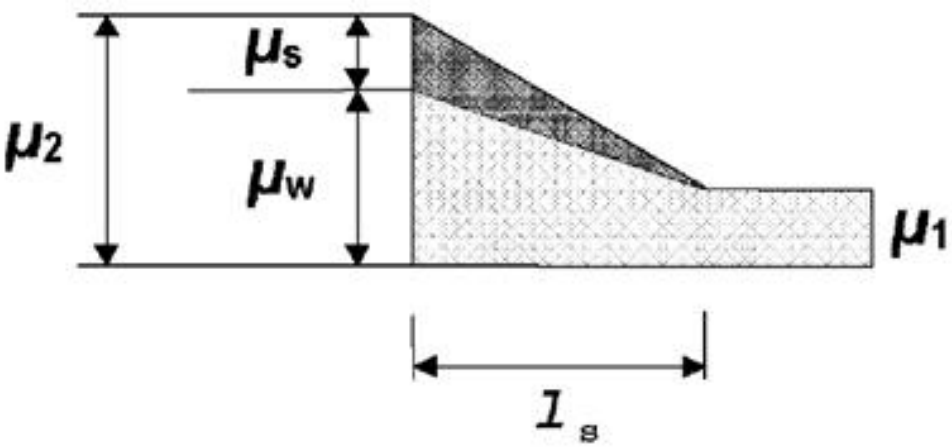
$$\mu_2=\mu_s+\mu_w$$

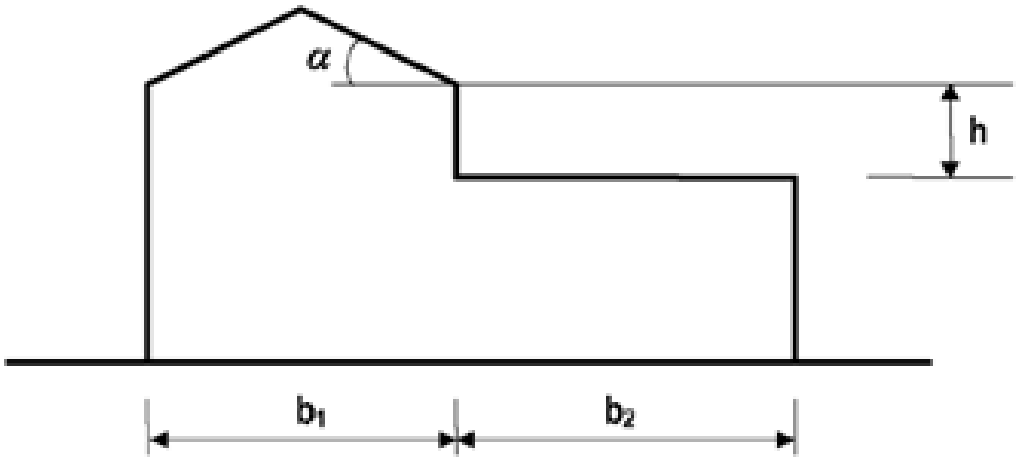
μ_s= is the coefficient of a load of snow due to snow sliding from the upper deck and is 0 if α=0.

μ_w= is the coefficient of a load of snow due to wind = $(b_1+b_2)/\ 2h \leq \gamma\ h/s_k$; and in Estonia this number has to verify $0,8 \leq \mu_w \leq 2,5$.

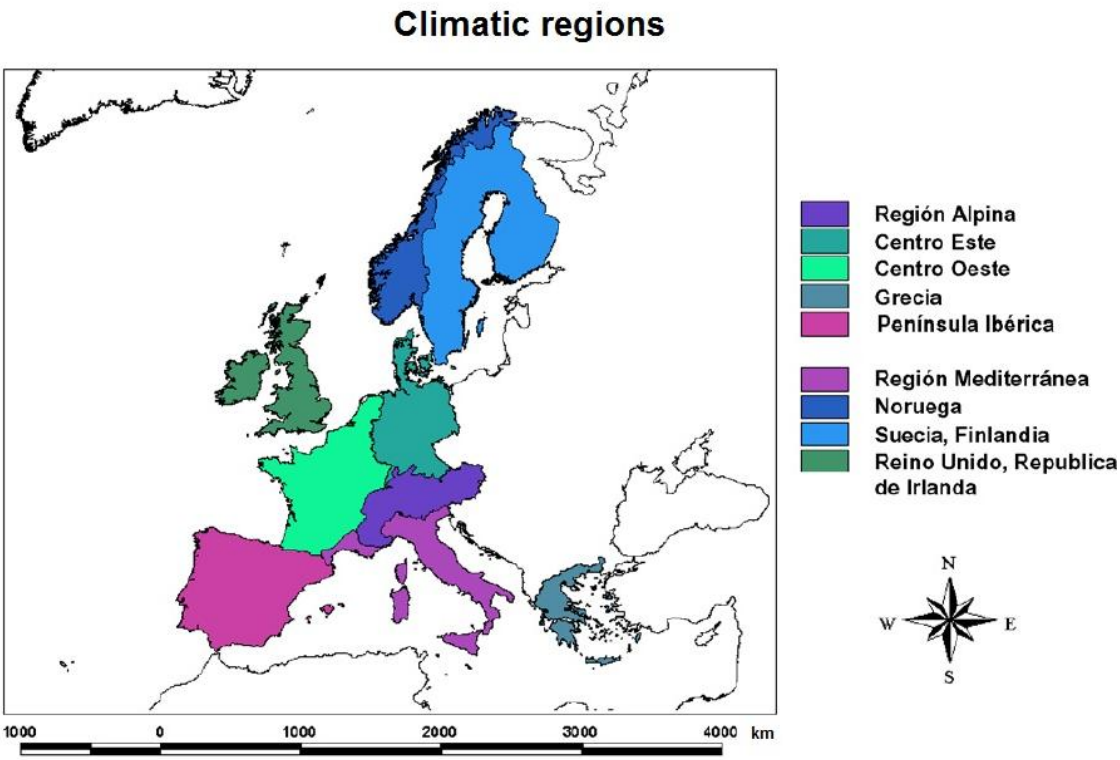
γ=is the specific weight that these effects can be taken equal to 2 kN/m^3.

l_s= 2 h = 2 · 11m = 22 m but it has to verify 2m ≤ l_s ≤ 6m.





To determine the value s_k we have used:



Our region is like Sweden and Finland. And the equation for this area is:

$$s_k = 0{,}790 \cdot Z + 0{,}375 + \frac{A}{336}$$

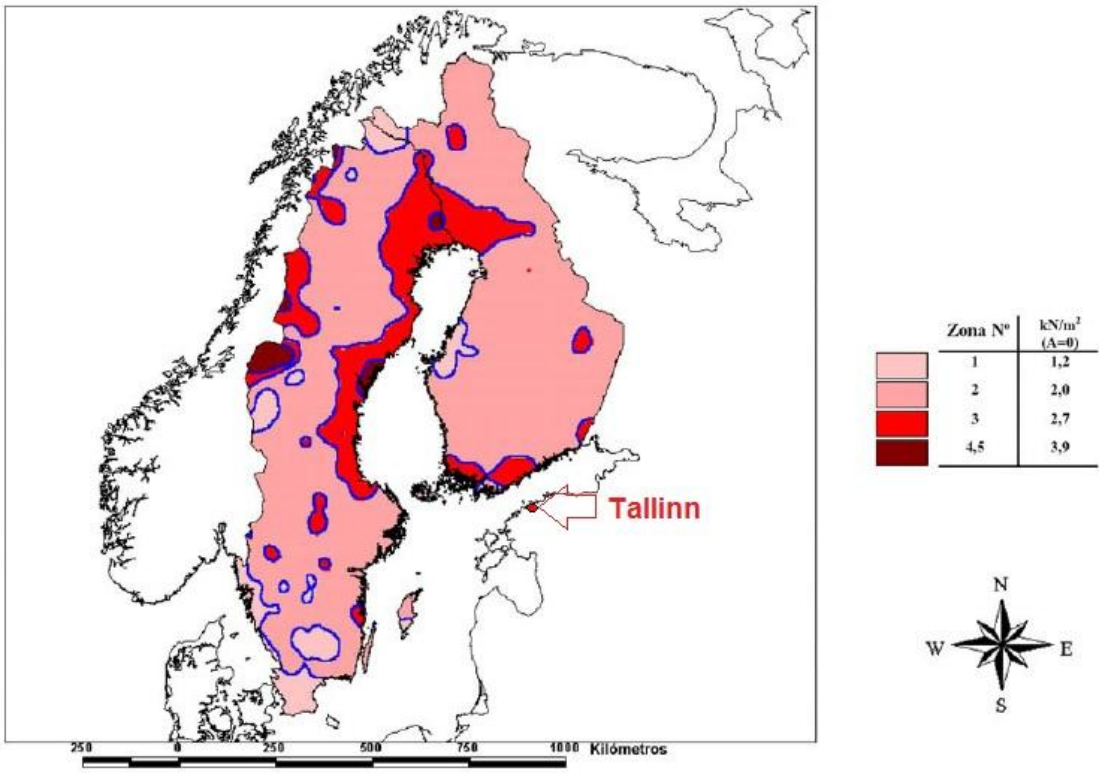
And:

s_k: is the characteristic value of snow load at ground level [kN/m^2].

A: is the site altitude above sea level [m].

Z: is the number of given area on the map.

Sweden, Finland: Loads of snow at sea



$\boxed{Z=3}$

The altitude at which we find our factory is 42 m above sea level:

$\boxed{A=42m}$

Accordingly:

$$\boxed{s_k = 0{,}790 \cdot Z + 0{,}375 + \frac{A}{336} = 0{,}790 \cdot 3 + 0{,}375 + \frac{42}{336} = 2{,}87 \; kN/m^2}$$

<u>Bottom Cover:</u>

$$\boxed{s = \mu_1 \cdot C_e \cdot C_t \cdot s_k = 0{,}8 \cdot 0{,}8 \cdot 1{,}0 \cdot 2{,}87 = 1{,}8368 \; kN/m^2}$$

<u>Upper Deck:</u>

$$\boxed{s = \mu_2 \cdot C_e \cdot C_t \cdot s_k = 2{,}34 \cdot 0{,}8 \cdot 1{,}0 \cdot 2{,}87 = 5{,}37 \; kN/m^2}$$

$$\mu_2 = \mu_s + \mu_w = 0 + 2{,}6275 = 2{,}34$$

$$\mu_w = \frac{b_1 + b_2}{2 \cdot h} = \frac{21{,}5 + 30}{2 \cdot 11\,m} = 2{,}34 \le \frac{\gamma \cdot h}{s_k} = \frac{2 \cdot 11}{2{,}87} = 7{,}67 \quad \text{OK}$$

4.2.6 Accidental Actions

4.2.6.1 Earthquake

The standard construction NCSE-02 establishes a classification of buildings based on the damage it can cause destruction regardless of the type of work concerned.

Then, the standard classifies buildings in buildings of moderate importance, of special importance and normal importance. The difference between them is as follows:

- *Constructions of moderate importance*: those with negligible probability that their destruction may be caused by the earthquake victims, an interrupt service parent or significant economic damage to third parties.
- *Constructions of normal importance*: those in which its destruction by the earthquake may cause victims, interrupting service to the community, or cause significant economic losses, without in any case, this is an essential service or can lead to catastrophic effects.
- *Buildings of particular importance*: those in which its destruction by the earthquake could disrupt an essential service or lead to catastrophic effects.

4.2.6.2 Fire

The structural design fire situation involves implementing related actions, both with thermal analysis, as with the mechanical analysis, under this part and other parts of EN 1991.

In this project we will focus more on the structural calculations and not on factors related to fire.

4.2.6.3 Vehicle Impact

The actions of a building caused by an impact depend on the mass, geometry and speed impactor, as well as the ability to both damping and deformation of the body such as the striking element against which impacts. The action of impact of vehicles from outside of the building will be considered when and where they set the bylaw.

To minimize the chances of a possible collision against the pillar structure of the ground floor of the building consider installing a pilot system to protect the bottom of the structure and provide security to people who are in it. Possible air accidents of collision against the top of the building will not be considered because they are not in a real probability. Although the planned building is considered tall, 35 meters does not pose a real risk on the aircraft or other flying.

In the specific case of this project is not considered necessary to take into account any practical action impacts due to land both for it's significantly too small, like air for its almost zero probability.

4.2.7 Combinations of actions

Combinations of actions to be taken into account in the relevant design situations should be appropriate for the requirements of serviceability and performance criteria that are being tested.

