

PEOPLE'S DEMOCRATIC REPUBLIC OF ALGERIA
MINISTRY OF HIGHER EDUCATION AND SCIENTIFIC RESEARCH
UNIVERSITY OF AKLI MOHAND OULHADJ-BOUIRA



Thesis statement for the diploma of

MASTER

Faculty of applied sciences

Civil Engineering course

Structural specialty

Theme:

**STUDY OF A STEEL STRUCTURE
WITH THE EFFECT OF TEMPERATURE TAKEN INTO ACCOUNT**

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Academic year 2024/2025

بِسْمِ اللَّهِ الرَّحْمَنِ الرَّحِيمِ

Thank you

First of all, we would like to express our gratitude to **Allah**, who has accompanied us throughout our years of study and has given us the determination, perseverance, and courage necessary to carry out this modest work.

We would like to express our deep thanks to our supervisor, Mr. **Arbaoui**, for his patience, thoughtful advice, and attentive listening, as well as his efforts in supervising this work.

We would like to thank all the jury members for evaluating our work.

We also wish to express our gratitude to all those who participated, directly or indirectly, in the development of this thesis.

DEDICATION

Praise be to God who teaches with the pen, guides to knowledge, and inspires
patience until the end of the road.

To the first person who instilled in me the love of learning and ignited in my heart
the flame of ambition,

To my honourable parents,
the crown of my head, the source of my giving, and my support in life.

To you, I present the fruit of years of toil and diligence, a token of love and loyalty
that words cannot do justice to.

To my brothers and family, you are the warmth of my heart, and behind every
achievement is your prayers and patience.

To my friends and companions on the path,
who shared moments of challenge and triumph, and you were the salt and light of
the journey.

To my esteemed professors,
who illuminated the darkness of ignorance with the light of their knowledge and
the generosity of their guidance.

I dedicate this memorandum to all of you, as it is the culmination of a joint effort
and a sign of gratitude and sincere gratitude.

DEDICATION

To the man whose name I proudly bear, my dear father, may God protect him and prolong his life.

To my angel in life, the symbol of love and tenderness, my mother, may God have mercy on her and make her grave a garden among the gardens of Paradise.

To my second mother, who did everything in her power to see me reach the highest ranks, I dedicate my graduation to you.

To my true friends, who have been a light in the darkness of the path and a smile in the midst of trials.

To my honourable family, who have been my elite and the source of my strength, I dedicate my graduation to you.

Résumé

La structure étudiée sera construite dans le sud du pays, dans la région de Biskra. Il s'agit d'un travail ayant pour objectif le dimensionnement des éléments métalliques porteurs et secondaires.

Elle est couverte par une toiture à deux versants symétriques avec des ouvertures. Cette toiture est réalisée en panneaux sandwich avec une âme injectée en polyuréthane, tout comme les parois extérieures.

Les calculs ont été effectués conformément à la réglementation en vigueur, aussi bien pour les charges que pour les hypothèses de calcul.

Les calculs ont été réalisés à l'aide d'outils avancés tels que le logiciel de calcul par éléments finis STRUCTURAL ROBOT.

Par la suite, le calcul et le dimensionnement des assemblages et des connexions des éléments de la structure, ainsi que l'étude sismique, ont été réalisés avec le même code de calcul.

Abstract

The structure under study will be built in the southern part of Biskra region. The project involves sizing the load bearing and secondary metal elements.

Covered by a symmetrical gable roof with openings, this roof consists of sandwich panels with injected polyurethane cores, as well as the exterior walls.

Calculations were performed in accordance with current regulations, both for loads and design assumptions.

The calculations were performed using advanced tools such as finite element calculation ROBOT STRUCTURAL ANALYSIS 2024.

The calculations and sizing of the assemblies and the connection of the assembled elements, as well as the seismic study, were then carried out using the same calculation code.

ملخص

سيتم بناء الهيكل قيد الدراسة في البلاد، في منطقة بسكرة، ويشمل المشروع تحديد أبعاد العناصر المعدنية والحاملة والثانوية يُغطى هذا السقف بسقف مائل متمائل يحتوي على فتحات، ويتكوّن من ألواح ذات نواة من البولي يوريثان المحقون، بالإضافة إلى الجدران الخارجية

تم إجراء الحسابات وفقاً للأنظمة المعمول بها حالياً، سواء من حيث الاحمال أو فرضيات التصميم

تم تنفيذ الحسابات باستخدام أدوات متقدمة مثل برنامج العناصر المحدودة
ROBOT STRUCTURAL ANALYSIS 2024.

بعد ذلك، تم حساب وتحديد أبعاد التجميعات والوصلات بين العناصر المُجمعة، بالإضافة إلى دراسة الزلازل، باستخدام نفس برنامج الحساب

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GENERAL INTRODUCTION

As part of our training in Civil Engineering specialising in "Structure" at the Akli Mohand Oulhadj University in Bouira, we have to carry out the study of a metal frame hangar in relation to our professional ambitions of mastering the calculation and design of metal structures.

Our project consists of studying the framework of a metal frame hangar for industrial use

Steel construction offers several advantages over concrete construction, such as

- The mechanical properties of steel make it possible to cover long spans, thus providing large free areas that are particularly advantageous for industries.
- Installation is quickly carried out via bolting or welding.
- It is possible to modify the structure.
- The material has good resistance to earthquakes due to its ductility and lightness.

These advantages make steel the preferred material, especially for industrial warehouses.

Our end-of-studies project consists of putting into practice and deepening the knowledge acquired during our training and this by complying with the construction standards in force in Algeria. The use of calculation tools such as calculation codes is highly recommended, if not essential, in the majority of cases of calculation and design.

Our work is organized into ten (10) chapters after the introduction.

CHAPTER I

PRESENTATION OF THE WORK

I.1 Introduction:

As far as the field of metal frame buildings is concerned. In the first place, once the requirements of the project owner are known, it is conceivable that the structure.

Our construction is located in the south of Algeria in the wilaya of Biskra, where the conditions are different from those existing in the north of the country, because the temperature of exposure and of the structures, can reach records on the day, according to the seasons, and the night can go down, to negative values.

I.2 Presentation of project:

The framework will be located in Biskra. It has a surface area of 1188m², 33m long, 36m wide and 6m for the space between the porticoes, with a 10m high pole.

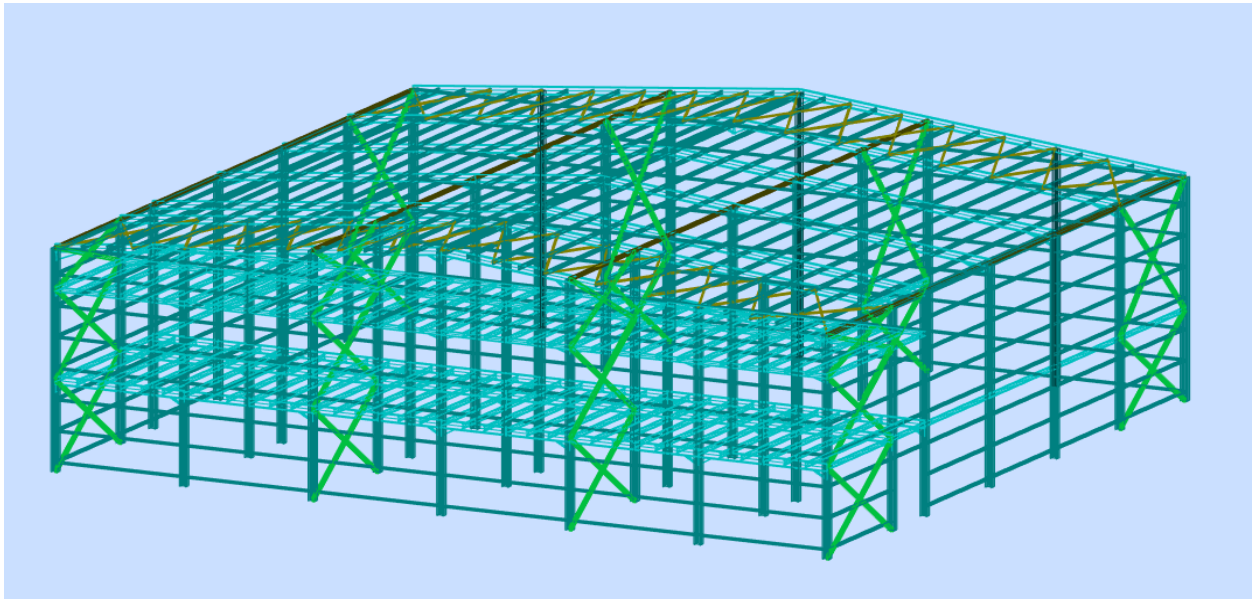


FIGURE I 1 Modeling of the framework

I.2.1 Project location:

The framework is in the wilaya of Biskra, classified according to the Algerian seismic regulation (RPA2024) as a seismicity zone III.

I.2.3 Site Data:

- Altitude: 87 m.
- Seismic zone (III) according to the classification established by RPA 2024.
- Snow zone (C) according to DTR C2-47 RNV version 2013.
- Wind zone (III) according to DTR C2-47 RNV version 2013.
- Allowable soil pressure $\sigma_{\text{soil}} = 1.5$ bars (from the geotechnical soil report).
- Site category (S3) (soft soil).

I.2.4 Regulations Used:

The technical regulations used in this study are:

- EUROCODE 3: Design of steel structures.
- DTR.BC.2.2: Permanent and imposed loads.
- DTR.BC.2.4: Design rules for steel structures "CCM97".
- DTR. C.2.47: Snow and Wind Regulations "RNV2013".
- BAEL91: Design of concrete structures.
- RPA 2024: Algerian seismic regulations version 2024.

I.3 Structural Elements:

- Columns: HEA
- Small columns: HEA
- Purlins: IPE
- Eave purlins: HEA
- Cladding rail: UPN
- Bracings: Angle sections
- Cladding: Sandwich panels

I.4 Materials Used:

For our project, the following construction materials were chosen:

I.4.1 Steel:

Steel is mainly composed of iron and a small amount of carbon, extracted from natural raw materials such as iron ore and carbon.

Carbon makes up only a tiny part of the composition of steel, generally less than 1%.

In addition to iron and carbon, steel may contain other associated elements, such as phosphorus and sulfur, which are considered impurities that alter the properties of steel.

Certain elements like silicon manganese nickel chromium tungsten and vanadium can be deliberately added to steel to improve its mechanical characteristics such as tensile strength hardness yield strength ductility toughness weldability corrosion resistance etc.

I.4.1.1 Properties of Steel:

- Strength.
- Ductility.

I.4.1.2 Mechanical Properties of Steel (§ 3.2.3 CCM97):

- $F_y = 235 \text{ N/mm}^2$
- $F_u = 430 \text{ N/mm}^2$
- $G = 81,000 \text{ N/mm}^2$
- ρ (density) = $78,500 \text{ N/m}^3$
- E (Young's modulus) = $210,000 \text{ N/mm}^2$
- ν (Poisson's ratio) = 0.3
- αT (thermal expansion coefficient) = $10^{-5} / ^\circ\text{C}$

I.4.2 Concrete:

In our structure, the concrete is used for the foundations.

It is a construction material composed of aggregates, sand, cement, water, and possibly additives to modify its properties.

I.4.2.1 Characteristics of Concrete:

- Density of concrete: $\rho = 25 \text{ kN/m}^3$
- Reinforced concrete: 350 kg/m^3
- Lean concrete: 150 kg/m^3
- Compressive strength at 28 days: $f_{c28} = 25 \text{ N/mm}^2$
- Tensile strength: $f_{t28} = 2.1 \text{ MPa}$

CHAPTER II

CLIAMTE ACTIONS

II.1 Introduction:

We are going to carry out an analysis to determine the magnitude and value of the wind loads and of the snow that will be applied to the structure.

II.2 Study of snow:

II.2.1 Scope of application:

This regulation concerns all buildings in Algeria which are located at an altitude of 2000 metres or less.

II.2.2, The load of the snow on the ground (S_k):

The altitude of the project is about 87 m $\Rightarrow H = 87$ m

Our project is located in BISKRA belongs to the C zone, so in our case S_k is given by the following equation,

$$S_k = (0.0325 \times H + 10) / 100 = (0.0325 \times 87 + 10) / 100$$
$$S_k = 0.1282 \text{ KN/m}^2$$

II.2.3 Coefficient of the roof :

The hangar has a roof with 2 slopes with an angle $\alpha = 5.71^\circ$, so we choose $\mu_1 = 0.8$

II.2.4 Calculation of snow loads on roof :

$$S = \mu_1 * S_k$$

$$S = 0.8 * 0.1282$$

$$S = 0.1025 \text{ KN/m}^2$$

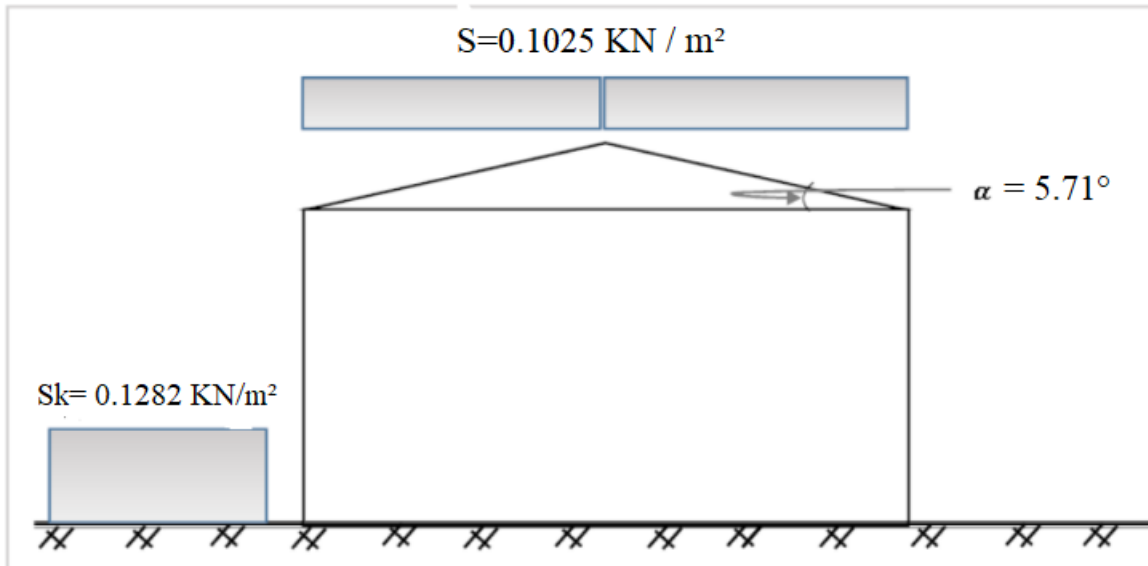


FIGURE II 1 Representation of static snow loads on the roof and floor

II.3 Wind action:

The impact of wind on a structure plays a significant role in the stability of the building. Their Horizontal and vertical movements have a direct impact on the structure.

It is therefore necessary to design a detailed analysis to determine the

Various actions caused by the wind in all possible orientations.

The calculation will be made in accordance with the 2013 RNV.

II.3.1 Wind Direction:

The calculation must be carried out separately for each of the orientations perpendicular to the various walls of the building.

The wilaya of Biskra is located in wind zone 3.

(Wind Plan RNV2013)

The site is classified as a category 2 area.

(Form 2.4 RNV2013)

The two main wind orientations

- Direction V1 the wind that blows penpendicular to the gable.
- Sense V2 the wind which is perpendicular to the lonng pann.

II.3.2.1 the dynamic reference pressure:

According to the 2013 RNV, our establishment is located in the wilaya of Biskra, which is classified as a zone, 3 on the climatic, wind map and based on the table (Table 2.2-RNV 2013)

Thus $q_{\text{réf}}=500\text{N/m}^2$

II.3. Determination of the dynamic pressure of the points:

II.3.2.2 Reference height z_e :

Please refer to Figure 2.1 of the 2013 RNV.

The reference height $z_e=h=11.8\text{m}$ and the width $b=33\text{m}$, therefore h cannot exceed b consequently $z_e=h=7\text{m}$.

Concerning roofs z_e is defined as the maximum height of structures

$Z_e=h=11.8\text{m}$.

(As per the SVSR 2013, c.2, section 2.3.2)

II.3.2.3 Exposure factor C_e :

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

II.3.2.3.1 Topography coefficient C_t :

The topography factor $C_t(z)$ considers the increase in wind speed when encountering obstacles such as hills isolated reliefs etc.

