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DEPARTMENT OF CIVIL ENGINEERING

A THESIS

Submitted in Partial Fulfilment of the Requirement for the degree of

MASTER

Field: **Civil Engineering**

Option: **Structures**

Presented by

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Title

Analysis and Design of a steel cold storage hangar according to Eurocode 03

Defended on 27/06/2024 in front of the jury composed of:

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بسم الله الرحمن الرحيم

Acknowledgment

First of all, I thank ALLAH, the almighty for giving me strength, courage, and
the will to carry out this modest work.
I would like to warmly and sincerely thank my supervisor
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Their expertise and generosity have significantly enriched my research
Finally, thanks go to all the people who have, from far or near, brought help and encouragement

Dedication

From the depths of my heart, I dedicate this work to all those dear to me.

To my beloved mother,

No dedication can fully express my respect, eternal love, and gratitude for the sacrifices you have made for my education and well-being.

I thank you for the unwavering support and love you have given me since my childhood, and I hope that your blessings will always be with me.

May this humble work fulfil your expressed wishes and be a testament to the countless sacrifices you have made. May God, the Most High, grant you good health, happiness, and a long life.

In loving memory of my father,

I dedicate this work to him. He left this world too soon, but he always encouraged and motivated me in my studies.

I hope that he, from his world now, appreciates this humble gesture as a token of recognition from a daughter who has always prayed for the salvation of his soul. May God, the Almighty, have mercy on him.

Abstract:

This master thesis consists of analyzing and designing a steel cold storage hangar located in the province of ADRAR in accordance with the Eurocode 03 standard. This study is carried out in several stages. Firstly, an assessment of the loads and overloads and the effects of climatic conditions (wind and sand) is carried out according to the Algerian regulation RNV2013. Then, the different structural elements are dimensioned, followed by studying the connections according to Eurocode 03. Finally, the infrastructure study is carried out according to CBA 93. The Autodesk Robot Structural Analysis Professional software was used for this study.

Keywords: Cold storage hangar, Steel structure, Structural analysis, Steel connections,

Résumé :

Cette thèse de master consiste à étudier et dimensionner un entrepot frigorifique en charpente métallique situé dans la Wilaya de ADRAR conformément à la norme Eurocode03. Cette étude est réalisée en plusieurs étapes. Tout d'abord, une évaluation des charges et surcharges ainsi que des effets des conditions climatiques (vent et sable) est effectuée selon le règlement algérien RNV2013. Ensuite, le dimensionnement des différents éléments structurels est réalisé, suivi de l'étude des assemblages selon le « Eurocode03 ». Enfin, l'étude de l'infrastructure est effectuée selon le CBA93 . Le logiciel Autodesk Robot Structural Analysis Professional est utilisé pour cette étude.

Mots clés : Entrepot frigorifique, Charpente métallique, Analyse structrale, assemblage métallique.

ملخص:

يتكون هذا الماستر من طالب وأبعاد لمستودع تبريد وداخل معدني يقع في ولاية وفقًا لمعيار Eurocode 03. هذا النمط متاح في المزيد من الكتب. بادئ ذي بدء، يتم إجراء تقييم للأحمال والزيادات وكذلك تأثيرات الظروف المناخية (الرياح والرمال) على القوانين الجزائرية RNV 2013. ومن ثم، تكون أبعاد العناصر الهيكلية المختلفة حقيقية، حيث أن التجميع سوف يمكن استخدامه في "Eurocode 03". وأخيرًا، سيؤثر تصميم البنية التحتية على CBA93. تم استخدام برنامج Autodesk Robot للتحليل الإنشائي الاحترافي في هذه الدراسة.

الكلمات المفتاحية: مدخل التبريد، السحر المعدني، التحليل الإنشائي، التجميع المعدني.

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Chapter I:
General Introduction

I.1.Introduction:

The construction field is vast and encompasses various aspects of design and execution, which can differ, based on the materials used and structure types. However, the main design goals are to create buildings that can withstand external forces and natural phenomena such as earthquakes and strong winds. The structure is designed to direct these external forces to the foundations, so understanding the constituent elements and their connections is crucial. Steel is the most commonly used material for new construction techniques due to its numerous advantages, including reliability, load-bearing capacity, speed of execution, and mechanical properties that allow for designing beams with long spans. Metal frame structures and roofs are typically made up of slender bars or thin elements, which makes them generally flexible. It is important to consider this flexibility when studying these structures, as issues such as flexibility, buckling, deflection of bent beams, and buckling of compressed elements are critical in verifying and designing steel structures. In our Master's thesis project, we focused on a steel frame structure – a cold storage hangar in the ADRAR province constructed with galvanized cold-formed profiles.

I.2.Problematic:

A cold storage building is a warehouse equipped with refrigeration equipment, or it can be described as a large-scale refrigeration system. Similar to a refrigerator, it primarily utilizes artificial means to create an environment that differs from the external temperature and humidity. It also serves as a storage facility with a consistent temperature and humidity for the products stored within.

Based on this introduction, we can generate the following questions:

- What is the lightest and strongest type of steel? How can it be made stainless?
- Which regulations are utilized to study and determine the size of this hangar?
- Should the structure be inspected after the installation of the equipment?

I.3.Hypotheses:

- Conducting a comprehensive analysis of a steel cold storage hangar in accordance with the applicable standards and codes.

I.4.Objectives:

The main goal of civil engineering studies is to design structures that can endure various natural events, such as earthquakes and strong winds. Achieving this requires creating structural systems that effectively combine the necessary properties to withstand these forces and transfer them to the foundations.

I.5.Motivation for choosing the theme (a steel cold storage hangar):

The use of refrigerated warehouses is becoming more and more prevalent in our daily lives. They are used in various industries, such as food factories, dairies, warehouses for fruits and vegetables, poultry and egg storage, aquatic product storage, pharmaceutical factories, chemical factories, vaccine storage, blood transfusion stations, laboratories, and more. Refrigerated warehouses offer the convenience of long-term storage for items that require low-temperature refrigeration. This allows us to manually set specific temperatures for these items, making our lives much easier.

I.6.Methodological approach:

The proposed work plan for structuring the thesis is as follows:

- Chapter I: General Introduction.
- Chapter II: Presentation of the structure.
- Chapter III: Evaluation of wind and sand loads according to RNV 2013.
- Chapter IV: Modelling and Numerical Analysis.
- Chapter V: Calculation of structural elements.
- Chapter VI: Calculation of assemblies.

Finally, we will conclude with a general conclusion.

Chapter II: Presentation of the Project

II.1 Introduction :

The study focuses on a steel cold storage hangar (Figure II.1) located in the ADRAR province . The hangar measures 98 meters in length and 62 meters in width and features a 40 cm expansion joint. According to the RPA2003 [1] ,ADRAR is categorized as zone 0, indicating minimal seismic activity. Additionally, the hangar is classified under snow region D, which means that there is no snow load. When designing the structure and choosing the materials, it is important to consider the key factors in this region, which are sand and wind (region III). These factors play a crucial role in ensuring that the hangar remains intact and fully functional in the local environmental climate conditions.

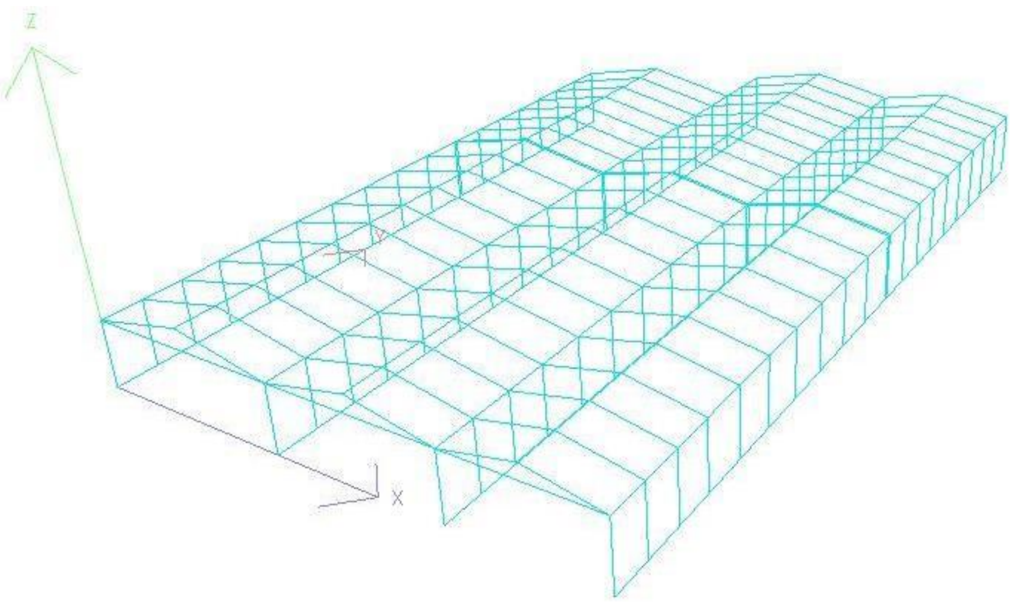


Figure II. 1: 3D model of the cold Storage hangar

II.2 Geotechnical study

The preliminary soil report for the project location provides information on:

- the load- bearing capacity of the soil. $\sigma_s = 2bars$

II.3 Geometric characteristics of the structure:

The cold storage warehouse currently in use has a rectangular shape, as shown in Figures 2 and 3. It is defined by the following dimensions:

- Building length: 98m, with an expansion joint of 40cm
- Building width: 62m
- Building height: 10.4m
- Column height: 8m
- Maximum roof height: 2.437m
- Maximum roof slope: 11.5°

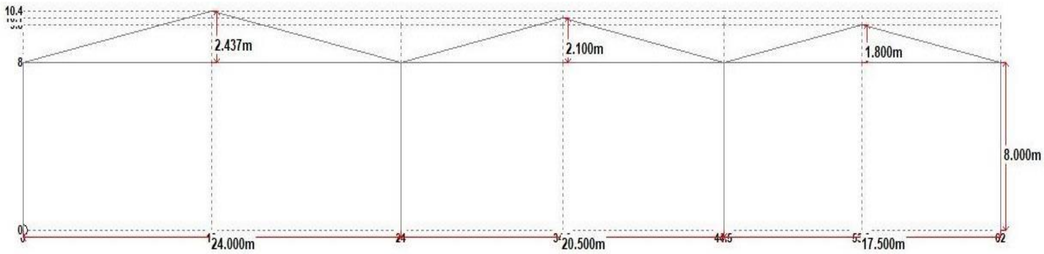


Figure II. 2 : Gable end view

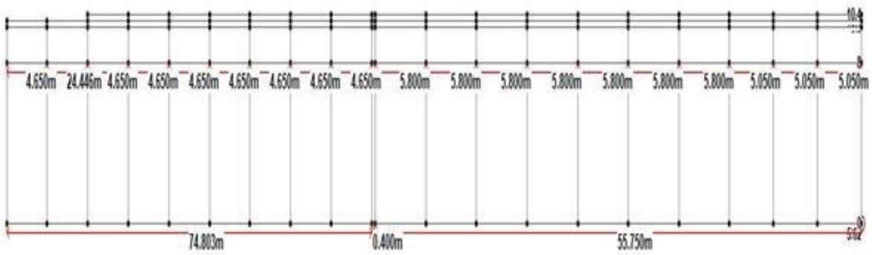


Figure II. 3: Long-span view

II.4 Structure elements

- ***Structure frame :***

The structure is comprised of three-span frames, which guarantee the transverse stability of the structure. Stability braces are responsible for ensuring longitudinal stability.

- ***Openings:***

Main gable end: 5 doors of (2x3) m²

Secondary gable end: 5 doors of (2x3) m²

Both faces of the long panel have no openings.

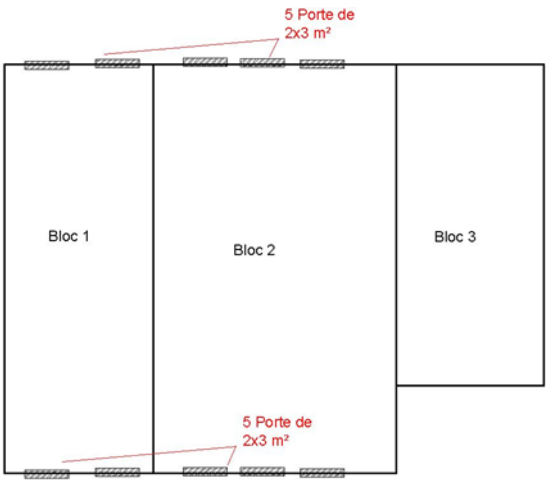


Figure II. 4: warehouse openings

- ***Roof:***

Generally, when designing industrial warehouses, the clearance of interior space is a top priority. This leads to the use of a steel frame roof, which offers various advantages, the most significant of which include:

- Elimination of interior columns, allowing for a more flexible and efficient utilization of space.
- The lightweight nature of the metal frame roof in comparison to a reinforced concrete slab or composite floor.
- Ease and efficiency of assembly

- ***Covering:***

The covering will be made of sandwich panels, also known as monobloc double-skin panels, which consist of two interior and exterior cladding sheets, an insulating foam core, and lateral profiles designed to protect the insulation and facilitate connections. Sandwich panels provide several advantages on site, including:

- Vapor barrier
- Insulation and waterproofing
- Strong load-bearing capacity
- Significant time savings during assembly.

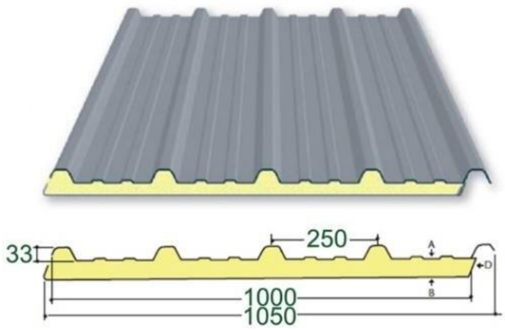


Figure II. 5: sandwich panels *[3]*

- ***Posts:***

The posts consist of metal profiles with a composite section.

- ***Purlins:***

The purlins consist of metal profiles with a composite section.

- ***Bracing:***

Stability braces in the shape of (x) in both directions ensure that the posts remain vertical and help bear wind loads by transmitting them to the foundations.

II.5 materials Used

- ***Concrete***

To construct the infrastructure and slabs, we use a specific type of cement known as CPA. This cement has a dosage of 350Kg/ and possesses the following properties:

Unit weight: $\gamma_{\ concrete} = 25\ KN/m3$

Compression strength: $f_{c28}= 25\ MPa$

Tensile strength: $f_{t28}= 2.1\ MPa$

- ***Steel :***

Steel consists mainly of iron and has a low carbon content. It is extracted from natural materials found in the subsoil, such as iron and coal mines. Carbon only makes up a small portion of its composition, generally less than 1%. In addition to iron and carbon, steel can contain other associated elements.

These elements can either be involuntarily present as impurities, such as phosphorus or sulfur, which can alter the properties of the steel. Or they can be intentionally added, such as silicon, manganese, chromium, and tungsten, which can improve characteristics like resistance to rupture, hardness, yield strength, ductility, and corrosion. In these cases, the steel is referred to as alloyed steel.

However, along with its advantages, steel also has disadvantages, which are presented below:

- ***Disadvantages:***

Steel has two major disadvantages:

- Its susceptibility to corrosion and low fire resistance due to loss of strength and rapid flow at relatively high temperatures.

- High cost.

- ***Properties:***

- Strength: Eurocode 03 provides the ultimate strengths of common steel grades.

- Durability: Structural steel must meet the following conditions:

- The ratio of ultimate deformation to elastic deformation must be greater than 20.

- ***Mechanical properties***

Yield strength: f_y = 235 MPa.

Ultimate strength: f_u = 360 MPa.

Density: ρ = 7850 Kg/m3

Longitudinal modulus of elasticity: E = 210000 MPa

Poisson's ratio: υ = 0.3

Thermal expansion coefficient α= 12.10

Transverse modulus of elasticity G = E/2(1+υ)

- ***Galvanized steel***

Galvanized steel is steel coated with a layer of zinc to protect it from corrosion. It is ideal for outdoor or industrial applications where exposure to moisture and other corrosive elements is common. Here are some reasons why galvanized steel is an excellent choice:

1. Corrosion Protection: The zinc coating acts as a barrier, protecting the steel from rust and corrosion.
2. Longevity: Galvanized steel has a longer lifespan than untreated steel due to its corrosion resistance.
3. Cost-Effectiveness: Although the initial cost of galvanizing steel may be higher than untreated steel, the long-term savings in maintenance, repair, and replacement costs often make it a cost-effective choice.
4. Versatility: Galvanized steel can be used in various applications thanks to its corrosion resistance and durability.
5. Environmental Benefits: Galvanizing steel conserves resources, reduces waste, and is recyclable.
6. Ease of Maintenance: Galvanized steel requires minimal maintenance compared to untreated steel. Regular inspections and occasional cleaning are typically all that's needed to maintain the protective zinc coating and ensure long-term corrosion resistance.
7. Overall, the use of galvanized steel offers numerous benefits, including enhanced corrosion protection, longevity, cost-effectiveness, versatility, and environmental sustainability, making it a preferred material choice in various industries and applications.

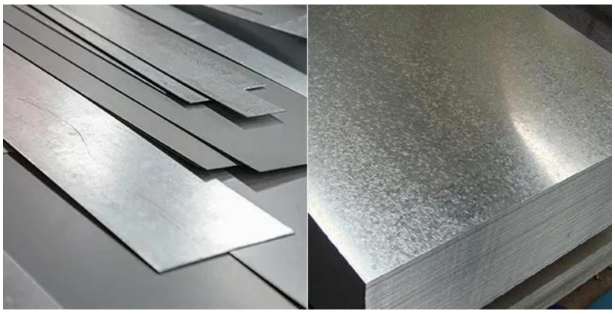


Figure II. 6: galvanized *steel sheets [4]*

II.6 Loads:

- dead loads (G) refer to the weight of the structure itself and all fixed components, including walls, roof, and carpeting.
- Live loads (Q) are temporary additional loads that exert extra force on a structure and its foundation.
- Climatic loads (S and W) encompass various environmental construction loads, such as sand and wind loads.

II.7 Codes Used :

The technical codes and standards used for the dimensioning of the studied structure are:

- Eurocode 01: Load Ponderations [4]
- Eurocode 3- part 1-1: Design of steel structures [4]
- Eurocode 3- part 1-8: design of joints [4]
- RNV 2013: Algerian code for snow and wind calculation [5]
- DTR-BC 2.2: Technical document regulation dead loads and live overloads [6]
- CBA 93 : Algerian concrete code [7]

II.8 Used software:

Autodesk's Robot Structural Analysis Professional (RSAP2023) and AutoCAD 2023 software are utilized for numerical analysis and design stages. Autodesk provides free educational versions of both software. By using my academic email (lagh-univ), I can ensure legal and cost-free access to these tools. RSAP2023 provides a wide range of tools for simulating, analyzing, and designing steel structures, steel joints, and concrete footings with accuracy and efficiency. Additionally, RSAP2023 offers an extensive selection of design codes and a vast library of hot-rolled and cold-formed profiles.

Chapter III:

Evaluation of Wind and Sand Loads According to RNV 2013

III.1 Introduction:

Any structure must be able to withstand both vertical and horizontal climatic forces. One of the significant horizontal forces is wind, which usually has a significant impact on steel structures. Therefore, a thorough study needs to be conducted to determine the various wind forces acting in all principal directions. The calculation of wind forces will be carried out in accordance with the Snow and Wind regulations specified in the RNVA2013 [5]. This regulatory technical document (DTR) [6] provides the general procedures and principles for determining wind forces on an entire construction and its various structural components.

The wind pressure values depend on several factors, including:

- The location of the construction
- The exposure of the construction
- The height of the construction
- The geometrical characteristics of the construction
- The stiffness of the construction, and so on.

III.2 Snow action:

III.2.1 Purpose and field of application:

The Algerian regulation establishes the standard values for the static snow load on any surface positioned above ground level and susceptible to snow accumulation, particularly on rooftops. It is applicable to all constructions in Algeria situated at an altitude below 2000 meters. For altitudes exceeding 2000 meters, the specifier must specify the snow load value to be considered.

III.3 Over all action:

ADRAR, where the warehouse is located, is classified as ZONE D. Only the sand load will be considered, which is the accumulation of sand grains deposited by the wind on the roof.

III.4 Sand load:

In the case of sloping roofs, it is necessary to consider a linear load located along the lower edges of the slopes (TB.5 p33) [5]. The studied cold storage warehouse has 3 different high-slope roofs (slope 11.5° >5°) [5], the application procedure of wind loads is detailed in the regulation, two values of sand linear loads noted q1 and q2 (in KN/ml) are given

(See table III.1)

Table III. 1:Sand load in zone D according to RNV2013 regulation

Wilaya	Surface load (N/m²)	Linear load q 1(KN/m)	Linear load q 2(KN/m)
ADRAR	**0.2**	**0.3**	**0.4**

III.5 Wind load:

III.5.1 Introduction:

Any steel construction work must resist various horizontal and vertical actions.

Among the horizontal actions we can cite the wind.

The effect of wind on a construction is quite significant and has a great influence on the stability of the structure. For this, an in-depth study must be carried out to determine the different actions due to the wind in all possible directions.

The calculation will be carried out in accordance with the Snow and Wind Regulations RNV2013 [5]. This regulatory technical document (DTR) [6] provides the general procedures and principles for determining the actions of the wind on an entire construction and on its different parts.

The actions of the wind applied to the walls depend on:

- The direction.

- Intensity.

- The region.

- The location of the structure and their environment.

-The geometric shape and openings of the structure.

III.5.2 Wind direction:

The calculation must be carried out separately for each of the directions perpendicular to the different walls of the construction.

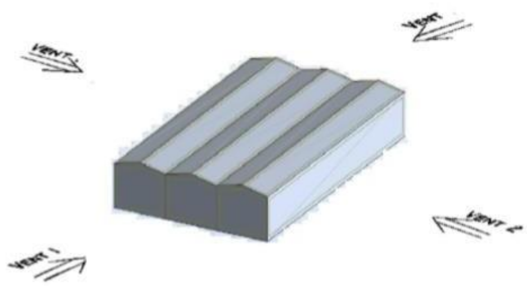


Figure III. 1: Wind directions

III.5.3 Determination of the different parameters and calculation coefficients:

a. Effect of region:

Dynamic reference pressure value ref:

The peak dynamic pressure (Ze), at the reference height ze is given by:

$$(Ze) = q_{ref} \times (Ze)$$

q_{ref} is the reference dynamic pressure given for permanent constructions by the table below depending on the wind zone. The project is located in the wilaya of ADRAR, and according to RNV2013 [5] the ADRAR province is classified in zone III, and the reference dynamic pressure is:

$$q_{ref} = 500 \text{ N}/m^2$$

Table III.2 : Reference dynamic pressure

Area	q_{ref} N/m^2
I	370
II	435
III	500
IV	575

b. Terrain category:

The structure is located in the ADRAR province and according to the RNV2013 regulation [5] the land is category I:

Table III.3: Determination of terrain category (RNV2013)

Category of land	Kt	Z0 (m)	Zmin (m)	
0 sea, or coastal area exposed to sea winds.	0.156	0.003	1	0.38
I Lakes or flat, horizontal area with negligible vegetation and free of any obstacle.	0.17	0.01	1	0.44
III Areas with regular vegetation cover or buildings, or with isolated obstacles	0.19	0.05	2	O.52

separated by more than 20 times their height (e.g. villages, suburban areas, permanent forests).				
IV Urban areas where at least 15% of the surface area is occupied by buildings with an average height greater than 15 m.	0.215	0.3	5	0.61

With:

K_t: field factor

Z_0: roughness parameter

: minimum height

c. Topography coefficient :

The topography coefficient $\boldsymbol{C_t}$ takes into account the increase in wind speed when it blows over obstacles such as hills, isolated drops, etc.

For practical reasons the values given in table 5 are used.

The structure studied is located on a flat site, therefore the topography coefficient $\boldsymbol{C_t}$ is equal to: $\boldsymbol{C_t}$=1

Table III.4:Topography coefficient

Site	Ct
Flat site	1
Site around valleys without funnel effect	1
Site around the valleys with funnel effect	1.3
Site around the plateaus	1.15

Site around the hills	1.15
Mountainous site	1.5

b. Calculation of the dynamic coefficient Cd:

The dynamic coefficient Cd accounts for the reduction effects caused by the imperfect correlation of pressures exerted on the walls, as well as the amplification effects caused by the turbulence component having a frequency close to the fundamental oscillation frequency of the structure.

- Simplified value:

A conservative value of Cd=1.0 can be considered in the following cases:

Buildings with a height of less than 15 m;

- Facade and roof elements with a natural frequency of less than 5 Hz;
- Frame buildings consisting of walls with a height less than 100 m and 4 times the dimension of the building measured perpendicular to the wind direction;
- Chimneys with a circular cross-section and a height less than 60 m, and 6.5 times the diameter.

The structure of the building is a metal structure with a height of less than 15 m (H = 10.43 m).

We take Cd=1

We have: Cd < 1.2.

Therefore, the construction is not very sensitive to dynamic excitations.

c. Determination of aerodynamic pressure W(Z):

The aerodynamic pressure W(Z) acting on the walls in (daN / m^2) is given by the following formula:

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

Where: q_p **(z):** peak dynamic pressure calculated at the height Z considered in

(daN/ m^2) .

: external pressure coefficient.

: internal pressure coefficient.

e.1 Calculation of the peak dynamic pressure q_p (z) :

The peak dynamic pressure q_p (Z) which is exerted on a surface element at the height Z is given by the relation:

$$q_p\ (z) = \quad Ce(Z)$$

q_{ref}: is the reference dynamic pressure for permanent constructions depending on the wind zone.

Ce(z): is the wind exposure coefficient

Zi: reference height

In our case: $Ze = Zi = Z$ with (Z=h and b=62m)

h<b$\Rightarrow$ Ze =h (q_p (z) = ( ))

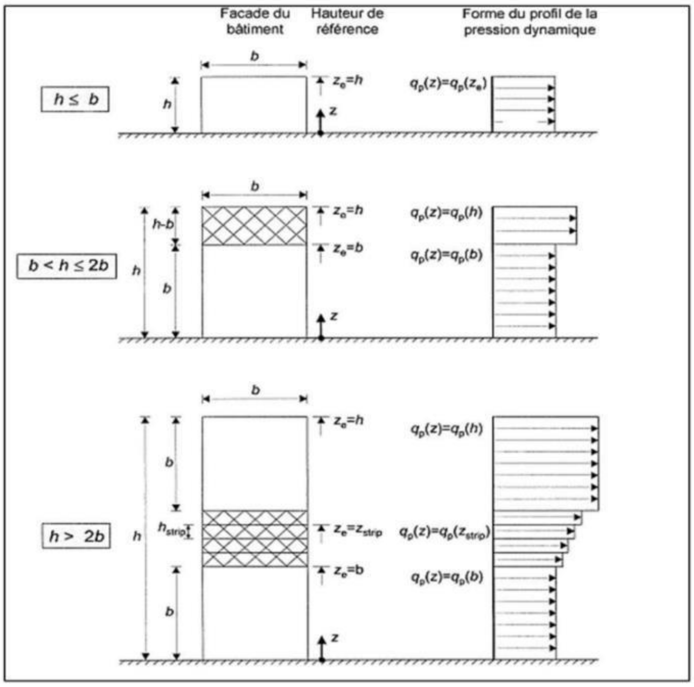


Figure III. 2: Reference height Ze and the corresponding dynamic pressure profile [6]

e.2 Wind exposure coefficient C_e (z) :

The wind exposure coefficient C_e (z) takes into account the effects of the roughness of the terrain, the topography of the site and the height z above the ground.

Additionally, it takes into account the turbulent nature of the wind.

$$C_e(z) = C_t^2(z)\ C_r^2(z)\ [1+7I_v(z)]$$

With:

: roughness coefficient.

: topography coefficient.

Iv: the intensity of the turbulence

e.3 Roughness coefficient :

The roughness coefficient C_r reflects the influence of roughness and height on the average wind speed.

The roughness of the ground creates friction which slows the wind at low altitude.

It is defined by the following law: (RNV 2013 §2.4.4) [6]

$$C_r(Z) = K_t\, ln5\left(\frac{Z}{Z_0}\right) for\ Z_{min} \leq Z \leq Z_{max} = 200$$

With:

: terrain factor.

z_0: roughness length (in m).

:: minimum height (in m). Z:

height considered (in m).

Table III.5:Tterrain category parameters

Category of land	**Kt**	z_0 (m)	z_{min} (m)
I	0.17	0.01	1

Cr (**z**) $= Kt \times \ln\left(\frac{}{Z0}\right)$ For $Zmin \leq Z \leq Zmax$=200m

For z=8m; Cr(z) = $0{,}17 \times \ln\left(\frac{8}{0{,}01}\right)$= 1.13

For z=10.43 ; Cr(z) = $0{,}17 \times \ln\left(\frac{10{,}43}{0{,}01}\right)$= 1.18

e.4 Turbulence intensity:

The intensity of turbulence is defined as the standard deviation of turbulence divided by the average wind speed and is given by the relationship:

$$\mathbf{IV(z)} = \frac{1}{ct(z)\ln\left(\frac{z}{z0}\right)}\ \mathbf{for\ Z > Zmin}$$

$$\mathbf{IV(z)} = \frac{1}{ct(z)\ln\left(\frac{zmin}{z0}\right)}\ \mathbf{for\ Z \leq Z_{min}}$$

Or:

C_t topography coefficient

Z_0: roughness length (in m).

: minimum height (in m). Z:

height considered (in m).

IV(z) = $\frac{1}{(z)\times\ln\left(\frac{}{z0}\right)}$

for Z=8m;Iv(z) $= \frac{1}{1\times\ln\left(\frac{8}{0{,}01}\right)} = 0{,}15$

for Z=10.43m ; Iv(z) = $\frac{1}{1 \times \ln\left(\frac{10,43}{0,01}\right)}$ = 0.14

Ce (z): Wind exposure coefficient:
This(z)=$ct^2(z) \times cr^2 \times [1 + 7\mathrm{Iv}(z)]$

For z=8m; C_e (z)= $1^2 \times 1{,}136^2 \times$[1+7x 0.1496]=2.64

For z=10.43m ; C_e (z)= $1^2 \times 1{,}181^2 \times$[1+7x 0.144]=2.80

q_p (z): peak dynamic pressure:

$$q_p \text{ (Z)} = q_{ref} \text{X } C_e \text{ (Z)}$$

for z=8m; q_p (z) = 500 $\times$ 2,642 = 1320,95N

for z=10.43m; q_p (z) = 500$\times$ 2,80 = 1400,50N

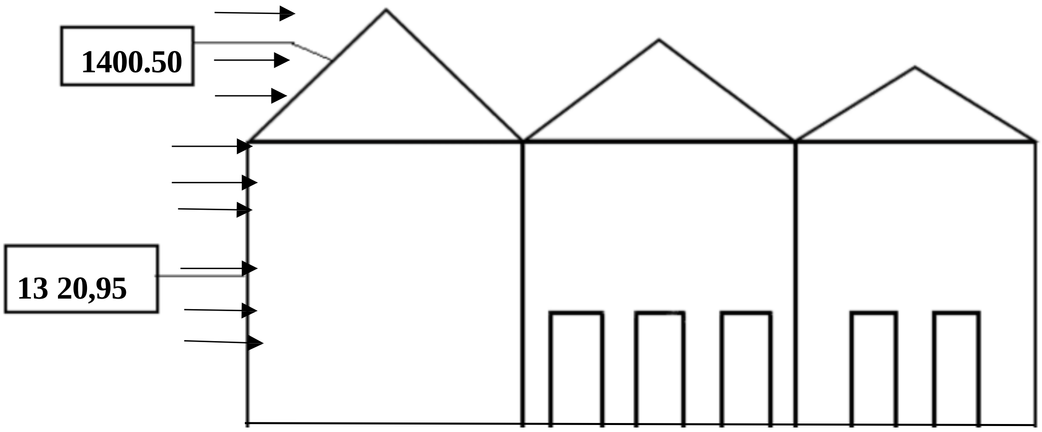


Figure III. 3: the distribution of the peak dynamic pressure over the height Ze

e.5 Determination of external pressure coefficient C_{pe}:

The external pressure coefficients C_{pe} of rectangular-based constructions and their individual constituent elements depend on the size of the loaded surface. They are defined for surfaces loaded with 1 $m^2$and 10 $m^2$to which corresponding pressure coefficients noted respectively C_{pe1} and C_{pe10}.