The value of calculating the effects of actions related to a persistent or transient state, is determined by combinations of actions from this expression:

$$E_d = \gamma_{Sd} \cdot E\left\{\gamma_{g,j} \cdot G_{k,j}; \gamma_p \cdot P; \gamma_{q,1} \cdot Q_{k,1}; \gamma_{q,i} \cdot \psi_{0,i} \cdot Q_{k,i}\right\} \quad j \geq 1 \; i > 1$$

The combined effects of the actions to be considered should be based on:

- The value of calculating the dominant variable action

- Combining the values of calculation variables accompanying actions:

$$E_d = E\left\{\gamma_{G,j} \cdot G_{k,j}; \gamma_P \cdot P; \gamma_{Q,1} \cdot Q_{k,1}; \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_{k,i}\right\} \quad j \geq 1 \; i > 1$$

The combination of actions between brackets can either be expressed as:

$$\sum_{j \geq 1} \gamma_{G,j} \cdot G_{k,j} + \gamma_P \cdot P + \gamma_{Q,1} \cdot Q_{k,1} + \sum_{i>1} \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_{k,i}$$

Where:

P: Value relevant representative of a prestressing action

$G_{k,j}$: Characteristic value of permanent action j

$Q_{k,1}$: Value characteristic of the dominant variable action 1

$Q_{k,i}$: Value characteristic of the action variable associated with i

Performing all possible combination and considering that the wind affects only one direction (V_1, V_2, V_3 and V_4 incompatible in the same case) and knowing that N_1 is the snow load (finally we haven't added the overload Q_1 because the snow is really bigger than this). All the possibilities that can occur are:

$$\gamma_{G,j} \cdot G_{k,j} + \gamma_P \cdot P + \gamma_{N1} \cdot N_1$$
$$\gamma_{G,j} \cdot G_{k,j} + \gamma_P \cdot P + \gamma_{V1} \cdot V_1$$
$$\gamma_{G,j} \cdot G_{k,j} + \gamma_P \cdot P + \gamma_{V2} \cdot V_2$$
$$\gamma_{G,j} \cdot G_{k,j} + \gamma_P \cdot P + \gamma_{V3} \cdot V_3$$
$$\gamma_{G,j} \cdot G_{k,j} + \gamma_P \cdot P + \gamma_{V4} \cdot V_4$$
$$\gamma_{G,j} \cdot G_{k,j} + \gamma_P \cdot P + \gamma_{N1} \cdot N_1 + \gamma_{V1} \cdot \psi_{0,V1} \cdot V_1$$
$$\gamma_{G,j} \cdot G_{k,j} + \gamma_P \cdot P + \gamma_{N1} \cdot N_1 + \gamma_{V2} \cdot \psi_{0,V2} \cdot V_2$$
$$\gamma_{G,j} \cdot G_{k,j} + \gamma_P \cdot P + \gamma_{N1} \cdot N_1 + \gamma_{V3} \cdot \psi_{0,V3} \cdot V_3$$
$$\gamma_{G,j} \cdot G_{k,j} + \gamma_P \cdot P + \gamma_{N1} \cdot N_1 + \gamma_{V4} \cdot \psi_{0,V4} \cdot V_4$$
$$\gamma_{G,j} \cdot G_{k,j} + \gamma_P \cdot P + \gamma_{V1} \cdot V_1 + \gamma_{N1} \cdot \psi_{0,N1} \cdot N_1$$
$$\gamma_{G,j} \cdot G_{k,j} + \gamma_P \cdot P + \gamma_{V2} \cdot V_2 + \gamma_{N1} \cdot \psi_{0,N1} \cdot N_1$$
$$\gamma_{G,j} \cdot G_{k,j} + \gamma_P \cdot P + \gamma_{V3} \cdot V_3 + \gamma_{N1} \cdot \psi_{0,N1} \cdot N_1$$
$$\gamma_{G,j} \cdot G_{k,j} + \gamma_P \cdot P + \gamma_{V4} \cdot V_4 + \gamma_{N1} \cdot \psi_{0,N1} \cdot N_1$$

To carry out all calculations, including all possible combinations we have taken the equation was worse and more unfavorable for our metal frame. Noting the values of the loads we have seen that the most crucial is the overload of snow due to its high value. So, we have taken N_1 as the main action. This action will be accompanied with the overload of use (Q_1) and the wind. We must choose a wind because the wind may blow in many directions at once. We have taken the wind V_2 (or V_4) because they are directions in which the area affected by wind is larger than in V_1 or V_3 and therefore, impact the behavior of our construktion.

So, this is our equation of charges with which we begin to make our calculations:

$$\gamma_{G,j} \cdot G_{k,j} + \gamma_P \cdot P + \gamma_{N1} \cdot N_1 + \gamma_{V2} \cdot \psi_{0,V2} \cdot V_2$$

If we apply the coefficients found in the sections of loads get:

$$\boxed{1{,}35 \cdot G_{k,j} + 1{,}50 \cdot N_1 + 1{,}50 \cdot 0{,}6 \cdot V_2}$$

These values we have determined the following tables:

Table A1.1 - Recommended values of ψ factors for buildings

Action	ψ_0	ψ_1	ψ_2
Imposed loads in buildings, category (see EN 1991-1-1)			
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	0,9	0,8
Category F : traffic area, vehicle weight ≤ 30kN	0,7	0,7	0,6
Category G : traffic area, 30kN < vehicle weight ≤ 160kN	0,7	0,5	0,3
Category H : roofs	0	0	0
Snow loads on buildings (see EN 1991-1-3)*			
Finland, Iceland, Norway, Sweden	0,70	0,50	0,20
Remainder of CEN Member States, for sites located at altitude H > 1000 m a.s.l.	0,70	0,50	0,20
Remainder of CEN Member States, for sites located at altitude H ≤ 1000 m a.s.l.	0,50	0,20	0
Wind loads on buildings (see EN 1991-1-4)	0,6	0,2	0
Temperature (non-fire) in buildings (see EN 1991-1-5)	0,6	0,5	0
NOTE The ψ values may be set by the National annex. * For countries not mentioned below, see relevant local conditions.			

Table A1.2(B) - Design values of actions (STR/GEO) (Set B)

Persistent and transient design situations	Permanent actions		Leading variable action	Accompanying variable actions (*)	
	Unfavourable	Favourable		Main (if any)	Others
(Eq. 6.10)	$\gamma_{Gj,sup}G_{k,j,sup}$	$\gamma_{Gj,inf}G_{k,j,inf}$	$\gamma_{Q,1}Q_{k,1}$		$\gamma_{Q,i}\psi_{0,i}Q_{k,i}$

Persistent and transient design situations	Permanent actions		Leading variable action (*)	Accompanying variable actions (*)	
	Unfavourable	Favourable	Action	Main	Others
(Eq. 6.10a)	$\gamma_{Gj,sup}G_{k,j,sup}$	$\gamma_{Gj,inf}G_{k,j,inf}$		$\gamma_{Q,1}\psi_{0,1}Q_{k,1}$	$\gamma_{Q,i}\psi_{0,i}Q_{k,i}$
(Eq. 6.10b)	$\xi\gamma_{Gj,sup}G_{k,j,sup}$	$\gamma_{Gj,inf}G_{k,j,inf}$	$\gamma_{Q,1}Q_{k,1}$		$\gamma_{Q,i}\psi_{0,i}Q_{k,i}$

(*) Variable actions are those considered in Table A1.1

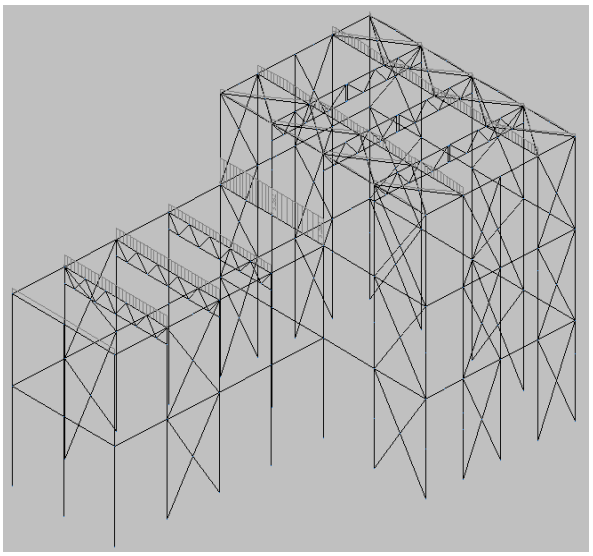
NOTE 1 The choice between 6.10, or 6.10a and 6.10b will be in the National annex. In case of 6.10a and 6.10b, the National annex may in addition modify 6.10a to include permanent actions only.

NOTE 2 The γ and ξ values may be set by the National annex. The following values for γ and ξ are recommended when using expressions 6.10, or 6.10a and 6.10b.
$\gamma_{Gj,sup} = 1,35$
$\gamma_{Gj,inf} = 1,00$
$\gamma_{Q,1} = 1,50$ where unfavourable (0 where favourable)
$\gamma_{Q,i} = 1,50$ where unfavourable (0 where favourable)
$\xi = 0,85$ (so that $\xi\gamma_{Gj,sup} = 0,85 \times 1,35 \cong 1,15$).
See also EN 1991 to EN 1999 for γ values to be used for imposed deformations.

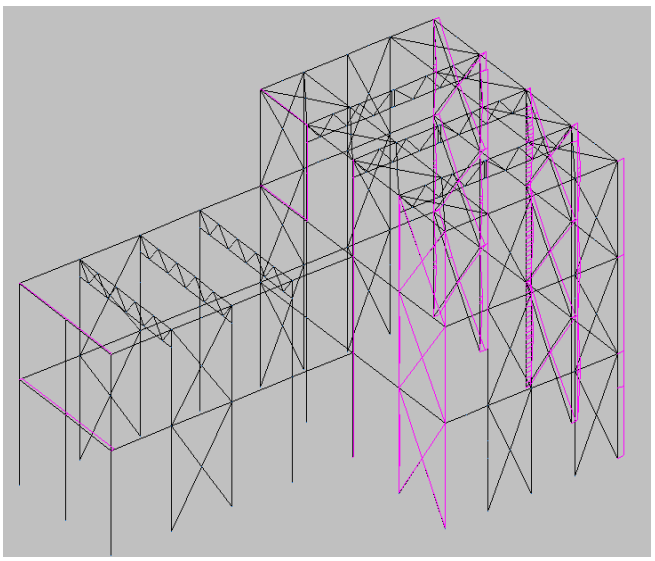
NOTE 3 The characteristic values of all permanent actions from one source are multiplied by $\gamma_{G,sup}$ if the total resulting action effect is unfavourable and $\gamma_{G,inf}$ if the total resulting action effect is favourable. For example, all actions originating from the self weight of the structure may be considered as coming from one source ; this also applies if different materials are involved.

NOTE 4 For particular verifications, the values for γ_G and γ_Q may be subdivided into γ_g and γ_q and the model uncertainty factor γ_{Sd}. A value of γ_{Sd} in the range 1,05 to 1,15 can be used in most common cases and can be modified in the National annex.

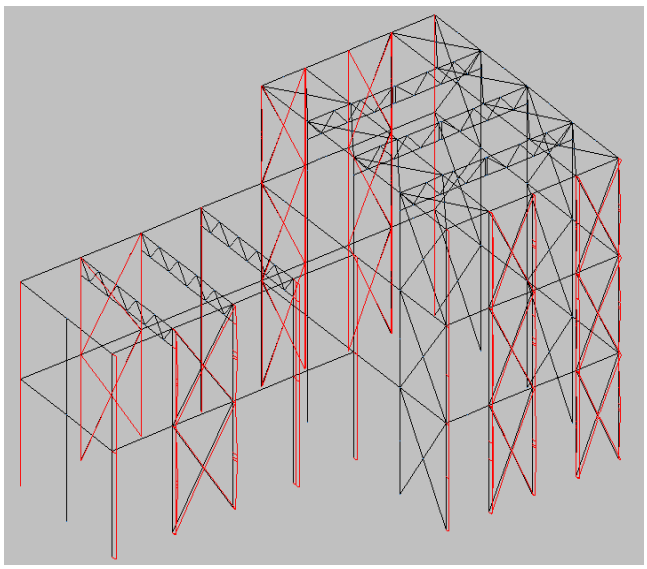
N1



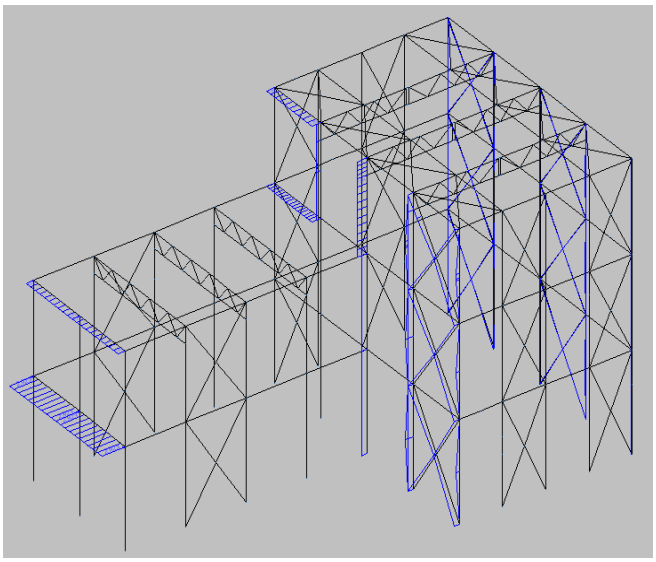
V1



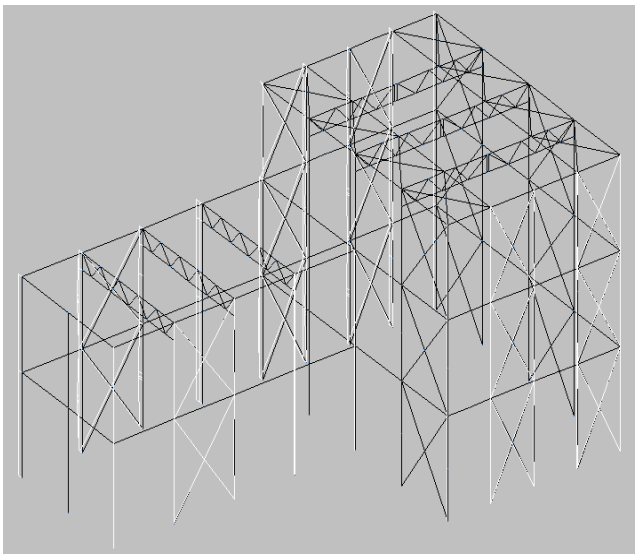
V2



V3



V4



4.3 Design Structure

4.3.1 Materials

When you want to design this factory, one of the first issues to be solved is the materials to be used. In the case of this thesis, we have chosen a steel structure.