(RNVA2013, Chapter 2, Formula 2.4)

Calculation of ϕ :

Since our installation will be located on a completely flat ground $H=0$, it will be exposed to all directions of the wind.

$$\phi = H/L_u = 0/33 = 0$$

$$\phi < 0.05 \text{ therefore; } C_t = 1$$

(Form 2.4 RNV2013)

II.3.2.3.2 Roughness coefficient C_r :

$$C_{r(z)} = K_T \times \ln \frac{z}{z_0} ; \text{ Pour } Z_{\min} \leq Z \leq 200 \text{ m.}$$

$$C_{r(z)} = K_T \times \ln \frac{z_{\min}}{z_0} ; \text{ Pour } Z < Z_{\min}$$

Land Category II

$$K_t = 0.19 ; Z_0 = 0.05\text{m} ; Z_{\min} = 2\text{m} ; \epsilon = 0.52$$

(Table 2.4, chapter 2 RNV 2013)

Cr for vertical walls :

$$Z=10\text{m} \Rightarrow Z_{\min} \leq Z \leq 200 \text{ m}$$

$$Cr(z) = KT \times \ln Z/Z_0 \Rightarrow 0.190 \times \ln 10/0.05 = 1.006$$

Cr for the roof :

$$Z=11.8\text{m} \Rightarrow Z_{\min} \leq Z \leq 200\text{m}$$

$$Cr(z) = KT \times \ln Z/Z_0 \Rightarrow 0.190 \times \ln 11.8/0.05 = 1.038$$

II.3.2.3.3 The intensity of turbulence Iv(z):

$$\left\{ \begin{array}{l} I_v(z) = \frac{1}{C_v(z) \times \ln\left(\frac{z}{z_0}\right)} \quad \text{pour } z > z_{\min} \quad (a) \\ I_v(z) = \frac{1}{C_v(z) \times \ln\left(\frac{z_{\min}}{z_0}\right)} \quad \text{pour } z \leq z_{\min} \quad (b) \end{array} \right.$$

Calculation of Iv(z) for vertical patois $Z_e = 10\text{m} > Z_{\min} = 2 \text{ m}$

$$I_v(z) = \frac{1}{1 \times \ln\left(\frac{10}{0.05}\right)} = 0.188$$

Calculation of Iv(z) for the roof $Z_e = 11.8\text{m} > Z_{\min} = 2\text{m}$

$$I_v(z) = \frac{1}{1 \times \ln\left(\frac{11.8}{0.05}\right)} = 0.183$$

Calculation of Exposure Coefficient and Peak Dynamic Pressure $qp_{(ze)}$:

TABULAR II 1 Calculation of exposure factor and dynamic pressure

Coefficient	Z_e (m)	Ct	Cr	Iv	Ce	$q_{\text{réf}}$ (N/m ²)	$qp_{(ze)}$ (N/m ²)
vertical walls	10	1	1.006	0.188	2.343	500	1171.5
roof	11.8	1	1.038	0.183	2.457	500	1228.5

II.3.3 Determination of coefficient dynamic Cd:

In our case the maximum height of the structure is 11.8 m:

$$11.8\text{m} < 15\text{m}$$

So: $C_d=1$

II.3.4 Determination of the dynamic pressure $W(z_j)$:

$W(z_j)$ Aerodynamics

$$W(z_j) = q_p(z_e) \times [C_{pe} - C_{pi}] \text{ [N/m}^2\text{]}$$

Interior Pressure Coefficient: C_{pi}

$$\mu_p = \frac{\sum \text{des surfaces des ouvertures ou } C_{pe} \leq 0}{\sum \text{des surfaces de toutes les ouvertures}}$$

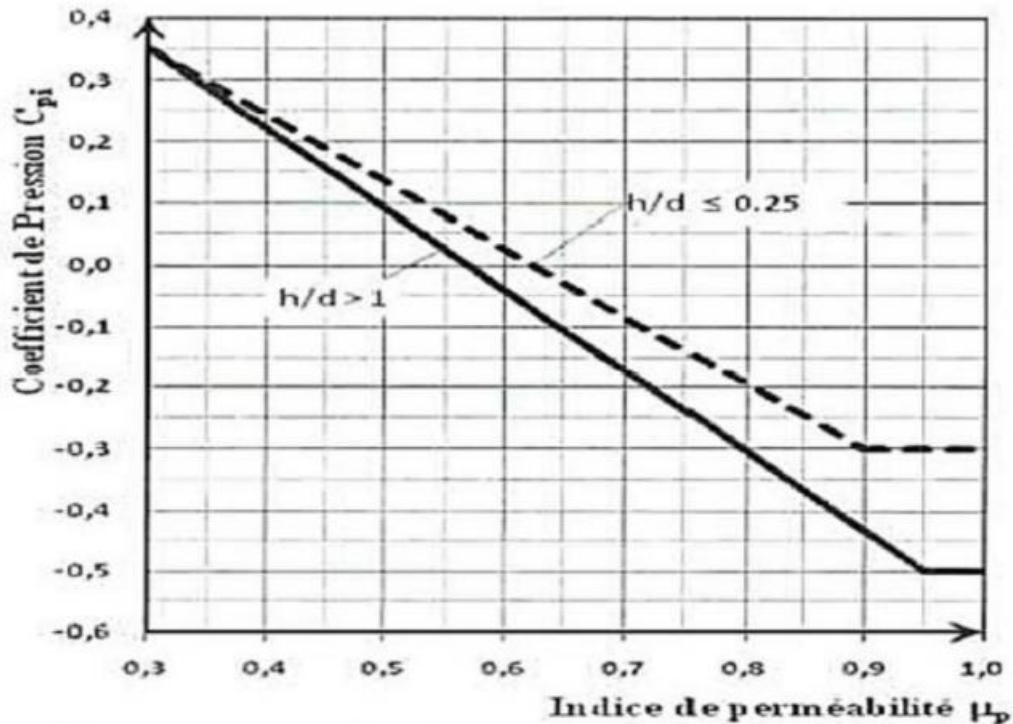


FIGURE II 2 Coefficient of pressure interior C_{pi} of buildings without face dominant

II.3.4.1.1 Calculation of C_{pe} wind perpendicular to long pan (direction V1):

a) Vertical walls

$$b = 33 \text{ m} ; d = 36 \text{ m} ; h = 10 \text{ m so } e = \min (b ; 2h) = \min (33 ; 2 \times 11,8) = 23,6 \text{ m}$$

We can see that $d > e$, so the vertical walls parallel to the wind V1 will be subdivided into 3 zones A,B,C as follows:

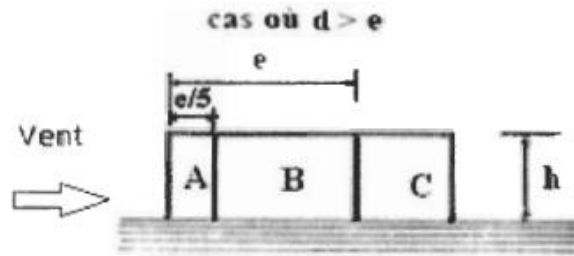


FIGURE II 3 Elevation view of the pressure zones on the vertical walls direction V1

- $A = e/5 = 23.6/5 = 4.72 \text{ m.}$
- $B = e - e/5 = 23.6 - 4.72 = 18.88 \text{ m.}$
- $C = b - e = 33 - 23.6 = 9.4 \text{ m}$
- $h = Z_e = 10 \text{ m (reference height).}$
- $D = 36 \text{ m}$
- $E = 36 \text{ m}$

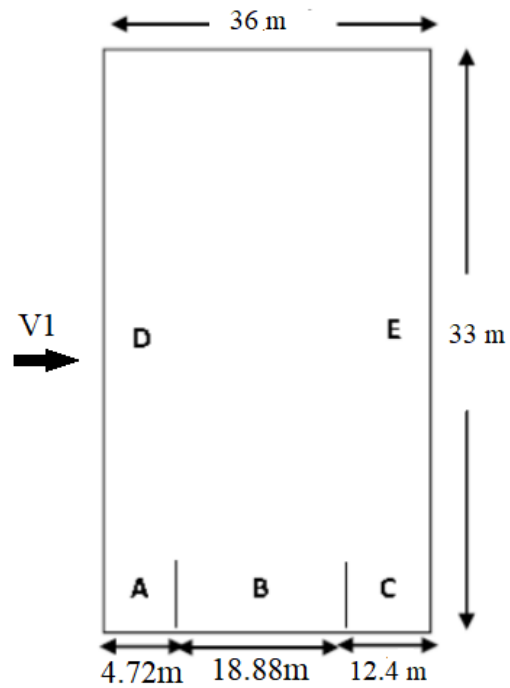


FIGURE II 4 Plan view of the pressure zones on the vertical walls direction V1

The areas of zones A, B, C, D and E are given by the following table:

TABULAR II 2 The values of the surfaces of the wind zones of the walls in the V1 direction

Zone	A		B		C		D		E	
Geometric Dimension (m)	$\frac{e}{5}$	h	$e - \frac{e}{5}$	h	d-e	h	b	h	b	h
	4,72	10	18.8	10	12.4	10	33	10	33	10
Surface(m ²)	47.2		188		124		330		330	

TABULAR II 3 Values of Cpe on walls in the V1 sense

Zone	A	B	C	D	E
C _{pe}	-1.0	-0.8	-0.5	+0.8	-0.3

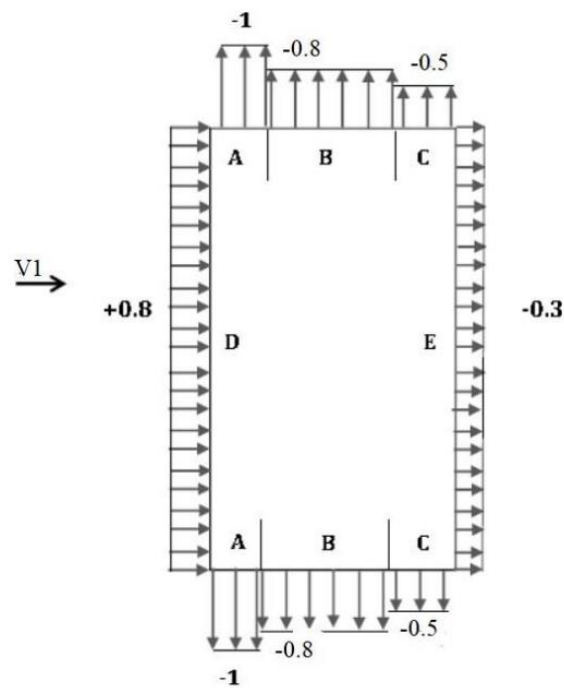


FIGURE II 5 Distribution of Cpe on the walls in the V1 direction

The structure is considered completely closed:

So $\mu_p=1$

$$h/d = 11.8/36 = 0.327$$

We take C_{pi} has two values

$$C_{pi} = -0.3$$

$$C_{pi}=0.35$$

b) The roof

In our case, the wind is perpendicular to the long pan (V1 direction) and also perpendicular to the generators, so $\theta = 0^\circ$.

Our roof is gable with an angle of $\alpha = 5.71^\circ$ so takes approximation $\alpha=5^\circ$

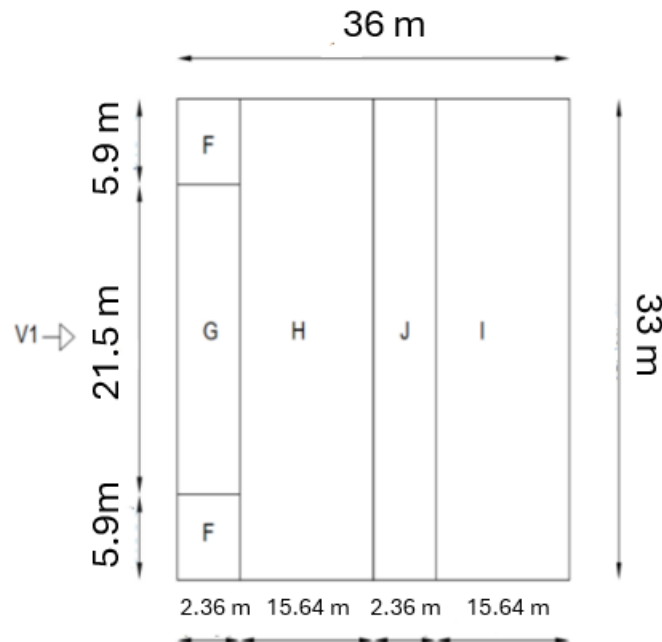


FIGURE II 6 Distribution of wind pressure zones on the roof direction V1

TABULAR II 4 The surface area of the loaded areas for the roof direction V1

Zone	F		G		H		J		I	
Geometric Dimension (m)	$\frac{e}{10}$	$\frac{e}{4}$	$\frac{e}{4}$	$b - \frac{e}{4}$	$\frac{d}{2}$ $-\frac{e}{10}$	b	$\frac{e}{10}$	b	$\frac{d}{2}$ $-\frac{e}{10}$	b
	2.36	5.9	5.9	21.2	15.64	33	2.36	33	15.64	33
Surface (m ²)	13.924		50.032		516.12		77.88		516.12	

C_{pe}=C_{pe,10} for all surfaces because they are more than 10m²

TABULAR II 5 Values of C_{pe} on the V1 roof

- CASE N°1

ZONE	F	G	H	I	J
C _{pe} 5°	-1.7	-1.2	-0.6	-0.6	+0.2

- CASE N°2

ZONE	F	G	H	I	J
C _{pe} 5°	+0.0	+0.0	+0.0	-0.6	-0.6

Calculation of C_{pi} wind perpendicular to the long pan direction V1 and θ = 0°:

The structure is considered as a completely closed structure

So $\mu_p=1$

$h/d = 11.8/36 = 0.327$

we take C_{pi} has two values:

C_{pi} = -0.3

C_{pi} =+ 0.35

II.3.4.1.2 Calculation of the wind Cpe perpendicular to the V2 pinion:

a) Vertical walls:

$$d=33\text{m}; b=36\text{m}, h=10\text{m}$$

$$e = \min [b; 2h]$$

$$e = \text{Min} [36; 2 \times 11.8]$$

$$e = 23.6 \text{ m}$$

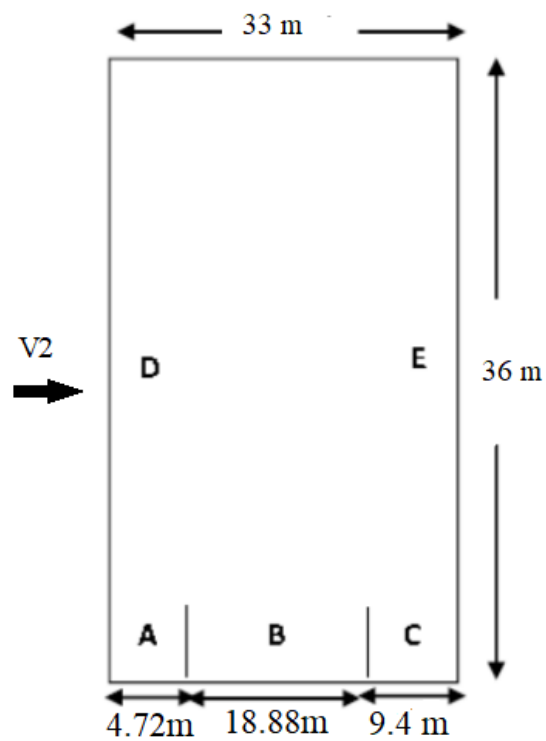


FIGURE II 7 Plan view of the pressure zones on the vertical walls direction V2

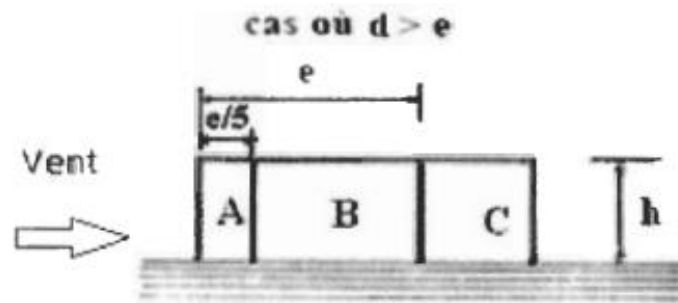


FIGURE II 8 Elevation view of the pressure zones on the vertical walls direction V2

TABULAR II 6 Surfaces of Loaded Areas for Vertical Walls

Zone	A		B		C		D		E	
Geometric Dimension (m)	$\frac{e}{5}$	h	$e - \frac{e}{5}$	h	d-e	h	b	h	b	h
	4,72	10	18.8	10	9.4	10	36	10	36	10
Surface(m ²)	47.2		188		94		360		360	

$C_{pe}=C_{pe,10}$ for all surfaces because they are 10m² higher, so:

TABULAR II 7 C_{pe} values on walls in the V1 sense

Zone	A	B	C	D	E
C_{pe}	-1.0	-0.8	-0.5	+0.8	-0.3

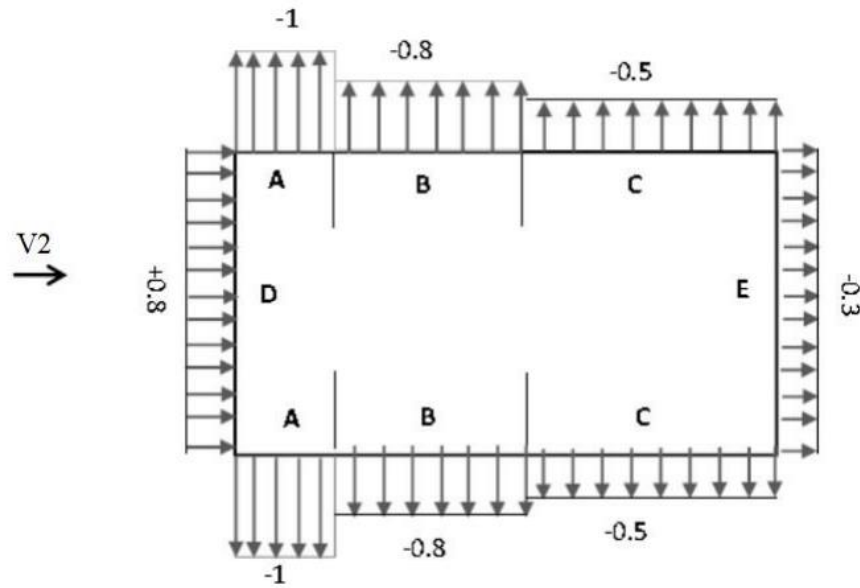


FIGURE II 9 The values of C_{pe} correspond to each area of the vertical walls direction V2

We consider the structure as a completely closed structure, so $\mu_p = 1$

$$h/d = 11.8/33 = 0.357$$

according to the figure of the permeability index to the function of the internal pressure coefficient (C_{pi}), we take C_{pi} with two values

$$C_{pi} = -0.3$$

$$C_{pi} = +0.35$$

b) Calculation of C_{pe} for the roof:

The wind direction is defined by angle θ , in our case the wind is perpendicular to the gable and parallel to the generators s , so $\theta = 90^\circ$. And the roof is gable with $\alpha = 5^\circ$. For $\theta = 90^\circ$, we define the different pressure zones F, G, H, I as follow :

$$\begin{aligned} h &= 11.8 \text{ m}; b = 36 \text{ m}; d = 33 \text{ m} \\ e &= \min [b ; 2 \times h] \\ e &= \min [36 ; 23.6] ; e = 23.6 \text{ m} \end{aligned}$$

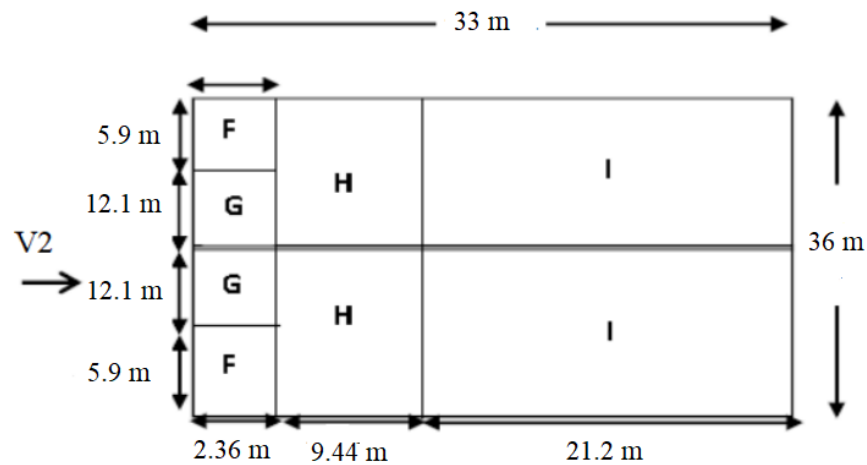


FIGURE II 10 The distribution of pressure zones on the roof direction V2

Calculation of the surface area for each zone

TABULAR II 8 Surfaces of the loaded areas for the roof

Zone	F	G	H	I
Surface (m ²)	13.924	28.556	169.92	381.6

$C_{pe} = C_{pe,10}$ for all surfaces because they are 10m² higher, so:

TABULAR II 9 The C_{pe} values corresponding to each roof area because the wind in the V2 direction

Zone	F	G	H	I
$C_{pe}(5^\circ)$	-1.6	-1.3	-0.7	-0.6

Calculation of C_{pi} wind perpendicular to the long pan direction V1 and $\theta=0^\circ$:

The structure is considered as a completely closed structure

So $\mu_p=1$

$$h/d = 11.8/36 = 0.327$$

according to the figure of the permeability index as a function of the internal pressure coefficient (C_{pi}) , we take C_{pi} with two values :

$$C_{pi} = -0.3$$

$$C_{pi} = 0.35$$

II.3.4.2 Calculation of aerodynamic pressure $W(z_j)$:

$$W(z_j) = q_p(z_e) \times [C_{pe} - C_{pi}] \text{ [N/m}^2\text{]}$$

II.3.4.2.1 Wind perpendicular to the pinion (SensV2):

Interior overpressure $C_{pi}=0.35$

a) Vertical walls:

TABULAR II 10 Aerodynamic pressure values on vertical walls in the case of overpressure

Zone	$q_p(Z_e)$	C_{pe}	C_{pi}	$W(z_j) \text{ [N/m}^2\text{]}$
A	1171.5	-1	0.35	-1581.525
B	1171.5	-0.8	0.35	-1347.225
C	1171.5	-0.5	0.35	-995.775
D	1171.5	+0.8	0.35	527.175
E	1171.5	-0.3	0.35	-761.475

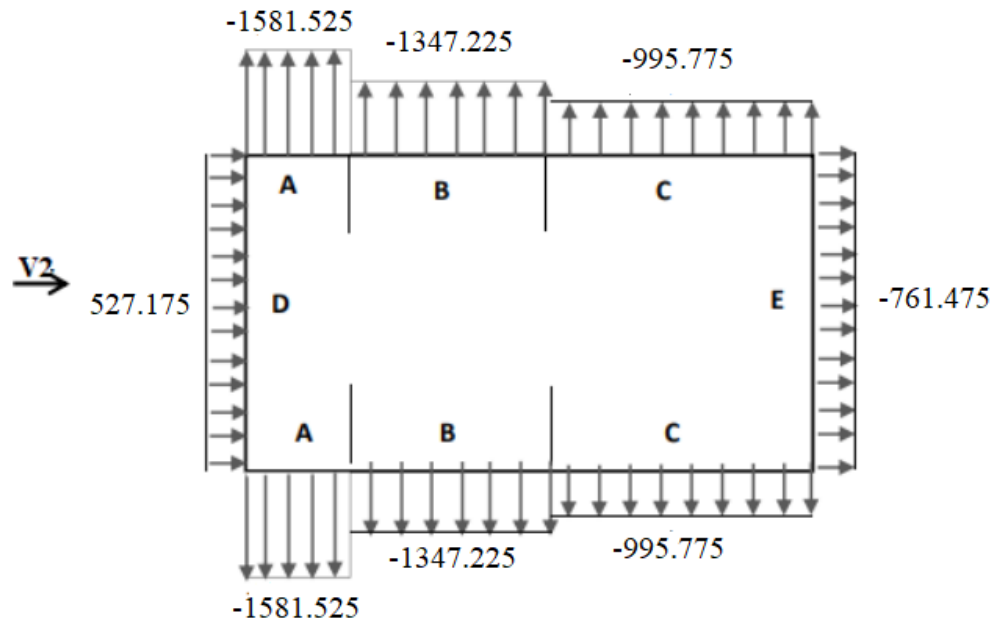


FIGURE II 11 The distribution of aerodynamic pressure on vertical walls in the case of overpressure

b) Roof $\theta=90^\circ$:

TABULAR II 11 Values of aerodynamic pressure on the roof in the case of overpressure

Zone	$q_p(Z_e)$	C_{pe}	C_{pi}	$W(z_j)$ [N/m ²]
F	1228.5	-1.6	0.35	-2395.575
G	1228.5	-1.3	0.35	-2027.025
H	1228.5	-0.7	0.35	-1289.925
I	1228.5	-0.6	0.35	-1167.075

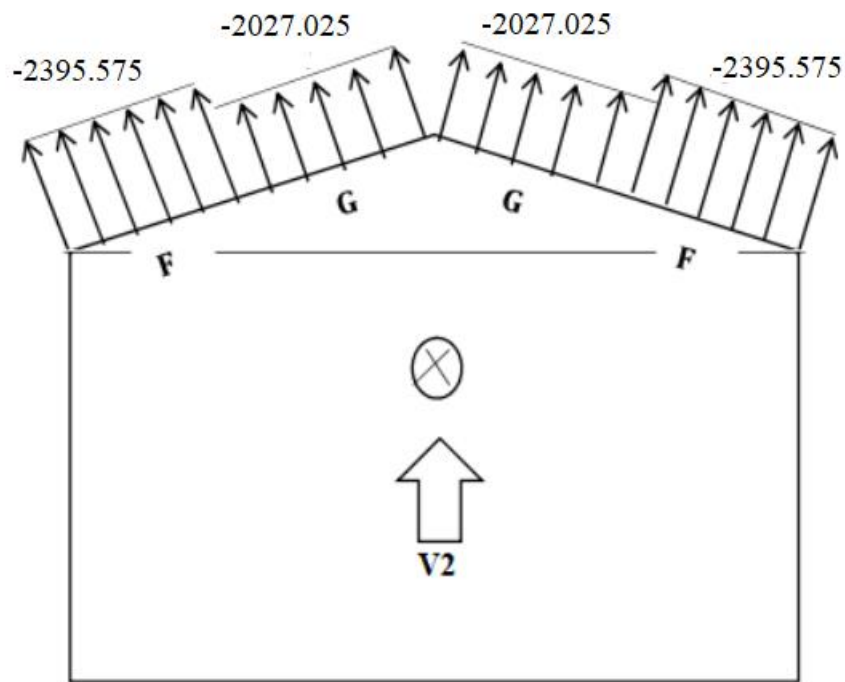


FIGURE II 12 The distribution of aerodynamic pressure on the vertical roof in the case of overpressure

Interior Depression $C_{pi}=0.35$

a) Vertical walls :

TABULAR II 12 Values of aerodynamic pressure on vertical walls in the case of vacuum

Zone	$q_p (Z_e)$	C_{pe}	C_{pi}	$W_{(zj)} [N/m^2]$
A	1171.5	-1	-0.3	-820.05
B	1171.5	-0.8	-0.3	-585.75
C	1171.5	-0.5	-0.3	-234.3
D	1171.5	0.8	-0.3	1288.5
E	1171.5	-0.3	-0.3	0

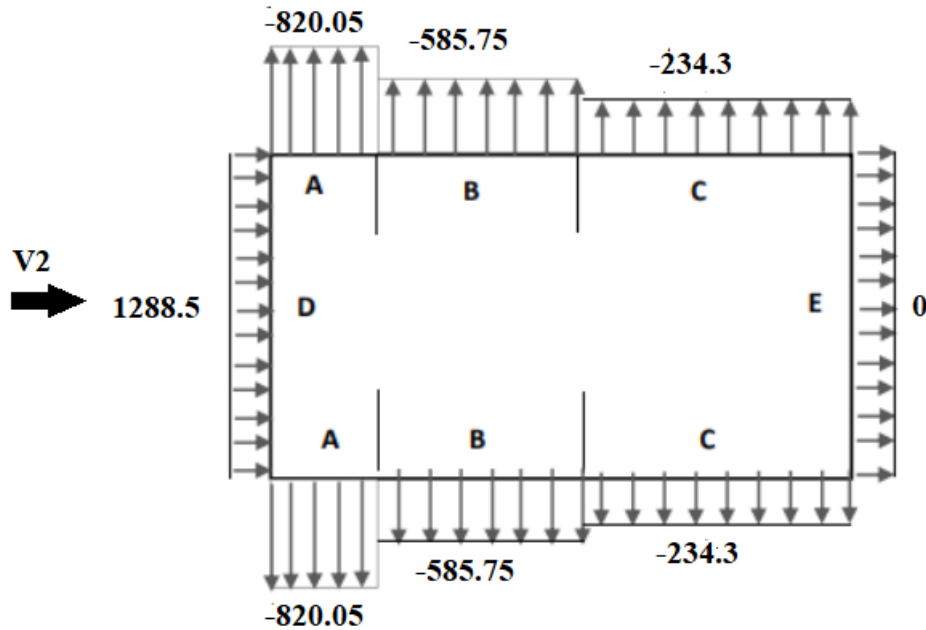


FIGURE II 13 The distribution of aerodynamic pressure on vertical walls in the case of depression

b) Roof:

TABULAR II 13 Values of aerodynamic pressure on the roof in the case of depression

	$q_p(z_e)$	C_{pe}	C_{pi}	$W_{(z)} [N/m^2]$
F	1228.5	-1.6	-0.3	-1597.05
G	1228.5	-1.3	-0.3	-1228.5
H	1228.5	-0.7	-0.3	-491.4
I	1228.5	-0.6	-0.3	-368.55

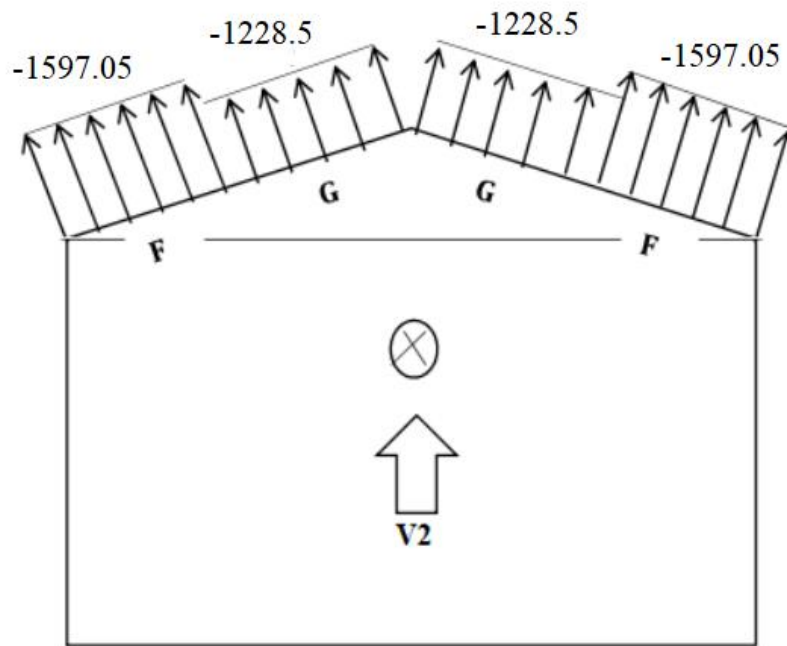


FIGURE II 14 The distribution of aerodynamic pressure on the roof in the case of the vacuum

II.3.4.2.2 Wind perpendicular to the long pan (Wind 1):

Interior overpressure

a) Vertical walls:

TABULAR II 14 Aerodynamic pressure values on vertical walls in the case of overpressure

Zone	$q_p (Z_e)$	C_{pe}	C_{pi}	$W_{(z)} [N/m^2]$
A	1171.5	-1	+0.35	-1581.525
B	1171.5	-0.8	+0.35	-1347.225
C	1171.5	-0.5	+0.35	-995.775
D	1171.5	+0.8	+0.35	527.175
E	1171.5	-0.3	+0.35	-761.475

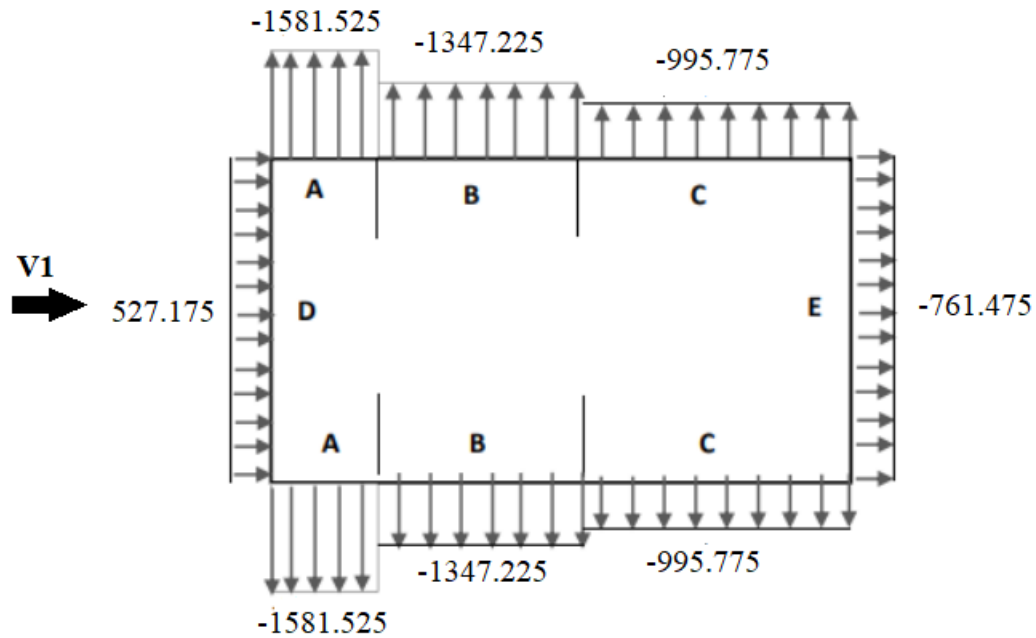


FIGURE II 15 The distribution of aerodynamic pressure on vertical walls in the case of overpressure