C_{pe} is obtained from the following formulas:

C_{pe} = Cpe1 if S≤ 1m2

C_{pe}= Cpe1 + (Cpe10- Cpe1) log10(s) if 1m2 <S< 10m2

C_{pe} = Cpe10 if S≥ 10m2

Such as : **S** : is the loaded surface of the wall considered in m²

e.5.1 Case of wind perpendicular to the gable (direction V1) :

e.5.1.1 Calculation of C_{pe} for vertical walls:

According to figure 5.1 of the RNVA 2013, the walls are divided as follows:

b=62 m; h=8m; d=98m; e=min[b; 2h] = 16m; d>e

Table III.6:the surfaces of the loaded zones for vertical walls

Area	A		B		C		D		E	
Geometric dimension (m)	e/5	H	e-A	H	b-e	H	d	H	d	h
	3.2	8	12.8	8	46	8	98	8	98	8
The surface (m^2)	25.6		102.4		368		784		784	

We notice that all the surfaces are greater than 10 m² Therefore: C_{pe} = Cpe,10 for each zone. (According to RNV 2013 chap5, article 5.1.1.2) [6].

- According to table 5.1 of RNV 2013 [6] we determine the C_{pe}values which correspond to each zone

Table III.7:Exterior pressure coefficients (vertical walls)

Area	A	B	C	D	E
Cpe10	-1	-0.8	-0.5	+0.8	-0.3

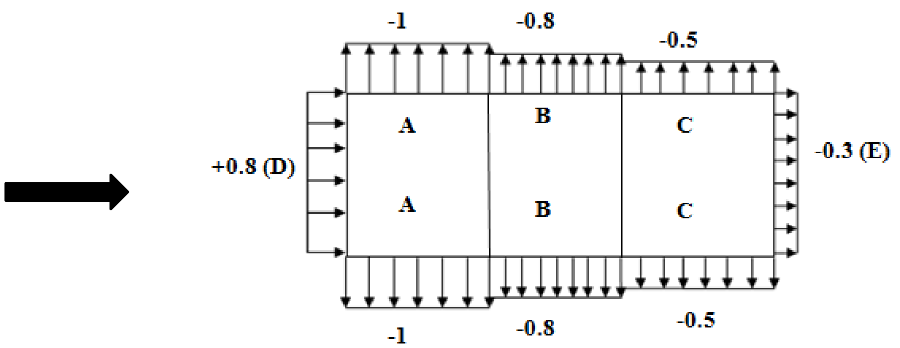


Figure III. 4: The values corresponding to each zone of the vertical walls

e.5.1.2 The roof :

The wind direction is defined by the angle θ, and in the present case:

The wind is perpendicular to the pinion (direction V1) and therefore parallel to the generators; θ = 90 ° (multi-slope) (According to RNVA 2013 [6] Chap5, article 5.1.5.1).

For θ =90 °, we will define the different pressure zones F, G, H and I which are represented in the following figure:

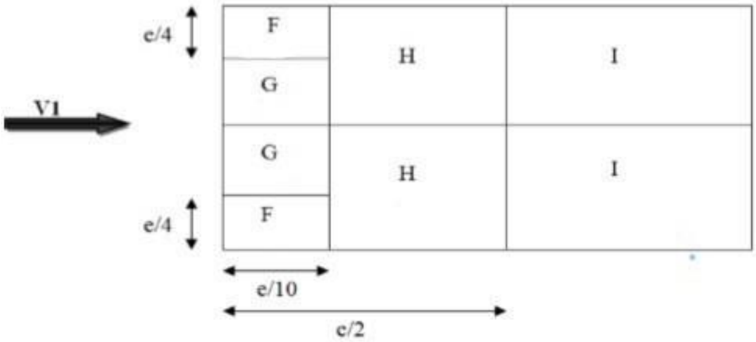


Figure III. 5: distribution of pressure zones on the roof, direction V1

b=62m; h=10.43m; d=98m And we ae= min [b, 2h]=20.86 → e=20.86m

Table III. 8:The surfaces of the loaded zones for the roof

The area	F		G		H		I	
Geometric dimension (m)	e/4	e/10	$b-\frac{e}{2}$	e/10	$\frac{e}{2}-\frac{e}{10}$	b	$d-\frac{e}{2}$	b
	5,215	2,086	51.57	2,086	8,344	62	87.57	62
The surface (m^2)	10,879		107,575		517,328		5429.34	

We note that:

- The external pressure coefficient depends on the size of the loaded surface, we define $\boldsymbol{C_{pe1}}$ and $\boldsymbol{C_{pe10}}$ the external pressure coefficients for a surface of 1m² and 10m², respectively, the values for other surfaces S are obtained by logarithmic interpolation:

C_{pe}=C_{pe1} if S<1m²

C_{pe} =C_{pe1}+ (C_{pe10} -C_{pe1}) Log10(S) if 1m²<S<10m²

C_{pe} =C_{pe10} if S>10m²

We note that all the surfaces of the pressure zones are greater than 10m2 (S ≥ 10m 2). So:

C_{pe10} = C_{pe10} for each zone.

Table III. 9:External pressure coefficient for gable roof

Slope	Directional wind zone = 90°							
	F		G		H		I	
	$C_{pe.10}$	$C_{pe.1}$	$C_{pe.10}$	$C_{pe.1}$	$C_{pe.10}$	$C_{pe.1}$	$C_{pe.10}$	$C_{pe.1}$
5 °	-1.6	-2.2	-1.3	-2.0	-0.7	-1.2	-0.6	
15 °	-1.3	-2	-1.3	-2.0	-0.6	-1.2	-0.5	
30 °	-1.1	-1.5	-1.4	-2.0	-0.8	-1.2	-0.5	
45 °	-1.1	-1.5	-1.4	-2.0	-0.9	-1.2	-0.5	
60 °	-1.1	-1.5	-1.2	-2.0	-0.8	-1	-0.5	
70 °	-1.1	-1.5	-1.2	-2.0	-0.8	-1	-0.5	

The values of the exposure coefficients $C_{pe}$are determined by a linear interpolation between the two values of the same sign for α = 5° and α = 15° taken from the table (table 5.4, chapter 2, RNVA2013) [6].

We have: α_0=5° < α =11.5° < α_1=15°

$F(x)$=$f(x_0) + \frac{f(x)-f(x_0)}{x-x_0}\ (\alpha - \alpha_0) \Rightarrow$

$$Cpe1(\alpha) = C_{pe1}(\alpha_0) + \frac{C_{pe1}(\alpha_1) - C_{pe1}(\alpha_0)}{(\alpha_1 - \alpha_0)}$$

$$Cpe10(\alpha) = C_{pe10}(\alpha_0) + \frac{C_{pe10}(\alpha_1) - C_{pe10}(\alpha_0)}{(\alpha_1 - \alpha_0)}$$

Example of linear interpolation:

Zone F:

$$cpe_1 = cpe_{10} = -1{,}6 + \frac{11{,}5 - 5}{15 - 5} \times (1{,}6 - 1{,}3) = -1{,}41$$

The results of cpe_{10} and cpe_1 for the angle $\propto$ =11.5 ° are shown in the following table:

Table III.10: C_{pe} values which correspond to each roof zone, case of wind in direction V1

Slope	F	G	H	I
	cpe_{10}			
α =11.5°	-1.40	-1.3	-0.63	-0.53

e.5.2 Wind perpendicular to the long side (V2):

e.5.2.1 Calculation of C_{pe} for vertical walls

For this wind direction we have: b=98m, d=62m, h=8m

And e= min [b, 2h] = 16m → e=16m

And we have: d=62m > e=16m.

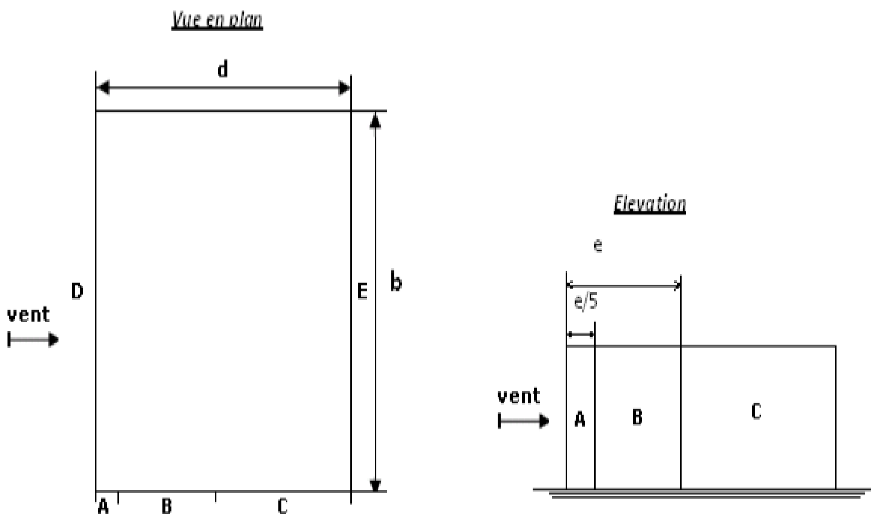


Figure III. 6: dimensions of zones A, B, C, D and E

e.5.2.2 Surface determinations:

Table III.11:the surfaces of the loaded zones for vertical walls

Area	A		B		C		D		E	
Geometric dimension (m)	e/5	H	e-A	H	Of	H	B	h	b	h
	3.2	8	12.8	8	46	8	98	8	98	8
Surface (m^2)	25.6		102.4		368		784		784	

We note that all surfaces are greater than 10m2 so the corresponding formula is: $C_{pe} = C_{pe10}$

And for the determination of the values of external pressure coefficients C_{pe}

And the external pressure coefficients C_{pe} in each zone are given in the following table:

Table III.12: C_{pe} values which correspond to each zone of the vertical walls, direction V2

Area	A	B	C	D	E
Cpe10	-1	-0.8	-0.5	+0.8	-0.3

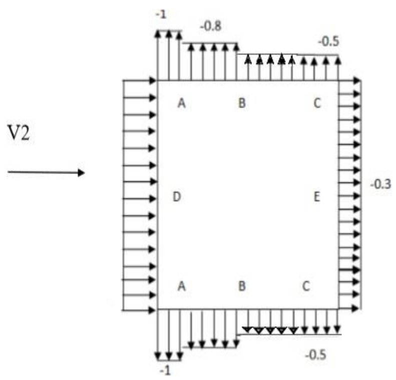


Figure III. 7: C_{pe} values which correspond to each zone of the vertical walls direction V2

e.5.2.3 Calculation of C_{pe} for Roof:

For a wind V2 perpendicular to the generator, for surfaces of 10 m², we will take Roofs whose slopes have a positive slope The C_{pe} values of a roof with one slope corresponding to Θ=0°

for the first slope

b=98m; d=62m; h=10.43m; e=min[b,2h]=20.86m

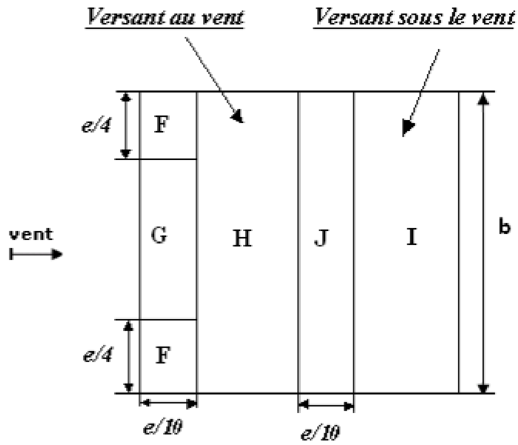


Figure III. 8:distribution of pressure zones on the roof, direction V2

e.5.2.4 Calculation of areas:

Table III.13:the surfaces of the pressure zones on the roof, direction V2

Area	F		G		H		J		I	
Geometric dimension (m^2)	e/10	e/4	e/10	b-$\frac{e}{2}$	b-$\frac{e}{5}$	b	e/10	b	b-$\frac{e}{5}$	b
	2,086	5,215	2,086	87.57	57,828	98	2,086	98	57,828	98
Surface (m²)	10,87		251,50		5667,14		273,26		5667,14	

We note that:

• all surfaces of pressure zones F, G, H and I are greater than 10 m² (S ≥ 10 m²).

So: C_{pe} = C_{pe10} for each zone. The values of the exposure coefficients C_{pe} are determined by a linear interpolation between the two values of the same Sign for α_0=5° and α_1=15° taken from the table RNV2013 [5].

Table III.14:the surfaces of the loaded zones for the roof

Slope	Zone for directional wind = 0°									
	F		G		H		I		J	
	cpe_{10}	cpe_1	cpe_{10}	cpe_1	cpe_{10}	cpe_1	cpe_{10}	cpe_1	cpe_{10}	cpe_1
5°	-1.7	-2.5	-1.2	-2.0	-0.6	-1.2	-0.6		-0.6	
									+0.2	
15 °	-0.9	-2.0	-0.8	-1.5	-0.3		-0.4		-1.0	-1.5
	+0.2		+0.2		+0.2					
30 °	-0.5	-1.5	-0.5	-1.5	-0.2		-0.4		-0.5	

	+0.7	+0.7	+0.4		
45 °	+0.7	+0.7	+0.6	-0.2	-0.3
60 °	+0.7	+0.7	+0.7	-0.2	-0.3
75 °	+0.8	+0.8	+0.8	-0.2	-0.3

We have: α_0=5° < α=11.5° <α_1=15°

Linear interpolation:

$$F(x) = f(x_0) + \frac{f(x)-f(x_0)}{x-x_0}(\alpha-\alpha_0) \Rightarrow$$

$$Cpe1(\alpha) = C_{pe1}(\alpha_0) + \frac{C_{pe1}(\alpha_1) - C_{pe1}(\alpha_0)}{(\alpha_1 - \alpha_0)}$$

$$Cpe10(\alpha) = C_{pe10}(\alpha_0) + \frac{C_{pe10}(\alpha_1) - C_{pe10}(\alpha_0)}{(\alpha_1 - \alpha_0)}$$

The results of Cpe,10 and Cpe,1 for the angle ∝=11.5° are represented in the following table:

Table III.15:Cpe values which correspond to each roof zone; case of wind in direction V2

Area	F	G	H	I	J
C_{pe}	**-1.18**	**-0.94**	**-0.40**	**-0.47**	**-0.86**

e.6 Determination of internal pressure coefficient C_{pi}:

For buildings without a dominant face, the interior pressure coefficient C_{pi} is determined from figure 5.14 of the RNVA 2013.

With: (h) the height of the building, (d) its depth and µp the permeability index given by:

$$\mu p = \frac{\Sigma \; des \; surface \; des \; ouvertures \; sous \; le \; vent \; et \; parallèle \; au \; vent}{\sum des \; surfaces \; de \; toutes \; les \; ouvertures}$$

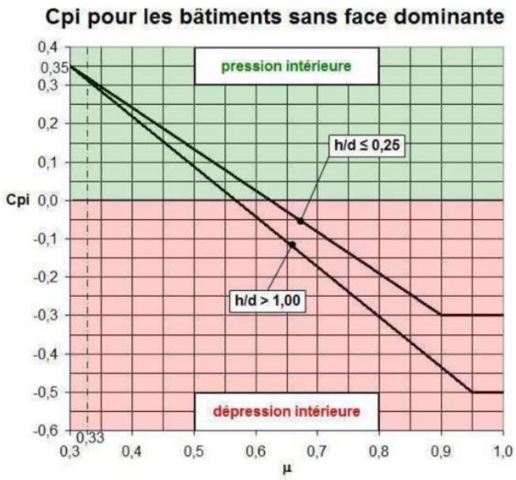


Figure III. 9:Interior pressure coefficient Cpi of buildings without dominant face Source : [6]

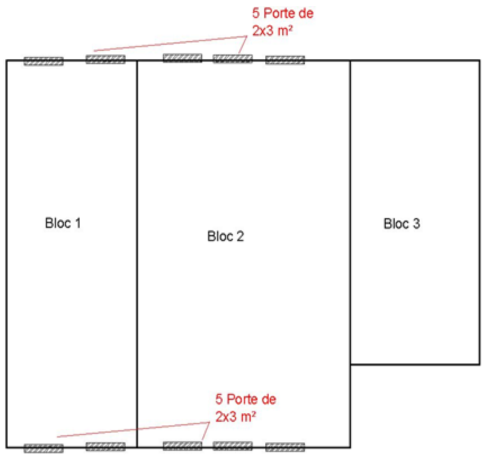


Figure III. 10:Arrangement of openings

e.6.1 Long-span: no openings at the long-pan

So: µp = 0

Values of the internal pressure coefficient Ci:

The interior pressure coefficient Ci of buildings without interior partitions (industrial hall for example) is given as a function of the permeability index µp

For µp2= µp4 =0 ⇒ Ci = 0.8

e.6.2 The pinion:

$\mu p = \frac{[(2\times3)\times5\times2]}{[(2\times3)\times5\times2]} = 1$

For $\mu p1 = \mu p3 \Rightarrow Ci = -0.5$

e.7 Determination of aerodynamic pressure :

e.7.1 Wind perpendicular to the pinion (direction V1)

e.7.2 Vertical Walls:

Table III.16:values of Wj corresponding to each zone of the walls with cpi =-0.5

Area	C_d	q_p (N/m²)	C_{pe}	C_{pi}	C_{pe}-C_{pi}	W_j (N/m²)
A	1	1320.95	-1	-0.5	-0.5	-660,47
B		1320.95	-0.8		-0.3	-396,28
C		1320.95	-0.5		0	0
D		1320.95	+0.8		+1.3	1717.23
E		1320.95	-0.3		+0.2	264.19

e.7.3 The roof:

Table III.17: Wj values correspond to each roof zone with cpi =-0.5

Area	C_d	q_p (N/m²)	C_{pe}	Cpi	C_{pe}-C_{pi}	W_j (N/m²)
F	1	1400.50	-1.40	-0.5	-0.90	-1267.45
G		1400.50	-1.3		-0.8	-1120.4
H		1400.50	-0.63		-0.13	-189.06
I		1400.50	-0.53		-0.03	-49.01

e.7.2 Wind perpendicular to the long section (direction V2)

e.7.2.1 Vertical Walls:

Table III.18:values of wjcorresponding to each wall zone with cpi =+0.8

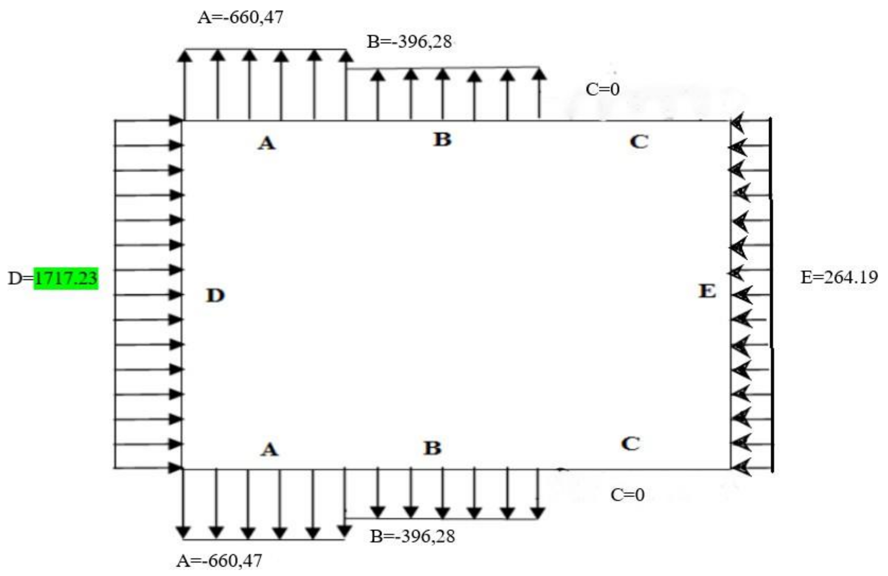
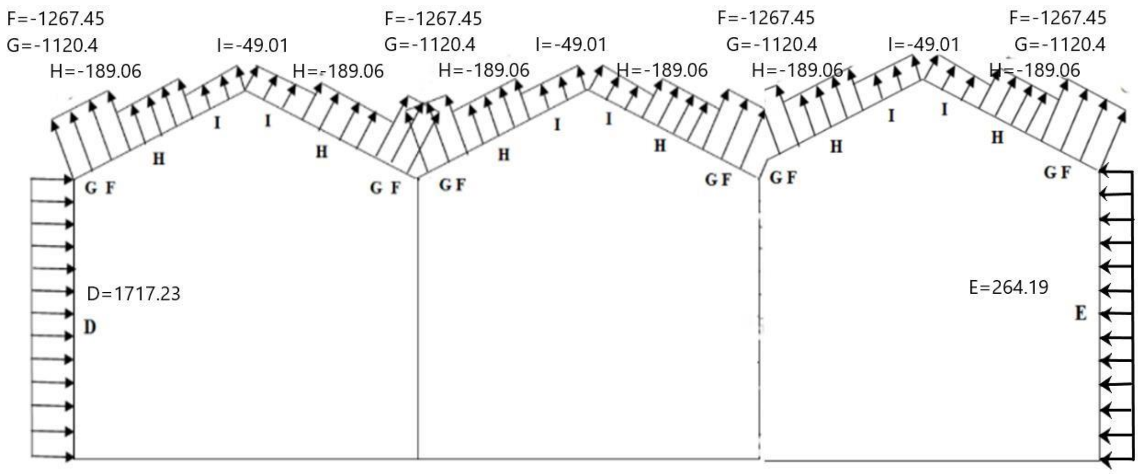
Area	C_d	q_p (N/m²)	C_{pe}	Cpi	C_{pe}-C_{pi}	w_j (N/m²)
A	1	1320.95	-1	+0.8	-1.8	-2377.71
B		1320.95	-0.8		-1.6	-2113.52
C		1320.95	-0.5		-1.3	-1717.23
D		1320.95	+0.8		0	0
E		1320.95	-0.3		+0.5	660,47

e.7.2.2 The roof:

Table III. 19:Values of wj corresponding to each roof zone with cpi =+0.8

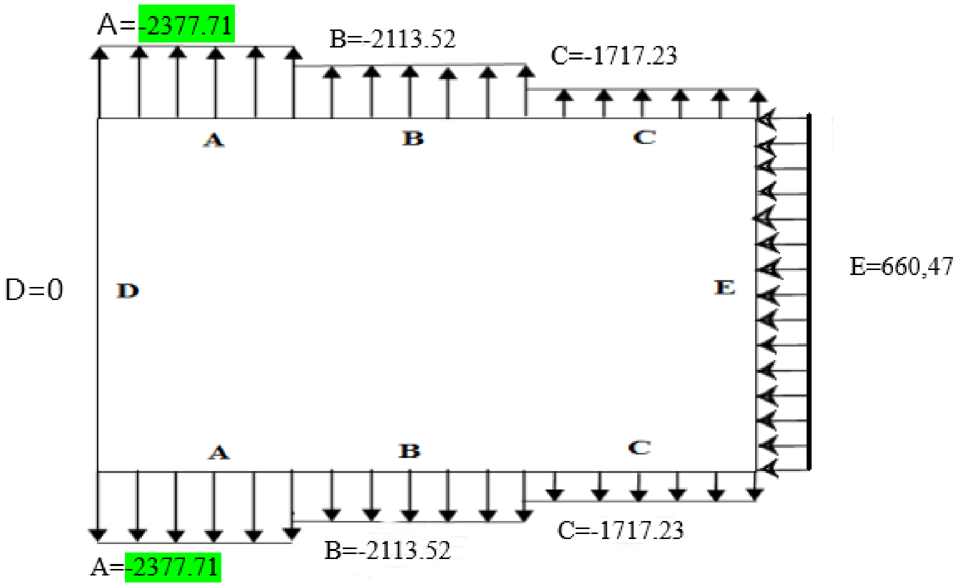
Area	C_d	q_p (N/m²)	C_{pe}	C_{pi}	C_{pe}-C_{pi}	w_j (N/m²)
F	1	1400.50	-1.18	+0.8	-1.98	-2772.99
G		1400.50	-0.94		-1.74	-2436.87
H		1400.50	-0.405		-1.205	-1687.60
I		1400.50	-0.47		-1.27	-1778.63
J		1400.50	-0.58		-1.38	-1932.69

e.8 Summary of the pressures exerted on the structure (wind on gable):

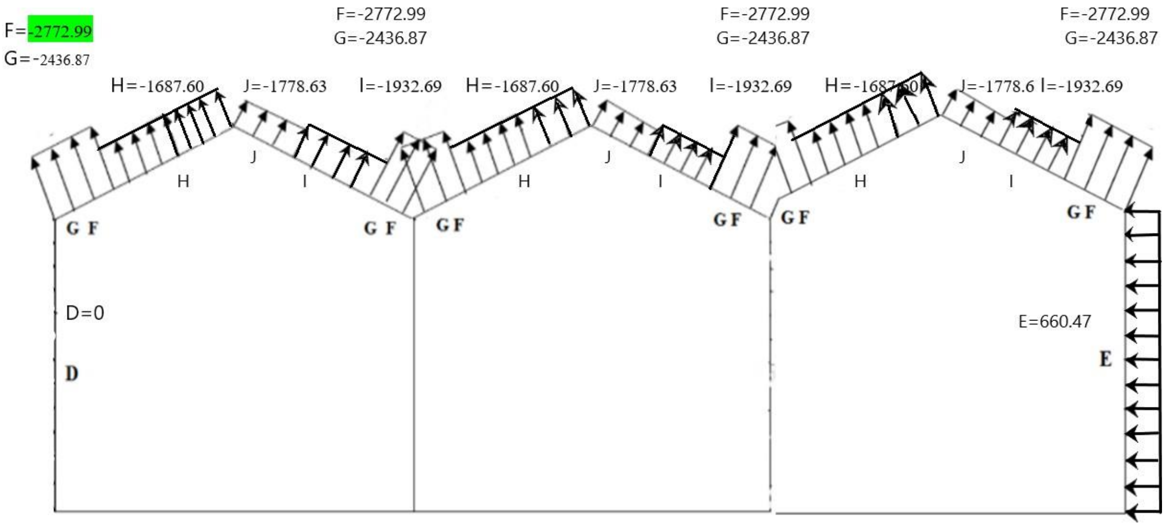
Wind On the Gable V1 Cpi = -0.5	**Pressure distributions on vertical walls** 
Wind on the Gable V1 Cpi = -0.5	**Pressure distribution on roofs** 

e.9 Summary of the pressures exerted on the structure (wind on long section):

<table>
<tr>
<td>on
the
Gable
V2
Cpi
=
+0.8</td>
<td>Pressure distributions on vertical walls



</td>
</tr>
<tr>
<td>on
the
Gable
V2
Cpi
=
+0.8</td>
<td>Pressure distributions on roofs



</td>
</tr>
</table>

III.6 Calculation of the friction force

III.6.1 Friction coefficients :

Table III.20:Values of friction coefficients

Surface condition	Friction coefficient C_{fr}
Smooth (Steel, smooth concrete, undulations parallel to the wind, coated wall, etc.)	0.01
Rough (rough concrete, uncoated wall, etc.)	0.02
Very rough (undulations perpendicular to the wind, ribs, folds, etc.)	0.04

The friction force F_{fr}given by:

$$F_{fr}=\Sigma\ (\ \ \times q_p(z_e) \times A_{fr})$$

Or:

: (in N/m²) is the dynamic peak wind pressure.

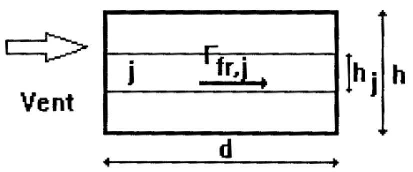
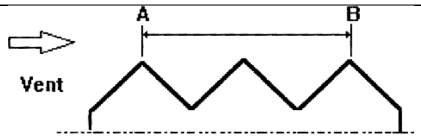
: (in m²) exterior surface parallel to the wind

: is the friction coefficient for the surface element j

III.6.2 The friction area:

The friction area must be determined as shown in the following table:

Table III.21:Friction area A_{fr}

Wall type	Plan	A_{fr}(m²)
Vertical wall		$A_{fr} = dxh$
Multi-sloped roof - wind parallel to generators		$A_{fr} = \left(\begin{matrix}\textit{somme des longueur devlopees}\\ \textit{de la toiture}\end{matrix}\right)$
Multi-sloped roof – wind perpendicular to generators		$A_{fr} = (longueur\ AB)xd$

a. *Wind perpendicular to the gable:*

a.1 Vertical wall

$A_{fr} = 8 \times 98$=784 m²

F_{fr} =784N $\times 0{,}04 \times 1320{,}95 = 414215{,}05$= 414.21KN

a.2 Roof slope

$A_{fr} = (24{,}871 + 20{,}92 + 17{,}86) \times 98 = 6239{,}05$m²

F_{fr}=6239.05x 0.01 x1400.5 = 87371.95N =87.37KN

So, the friction force is:

F_{fr} =87371.95+ $414215{,}05 = 501587{,}50$N=501.58KN

b. *Wind perpendicular to the long section:*

b.1 Vertical wall

$A_{fr} = 62 \times 8 = 496$

$F_{fr} = 496 \times 0{,}04 \times 1320{,}95 = 26207{,}68$

b.2 Roofing

$A_{fr} = 41{,}98 \times 98 = 4042{,}5$

$F_{fr} = 4042{,}5 \times 0{,}01 \times 1400{,}5 = 56615{,}21N$=56.61KN

So, the friction force is:

$F_{fr} = 26207{,}68 + 56615{,}21 = 82822{,}90N$=82.82KN

III.7 Over all action:

The force exerted by the wind $F_w$acting on a construction or a construction element can be determined by the vector summation of the forces $F_{w,}$, $F_{w,i}$and F_{fr}respectively given by: (According to RNVA 2013 Chap2, article 2.6.2) [6]

- External forces: $F_{w,\times} = C_d \times \sum w_e \times A_{ref}$

- Internal forces: $F_{w,} = \sum w_e \times A_{ref}$

- Friction force : $F_{fr} = C_{fr} \times q_p(Z_e) \times A_{ref}$

Or :

- w_e: is the external pressure exerted on the elementary surface of the heightZ_e given by the expression:

$$\boldsymbol{w_e = q_p(Z_e) \times C_{pe}}$$

- w_i : is the internal pressure exerted on the elementary surface of the height $Z_e$given by the expression:

$$w_e = q_p(Z_i) \times C_{pi}$$

A_{ref}: is the reference area of the elementary surface.

C_{fr}: is the coefficient of friction

A_{fr}: is the area of the exterior surface parallel to the wind.

Z_e, Z_i: respectively the reference heights of the external and internal pressures

a. Case of wind perpendicular to the gable (V1):

a.1 Determination of external force $F_{w,}$:

$$F_{w,} = C_d \times \sum w_e \times A_{ref} \text{With} \quad C_d = 1 \text{ and } \quad w_e = q_p(Z_e) \times C_{pe}$$

Table III.22:external pressure values W_e, corresponding to each zone

Area	q_p	C_{pe}	w_e
D	1320,95	+0.8	1056.76
E	1320,95	-0.3	-396,28
F	1400.5	-1.40	-1967.70
G	1400.5	-1.3	-1820.65
H	1400.5	-0.63	-889.32
I	1400.5	-0.53	-749.27

Table III.23:of external force values F_{we} corresponding to each zone

Element	Area	C_d	W_e (KN $/m^2$)	A_{ref}	Horizontal Components of F_{we}	Vertical Components of F_{we}(KN)
Walls.V	D	1	+1.06	784	+828.50	0
Walls.V	E		-0.39	784	-310.69	0
Roofing	F		-1.97	10,87	0	-21.41
Roofing	G		-1.82	107,57	0	-195.86
Roofing	H		-0.89	517,32	0	-460.07
Roofing	I		-0.75	5429.34	0	-4068.03
		F_{we}(Resultant)			+517.81	-4745.36

a.2 Determination of inner strength $F_{w,}$:

$$F_{w,} = \sum w_i \times A_{réf} \text{ With } w_i = q_{p(z_i)} \times C_{pi}$$

a.3 Calculation of internal pressure w_i:

Table III.24:values of internal pressure Wi corresponding to each zone

Area	q_p	C_{pi}	w_i
D	1320,95	-0.5	-0.66
E	1320,95	-0.5	-0.66
F	1400.5	-0.5	-0.70
G	1400.5	-0.5	-0.70
H	1400.5	-0.5	-0.70
I	1400.5	-0.5	-0.70

Table III.25:values of the internal force $F_{w,i}$ corresponding to each zone

Element	Area	w_i	$A_{réf}$	Horizontal Components of $F_{w,i}$	Vertical Components of $F_{w,i}$
Walls v	D	-0.66	784	-517.81	0
Walls v	E	-0.66	784	-517.81	0
Roofing	F	-0.70	10,87	0	-7.62
Roofing	G	-0.70	107,57	0	-75.33
Roofing	H	-0.70	517,32	0	-362.26
Roofing	I	-0.70	5429.34	0	-3801.89
	F_{wi}(Resultant)			-1035.62	-4247.10

b. Calculation of overall stability

b.1 Longitudinal stability (small into the wind):

The values of the internal and external forces and those of friction corresponding to the direction of the wind V1 and the horizontal and vertical resultants which are exerted on the construction are data in the table below:

Table III.26: The resulting force F_W, case V1

	Horizontal action (KN)	Vertical action (KN)
$F_{w.e}$	+517.81	0
$F_{w.e}$	0	-4745.36
$F_{w.i}$	-1035.62	0
$F_{w.i}$	0	-4247.10
F_{fr}	501,59	0
Resultant $F_{w.h}$	-16.22	0
Resultant $F_{w.v}$	0	-8992.46

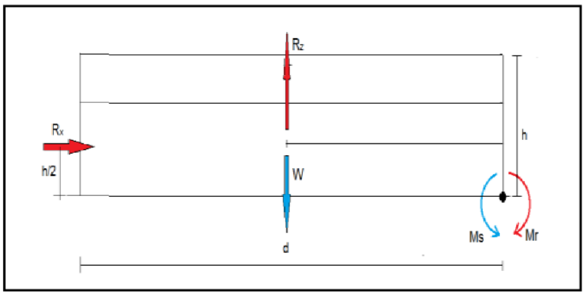


Figure III. 11: stability longitudinal

- ***Calculation of overturning moment***

$$Mr = (\ _{W.h} \times \frac{H}{2}) + (\ _{W.v} \times \frac{D}{2}) = (16.22\times \frac{10,43}{2}) + (8992.46 \times \frac{98}{2})$$

Mr = 440715.22 kN.m

- ***Calculation of the stabilizing moment (with an estimated self-weight of the hangar):***

Ms = (W × d/2)

With:

W: total surface weight of the shed (w = 0.5 KN/m2).