The steel structure presents some advantages and some disadvantage that we have to study if we want to take the best decision. The most important advantages of this material are:

- *High Strength:* The high strength of steel per unit weight allow to make structures relatively light, which can be important in building bridges, large buildings or when the ground is soft consistency.
- *Uniformity*: The properties of steel do not alter with time or vary with localization in structural elements.
- *Elasticity*: The elastic steel material that is more similar to behave linearly elastic (Hooke's Law).
- *Dimensional precision rolled*: The namin profiles have been created with a standards that are manufactured for allow very precise geometric properties of the section.
- *Ductility:* The steel can withstand large deformations without failure, achieving high tension efforts.
- *Tenacity:* This material has the ability to absorb large amounts of energy deformation (elastic and inelastic).
- *Ease of union with other members*: The steel profiles can easily connect using rivets, screws or welding to other profiles.
- *Fast assembly:* The speed of construction steel is much higher other materials.
- *Availability of sections and sizes*: Today has a lot profiles that optimize its use.
- *Cost recovery*: The steel structures are cost recovery in the worst cases as scrap steel.
- *Recyclable*: Steel is 100% recyclable material. It is biodegradable and it doesn't pollute.
- *Allows easy expansion to steel*: It is easy to change a steel structure by adding components.
- *Can be prefabricated structures*: The structures of this material can workshop carried out in reducing as far as possible site work. In this way, you get more accuracy.

Also, the steel have some disadvantage like:

- *Corrosion*: The steel exposed to the elements undergoes corrosion, so the structures must always be covered with enamel (primary corrosive), except for special steels such as stainless steel.

- *Heat, fire*: In the case of fire, heat spreads quickly to the structures by reducing its resistance up to temperatures where the steel behaves plastically. It should have an insulating coating protection from heat and fire (retardant), as mortar, asbestos, etc...
- *Fatigue*: The resistance of the steel can decrease when subjected to a large number investment load or frequent changes of magnitude efforts to tension (pulsating loads and alternatives).

Common characteristics of steels:

-Modulus of Elasticity (E) 210 000 N/mm^2

-Modulus of Rigidity (G) 81 000 N/mm^2

-Poisson coefficient (v) 0.3

-Coefficient of thermal expansion (α) $1{,}2 \cdot 10^{-5}$ $(^{o}C)^{-1}$

-Density (ρ) 7850 kg/m^3

4.3.2 Structural Types

The metal structure calculation is the most delicate engineering of an industrial warehouse in order to give the greatest possible security at the lowest price. There are metal structures or frames of variable geometry. There are different types, always chosen according to the type of surface ship applications: tubular, prestressed, lattice, profile variable section soul full profile, etc...

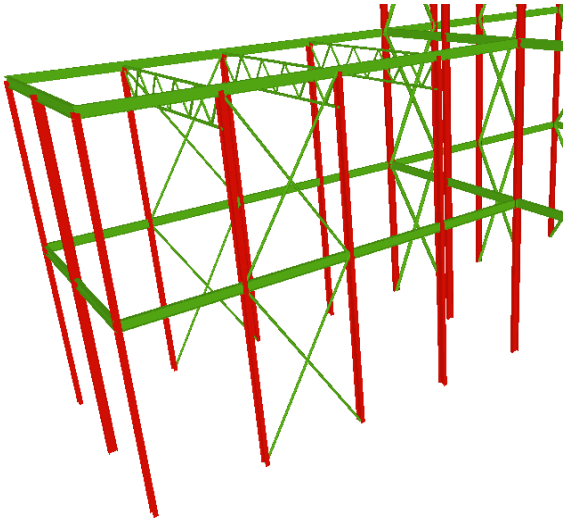
Therefore, we decided to use the porch with lattice frame type as the ship because, firstly, is the most economical solution and, secondly, is a structural typology diaphanous and light, not to be a solid structure, and besides, its elements work primarily in tension and compression.

The gantries are hiperestàtics, so that the pillars are built into the base and therefore the forces and moments are transmitted to the foundations. This prevents too many nodes are required.

4.3.3 Predimensionat jealousy

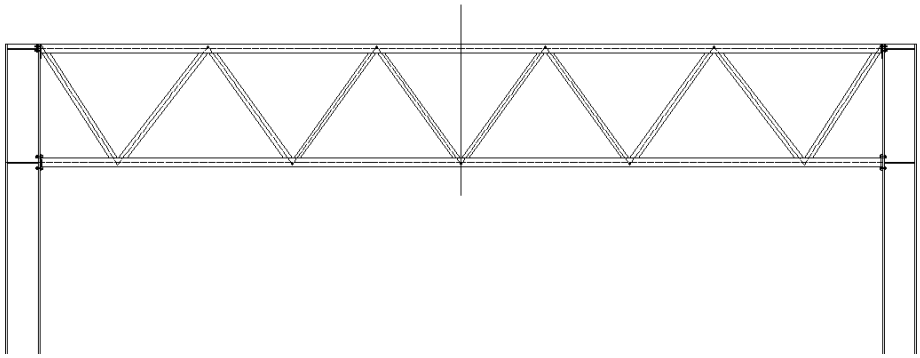
Secondly we must choose the type used for structural design of the ship industry. The breach is the most widely used structural system for single storey buildings, as is the case of industrial buildings, due to the economic impact that is its ease of assembly. The portico is composed of a lintel supported on two pillars. If you are trying, as seen above, there is a rather large area building.

Shortest building:



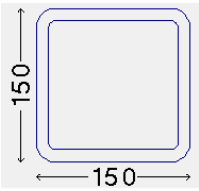
For the structural design of this building we have started from 5 separate porches the distance between two consecutive spoils is 7 meters. We have separated the two storey building of 12 meters (down) and 11 meters (up). The pillars of this structure will be formed by a HEB 600 profile.

On the deck beams have also used the same profile (HEB 600) but just in the case that the distance between pillars is not big. We are not interested in put a large beams as its own weight can significantly affect the pillars below. Also, we have need a king of beams or pillar between the pilars that are more separate. We have choosen a kind of tubular profile becouse we don't want that the pillar need to suport so much stress and the roof not so heavy.

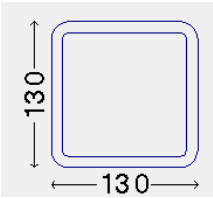


The profile that we have used is formed with three diferents types of square tube all of them have a thickness of 10 mm: the horizontal beams measures 150x150 mm, the exterior diagonal beams have a side of 130x130mm and in the middle are quites maller than the others, 100x100mm. This reason is because in the middle of the portic we can see that the beams works less than the others so we don't need to use so big profile.

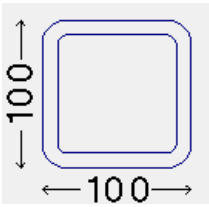
These follow pictures you can see a sketch of the differents kind of profile and their properties:



Área (cm²)	$I_y^{(1)}$ (cm4)	$I_z^{(1)}$ (cm4)	$I_t^{(2)}$ (cm4)
63.71	1974.59	1974.59	3255.50



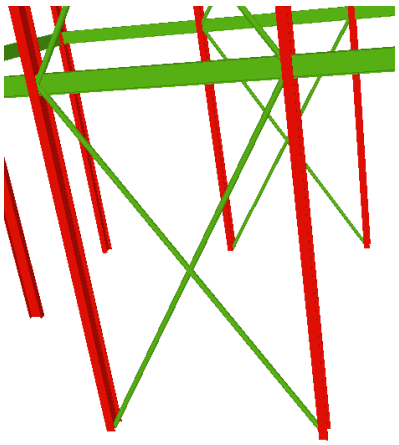
Área (cm²)	$I_y^{(1)}$ (cm4)	$I_z^{(1)}$ (cm4)	$I_t^{(2)}$ (cm4)
46.07	1077.92	1077.92	1783.73

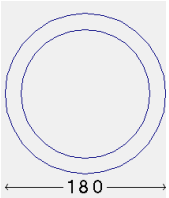


Área (cm²)	$I_y^{(1)}$ (cm4)	$I_z^{(1)}$ (cm4)	$I_t^{(2)}$ (cm4)
34.07	443.52	443.52	757.43

Also, to ensure the stability of the structure compared to efforts in the direction perpendicular to the gantry, efforts produced by the wind, they placed crosses of St. Andrew. These items are considered to work only in tension is the same or only work when the wind (horizontal or action) causes a destabilization of the structure. These are in the facades, are formed by a tubular profile(diameter of

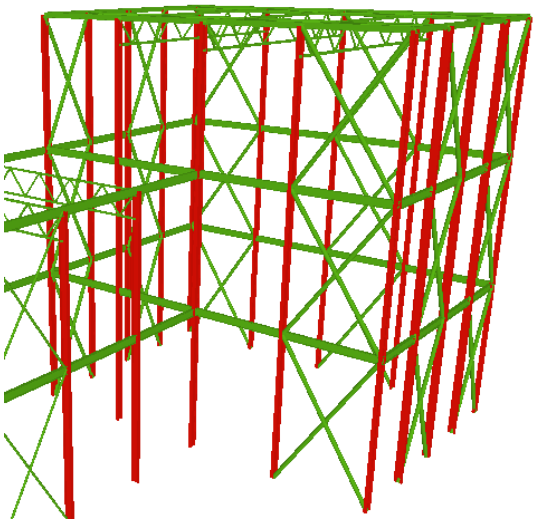
180 mm and 18 mm thick). We've been changing the placement of the crosses for the location of a possible door and to improve stability.





Área (cm²)	$I_y^{(1)}$ (cm4)	$I_z^{(1)}$ (cm4)	$I_t^{(2)}$ (cm4)
91.61	3042.33	3042.33	6084.66

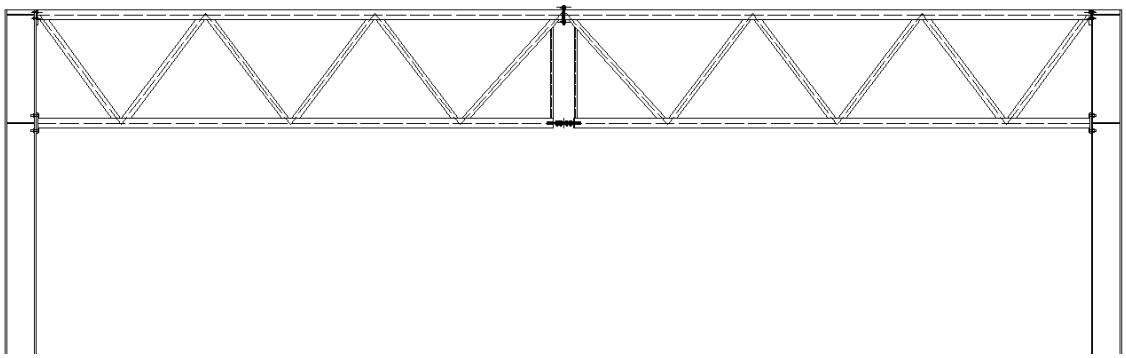
Tallest building



The tall building is 34 meters and consists of three floors: the ground floor of 12 meters and the other from 11 meters. For the pillars has been used HEB profile section 600. This pilars are separated 5,375 m(short side) and 7,875 m (long side).