Case of overpressure

b) Roof:

Case No. 1:

TABULAR II 15 Values of the aerodynamic pressure on the roof in the case of overpressure

Zone	$Q_p(Z_e)$	C_{pe}	C_{pi}	$W_{(z)} [N/m^2]$
F	1228.5	-1.7	+0.35	-2518.425
G	1228.5	-1.2	+0.35	-1904.175
H	1228.5	-0.6	+0.35	-1167.075
I	1228.5	-0.6	+0.35	-1167.75
J	1228.5	+0.2	+0.35	-184.275

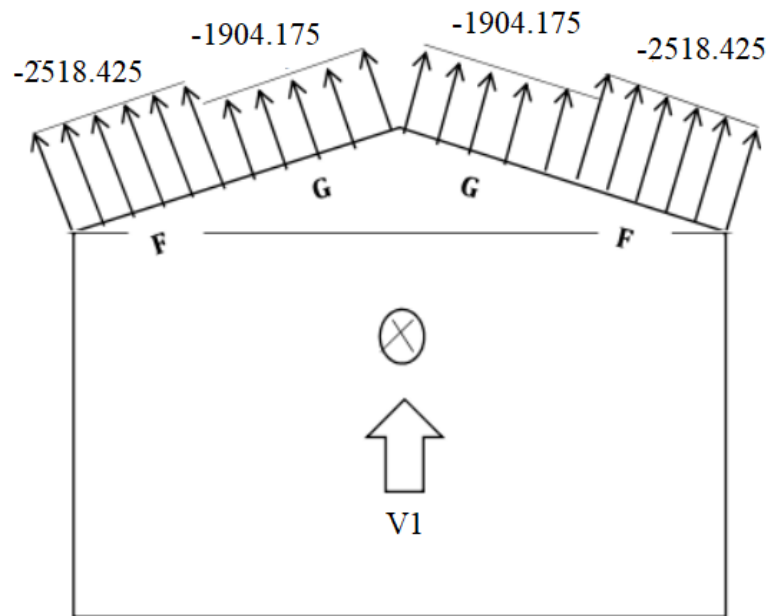


FIGURE II 16 The distribution of aerodynamic pressure on the roof in the case of a vacuum

Case N. 2:

TABULAR II 16 Values of the aerodynamic pressure on the roof in the case of overpressure.

Zone	$q_p (Z_e)$	C_{pe}	C_{pi}	$W_{(z)} (N/m^2)$
F	1228.5	+0	+0.35	-429.975
G	1228.5	+0	+0.35	-429.975
H	1228.5	+0	+0.35	-429.975
I	1228.5	-0.6	+0.35	-1167.075
J	1228.5	-0.6	+0.35	-1167.075

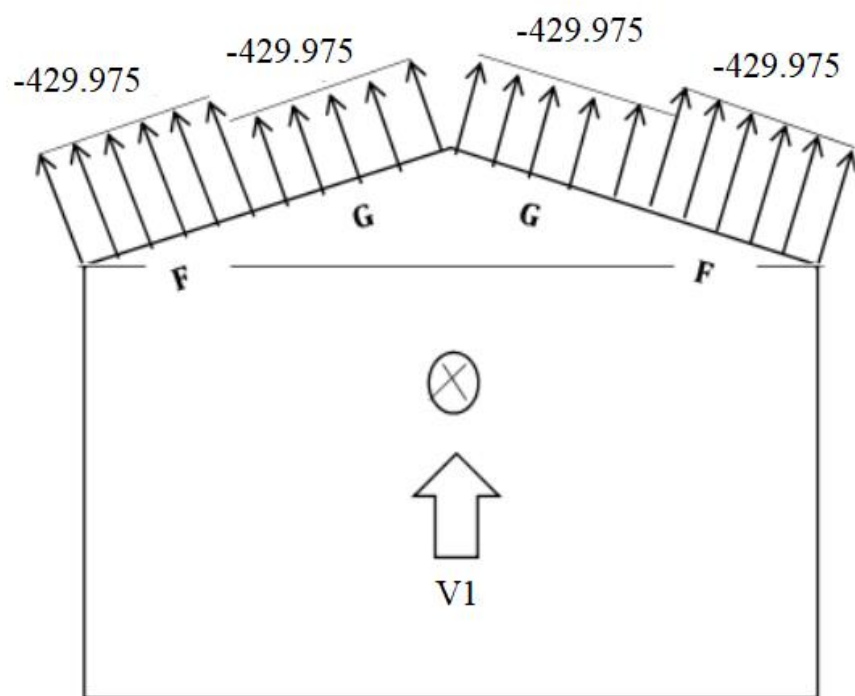


FIGURE II 17 The distribution of aerodynamic pressure on the roof in the case of the vacuum

Interior Depression $C_{pi}=0.35$

a) Vertical walls :

TABULAR II 17 Values of aerodynamic pressure on vertical walls in the case of depression.

Zone	$q_p (Z_e)$	C_{pe}	C_{pi}	$W_{(z_j)} [N/m^2]$
A	1171.5	-1	-0.3	-820.05
B	1171.5	-0.8	-0.3	-585.75
C	1171.5	-0.5	-0.3	-234.3
D	1171.5	0.8	-0.3	1288.5
E	1171.5	-0.3	-0.3	0

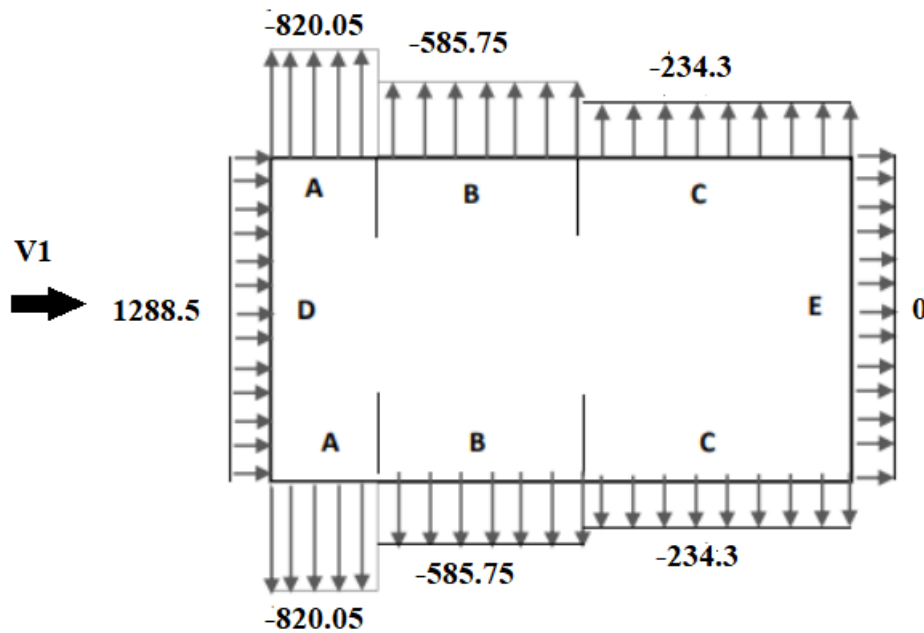


FIGURE II 18 The distribution of aerodynamic pressure on vertical walls in the case of depression

b) Roof:

TABULAR II 18 Values of aerodynamic pressure on vertical walls in the case of depression.

Zone	$q_p(z_e)$	C_{pe}	C_{pi}	$W_{(z)} [N/m^2]$
F	1228.5	-1.7	-0.3	-1719.9
G	1228.5	-1.2	-0.3	-1105.65
H	1228.5	-0.6	-0.3	-368.55
I	1228.5	-0.6	-0.3	-368.55
J	1228.5	+0.2	-0.3	614.25

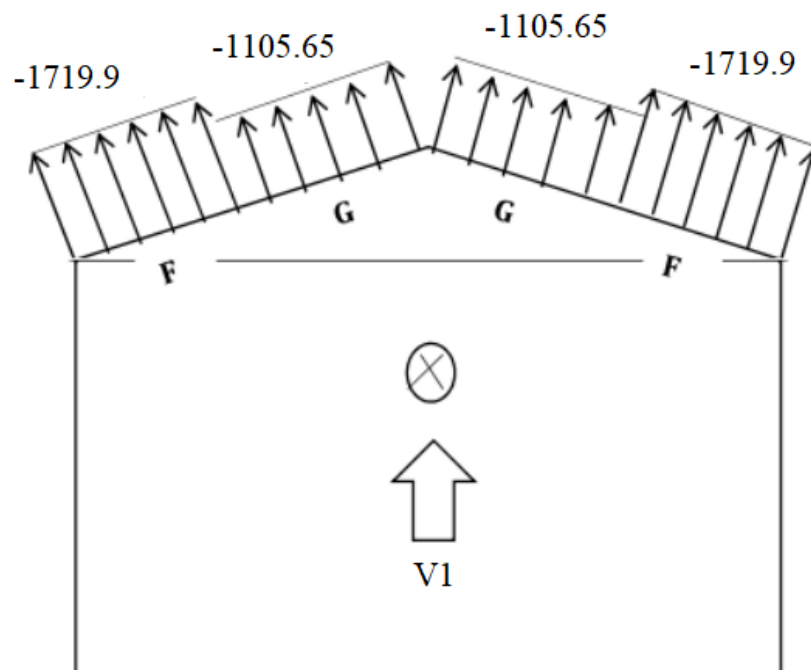


FIGURE II 19 The distribution of aerodynamic pressure on vertical walls in the case of vacuum

TABULAR II 19 Values of the aerodynamic pressure on the vertical walls, in the case of the vacuum.

Zone	$q_p(Z_e)$	C_{pe}	C_{pi}	$W_{(zj)} [N/m^2]$
F	1228.5	+0	-0.3	368.55
G	1228.5	+0	-0.3	368.55
H	1228.5	+0	-0.3	368.55
I	1228.5	-0.6	-0.3	-368.55
J	1228.5	-0.6	-0.3	-368.55

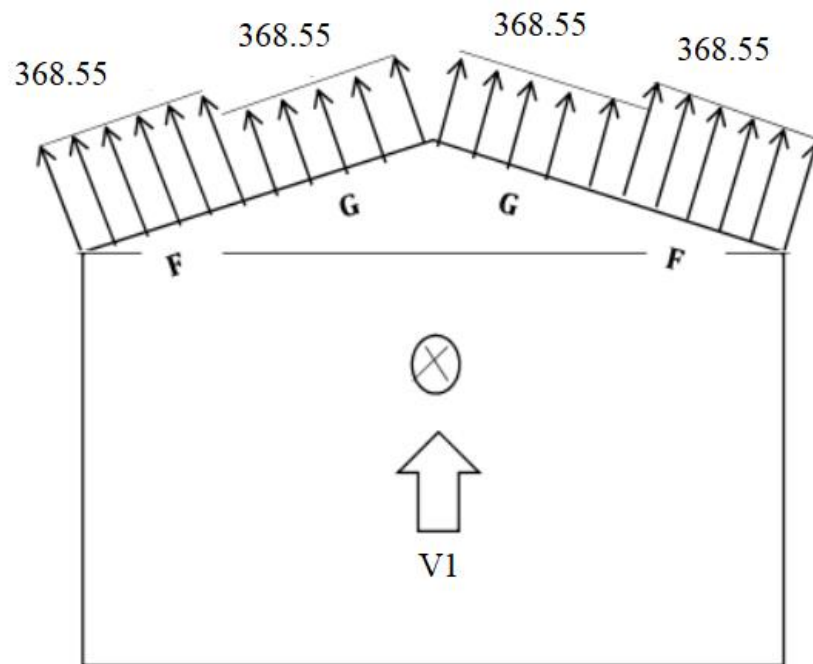


FIGURE II 20 The distribution of aerodynamic pressure on vertical walls: the case of vacuum

II.3.5 Overall action :

II.3.5.1 Determination of the external force $F_{w,e}$ steering case towards the pinion (V2) :

Calculation of external pressure :

TABULAR II 20 The results of the calculation of the external pressure W_e corresponding to each zone

Zone	$q_p (Z_e)$	C_{pe}	$W_e [KN/m^2]$
D	1171.5	+0.8	0.9372
E	1171.5	-0.3	-0.35145
F	1228.5	-1.6	-1.9656
G	1228.5	-1.3	-1.59705
H	1228.5	-0.7	-0.85995
I	1228.5	-0.6	-0.7371

Calculation of external force:

TABULAR II 21 The results of the calculation of the external force $F_{w,e}$ corresponding to each zone

Element	Zone	C_d	W_e (KN/m ²)	A_{ref} (m ²)	Horizontal components $F_{w,e}$ (KN)	Vertical components $F_{w,e}$ (KN)
V wall	D	1	0.9372	1188	1113.39	0
V wall	E	1	-0.35145	1188	-417.52	0
Roof	F	1	-1.9656	13.924	0	-27.37
Roof	G	1	-1.59705	28.556	0	-45.60
Roof	H	1	-0.85995	169.92	0	-146.12
Roof	I	1	-0.7371	381.6	0	-281.27
					$\Sigma = 698.87$	$\Sigma = -500.36$

II.3.5.2 Determination of the internal force $F_{w,i}$ steering case towards the pinion (V2) :

Calculation of the internal pressure case N°1 :

TABULAR II 22 The results of the calculation of the internal pressure W_i corresponding to each zone

Zone	$q_p (Z_e)$	C_{pi}	W_i [N/m ²]
D	1171.5	-0.3	-351.45
E	1171.5	-0.3	-351.45
F	1228.5	-0.3	-368.55
G	1228.5	-0.3	-368.55
H	1228.5	-0.3	-368.55
I	1228.5	-0.3	-368.55

Calculation of the inner strength :

TABULAR II 23 The results of the calculation of the internal force F_{wi} corresponding to each zone

Elément	Zone	C_d	W_i [N/m ²]	A_{ref} [m ²]	Horizontal components $F_{w,i}$ [KN]	Vertical components $F_{w,i}$ [KN]
V wall	D	1	-351.45	1188	-417.522	0
V wall	E	1	-351.45	1188	-417.522	0
Roof	F	1	-368.55	13.924	0	-5.131
Roof	G	1	-368.55	28.556	0	-10.52
Roof	H	1	-368.55	169.92	0	-62.624
Roof	I	1	-368.55	381.6	0	-140.638
					$\Sigma=-835.044$	$\Sigma=-218.913$

Calculation of the internal pressure case N°2 :

TABULAR II 24 The results of the W_i internal pressure calculation corresponding to each zone

Zone	q_p (Ze)	C_{pi}	W_i [N/m ²]
D	1171.5	0.35	410.025
E	1171.5	0.35	410.025
F	1228.5	0.35	429.975
G	1228.5	0.35	429.975
H	1228.5	0.35	429.975
I	1228.5	0.35	429.975

Calculation of the inner strength :

TABULAR II 25 The results of the calculation of the internal force Fwi corresponding to each zone

Elément	Zone	C _d	W _i [N/m ²]	A _{ref} [m ²]	Horizontal components F _{w, i} [KN]	Vertical components F _{w, i} [KN]
V wall	D	1	410.025	1188	487.109	0
V wall	E	1	410.025	1188	487.109	0
Roof	F	1	429.975	13.924	0	5.986
Roof	G	1	429.975	28.556	0	12.278
Roof	H	1	429.975	169.92	0	73.061
Roof	I	1	429.975	381.6	0	164.078
					Σ= 974.218	Σ= 255.403

II.3.5.3 Determination of the frictional force :

When the cumulative area of all windward surfaces does not exceed four times the total area of all exterior surfaces perpendicular to the wind.

Case 1 the wind direction is perpendicular to the V2 pinion

Calculation of Wind-Parallel Surfaces

The vertical walls of long pan = $33 \times 10 \times 2 = 660 \text{ m}^2$.

The long-sloped roof

Roof = $[2 \times (18 \times 33)] = 1188 \text{ m}^2$.

⇒ Total wind-parallel areas = $660 + 1188 = 1848 \text{ m}^2$.