W = 0.5 × S_{sol}

W = 0.5 × 62 × 98

W = 3038 KN

Ms = $(3038\times \frac{98}{2})$ = 148862 kN.m

Ms=148862 kN.m < Mr=440715.22 kN.m ....longitudinal stability is checked

b.2 Case of wind perpendicular to the long section (V2):

b.2.1 Determination of external force $F_{w,}$:

$$F_{w,} = C_d \times \sum w_e \times A_{réf} \text{ with: } C_d = 1 \text{ and } w_e = q_p\ (z_e) \times \text{Cpe}$$

Table III.27: external pressure values w_e, corresponding to each zone

Area	q_P	C_{pe}	w_e(KN/m2)
D	1320,95	+0.8	1056.76
E	1320,95	-0.3	-396,28
F	1400.5	-1.18	-1652.59
G	1400.5	-0.94	-1316.47
H	1400.5	-0.41	-567.20
I	1400.5	-0.47	-658,24
J	1400.5	-0.86	-1204.43

Table III.28: external force values $F_{w,}$ corresponding to each zone

Element	Area	c_d	w_e	$A_{réf}$	Horizontal Components of $F_{w,e}$	Vertical Components of $F_{w,e}$
Walls.v	D	1	1.06	784	828.50	0
Walls.v	E		-0.40	784	-310.68	0
Roofing	F		-1.65	10,87	0	-17.98
Roofing	G		-1.32	215,50	0	-283.71
Roofing	H		-0.57	5667,14	0	-3214.42
Roofing	I		-0.66	273,26	0	-179.87
Roofing	J		-1.20	5667,14	0	-6825.68
			$F_{w,e}$(Resultant)		517.81	-10521.66

c. Determination of inner strength $F_{w,}$:

$$F_{w,} = \sum w_i \times A_{ref} \text{With: } w_i = \text{qp (zi)} \times \text{Cpi}$$

Table III.29: values of interior pressure Wi corresponding to each zone.

Area	q_p	C_{pi}	w_i
D	1320.95	+0.8	1056.76
E	1320.95	+0.8	156.76
F	1400.5	+0.8	1120.4
G	1400.5	+0.8	1120.4
H	1400.5	+0.8	1120.4
I	1400.5	+0.8	1120.4
J	1400.5	+0.8	1120.4

Table III.30: internal force values $F_{w,}$ corresponding to each zone

Element	Area	w_i	A_{ref}	Horizontal Components of $F_{w,i}$	Vertical Components of $F_{w,i}$
Walls.v	D	1.06	784	828.49	0
Walls.v	E	1.06	784	828.49	0
Roofing	F	1.12	10,87	0	12.18
Roofing	G	1.12	251,50	0	281.79
Roofing	H	1.12	5667,14	0	6349.46
Roofing	I	1.12	273,26	0	306.16
Roofing	J	1.12	5667,14	0	6349.46
			$F_{w,i}$ $(resultante)$	1656.99	13299.08

c.1 Calculation of overall stability:

c.1.1 Transverse stability (high into the wind):

The values of the internal and external forces and those of the friction corresponding to the direction of the wind V2 and the horizontal and vertical resultants which are exerted on the construction are given in the table below:

Table III.31: strength $F_w$resulting case V2

	horizontal Action [KN]	vertical Action [KN]
$F_{w,e}$	517.81	0
$F_{w,e}$	0	-10521.66
$F_{w,i}$	1656.99	0
$F_{w,i}$	0	13299.08
F_{fr}	82,823	0
Resultant$F_{w,h}$	2257.64	0
Resultant$F_{w,v}$	0	2777.42

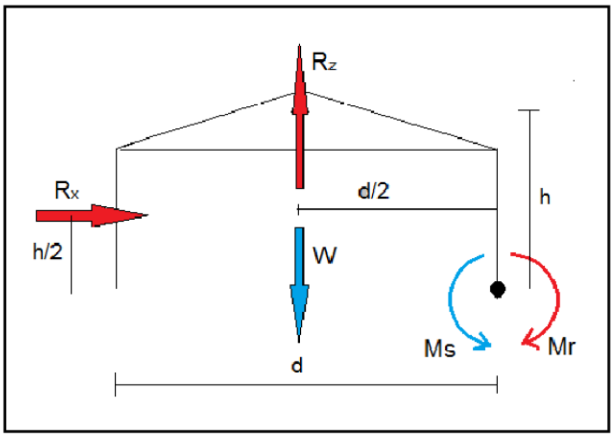


Figure III. 12: transverse stability

- ***Calculation of overturning moment***

Mr = $(\ _{w,h} \times \frac{h}{2})+(F_{w,v} \times \frac{d}{2})=(2257.6351\times \frac{10{,}43}{2}) + (2777.4236\times \frac{62}{2})$

Mr = 97873.6986 kN.m

- ***Calculation of the stabilizing moment (with an estimated self-weight of the hangar):***

Ms = $(W \times \frac{d}{2})$

With:

W: total surface weight of the shed (w = 0.5 N/ m^2).

W = 0.5 $\times S_{sol}$

$W = 0.5 \times 98 \times 62$

$W = 3038$ kN

Ms= $(3038 \times \frac{62}{2}) = 94178$ kN.m

Ms = 94178 kN.m < Mr = 97873.6986 kN…. transverse stability is not verified

III.7 Action of the seismic load:

The ADRAR province is classified by the Algerian seismic regulation RPA 2003 [1] as a zone with negligible seismicity, noted ZONE 0. So, a seismic justification is not necessary for this project.

III.8 Conclusion :

Our study area is located at ADRAR at a significant altitude so the actions of the winds have a great impact on the structure. After verification with the calculations of the actions of the wind on walls as well as on the roof, also by the calculation of the sand load, this used roof resists well to the dead and live loads of climatic effects.

Chapter IV: Numerical Analysis and Design

VI.1 Presentation of software and preliminary design

VI.1.1 Introduction

Steel sheds are part of discrete structures composed of bar elements assembled by bolting at nodes called nodes and subject to external forces, which are pressure due to wind and sand and the self-weight of the shed. Under the effect of these forces, the shed can deform, and internal stresses in each element are generated. The latter is entirely defined by the geometric characteristics of the current section (area, Young's modulus, etc.) and the geometry of the average fibre. The forces applied to each bar are schematised as point loads.

VI.1.2 Shed sizing method:

The calculation software currently on the market is based on the finite element method (FEM). Their fields of application are varied: structural calculations, thermal calculations, electromagnetic calculations, and fluid flows. Concerning calculations relating to structures, it is possible to deal with the majority of cases encountered in practice:

- Isotropic or anisotropic materials ;
- Elastic, elasto -plastic, and plastic behaviors;
- Calculations of thermal fields, stationary or not, thermoelastic coupling;
- Homogeneous or composite structures under static loading;
- Nonlinear behavior such as buckling;
- Dynamic analysis: frequencies, modes, forced vibrations under various excitations, transient vibrations.

It is possible to consider several types of finite element in the calculations:

Bar element,

Beam element,

Plate element: the present study , the bar element concerns us because all the bars of the structure in work either in tension or in compression.

IV.1.3 Presentation of ROBOT SA software:

The Robot system is CAD software intended to model, analyze and dimension different types of structures. ROBOT allows to create structures, calculate them, check the results obtained, and size the specific elements of the structure; the last step managed by ROBOT is the creation of the documentation for the calculated and sized structure.

IV.1.4 The main characteristics of the Robot software:

- Definition of the structure carried out in fully graphical mode in the editor designed for this purpose (you can also open a file in DXF format and import the geometry of a structure defined in another CAD/CAD software).
- Possibility of graphic presentation of the structure studied and of representing on the screen the different types of calculation results (efforts, displacements, simultaneous work in several open windows etc.),
- Possibility of calculating (dimensioning) one structure and simultaneously studying another (multithreaded architecture);
- Possibility of carrying out static and dynamic analysis of the structure,
- Possibility of affecting the type of bars when defining the structure model and not only in the business modules.
- Ability to freely compose prints (calculation notes, screenshots, print composition, copying objects to other software).

The Robot system brings together several parts (modules) specialised in each of the stages of the study of the structure (creation of the structural model, calculations of the structure, sizing). The modules work in the same environment.

IV.1.5 Presentation of the software and its environment:

It brings together several modules, among which we can cite:

- The study of a hull,
- The study of a spatial lattice,

- The study of a space gantry, etc... This software integrates the LIMITED STATES calculation method. It takes into account the regulatory safety coefficients according to several insecurity factors with regard to:
- On the one hand, the ultimate limit state (ULS) corresponds to the ruin of one of the elements of the structure.
- On the other hand, the service limit state (ELS) corresponds to the limit state of cracking and deformation.

The following figures show the AUTODESK ROBOT STRUCTURAL ANALYSIS Professional environment.

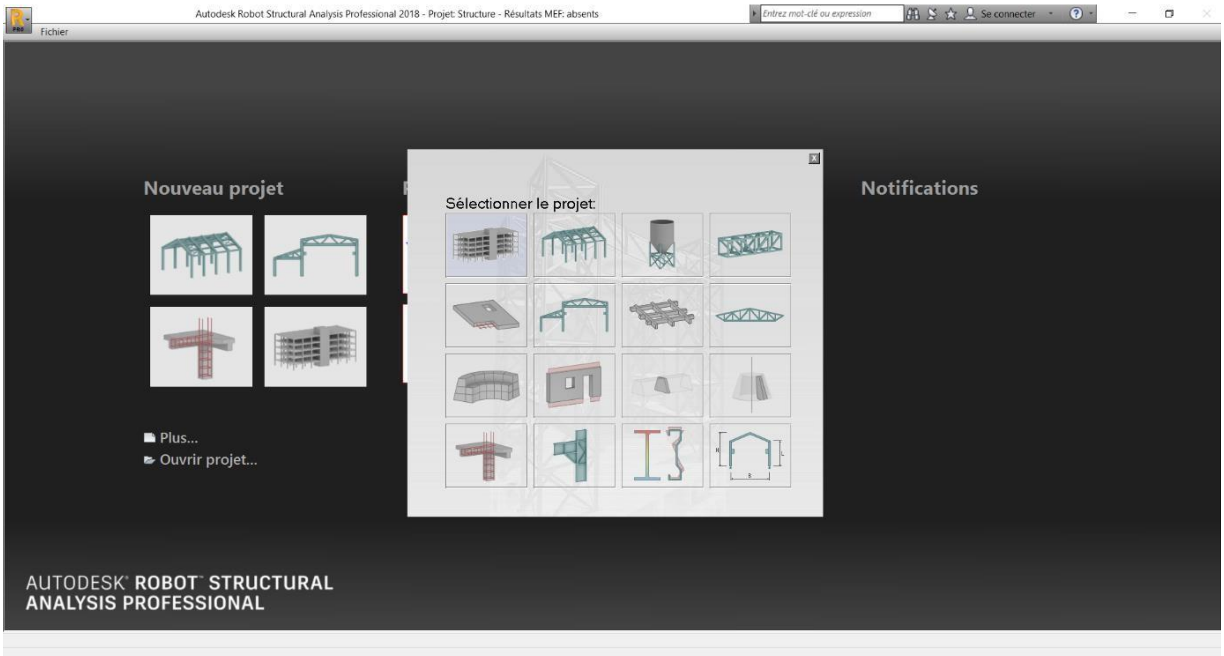


Figure IV.1: ROBOT structural analysis Professional

IV.1.6 Steps of modelling on ROBOT:

- The definition of construction lines.
- The selection of standards.
- The definition of the structure bars.
- The definition of supports.

After completing the modelling, we moved on to defining the different calculated loads.

- The definition of load cases.
- The definition of loads for the defined load cases.
- The definition of climatic loads on the hangar.

IV.1.7 Modelling the structure and drawing the frame

The modelling of the metal frame conducted in ROBOT is shown in the following figure:

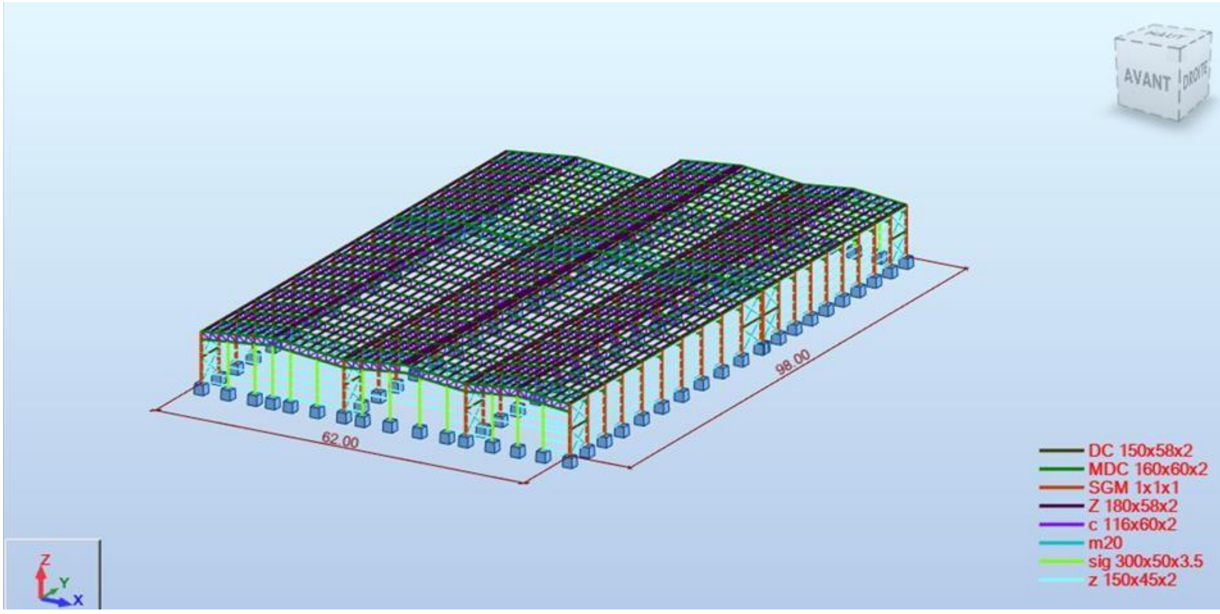


Figure IV.2 : 3D Model of the cold storage hangar.

- **Load Cases:**

The steel frame experiences three types of loads: dead loads, which include the weight of the framework, roof, and cladding; live loads, which consist of the load to be stored, the rolling load of the cart, and maintenance; and wind load, which is automatically generated by the software.

IV.1.8 Load combinations:

There are two ways to do this operation: manual and automatic. For our work, we chose the automatic combination.

Here is a table that lists all the current loads:

Table IV. 1: load combinations

Combinaison	Nom	Type d'analyse	Type de la	Nature du cas	Définition
7 (C)	COMB1	Combinaison lin	ELU	Structurelle	(1+2)*1.35+3*1.50+6*0.90
8 (C)	COMB2	Combinaison lin	ELU	Structurelle	(1+2)*1.35+(3+4)*1.50
9 (C)	COMB3	Combinaison lin	ELU	Structurelle	(1+2)*1.00+3*1.50+6*0.90
10 (C)	COMB4	Combinaison lin	ELU	Structurelle	(1+2)*1.00+(3+4)*1.50
11 (C)	COMB5	Combinaison lin	ELU	Structurelle	(1+2)*1.35+3*1.05+6*1.50
12 (C)	COMB6	Combinaison lin	ELU	Structurelle	(1+2)*1.35+6*1.50
13 (C)	COMB7	Combinaison lin	ELU	Structurelle	(1+2)*1.00+3*1.05+6*1.50
14 (C)	COMB8	Combinaison lin	ELU	Structurelle	(1+2)*1.00+6*1.50
15 (C)	COMB9	Combinaison lin	ELS	Structurelle	(1+2+3)*1.00+6*0.60
16 (C)	COMB10	Combinaison lin	ELS	Structurelle	(1+2+3+4)*1.00
17 (C)	COMB11	Combinaison lin	ELS	Structurelle	(1+2+6)*1.00+3*0.70
18 (C)	COMB12	Combinaison lin	ELS	Structurelle	(1+2+6)*1.00

IV.1.9 Static analysis and results:

- After having carried out the modelling of the pylon, it is asked to carry out the calculations.
- The results obtained by the ROBOT software show the efforts and stresses that are expressed in the local reference frame and the displacements in the global reference frame.

IV.1.10 Sizing :

- **Creation of bar types:**

Column:

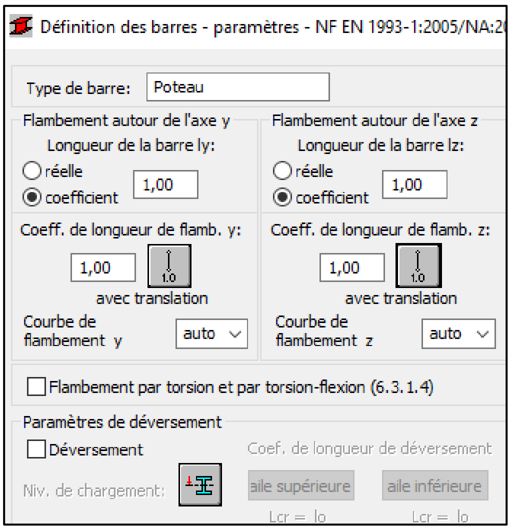


Figure IV.3 : Column settings

Purlins:

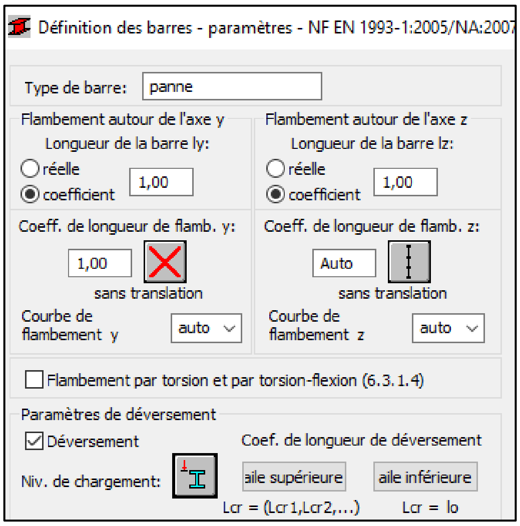


Figure IV.4 : Purlin settings

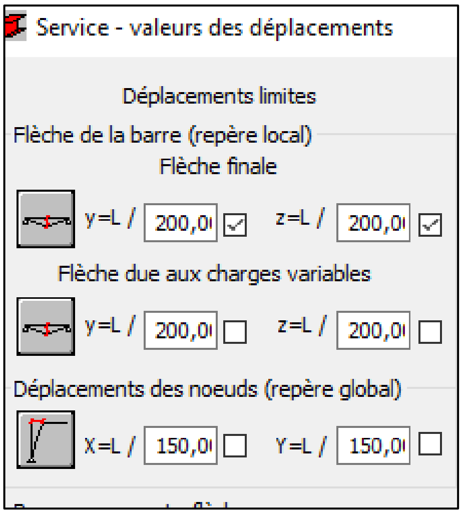


Figure IV.5 : Deflection settings

IV.1.11 Creation of families

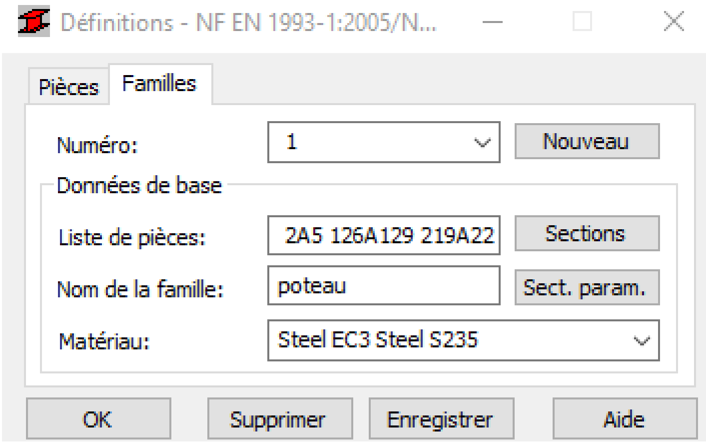


Figure IV.6 : Creation of families

The same procedures will be applied to the other elements of the structure.

IV.1.12 Geometrical characteristics of different profiles:

Table IV. 2: Characteristics of steel profiles.

Nom de la section	Liste des barres	AX [cm2]	AY [cm2]	AZ [cm2]	IX [cm4]	IY [cm4]	IZ [cm4]
DC 150x58x2	2998A3033 3440 3530	11,47	4,58	6,17	0,41	397,86	87,61
DC 160x60x2	6A17 130A141 223A23	12,27	0,0	0,0	0,20	782,71	482,02
Z 180x58x2	1986A2007 2011A212	6,33	5,50	3,96	0,09	328,46	18,25
c 116x60x2	18A47 77A84 87A125	5,24	2,32	2,53	0,08	113,55	27,36
m20	3262 3263 3266A3269	3,14	2,65	2,65	1,57	0,79	0,79
sig 300x50x3.5	3651 3655A3662 3665	29,35	8,29	18,74	1245,89	3555,55	654,21
sig 350x44x4	2A5 126A129 219A222	46,33	5,12	26,81	3347,07	7935,41	1717,98
z 150x45x2	3034A3261 3666 3671	5,22	4,55	3,26	0,08	183,06	9,56

IV.1.13 Verification of families:

- **ULS :**

Table IV. 3: verification of families under ULS combination.

Pièce	Profil	Matériau	Lay	Laz	Ratio	Cas
Famille : 1 poteau						
405	SGM 1x1x1	Steel	61.12	131.37	0.32	11 COMB5
Famille : 2 potelet						
4345	sig 300x50x3.5	Steel	72.68	169.44	0.07	11 COMB5
Famille : 3 panne						
2488	Z 180x58x2	Steel	80.52	112.71	0.85	12 COMB6
Famille : 4 Membrure Sup et inf						
877	MDC	Steel	150.57	192.44	0.36	8 COMB2
Famille : 6 Contreventement						
3273	m20	Steel	1087.13	1087.13	0.47	8 COMB2
Famille : 7 sablière						
3010	DC 150x58x2	Steel	98.48	69.25	0.18	11 COMB5
Famille : 8 palée stabilité						
3566	DC 150x58x2	Steel	78.95	168.25	0.07	11 COMB5
Famille : 9 lisses						
3095	z 150x45x2	Steel	97.99	141.47	0.55	14 COMB8
Famille : 10 montant et diagonale ferme						
902	c 116x60x2	Steel	20.62	42.01	0.34	7 COMB1

- **SLS:**

Table IV. 4: verification of families under SLS combination.

Pièce	Profil	Matériau	Ratio(uy)	Cas (uy)	Ratio(uz)	Cas (uz)
Famille : 1 poteau						
501	SGM 1x1x1	S 235	-	-	-	-
Famille : 2 potelet						
3665	sig 300x50x3.5	S 235	-	-	-	-
Famille : 3 panne						
4427	Z 180x58x2	S 235	0.24	1 PERM1	0.03	1 PERM1
Famille : 7 sablière						
3026	DC 150x58x2	S 235	0.00	1 PERM1	0.01	1 PERM1
Famille : 8 palée stabilité						
4380	DC 150x58x2	S 235	0.00	1 PERM1	0.01	1 PERM1
Famille : 9 lisses						
3093	z 150x45x2	S 235	0.98	1 PERM1	0.01	1 PERM1

We notice that all profiles' families are verified against ULS and SLS combinations.

IV.2 Design of structural elements

IV.2.1 Introduction

Having defined this structure's characteristics, we will focus on dimensioning and verifying the hangar supporting elements in this chapter. Robot Structural Analysis software is used to study the elements of our structure. This software uses the finite element analysis method to study planar and spatial structures such as Lattices, gantries, mixed structures, etc.

The calculation makes it possible to examine all the loads applied to the structure taking into account the different combinations and provides results with better precision.

IV.2.2 Definition of cold rolled profiles:

Cold-rolled profiles are steel parts that have been formed from metal sheets or strips by a cold-forming process. Unlike hot rolling, where the metal is heated to high temperatures, cold rolling is done at room temperature or slightly lower temperatures. This process achieves specific mechanical properties and high dimensional accuracy

Cold rolled profiles can take different shapes, such as Z, C, Σ and Ω sections

These profiles offer several advantages, including excellent dimensional accuracy, a high-quality surface finish, precise mechanical properties, ease of assembly, and corrosion resistance.

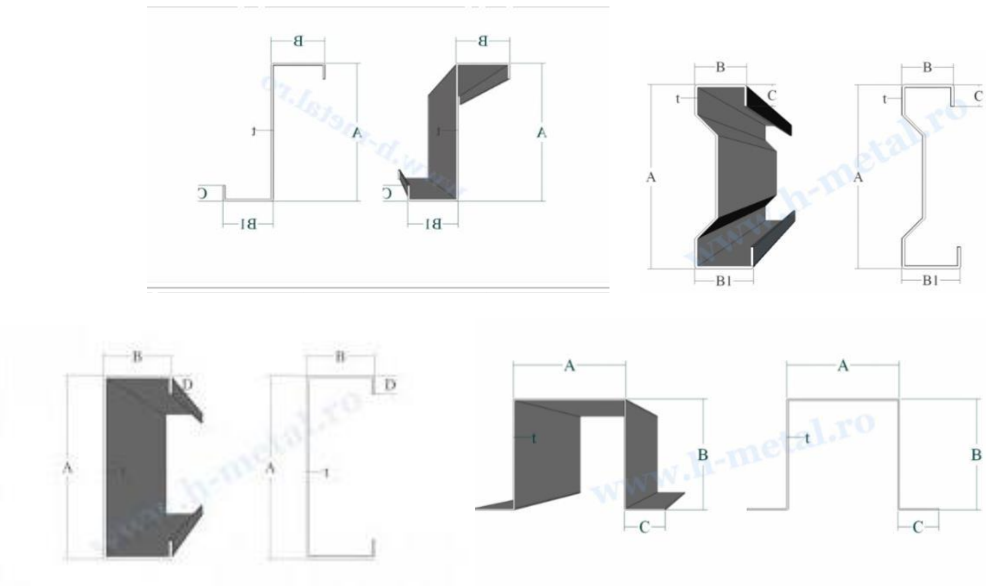


Figure IV.7: cold rolled profiles

IV.2.2.1 Z-shaped profile:

“Z” shaped metal profiles are structural elements that have a cross-section shaped like the letter “Z”. These profiles are formed using cold rolling or cold bending techniques on metal sheets or metal strips.

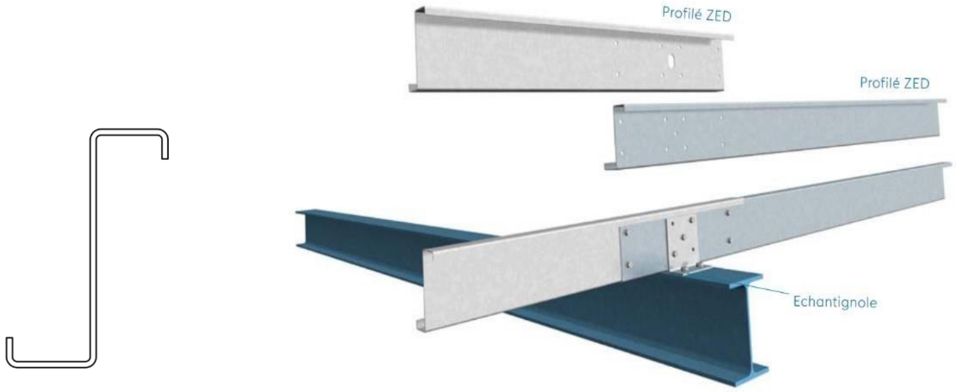


Figure IV.8: Z profiles [9]

Here are some characteristics and uses of metal “Z” profiles:

a. Features :

- "Z" shaped cross section with two parallel wings connected by a central part.
- The wings are usually equal or different sizes depending on the design specifications.
- The central part can vary in thickness and width depending on strength and rigidity requirements.

b. Uses:

- Metal "Z" profiles are often used as beams or uprights in the construction of lightweight structures or in applications where strength and rigidity are required.
- They are commonly used in metal building systems to form wall frames, roof supports, struts, etc.
- In the energy sector, "Z" profiles can be used to construct electrical transmission pylons or support structures for solar panels.
- They can also be used in storage applications, industrial shelving, machine and equipment frames, etc.

c. *Benefits :*

- "Z" profiles offer good resistance to bending and compression.
- Their "Z" shaped design allows for efficient load distribution and optimal use of the material.
- They are generally lightweight while providing adequate strength, making them suitable for various applications where weight is critical.

IV.2.2.2 C-shaped profiles:

"C"-shaped metal profiles are structural elements having a cross-section in the shape of the letter "C". These profiles are commonly used in various construction and engineering applications.

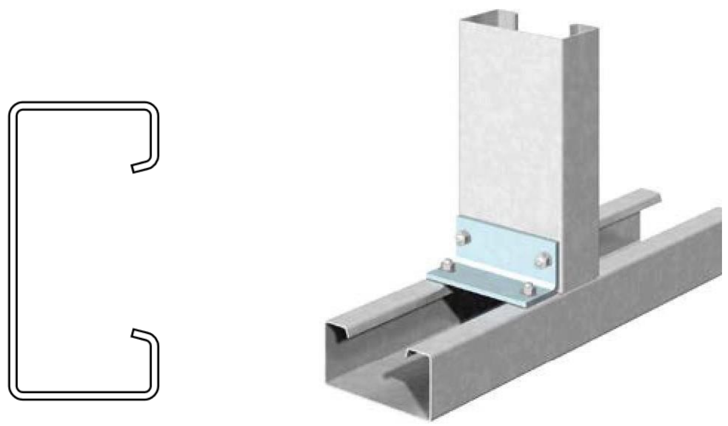


Figure IV.9 :C profiles [9]

Here is some information about these profiles:

a. *Features :*

- "C" shaped profiles generally consist of two parallel wings connected by an open central part.
- Wings can be equal or different heights, depending on design requirements.
- The open center section allows for flexibility when installing or assembling structures.

b. Uses:

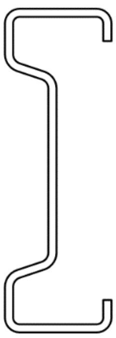
- "C" shaped channels are often used as studs or rails in lightweight construction systems, such as drywall, suspended ceilings and wall framing.
- They can also be used as beams or supporting elements in applications where a "C" shaped section is more suitable than other profile shapes.
- In the industrial sector, "C" profiles are used to construct supports for machines, shelves, storage racks and other equipment.

c. Benefits :

- "C" channels offer good resistance to bending and compression, making them suitable for various supporting applications.
- Their open design facilitates the installation of fasteners and allows the passage of cables, pipes or other elements through the central part of the profile.
- They are generally lightweight while providing adequate strength, making them easy to handle and install on the job site.

IV.2.2.3 SIGMA shaped profiles:

"Sigma" shaped metal profiles are structural elements having a cross-section that resembles the Greek letter " Σ " (sigma). These profiles are often used in applications requiring high resistance to bending and torsion.



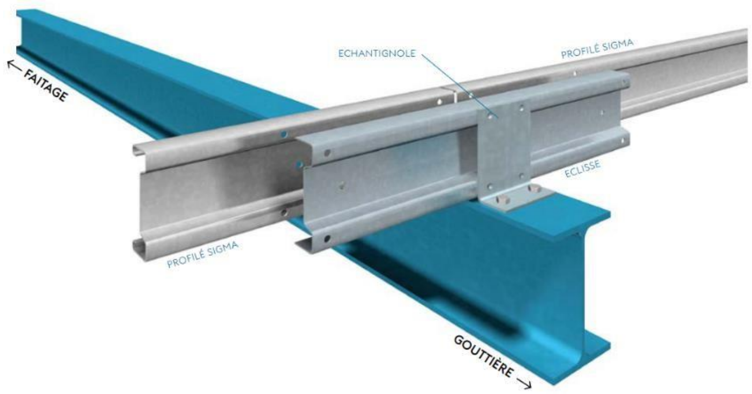


Figure IV.10 : Sigma profiles [9]

Here is some information about these profiles:

a. Features :

- "Sigma" shaped profiles generally consist of two parallel wings connected by a central “S” shaped part.