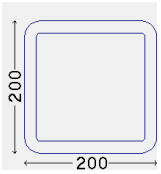
As in the previous case, we used the bigger beams (HEB 600) at short distances between pillars. On the other hand, larger distances (21 meters) have resorted to the previous solution using hollow so that the pillars do not suffer much.

The solution used was as follows, but in this case with larger profiles than the small building:

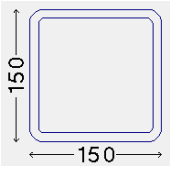


We have designed this portic in two parts, because if we didn't do it, we would have so large and heavy beams to transportation. This joints that we have created will be showed later.

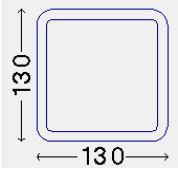
The beams profile consists of the association of three different a square tubular profile: the horitzontal beams have 200x200 mm of side and a stickness of 12 mm; the exterior diagonals have 140x140 mm of side and 10 mm of stickness and the internal diagonals 120x120 mm and the same thickness.



Área (cm²)	$I_y^{(1)}$ (cm4)	$I_z^{(1)}$ (cm4)	$I_t^{(2)}$ (cm4)
87.71	5082.00	5082.00	8172.59

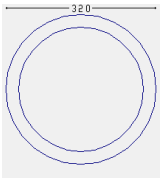


Área (cm²)	$I_y^{(1)}$ (cm4)	$I_z^{(1)}$ (cm4)	$I_t^{(2)}$ (cm4)
54.07	1729.35	1729.35	2822.77



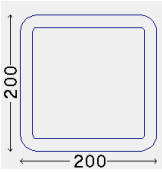
Área (cm²)	$I_y^{(1)}$ (cm4)	$I_z^{(1)}$ (cm4)	$I_t^{(2)}$ (cm4)
46.07	1077.92	1077.92	1783.73

To improve the stability and strength of our structure we have also used crosses of San Andres. We have used crosses to support our structure. This crosses have been formed with a tubular profile(diameter of 350 mm and 30 mm thick). It was not necessary to put in all the floors, just in the first and the second one. The colums that don't have crosses, have been replaced with two beams of HEB 600 of simple profile.



Área (cm²)	$I_y^{(1)}$ (cm4)	$I_z^{(1)}$ (cm4)	$I_t^{(2)}$ (cm4)
301.59	38943.18	38943.18	77886.37

Also, in the roof, we have put along the longest side some crosses to do the structure more rigid and stable against the wind acction. To place the crosses of St. Andrew in the ceiling beams are made connecting the firstframe with the second, this with the third and follow this structure thresholds at the height of the pillars are not ends of the first and last gate. This crosses and beams have a empty profile because we don't want the pillars to support so much weight. It is a square tubular profile of 200 mm of sife and 18 mm of thickness.



Área (cm²)	$I_y^{(1)}$ (cm4)	$I_z^{(1)}$ (cm4)	$I_t^{(2)}$ (cm4)
312.00	68130.00	19710.00	1501.00

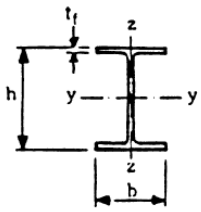
4.3.4 Buckling lenght

The calculation of the effective buckling length of structural elements in steel any metallic structure is of great importance. In fact, the definition of the parameter β (buckling ratio) is usually one of the issues involved in the calculation since the use of values higher than really necessary leads, in many cases, an oversizing of the profiles. This is because checking the stability of the bars is part of the verification States Last limits (ELU) and, therefore, considered too great lengths buckling means having to raise the profiles or add new to ensure that no buckling occurs.

So we need to know the lengths of each bar buckling of the structure to verify that they do not subsequently fail to buckling. Therefore, before setting the criteria for verification of the ELU, these lengths are calculated.

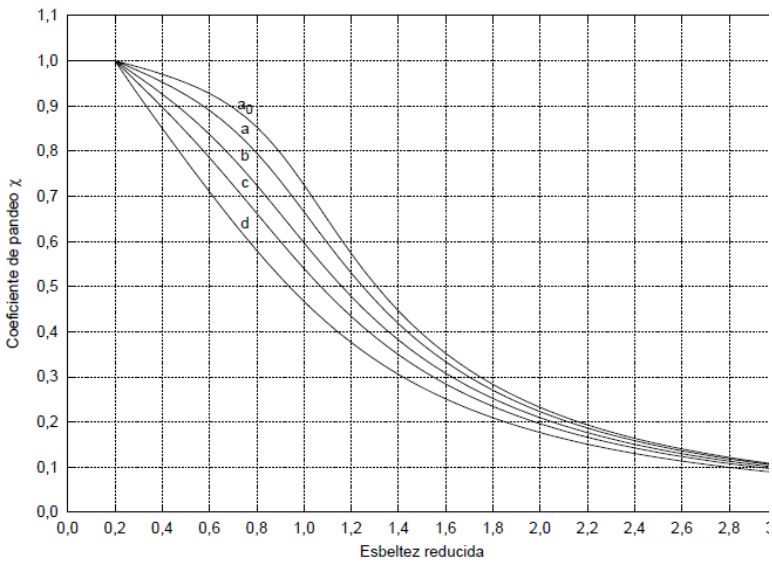
For each type of bar you must define the length of buckling in the strong axis of the section, the length of buckling in the weak axis of the section and length of lateral buckling. A bar subjected to compression when has excess load can fail by buckling. In contrast, when a bar is subjected to bending, part of the section is subjected to traction and the other in compression. Because of this compression, this bar can fail for lateral buckling.

In our case we have used profiles are of the following types:

Cross section	Limits	Buckling about axis	Buckling curve
Rolled I-sections 	h/b > 1,2:		
	$t_f \leq 40$ mm	y – y z – z	a b
	40 mm < $t_f \leq 100$ mm	y – y z – z	b c
	h/b ≤ 1,2:		
	$t_f \leq 100$ mm	y – y z – z	a b
	$t_f > 100$ mm	y – y z – z	d d

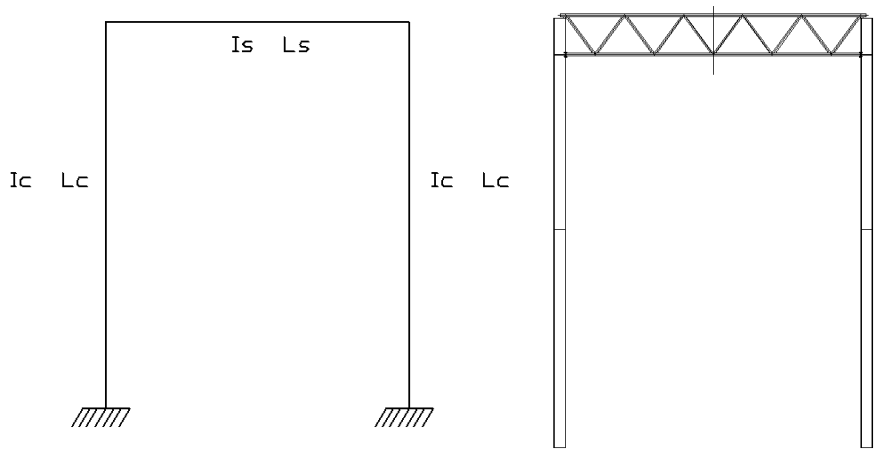
Hollow sections 	hot rolled	any	a
	cold formed — using f_{yb}[a]	any	b
	cold formed — using f_{ya}[a]	any	c

In the following table we can easily obtain the value of buckling coefficient according to the beam profile:



Bucking curve	a	b	c	d
Imperfection factor α	0,21	0,34	0,49	0,76

Sometimes, is difficult to calcúlate the buckling lenght because the geometry is quite complicated.In our case, if we want to find the buckling lenght of our pillars that are formed portic we have had to do some variations and calculations.



We have some relations and equations to know which is the buckling length of the pillar:

$$n = \frac{Is \cdot Lc}{Ls \cdot Ic}$$

$$\mu = \sqrt{\frac{n + 0{,}56}{n + 0{,}14}}$$

Where:

$$Ic = 171000 \cdot 10^4 mm^4 (HEB\ 600)$$
$$Lc = 11 + 12 = 23 m = 23000\ mm$$
$$Ls = 15,75 m = 15750\ mm$$

To calculate Is we have to reduce the inertia of the bottom bar of the portic to the axis of the top bar.

$$Is = I_{150\ x150} + I_{red} = 1974,59 \cdot 10^4 mm^4 + 2(10 mm \cdot 150\ mm \cdot (2000\ mm)^2 + 10 mm \cdot 130\ mm \cdot (2000\ mm - 70 mm)^2$$
$$+ 10 mm \cdot 130\ mm \cdot (2000\ mm + 70 mm)^2 = 2243249.34 \cdot 10^4 mm^4$$

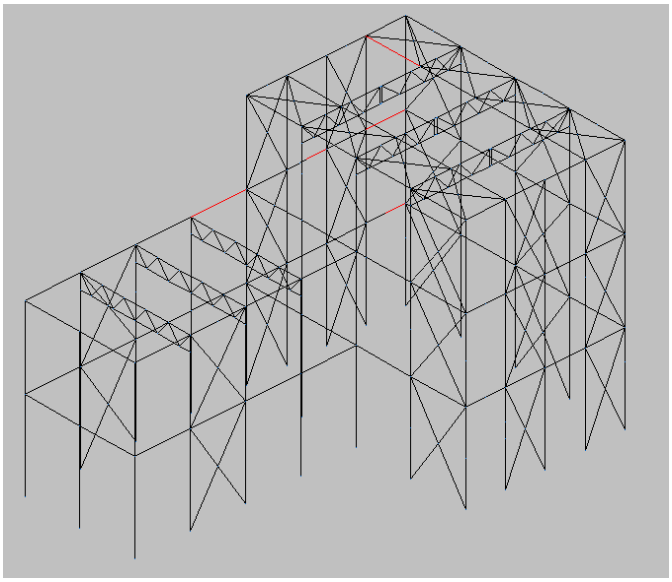
So:

$$\boxed{\begin{array}{l} n = 19,157 \\ \mu = 1,02 \end{array}}$$

Now, we know that the buckling lenght of this pillar is 1,02 times the real lenght and we can check and do all the calculations that we will do later.

4.3.5 Jealousy study

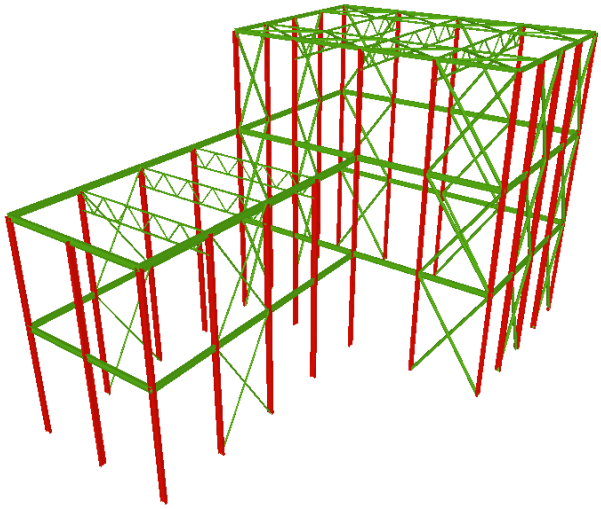
To study the structure we have used different methods. First, using the software sanity check fails that the structure has endured all load combinations. During the election of the sections and structural design, the beams have appeared red because for whatever reason did not support the requirements.



In the final model that we have chosen, It appears three beams that are red, but the use of resistence is arround 100% (101.15%; 103.87%, 108,53% and

108.33%) and considering that during the calculation is made by using various ratios mostly, we could take the value as correct.

Making Changes in profile and playing with the structure we arrive at a configuration that supports the efforts and all the beams operating below full capacity.