Calculation of surfaces perpendicular to the wind

The surfaces of the two gables

Condition check

$1848 \text{ m}^2 \leq 4 \times 784.8 = 3139.2 \text{ m}^2 \rightarrow$ condition is verified.

The friction effect can be neglected.

Case 2 wind direction perpendicular to long pan V1

Calculation of Wind-Parallel Surfaces

The surface of the two gables

$$\text{Roof} = [2 \times (18 \times 33)] = 1188 \text{ m}^2.$$

$$\Rightarrow \text{Total windward areas} = 784.8 + 1188 = 1848 \text{ m}^2.$$

Calculation of surfaces perpendicular to the wind

The surfaces of the two vertical gables (long pan)

$$= (33 \times 10) \times 2 = 660 \text{ m}^2$$

Condition check

$$1848 \text{ m}^2 \leq 4 \times 660 = 2640 \text{ m}^2 \rightarrow \text{condition is verified}$$

The friction effect can be neglected.

Chapter III

Pre-sizing of Elements

III . Introduction :

This chapter deals with the calculation of the resistant components which form the hangar structure and which will be exposed to various loads. This calculation will give us the profiles capable of guaranteeing both the durability and the balance of the structure.

III.1 Dimensioning of the roof panels and cladding:

III.1.1 Definition of sandwich panels:

It consists of an outer metal cladding, an insulating core made of a polyurethane foam and a metal inner cladding, all firmly attached to the insulating core by adhesion of the foam. These elements work synergistically to form a single self-sustaining entity that offers diverse.

III.1.2 Cover panels:

To select the appropriate roof panel for our structure, it is essential to know the maximum wind load that the roof can withstand, the number of support points for the roof panel and the thickness of the roof material. In accordance with a technical sheet, we will determine the thickness of the cover panel and its own weight

TABULAR III 1 proper weight of panels with its thickness

CARACTÉRISTIQUES TECHNIQUES											
Ep. mm	Coefficient de transmission thermique : W/m². °C	Poids Kg/m²	Charges non pondérées daN/m²								
			50	75	100	125	150	175	200	225	250
35	0.67	12.4	3.75	3.25	2.85	2.60	2.45	2.30	2.15	2.05	1.90
40	0.51	12.9	4.25	3.70	3.30	3.00	2.80	2.60	2.45	2.35	2.20
45	0.42	13.2	4.80	4.15	3.75	3.40	3.15	2.95	2.80	2.65	2.50

Ep. mm	Coefficient de transmission thermique : W/m². °C	Poids Kg/m²	Charges non pondérées daN/m²								
			50	75	100	125	150	175	200	225	250
35	0.67	12.4	4.80	4.15	3.75	3.40	3.15	2.90	2.65	2.50	2.35
40	0.51	12.9	5.40	4.65	4.15	3.75	3.45	3.20	2.95	2.80	2.65
45	0.42	13.2	5.95	5.25	4.65	4.25	3.90	3.60	3.35	3.15	2.95

Les valeurs indiquées dans les tableaux ci-dessus prévoient une flèche $f < 1/200$ des écartements des appuis l (m) et se réfèrent aux panneaux avec épaisseurs des parements acier, $0.5 + 0.5$ mm.

According to the technical sheet, we will choose a 45mm thick cover panel which gives a weight of 13.2 kg/m²

III.1.3 Cladding panel:

TABULAR III 2 proper weight of panels with its thickness

★ ★ ★ ★

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B- PANNEAUX DE BARDAGE PRELAQUES (OU GALVANISES)

Tôle épaisseur: 0.5 mm

Type	Longueur Max ml	Largeur standard mm	Epaisseurs Mm	Poids Kg/m ²	Résistance à la Conductibilité $\frac{1}{\sqrt{\Omega}}$ m ² K/W	Coefficient de Transmission Thermique K W/m ² .K	Distance entre traverses maxi en Mètres
B 35	18	1000	35	10,17	1,70	0,53	*
B 40	18	1000	40	10,42	1,90	0,45	*
B 50	18	1000	50	10,92	2,46	0,34	*
B 60	18	1000	60	11,42	2,95	0,29	*

N.B : * selon les études et calculs statiques pour chaque cas.

According to this table above, a 60mm thick cladding panel is chosen, which gives a weight of 11 kg/m².

III.2 Study of purlins:

III.2.1 Definitions:

The function of purlin beams is to support the roof and to transfer the loads and overloads exerted on it to the crossbeam. They are aligned in parallel with the ridge line and their dimensioning is done by means of deflected bending, under the influence of permanent loads, operating loads and climatic loads. They are made either of hot-formed profiles in (I) or in (U) or in cold-formed profiles in (Z), (U)

We study the most stressed purlin, which is the intermediate purlin with a span of L=6m, inclined at an angle $\alpha = 5^\circ$ and in the centre distance e = 1.2m

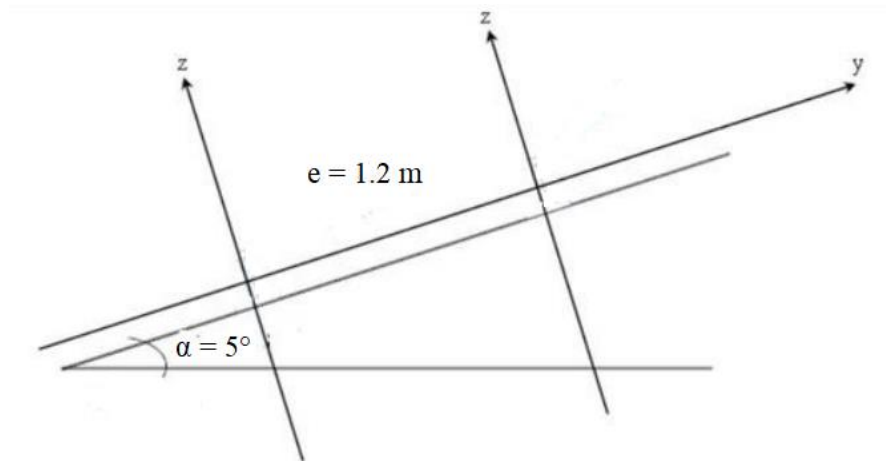


FIGURE III 1 placement of purlins on the roof with center distances

III.2.2 Determination of Demands:

III.2.2.1 Assessment of loads and overloads:

The dead loads G:

Self-weight of cover (sandwich panels) 13.2 kg/m^2

Self-weight of attachment accessory 1.5 kg/m^2

Self-weight of the purlin (estimated) 16 kg/ml

Spacing between purlins $e = 1.16 \text{ m}$ $G = (13.2 + 1.5) \times 1.16 + (16) = 33.05 \text{ kg/ml}$

$G = 0.33 \text{ kg/ml}$.

Maintenance Surcharges P:

In the case of inaccessible roofs, only a maintenance load of 100 kg equal to the weight of a worker and his assistant which is equivalent to two concentrated loads of 100 kg located at $1/3$ and $2/3$ of the span of the purlin are considered.

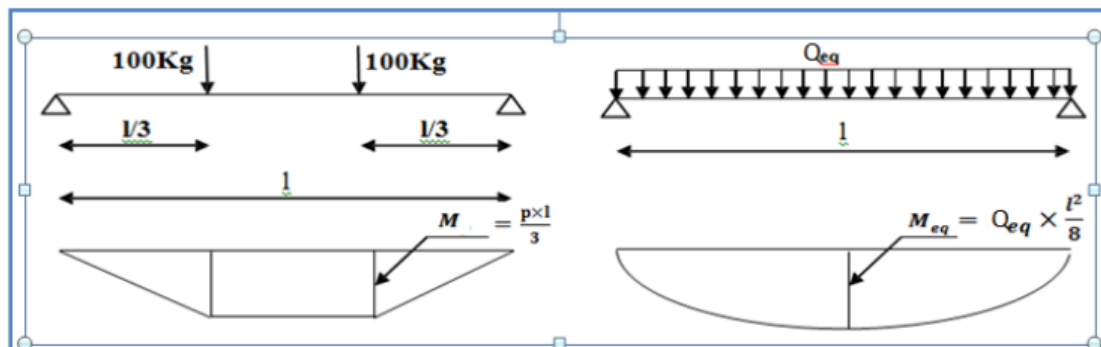


FIGURE III 2 static pattern of maitenance overloads on purlins

$$M_{\max} = \frac{P' \times L}{3} \Rightarrow \frac{P \times L}{8} P = \frac{P' \times 8}{3 \times L} \times \frac{100 \times 8}{3 \times 6} = 44.44 \text{ Kg/ml}$$

$$P_{ep} = 0.467 \text{ kN/ml}$$

C) Climatic loads:

1) Wind overload W:

The extreme load is in the case of wind on a long slope with internal overpressure:

$$C_{pi} = +0.35 \text{ Zone F}$$

$$W = -2.518 \text{ KN/m}^2$$

The linear wind load is equal to $W = -2.518 \times 1.16$

$$W = -2.92 \text{ kN /ml.}$$

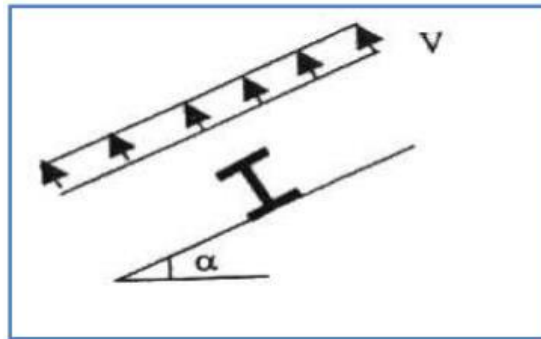


FIGURE III 3 Wind load on purlins

2) Snow overload:

The snow overload depends on the location of the site and the shape of the roof

S Snow overload on the roof .

$$S = 0.1025 \text{ kN/m}^2$$

Linear snow load on the roof

$$S = 0.1025 \times 1.16 = \times$$

$$S = 0.1189 \text{ kN/ml}$$

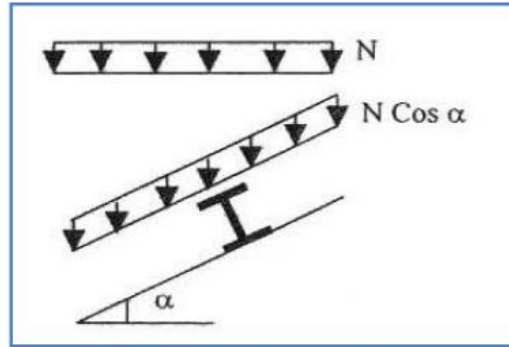


FIGURE III 4 Snow load on the roof

III.2.2.2 Applied loads and surcharges:

$$G = 0.33 \text{ kN/ml}$$

$$Q = 0.444 \text{ kN/ml}$$

$$W = -2.92 \text{ kN/ml}$$

$$S = 0.1189 \text{ kN/ml}$$

TABULAR III 3 Decomposition of loads along the Z-Z and Y-Y axis

Following Z-Z axis	Following Y-Y axis
$G_{ZZ} = G \cos \alpha = 0.328$	$G_{YY} = G \sin \alpha = 0.028$
$Q_{ZZ} = Q \cos \alpha = 0.442$	$Q_{YY} = Q \sin \alpha = 0.038$
$W_{ZZ} = W = -2.92$	$W_{YY} = 0$
$S_{ZZ} = S \cos \alpha = 0.118$	$S_{YY} = S \sin \alpha = 0.010$

-Thus

$$\bullet Q_{zEd} = -2.92 \text{ kN/m}$$

$$\bullet Q_{yEd} = 0.038 \text{ kN/m}$$

III.2.2.3 Combinations of actions:

a)Ultimate limit stat (ULS):

1ST case only wind load:

$$Q_{uz} = G_{zz} + 1.5W_z = 0.328 + 1.5 \times (-2.92) = -4.7 \text{ kN/ml.}$$

$$Q_{uy} = 1.35G_{yy} + 1.5W_{yy} = 1.35 \times 0.028 + 1.5 \times 0 = 0.037 \text{ kN/ml.}$$

2ND only snow load:

$$Q_{uz} = 1.35 G_{zz} + 1.5 S_{zz} = 1.35 \times 0.328 + 1.5 \times 0.118 = 0.62 \text{ KN/ml.}$$

$$Q_{uy} = 1.35 G_{yy} + 1.5 S_{yy} = 1.35 \times 0.028 + 1.5 \times 0.010 = 0.052 \text{ KN/ml.}$$

3RD maintenance overload:

$$Q_{uz} = 1.35 G_{zz} + 1.5 Q_{uz} = 1.35 \times 0.328 + 1.5 \times 0.442 = 1.10 \text{ KN/ml.}$$

$$Q_{uy} = 1.35 G_{yy} + 1.5 Q_{yy} = 1.35 \times 0.028 + 1.5 \times 0.038 = 0.09 \text{ KN/ml.}$$

$$Q_{uz} = G_{zz} + 1.5 W_z = 0.328 + 1.5(-2.92) = -4.7 \text{ KN/ml.}$$

$$Q_{uy} = 1.35 G_{yy} + 1.5 Q_{yy} = 1.35 \times 0.028 + 1.5 \times 0.038 = 0.09 \text{ KN/ml.}$$

b) The Service Limit State (SLS):**1st case the wind acts alone:**

$$Q_{uz} = G_{zz} + W_z = 0.328 + (-2.92) = -2.59 \text{ KN/ml.}$$

$$Q_{uy} = G_{yy} + W_{yy} = 0.028 + 0 = 0.028 \text{ KN/ml.}$$

2nd case the snow acts alone:

$$Q_{uz} = G_{zz} + S_{zz} = 0.328 + 0.118 = 0.446 \text{ KN/ml.}$$

$$Q_{uy} = G_{yy} + S_{yy} = 0.028 + 0.01 = 0.038 \text{ KN/ml.}$$

3rd case the maintenance overload:

$$Q_{uz} = G_{zz} + Q_{uz} = 0.328 + 0.442 = 0.77 \text{ KN/ml.}$$

$$Q_{uy} = G_{yy} + Q_{yy} = 0.028 + 0.038 = 0.066 \text{ kN/ml}$$

Takes the maximum load at the ULS back to the most stressed failure

$$Q_{uz} = -2.59 \text{ kN/ml}$$

$$Q_{uy} = 0.066 \text{ kN/ml}$$

III.2.3 Pre-sizing:

Purlins are stressed to deflected bending, they must satisfy the following two conditions

The SLS

The ULS

We do the pre-sizing of the purlins by using the deflection condition, then we do the verification of the resistance condition.

Checking the Arrow:

$$f_{adm} \leq \frac{L}{200} \text{ avec : } f_z = \frac{5 \times q_{sz} \times L^4}{384 \times E \times I_y}$$

$$\text{et : } \frac{L}{200} = \frac{600}{200} = 3 \text{ cm.}$$

$$I_y \geq \frac{5 \times q_{sz} \times L^4}{384 \times E \times 3}$$

$$I_y \geq \frac{5 \times 2.59 \times 10^{-2} \times 600^4}{384 \times 21000 \times 3}$$

$$I_y \geq 693.75 \text{ cm}^4.$$

Which corresponds to an IPE 160 with $I_y = 869.3 \text{ cm}^4$.

TABULAR III 4 CHARACTERISTICS OF IPE 160

Profile	h(mm)	b(mm)	t_w (mm)	t_f (mm)	r (mm)	d (mm)	P (kg/m)
IPE 160	160.0	82.0	5.0	7.4	9.0	127.2	15.8
	A(cm ²)	I_y (cm ⁴)	i_y (cm)	W_{ply} (cm ³)	I_z (cm ⁴)	i_z (cm)	W_{plz} (cm ³)
	20.1	869.3	6.58	123.9	68.3	1.84	26.10
	W_{ely} (cm ³)	W_{elz} (cm ³)					
	108.7	16.66					

III.2.4 Sizing of the purlins:**III.2.4.1 Confiton of the boom:**

$$f \leq f_{adm}$$

a) Verification of the deflection along the Z-Z axis :

$$f_z = \frac{5 \times q_{sz} \times L^4}{384 \times E \times I_y}$$

$$f_z = \frac{5 \times 2.59 \times 10^{-2} \times 600^4}{384 \times 21000 \times 869.3}$$

$$f_z = 2.39 \text{ cm} < f_{adm} = 3 \text{ cm}$$

The condition of the arrow along the Z-Z axis is verified

(b) Y-Y Axis Deflection Verification :

$$f_y = \frac{5 \times q_{sy} \times L^4}{384 \times E \times I_z}$$
$$f_y = \frac{5 \times 0.066 \times 10^{-2} \times 600^4}{384 \times 21000 \times 68.3}$$
$$f_y = 0.77 < f_{adm} = 3 \text{ cm}$$

The condition is check according to Y-Y:

The arrow is checked on both axes so we adopt an IPE160

III.2.4.2 The condition of resistance:

Deflected bending verification:

Determination of the profile class:

- Web

$$\varepsilon = \sqrt{\left(\frac{275}{f_y}\right)} = \sqrt{\left(\frac{275}{275}\right)} = 1$$
$$\frac{d}{t_w} = \frac{127.2}{5} = 25.44 < 33\varepsilon = 33 \times 1 = 33 \Rightarrow \text{Web classe 1.}$$

- Flange :

$$\frac{c}{t_f} = \frac{\frac{b}{2}}{t_f} = \frac{\frac{82}{2}}{7.4} = 5.54 < 10\varepsilon = 10 \times 1 = 10 \Rightarrow \text{Flange de classe 1.}$$

So the class section is 1 $\Rightarrow \gamma_{m0} = 1$.