- Wings can be equal or different sizes, depending on design specifications and strength requirements.
- The central "S" shaped part helps to reinforce the structure by distributing loads efficiently.

b. Uses:

“Sigma” shaped profiles are widely used in applications requiring high strength, such as heavy supporting structures, bridges, cranes and other civil engineering structures.

They are also used in the maritime industry to construct ship hulls, offshore platforms, and other marine structures requiring high strength. In the field of industrial construction, "Sigma" profiles can be used as beams or uprights in support systems for heavy equipment, warehouses and storage structures.

c. Benefits :

- “Sigma” profiles offer excellent resistance to bending and torsion, making them suitable for high loads and harsh environmental conditions.
- Their “S” shaped design allows for efficient load distribution, which helps improve the stability and durability of structures.
- They are available in various sizes and thicknesses to meet the specific needs of construction and engineering projects.

In summary, cold-rolled metal profiles are robust and versatile structural elements widely used in construction, civil engineering, the marine industry, and other applications where strength and durability are paramount.

IV.2.3 Summary of loading:

IV.2.3.1 Dead loads:

- Self-weight of the structure and a dead load G (sandwich pane, cables, equipment, etc)

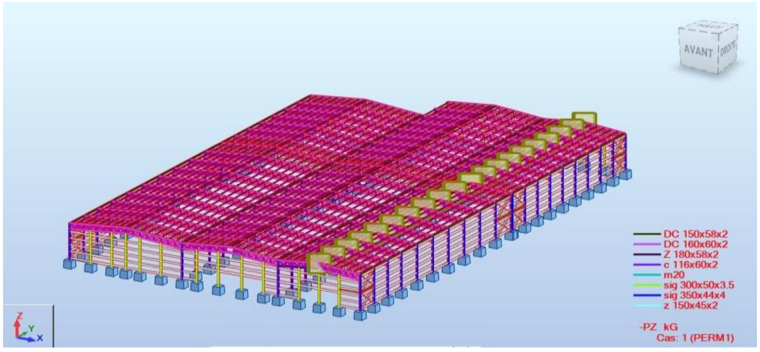


Figure IV.11: Dead loads definition

IV.2.3.2 The load of the sand:

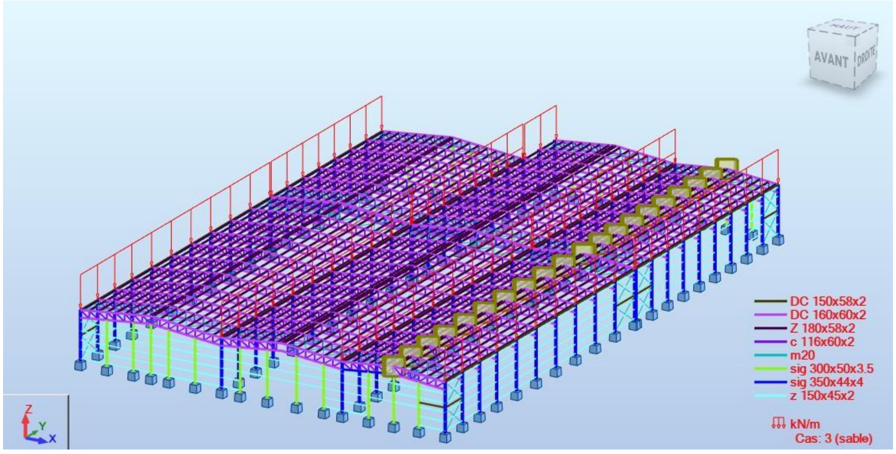


Figure IV.12: Sand load definition

IV.2.3.3 Wind load:

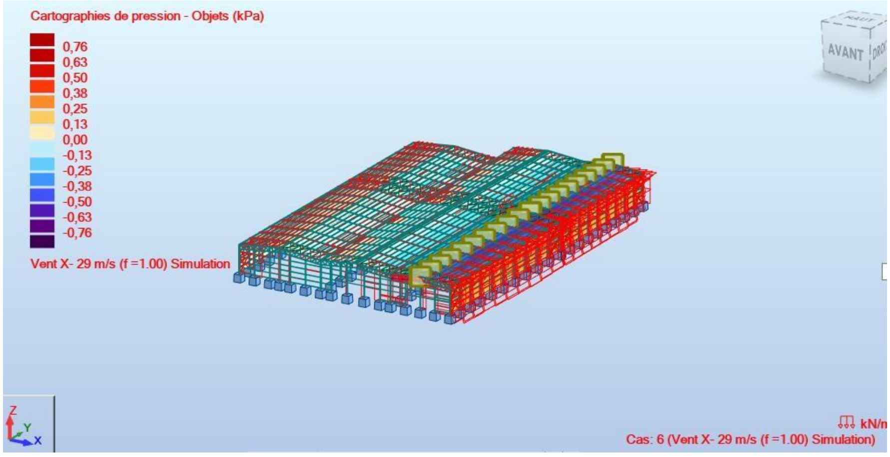


Figure IV. 1: wind action

IV.2.4 Study of secondary elements:

The secondary elements represent the framework necessary to support the roofing and the cladding. In this part, we are interested in defining the profiles which will have to resist the various stresses according to the EUROCODE3 [5] code. The principle of verification requires resistance and stability; the profiles concerned by this study are:

-Purlins

-The vines

-The cladding rails

-The posts

-The chain beam

IV.2.4.1 Purlins:

The purlins are beams intended to support the roofing and transmit the loads and overloads applied to it to the crosspiece or to the truss. They are arranged parallel to the ridge line and calculated in deflected bending under the effect of dead , live , and climatic loads.

They are made either in a hot-formed profile in (I), or in (U), or in a cold-formed profile in (Z), (U), (Σ), and cold-rolled Z profiles will be used.

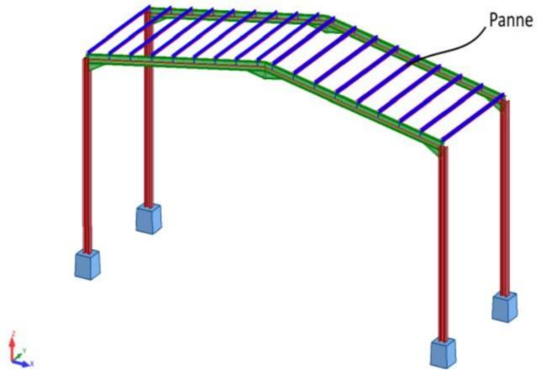


Figure IV. 2: Purlins

IV.2.4.1.1 Calculation principle

- The dead and snow loads are applied in the direction of gravity.
- The wind acts perpendicular to the face of the elements (axis of great inertia).
- We consider the most unfavorable combination.
- The purlins must be checked for resistance to bending, shearing, and overturning

IV.2.4.1.2 The characteristics of the purlins:

- The span between purlin axes is d = 1,55m (the space between 2 purlins).
- The angle of the slopes of the hall is equal to $\alpha = 11.5°$.
- The purlins are made of S235 steel, with the following characteristics:
- $f_y = 23.5$ daN/mm² (yield strength of steel)
- $E = 21000$ daN/mm² (Young modulus of steel)

IV.2.4.1.3 Evaluation of dead and live loads:

- ***Dead loads:***

Sandwich panel weight =0.15 kg/m²

Cables and equipment weight =0.05 kg/m²

- ***Climate and live loads:***

- Sand load, according to RNV 2013 → ADRAR : q1=0.3Kn/ml ; q2=0.4Kn/ml

  Wind load (we take the most unfavorable case of the wind:

- Maintenance load, according to DTR BC.2.2, a concentrated load of 1 kN is applied at L/3 and 2L/3 of the purlins.

IV.2.4.1.4 Sizing:

- ULS :

Famille : 3 panne	
2488	OK Z 180x58x2

- SLS:

Famille : 3 panne	
4427	OK Z 180x58x2

We adopt the Z180 profile as roofing purlins.

IV.2.4.1.5 VERIFICATION

THE CALCULATION NOTE

- **ULU :**

STANDARD: NF EN 1993-1:2005/NA:2007/AC:2009, Eurocode 3: Design of steel structures.

TYPE OF ANALYSIS: Family Verification

FAMILY : 3 breakdown

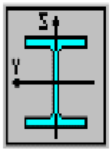
PIECE : 2488

POINT : 4

COORDINATE : x = 0.50 L = 2.90 m

LOADINGS: ***Decisive load case:*** **12 COMB6 (1+2) *1.35+6*1.50**

MATERIAL : Steel (S235) fy = 235.00 MPa



SECTION PARAMETERS : Z 180x58x2

h=20.3 cm g M0=1.00 g M1=1.00

b=6.9 cm Ay=5.50 cm2 Az =3.96 cm2 Ax=6.33 cm2

tw =0.0 cm Iy =328.46 cm4 Iz =18.25 cm4 Ix =0.09 cm4 tf =0.0 cm Wply =41.07 cm3 Wplz =9.13 cm3

INTERNAL EFFORTS AND ULTIMATE RESISTANCES:

N,Ed = -0.49 kN My,Ed = 2.13 kN *m Mz,Ed = 1.34 kN *m

Nt,Rd = 148.76 kN My,pl,Rd = 9.65 kN *m Mz,pl,Rd = 2.15 kN *m

My,c,Rd = 9.65 kN *m Mz,c,Rd = 2.15 kN *m

MN,y,Rd = 9.65 kN*m MN,z,Rd = 2.15 kN*m

Tt,Ed = 0.00 kN*m

Section class = 1

SPILL PARAMETERS:

BUCKING PARAMETERS:

in y: in z:

VERIFICATION FORMULAS:

Checking the resistance of the section:

N,Ed / Nt,Rd = 0.00 < 1.00 (6.2.3.(1))

My,Ed / MN,y,Rd = 0.22 < 1.00 (6.2.9.1.(2))

Mz,Ed / MN,z,Rd = 0.63 < 1.00 (6.2.9.1.(2))

(My,Ed / MN,y,Rd)^ 1.00 + (Mz,Ed / MN,z,Rd)^1.00 = 0.85 < 1.00 (6.2.9.1.(6))

Correct profile!!!

- **ULS**

STANDARD : NF EN 1993-1:2005/NA:2007/AC:2009, Eurocode 3: Design of steel structures.

ANALYSIS TYPE: Family Verification

FAMILY : 3 breakdown

PIECE: 4427 ITEM: CONTACT INFORMATION:

SECTION PARAMETERS : Z 180x58x2

height =20.3 cm

bf=6.9 cm Ay=5.50 cm2 Az=3.96 cm2 Ax =6.33 cm2

ea =0.0 cm Iy =328.46 cm4 Iz =18.25 cm4 Ix =0.09 cm4

es=0.0 cm Wely =32.30 cm3 Welz =5.26 cm3

LIMITED TRAVEL

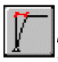
Arrows (LOCAL MARK):

uy = 0.7 cm < uy max = L/200.00 = 2.9 cm Checked

Decisive load case: 1 PERM1

uz = 0.1 cm < uz max = L/200.00 = 2.9 cm Checked

Decisive load case : 1 PERM1

*Travel (OVERALL REFERENCE):* Not analyzed

Correct profile!!!

IV.2.4.1.6 Conclusion:

The Z180 is verified and adopted as a roof purlin.

IV.2.4.2 The lianas:

The straps are tie rods that work in traction arranged at the mid-span of the purlins perpendicular to the latter in the plane of the roof. They are generally formed as round bars or small angles. Their main role is to avoid lateral deformation of the purlins and limit the spill length and lateral buckling for the compressed parts. We assumed a round bar of 10 mm in diameter for the lugs.

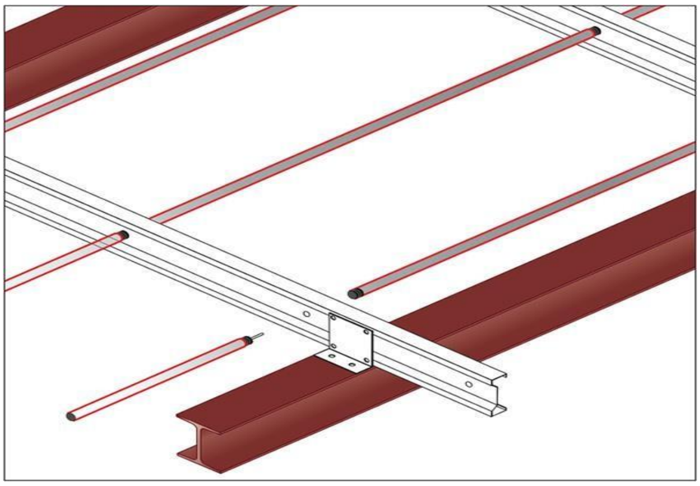


Figure IV. 3: lianas [10]

IV.2.4.2 .1 Calculation of the cross-section:

Nsd ≤ Npl.Rd “Formula 5.16 – Page 5-55 – EC3” [5]

For safety and practical reasons, we opt for a round bar with a diameter Ø = 10 mm

IV.2.4.3 The cladding rails:

Cladding rails are generally made of beams (IPE, UAP, HEA) or thin folded profiles. They are placed horizontally on the "long-pan" gantry posts and also on the intermediate "gable" posts. In addition to their own weight and the weight of the cladding, they are subject to the actions of the wind transmitted by the latter. They must be checked for bending, overturning, and cutting force.

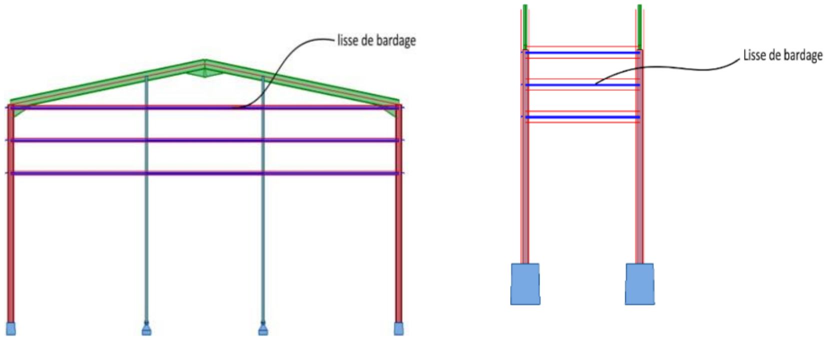


Figure IV. 4: Cladding rails

IV.2.4.3.1 Evaluation of loads and overloads:

- ***Dead loads (G):***
  - the weight of the rail and its corresponding cladding.
  - Possible additional loads.
  - Cladding weight : kg/m2
  - weight of the boom (Estimated)
- ***Climatic loads:***

We take the most unfavorable case of the wind obtained from wind calculations (see Chapter III)

IV.2.4.3.2 SIZING:

- **ULS :**

Famille : 9 lisses

3095	OK	z 150x45x2

- **SLS:**

Famille : 9 lisses

3093	OK	z 150x45x2

IV.2.4.3.3 VERIFICATION :

- *ULS*

THE CALCULATION NOTE

STANDARD: NF EN 1993-1:2005/NA:2007/AC:2009, Eurocode 3: Design of steel structures.

TYPE OF ANALYSIS : Family verification

FAMILY : 9 cladding rails

PIECE : 3095

POINT : 4

CONTACT INFORMATION: x = 0.50, L =2.90 m

LOADINGS: Decisive load case: 14 COMB8 (1+2)*1.00+6*1.50

MATERIAL: Steel (S235) fy = 235.00 MPa

SECTION PARAMETERS: Z150x45x2

h=16.7 cm gM0=1.00 gM1=1.00

b=5.4 cm Ay=4.55 cm2 Az =3.26 cm2 Ax=5.22 cm2

tw =0.0 cm Iy =183.06 cm4 Iz =9.56 cm4 Ix =0.08 cm4

tf =0.0 cm Wply =27.89 cm3 Wplz =6.02 cm3

INTERNAL EFFORTS AND ULTIMATE RESISTANCES:

N,Ed = -0.05 kN My,Ed = 0.29 kN *m Mz,Ed = 0.71 kN *m

Nt,Rd = 122.79 kN My,pl,Rd = 6.55 kN *m Mz,pl,Rd = 1.42 kN *m

My,c,Rd = 6.55 kN *m Mz,c,Rd = 1.42 kN *m

MN,y,Rd = 6.55 kN*m MN,z,Rd = 1.42 kN*m

Tt,Ed = -0.00 kN*m

Section class = 1

SPILL PARAMETERS:

BUCKING PARAMETERS:

in y: in z:

VERIFICATION FORMULAS:

Section resistance check:

N,Ed / Nt,Rd = 0.00 < 1.00 (6.2.3.(1))

My,Ed / MN,y,Rd = 0.04 < 1.00 (6.2.9.1.(2))

Mz,Ed / MN,z,Rd = 0.50 < 1.00 (6.2.9.1.(2))

(My,Ed / MN,y,Rd)^ 1.00 + (Mz,Ed / MN,z,Rd)^1.00 = 0.55 < 1.00 (6.2.9.1.(6))

Correct profile!!!

- ***SLS:***

STANDARD : NF EN 1993-1:2005/NA:2007/AC:2009, Eurocode 3: Design of steel structures.

ANALYSIS TYPE: Family Verification

FAMILY: 9 cladding rails

PIECE: 3093 **POINT: COORDINATE:**

SECTION PARAMETERS : z 150x45x2

height =16.7 cm

bf=5.4 cm Ay=4.55 cm2 Az=3.26 cm2 Ax =5.22 cm2

ea =0.0 cm Iy =183.06 cm4 Iz =9.56 cm4 Ix =0.08 cm4

es=0.0 cm Wely =21.93 cm3 Welz =3.51 cm3

LIMITED TRAVEL

Arrows (LOCAL MARK):

uy = 2.8 cm < uy max = L/200.00 = 2.9 cm Checked

Decisive load case: 1 PERM1

uz = 0.0 cm < uz max = L/200.00 = 2.9 cm Checked

Decisive load case: 1 PERM1

Travel (OVERALL REFERENCE): Not analyzed

Correct profile!!!

IV.2.4.3.4 Conclusion:

Z profile 150 is verified and adopted for cladding rails.

IV.2.4.4 The posts:

Posts are structural elements that are attached to gable walls in order to reduce the distance between posts and provide support for the insulating covering. They serve two purposes:

- They help to mitigate the effects of compound flexion, which occurs when the gable wall bends due to wind forces.
- They provide support by bearing the weight of the rails, cladding, and the post itself, which results in compression.

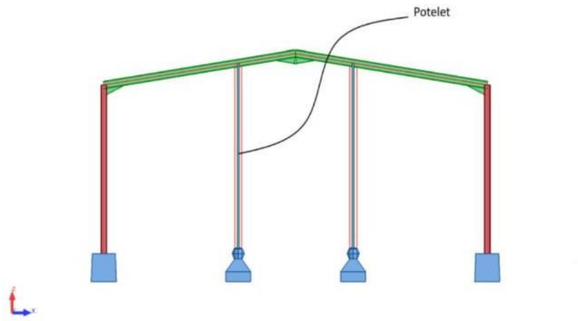


Figure IV. 5: The posts

IV.2.4.4.1 SIZING :

- **ULU:**

Famille : 2 potelet

4345	OK sig 300x50x3.5

IV.2.4.4.2 VERIFICATION :

- **ULS:**

THE CALCULATION NOTE

STANDARD : NF EN 1993-1:2005/NA:2007/AC:2009, Eurocode 3: Design of steel structures.

TYPE OF ANALYSIS: Family Verification

FAMILY: 2 posts

PIECE: 4345 potelet01_4345

POINT: 7

COORDINATE: x = 1.00L =8.00 m

LOADINGS: ***Decisive load case:*** **11 COMB5 (1+2)*1.35+3*1.05+6*1.50**

MATERIAL: Steel (S235) fy = 235.00 MPa
SECTION PARAMETERS : sig 300x50x3.5

h=30.0 cm gM0=1.00 gM1=1.00

b=14.0 cm Ay=8.29 cm2 Az =18.74 cm2 Ax=29.35 cm2

tw =0.0 cm Iy =3555.55 cm4 Iz =654.21 cm4 Ix =1245.89 cm4

tf =0.0 cm Wply =290.70 cm3 Wplz =128.74 cm3

INTERNAL EFFORTS AND ULTIMATE RESISTANCES:

N,Ed = 8.72 kN My,Ed = -0.66 kN *m Mz,Ed = 0.70 kN *m Vy,Ed = -0.21 kN

Nc,Rd = 689.66 kN My,Ed,max = 0.74 kN *m Mz,Ed,max = 0.70 kN *m Vy,T,Rd = 112.49 kN

Nb,Rd = 194.51 kN My,c,Rd = 68.31 kN *m Mz,c,Rd = 30.25 kN *m Vz,Ed = -0.27 kN

MN,y,Rd = 68.30 kN*m MN,z,Rd = 30.25 kN*m Vz,T,Rd = 254.32 kN

Tt,Ed = -0.03kN*m

Section class = 1

SPILL PARAMETERS:

BUCKLING PARAMETERS:

in y:   in z:

Ly = 8.00 m Lam_y = 0.77m Lz = 8.00 m Lam_z = 1.80

Lcr,y = 8.00 m Xy = 0.87m Lcr,z = 8.00 m Xz = 0.28

Lamy = 72.68 kzy = 0.57m Lamz = 169.44 kzz = 1.00

VERIFICATION FORMULAS:

Checking the resistance of the section:

N,Ed / Nc,Rd = 0.01 < 1.00 (6.2.4.(1))

My,Ed / MN,y,Rd = 0.01 < 1.00 (6.2.9.1.(2))

Mz,Ed / MN,z,Rd = 0.02 < 1.00 (6.2.9.1.(2))

(My,Ed / MN,y,Rd)^ 1.00 + (Mz,Ed / MN,z,Rd)^1.00 = 0.03 < 1.00 (6.2.9.1.(6))

Vy,Ed / Vy,c,Rd = 0.00 < 1.00 (6.2.6.(1))

Vz,Ed / Vz,c,Rd = 0.00 < 1.00 (6.2.6.(1))

Checking the overall stability of the bar:

Lambda,y = 72.68 < Lambda,max = 210.00 Lambda,z = 169.44 < Lambda,max = 210.00 STABLE

N , Ed / (1.00 (6.3.3.(4))

N , Ed / (1.00 (6.3.3.(4))

Correct profile!!

IV.2.4.4.3 Conclusion:

Σ 300 profile has been checked and is adopted for the posts.

IV.2.4.5 Chain beam (sand pit):

The chain beams serve as the link between the various porticos of the structure. They are primarily subjected to normal forces but also experience bending moments caused by their own weight and wind loads.

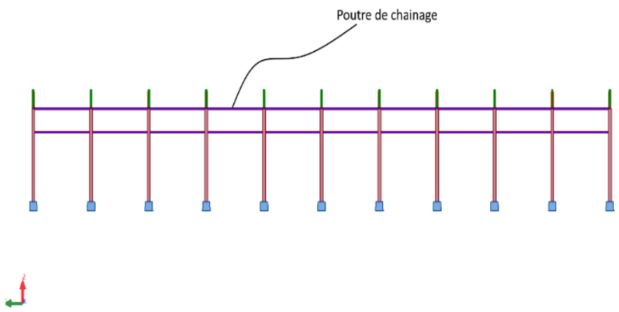


Figure IV. 6: Chain beam

IV.2.4.5.1 SIZING:

- **ULS :**

Famille : 7 sablière		
3010	OK	DC 150x58x2

- **SLS** :

Famille : 7 sablière		
3026	OK	DC 150x58x2

IV.2.4.5.2 VERIFICATION

THE CALCULATION NOTE

- **ULU:**

STANDARD: NF EN 1993-1:2005/NA:2007/AC:2009, Eurocode 3: Design of steel structures.

ANALYSIS TYPE: Family Verification

FAMILY: 7 sand pit

ROOM: 3010 POINT: 1 COORDINATE: x = 0.00 L =0.00 m

LOADINGS: ***Decisive load case:*** **11 COMB5 (1+2)*1.35+3*1.05+6*1.50**

MATERIAL: Steel (S235) fy = 235.00 MPa

SECTION SETTINGS: DC 150 x58x2

h=15.0 cm gM0=1.00 gM1=1.00

b=11.6 cm Ay=4.58 cm2 Az =6.17 cm2 Ax=11.47 cm2

tw =0.0 cm Iy =397.86 cm4 Iz =87.61 cm4 Ix =0.41 cm4

tf =0.0 cm Wply =62.00 cm3 Wplz =20.29 cm3

INTERNAL EFFORTS AND ULTIMATE RESISTANCES:

N,Ed = 0.07 kN My,Ed = -1.18 kN *m Mz,Ed = -0.46 kN *m Vy,Ed = -0.46 kN

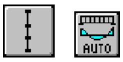
Nc,Rd = 269.55 kN My,Ed,max = -1.18 kN *m Mz,Ed,max = -0.46 kN *m Vy,T,Rd = 62.11 kN

Nb,Rd = 237.65 kN My,c,Rd = 14.57 kN *m Mz,c,Rd = 4.77 kN *m Vz,Ed = 1.20 kN

MN,y,Rd = 14.57 kN*m MN,z,Rd = 4.77 kN*m Vz,T,Rd = 83.70 kN

**SPILL PARAMETERS:**

BUCKLING PARAMETERS:

in y:  in z:

Lz = 5.80 m Lam_z = 0.74

Lcr,z = 1.91 m Xz = 0.88

kzy = 0.56 Lamz = 69.25 kzz = 1.00
Tt,Ed = -0.00 kN*m

Section class = 1

VERIFICATION FORMULAS:

Section resistance check:

N,Ed / Nc,Rd = 0.00 < 1.00 (6.2.4.(1))

My,Ed / MN,y,Rd = 0.08 < 1.00 (6.2.9.1.(2))

Mz,Ed / MN,z,Rd = 0.10 < 1.00 (6.2.9.1.(2))

(My,Ed / MN,y,Rd)^ 1.00 + (Mz,Ed / MN,z,Rd)^1.00 = 0.18 < 1.00 (6.2.9.1.(6))

Vy,Ed / Vy,c,Rd = 0.01 < 1.00 (6.2.6.(1))

Vz,Ed / Vz,c,Rd = 0.01 < 1.00 (6.2.6.(1))

Overall bar stability control:

Lambda,z = 69.25 < Lambda,max = 210.00 STABLE

N , Ed / (1.00 (6.3.3.(4))

Correct profile!!!

- **SLS:**

STANDARD : NF EN 1993-1:2005/NA:2007/AC:2009, Eurocode 3: Design of steel structures.

ANALYSIS TYPE: Family Verification

FAMILY: 7 hourglass

PIECE: 3026 POINT: COORDINATE:

SECTION PARAMETERS : DC 150x58x2

height =15.0 cm

bf=11.6 cm Ay=4.58 cm2 Az=6.17 cm2 Ax =11.47 cm2

ea =0.0 cm Iy =397.86 cm4 Iz =87.61 cm4 Ix =0.41 cm4

es=0.0 cm Wely =53.05 cm3 Welz =15.11 cm3

LIMITED TRAVEL

Arrows (LOCAL MARK):

uy = 0.0 cm < uy max = L/200.00 = 2.9 cm Checked

Decisive load case: 1 PERM1

uz = 0.0 cm < uz max = L/200.00 = 2.9 cm Checked

Decisive load case: 1 PERM1

Travel (OVERALL REFERENCE): Not analyzed

Correct profile!!!

IV.2.4.5.3 Conclusion :

The DC 150 profile has been verified and is suitable as a sand pit.

IV.2.4.6. Study of bracing:

IV.2.4.6.1 Introduction:

Bracing is a static system that provides stability to a structure by counteracting horizontal forces caused by various actions such as wind, earthquakes, and shocks. Its purpose is to transfer the forces acting on the structure due to wind to the ground. Bracing elements can be placed either on the roof of an inclined plane (wind beams) or on the façade (stabilization columns). These elements play a crucial role in resisting wind forces that act on the gables and long sides of the structure.

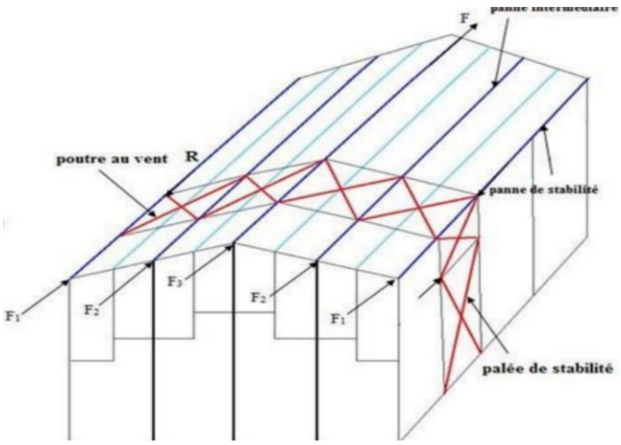


Figure IV. 7: bracing

Horizontal bracing:

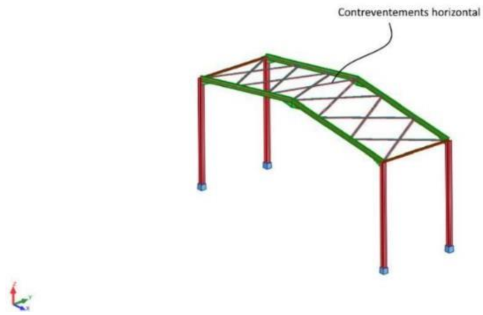


Figure IV. 8: horizontal bracing

IV.2.4.6.2 Sizing

Famille : 6 Contreventement	
3273	OK m20

IV.2.4.6.3 VERIFICATION

- **ULU :**

THE CALCULATION NOTE

STANDARD: NF EN 1993-1:2005/NA:2007/AC:2009, Eurocode 3: Design of steel structures.

TYPE OF ANALYSIS: Family Verification

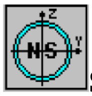
FAMILY: 6 Bracing

PIECE : 3273contraventement_3273 POINT: 7 **CONTACT:** x =0.54 L = 2.91 m

LOADINGS:

Decisive load case: 8 COMB2 (1+2)*1.35+(3+4)*1.50

MATERIAL: Steel (S235) fy = 235.00 MPa



SECTION PARAMETERS: m20

h=2.0 cm gM0=1.00 gM1=1.00

Ay=2.00 cm2 Az =2.00 cm2 Ax=3.14 cm2

tw =1.0 cm Iy =0.79 cm4 Iz =0.79 cm4 Ix=1.57 cm4

Wply =1.33 cm3 Wplz =1.33 cm3

INTERNAL EFFORTS AND ULTIMATE RESISTANCES:

N,Ed = -2.33 kN My,Ed = 0.01 kN *m Mz,Ed = 0.15 kN *m Vy,Ed = -1.05 kN

Nt,Rd = 73.83 kN My,pl,Rd = 0.31 kN *m Mz,pl,Rd = 0.31 kN *m Vy,T,Rd = 26.05 kN

My,c,Rd = 0.31 kN *m Mz,c,Rd = 0.31 kN *m Vz,Ed = 0.11 kN

MN,y,Rd = 0.31 kN*m MN,z,Rd = 0.31 kN*m Vz,T,Rd = 26.05 kN

Tt,Ed = -0.01 kN*m

Section class = 1

SPILL PARAMETERS:

BUCKLING PARAMETERS:

in y: in z:

VERIFICATION FORMULAS:

Checking the resistance of the section:

N,Ed / Nt,Rd = 0.03 < 1.00 (6.2.3.(1))

My,Ed / MN,y,Rd = 0.04 < 1.00 (6.2.9.1.(2))

Mz,Ed / MN,z,Rd = 0.47 < 1.00 (6.2.9.1.(2))

(My,Ed / MN,y,Rd)^ 2.00 + (Mz,Ed / MN,z,Rd)^2.00 = 0.23 < 1.00 (6.2.9.1.(6))

Vy,Ed / Vy,T,Rd = 0.04 < 1.00 (6.2.6-7)

Vz,Ed / Vz,T,Rd = 0.00 < 1.00 (6.2.6-7)

Tau,ty,Ed /(fy /(sqrt(3)*gM0)) = 0.04 < 1.00 (6.2.6)

Tau,tz,Ed /(fy /(sqrt(3)*gM0)) = 0.04 < 1.00 (6.2.6)

Correct profile!!!

IV.2.4.6.4 Conclusion:

The M20 section is checked and is adopted for bracing.

IV.2.4.7 Long-sided stability platform :

The stability supports the horizontal forces caused by the beam's reaction to the wind (slopes bracing) and the horizontal longitudinal response resulting from the service brakes. We will disregard compressed diagonals when considering the latter dimensions and only utilise stretched diagonals, provided they are curved.