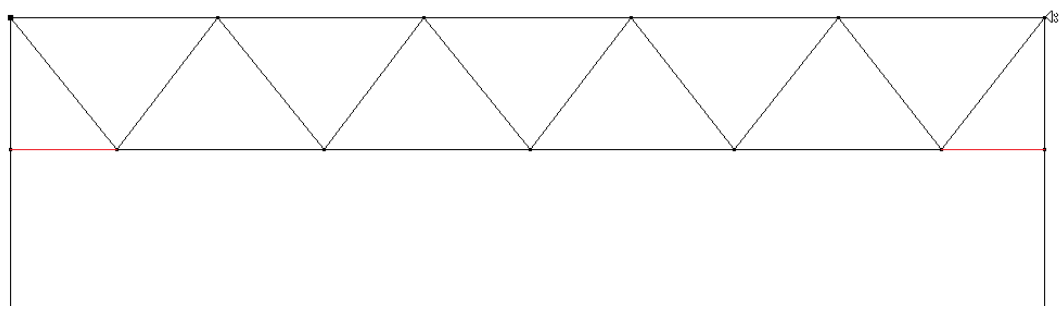


4.4 Results of analysis with differents softwares

4.4.1 PowerFrame

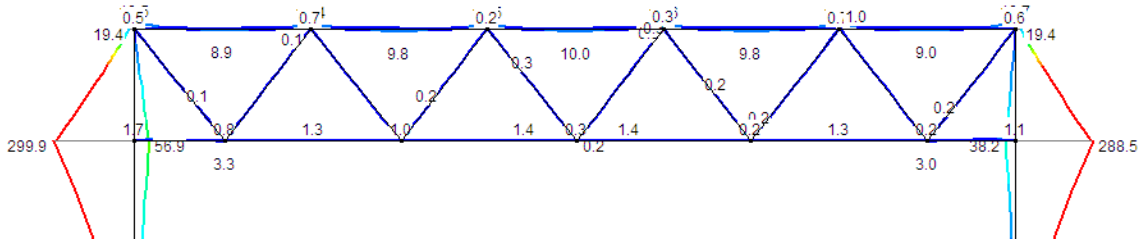
To make a concrete study of the porch (high and low) we used a computer program that works in 2D. This helped us decide to small changes in our design because it allows us to compare the moment diagrams and cutting with ease, as well, an outline of the possible deformation of the porch.

We started with the small porch and checked the time and strain variation between operating or not a biga (red in the drawing).

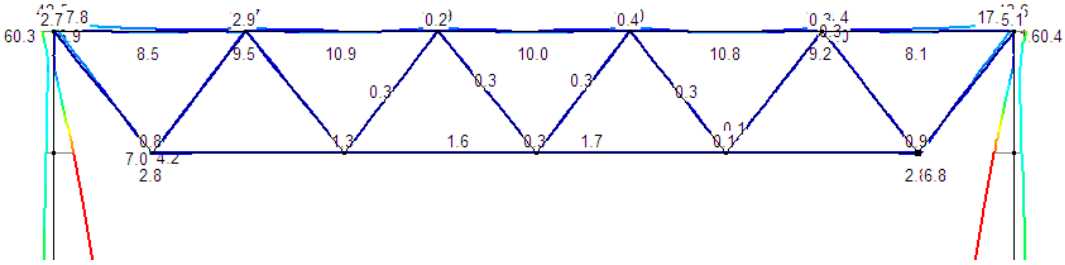


As we work in 2D but really this porch is in a three-dimensional network, we configured blocking links to such degrees of freedom to reach a result that can compare and look at the real example.

With this beam, the moments are(ultimate limit state)[kN·m]:

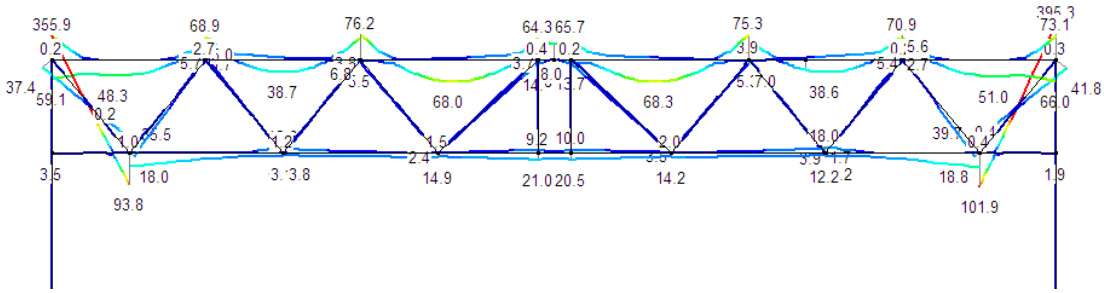


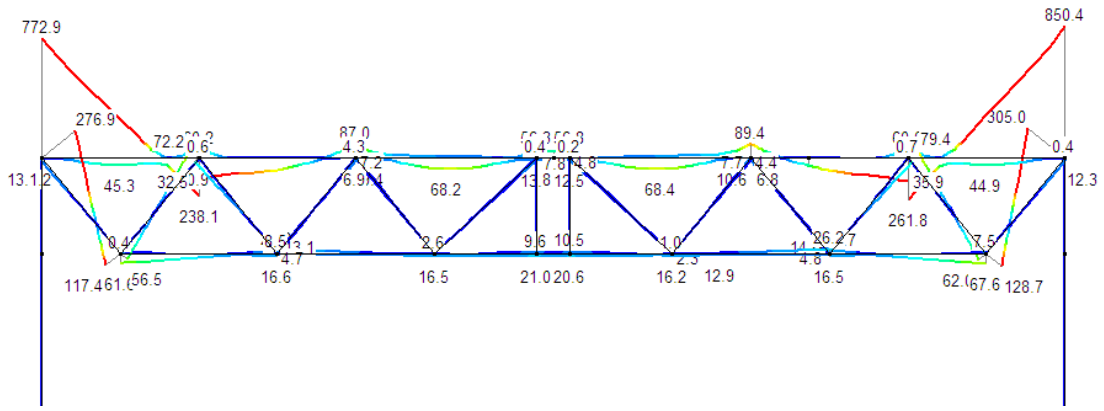
Other hand, without this beam(ultimate limit state)[kN•m]:



The big difference between the two cases is that in the second pillar is longer and so that it directly affects the buckling length and that there is more distance between the two junctions. Moreover, the moments in the horizontal beams are greater when we have the biga. When we put this beam, we have more rigidity and stability without it and our structure undergoes minor deformations in the case without it. Using the software used is easy to verify that the maximum deflection occurs at the midpoint and if we have not placed the bar at the ends.

We have done the same with the taller one[kN•m]:



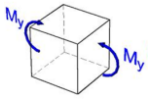


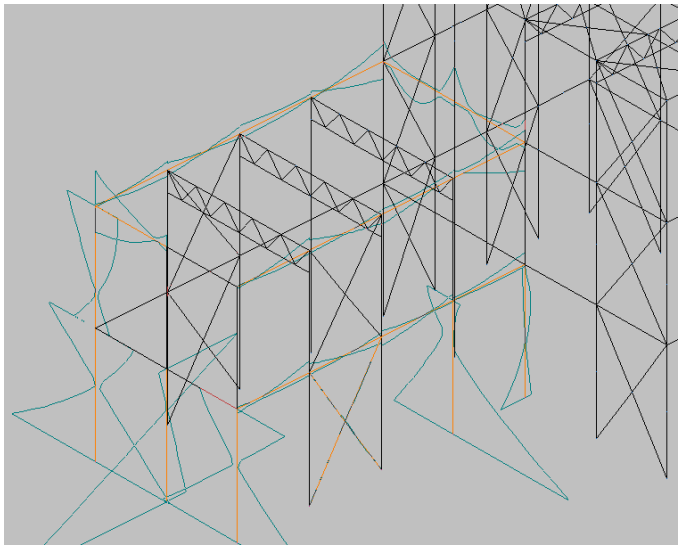
In this case the difference is greater than in the previous case as can be seen in the diagrams of moments. It is also logical because the distance between the two porticos is higher.

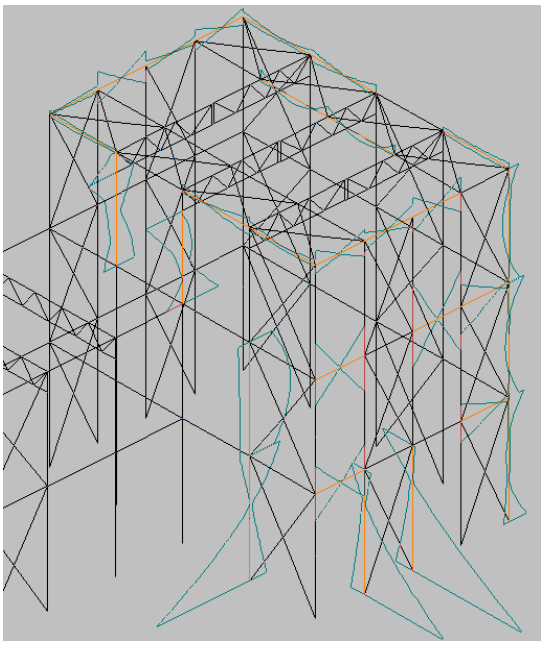
4.4.2 CYPE

The graphs shown below give us information about the range of values of the efforts to implement all the combinations described in previous sections.

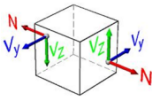
4.4.2.1 Bending moments(My)

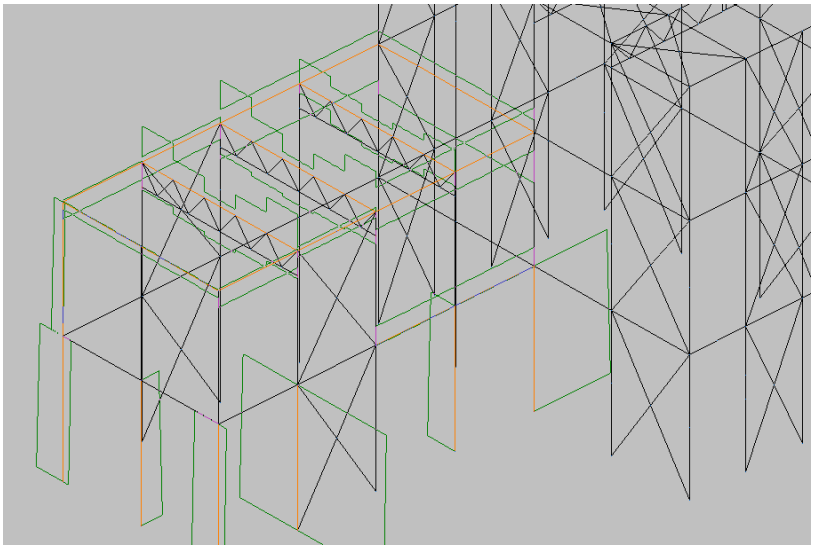


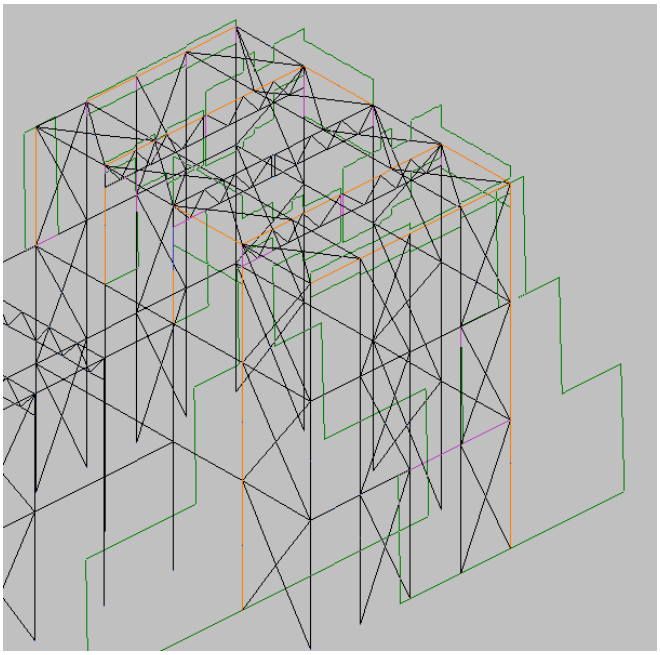




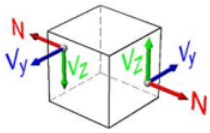
4.4.2.2 Normal stress(N)