Checking for deflected bending

$$\left(\frac{M_{ysd}}{M_{pl.y.rd}}\right)^\alpha + \left(\frac{M_{zsd}}{M_{pl.z.rd}}\right)^\alpha \leq 1$$

With $\alpha=1$ and $\beta=1$ in case of safety

$$\text{And } M_{sd} = \frac{q_u \times L^2}{8}$$

Axe Z-Z

$$M_{ysd} = \frac{q_{uz} \times L^2}{8} = \frac{2.59 \times 6^2}{8} = 11.65 \text{ KN.m}$$

Axe Y-Y

$$M_{zsd} = \frac{q_{uy} \times L^2}{8} = \frac{0.066 \times 6^2}{8} = 0.297 \text{ KN.m}$$

$$M_{pl} = \frac{W_{pl} \times f_y}{\gamma_{m0}}$$

$$M_{ply,rd} = \frac{W_{ply} \times f_y}{\gamma_{m0}} = \frac{123.9 \times 27.5 \times 10^{-2}}{1} = 34.07 \text{ KN.m}$$

$$M_{pl,z,rd} = \frac{W_{plz} \times f_y}{\gamma_{m0}} = \frac{26.1 \times 27.5 \times 10^{-2}}{1} = 7.17 \text{ KN.m}$$

So:

$$\left(\frac{M_{ysd}}{M_{ply,rd}} \right)^\alpha + \left(\frac{M_{zsd}}{M_{pl,z,rd}} \right)^\alpha \leq 1$$

$$\left(\frac{11.65}{34.07} \right)^1 + \left(\frac{0.297}{7.17} \right)^1 \leq 1$$

$0.38 < 1$ the biaxial bending is verified.

b) Shear Verification :

We use the following relationship

$$V_{zsd} < V_{plz,rd} \text{ and } V_{ysd} < V_{ply,rd}$$

(Eurocode. 3 p158)

- Axis (Z-Z)

$$V_{zsd} = \frac{Q_{uz} \times L}{2} = \frac{2.59 \times 6}{2}$$

$$V_{zsd} = 7.77 \text{ KN.}$$

$$A_{vz} = A - 2 \times b \times t_f + (t_w + r)t_f = 20.1 - 2 \times 8.2 \times 0.74 + (0.5 + 0.9)0.74$$

$$A_{vz} = 9 \text{ cm}^2$$

$$V_{plz,rd} = A_{vz} \times \frac{f_y}{\sqrt{3} \times \gamma_{m0}} = 9 \times \frac{27.5}{\sqrt{3} \times 1.1}$$

$$V_{plz,rd} = 129.9 \text{ KN.}$$

- Axis (Y-Y)

$$V_{ysd} = \frac{Q_{yz} \times L}{2} = \frac{0.066 \times 6}{2}$$

$$V_{ysd} = 0.198 \text{ KN.}$$

$$A_{vy} = A - A_{vz} = 20.1 - 9$$

$$A_{vy} = 11.1 \text{ cm}^2$$

$$V_{ply,rd} = A_{vy} \times \frac{f_y}{\sqrt{3} \times \gamma_{m0}} = 11.1 \times \frac{27.5}{\sqrt{3} \times 1.1}$$

$$V_{ply,rd} = 160.21 \text{ KN.}$$

$$V_{zsd} = 7.63 \text{ KN} < V_{plz,rd} = 129.9 \text{ KN}$$

$$V_{ysd} = 0.198 \text{ KN} < V_{ply,rd} = 160.21 \text{ KN}$$

So the resistance of the purlins with **IPE160** to shear is verified.

C) Spill Verification:

Upper sole:

It is possible that the upper sole compresses under the effect of downward vertical loads, overturns. Since it is securely anchored to the roof, there is no risk of tipping over it is compressed under the effect of the lifting wind, can spill as soon as it is cleared over its entire reach. According to the EC3, the resistance to spillage of the profile is verified by the following condition $M_{ysd} < M_{brd}$

$$M_{ysd} = \frac{q_{uz} \times L^2}{8} = \frac{2.59 \times 6^2}{8}$$

$$M_{ysd} = 11.65 \text{ KN.m}$$

$$\beta_w = 1 \text{ (Class 1).}$$

$$\lambda_{LT} = \frac{\frac{L}{i_z}}{\sqrt{C1} \left[1 + \frac{1}{20} \left(\frac{\frac{L}{i_z}}{\frac{h}{t_f}} \right)^2 \right]^{0.25}}$$

(Annexe F EC03 partie 1-1)

K=1 appui simple

So :

$$C1 = 1.132$$

$$\lambda_{LT} = \frac{\frac{600}{1.84}}{\sqrt{1.132} \left[1 + \frac{1}{20} \left(\frac{\frac{600}{1.84}}{\frac{16}{0.74}} \right)^2 \right]^{0.25}} = 163.41$$

$$\bar{\lambda}_{LT} = \frac{\lambda_{LT}}{93.9\varepsilon} = \frac{163.41}{93.9 \times 1} = 1.74$$

1.74 > 0.4 so there is a risk of spillage

Laminated Profile = 0.21α

$$X_{LT} = \frac{1}{\varphi_{LT} + [\varphi_{LT}^2 - \bar{\lambda}_{LT}^2]^{0.5}}$$

$$X_{LT} = \frac{1}{2.17 + [2.17^2 - 1.74^2]^{0.5}} = 0.35$$

$$M_{brd} = \frac{X_{LT} \times \beta_w \times W_{ply} \times f_y}{\gamma M_0}$$

$$M_{brd} = \frac{0.35 \times 1 \times 123.9 \times .275}{1.1} = 10.84 \text{ KN.m}$$

$$M_{ysd} = 11.65 \text{ KN.m} > M_{brd} = 10.84 \text{ KN.m}$$

Risk of spillage so we add the liernes.

If the liernes are added, the length is divided into two $\Rightarrow L = 3 \text{ m}$

Same previous application we change the length 6 [m] by 3 [m],

The following values are obtained:

$$\lambda_{LT} = 109,44$$

$$\bar{\lambda}_{LT} = 1.17$$

$$\varphi_{LT} = 1,28$$

$$X_{LT} = 0,55$$

$$M_{brd} = 17,036 \text{ Kn.m} > M_{ysd} = 11.65 \text{ KN.m}$$

Spillage checked for **IPE160 section** purlins (with linkers)

III.3 Study of tie rods:

III.3.1 Introduction:

The tie rods operate in tension, they are formed of cylendric shapes or small angles, their primary function is to prevent lateral distortion of the purlins.

III.3.2 Calculation of the maximum force of the tie rod:

The reaction R at the level of 1st ivy

$$Q_{yEd} = 0.038 \text{ kN/m}$$

$$R = \frac{1.25 Q_{yEd} \times L}{2} = \frac{1.25 \times 0.038 \times 6}{2} = 0.1425 \text{ KN}$$

$$\theta = \tan^{-1} \frac{1.2}{3} = 21.8^\circ$$

$$\text{Lierne L1 } T1 = R = 0.1425 \text{ KN}$$

$$\text{Lierne L2 } T2 = R + T1 = 0.285 \text{ KN}$$

$$\text{Lierne L3 } T3 = R + T2 = 0.4275 \text{ KN}$$

$$\text{Lierne L4 } T4 = R + T3 = 0.57 \text{ KN}$$

$$\text{Lierne L5 } T5 = R + T4 = 0.7125 \text{ KN}$$

$$\text{Lierne L6 } T6 = R + T5 = 0.855 \text{ KN}$$

$$\text{Lierne L7 } T7 = R + T6 = 0.9975 \text{ K}$$

$$\text{Lierne L8 } T8 = R + T7 = 1.14 \text{ KN}$$

$$\text{Lierne L9 } T9 = R + T8 = 1.28 \text{ KN}$$

$$\text{Lierne L10 } T10 = R + T9 = 1.42 \text{ KN}$$

$$\text{Lierne L11 } T11 = R + T10 = 1.56 \text{ KN}$$

$$\text{Lierne L12 } T12 = R + T11 = 1.7 \text{ KN}$$

$$\text{Lierne L13 } T13 = R + T12 = 1.84 \text{ KN}$$

$$\text{Lierne L14 } T14 = T13 / (2 \times \sin \theta) = 2.47 \text{ KN}$$

III.3.3 Sizing of the tie rods:

The most used ransom is L14

$$N_{Ed} \leq N_{pl,rd}$$

$$N_{Ed} \leq \frac{A_s \cdot f_y}{\gamma_{M0}} \rightarrow A_s \leq \frac{N_{Ed} \cdot \gamma_{M0}}{f_y}$$

$$A_s \geq \frac{2.47 \times 1.1}{0.275} = 9.88 \text{ mm}^2$$

$$A_{s-min} \geq \frac{\pi \times d_{min}^2}{4} \rightarrow d_{min} \geq \sqrt{\frac{4 \times A_{s-min}}{\pi}}$$

$$d_{min} \geq \sqrt{\frac{4 \times 9.88}{\pi}} = 3.54 \text{ mm} \quad ; \quad d_{min} = 10 \text{ mm}$$

III.4 Calculation of the strut:

The strut is a fastening tool that is used to fix purlins on the structure of the work.

It resists mainly at the moment of overturning due to the load, particularly under the effect of the wind lifting.

The spacing 't' is limited by the following condition

$$2\left(\frac{b}{2}\right) \leq t \leq 3\left(\frac{b}{2}\right)$$

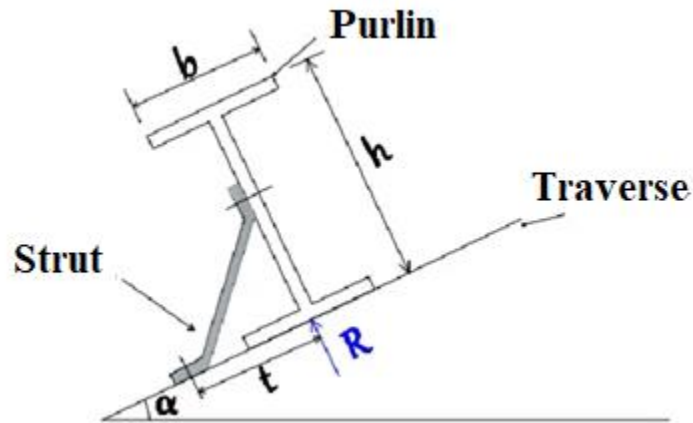


FIGURE III 5 Cross section of the sample

The purlins are **IPE160** with

B = 8.2 cm and h = 16 cm

$$2\left(\frac{8.2}{2}\right) \leq t \leq 3\left(\frac{8.2}{2}\right)$$

8.2cm < t < 12.3 cm; so, we adopt t=10 cm

Under the most unfavourable load combination $G \cdot \cos -1.5W\alpha$

The combination becomes $G_z - 1.5W_z$

$G = 0.33 \text{ KN/m}$

$G_z = G \cdot \cos \alpha = 0.33 \times \cos (5.71) = 0.328 \text{ KN}$

$P_{uz} = - G_z + 1.5W_z = - 0.382 + 1.5 \times 2.02 = 2.648 \text{ kN/m}^2$

Load returning to failure per ml and as account of continuity

The condition is verified

Strut Bending Risk Check:

$MR \leq M_{Pliage}$

With $MR = R \cdot t$

For our case we have PEI 160

$MR = R \cdot t = 24.825 \times 0.1 = 2.48 \text{ kN/m}$

The Struts are cold-formed elements, the class of the section is at least class 3

$$M_{plage} = \frac{W_{ely} \times f_y}{\gamma_{m0}} \leftrightarrow W_{ely} \geq \frac{M_R \times \gamma_{m0}}{f_y}$$

$$W_{ely} \geq \frac{248 \times 1.1}{27.5}$$

$$W_{ely} \geq 9.92 \text{ cm}^3$$

$$\text{Pour une section rectangulaire: } W_{ely} = \frac{b \times e^2}{6}$$

Note the width of the sample 'b' is taken after having dimensioned the crossbar, we take b = 17 cm

$$W_{ely} = \frac{b \times e^2}{6}$$

$$e = \sqrt{\frac{9.92 \times 6}{17}} = 1.87 \text{ cm}$$

So, we adopt a **20 mm** thick sample

III.5 Study of the cladding beams :

III.5.1 Definition:

Beams are horizontal elements used to support cladding and wind loads. Made of rolled profiles or folded sheets, they are fixed to the posts with a constant centre distance, depending on the permissible span of the badging trays.

PP loads and loads :

of cladding 11 kg/m²

Fastener Accessory PP 2kg/m²

PP d'UPN 14kg/ml

Spacing between beams e = 1.36 m

$$G = (11+2) \times 1.36 + 14 = 31.68 \text{ Kg/ml}$$

$$G = 0.316 \text{ kN/m}$$

Climatic wind loads:

Maximum wind load on the long W pan (internal overpressure)

$$W = 1.28 \times 1.36 = 1.75 \text{ kN/m}$$

The long-sided beams are supported on two 6m span posts, subject to bending

Bi-axial.

ULS:

Sur l'axe Z-Z $q_{uz} = 1.5 \times 1.75 = 2.625 \text{ Kn/ml}$.

Sur l'axe Y-Y $q_{uy} = 1.35 \times 0.316 = 0.426 \text{ Kn/ml}$.

SLS:

Sur l'axe Z-Z $q_{sz} = 1.75 \text{ Kn/ml}$.

Sur l'axe Y-Y $q_{sy} = 0.316 \text{ Kn/ml}$.

III.5.2 Deflection Condition;

The arrow to the els is done with the loads and service overloads $f \leq f_{adm}$

For a beam with two uniformly spaced supports (Z-Z axis)

$$f_{adm} \leq \frac{L}{200} \text{ avec : } f_z = \frac{5 \times q_{sz} \times L^4}{384 \times E \times I_y}$$

$$\text{et : } \frac{L}{200} = \frac{600}{200} = 3 \text{ cm.}$$

$$I_y \geq \frac{5 \times q_{sz} \times L^4}{384 \times E \times 3}$$

$$I_y \geq \frac{5 \times 1.75 \times 10^{-2} \times 600^4}{384 \times 21000 \times 3}$$

$I_y \geq 468.75$ Which corresponds to a **UPN 140** with $I_y = 605 \text{ cmcm}^4$.

TABULAR III 5 Characteristics of UPN140.

Profile	h (mm)	b (mm)	tw (mm)	TF (mm)	r (mm)	d(mm)	P (kg/m)
UPN140	140.0	60.0	7.0	10.0	10.0	97.9	16
	A(cm ²)	$I_y(\text{cm}^4)$	$i_y(\text{cm})$	$W_{ply}(\text{cm}^3)$	$I_z(\text{cm}^4)$	$i_z(\text{cm})$	$W_{plz}(\text{cm}^3)$
	20.4	605	5.45	103	63	1.75	28

The load G then becomes:

$$G = (11+2) \times 1.36 + 16 = 33.68 \text{ Kg/m}$$

$$G = 0.336 \text{ KN/m}$$

Combination at the ULS:

$$Q_{uy} = (1.35 G) = 1.35 \times 0.336 = 0.435 \text{ kN/m}$$

Combination at the SLS:

$$Q_{sy} = G = 0.336 \text{ kN/m}$$

Checking the Boom Condition :

a) According to Z-Z axis:

$$f_z = \frac{5 \times q_{sz} \times L^4}{384 \times E \times I_y} = \frac{5 \times 1.75 \times 10^{-2} \times 600^4}{384 \times 21000 \times 605}$$

$$f_z = 2.32 \text{ cm} < f_{adm} = 3 \text{ cm}$$

b) According to Y-Y axis:

$$f_y = \frac{5 \times q_{sy} \times L^4}{384 \times E \times I_z} = \frac{5 \times 0.336 \times 10^{-2} \times 600^4}{384 \times 21000 \times 63}$$

$$f_y = 0.42 < f_{adm} = 3 \text{ cm}$$

The deflection is checked on both axes so we adopt a **UPN180** cladding beam.