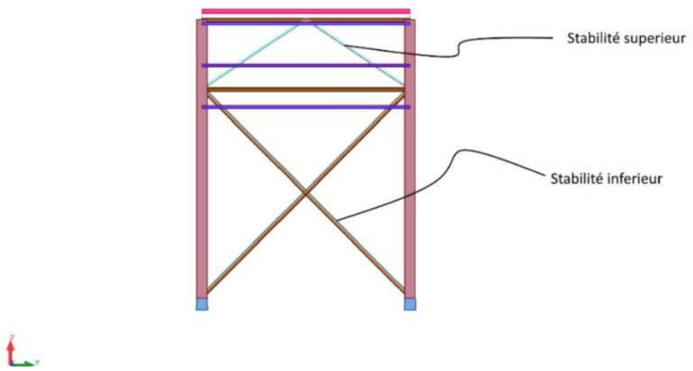


Figure IV. 9:long-sided stability platform

IV.2.4.7.1 SIZING:

- **ULU:**

Famille : 8 palée stabilité	
3566	OK DC 150x58x2

- **ULS:**

Famille : 8 palée stabilité	
4380	OK DC 150x58x2

IV.2.4.7.2 VERIFICATION:

THE CALCULATION NOTE:
STANDARD: NF EN 1993-1:2005/NA:2007/AC:2009, Eurocode 3: Design of steel structures.

TYPE OF ANALYSIS : Family verification

FAMILY : 8 stability platform

ROOM : 3566 Beam_3566 POINT: 7 COORDINATE: x = 1.00 L =4.65 m

LOADINGS : Decisive load case: 11 COMB5 (1+2)*1.35+3*1.05+6*1.50

MATERIAL: Steel (S23) fy = 235.00 MPa

SECTION PARAMETERS: DC 150x58x2
h=15.0 cm gM0=1.00 gM1=1.00

b=11.6 cm Ay=4.58 cm2 Az =6.17 cm2 Ax=11.47 cm2

tw =0.0 cm Iy =397.86 cm4 Iz =87.61 cm4 Ix =0.41 cm4

tf =0.0 cm Wply =62.00 cm3 Wplz =20.29 cm3

INTERNAL EFFORTS AND ULTIMATE RESISTANCES:

N,Ed = -0.83 kN My,Ed = -0.26 kN *m Mz,Ed = -0.27 kN *m Vy,Ed = 0.16 kN

Nt,Rd = 269.55 kN My,pl,Rd = 14.57 kN *m Mz,pl,Rd = 4.77 kN *m Vy,T,Rd = 62.11 kN

My,c,Rd = 14.57 kN *m Mz,c,Rd = 4.77 kN *m Vz,Ed = -0.31 kN

MN,y,Rd = 14.57 kN*m MN,z,Rd = 4.77 kN*m Vz,T,Rd = 83.70 kN

Tt,Ed = -0.00 kN*m

Section class = 1

SPILL PARAMETERS:

BUCKLING PARAMETERS:

in y: in z:

VERIFICATION FORMULAS:

Checking the resistance of the section:

N,Ed / Nt,Rd = 0.00 < 1.00 (6.2.3.(1))

My,Ed / MN,y,Rd = 0.02 < 1.00 (6.2.9.1.(2))

Mz,Ed / MN,z,Rd = 0.06 < 1.00 (6.2.9.1.(2))

(My,Ed / MN,y,Rd)^ 1.00 + (Mz,Ed / MN,z,Rd)^1.00 = 0.07 < 1.00 (6.2.9.1.(6))

Vy,Ed / Vy,c,Rd = 0.00 < 1.00 (6.2.6.(1))

Vz,Ed / Vz,c,Rd = 0.00 < 1.00 (6.2.6.(1))

Correct profile!!!

IV.2.4.7.3 Conclusions:

This study allows us to identify the profiles of the secondary elements that need to withstand various stresses. Here are the profiles that were selected after the checks and sizing:

- Purlins: Z 180x58x2
- Cladding rails: Z 150x45x2
- Posts: SIG 300x50x3.5
- Sand pits: DC 150x58x2
- Lines: Round bar with a 10 mm diameter
- Stability platform: DC 150x58x2

IV.2.5 Study of the main elements:

IV.2.5.1 Introduction:

Static stability must be ensured at the overall structure and individual element levels. To achieve this, the structure must be calculated in all possible combinations as defined by regulations. The actions exerted on the structure generate various stresses within the materials, which can lead to element deformation. Therefore, it is crucial to verify that the stresses and deformations remain within permissible limits to ensure the desired level of safety.

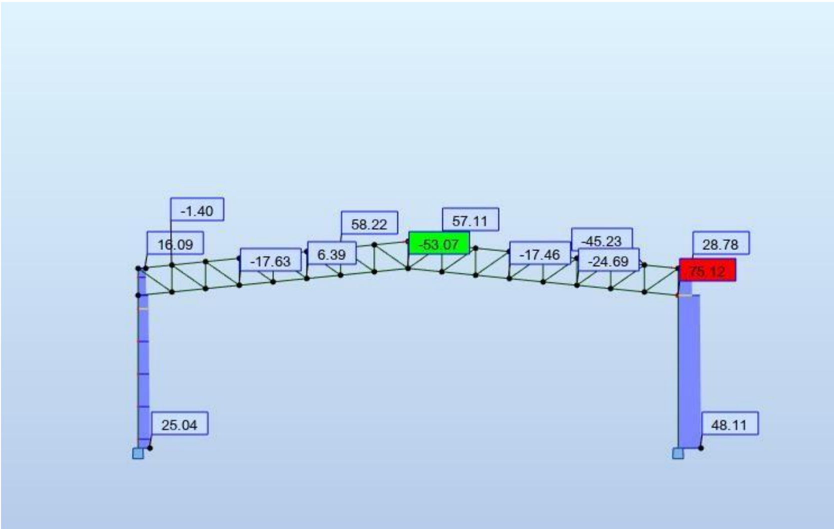


Figure IV. 10: Maximum effort Fx(kN) under ULS combination.

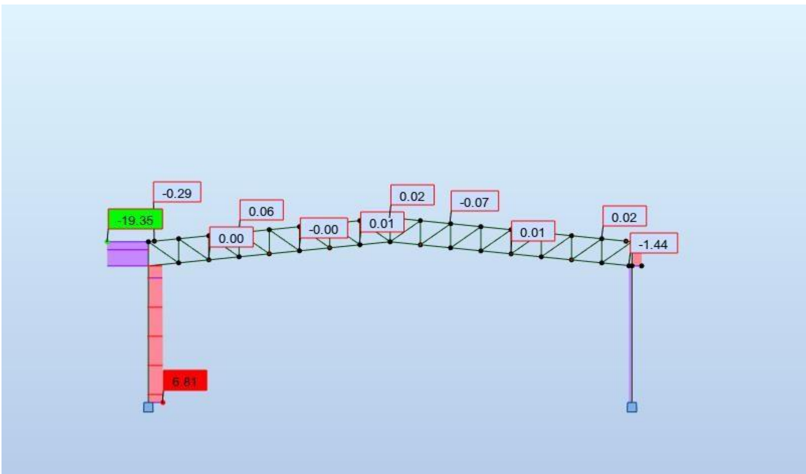


Figure IV. 11: maximum effort Fz(kN) under ULS combination.

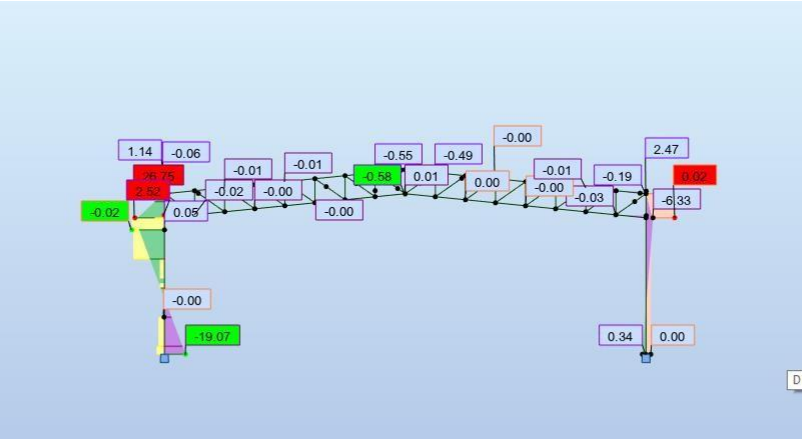


Figure IV. 12: maximum moment (kN.m) under ULS combination

IV.2.5.2 The sleepers (beams):

- **ULS :**

The calculation note:

STANDARD: *NF EN 1993-1:2005/NA:2007/AC:2009, Eurocode 3: Design of steel structures.*

TYPE OF ANALYSIS: family verification

FAMILY : 11 beam

PIECE : 6943 **POINT :** **CONTACT:**

SECTION PARAMETERS: MDC 160x60x2

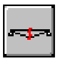
height =23.7 cm

bf=16.0 cm Ay=0.00 cm2 Az=0.00 cm2 Ax =12.27 cm2

ea =0.0 cm Iy =787.42 cm4 Iz =482.02 cm4 Ix =0.20 cm4

es=0.0 cm Wely =66.59 cm3 Welz =60.25 cm3

LIMITED TRAVEL

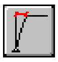
 ***Arrows (LOCAL MARK):***

uy = 0.0 cm < uy max = L/200.00 = 4.4 cm Checked

Decisive load case: 1 PERM1

uz = 0.0 cm < uz max = L/200.00 = 4.4 cm Checked

Decisive load case: 1 PERM1

 ***Travel (OVERALL REFERENCE):*** *Not analyzed*

Correct profile!!!

IV.2.5.3 Poles:

IV.2.5.3.1 Sizing:

- **ULU :**

Famille : 1 poteau		
405	OK	SGM 1x1x1

IV.2.5.3.2 Verification:

The calculation note:

STANDARD: NF EN 1993-1:2005/NA:2007/AC:2009, Eurocode 3: Design of steel structures.

TYPE OF ANALYSIS: Family verification

FAMILY: 1 post

PIECE: 405 POINTS: 7 COORDINATE: x = 0.85 L =6.80 m

LOADINGS:

Decisive load case: 11 COMB5 (1+2) *1.35+3*1.05+6*1.50

MATERIAL:

Steel (S235) fy = 235.00 MPa

SECTION PARAMETERS: SGM 1x1x1

h=35.0 cm gM0=1.00 gM1=1.00

b=17.6 cm Ay=5.12 cm2 Az=26.81 cm2 Ax =46.33 cm2

tw =0.0 cm Iy =7935.41 cm4 Iz =1717.98 cm4 Ix =3347.01 cm4

tf =0.0 cm Wply =549.39 cm3 Wplz =255.56 cm3

INTERNAL EFFORTS AND ULTIMATE RESISTANCES:

N,Ed = 30.58 kN My,Ed = 36.44 kN *m Mz,Ed = 0.05 kN *m Vy,Ed = 0.01 kN

Nc,Rd = 1088.65 kN My,Ed,max = 36.44 kN *m Mz,Ed,max = -0.12 kN *m Vy,T,Rd = 69.44 kN

Nb,Rd = 486.35 kN My,c,Rd = 129.11 kN *m Mz,c,Rd = 60.06 kN *m Vz,Ed = 8.97 kN

MN,y,Rd = 129.01 kN*m MN,z,Rd = 60.01 kN*m Vz,T,Rd = 363.70 kN

Tt,Ed = -0.00 kN*m

Section class = 1

SPILL PARAMETERS:

BUCKLING PARAMETERS:

in y: in z:

Ly = 8.00 m Lam_y = 0.65 Lz = 8.00 m Lam_z = 1.40

Lcr,y = 8.00 m Xy = 0.91 Lcr,z = 8.00 m Xz = 0.45

Lamy = 61.12 kyy = 1.02 Lamz = 131.37 kyz = 0.66

VERIFICATION FORMULAS:

Checking the resistance of the section:

N,Ed / Nc,Rd = 0.03 < 1.00 (6.2.4.(1))

My,Ed / MN,y,Rd = 0.28 < 1.00 (6.2.9.1.(2))

Mz,Ed / MN,z,Rd = 0.00 < 1.00 (6.2.9.1.(2))

(My,Ed / MN,y,Rd)^ 1.00 + (Mz,Ed / MN,z,Rd)^1.00 = 0.28 < 1.00 (6.2.9.1.(6))

Vy,Ed / Vy,c,Rd = 0.00 < 1.00 (6.2.6.(1))

Vz,Ed / Vz,c,Rd = 0.02 < 1.00 (6.2.6.(1))

Checking the overall stability of the bar:

Lambda,y = 61.12 < Lambda,max = 210.00 Lambda,z = 131.37 < Lambda,max = 210.00 STABLE

N , Ed / (1.00(6.3.3.(4))

N , Ed / (1.00(6.3.3.(4))

Correct profile !!!

IV.2.5.3.3 Conclusions:

- In this chapter, we will discuss the hangar modelling and sizing software, Robot Structural Analyses, and present the results from the static study. The maximum stresses caused by the applied forces have been thoroughly verified and indicate that the hangar is stable. Based on our calculations, we have identified the profiles of the gantry elements that must withstand different stresses. Below are the profiles that have been selected after conducting checks and sizing:
- SGM 1x1x1 posts
- MDC 160x60x2 sleepers

IV.3 Design of steel connections

IV.3.1 Introduction :

The calculation and design of steel connections are just as important as the sizing of the parts that make up the structure. Assemblies serve to bring together and join the parts, facilitating the transmission and distribution of various stresses within different structural components. If the joint fails, the overall functionality of the structure is compromised.

IV.3.2 Definition:

An assembly is a device that facilitates the joining of multiple parts by ensuring the transmission and distribution of various requests, without causing any unnecessary stresses.

In metal construction, the elements to be assembled can be placed:

- Either end to end (splicing, joining);
- -Or they can be placed concurrently (beam/column connections, trellis, and articulated systems).

IV.3.3 How de the joints work:

The main methods of assembly are:

- Bolting;
- Welding.

There are two main modes of operation for assemblies:

- Operation by obstruction: this applies to ordinary, non-restressed bolts, where the rods bear the forces and operate in shear.
- Operation by adhesion: in this case, the forces are transmitted through the adhesion of the surfaces of the parts in contact. This applies to welding and bolting with HR bolts.

IV.3.4 Calculation of assemblies:

IV.3.4.1 Constructive Precautions:

-Always use a washer to prevent damage to the parts.

-The pre-tensioning force should be applied according to the design value. Therefore, torque wrenches are necessary.

-The coefficient of friction should match its calculated value. To achieve this, it is important to prepare the surfaces by brushing or shot-blasting to remove any rust, scale, etc. (Never skip this step).

-We accept coefficients of 0.3 for brushed surfaces and 0.45 for shot-blasted surfaces.

-Tightening should be done gradually, following the sequence established by standards NF P.22464, to prevent deformation of the support plates and maintain their flatness.

IV.3.4.2 Bolt Tightening Calculation:

The formula for calculating the tightening of a bolt is: $M_s = Kd \times P_v$

Where:

d = diameter

K = coefficient that depends on the condition of the bolts. For lightly greased bolts (typical delivery case), K = 0.18. To ensure the accuracy of the tightening moment, we require a moment greater than 10.

IV.3.4.3 Bolt Calculation:

Based on the results obtained at the base of the uprights, we have a lifting force of N=23562daN.

The bolts used are HR 8.8M16 type bolts, with a resistance section area of the M16 bolt.

As=157mm2

$F_{sd} \leq F_{srd} = K_s \times n \times \mu(F_{pcd})/\gamma_{ms}$

Ms=1.25 ELU

Ks=1

n = number of friction interfaces

μ =0.3

F_{pcd} =0.7× f_{bu} × A_s=8792daN

F_{srd} =1x1x0.3x8692/1.25 =2637.6 daN is the design sliding resistance of an assembly made with an HR 8.8 bolt

N= $\frac{}{F_{srd}}$=4.73 we adopt for practical reasons 10 bolts

Spacing = 60mm level 1 bolts = 60 mm

IV.3.4.4 Thickness of the assembly part:

According to the tensile strength condition:

$N_{sd} \leq (0.9 \times A_{net} \times f_u) / \gamma m^2$

$A_{net} \geq (N_{sd} \times \gamma m^2)/0.9 x f_u$ =4.43cm ^{2}

$A_{tr}= A_{brut}-A_{net}$=62.21×T-4.43

D_{tr} =62.21×T-5

T=0.08cm

We adopt for practical reasons T=1cm

Stud diameter: 230.62

Tensile force per stud is: N'/4= 13062/4 = 3265.5kg

Allowable force per stud:

$$N_a = \frac{0.1(1+7\times\frac{350}{1000}).}{(1+\frac{\emptyset}{250})\ 2\ (20\emptyset+19.2\emptyset+7\emptyset)} \geq \frac{N`}{4}$$

Ø≥24

we will adopt Ø=30mm Platinum

The compression force N=1919DAN see the table

The lifting force is N'=13062 DAN (see table)

The bending moment M=2695DAN.m

By balancing the forces at the axis of the left stud, we can obtain:

$$\sigma_b = \frac{2Nl}{bh`(h-\frac{h`}{3})} \leq F_{ub}$$

With h`$^3$ +3(l- h) h`^{2} +90Ah'lb-90Ahlb

A=353x2mm ^{2}

L=639.4mm

e= $\frac{M}{N}$=464.4 mm

h=395mm; b=440mm

h`$^3$ +228h`^{2} +1062.46h` – 74372.4=0

- ⇨ h`=13.09cm
- ⇨ σ_b=**7.89N/mm ^{2} ≤ 25/1.5=16.66N/mm ^{2}**

Base plate support surface :

$$\frac{N}{\sigma_b} = h_p b_p = 44 \times 32 = 1048 >>> \frac{N}{\sigma_b} = 15$$

Thickness of the plate :

$$t > 31\sqrt{\frac{3\gamma\, m\, \sigma_{moy}}{fy}}$$

$$t > 31\sqrt{\frac{3\times1\times5.47}{235}} = 8.191mm$$

IV.3.4.5 Welding beads:

IV.3.4.5.1 Resistance of the weld bead

a $\sum L_i$>(N $\sqrt{2}.\gamma_m B_m)f_u$

S235 $\gamma_m B_m = 1$steel ; $\sum L_i = 649mm; f_u = 360$

a = 1mm, we adopt a welding seam with a thickness of a = 5mm.

NB :

for practical reasons, the assembly is done using a standard. In this assembly part, we can determine five types of assemblies, and it is based on the nodes at the truss level.

- ***Type 1:***

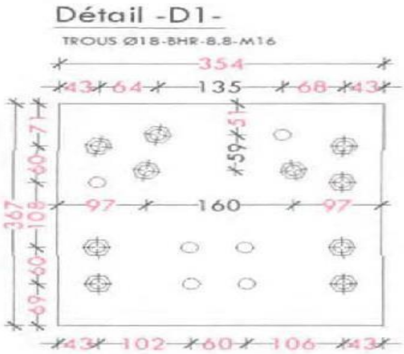


Figure IV. 13:Layout of the joint n°1

Table IV. 5: Joint 01 efforts

NODES	Type	EFFORT(DaN)
01	Firm amount	22988.53

the resistance of the bolts to sliding is : F_{sd}< F_{srd} = Ks.n. µ (F_{pcd}) / γ_{ms}

γ_{ms}=1.25 ELU

Ks=1

n = number of friction interfaces

µ =0.3

F_{pcd} =0.7fubxAs=8792daN M16:AS=157mm2

F_{srd} = 1x1x0.3x8792/1.25=2637.6daN is the sliding resistance of an assembly made by an HR 8.8 bolt

n = $\frac{}{F_{srd}}$=8.71 we adopt for practical reasons 10 bolts

spacing = 60mm

- ***Type 02:***

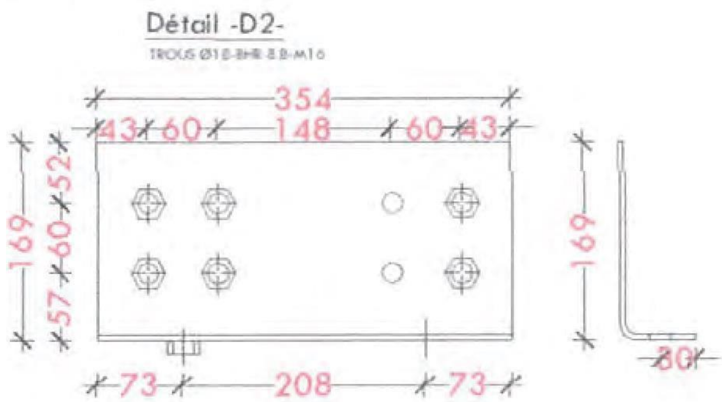


Figure IV. 14: Layout of the joint n°2

Table IV. 6: Joint 02 efforts

Knots	Type	Effort (daN)
02	Firm amount	2622.79

The resistance of the bolts to sliding is : F_{sd}< F_{srd} = Ks.n. μ (F_{pcd}) /γ_{ms}

γ_{ms}=1.25 ELU

Ks=1

N=number of friction interfaces

M =0.3

F_{pcd}=0.7xfubxAs=8792 daN M 16: As =157mm2

F_{srd} =1x1x0.3x8692/1.25=2637.6daN is the design sliding resistance of an assembly made with HR8.8 bolt

n = $\frac{}{F_{srd}}$=0.994 we adopt for practical reasons 6 bolts

Spacing =60mm

- ***Type 03:***

Table IV. 7: Joint 03 efforts

Knots	Type	Effort (daN)
02	Firm amount	11633.85

The resistance of the bolts to sliding is: F_{sd}< F_{srd} = Ks.n. μ (F_{pcd}) / γ_{ms}

γ_{ms}=1.25 ELU

Ks=1

n = number of friction interfaces

μ =0.3

F_{sd}=0.7xfubxAs=8792 daN M 16: As =157mm2

$F_{srd}=\frac{1\times1\times0{,}3\times8692}{1{,}25}$=2637.6 daN is the design sliding resistance of an assembly made by HR 8.8 bolts

n = $\frac{}{F_{srd}}$=4.41 we adopt for practical reasons 8 bolts

Spacing =60mm

- ***Type 04:***

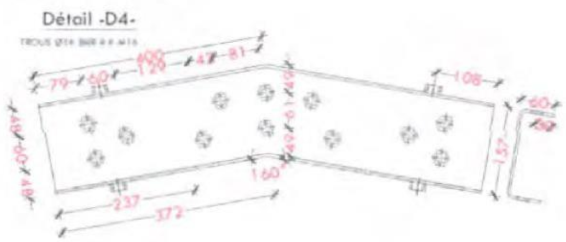


Figure IV. 15: Layout of the joint n°4

Table IV. 8: Joint 04 efforts

Knots	Type	Effort (daN)
03	Firm amount	15176.23

The resistance of the bolts to sliding is: F_{sd}< F_{srd} = Ks.n. µ (F_{pcd}) / γ_{ms}

γ_{ms}=1.25 ELU

Ks=1

n = number of friction interfaces

µ =0.3

F_{pcd} =0.7xfubxAs=8792 daN M 16: As =157mm28.8 bolts
n = _ =5.75 we adopt for practical reasons 20 bolts
$Fsrd$
Spacing =60mm

$F_{srd} = \frac{1\times1\times0,3\times8692}{1,25}$=2637.6 daN is the design sliding resistance of an assembly made by HR

- ***Type 05:***

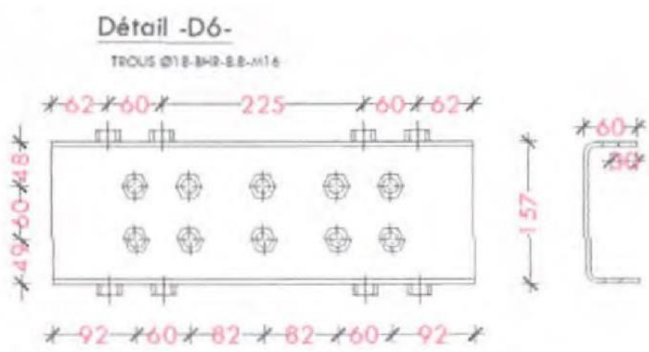


Figure IV. 16: Layout of the joint n°5

Table IV. 9: Joint 05 efforts

Knots	Type	Effort(daN)
04	Firm amount	20282.44

The resistance of the bolts to sliding is : $F_{srd} < F_{pcd}$= Ks.n. μ (F_{pcd}) / γ_{ms}

γ_{ms}=1.25 ELU

Ks=1

n = number of friction interfaces

μ =0.3

F_{pcd} =0.7xfubxAs=8792 daN M 16: As =157mm^2

$F_{srd} = \frac{1 \times 1 \times 0{,}3 \times 8692}{1{,}25}$=2637.6 daN is the design sliding resistance of an assembly made with HR 8.8 bolts

n = $\frac{}{F_{srd}}$=7.68 , for practical reasons M20 bolt is adopted

Spacing =60mm

Chapter V:
Foundation Design

V.1 Introduction:

A construction project, regardless of its form and purpose, always relies on a foundation. The components that serve as a connection between the structure and the ground are known as foundations. The size of the foundation depends on the site where it will be installed. The foundations of a building are the sections that directly touch the ground. They bear the weight of the superstructure, making them a crucial element as their effective design and construction ensure the overall structure performs well. The dimensions of the foundations are determined according to the BAEL91 code [8].

- **Selecting of foundation type**:

The selection of the foundation type is based on three criteria:

- The characteristics and weight of the structure.
- The intensity and distribution of the loads on the building.
- The quality of the underlying soil.

The bearing capacity of the soil for the presented project is σ_{sol} = 2 bars at a depth of 2 m (obtained from the geotechnical report).

V.2 Effort's calculation:

The most unfavourable reactions on the "Column feet" connection have been calculated using the ROBOT software

- Vsd = 9,32kN
- Nsd = 68,89kN
- Msd = 26,39 kN.m

V.3 Load to be considered:

Table V. 1: Load to be considered

	Effort	**ULS**	**SLS**
sole	N (KN)	78,41	102,52
	M (KN.m)	56,39	41,77
$\boldsymbol{\sigma_{sol}}$	2 bar = 0.2 Mpa = 20000 daN/m²		

V.4 Pre-sizing of the column flange:

The dimensions of the sole are chosen to be similar to those of the base of the post. In the structure, the posts and columns have a rectangular base (h x b), so the soles are also rectangular (H x B).

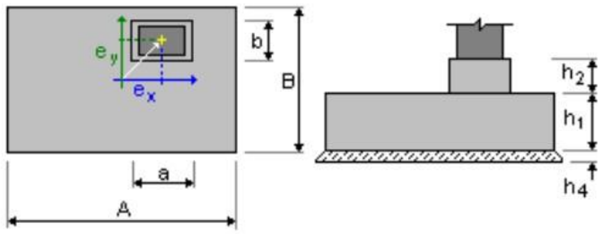
h and b: dimensions of the post plate being considered

H and B: dimensions of the sole.

h1: d + c, where c = 5 cm (concrete for cleanliness).

d: the useful height of the sole is determined by

Géométrie:



A	= 2,60 (m)	a	= 0,50 (m)
B	= 1,90 (m)	b	= 0,50 (m)
h1	= 0,55 (m)	e_x	= 0,00 (m)
h2	= 1,10 (m)	e_y	= 0,00 (m)
h4	= 0,10 (m)		

- ***Determination of A and B:***

Given: a = 0.5m

b = 0.5m

$$\sigma_{SOL} = \frac{\_}{A\times B} \Rightarrow A\times B \geq \frac{\_}{\sigma_{SOL}} \Rightarrow A\times B \geq \frac{102520}{0.2} = 512600\ mm^2 \Rightarrow A\times B \geq 0.51 m^2$$

Therefore, we choose a rectangular sole with dimensions: $\left\{ \begin{matrix} A = 2.1m \\ B = 1.9m \end{matrix} \right\}$

- **Determination of d and h:**

h = d + 5 cm

$\frac{B-b}{4} \leq$ d ≤ A – a ⇒ $\frac{1.5-0.5}{4} \leq$ d ≤ 2.1 – 0.5

0.25 m ≤ d ≤ 1.6 m ⇒ 25 cm ≤ d ≤ 160 cm

d_{min} = 40 cm we select d = 95 cm

Then: h = d + 5 = 95 + 5⇒ h = 1m

V.5 Verification of overturning stability:

It must be verified that the eccentricity of the resultant vertical gravity forces and seismic forces remains within the central half of the foundation base, resisting overturning.

$e0 = \frac{Ms}{N_s} \leq \frac{B}{4}$

e0 =0,38 m ≤ $\frac{B}{4} = \frac{1.9}{4}$ = 0.475cm ………condition checked .

V.6 Calculation of reinforcement:

a. ULU :

$Au = \frac{(A-a)}{8 \times d \times \sigma_{st}}$

Avec : $\sigma_{st} = \frac{fe}{\gamma} = \frac{400}{1.15}$ = 347.82 Mpa

Nu =78410N

$Au = \frac{78.41 \times 10^{-3} \times (2.1-0.5)}{8 \times 0.95 \times 347.82} = 4.75 cm^2$

b. ULS :

$As = \frac{(A-a)}{8 \times d \times \sigma_{st}}$

Avec : σ_{st} =min ($\frac{2}{3}f$; $110\sqrt{nf_{c28}}$) = 201.63 Mpa

Ns = 102520N

As=$\frac{102.52\times10^{-3}\times(2.1-0.5)}{8\times0.95\times201.63} = 10.70cm^2$

We have As > Au, so we need to reinforce with 10HA14=11.00 cm².

V.7 Sills calculation :

The sills serve to connect the soles together and are subjected to tensile forces. A sill is placed directly on clean concrete to prevent contamination of the fresh concrete by the supporting soil during pouring. The concrete also provides uniform support for the sill.

V.7.1 Sills sizing :

According to RPA2003 [1], for a type S3 floor, the minimum dimensions of the sill's cross section are 40 cm x 40 cm.

V.8 Calculation of reinforcement:

The sills must be designed to withstand traction caused by a force equal to:

F = max [$\frac{N}{\alpha}$; 20KN]

Avec :

N : Equal to the maximum vertical gravity load value exerted by the connected support points.