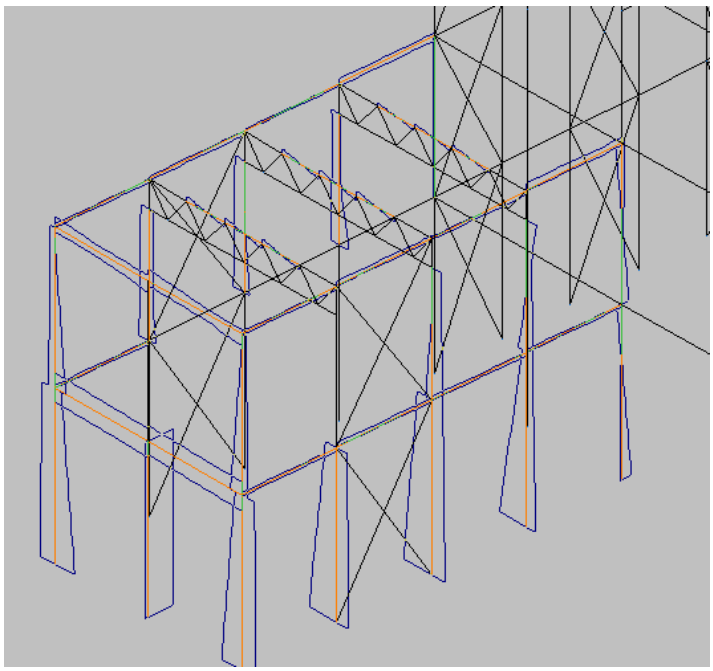


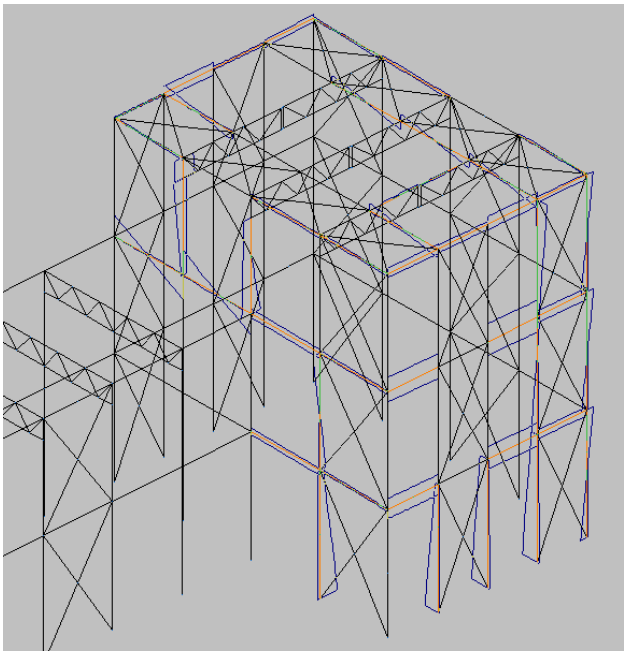




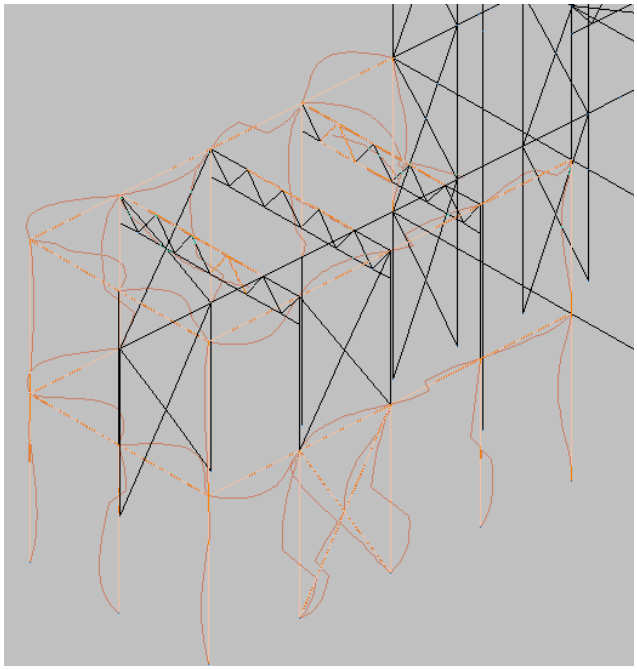
4.4.2.3 Shear stress(V_z)

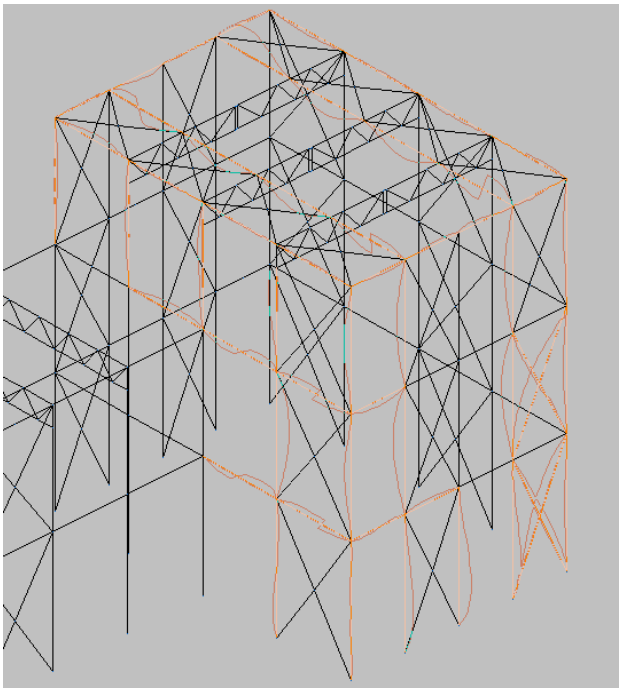






4.4.2.4 Deformed





4.5 Results of manual checks

To compare the results with those of CYPE manually, you should see Annexe A, which have been made each and every one of the tests of resistance and stability according to the instructions of the part EN 1993-1-1 in the Eurocode 3.

4.5.1 Strength of beams and metal pillars

To check whether the pillars and beams of our structure work in a healthy state, has selected a series of bars and beams in building a randomly distributed. Here are three summary tables that show the checks were carried out at Annexe A separated by beams, columns and cross bars.

Columns

We have choosen five columns of our structure design situated in differents parts. For each one, we have done some checkings to verificate if these columns are working in a good conditions. We have checked:

-Tensile

-Compressive strength

-Buckling resistance

-Flexural axis Y

-Flexural axis Z

-Shear strength Z

-Shear buckling of the soul

-Shear strength Y

-Resistance to bending moment Y and Z combined shear

- Resistance to bending moment Z and Y combined shear

- Resistance to bending and combined axial

-Torsional Strength

The following table summarizes the calculations made that are attached at Annexe. The numbers signify the working capacity in the backbone as the time or effort. (0 does not work, 100 working at maximum level):

η(%)	Checkings (Eurocode 3, EN 1993-1-1; 2005)							
	N_t	N_c	M_Y	M_Z	V_z	V_y	NM_YM_Z	M_t
1	11.0	77.0	59.6	2.7	11.4	0.1	84.6	1.5
2	0	12.4	88.8	26.8	13.0	0.6	93.6	0.3
3	0	7.8	33.9	4.9	5.3	0.1	36.7	0.9
4	7.1	56.6	76	1.2	15.5	0.1	83.7	1.4
5	1.1	8.4	6.3	23.8	1.3	1.6	23.8	0.3

Beams

To study the beams have followed the same procedure as abutments. We have selected 5 beams of different areas of the structure and verified if it is working load conditions. Apart from the above tests, in this case we have also studied

the shear resistance Z and torque together. All calculations have been added in the Annex A.

Below is also a summary table of the beams studied:

η(%)	Checkings (Eurocode 3, EN 1993-1-1; 2005)								
	N_t	N_c	M_Y	M_Z	V_z	V_y	NM_YM_Z	M_t	M_tV_Z
1	0.1	0.8	17.7	93.3	5.3	4.7	97.6	0.6	5.3
2	0.7	3.1	21.4	70.0	8.5	3.0	73.7	0.5	0.7
3	0.0	1.3	15.6	77.9	7.3	1.5	78.8	3.0	0.6
4	4.9	11.9	64.2	10.9	8.6	0.4	75.5	0.5	7.1
5	0.4	2.5	19.8	75.6	9.1	3.2	80.2	1.4	4.0

Crosses of Saint Andrew

Also, we have checked some posts(5) that are making a cross for make the structure more rigid. Each cross is formed with four bars that are joined by a union to be studied in the next sections.

It has created a table with the results of the tests:

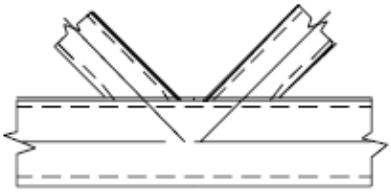
η(%)	Checkings (Eurocode 3, EN 1993-1-1; 2005)							
	N_t	N_c	M_Y	M_Z	V_z	V_y	NM_YM_Z	M_t
1	9.5	36.5	2.2	71.2	0.2	3.8	75.8	6.1
2	5.5	7.7	3.9	36.0	0.3	2.2	37.5	10.1
3	4.7	31.6	2.8	39.9	0.2	3.2	41.6	3.4
4	6.3	5.9	47.0	16.7	4.4	1.1	62.1	6.9
5	1.8	4.2	2.0	25.4	0.3	2.5	27.2	10.8

4.5.2 Checking the connections and joints

To verify that all connections are working in good condition without compromising the structure, has identified a number of unions of different types and located in different parts of the structure and have been checked one by one using the Eurocode 3.

Type 1

This connection is formed by the union of the joint of three beams: the main one is horitzontal and the others diagonal. We have solved these joints with weldings.



This union can fail for various reasons which are shown below:

a) Plastic failure of the chord face or the chord cross section.

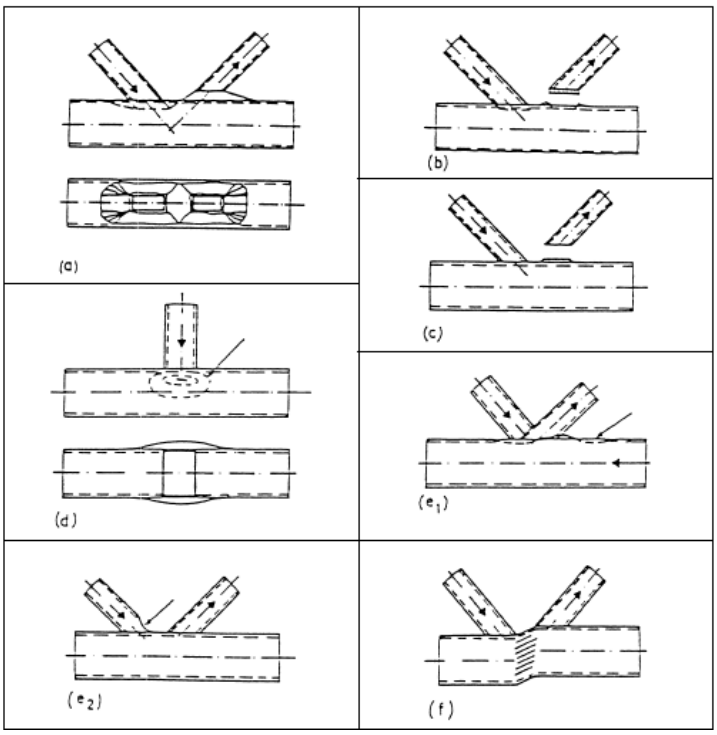
b) Crack initiation leading to rupture of the bracings from the chord (punching shear).

c) Cracking in the welds or in the bracings (effective width).

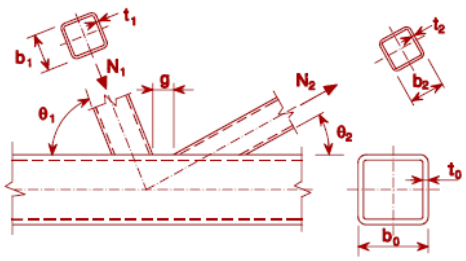
d) Chord wall bearing or local buckling under the compression bracing.

e) Local buckling in the compressive areas of the members.

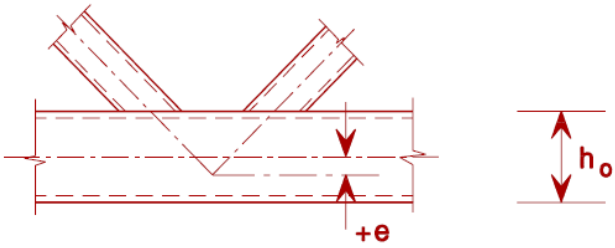
f) Shear failure of the chord.



To check it, to assimilate the tubular hollow squares as shown in the figure below:

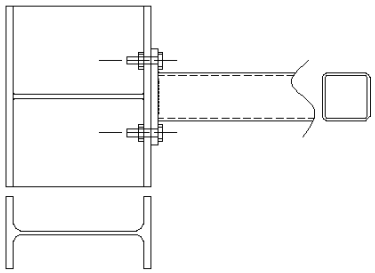
K and N gap joints	Chord face failure $\beta \leq 1{,}0$
	$N_{i,Rd} = \frac{8{,}9\gamma^{0,5} k_n f_{y0} t_0^2}{\sin\theta_i}\left(\frac{b_1+b_2}{2b_0}\right) / \gamma_{M5}$

In our case, all this kind of joints have a positive eccentricity:



Type 2

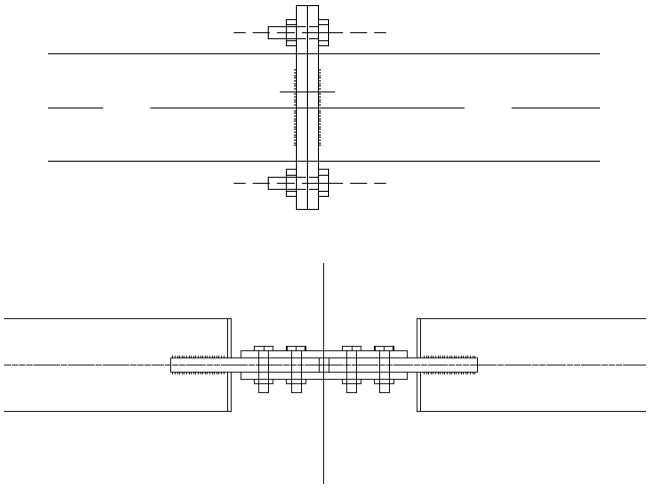
This kind of unions is usefull between horitzontal beams and pillars. In our case we have joined square hollow profiles with HEB prfiles. Is quite difficult combination to join but with some bolts and welding is possible to make a safe joint:



the problem of using a hollow tubular profile is not easily attach to the column, so, heplaced a thin plate welded on the end which is attached to the pillar by screws. This method helps us create an atmosphere inside the tubular profile without humidity or other factors that may affect the live of the beam.