III.5.2.1 Condition of Resistance:

Sur l'axe Z-Z :

$$q_{uz} = 1.5 W = 1.5 \times 1.75 = 2.62 \text{ KN/ml.}$$

Sur l'axe Y-Y :

$$q_{uy} = 1.35 G = 1.35 \times 0.336 = 0.453 \text{ KN/ml.}$$

a) Deflected bending verification :

Z-Z axis:

$$M_{ysd} = \frac{q_{uz} \times L^2}{8} = \frac{2.62 \times 6^2}{8} = 11.8 \text{ KN.m}$$

Y-Y axis:

$$M_{zsd} = \frac{q_{uy} \times L^2}{8} = \frac{0.453 \times 6^2}{8} = 2.03 \text{ KN.m}$$

Determination of the class of profiles :

- web:

$$\varepsilon = \sqrt{\left(\frac{275}{f_y}\right)} = \sqrt{\left(\frac{275}{275}\right)} = 1$$

$$\frac{d}{t_w} = \frac{97.9}{7} = 13.98 < 72\varepsilon = 72 \times 1 = 72 \Rightarrow \text{Web class 1}$$

- Flange:

$$\frac{c}{t_f} = \frac{30}{10} = 3 < 10\varepsilon = 10 \times 1 = 10 \Rightarrow \text{Flange class 1}$$

So the section is class 1 $\Rightarrow \gamma_{m0} = 1$.

$$M_{pl} = \frac{W_{pl} \times f_y}{\gamma_{m0}}$$

$$M_{pl,y,rd} = \frac{W_{ply} \times f_y}{\gamma_{m0}} = \frac{103 \times 27.5 \times 10^{-2}}{1} = 28.32 \text{ KN.m}$$

$$M_{pl,z,rd} = \frac{W_{plz} \times f_y}{\gamma_{m0}} = \frac{28 \times 27.5 \times 10^{-2}}{1} = 7.7 \text{ KN.m}$$

Therefore:

$$\left(\frac{M_{ysd}}{M_{pl,y,rd}}\right)^\alpha + \left(\frac{M_{zsd}}{M_{pl,z,rd}}\right)^\alpha \leq 1$$

$$\left(\frac{11.8}{28.32}\right)^1 + \left(\frac{2.03}{7.7}\right)^1 \leq 1$$

$0.68 < 1$; The biaxial bending is verified.

b) Shear Verification:

We use the following equation:

$$V_{zsd} < V_{plz,rd} \text{ and } V_{ysd} < V_{ply,rd}$$

- Axis (Z-Z) :

$$V_{zsd} = \frac{q_{uz} \times L}{2} = \frac{2.62 \times 6}{2}$$

$$V_{zsd} = 7.86 \text{ KN.}$$

$$A_{vz} = A - 2 \times b \times t_f + (t_w + r)t_f = 20.4 - 2 \times 6 \times 1 + (0.7 + 1)1$$

$$A_{vz} = 10.1 \text{ cm}^4$$

$$V_{plz,rd} = A_{vz} \times \frac{f_y}{\sqrt{3} \times \gamma_{m0}} = 10.1 \times \frac{27.5}{\sqrt{3} \times 1.1}$$

$$V_{plz,rd} = 145.78 \text{ KN.}$$

- Axis (Y-Y) :

$$V_{ysd} = \frac{q_{yz} \times L}{2} = \frac{0.453 \times 6}{2}$$

$$V_{ysd} = 1.36 \text{ KN.}$$

$$A_{vy} = A - A_{vz} = 20.4 - 10.1$$

$$A_{vy} = 10.3 \text{ cm}^2$$

$$V_{ply,rd} = A_{vy} \times \frac{f_y}{\sqrt{3} \times \gamma_{m0}} = 10.3 \times \frac{27.5}{\sqrt{3} \times 1.1}$$

$$V_{ply,rd} = 148.66 \text{ KN.}$$

$$V_{zsd} = 7.86 \text{ KN} < V_{plz,rd} = 145.78 \text{ KN}$$

$$V_{ysd} = 1.36 \text{ KN} < V_{ply,rd} = 148.66 \text{ KN}$$

The shear strength of **UPN180** is verified.

C) Spill Verification:

Superior flange :

It is securely anchored to the roof, so there is no risk of spillage

Lower sole:

According to the EC3, the spill resistance can be verified by :

$$M_{ysd} < M_{brd}$$

(Eurocode 3 (IV.4))

$$M_{ysd} = \frac{q_{uz} \times L^2}{8} = \frac{2.62 \times 6^2}{8}$$

$$M_{ysd} = 11.8 \text{ KN.m}$$

$$\beta_w = 1 \text{ (Class 1).}$$

$$\lambda_{LT} = \frac{\frac{L}{i_z}}{\sqrt{C1} \left[1 + \frac{1}{20} \left(\frac{\frac{L}{i_z}}{\frac{h}{t_f}} \right)^2 \right]^{0.25}}$$

(Annex F EC03 Part 1-1)

K=1 single press

(Table F.1.2 Eurocode 3)

So:

$$C1 = 1.132$$

$$\lambda_{LT} = \frac{\frac{600}{0.75}}{\sqrt{1.132} \left[1 + \frac{1}{20} \left(\frac{\frac{600}{0.75}}{\frac{14}{1}} \right)^2 \right]^{0.25}} = 172.26$$

$$\bar{\lambda}_{LT} = \frac{\lambda_{LT}}{93.9\varepsilon} = \frac{172.26}{93.9 \times 1} = 1.83$$

$0.28 < 0.4$ so there is no risk of spillage.

Therefore: $X_{LT} = 1$

$$M_{brd} = \frac{X_{LT} \times \beta_w \times W_{ply} \times f_y}{\gamma M0}$$
$$M_{brd} = \frac{1 \times 1 \times 103 \times 27.5 \times 10^{-2}}{1.1} = 25.75 \text{ KN.m}$$
$$M_{ysd} = 11.8 \text{ KN.m} < M_{brd} = 25.75 \text{ KN.m}$$

UPN180 purlins are verified for spill stability.

III.5.3 Gable Cladding Beams:

III.5.3.1 Evaluation of charges:

a) Dead loads :

PP cladding 12.5 kg/m²

Fastener Accessory PP 2kg/m²

Smooth PP 14kg/ml

Spacing between beams e= 1.36 m

$$G = (11+3) \times 1.6 + 14 = 33.04 \text{ kg/ml}$$

$$G = 0.33 \text{ kN/m}$$

b) Climatic wind loads :

Maximum wind load on the pinion $W = -2518.425 \text{ N/m}^2$

$$C_{pi} = +0.35.W = -2.51 \times 1.36 = -3.41 \text{ KN/m.}$$

ULS:

$$\text{Sue z-z } q_{uz} = 1.5 \times -3.41 = -5.11 \text{ KN/ml.}$$

$$\text{Sur y-y } q_{uy} = 1.35 \times 0.33 = 0.44 \text{ KN/ml.}$$

SLS:

Sur z-z $q_{sz} = -3.41$ KN/ml.

Sur y-y $q_{sy} = 0.33$ KN/ml

III.5.3.2 Deflection Condition:

$f \leq f_{adm}$

$$f_{adm} \leq \frac{L}{200} \text{ avec : } f_z = \frac{5 \times q_{sz} \times L^4}{384 \times E \times I_y}$$

$$\text{et : } \frac{L}{200} = \frac{600}{200} = 3 \text{ cm.}$$

$$I_y \geq \frac{5 \times q_{sz} \times L^4}{384 \times E \times 3}$$

$$I_y \geq \frac{5 \times 3.41 \times 10^{-2} \times 600^4}{384 \times 21000 \times 3}$$

$I_y \geq 913.4$ Which corresponds to a **UPN 180** with $I_y = 1350 \text{ cm}^4$.

TABULAR III 6 Characterisric of UPN 180

Profile	h(mm)	b(mm)	t_w (mm)	t_f (mm)	r(mm)	d(mm)	P(Kg/m)
UPN180	180.0	70.0	8.0	11.0	11.0	133.4	22
	A(cm ²)	I_y (cm ⁴)	i_y (cm)	W_{ply} (cm ³)	I_z (cm ⁴)	i_z (cm)	W_{plz} (cm ³)
	28	1350	69.5	179.13	114	20.5	43.06

The load G then becomes:

$$G = (11+2) \times 1.36 + 22 = 39.68 \text{ Kg/m} \quad G = 0.396 \text{ KN/m}$$

The combination at the ULS:

$$q_{uy} = (1.35 G) = 1.35 \times 0.396 = 0.534 \text{ KN/m}$$

The combination at the SLS:

$$q_{sy} = G = 0.396 \text{ KN/m}$$

Verification of the condition of the deflection a) Verification of the deflection along the Z-Z axis:

$$f_z = \frac{5 \times q_{sz} \times L^4}{384 \times E \times I_y} = \frac{5 \times 3.41 \times 10^{-2} \times 600^4}{384 \times 21000 \times 1350}$$

$$f_z = 2.03\text{cm} < f_{\text{adm}} = 3\text{cm}$$

The deflection is checked on both axes so a **UPN180** is adopted.

b) Verification of the deflection along the Y-Y axis

$$f_y = \frac{5 \times q_{sy} \times L^4}{384 \times E \times I_z} = \frac{5 \times 0.396 \times 10^{-2} \times 600^4}{384 \times 21000 \times 114}$$

$$f_y = 2.79 < f_{\text{adm}} = 3\text{cm}$$

The deflection is checked on both axes so a **UPN180** is adopted.

III.5.3.3 Resistance Condition (ULS):

On z-z $q_{uz} = 1.5 \text{ W} = 1.5 \times 3.41 = 5.11 \text{ KN/ml}$.

On y-y $q_{uy} = 1.35 \text{ G} = 1.35 \times 0.396 = 0.534 \text{ KN/ml}$.

a) Deflected bending design:

Z-Z axis:

$$M_{ysd} = \frac{q_{uz} \times L^2}{8} = \frac{5.11 \times 6^2}{8} = 23 \text{ KN.m}$$

Y-Y axis:

$$M_{zsd} = \frac{q_{uy} \times L^2}{8} = \frac{0.534 \times 6^2}{8} = 2.40 \text{ KN.m}$$

Class of profiles :

Web:

$$\varepsilon = \sqrt{\left(\frac{275}{f_y}\right)} = \sqrt{\left(\frac{275}{275}\right)} = 1$$

$$\frac{d}{t_w} = \frac{133.4}{8} = 16.67 < 72\varepsilon = 72 \times 1 = 72 ; \text{Class 1 web}$$

Flange:

$$\frac{c}{t_f} = \frac{68}{11} = 6.18 < 10\varepsilon = 10 \times 1 = 10 ; \text{Class 1 flange}$$

So, the cross-section is class 1 $\Rightarrow \gamma_{m0} = 1$

$$M_{pl} = \frac{W_{pl} \times f_y}{\gamma_{m0}}$$

$$M_{pl,y,rd} = \frac{W_{ply} \times f_y}{\gamma_{m0}} = \frac{179.13 \times 27.5 \times 10^{-2}}{1} = 49.26 \text{ KN.m}$$

$$M_{pl,z,rd} = \frac{W_{plz} \times f_y}{\gamma_{m0}} = \frac{43.06 \times 27.5 \times 10^{-2}}{1} = 11.84 \text{ KN.m}$$

So:

$$\left(\frac{M_{ysd}}{M_{pl,y,rd}} \right)^\alpha + \left(\frac{M_{zsd}}{M_{pl,z,rd}} \right)^\alpha \leq 1$$

$$\left(\frac{23}{49.26} \right)^1 + \left(\frac{2.40}{11.84} \right)^1 \leq 1$$

0.78 < 1 the biaxial bending is true.

b) Shear Verification :

We use the following formula:

$$V_{zsd} < V_{plz,rd} \text{ and } V_{ysd} < V_{ply,rd}$$

(Eurocode. 3 p158)

- Axis (Z-Z) :

$$V_{zsd} = \frac{q_{uz} \times L}{2} = \frac{5.11 \times 6}{2}$$

$$V_{zsd} = 15.33 \text{ KN.}$$

$$A_{vz} = A - 2 \times b \times t_f + (t_w + r)t_f = 28 - 2 \times 7 \times 1.1 + (1.1 + 0.8)1.1$$

$$A_{vz} = 14.69 \text{ cm}^2$$

$$V_{plz,rd} = A_{vz} \times \frac{f_y}{\sqrt{3} \times \gamma_{m0}} = 14.69 \times \frac{27.5}{\sqrt{3} \times 1.1}$$

$$V_{plz,rd} = 212.03 \text{ KN.}$$

- Axis (Y-Y) :

$$V_{ysd} = \frac{q_{yz} \times L}{2} = \frac{0.534 \times 6}{2}$$

$$V_{ysd} = 1.60 \text{ KN.}$$

$$A_{vy} = A - A_{vz} = 28 - 14.69$$

$$A_{vy} = 13.31 \text{ cm}^2$$

$$V_{ply,rd} = A_{vy} \times \frac{f_y}{\sqrt{3} \times \gamma_{m0}} = 13.31 \times \frac{27.5}{\sqrt{3} \times 1.1}$$

$$V_{ply,rd} = 192.11 \text{ KN.}$$

$$V_{zsd} = 15.33 \text{ KN} < V_{plz,rd} = 212.03 \text{ KN}$$

$$V_{ysd} = 1.60 \text{ KN} < V_{ply,rd} = 192.11 \text{ KN}$$

Therefore shear strength of the cladding beam **UPN 180** is verified.

C) Spill CheckTop Sole:

It is possible that the upper sole, compressed under the effect of downward vertical loads, will tip over. Since it is firmly anchored to the roof, there is no risk of tipping over. The lower footing, compressed by the effect of the lifting wind, can spill as soon as it is cleared over its entire reach.

According to Eurocode 3, the spill resistance of the profile is verified if the following condition is met : $M_{ysd} < M_{brd}$

C1: factor depending on the load and embedding conditions.

X_{LT} :is the reduction coefficient for the spillage as a function of the reduced slenderness.

$\bar{\lambda}_{LT}$:is the coefficient of imperfection.

$$M_{ysd} = \frac{q_{uz} \times L^2}{8} = \frac{5.11 \times 6^2}{8}$$

$$M_{ysd} = 23 \text{ KN.m}$$

$$\beta_w = 1 \text{ (Class 1).}$$

$$\lambda_{LT} = \frac{\frac{L}{i_z}}{\sqrt{C1} \left[1 + \frac{1}{20} \left(\frac{\frac{L}{i_z}}{\frac{h}{t_f}} \right)^2 \right]^{0.25}}$$

(Annex F EC03 Part 1-1)

K=1 single press

(Table F.1.2 Eurocode 3)

So:

$$C1 = 1.132$$

$$\lambda_{LT} = \frac{\frac{600}{20.5}}{\sqrt{1.132} \left[1 + \frac{1}{20} \left(\frac{600}{\frac{20.5}{1.1}} \right)^2 \right]^{0.25}} = 26.50$$

$$\bar{\lambda}_{LT} = \frac{\lambda_{LT}}{93.9\epsilon} = \frac{26.50}{93.9 \times 1} = 0.28$$

$0.28 < 0.4$ donc il n'y a pas risque de déversement

Therefore: $X_{LT} = 1$

$$M_{brd} = \frac{X_{LT} \times \beta_w \times W_{ply} \times f_y}{\gamma_{M0}}$$

$$M_{brd} = \frac{1 \times 1 \times 179.13 \times 27.5 \times 10^{-2}}{1.1} = 44.78 \text{ KN.m}$$

$$M_{ysd} = 23 \text{ KN.m} < M_{brd} = 44.78 \text{ KN.m}$$

The spill stability of **UPN 180** purlins is checked.

III.6 Sleeper Design:

III.6.1 Definition:

The main beams of a gable roof are called transoms. They are usually manufactured in IPE or HEA profiles.

The function of the roof sleepers is to support the roof components and to anchor the loads and loads applied to them.