α: Coefficient of category of site being considered, for S3 soils (α = 15).

a. ULU : $\frac{Nu}{\alpha} = \frac{78410}{15}$= 5227.33N
b. ULS : $\frac{Ns}{\alpha} = \frac{102520}{15} = 6834.66$N

F = max [5.23KN; 6.83 KN; 20KN] = 20KN

Ast = $\frac{F}{\sigma_{st}}$

Astu =$\frac{}{\sigma_{stu}} = \frac{0.2}{347.82} = 0.57cm^2$

Asts = $\frac{F}{\sigma_{sts}} = \frac{0.2}{201.6} = 0.99cm^2$

RPA2003 requires a minimum section:

Amin =0.6% (40×40)

Amin = 9.6cm^2

So we take: Ast = 11.00cm^2

V.9 Verification of non-fragility requirement:

Ast $\leq 0.23\text{b}\times\text{d}\times\frac{f_{c28}}{f_e}$

$$\left\{\begin{array}{l} A_{st}=11.00cm^2 \\ 0.23b\times d\times\frac{f_{c28}}{f_e}=23cm^2 \end{array}\right\}$$

Ast $=11.00cm^2 \leq 23cm^2$ …condition check

V.10 calculation of transverse reinforcements:

calculation of transverse reinforcements:

$\emptyset t \leq \min\left(\frac{h}{35};\ \emptyset\min;\frac{b}{10}\right)$

$\emptyset t \leq \min\left(\frac{400}{35};\ 14mm;\frac{400}{10}\right)$

$\emptyset t \leq \min$ (11.42mm; 14mm; 40mm)

So we take: $\emptyset$t=10 mm

V.11 Frame spacing calculation:

RPA2003 [1] requires frames who's spacing must not exceed:

St $\leq$ (20cm; 15$\emptyset$t)

St $\leq$ (20cm; 15$\times$ 1)
So we adopt a spacing: St = 15 cm

THE CALCULATION NOTE:

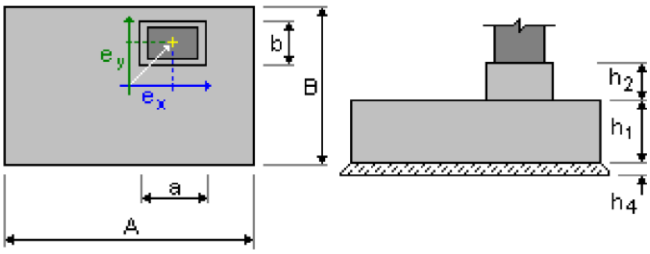
sole: Sole1 Number
of identical elements: 1

Basic data

Principles

- Standard for geotechnical calculations: DTU 13.12
- Standard for reinforced concrete calculations: BAEL 91 mod. 99
- Sole shape: free

Geometry:



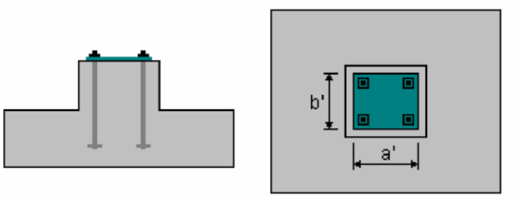
A = 2.60 (m) a = 0.50 (m)

B = 1.90 (m) b = 0.50 (m)

h 1 = 0.55 (m) ex = 0.00 (m)

h 2 = 1.10 (m) ey = 0.00 (m)

h 4 = 0.10 (m)



a' = 40.0 (cm)

b' = 40.0 (cm)

c1 = 5.0 (cm)

c 2 = 3.0 (cm)

Materials

- Concrete : C25/30; characteristic resistance = 25.00MPa

Density weight = 2501.36 (kG /m3)

- Longitudinal reinforcement : type HA 400 resistance

Characteristic = 400.00 MPa

- Transverse reinforcement : type HA 400 resistance

Characteristic = 400.00 MPa

- Additional reinforcement :: type HA 400 resistance

Characteristic = 400.00 MPa

Loadings:

Loads on the sole:

Case Nature Group N Fx Fy Mx My

(kN) (kN) (kN) (kN*m) (kN*m)

G1 permanent

1 -68.89 9.32 0.00 0.00 26.39

Loads on the slope:

Case Nature Q1 (kN /m2)

List of combinations

1/ ULU: 1.35G1

2/ ULU: 1.00G1

3/ ULS: 1.00G1

4/* ULU: 1.35G1

5/* ULU: 1.00G1

6/*ULS: 1.00G1

Geotechnical sizing

Principles

Sizing the foundation on:

- Charge capacity
- Sliding
- Reversal
- Uprising

Ground :

Stresses in the ground: σ ULU = 0.10 (MPa) σ ULS = 0.07 (MPa)

Ground level : N 1 = 0.00 (m)

Maximum level of the sole **:** Na = 0.00 (m)

Level of the bottom of the excavation: N f = -1.50 (m)

Compact sands and gravels

- Ground level: 0.00 (m)
- Density weight: 1937.46 (kG /m3)
- Unit density weight: 2692.05 (kG /m3)
- Internal friction angle: 35.00 (Deg)
- Cohesion: 0.00 (MPa)

Limit states

Calculation of constraints

Type of soil under the foundation: uniform

Sizing combination ULU: 1.00G1

Loading coefficients: 1.35 * weight of the foundation

1.35 * ground weight

Calculation results: at ground level

Weight of the foundation and the ground above the foundation: Gr = 231.41(kN)

Sizing load:

Nr = 162.52 (kN) Mx = -0.00 (kN*m) My = 41.77 (kN*m)

Equivalent dimensions of the foundation:

B' = 1

L' = 1

Level thickness: D_{min}= 1.65 (m)

Breaking stress calculation method : stress pressure meter

(DTU 13.12, 3.22)

q_u= 0.20 (MPa)

Soil calculation stop:

q_{lim}= q_u/ γ f = 0.10 (MPa)

γf = 2.00

Stress in the ground: q_{ref}= 0.04 (MPa)

Safety coefficient: q_{lim}/ q_{ref}= 2.344 > 1

Uprising

ELECTED uprising

Sizing combination ULU: 1.35G1

Loading coefficients: 1.00 * weight of the foundation

1.00 * ground weight

Weight of the foundation and the ground above the foundation: Gr = 171.41(kN)

Sizing load:

Nr = 78.41 (kN) Mx = -0.00 (kN*m) My = 56.39 (kN*m)

Contact surface s = 67.03 (%)

Slim = 10.00 (%)

ULS uprising

Unfavourable combination: ULS: 1.00G1

Loading coefficients: 1.00 * weight of the foundation

1.00 * ground weight

Weight of the foundation and the ground above the foundation: Gr = 171.41(kN)

Sizing load:

Nr = 102.52 (kN) Mx = -0.00 (kN*m) M_y= 41.77 (kN*m)

Contact area s = 100.00 (%)

Slim = 100.00 (%)

Slip

Sizing combination ULU: 1.35G1

Loading coefficients: 1.00 * weight of the foundation

1.00 * ground weight

Weight of the foundation and the ground above the foundation: Gr = 171.41(kN)

Sizing load:

Nr = 78.41 (kN) Mx = -0.00 (kN*m) M_y= 56.39 (kN*m)

Equivalent dimensions of the foundation: A_ = 2.60 (m) B_ = 1.90 (m)

Sliding surface: 3.31 (m2)

Cohesion: C = 0.00 (MPa)

Foundation-soil friction coefficient: tg(φ) = 0.70

Sliding force value F = 12.58 (kN)

Value of the force preventing the foundation from sliding:

- at ground level: F(stab) = 39.21 (kN)

Sliding stability: 3.116 > 1

Reversal

Around the OX axis

Sizing combination ULU: 1.35G1

Loading coefficients: 1.00 * weight of the foundation

1.00 * ground weight

Weight of the foundation and the ground above the foundation: Gr = 171.41(kN)

Sizing load:

Nr = 78.41 (kN) Mx = -0.00 (kN*m) M_y= 56.39 (kN*m)

Stabilizing moment: M_{stab}= 162.84 (kN*m)

Overturning moment: M_{renv}= 88.35 (kN*m)

stability : 1.843 > 1

Around the OY axis

Unfavourable combination: ULU: 1.35G1

Loading coefficients: 1.00 * weight of the foundation

1.00 * ground weight

Weight of the foundation and the ground above the foundation: Gr = 171.41(kN)

Sizing load:

Nr = 78.41 (kN) M_x= -0.00 (kN*m) M_y= 56.39 (kN*m)

Stabilizing moment: M_{stab}= 222.84 (kN*m)

Overturning moment: M_{renv}= 177.29 (kN*m)

Rollover stability: 1.257 > 1

Reinforced Concrete Sizing:

Principles:

• Cracking: not very detrimental

• Environment: non-aggressive

• Taking into account the condition of non-fragility: yes

Punching and shear analysis:

Shear:

Sizing combination ULU: 1.35G1

Loading coefficients: 1.00 * weight of the foundation

1.00 * ground weight

Sizing load:

Nr = 78.41 (kN) Mx = -0.00 (kN *m) M_y= 56.39 (kN*m)

Length of critical perimeter: 1.90 (m)

Shear force: 4.25 (kN)

Effective height of section heff = 0.49 (m)

Shear area: A = 0.93 (m2)

Shear stress: 0.00 (MPa)

Allowable shear stress: 1.17 (MPa)

Safety coefficient: 255.4 > 1

Theoretical reinforcement

Insulated sole:

Lower steels:

ULU: 1.00G1

M_y= 0.49 (kN*m) Asx = 4.90 (cm2/m)

M_x= 0.00 (kN*m) Asy = 4.90 (cm2/m)

As min = 4.90 (cm2/m)

Superior steels:

A' sx = 0.00 (cm2/m)

A' sy = 0.00 (cm2/m)

As min = 0.00 (cm2/m)

Maximum regulatory spacing e_{max} = 0.25 (m)

Longitudinal reinforcement A = 8.87 (cm2) A_{min}= 8.00 (cm2)

A = 2 * (+ A_{sy})

A_{sx} = 4.43 (cm2) A_{sy}= 0.00 (cm2)

Actual reinforcement

Insulated sole:

Lower steels:

In X: 7 HA 400 14 l = 2.99 (m) e = 1*-0.74 + 6*0.25

In Y: 10 HA 400 14 l = 2.29 (m) e = 1*-1.12 + 9*0.25

Superior steels:

Was:

Longitudinal reinforcement

In Y: 8 HA 400 14 l = 1.76 (m) e = 1*-0.20 + 3*0.14

Transverse reinforcement

8 HA 400 10 l = 1.96 (m) e = 1*0.39 + 5*0.20 + 2*0.11

Quantitative:

• Volume of Concrete = 2.99 (m3)

• Formwork Area = 7.15 (m2)

- HA 400 steel

• Total weight = 79.65 (kG)

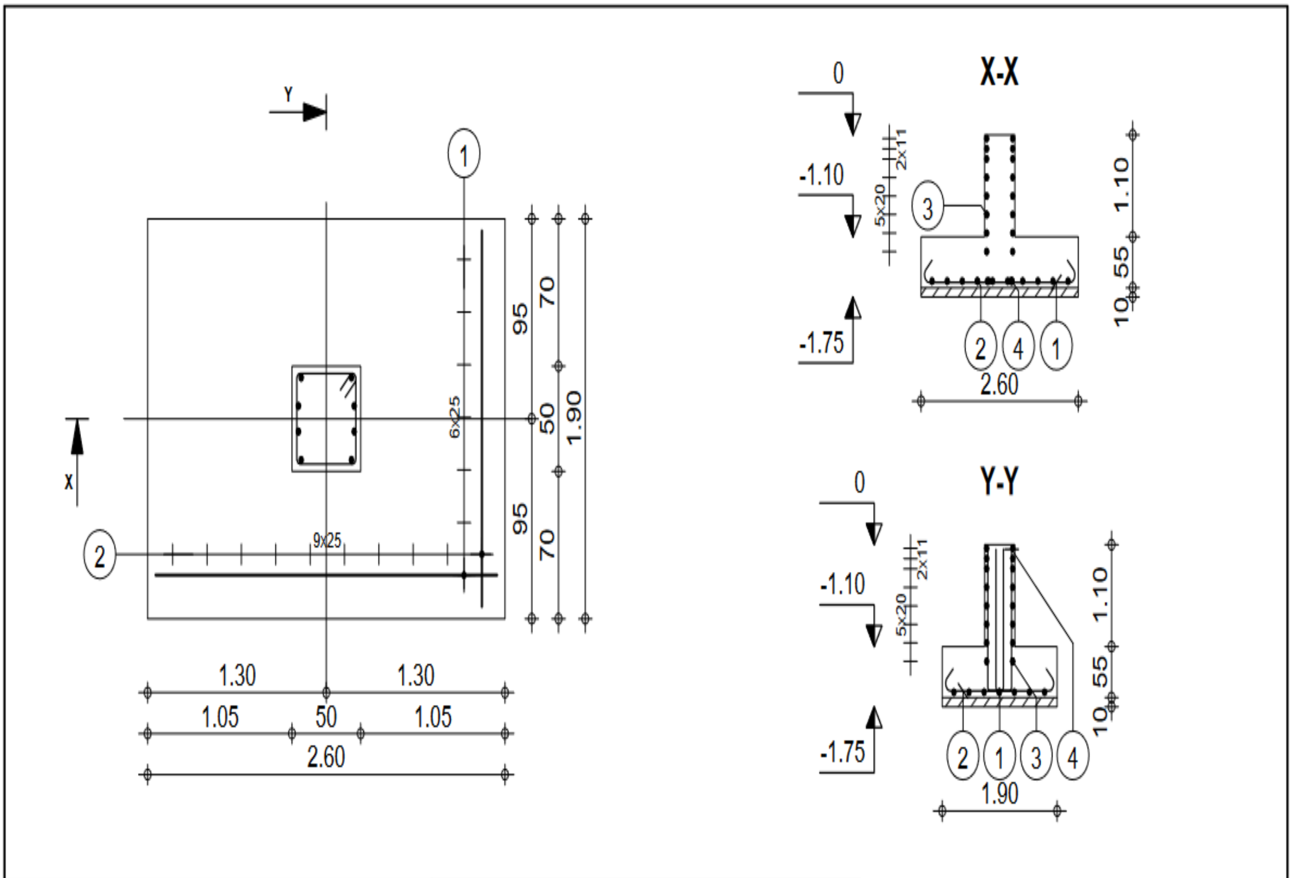
• Density = 26.62 (kG /m3)

• Average diameter = 13.1 (mm)

• List by diameters:

Diameter	Length (m)	Weight (kg)
10	15.64	9.65
14	57.91	70.00

V.13 Reinforcement layout:



Pos.	Armature		Code	Forme
1	7HA 14	l=2.99	00	2.50
2	10HA 14	l=2.29	00	1.80
3	8HA 10	l=1.96	31	44 / 44 / 13

Pos.	Armature		Code	Forme
4	8HA 14	l=1.76	00	1.54

Tél. Fax		Acier HA 400 = 52.9 kg
Fissuration peu préjudiciable	Béton : C25/30 = 2.99 m3	Acier HA 400 = 26.7 kg
Semelle1 Nombre 1	Surface du coffrage = 7.15 m2	Enrobage c1 = 5 cm, c2 = 3 cm
	Densité = 26.62 kg/ m3	
		Echelle pour la vue 1/33 — Page 1/1

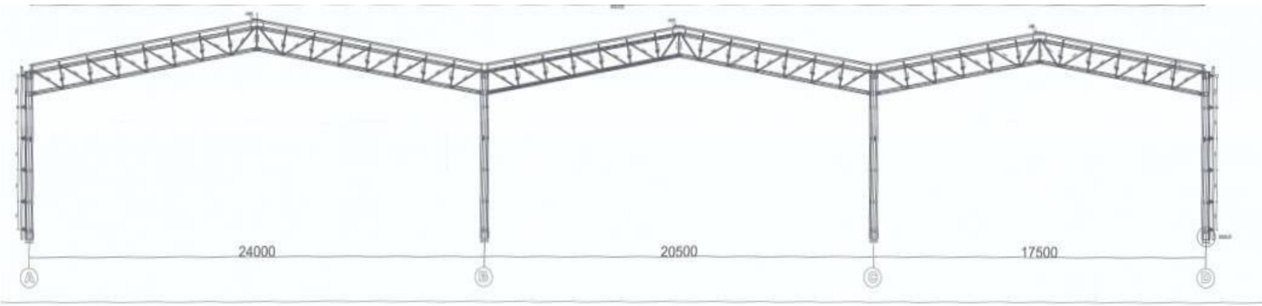
Figure V. 1: Reinforcement layout

Chapter VI:

Verification of the hangar's capacity to install supplementary equipment

VI.1 Introduction:

Trusses play a crucial role in the structural integrity of the roof. These trusses are designed using a triangular system, where the upper member (rafter) is positioned beneath the attic's exterior surface, and the ends are connected to the lower frame (tie beam) using drop gussets. A lattice system of uprights and diagonals links these frames. The trusses are supported by posts, walls, or sometimes hourglasses. This chapter focuses on lightweight roofing trusses designed to support roofing materials.



VI.2 Objective

The goal is to verify if the existing truss structure can support the weight of the newly installed evaporators. The equipment is mounted using heavy black angle brackets measuring 60x60x6 mm, welded to the truss at the junctions of the lower member and diagonals.

Table VI. 1.: Weights of additional equipment.

Equipment	Weight (kg)
Evaporator F50HC 1908 E10	167
FHD 932 E7 evaporator	48.4
Evaporator F62HC 2214 E6	396
Heat exchanger CD45W 8208-7	52

Figure VI. 1: F50HC 1908 E10 evaporator

Figure VI. 2: FHD 932 E7 evaporator

Figure VI. 3: Evaporator F62HC 2214 E6

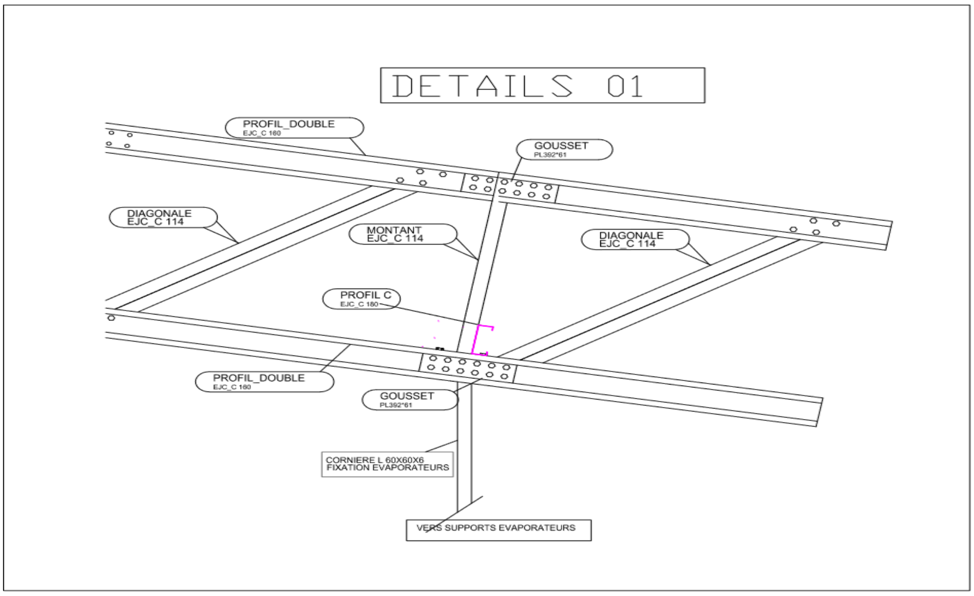
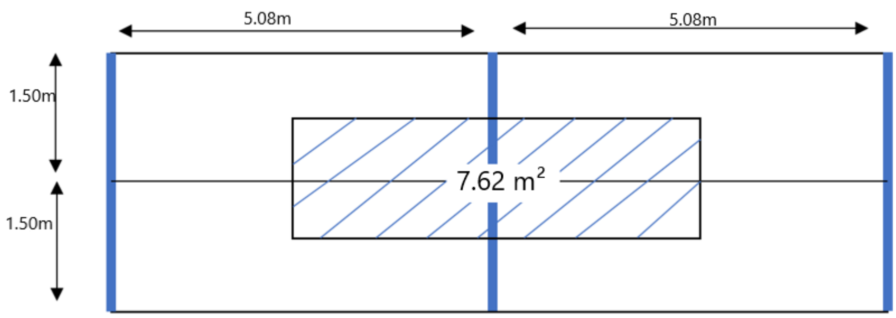


Figure VI. 4: Detail of the equipment's support

VI.3 Calculations

The farm is made up of an N-shaped trellis system with the characteristics: Height: H=2.43m and span: L=24 m; The trusses are spaced 5.05 m apart with a slope α=11.5°



VI.4 Calculation of charges:

- Wind and sand loads → we keep the same values as chapter III.
- Permanent loads →TN40 + false suspended ceiling, G = 1,376 kN/m²
- Maintenance charge which is Q=0.4 kN/ml

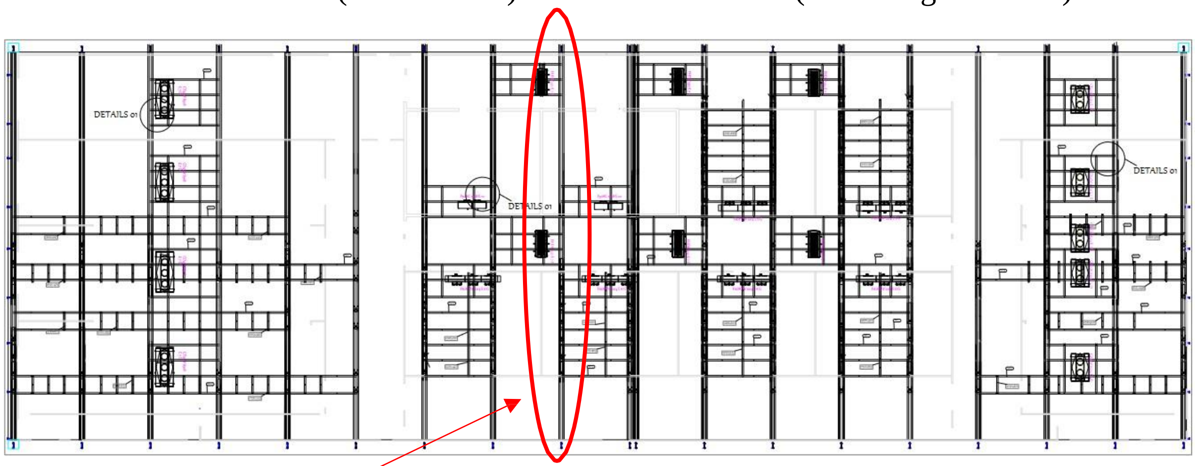
Table VI. 2: Different loads applied on the busiesrt farm.

	Loading	**Distributed load (kN/m²)**	**Linear load (kN/ml)**	**Concentrated Load (kN)**	**Application area**
Permanent load (G)	Own weight of the breakdown	5.2 kg x 5.08/100		0.263	Top chord
	TN40 sheet metal + false ceiling (thickness = 4mm)	0.0317x7.62 m²	0.24		Top chord
	insulated suspended ceiling	0.15 x7.62 m²	1.143		Top chord
Operating expenses (Q)	**a) Loads uniformly distributed**				
	Maintenance charge			0.4	Applied to L/3 and 2L/3
	b) Concentrated load				
	Evaporator 48.4kg			0.161	Lower member
	Evaporator 396kg			1,320	Lower member
	Evaporator 167 kg			0.557	Lower member
N.B - The edge nodes carry half the load of the sheet metal. - Maintenance charges are applied to nodes at 1/3 and 2/3 spans (DTR BC-2-2)					

N.B

➔ The truss carries a half on each side, which means an area of 5.05 m along the truss.

We will choose the busiest (intermediate) farm for verification (see the figure below):

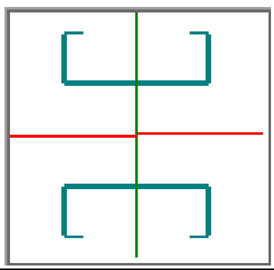


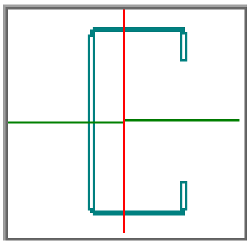
The busiest farm Long pan:

The farm carries the following loads:

- Two evaporators of 48.4 kg.
- One evaporator of 396 kg.
- One evaporator of 167 kg
- Weight of power cables
- Weight of TN 40 sheet
- Maintenance surcharges.

Table VI. 3: Characteristics of double C and C profiles.

	• Section: DC 160x60x2 • Element: Bottom and top chord • Cold rolled steel • yielding stress f_e =235 MPa (E24) • Moment of inertia ZZ: Iz = 480.27 cm4 • Moment of inertia YY: Iy = 787.58 cm4 • Moment of inertia XX: Ix = 0.15 cm4 • Area: 12.28 cm²

	• Section: C 116x60x2 • Element: Diagonals and amounts • Cold rolled steel • Elastic limit f_e=235 MPa (E24) • Moment of inertia ZZ: $I_z = 28.30\ cm^4$ • Moment of inertia YY: $I_y = 129.96\ cm^4$ • Moment of inertia XX: $I_x = 0.06\ cm^4$ • Area: 5.4 cm²

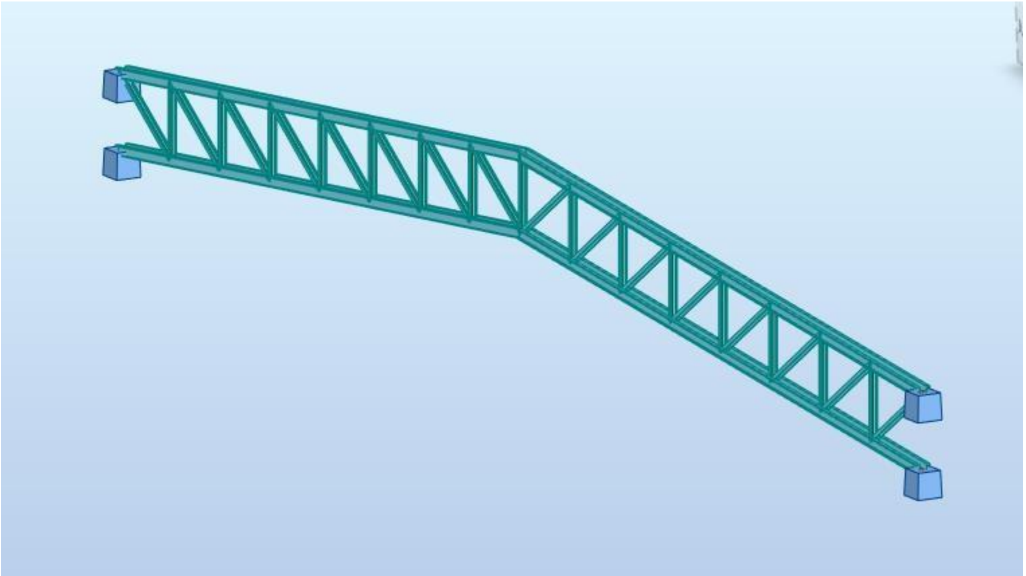


Figure VI. 5: 3D model of the farm

: Used combinations

Combinaison	Nom	Type d'analyse	Type de la	Nature du cas	Définition
7 (C)	COMB1	Combinaison lin	ELU	Structurelle	(1+2)*1.35+3*1.50+6*0.90
8 (C)	COMB2	Combinaison lin	ELU	Structurelle	(1+2)*1.35+(3+4)*1.50
9 (C)	COMB3	Combinaison lin	ELU	Structurelle	(1+2)*1.00+3*1.50+6*0.90
10 (C)	COMB4	Combinaison lin	ELU	Structurelle	(1+2)*1.00+(3+4)*1.50
11 (C)	COMB5	Combinaison lin	ELU	Structurelle	(1+2)*1.35+3*1.05+6*1.50
12 (C)	COMB6	Combinaison lin	ELU	Structurelle	(1+2)*1.35+6*1.50
13 (C)	COMB7	Combinaison lin	ELU	Structurelle	(1+2)*1.00+3*1.05+6*1.50
14 (C)	COMB8	Combinaison lin	ELU	Structurelle	(1+2)*1.00+6*1.50
15 (C)	COMB9	Combinaison lin	ELS	Structurelle	(1+2+3)*1.00+6*0.60
16 (C)	COMB10	Combinaison lin	ELS	Structurelle	(1+2+3+4)*1.00
17 (C)	COMB11	Combinaison lin	ELS	Structurelle	(1+2+6)*1.00+3*0.70
18 (C)	COMB12	Combinaison lin	ELS	Structurelle	(1+2+6)*1.00

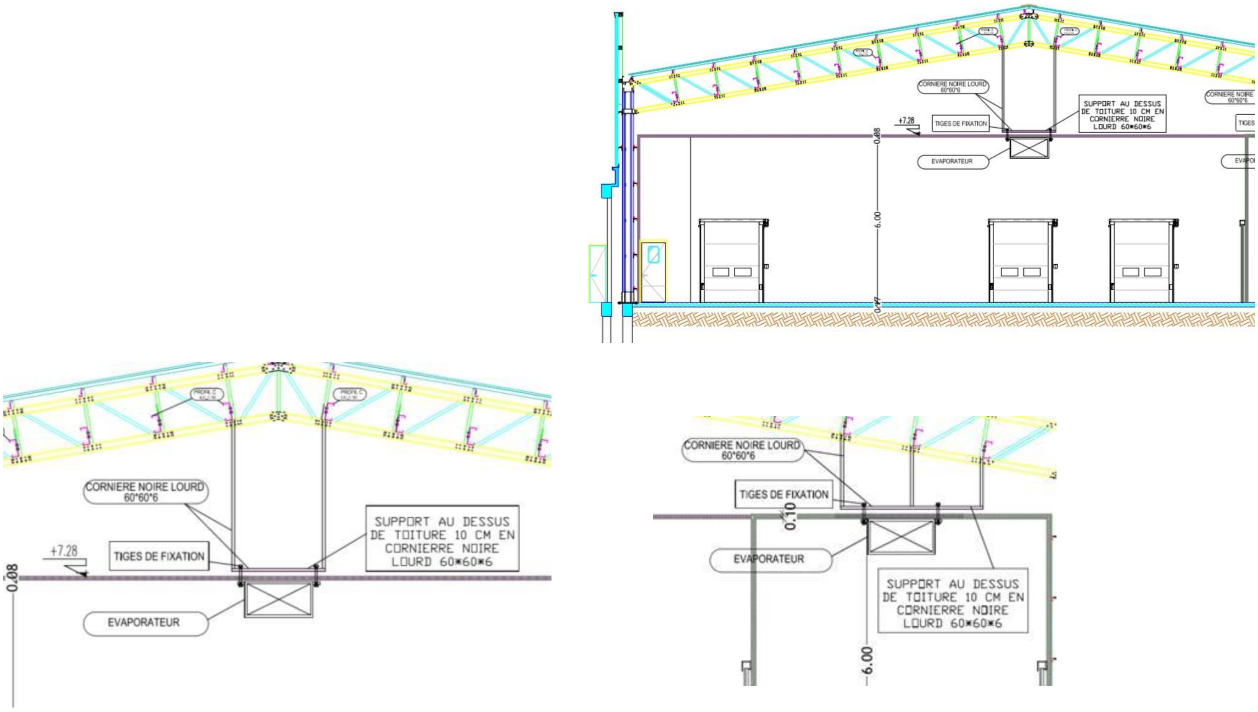


Figure VI. 6: equipment details

1. RSAP 2023 software (Robot Structural Analysis Profissional) is used for structural truss analysis.
2. The structural elements are not verified according to the EC03.
3. A stress analysis will be carried out to verify the capacity of the lower chords to support the equipment attached to them.

VI.5 Calculation of steel structures

STANDARD: *NF EN 1990 /NA*
ANALYSIS TYPE : Checking parts
FAMILY:
PIECE: 3 Membrane_inf **POINT:**
SECTION **SETTINGS :** **DC 160x60x2**
height =23.6 cm

bf=16.0 cm	Ay=0.00 cm2 Az=0.00 cm2		Ax =12.27 cm2
ea =0.0 cm	Iy =782.71 cm4	Iz =482.02 cm4	Ix =0.20 cm4
es=0.0 cm	Wely =66.33 cm3	Welz =60.25 cm3	

LIMITED TRAVEL

Arrows (LOCAL MARK):
uz = 0.9 cm < uz max = L/200.00 = 6.0 cm Checked
Decisive load case: 16 DEP /1/ 1*1.00 + 2*1.00 + 3*1.00 + 4*1.00

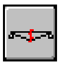
Travel (OVERALL REFERENCE): *Not analyzed*

Correct profile!!!
STANDARD: *NF EN 1990 /NA*
ANALYSIS TYPE: Checking part

FAMILY:
PIECE: 4 Membrare_sup **POINT:**
SECTION PARAMETERS: DC 160x60x2

ht=23.6 cm	
bf=16.0 cm	Ay=0.00 cm2 Az=0.00 cm2 Ax=12.27 cm2
ea=0.0 cm	Iy=782.71 cm4 Iz=482.02 cm4 Ix=0.20 cm4
es=0.0 cm	Wely=66.33 cm3 Welz=60.25 cm3

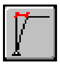
LIMITED TRAVEL



Arrows (LOCAL MARK):

uz = 0.8 cm < uz max = L/200.00 = 6.0 cm Checked

Decisive load case: 16 DEP /1/ 1*1.00 + 2*1.00 + 3*1.00 + 4*1.00



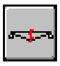
Travel (OVERALL REFERENCE): *Not analyzed*

Correct profile!!!

STANDARD: *NF EN 1990 /NA*
ANALYSIS TYPE: Checking parts
FAMILY:
PIECE: 5 Membrare_sup **POINT:**
SECTION PARAMETERS: DC 160x60x2

ht=23.6 cm	
bf=16.0 cm	Ay=0.00 cm2 Az=0.00 cm2 Ax=12.27 cm2
ea=0.0 cm	Iy=782.71 cm4 Iz=482.02 cm4 Ix=0.20 cm4
es=0.0 cm	Wely=66.33 cm3 Welz=60.25 cm3

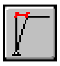
LIMITED TRAVEL



Arrows (LOCAL MARK):

uz = 0.8 cm < uz max = L/200.00 = 6.0 cm Checked

Decisive load case: 16 DEP /1/ 1*1.00 + 2*1.00 + 3*1.00 + 4*1.00



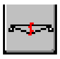
Travel (OVERALL REFERENCE): *Not analyzed*

Correct profile!!!

STANDARD: *NF EN 1990 /NA*
ANALYSIS TYPE: Checking parts
FAMILY:
PIECE: 6 Membrane_inf **POINT:** **CONTACT INFORMATION:**
SECTION PARAMETERS: DC 160x60x2

ht=23.6 cm	
bf=16.0 cm	Ay=0.00 cm2 Az=0.00 cm2 Ax=12.27 cm2
ea=0.0 cm	Iy=782.71 cm4 Iz=482.02 cm4 Ix=0.20 cm4
es=0.0 cm	Wely=66.33 cm3 Welz=60.25 cm3

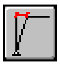
LIMITED TRAVEL



Arrows (LOCAL MARK):

uz = 0.8 cm < uz max = L/200.00 = 6.0 cm Checked

Decisive load case: 16 DEP /1/ 1*1.00 + 2*1.00 + 3*1.00 + 4*1.00



Travel (OVERALL REFERENCE): *Not analyzed*

Correct profile!!!