Type 3

These unions have been created since the length of the beams was very big and wedecided to create two boards to make it easier to transport, manufacture and assemble the structure.



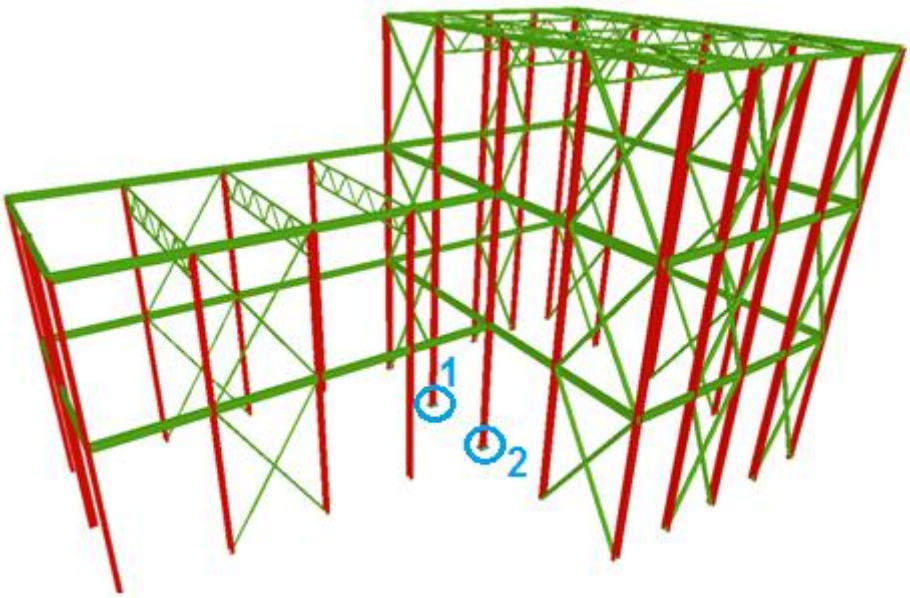
Sometimes, these kind of unions have some bolts that never work to traction so just make the function to join the board. In this case we used a minimum metric M20.

4.6 Foundations. Basic and pillar joined.

The design of the union of the base of the pillar was performed using the program CYPE, but checks have also been resistance from the union of the basic components as described in section 6.2.6 of the EN 1993-1-8 the Eurocode3.

The union was performed using a base plate welded to the pillar attached to the shoe using anchor bolts. Screws have been used quite ready next to the center of the union giving you some ability to rotate.

Now, we will study the two simplest shoes. This shoes don't have joint and it is only one kind of profile (HEM 600). The pillars that are supported with this shoes are separating the tall and the short building. You can see this in the follow picture:



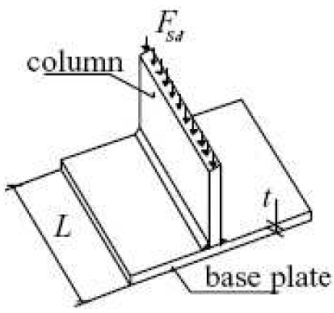
4.6.1 Motherboard and concrete under compression

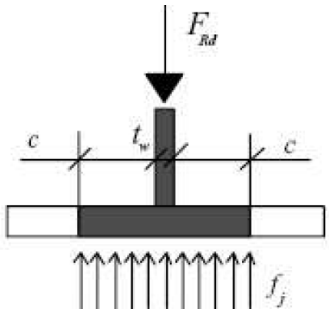
To evaluate the resistance of the base plate and concrete in compression must be account:

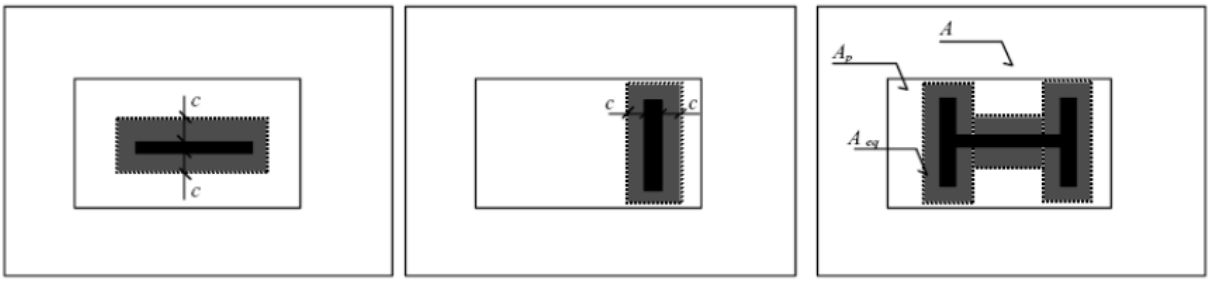
- The effect of confinement of concrete: strength than their own characteristic resistance to compression due to movement restrictions.
- The elastic behavior of the motherboard (flexible) and non-uniform distribution of stresses on this approach, according to the model

calculation, an effective section (rigid) less than the dimensions of the plate (defined by the footprint) with a uniform stress distribution.

The model calculation can be represented by the two following figures:







Where:

c: trace

A: foundation area

A_p: motherboard area(flexible)

$A_{eq} = A_{ef}$: equivalent or effective area of the motherboard (rigid)

The maximum calculation of resistance to compression of the base of the pillar is:

$$F_{c,rd}=R_z$$

For design our brake shoes, we took the pillar that the compressive strength of the base of the pillar:

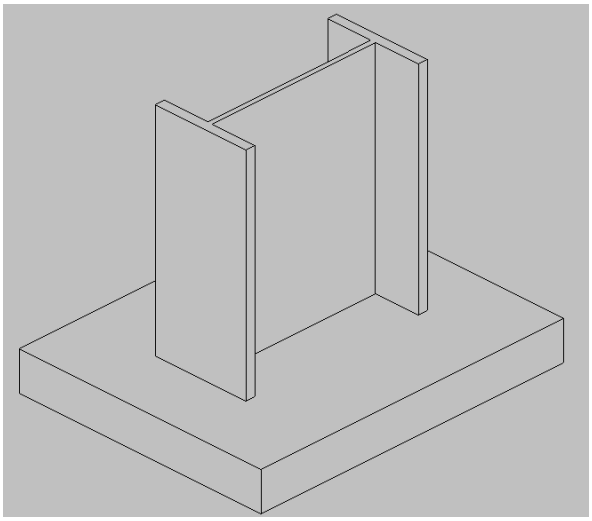
	1	2
Max compressive strength R_z(kN)	865,11	714,17

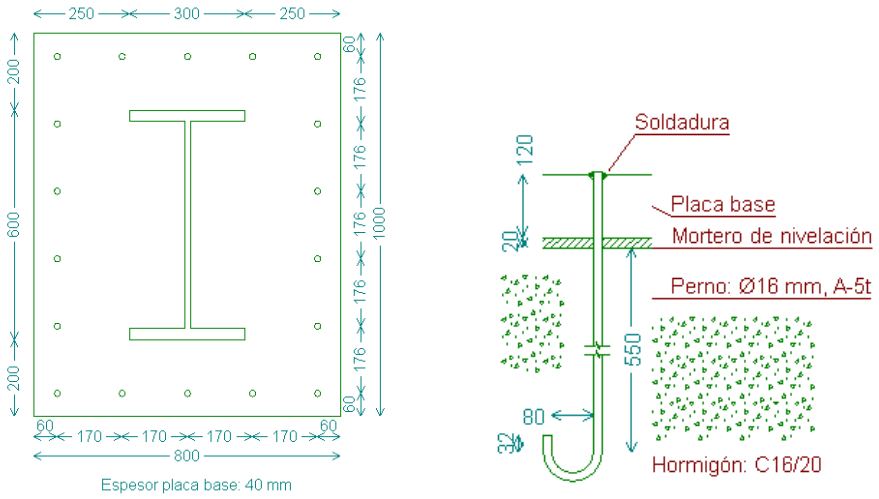
4.6.2 Selected baseplates

To set the pillars on the ground we have used concrete footings. These shoes are designed to withstand all efforts to absorb the pillar forward. It has placed our profile HEM 600 soldier in the center of the brake shoe. The follow solution is not the only one, it is one possibility as correct as the others. For this part we have used the Eurocode 2 of concrate structures.

Brake shoe number 1:

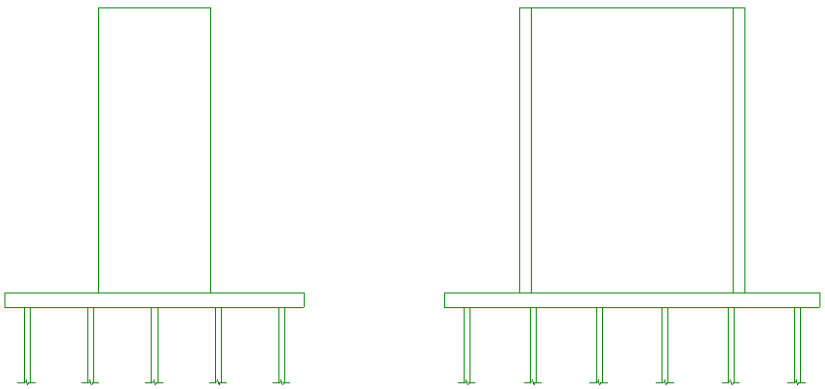
The pillar is we have to endure double profile HEM 600. In this brake shoe we have used cement for a base plate for a size of 800x1000x120 mm. We put 18 bolts(more in the long side) around the base to absorb forces with a diameter of 16 mm each. These have been manufactured with a steel S275. In this case we don't need to put stiffeners because the applied stress is not so big and it is enough with the concrete and the bolts. We caught a C20/25 concrete type for all calculations.





Brake shoe number 2:

In this brake shoe we have used cement for a base plate for a size of 800x 1000x 120 mm. We put 18 bolts(more in the long side) around the base to absorb forces with a diameter of 16 mm each. Also, we have placed 18 pins on the edges of the shoe. Bolt or stud is called a piece of metal, usually steel or iron, long, cylindrical, like a screw but larger, with a round head end and another end that is usually threaded. At this end is screwed a pin, screw or rivet, and can hold one or more pieces in a structure, usually of large volume.



4.6.3 Computer programs used

4.6.3.1 PowerFrame

PowerFrame is a complete system for calculating 2D and 3D structures of steel, concrete and wood. The ease of data entry and interpretation of results make this powerful software program. The calculation and verification is not limited to rolled, but also works with variable section profiles armed.

4.6.3.2 CYPE

CYPECAD is designed to perform the analysis and design of reinforced concrete and steel, subject to horizontal and vertical, for homes, buildings and civil engineering projects. This program performs an analysis of the matrix structure, considering a linear elastic behavior of materials. The bars are defined as linear elements. Each state charges the program generates all possible combinations according to the rules chosen by the user. From the geometry and charges are introduced, the program obtains the matrix structure and stiffness matrices of simple loading scenarios. The matrix displacement nodes of the structure obtained by inverting the stiffness matrix of frontal methods. After finding the displacement hypothesis is calculated for all combinations of charge for all states, and efforts in any section from efforts at the ends of the bars and loads applied to the bar.

4.6.3.3 AutoCad

Autodesk AutoCAD is a program for computer-aided design drawing in two and three dimensions. He is currently developed and marketed by Autodesk.