VI.6 Results:

Table VI. 4: The maximum stresses in the farm under the most unfavourable combination.

	S max [MPa]
MAX	164,63
Barre	4
Noeud	6
Cas	ELU/1
MIN	-163,44
Barre	3
Noeud	4
Cas	ELU/1

$\boldsymbol{\sigma_{calculé} < \sigma_e = 235\ \mathrm{MPa}}$ → the bars do not reach the elastic limit

Table VI. 5: The support reactions under the most unfavourable combination in the two base nodes of the truss.

	FX [kN]	FZ [kN]
MAX	0,0	40,36
Noeud	38	38
Cas	1	ELU/1
MIN	0,0	-32,46
Noeud	38	39
Cas	1	ELU/84

Table VI. 6: The maximum efforts on the farm are under the most unfavourable combination.

	FX [kN]
MAX	202,07
Barre	4
Noeud	6
Cas	ELU/1
MIN	-200,61
Barre	3
Noeud	4
Cas	ELU/1

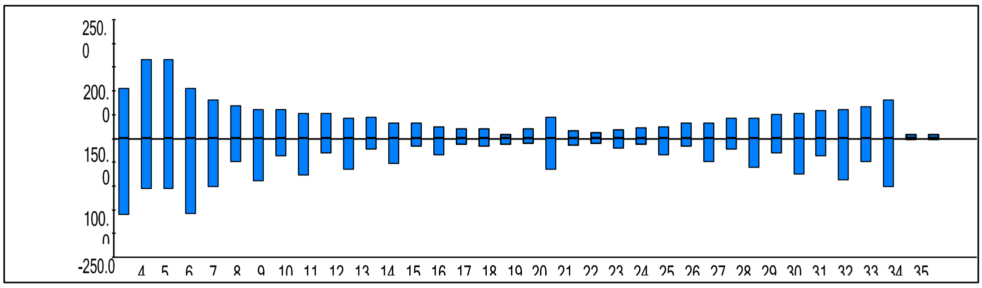


Figure VI. 7: The elastic stresses in the bars are less than the elastic limit f e = 235 MPa.

Pièce		Profil	Matériau	Ratio(uz	Cas (uz)
3 Membrure_sup	OK	DC 160x60x2	S 235	0.14	16 DEP /1/
4 Membrure_sup	OK	DC 160x60x2	S 235	0.14	16 DEP /1/
5 Membrure_sup	OK	DC 160x60x2	S 235	0.13	16 DEP /1/
6 Membrure_sup	OK	DC 160x60x2	S 235	0.13	16 DEP /1/

VI.7 Conclusion:

This chapter provides a detailed analysis of the truss structure's stability and resistance after installing additional equipment. The trusses are verified based on their load-bearing capacity, taking into account various dead and live loads, as well as material properties and geometrical characteristics. The verification ensures that the trusses can support the additional weights without compromising structural integrity.

General conclusion

General conclusion:

This project has enabled us to apply and further deepen the knowledge acquired during our master's training in civil engineering. The design of a steel structure follows limit states design principles, which encompass the most severe external forces such as overloads, operational conditions, snow, wind, and earthquakes.

This project's scope involved studying and designing a steel cold storage hangar. After determining the applied loads on the structure, the structural elements such as columns, beams, bracing systems for stability, purlins, and cladding rails were dimensioned accordingly.

The design process comprehensively addressed each structural element, connection, and critical construction detail. Precise calculations, verifications, and meticulous definitions of construction specifics were essential. The structure was modelled and subjected to numerical analysis using ROBOT structural analysis software.

This experience has significantly enhanced our understanding of steel construction, encompassing various modelling techniques, numerical analysis methods, and adherence to regulatory design and calculation principles in this field. An extensive review of pertinent literature has further enriched our knowledge base.

This project marks our initial foray into this expansive field, providing invaluable insights as we embark on our professional journey.

This version maintains the technical detail and academic tone appropriate for a master's thesis conclusion in civil engineering.

References

[1] Algerian seismic code RPA 2003.

[2] «Maghreb panneaux sandwich,» 2024. [En ligne]. Available: https://www.maghrebpanneaux.com/panneaux-sandwich.php.

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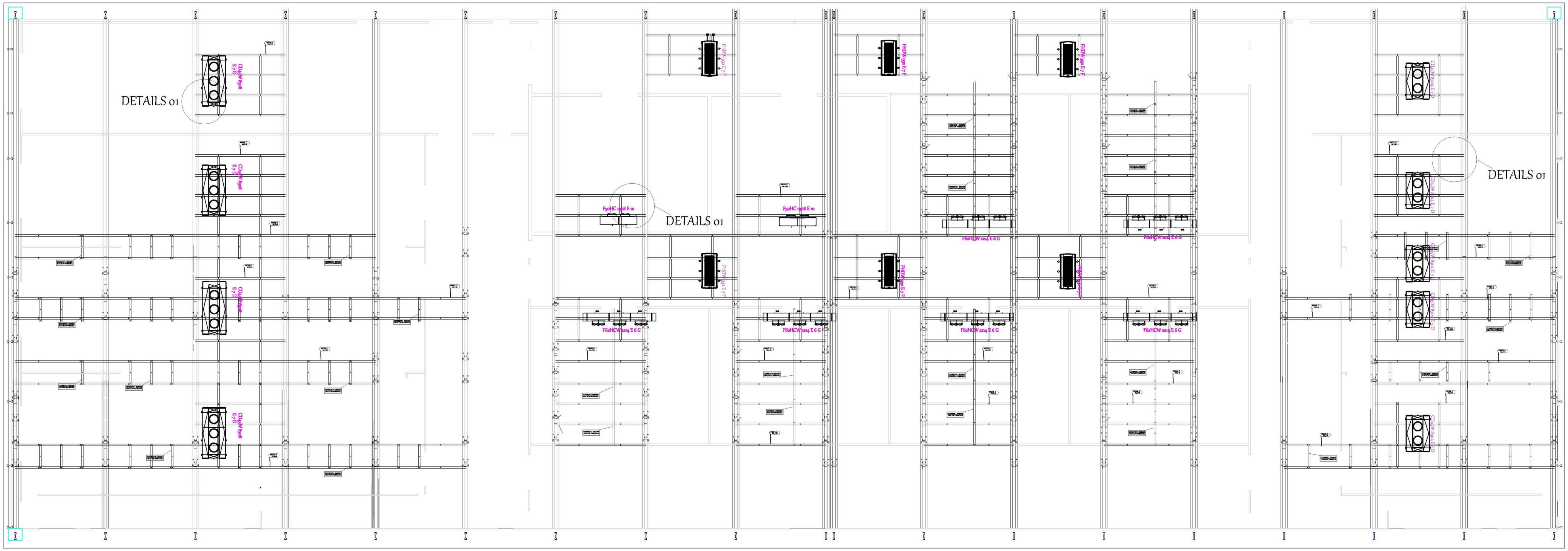
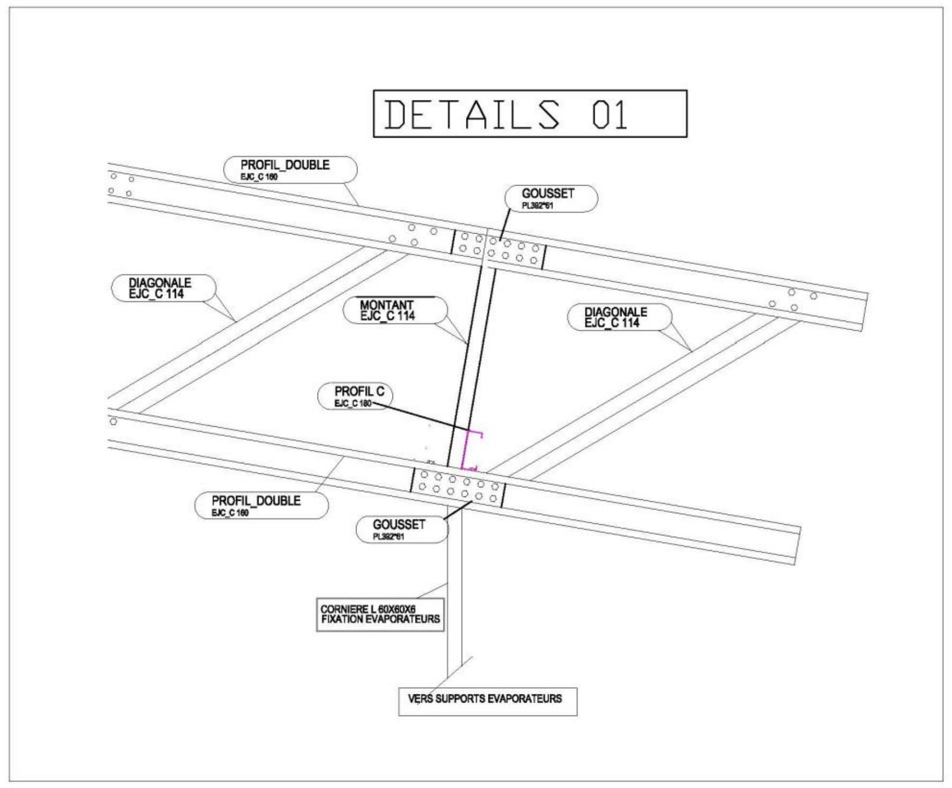
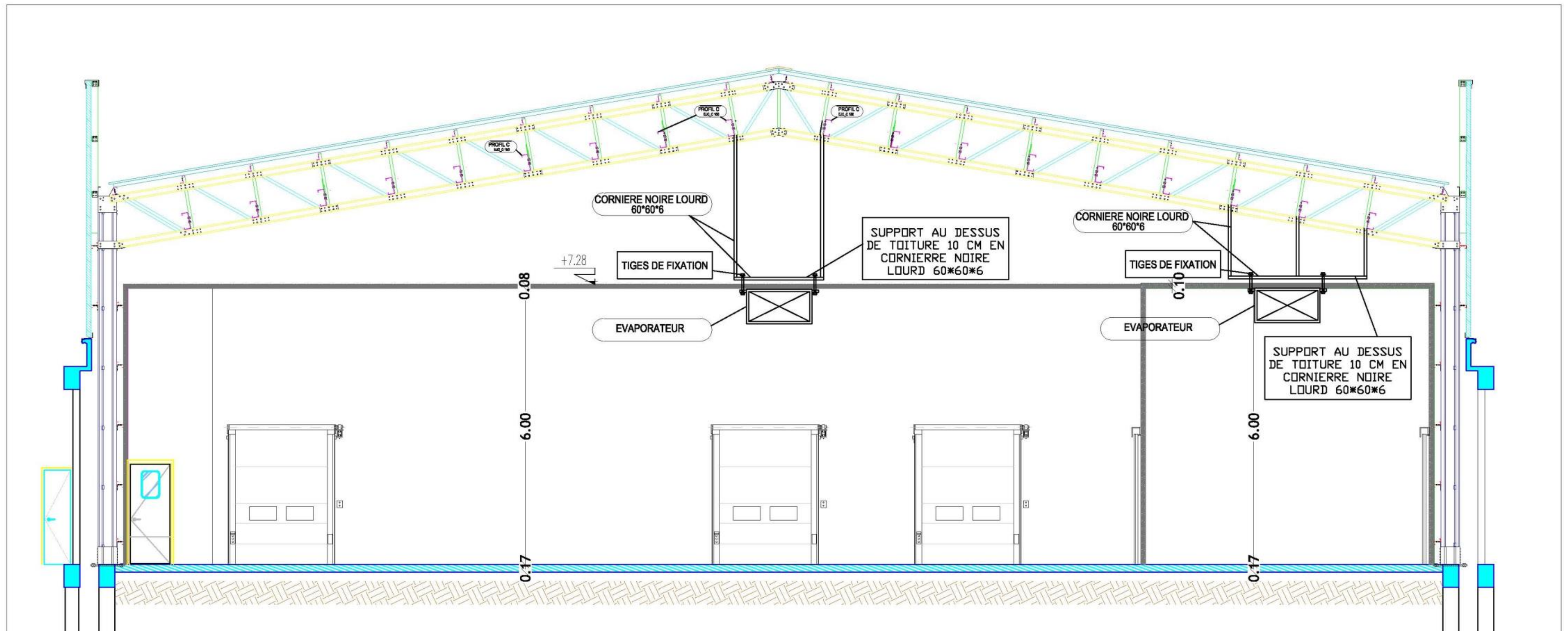
Appendixes

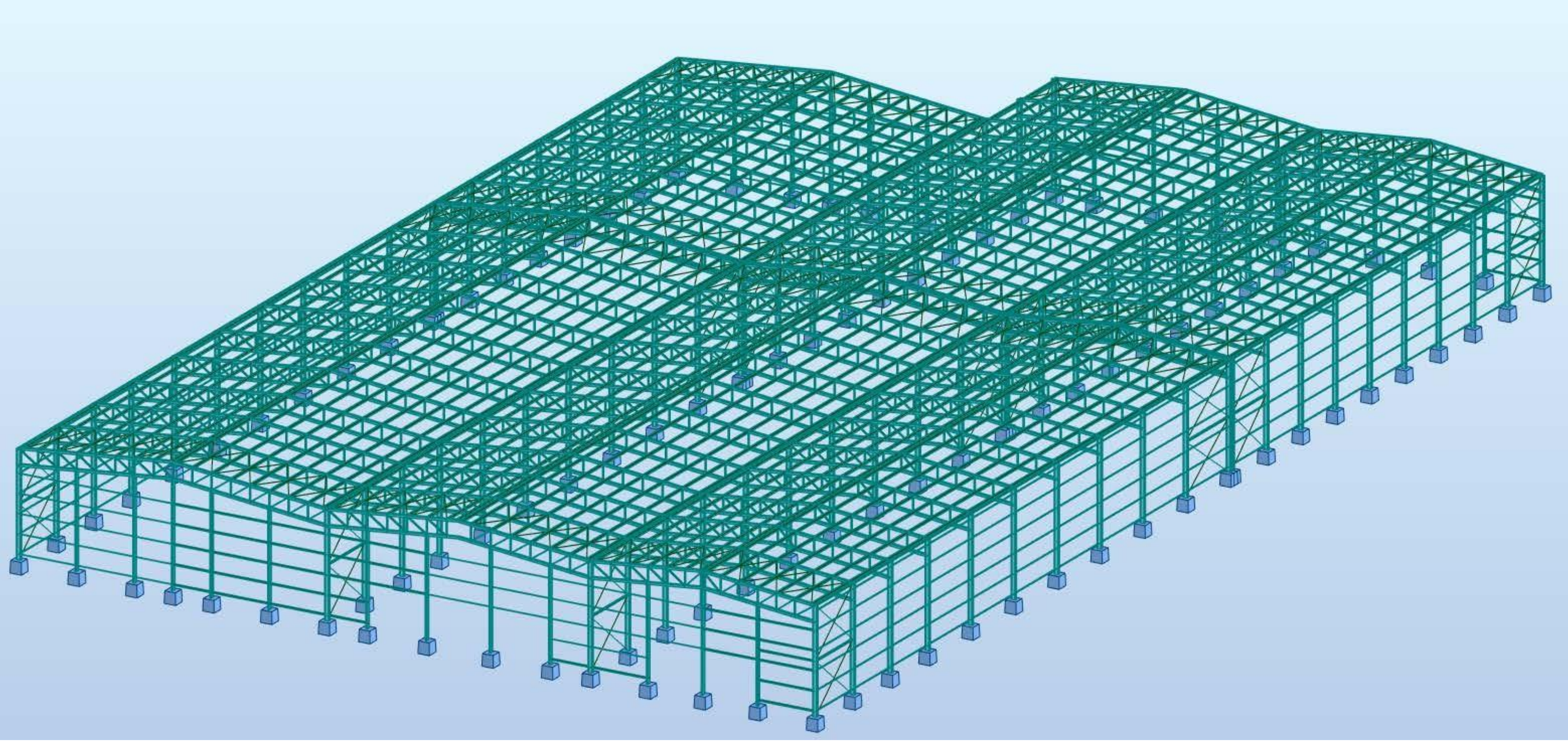




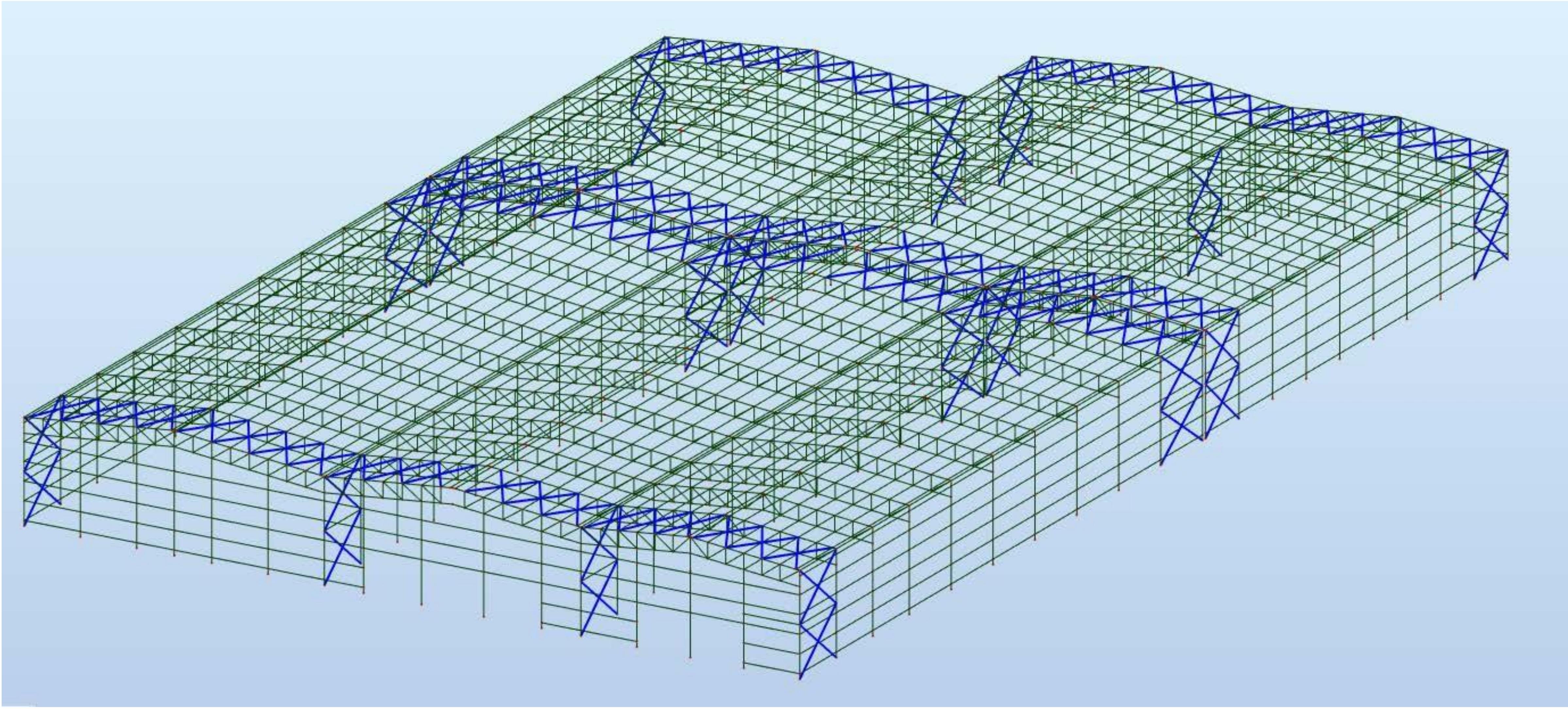


COUPE A-A

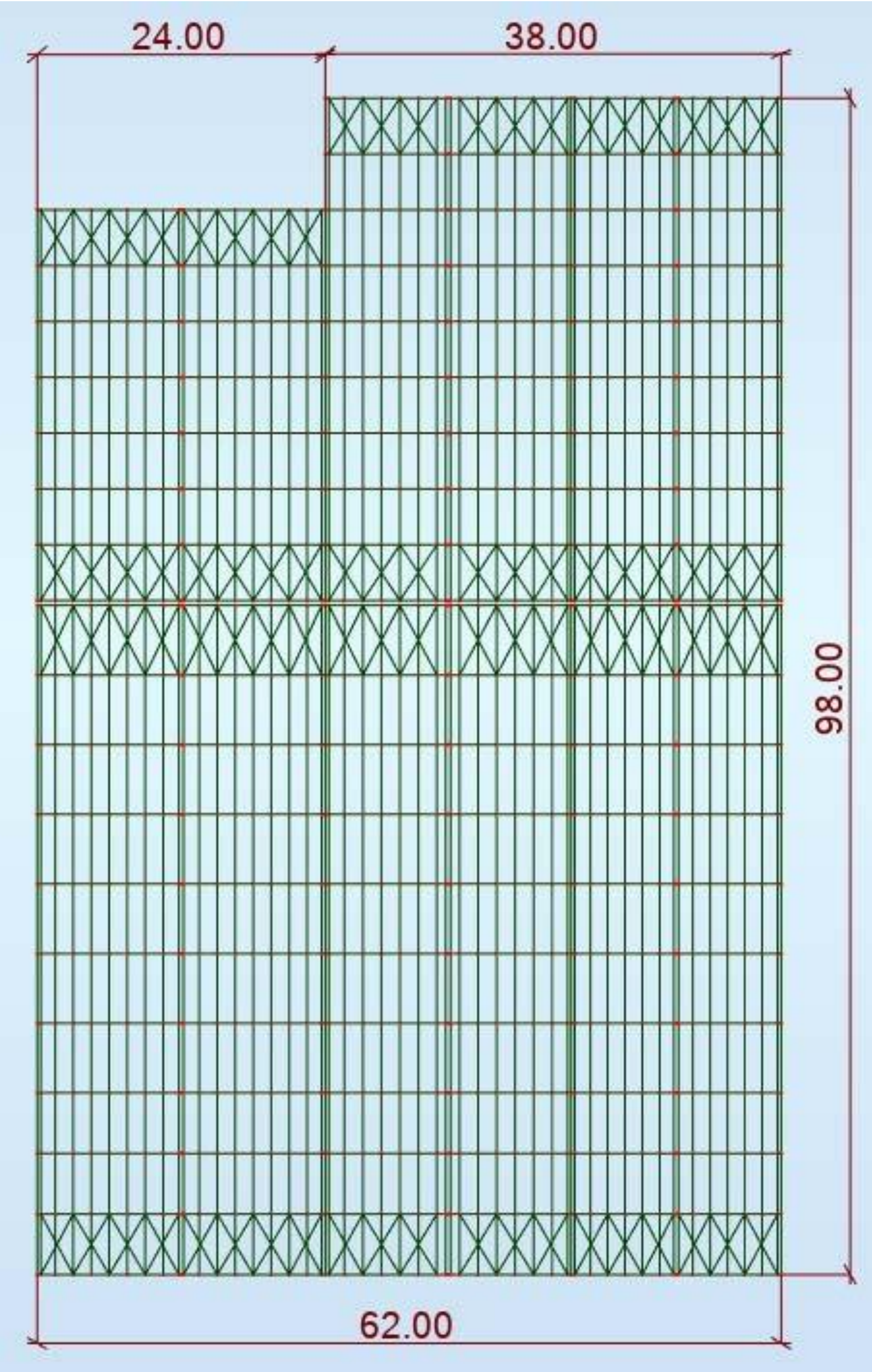
Plan d'installation des équipements



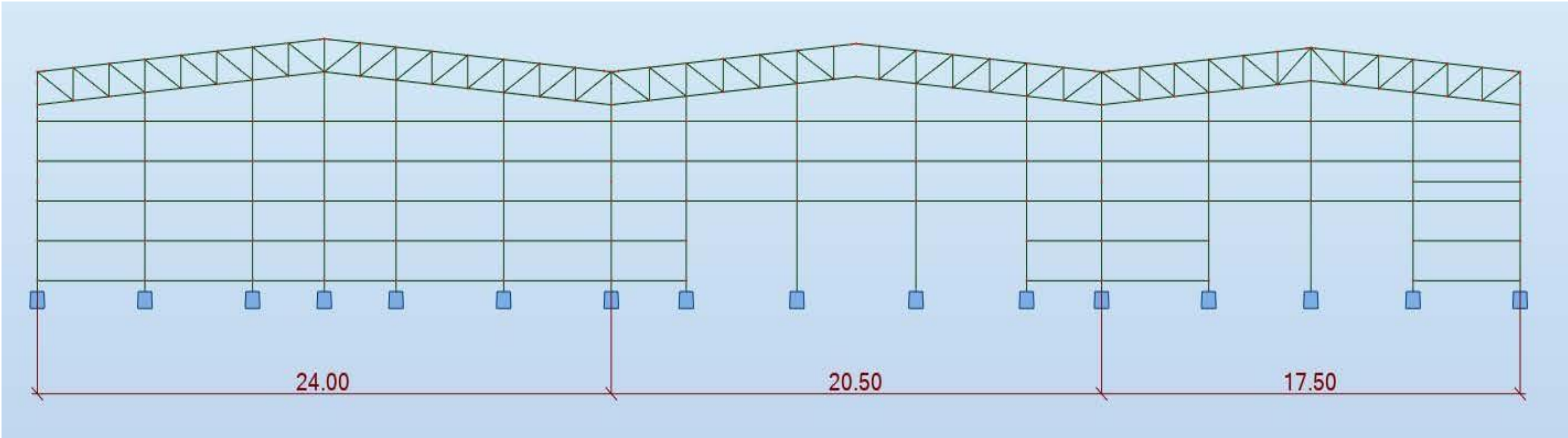
3D model of the cold storage hangar



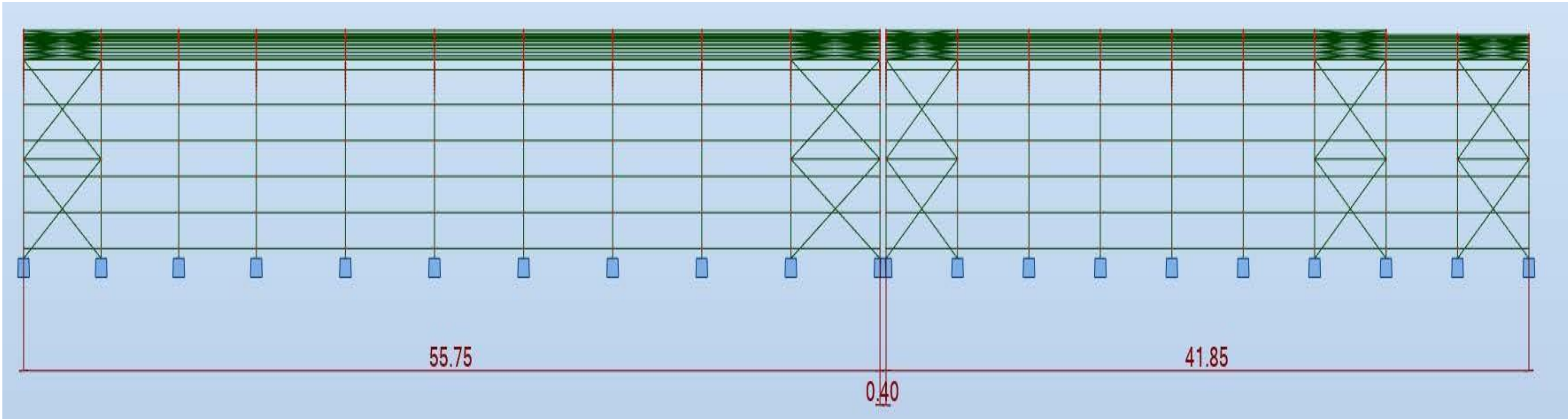
Ramp and vertical bracing of the cold storage hangar



Plan view of the cold storage hangar



Front side



long side with an expantion joint

Pièce	Profil	Matériau	Ratio(uy)	Cas (uy)	Ratio(uz)	Cas (uz)
Famille : 1 poteau						
501	OK SGM 1x1x1	S 235	-	-	-	-
Famille : 2 potelet						
3665	OK sig 300x50x3.5	S 235	-	-	-	-
Famille : 3 panne						
4427	OK Z 180x58x2	S 235	0.24	1 PERM1	0.03	1 PERM1
Famille : 7 sablière						
3026	OK DC 150x58x2	S 235	0.00	1 PERM1	0.01	1 PERM1
Famille : 8 palée stabilité						
4380	OK DC 150x58x2	S 235	0.00	1 PERM1	0.01	1 PERM1
Famille : 9 lisses						
3093	OK z 150x45x2	S 235	0.98	1 PERM1	0.01	1 PERM1

Ratio(vx)	Cas (vx)	Ratio(vy)	Cas (vy)
Famille : 1 poteau			
0.01	1 PERM1	0.00	1 PERM1
Famille : 2 potelet			
0.00	1 PERM1	0.01	1 PERM1
Famille : 3 panne			
-	-	-	-
Famille : 7 sablière			
-	-	-	-
Famille : 8 palée stabilité			
-	-	-	-
Famille : 9 lisses			
-	-	-	-

Pièce	Profil	Matériau	Lay	Laz	Ratio	Cas
Famille : 1 poteau						
405	SGM 1x1x1	Steel	61.12	131.37	0.32	11 COMB5
Famille : 2 potelet						
4345	sig 300x50x3.5	Steel	72.68	169.44	0.07	11 COMB5
Famille : 3 panne						
2488	Z 180x58x2	Steel	80.52	112.71	0.85	12 COMB6
Famille : 4 Membrure Sup et inf						
877	MDC	Steel	150.57	192.44	0.36	8 COMB2
Famille : 6 Contreventement						
3273	m20	Steel	1087.13	1087.13	0.47	8 COMB2
Famille : 7 sablière						
3010	DC 150x58x2	Steel	98.48	69.25	0.18	11 COMB5
Famille : 8 palée stabilité						
3566	DC 150x58x2	Steel	78.95	168.25	0.07	11 COMB5
Famille : 9 lisses						
3095	z 150x45x2	Steel	97.99	141.47	0.55	14 COMB8
Famille : 10 montant et diagonale ferme						
902	c 116x60x2	Steel	20.62	42.01	0.34	7 COMB1

CALCUL DES STRUCTURES ACIER

NORME: *NF EN 1993-1:2005/NA:2007/AC:2009, Eurocode 3: Design of steel structures.*
TYPE D'ANALYSE: Vérification des familles

FAMILLE: 1 poteau
PIECE: 405 **POINT:** 7 **COORDONNEE:** x = 0.85 L = 6.80 m

CHARGEMENTS:
Cas de charge décisif: 11 COMB5 (1+2)*1.35+3*1.05+6*1.50

MATERIAU:
Steel (S235) fy = 235.00 MPa

PARAMETRES DE LA SECTION: SGM 1x1x1

h=35.0 cm	gM0=1.00	gM1=1.00	
b=17.6 cm	Ay=5.12 cm2	Az=26.81 cm2	Ax=46.33 cm2
tw=0.0 cm	Iy=7935.41 cm4	Iz=1717.98 cm4	Ix=3347.01 cm4
tf=0.0 cm	Wply=549.39 cm3	Wplz=255.56 cm3	

EFFORTS INTERNES ET RESISTANCES ULTIMES:

N,Ed = 30.58 kN	My,Ed = 36.44 kN*m	Mz,Ed = 0.05 kN*m	Vy,Ed = 0.01 kN
Nc,Rd = 1088.65 kN	My,Ed,max = 36.44 kN*m	Mz,Ed,max = -0.12 kN*m	Vy,T,Rd = 69.44 kN
Nb,Rd = 486.35 kN	My,c,Rd = 129.11 kN*m	Mz,c,Rd = 60.06 kN*m	Vz,Ed = 8.97 kN
	MN,y,Rd = 129.01 kN*m	MN,z,Rd = 60.01 kN*m	Vz,T,Rd = 363.70 kN
			Tt,Ed = -0.00 kN*m
			Classe de la section = 1

PARAMETRES DE DEVERSEMENT:

PARAMETRES DE FLAMBEMENT:

en y:		en z:	
Ly = 8.00 m	Lam_y = 0.65	Lz = 8.00 m	Lam_z = 1.40
Lcr,y = 8.00 m	Xy = 0.91	Lcr,z = 8.00 m	Xz = 0.45
Lamy = 61.12	kyy = 1.02	Lamz = 131.37	kyz = 0.66

FORMULES DE VERIFICATION:
Contrôle de la résistance de la section:
N,Ed/Nc,Rd = 0.03 < 1.00 (6.2.4.(1))
My,Ed/MN,y,Rd = 0.28 < 1.00 (6.2.9.1.(2))
Mz,Ed/MN,z,Rd = 0.00 < 1.00 (6.2.9.1.(2))
(My,Ed/MN,y,Rd)^ 1.00 + (Mz,Ed/MN,z,Rd)^1.00 = 0.28 < 1.00 (6.2.9.1.(6))
Vy,Ed/Vy,c,Rd = 0.00 < 1.00 (6.2.6.(1))
Vz,Ed/Vz,c,Rd = 0.02 < 1.00 (6.2.6.(1))
Contrôle de la stabilité globale de la barre:
Lambda,y = 61.12 < Lambda,max = 210.00 Lambda,z = 131.37 < Lambda,max = 210.00 STABLE
N,Ed/(Xy*N,Rk/gM1) + kyy*My,Ed,max/(XLT*My,Rk/gM1) + kyz*Mz,Ed,max/(Mz,Rk/gM1) = 0.32 < 1.00 (6.3.3.(4))
N,Ed/(Xz*N,Rk/gM1) + kzy*My,Ed,max/(XLT*My,Rk/gM1) + kzz*Mz,Ed,max/(Mz,Rk/gM1) = 0.23 < 1.00 (6.3.3.(4))

Profil correct !!!

CALCUL DES STRUCTURES ACIER

NORME: *NF EN 1993-1:2005/NA:2007/AC:2009, Eurocode 3: Design of steel structures.*
TYPE D'ANALYSE: Vérification des familles

FAMILLE: 2 potelet
PIECE: 4345 potelet01_4345 **POINT:** 7 **COORDONNEE:** x = 1.00 L = 8.00 m

---

CHARGEMENTS:
Cas de charge décisif: 11 COMB5 (1+2)*1.35+3*1.05+6*1.50

MATERIAU:
Steel (S235) fy = 235.00 MPa

PARAMETRES DE LA SECTION: sig 300x50x3.5

h=30.0 cm	gM0=1.00	gM1=1.00	
b=14.0 cm	Ay=8.29 cm2	Az=18.74 cm2	Ax=29.35 cm2
tw=0.0 cm	Iy=3555.55 cm4	Iz=654.21 cm4	Ix=1245.89 cm4
tf=0.0 cm	Wply=290.70 cm3	Wplz=128.74 cm3	

EFFORTS INTERNES ET RESISTANCES ULTIMES:

N,Ed = 8.72 kN	My,Ed = -0.66 kN*m	Mz,Ed = 0.70 kN*m	Vy,Ed = -0.21 kN
Nc,Rd = 689.66 kN	My,Ed,max = 0.74 kN*m	Mz,Ed,max = 0.70 kN*m	Vy,T,Rd = 112.49 kN
Nb,Rd = 194.51 kN	My,c,Rd = 68.31 kN*m	Mz,c,Rd = 30.25 kN*m	Vz,Ed = -0.27 kN
	MN,y,Rd = 68.30 kN*m	MN,z,Rd = 30.25 kN*m	Vz,T,Rd = 254.32 kN
			Tt,Ed = -0.03 kN*m
			Classe de la section = 1

PARAMETRES DE DEVERSEMENT:

PARAMETRES DE FLAMBEMENT:

en y:		en z:	
Ly = 8.00 m	Lam_y = 0.77	Lz = 8.00 m	Lam_z = 1.80
Lcr,y = 8.00 m	Xy = 0.87	Lcr,z = 8.00 m	Xz = 0.28
Lamy = 72.68	kzy = 0.57	Lamz = 169.44	kzz = 1.00

FORMULES DE VERIFICATION:
Contrôle de la résistance de la section:
N,Ed/Nc,Rd = 0.01 < 1.00 (6.2.4.(1))
My,Ed/MN,y,Rd = 0.01 < 1.00 (6.2.9.1.(2))
Mz,Ed/MN,z,Rd = 0.02 < 1.00 (6.2.9.1.(2))
(My,Ed/MN,y,Rd)^ 1.00 + (Mz,Ed/MN,z,Rd)^1.00 = 0.03 < 1.00 (6.2.9.1.(6))
Vy,Ed/Vy,c,Rd = 0.00 < 1.00 (6.2.6.(1))
Vz,Ed/Vz,c,Rd = 0.00 < 1.00 (6.2.6.(1))
Contrôle de la stabilité globale de la barre:
Lambda,y = 72.68 < Lambda,max = 210.00 Lambda,z = 169.44 < Lambda,max = 210.00 STABLE
N,Ed/(Xy*N,Rk/gM1) + kyy*My,Ed,max/(XLT*My,Rk/gM1) + kyz*Mz,Ed,max/(Mz,Rk/gM1) = 0.04 < 1.00 (6.3.3.(4))
N,Ed/(Xz*N,Rk/gM1) + kzy*My,Ed,max/(XLT*My,Rk/gM1) + kzz*Mz,Ed,max/(Mz,Rk/gM1) = 0.07 < 1.00 (6.3.3.(4))

Profil correct !!!

CALCUL DES STRUCTURES ACIER

NORME: *NF EN 1993-1:2005/NA:2007/AC:2009, Eurocode 3: Design of steel structures.*
TYPE D'ANALYSE: Vérification des familles

FAMILLE: 3 panne
PIECE: 2488 **POINT:** 4 **COORDONNEE:** x = 0.50 L = 2.90 m

CHARGEMENTS:
Cas de charge décisif: 12 COMB6 (1+2)*1.35+6*1.50

MATERIAU:
Steel (S235) fy = 235.00 MPa

PARAMETRES DE LA SECTION: Z 180x58x2

h=20.3 cm	gM0=1.00	gM1=1.00	
b=6.9 cm	Ay=5.50 cm2	Az=3.96 cm2	Ax=6.33 cm2
tw=0.0 cm	Iy=328.46 cm4	Iz=18.25 cm4	Ix=0.09 cm4
tf=0.0 cm	Wply=41.07 cm3	Wplz=9.13 cm3	

EFFORTS INTERNES ET RESISTANCES ULTIMES:

N,Ed = -0.49 kN	My,Ed = 2.13 kN*m	Mz,Ed = 1.34 kN*m	
Nt,Rd = 148.76 kN	My,pl,Rd = 9.65 kN*m	Mz,pl,Rd = 2.15 kN*m	
	My,c,Rd = 9.65 kN*m	Mz,c,Rd = 2.15 kN*m	
	MN,y,Rd = 9.65 kN*m	MN,z,Rd = 2.15 kN*m	
			Tt,Ed = 0.00 kN*m
			Classe de la section = 1

PARAMETRES DE DEVERSEMENT:

PARAMETRES DE FLAMBEMENT:

en y: en z:

FORMULES DE VERIFICATION:
Contrôle de la résistance de la section:
N,Ed/Nt,Rd = 0.00 < 1.00 (6.2.3.(1))
My,Ed/MN,y,Rd = 0.22 < 1.00 (6.2.9.1.(2))
Mz,Ed/MN,z,Rd = 0.63 < 1.00 (6.2.9.1.(2))
(My,Ed/MN,y,Rd)^ 1.00 + (Mz,Ed/MN,z,Rd)^1.00 = 0.85 < 1.00 (6.2.9.1.(6))

Profil correct !!!

CALCUL DES STRUCTURES ACIER

NORME: *NF EN 1993-1:2005/NA:2007/AC:2009, Eurocode 3: Design of steel structures.*
TYPE D'ANALYSE: Vérification des familles

FAMILLE: 4 Membrure Sup et inf
PIECE: 877 **POINT:** 1 **COORDONNEE:** x = 0.00 L = 0.00 m

CHARGEMENTS:

Cas de charge décisif: 8 COMB2 (1+2)*1.35+(3+4)*1.50

MATERIAU:
Steel (S235) fy = 235.00 MPa

PARAMETRES DE LA SECTION: MDC 160x60x2

h=23.7 cm	gM0=1.00	gM1=1.00	
b=16.0 cm	Ay=0.00 cm2	Az=0.00 cm2	Ax=12.27 cm2
tw=0.0 cm	Iy=787.42 cm4	Iz=482.02 cm4	Ix=0.20 cm4
tf=0.0 cm	Wply=96.24 cm3	Wplz=70.55 cm3	

EFFORTS INTERNES ET RESISTANCES ULTIMES:

N,Ed = 105.15 kN	My,Ed = 0.18 kN*m	Mz,Ed = 2.85 kN*m	Vy,Ed = 2.03 kN
Nc,Rd = 288.44 kN	My,pl,Rd = 22.62 kN*m	Mz,pl,Rd = 16.58 kN*m	
Nb,Rd = 288.44 kN	My,c,Rd = 22.62 kN*m	Mz,c,Rd = 16.58 kN*m	Vz,Ed = -0.14 kN
	MN,y,Rd = 19.61 kN*m	MN,z,Rd = 14.38 kN*m	
			Tt,Ed = 0.00 kN*m
			Classe de la section = 1

PARAMETRES DE DEVERSEMENT:

PARAMETRES DE FLAMBEMENT:

en y: en z:

FORMULES DE VERIFICATION:
Contrôle de la résistance de la section:
N,Ed/Nc,Rd = 0.36 < 1.00 (6.2.4.(1))
My,Ed/MN,y,Rd = 0.01 < 1.00 (6.2.9.1.(2))
Mz,Ed/MN,z,Rd = 0.20 < 1.00 (6.2.9.1.(2))
(My,Ed/MN,y,Rd)^ 1.00 + (Mz,Ed/MN,z,Rd)^1.00 = 0.21 < 1.00 (6.2.9.1.(6))

Profil correct !!!

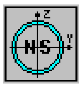
CALCUL DES STRUCTURES ACIER

NORME: *NF EN 1993-1:2005/NA:2007/AC:2009, Eurocode 3: Design of steel structures.*
TYPE D'ANALYSE: Vérification des familles

FAMILLE: 6 Contreventement
PIECE: 3273 contreventement_3273 **POINT:** 7 **COORDONNEE:** x = 0.54 L = 2.91 m

CHARGEMENTS:
Cas de charge décisif: 8 COMB2 (1+2)*1.35+(3+4)*1.50

MATERIAU:
Steel (S235) fy = 235.00 MPa



PARAMETRES DE LA SECTION: m20

h=2.0 cm	gM0=1.00	gM1=1.00	
	Ay=2.00 cm2	Az=2.00 cm2	Ax=3.14 cm2
tw=1.0 cm	Iy=0.79 cm4	Iz=0.79 cm4	Ix=1.57 cm4

	Wply=1.33 cm3	Wplz=1.33 cm3	

EFFORTS INTERNES ET RESISTANCES ULTIMES:

N,Ed = -2.33 kN	My,Ed = 0.01 kN*m	Mz,Ed = 0.15 kN*m	Vy,Ed = -1.05 kN
Nt,Rd = 73.83 kN	My,pl,Rd = 0.31 kN*m	Mz,pl,Rd = 0.31 kN*m	Vy,T,Rd = 26.05 kN
	My,c,Rd = 0.31 kN*m	Mz,c,Rd = 0.31 kN*m	Vz,Ed = 0.11 kN
	MN,y,Rd = 0.31 kN*m	MN,z,Rd = 0.31 kN*m	Vz,T,Rd = 26.05 kN
			Tt,Ed = -0.01 kN*m
			Classe de la section = 1

PARAMETRES DE DEVERSEMENT:

PARAMETRES DE FLAMBEMENT:

en y: en z:

FORMULES DE VERIFICATION:

Contrôle de la résistance de la section:

N,Ed/Nt,Rd = 0.03 < 1.00 (6.2.3.(1))
My,Ed/MN,y,Rd = 0.04 < 1.00 (6.2.9.1.(2))
Mz,Ed/MN,z,Rd = 0.47 < 1.00 (6.2.9.1.(2))
(My,Ed/MN,y,Rd)^ 2.00 + (Mz,Ed/MN,z,Rd)^2.00 = 0.23 < 1.00 (6.2.9.1.(6))
Vy,Ed/Vy,T,Rd = 0.04 < 1.00 (6.2.6-7)
Vz,Ed/Vz,T,Rd = 0.00 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.04 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.04 < 1.00 (6.2.6)

Profil correct !!!

CALCUL DES STRUCTURES ACIER

NORME: *NF EN 1993-1:2005/NA:2007/AC:2009, Eurocode 3: Design of steel structures.*
TYPE D'ANALYSE: Vérification des familles

FAMILLE: 7 sablière
PIECE: 3010 **POINT:** 1 **COORDONNEE:** x = 0.00 L = 0.00 m

CHARGEMENTS:
Cas de charge décisif: 11 COMB5 (1+2)*1.35+3*1.05+6*1.50

MATERIAU:
Steel (S235) fy = 235.00 MPa

PARAMETRES DE LA SECTION: DC 150x58x2

h=15.0 cm	gM0=1.00	gM1=1.00	
b=11.6 cm	Ay=4.58 cm2	Az=6.17 cm2	Ax=11.47 cm2
tw=0.0 cm	Iy=397.86 cm4	Iz=87.61 cm4	Ix=0.41 cm4
tf=0.0 cm	Wply=62.00 cm3	Wplz=20.29 cm3	

EFFORTS INTERNES ET RESISTANCES ULTIMES:

N,Ed = 0.07 kN	My,Ed = -1.18 kN*m	Mz,Ed = -0.46 kN*m	Vy,Ed = -0.46 kN
Nc,Rd = 269.55 kN	My,Ed,max = -1.18 kN*m	Mz,Ed,max = -0.46 kN*m	Vy,T,Rd = 62.11 kN
Nb,Rd = 237.65 kN	My,c,Rd = 14.57 kN*m	Mz,c,Rd = 4.77 kN*m	Vz,Ed = 1.20 kN
	MN,y,Rd = 14.57 kN*m	MN,z,Rd = 4.77 kN*m	Vz,T,Rd = 83.70 kN

Tt,Ed = -0.00 kN*m
Classe de la section = 1

PARAMETRES DE DEVERSEMENT:

PARAMETRES DE FLAMBEMENT:

en y:		en z:	
		Lz = 5.80 m	Lam_z = 0.74
		Lcr,z = 1.91 m	Xz = 0.88
	kzy = 0.56	Lamz = 69.25	kzz = 1.00

FORMULES DE VERIFICATION:

Contrôle de la résistance de la section:

N,Ed/Nc,Rd = 0.00 < 1.00 (6.2.4.(1))
My,Ed/MN,y,Rd = 0.08 < 1.00 (6.2.9.1.(2))
Mz,Ed/MN,z,Rd = 0.10 < 1.00 (6.2.9.1.(2))
(My,Ed/MN,y,Rd)^ 1.00 + (Mz,Ed/MN,z,Rd)^1.00 = 0.18 < 1.00 (6.2.9.1.(6))
Vy,Ed/Vy,c,Rd = 0.01 < 1.00 (6.2.6.(1))
Vz,Ed/Vz,c,Rd = 0.01 < 1.00 (6.2.6.(1))

Contrôle de la stabilité globale de la barre:

Lambda,z = 69.25 < Lambda,max = 210.00 STABLE
N,Ed/(Xz*N,Rk/gM1) + kzy*My,Ed,max/(XLT*My,Rk/gM1) + kzz*Mz,Ed,max/(Mz,Rk/gM1) = 0.14 < 1.00 (6.3.3.(4))

Profil correct !!!

CALCUL DES STRUCTURES ACIER

NORME: *NF EN 1993-1:2005/NA:2007/AC:2009, Eurocode 3: Design of steel structures.*
TYPE D'ANALYSE: Vérification des familles

FAMILLE: 8 palée stabilité
PIECE: 3566 Poutre_3566 **POINT:** 7 **COORDONNEE:** x = 1.00 L = 4.65 m

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CHARGEMENTS:
Cas de charge décisif: 11 COMB5 (1+2)*1.35+3*1.05+6*1.50

MATERIAU:
Steel (S235) fy = 235.00 MPa

PARAMETRES DE LA SECTION: DC 150x58x2

h=15.0 cm	gM0=1.00	gM1=1.00	
b=11.6 cm	Ay=4.58 cm2	Az=6.17 cm2	Ax=11.47 cm2
tw=0.0 cm	Iy=397.86 cm4	Iz=87.61 cm4	Ix=0.41 cm4
tf=0.0 cm	Wply=62.00 cm3	Wplz=20.29 cm3	

EFFORTS INTERNES ET RESISTANCES ULTIMES:

N,Ed = -0.83 kN	My,Ed = -0.26 kN*m	Mz,Ed = -0.27 kN*m	Vy,Ed = 0.16 kN
Nt,Rd = 269.55 kN	My,pl,Rd = 14.57 kN*m	Mz,pl,Rd = 4.77 kN*m	Vy,T,Rd = 62.11 kN
	My,c,Rd = 14.57 kN*m	Mz,c,Rd = 4.77 kN*m	Vz,Ed = -0.31 kN
	MN,y,Rd = 14.57 kN*m	MN,z,Rd = 4.77 kN*m	Vz,T,Rd = 83.70 kN
			Tt,Ed = -0.00 kN*m
			Classe de la section = 1

PARAMETRES DE DEVERSEMENT:

PARAMETRES DE FLAMBEMENT:

en y: en z:

FORMULES DE VERIFICATION:

Contrôle de la résistance de la section:

N,Ed/Nt,Rd = 0.00 < 1.00 (6.2.3.(1))
My,Ed/MN,y,Rd = 0.02 < 1.00 (6.2.9.1.(2))
Mz,Ed/MN,z,Rd = 0.06 < 1.00 (6.2.9.1.(2))
(My,Ed/MN,y,Rd)^ 1.00 + (Mz,Ed/MN,z,Rd)^1.00 = 0.07 < 1.00 (6.2.9.1.(6))
Vy,Ed/Vy,c,Rd = 0.00 < 1.00 (6.2.6.(1))
Vz,Ed/Vz,c,Rd = 0.00 < 1.00 (6.2.6.(1))

Profil correct !!!

CALCUL DES STRUCTURES ACIER

NORME: *NF EN 1993-1:2005/NA:2007/AC:2009, Eurocode 3: Design of steel structures.*
TYPE D'ANALYSE: Vérification des familles

FAMILLE: 9 lisses
PIECE: 3095 **POINT:** 4 **COORDONNEE:** x = 0.50 L = 2.90 m

CHARGEMENTS:
Cas de charge décisif: 14 COMB8 (1+2)*1.00+6*1.50

MATERIAU:
Steel (S235) fy = 235.00 MPa

PARAMETRES DE LA SECTION: z 150x45x2

h=16.7 cm	gM0=1.00	gM1=1.00	
b=5.4 cm	Ay=4.55 cm2	Az=3.26 cm2	Ax=5.22 cm2
tw=0.0 cm	Iy=183.06 cm4	Iz=9.56 cm4	Ix=0.08 cm4
tf=0.0 cm	Wply=27.89 cm3	Wplz=6.02 cm3	

EFFORTS INTERNES ET RESISTANCES ULTIMES:

N,Ed = -0.05 kN	My,Ed = 0.29 kN*m	Mz,Ed = 0.71 kN*m	
Nt,Rd = 122.79 kN	My,pl,Rd = 6.55 kN*m	Mz,pl,Rd = 1.42 kN*m	
	My,c,Rd = 6.55 kN*m	Mz,c,Rd = 1.42 kN*m	
	MN,y,Rd = 6.55 kN*m	MN,z,Rd = 1.42 kN*m	
			Tt,Ed = -0.00 kN*m
			Classe de la section = 1

PARAMETRES DE DEVERSEMENT:

PARAMETRES DE FLAMBEMENT:

en y: en z:

FORMULES DE VERIFICATION:

Contrôle de la résistance de la section:
N,Ed/Nt,Rd = 0.00 < 1.00   (6.2.3.(1))
My,Ed/MN,y,Rd = 0.04 < 1.00   (6.2.9.1.(2))
Mz,Ed/MN,z,Rd = 0.50 < 1.00   (6.2.9.1.(2))
(My,Ed/MN,y,Rd)^ 1.00 + (Mz,Ed/MN,z,Rd)^1.00 = 0.55 < 1.00   (6.2.9.1.(6))

Profil correct !!!

CALCUL DES STRUCTURES ACIER

NORME: *NF EN 1993-1:2005/NA:2007/AC:2009, Eurocode 3: Design of steel structures.*
TYPE D'ANALYSE: Vérification des familles

FAMILLE: 10 montant et diagonale ferme
PIECE: 902 **POINT:** 1 **COORDONNEE:** x = 0.00 L = 0.00 m

CHARGEMENTS:
Cas de charge décisif: 7 COMB1 (1+2)*1.35+3*1.50+6*0.90

MATERIAU:
Steel (S235)   fy = 235.00 MPa

PARAMETRES DE LA SECTION: **c 116x60x2**

h=11.6 cm	gM0=1.00	gM1=1.00	
b=6.0 cm	Ay=2.32 cm2	Az=2.53 cm2	Ax=5.24 cm2
tw=0.0 cm	Iy=113.55 cm4	Iz=27.36 cm4	Ix=0.08 cm4
tf=0.0 cm	Wply=22.65 cm3	Wplz=10.39 cm3	

EFFORTS INTERNES ET RESISTANCES ULTIMES:

N,Ed = 26.76 kN	My,Ed = 0.00 kN*m	Mz,Ed = 0.29 kN*m	Vy,Ed = 0.49 kN
Nc,Rd = 123.14 kN	My,Ed,max = 0.08 kN*m	Mz,Ed,max = -0.30 kN*m	Vy,T,Rd = 31.47 kN
Nb,Rd = 118.43 kN	My,c,Rd = 5.32 kN*m	Mz,c,Rd = 2.44 kN*m	Vz,Ed = 0.07 kN
	MN,y,Rd = 5.07 kN*m	MN,z,Rd = 2.33 kN*m	Vz,T,Rd = 34.29 kN
			Tt,Ed = -0.00 kN*m
			Classe de la section = 1

PARAMETRES DE DEVERSEMENT:

PARAMETRES DE FLAMBEMENT:

en y:		en z:	
Ly = 1.20 m	Lam_y = 0.22	Lz = 1.20 m	Lam_z = 0.45
Lcr,y = 0.96 m	Xy = 1.00	Lcr,z = 0.96 m	Xz = 0.96
Lamy = 20.62	kzy = 0.41	Lamz = 42.01	kzz = 0.89

FORMULES DE VERIFICATION:
Contrôle de la résistance de la section:
N,Ed/Nc,Rd = 0.22 < 1.00   (6.2.4.(1))
My,Ed/MN,y,Rd = 0.00 < 1.00   (6.2.9.1.(2))
Mz,Ed/MN,z,Rd = 0.13 < 1.00   (6.2.9.1.(2))
(My,Ed/MN,y,Rd)^ 1.00 + (Mz,Ed/MN,z,Rd)^1.00 = 0.13 < 1.00   (6.2.9.1.(6))
Vy,Ed/Vy,c,Rd = 0.02 < 1.00   (6.2.6.(1))
Vz,Ed/Vz,c,Rd = 0.00 < 1.00   (6.2.6.(1))
Contrôle de la stabilité globale de la barre:
Lambda,y = 20.62 < Lambda,max = 210.00   Lambda,z = 42.01 < Lambda,max = 210.00   STABLE

N,Ed/(Xy*N,Rk/gM1) + kyy*My,Ed,max/(XLT*My,Rk/gM1) + kyz*Mz,Ed,max/(Mz,Rk/gM1) = 0.30 < 1.00 (6.3.3.(4))
N,Ed/(Xz*N,Rk/gM1) + kzy*My,Ed,max/(XLT*My,Rk/gM1) + kzz*Mz,Ed,max/(Mz,Rk/gM1) = 0.34 < 1.00 (6.3.3.(4))

Profil correct !!!

CALCUL DES STRUCTURES ACIER

NORME: *NF EN 1993-1:2005/NA:2007/AC:2009, Eurocode 3: Design of steel structures.*
TYPE D'ANALYSE: Vérification des familles

FAMILLE: 1 poteau
PIECE: 501 **POINT:** **COORDONNEE:**

PARAMETRES DE LA SECTION: SGM 1x1x1

ht=35.0 cm			
bf=17.6 cm	Ay=5.12 cm2	Az=26.81 cm2	Ax=46.33 cm2
ea=0.0 cm	Iy=7935.41 cm4	Iz=1717.98 cm4	Ix=3347.01 cm4
es=0.0 cm	Wely=453.45 cm3	Welz=195.23 cm3	

DEPLACEMENTS LIMITES

Flèches (REPERE LOCAL): *Non analysé*

Déplacements (REPERE GLOBAL):
vx = 0.1 cm < vx max = L/150.00 = 5.3 cm Vérifié
Cas de charge décisif: 1 PERM1
vy = 0.0 cm < vy max = L/150.00 = 5.3 cm Vérifié
Cas de charge décisif: 1 PERM1

Profil correct !!!

CALCUL DES STRUCTURES ACIER

NORME: *NF EN 1993-1:2005/NA:2007/AC:2009, Eurocode 3: Design of steel structures.*
TYPE D'ANALYSE: Vérification des familles

FAMILLE: 2 potelet
PIECE: 3665 **POINT:** **COORDONNEE:**

PARAMETRES DE LA SECTION: sig 300x50x3.5

ht=30.0 cm			
bf=14.0 cm	Ay=8.29 cm2	Az=18.74 cm2	Ax=29.35 cm2
ea=0.0 cm	Iy=3555.55 cm4	Iz=654.21 cm4	Ix=1245.89 cm4
es=0.0 cm	Wely=237.04 cm3	Welz=93.46 cm3	

DEPLACEMENTS LIMITES

Flèches (REPERE LOCAL): *Non analysé*

Déplacements (REPERE GLOBAL):
vx = 0.0 cm < vx max = L/150.00 = 4.8 cm Vérifié
Cas de charge décisif: 1 PERM1
vy = 0.0 cm < vy max = L/150.00 = 4.8 cm Vérifié
Cas de charge décisif: 1 PERM1

Profil correct !!!

CALCUL DES STRUCTURES ACIER

NORME: *NF EN 1993-1:2005/NA:2007/AC:2009, Eurocode 3: Design of steel structures.*
TYPE D'ANALYSE: Vérification des familles

FAMILLE: 3 panne
PIECE: 4427 **POINT:** **COORDONNEE:**

PARAMETRES DE LA SECTION: **Z 180x58x2**

ht=20.3 cm			
bf=6.9 cm	Ay=5.50 cm2	Az=3.96 cm2	Ax=6.33 cm2
ea=0.0 cm	Iy=328.46 cm4	Iz=18.25 cm4	Ix=0.09 cm4
es=0.0 cm	Wely=32.30 cm3	Welz=5.26 cm3	

DEPLACEMENTS LIMITES

Flèches (REPERE LOCAL):
uy = 0.7 cm < uy max = L/200.00 = 2.9 cm Vérifié
Cas de charge décisif: 1 PERM1
uz = 0.1 cm < uz max = L/200.00 = 2.9 cm Vérifié
Cas de charge décisif: 1 PERM1

Déplacements (REPERE GLOBAL): *Non analysé*

Profil correct !!!

CALCUL DES STRUCTURES ACIER

NORME: *NF EN 1993-1:2005/NA:2007/AC:2009, Eurocode 3: Design of steel structures.*
TYPE D'ANALYSE: Vérification des familles

FAMILLE: 7 sablière
PIECE: 3026 **POINT:** **COORDONNEE:**

PARAMETRES DE LA SECTION: **DC 150x58x2**

ht=15.0 cm			
bf=11.6 cm	Ay=4.58 cm2	Az=6.17 cm2	Ax=11.47 cm2
ea=0.0 cm	Iy=397.86 cm4	Iz=87.61 cm4	Ix=0.41 cm4
es=0.0 cm	Wely=53.05 cm3	Welz=15.11 cm3	

DEPLACEMENTS LIMITES

Flèches (REPERE LOCAL):
uy = 0.0 cm < uy max = L/200.00 = 2.9 cm Vérifié
Cas de charge décisif: 1 PERM1
uz = 0.0 cm < uz max = L/200.00 = 2.9 cm Vérifié
Cas de charge décisif: 1 PERM1

Déplacements (REPERE GLOBAL): *Non analysé*

Profil correct !!!

CALCUL DES STRUCTURES ACIER

NORME: *NF EN 1993-1:2005/NA:2007/AC:2009, Eurocode 3: Design of steel structures.*
TYPE D'ANALYSE: Vérification des familles

FAMILLE: 8 palée stabilité
PIECE: 4380 **POINT:** **COORDONNEE:**

PARAMETRES DE LA SECTION: DC 150x58x2

ht=15.0 cm

bf=11.6 cm	Ay=4.58 cm2	Az=6.17 cm2	Ax=11.47 cm2
ea=0.0 cm	Iy=397.86 cm4	Iz=87.61 cm4	Ix=0.41 cm4
es=0.0 cm	Wely=53.05 cm3	Welz=15.11 cm3	

DEPLACEMENTS LIMITES

Flèches (REPERE LOCAL):

uy = 0.0 cm < uy max = L/200.00 = 2.9 cm Vérifié

Cas de charge décisif: 1 PERM1

uz = 0.0 cm < uz max = L/200.00 = 2.9 cm Vérifié

Cas de charge décisif: 1 PERM1

Déplacements (REPERE GLOBAL): *Non analysé*

Profil correct !!!

CALCUL DES STRUCTURES ACIER

NORME: *NF EN 1993-1:2005/NA:2007/AC:2009, Eurocode 3: Design of steel structures.*

TYPE D'ANALYSE: Vérification des familles

FAMILLE: 9 lisses

PIECE: 3093 **POINT:** **COORDONNEE:**

PARAMETRES DE LA SECTION: z 150x45x2

ht=16.7 cm

bf=5.4 cm	Ay=4.55 cm2	Az=3.26 cm2	Ax=5.22 cm2
ea=0.0 cm	Iy=183.06 cm4	Iz=9.56 cm4	Ix=0.08 cm4
es=0.0 cm	Wely=21.93 cm3	Welz=3.51 cm3	

DEPLACEMENTS LIMITES

Flèches (REPERE LOCAL):

uy = 2.8 cm < uy max = L/200.00 = 2.9 cm Vérifié

Cas de charge décisif: 1 PERM1

uz = 0.0 cm < uz max = L/200.00 = 2.9 cm Vérifié

Cas de charge décisif: 1 PERM1

Déplacements (REPERE GLOBAL): *Non analysé*

Profil correct !!!

Efforts - Cas: 7A14
Extrêmes globaux
1

- Cas: 7A14

Filtre	Barre	Cas
Liste complète	2A47 77A84 87A	1A18
Sélection	2A47 77A84 87A	7A14
Nombre total	3478	18
Nombre sélectionné	3478	8

- Cas: 7A14

	FX [kN]	FY [kN]	FZ [kN]	MX [kNm]	MY [kNm]
MAX	105,15	5,48	12,02	0,31	13,51
Barre	877	1811	594	4343	594
Noeud	987	2029	671	1948	670
Cas	8 (C)	8 (C)	8 (C)	8 (C)	8 (C)
MIN	-70,82	-6,33	-26,39	-0,53	-26,36
Barre	504	1808	405	1708	405
Noeud	570	2028	457	1915	456
Cas	12 (C)	8 (C)	12 (C)	12 (C)	11 (C)

	MZ [kNm]
MAX	3,42
Barre	411
Noeud	465
Cas	11 (C)
MIN	-2,85
Barre	1810
Noeud	2029
Cas	8 (C)

Contraintes - Cas: 7A14
Extrêmes globaux
1

- Cas: 7A14

Filtre	Barre	Cas
Liste complète	2A47 77A84 87A	1A18
Sélection	2A47 77A84 87A	7A14
Nombre total	3478	18
Nombre sélectionné	3478	8

- Cas: 7A14

	S max [MPa]	S min [MPa]	S max(My) [MPa]	S max(Mz) [MPa]	S min(My) [MPa]
MAX	135,69	60,00	82,01	56,83	0,00
Barre	877	782	3525	4389	1990
Noeud	987	880	1101	3774	171
Cas	8 (C)	8 (C)	12 (C)	13 (C)	11 (C)
MIN	-70,29	-109,58	-0,00	-0,00	-82,01
Barre	529	875	3714	3212	3525
Noeud	563	979	3026	2808	1101
Cas	8 (C)	12 (C)	13 (C)	13 (C)	12 (C)

	S min(Mz) [MPa]	Fx/Ax [MPa]
MAX	0,00	85,67
Barre	3212	877
Noeud	2808	987
Cas	13 (C)	8 (C)
MIN	-56,83	-75,08
Barre	4389	531
Noeud	3774	593
Cas	13 (C)	8 (C)

Réactions Repère global - Cas: 7A14
Extrêmes globaux
1

Repère global - Cas: 7A14

Filtre	Noeud	Cas
Liste complète	1A304 306 307A349P14 309A319 324A3	1A18
Sélection	1A7P2 144A150P2 248A254P2 352A358	7A14
Nombre total	2810	18
Nombre sélectior	106	8

Repère global - Cas: 7A14

	FX [kN]	FY [kN]	FZ [kN]	MX [kNm]	MY [kNm]
MAX	9,32	1,97	68,89	1,03	26,36
Noeud	456	1188	458	3727	456
Cas	11 (C)	11 (C)	8 (C)	11 (C)	11 (C)
MIN	-4,55	-2,41	2,10	-0,84	-13,51
Noeud	670	1082	3719	2975	670
Cas	8 (C)	8 (C)	10 (C)	11 (C)	8 (C)

	MZ [kNm]
MAX	0,23
Noeud	1914
Cas	11 (C)
MIN	-0,22
Noeud	1080
Cas	7 (C)