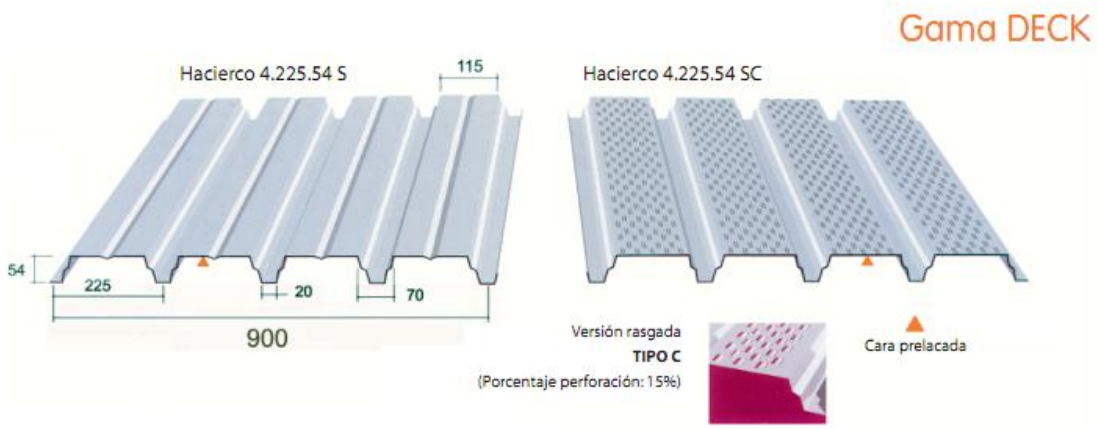
Part of the AutoCAD program is geared to the production of planes, employing the traditional resources of the graphics on the drawing such as color, line thickness and woven textures. AutoCAD, from Release 11, uses the concept of model space and paper space to separate the phases of design and 2D and 3D drawing of the specific plans drawn for the corresponding paper to scale.

4.7 Finishes and closures

The exterior closures have been made by metal sheet profiles and rock wool between creating an insulating chamber. All profiles roofs and facades have been selected from the company *ArcelorMittal*.

4.7.1 Closure of the roof

The closure of the roof is made of two different plates.The exterior is the develop the role of resistant element and has a profile Hacierco 4.225.54.



The ribbed steel profile Hacierco 4.225.54 S /SC is essential for the execution of enclosed decks. Provides support for insulation panels which adds a coating for roofs.

- Number of nerves: 4
- Step of fretwork: 225 mm
- Height of fret: 54 mm
- Width: 900 mm

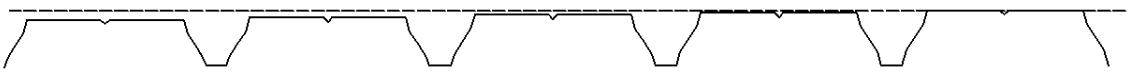
Terms of use
Sandwich deck

Material		Rules
Steel type	S 320 GD	EN 10346
Steel thickness	0,75/0,88/1,00/1,25 mm	EN 10143
Kind of protection	Galvanizado	EN 10346
	Galvanizado-prelacado	EN 10169-1
Prepainted	Carta Colorissime	

The problem we have is that the roof of the warehouse is pending and is not an important factor to consider when designing the fences for the rainwater.

For this reason we decided to implement small changes to the model chosen for that to facilitate water drainage.

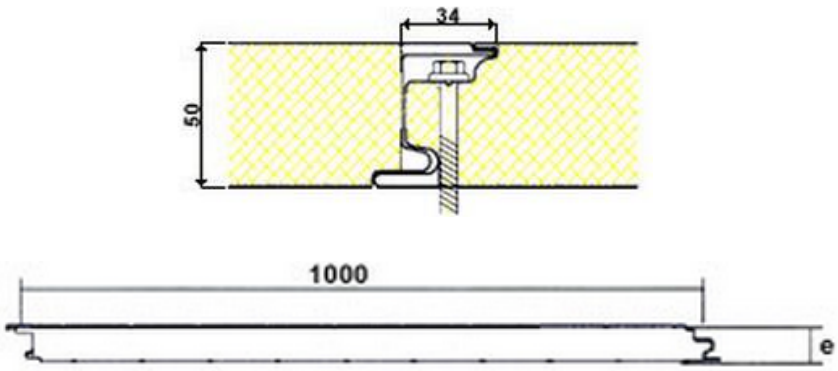
To be amended slightly the thickness of the plate getting the maximum thickness in the center of the ceiling in order to create a small slope.



4.7.2 Closure of the facade

This is a solution for the facades. It looks nice for the exterior and interior view and has a excellent performance: mechanical resistance, thermal and acoustics and good fire resistence.
With this innovative design it is possible to hide the structural elements and have some differents architectural advantages.

Luxsonor facade panels are highly organic products because they are composed at its core for mineral wool (inert elements). In addition, the hygienic properties of mineral wool (not allow the growth of microorganisms and insects inside, not food for rodents), are well suited for all types of buildings, especially in the food industry, supermarkets, etc.



DIMENSIONS	
Useful width	1150 mm
Thickness	50 mm (standar), 60, 80, 100, 120, 140 mm

COEFICIENTES TÉRMICOS			
Thickness panel in mm	50	80	100
Transmisión térmica K (W/m^2K)	0,66	0,43	0,35
Resistencia térmica R (m^2K/W)	1,50	2,30	2,83

LOADS (daN/m^2)						
	1,5 m	2 m	2,5 m	3 m	3,5 m	4 m
E=50 mm	92	64	45	31	20	13
E=80 mm	145	103	73	50	37	25
E=100 mm	194	148	108	77	58	44

Acoustic absorption coefficient		
Frecuencia (Hz)	Thickness 50 mm	Thickness 100 mm
100	0,33	0,44
500	0,67	0,66
1000	0,74	0,68
2000	0,73	0,65
5000	0,81	0,64

4.8 Economic assessment of writing project

The budget is broken down below does not refer the execution of the work, only the preparation of this project. The various stages that constitute the project are:

- *Project Analysis*: Structural alternative approach.

- *Global analysis of the structure of rods*: determination of the loads acting on structure and forces that are generated.
- *Calculation of joints*: study of the joints of the main porchs of the structure and the pillars.
- *Preparation of plans*: development of a 3D model for display in detail final structure and appearance of the ship, and preparation of 2D drawings.
- Amortization of computer support: computer programs (CYPE, PowerFrame and PowerConnect) and CAD program (AuroCad).

The purpose of this section is to obtain the total costs of this master thesis. These costs are broken down between the structure of the building, building materials and compounds for their execution, and the author considered the budget for the computation:

$$C_T = C + B$$

C_T is the total cost

C is the structure cost

B is the budget execution of the structure calculation

4.9 Environmental Impact Study

The number of pollutants that affect and degrade the environment are multiplied daily, and most have a common origin: they waste generated by humans in their daily activities.

Arround 50% of global resources allocated to the construction industry and this is where you generate more Rediu, which most are inert, but the problem which cause is the large volume generated. The consequences are on the one hand the environmental impact caused by the occupation of large areas for landfills and the degradation of the landscape and the other, unnecessary energy consumption used in the life cycle of waste.

At present, for approval of any project should include an executive environmental impact study. All this happens due to the need and awareness generated by taking care of environment and optimizes the resources needed to implement the project to be implemented. This final project is a design and calculation of a structure without considered the start of building work. Therefore, it is not thought necessary to conduct a study environmental impact, since this is considered a pre-executive. Moreover, taking into account current legislation, the task of implementing and present the environmental impact study of the implementation of a building is responsible for making all the executive project, including analysis of structure, facilities, etc.. Because this project was conducted in a company structure calculations has been taken into account established guidelines for development projects, which include non-conducting these tests. It is therefore not justifiable to carry out environmental impact analysis for this project.

4.9.1 Master thesis description

This project aims to design the project to define the structural design of an industrial warehouse located in Tallinn (Estonia).

The structure of the ship will be made with metal sections, the fundamentals of the ship will be isolated by rigid shoes connected reinforced concrete beams attached.

The phases of construction of the factory is divided into:

- Earthmoving
- Construction of foundation
- Construction of the metallic structure
- Placement of walls
- Paving

4.9.2 Analysis of alternatives

Location:

The warehouse is located in an existing plot, choosing this location was a decision known before the completion of the study. The site has all the necessary facilities for a possible activity, proximity to highways and other roads that reduce and facilitate travel.

Structural types

The ship was designed by optimizing the use of building materials, which will mainly steel and concrete. We have also ensured that most of the work of building the steel structure can be performed in workshop so you can better control the generation of waste, pollution and duration of construction works on the site.

4.9.3 Environmental analysis of steel

The warehouse is designed metal structure, ie, the main element is the steel. Therefore, this section will analyze the steel from the environmental point of view and applied in the construction sector.

Production and commissioning work

The steel is derived from iron, one of the most abundant elements on our planet. Can be obtained from the ore in facilities that have furnaces or from ferrous scrap in electric oven. Having two options for obtaining steel, presents the most important property of steel from the environmental point of view: the steel can be recycled once their initial use has reached its end. This means that once desballestada structure of the ship and turned into scrap this release are forming large compact is sent to foundries where achieved new steel products. It is estimated that recycled scrap metal covers 40% of global steel needs (number 2006). In less than 50 years, consumption of energy needed to manufacture steel and CO2 emissions have been reduced by half. Thus, in the manufacture of steel emissions to the environment have decreased due to filtration devices and recovery of gases and particles.

The waters are used systematically treated in facilities increasingly effective. The recycled to reduce consumption in natural reserves.

All waste is practically reused. Some are used for the manufacture of cement, which prevents the extraction of each year 4.5 million tons of limestone and reduced 2 million tons of CO2 emissions.

Recycling

For its virtues magnetic steel is selected very easily among all types of waste. It can be recycled indefinitely without losing any of its properties. In the field of construction, steel buildings are easily deconstruibles (dismantling and recycling of their components). This means that steel buildings at the end of the period of life do not require demolition. It disassembled easily, safely and cleanly, avoiding noise, dust and other damage to the local environment.

Cleaning works

Our steel belong to the denomination of "dry work". This is characterized by in situ assembly of products and components manufactured in factory. Therefore, the works are almost no waste, clean, dry and free of dust. The products can be supplied in the time required for assembly, limiting storage needs in work. The storage time is reduced and therefore the works are carried out cleanly and quickly. A steel construction requires far fewer trucks to work interventions, therefore, inert waste less and less air pollution.

Integration into the environment

The visual impact is also an aspect to consider. Therefore, the fact opt for building a metallic skeleton gives the designer freedom and flexibility of forms of intervention that can better adapt to their location.

4.9.4 Detection and identification of impacts

During construction of the factory there is a risk of negative impacts on the environment during this period. The effects on the environment will be minimal because the activity will take place in an urban environment, particularly in an industrial estate where there are no residents. The neighboring buildings could

be affected by these impacts, so we must apply the necessary corrective measures.

Here, are some environmental impacts that could result in their assessment (compatible, moderate, severe, critical) and the corrective measures proposed:

IMPACT	RATING	CORRECTIVE MEASURES
Dust	Moderate	Irrigated land to prevent the lifting of dust. Protection of cargo transport.
Machine noise	Compatible	Do not perform any activity that may generate noise from 8 am to 8 pm. It will check that the machine meets the current regulations regarding noise pollution.
Waste generation	Moderate	Minimization of waste generation. Separation of waste for reuse or recycling. Maintaining order and cleanliness of the area of work.
Generation of debris	Moderate	Minimising the generation of debris. Proper management of an entity authorized by the debris. Maintaining order and cleanliness of the area of work.
Machine gas emision	Moderate	Review and maintenance of machinery. Do not use the machinery when it is not essential
Spills of paints, oils and other pollutants	Moderate	Stored in a secure area to ensure no soil absorption or in case of spillage. Used containers suitable for each substance.

4.9.5 Environmental Impact Regulations

Regulations concerning environmental impact assessment procedures, and activities and projects subject to these procedures.

European Legislation

- Directive No. 1985/337 relating to the assessment of the effects of certain public and private projects on the environment. DOCE-L No. 175, of 05.07.1985.
- Directive No. 1997/11 on the evaluation of the effects of certain public and private projects on the environment. (amending Directive 1985/337). DOCE-L No. 73, 14.03.1997.
- Directive No. 2001/42 on the assessment of the effects of certain plans and programs on the environment. DOCE-L No. 197 of 21.07.2001.

5 Conclusions

My goal in this section is to provide a brief summary of the main points of this study.

First I would like to emphasize that I consider the main contribution of the document: to attempt a project that, in all respects as possible approaches to a fully realistic working environment.

In this regard, I have based on an existing site located in an industrial area of Tallinn, so I adapted to real conditions and problems, including the conditions imposed by the specific regulations that exist in the municipality regarding in industrial areas. Throughout the project we had in mind the big problem in this area: the snow and cold.

We have begun to calculate all the loads that occur in the area where the building stands. Then we had to play with the shape of the structure and different methods have been applied to improve the stability and rigidity without being used excessively large profiles. This point has been more complicated, lacking much experience in the sector has been more complicated, but with the help of guardian has been more enjoyable.

When we have reached a near-final model, we studied separately the two porches in 2D as well as various boards of our structure, all checking with the Eurocode 3.

To complete the study, we have chosen different types of cladding for the roof andfacades and have made a small environment study.

Finally, making an overall assessment of the project can be concluded that the result was satisfactory as the objectives have been met. Furthermore, adding that this project has allowed to touch different topics within the field structures and deal with some more depth.

6 Acknowledgements

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