III.6.2 Pre-dimensioning of sleepers:

Evaluation of loads and overloads:

a) Permanent load G :

Panel covers (sandwiches) 13.2 kg/m^2

Installation accessory 1.5 kg/m^2

IPE160 purlins 15.8 kg/m

The span of the crossbeam 12 m

Purlin spacing $e = 1.2 \text{ m}$

$$G = (P_{\text{couverture}} + P_{\text{acc}}) \times e + (P_{\text{panne}} \times n_b \times l_p)$$

$$G = (13.2 + 1.5) \times 1.2 + (15.8 \times 8 \times 6) = 776.04 \text{ kg/m}$$

$$G = 7.76 \text{ KN/m}$$

b) Climatic wind load (perpendicular to the wind):

The most charged area is G of which $C_{pi} = 0.35$

$$W = -q_j = 2.02 \times 12 = 24.24 \text{ KN/m}$$

$$\text{Joist span} = 12 \text{ m}$$

c) Climatic snow load (by horizontal projection):

$$S = 0.1025 \times 6 \times 8 = 4.92 \text{ KN/m}$$

Combinations of the most unfavourable loads:

Loads and surcharges applied

$$G = 7.76 \text{ KN/m}$$

$$Q = 0.444 \text{ KN/m}$$

$$W = 24.24 \text{ KN/m}$$

$$S = 4.92 \text{ KN/m}$$

The worst-case combination

$$1.35G + 1.5(W + S) = 1.35 \times 7.76 + 1.5(24.24 + 4.92)$$

$$= 54.21 \text{ KN/m}$$

Deflection Condition Check

$$f_{adm} = L/250 = 1200/250 = 6 \text{ cm}$$

$$f_z = \frac{5 \times Q \times L^4}{384 \times E \times I_y}$$

$$I_y > \frac{5 \times Q \times L^4}{384 \times E \times f_{adm}} = \frac{5 \times 54.21 \times 10^{-2} \times 1200^4}{384 \times 2.1 \times 10^5 \times 6}$$

$$I_y > 11616.42 \text{ cm}^4$$

We take IPE 360 ($I_y = 16270 \text{ cm}^4$)

The Resistance:

$$q_u = 56.16 \text{ KN/m}$$

$$M_{sd} = \frac{q_u \times L^2}{12} = \frac{56.16 \times 12^2}{12} = 673.92 \text{ KN.m}$$

$$B_n = \frac{W_{pl} \times f_y}{\gamma_{mo}} = \frac{1019 \times 2.75}{1.1} = 2547.2 \text{ KN.m}$$

$M_{brd} > M_{sd} \rightarrow$ Resistance is verified.

Shear:

$$V_{sd}^{max} = \frac{q_u \times L}{2} = \frac{56.16 \times 12}{2} = 336.72 \text{ KN}$$

$$V_{rd} = \frac{A_{vz} \times f_y}{\sqrt{3} \times \gamma_{mo}} = \frac{32.822 \times 275}{\sqrt{3} \times 1.1} = 498.67 \text{ KN}$$

$$V_{rd} > V_{sd}^{max}$$

The shear is checked, so we adopt an **IPE 360**.

III.7 Column Design:**III.7.1 Definition:**

Columns are vertical elements of the framework, are subject to compression and sometimes bending under the effect of climatic loads, in particular snow and wind.

III.7.2 Pre-dimensioning of columns:**a) Dead loads :**

$$\text{Influence area } S = 10 \times 6 = 60 \text{ m}^2$$

$$\text{Covers (sandwich panels)} 0.132 \times 60 = 7.92 \text{ KN}$$

$$\text{Installation accessory } 0.15 \times 60 = 9 \text{ KN}$$

$$\text{La panne IPE 160 } 0.158 \times 6 \times 6 = 5.688 \text{ KN}$$

$$\text{Solives IPE 200 } 0.224 \times 60 = 14.64 \text{ KN}$$

$$\text{IPE 360 beam } 0.571 \times 6 \times 2 = 6.852 \text{ KN}$$

$$\text{Sheet weight } 0.085 \times 6 \times 6 = 2.36 \text{ KN}$$

$$\text{Concrete Weight} = 0.10 \times 6 \times 25 = 11.55 \text{ KN}$$

b) Climatic snow loads :

$$S_n = 0.1025 \times 60 = 6.15 \text{ KN}$$

So, the snow load is $S = S_n \times \cos(\alpha) = 6.15 \times \cos(5.71) = 5.16 \text{ KN}$

a) Operating Overloads :

Maintenance load $Q = 0.35 \times 12 = 4.2 \text{ KN}$

So, $Q = Q \times \cos(\alpha) = 4.2 \times \cos(5.71) = 3.52 \text{ KN}$

Determination of the N Effort :

➤ Dead loads:

$G_t = (G_{\text{couverture}} + G_{\text{acc}} + G_{\text{panne}} + G_{\text{solive}}) \times \cos(5.71) + G_{\text{dalle}}$

$G_t = (7.92 + 9 + 5.688 + 14.64) \times \cos(5.71) + 74.15 = 111.21 \text{ KN}$

➤ Operating expenses:

$Q_t = Q_{\text{slab}} + Q_{\text{maintenance}} = 66.82 + 3.52 = 70.34 \text{ KN}$

➤ ULS:

$N_{\text{max}} = 1.35G + 1.5(Q + S)$

$N_{\text{max}} = 1.35 \times 111.21 + 1.5(70.34 + 5.16)$

$N_{\text{max}} = 263.383 \text{ KN}$

$$N_{\text{max}} = \frac{A \times f_y}{\gamma_{m0}}$$

$$A = \frac{N_{\text{max}} \times \gamma_{m0}}{f_y} = \frac{263383 \times 1.1}{2750}$$

$$A = 105.35 \text{ cm}^2$$

For the section obtained, we oversize our section and we adopt for an HEA 300 for safety reasons.

Buckling Checks:

The following condition must be verified

$$N_{\text{max}} \leq \frac{\beta_a \times f_y \times A \times \chi_{\text{min}}}{\gamma_{m1}}$$

Calculation of the reduced slenderness $\bar{\lambda}_i$

$$L_{ky} = 0.7 \times 10 = 7 \text{ m}$$

$$L_{kz} = 0.7 \times 10 = 7 \text{ m}$$

$$\lambda_y = L_{ky} / i_y = 7 / 0.127 = 55.11$$

$$\lambda_z = L_{kz} / i_z = 7/0.0749 = 93.45$$

$$\bar{\lambda}_y = \frac{\lambda_y \times \sqrt{\beta_a}}{93.9} = \frac{55.11 \times 1}{93.9} = 0.58$$

Around yy' → Curve at → $\chi_y = 0.89$

$$\bar{\lambda}_z = \frac{\lambda_z \times \sqrt{\beta_a}}{93.9} = \frac{93.45 \times 1}{93.9} = 0.99$$

Around zz' → Curve b → $\chi_z = 0.67$

$$\chi_{\min} = \min(\chi_y, \chi_z) = 0.67$$

$$N_{cal} = \frac{\beta_a \times f_y \times A \times \chi_{\min}}{\gamma_{m1}} = \frac{1 \times 275 \times 11250 \times 0.67}{1.1}$$

$$N_{cal} = 1884.37 \text{ KN}$$

$$N_{\max} = 256.89 \text{ KN} < N_{cal} = 1884.37 \text{ KN} \rightarrow \text{Buckling Verified Post}$$

The **HEA300** Post is checked.

III.7.3 Deduction:

According to all the verifications carried out above, the following results were obtained:

IPE360 for crossbeams.

HEA300 for Posts.

III.8 Calculation of bracing:

III.8.1 The forces acting on the wind beam:

$$F_i = B \times W + \frac{F_r}{n}$$

I_f :Surface to wind.

W: Worst wind load.

F_r : Force of friction.

n :Number of nodes.

With n = 15 knots.

$$Y_{es} = \frac{h_i}{2} \times e_i$$

h_i length of the post.

No espacement.

TABULAR III 7 Values of wind forces due to the wind

knots	1	2	3	4	5	6	7	8
h _i (m)	10	10.36	10.6	10.84	11.08	11.32	11.56	11.8
e (m)	1.81	3.01	2.41	2.41	2.41	2.41	2.41	2.41
I _f (m ²)	9.05	15.6	12.77	13.06	13.35	13.64	13.92	14.22
W(KN/m ²)	1.58	1.58	1.58	1.58	1.58	1.58	1.58	1.58
Fr (KN)	0	0	0	0	0	0	0	0
Fi (KN)	14.3	24.64	20.17	20.95	21.09	21.55	22	22.46

F₂ is the most unfavourable.

Calculation of reactions:

$$\sum F = 0$$

$$R_A + R_B - 2F_1 - 2F_2 - 2F_3 - F_4 = 0$$

$$R_A + R_B = R$$

$$2R = 2F_1 + 2F_2 + 2F_3 + F_4$$

$$R =$$

$$\frac{2F_1 + 2F_2 + 2F_3 + 2F_4 + 2F_5 + 2F_6 + 2F_7 + F_8}{2} = \frac{2 \times 14.3 + 2 \times 24.64 + 2 \times 20.17 + 2 \times 20.95 + 2 \times 21.09 + 2 \times 21.55 + 2 \times 22 + 22.46}{2}$$

$$R = R_A = R_B = 155.93 \text{ KN}$$

Calculation of internal forces in the diagonal:

$$\theta = \tan^{-1}(3.62/6)$$

$$\theta = 75.55^\circ$$

$$\sum F = 0$$

$$F_d \cos \theta + F_1 = R$$

$$F_d = \frac{R - F_1}{\cos \theta} = \frac{155.93 - 14.3}{\cos (75.55)}$$

$$F_d = 141.63 \text{ KN}$$

$$N_{sd} = 1.5 \times F_d = 1.5 \times 141.63 = 212.44 \text{ KN}$$

Diagonal section:

$$N_{sd} \leq N_{pl,rd}$$

$$N_{pl,rd} = \frac{A \cdot f_y}{\gamma_{m0}}$$

$$A \geq \frac{N_{sd} \cdot \gamma_{m0}}{f_y} = \frac{212.44 \times 10^3 \times 1.1}{275}$$

$$A \geq 849.46 \text{ mm}^2 = 8.5 \text{ cm}^2$$

This corresponds to an angle iron **L 60×60×8**

Diagonal check:

$$N_{sd} \leq N_{t,rd}$$

$$N_{pl,rd} = \frac{A \cdot f_y}{\gamma_{m0}} = \frac{903 \times 275}{1.1} = 225.75 \text{ KN}$$

$$N_{t,rd} = \min[N_{pl,rd}, N_{u,rd}, N_{net,rd}]$$

$$N_{u,rd} = 0.9 \times A_{net} \times \frac{f_u}{\gamma_{m2}} = 0.9 \times 759 \times \frac{430}{1.25} = 234.98 \text{ KN}$$

$$N_{net,rd} = A_{net} \times \frac{f_u}{\gamma_{m0}} = 759 \times \frac{430}{1.1} = 296.7 \text{ KN}$$

$$A_{net} = A - e \times d_0 = 903 - 8 \times 18 = 759 \text{ mm}^2$$

With d_0 drilling section.

A_{net} Section of the angle after drilling.

$$\gamma_{m0} = 1.1$$

$$\gamma_{m2} = 1.25$$

$$f_u = 430 \text{ [N/mm}^2\text{]}$$

$$f_y = 275 \text{ [N/mm}^2\text{]}$$

$$N_{t,rd} = \min[225.75, 234.98, 296.7]$$

$$N_{t,rd} = 225.75 \text{ KN}$$

$$N_{sd} = 212.44 \text{ KN} < N_{t,rd} = 225.75 \text{ KN}$$

Condition check: we opt for angles of **L 60×60×8** for the wind beam.

The **UPN 160** profile is suitable as beam uprights in the wind.

CHAPTER IV

FLOOR STUDY

IV.1 Introduction :

Composite floors are classified as composite construction because they combine the attractive technical properties of steel and concrete.

Concrete is an ideal material for compressive loading, while steel is perfect for tensile work. Cold-formed steel sheets, also known as steel trays (lost formwork), in combination with the connectors, unite the concrete slab and the metal which together contribute to the strength of the floor. They are used for the construction of floors in various sectors.

IV.2 Definition :

Composite flooring combines steel and concrete to combine tensile and compressive strength, offering an efficient, economical and high-performance solution.

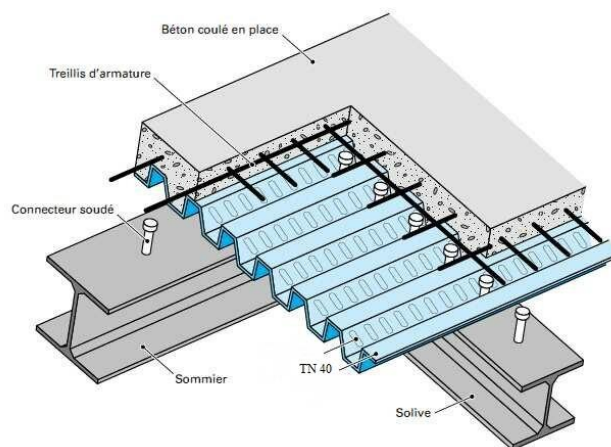


FIGURE IV 1 The Elements from floor collaborant

IV.3 Arrangement of joints:

Distance, between joist, $d = 1\text{m}$

Length, of joists $l = 6\text{m}$

Permanent loads :

-Weight of the reinforced concrete slab $G1 = \rho \cdot e = 2500 \times 0.1$ $G1 = 2.5 \text{ KN/m}^2$

-Sheet weight (TN40) + G2 accessory = 0.147 KN/m^2

$G = (G1 + G2) \times d = (2.5 + 0.147) \times 1 = 2.647 \text{ KN/m}$

$G = 2.647 \text{ KN/m}$

$Q = 0.1 \text{ kn/m}$

$$Q_s = G + Q$$

The eligible arrow:

$$f_{adm} = \frac{L}{250}$$

In the case of a uniformly loaded, two-hinged beam, the deflection is given by $f_{max} = \frac{5q_s l^4}{384 E I_y}$

The f_{max} check condition $f_{max} \leq f_{adm}$

$$\frac{5q_s l^4}{384 E I_y} \leq \frac{L}{250}$$

Therefore:

$$I_y \geq \frac{5q l^3 250}{384 E}$$

$$I_y \geq \frac{5 \times 2647 \times 250 \times 6^3}{384 \times 2.1 \times 10^5}$$

$$I_y \geq 886,272 \text{ cm}^4$$

So, we will choose IPE200 with $I_y = 1943 \text{ cm}^4$

IV.5 Evaluation of floor loads:

The dead loads G:

- Weight of the reinforced concrete slab $G1 = \rho \cdot e = 2500 \times 0.1 \text{ G1} = 2.5 \text{ kN/m}^2$
- Sheet weight (TN40) + accessory e $G2 = 0.147 \text{ kN/m}^2$
- Joist weight IPE180 $P = 22.4 \text{ kg/m}$

$$G = (G1 + G2) \times d + P = (2.5 + 0.147) \times 1 + 0.224 = 2.871 \text{ kN/m}$$

$$G = 2.871 \text{ kN/m}$$

IV.5.1 Combination of Loads:

➤ **At the ULS:**

$$\Sigma G + Q = 2.871 \text{ kN/m}$$

➤ **At the SLS:**

$$S = \gamma_G \cdot G + \gamma_Q \cdot Q = (1.35 \times 2.871) = 3.875 \text{ kN/m}$$

IV.5.2 Deflection Verification at ULS:

➤ **Verification of condition:**

$$f_{max} \leq f_{adm}$$

➤ **Calculation of the permissible deflection:**

$$f_{adm} = \frac{L}{250}$$

$$f_{adm} = 600/250 = 2.4 \text{ cm}$$

$$f_{adm} = 2.4 \text{ cm (1)}$$

➤ **Max deflection calculation:**

$$f_{max} = \frac{5q_s l^4}{384 E I_y}$$

$$f_{max} = \frac{5 \times 2835 \times 6^4}{384 \times 1317 \times 2.1 \times 10^5}$$

$$f_{max} = 1.18 \text{ cm (2)}$$

From (1) and (2) we have **Fmax** < **fadm** so the arrow is verified.

IV.5. 3 Shear force verification:

$$V_{sd} \leq V_{Pl, Rd}$$

$$V_{sd} = \frac{q_u \times l}{2}$$

$$V_{sd} = \frac{3.875 \times 6}{2}$$

$$V_{sd} = 11.625 \text{ KN}$$

$$V_{pl, rd} = \frac{A_{vz} \cdot \left(\frac{f_y}{\sqrt{3}} \right)}{\gamma_{M0}}$$

➤ **Determination of the profile class:**

Web:

$$\frac{d}{t_w} = \frac{159}{5.6} = 28.39 < 72 \varepsilon \text{ with : } \varepsilon = \sqrt{\frac{275}{275}} \rightarrow = 1$$

28.39 < 72 ; Then the web is class 1

Flange:

$$\frac{c}{t_f} = \frac{\frac{b}{2}}{t_f} = \frac{45.5}{8} = 5.68 < 10 \varepsilon = 10 \times 1 = 10$$

Class 1 flange

✓ **Class 1 Section**

$$A_v = A - 2b + (t_f t_w + 2r) t_f = 2850 - 2 \times 100 \times 8.5 + (5.6 + 2 \times 12) \times 8.5$$

$$A_v = 1401.6 \text{ mm}^2$$

$$V_{pl,rd} = \frac{1401.6 \times 235}{1.1 \times \sqrt{3}}$$

$$V_{pl,rd} = 172,877 \text{ KN}$$

✓ $V_{pl,rd} = 172.877 \text{ KN} \geq V_{sd} = 11.481 \text{ KN}$, So the shear strength is checked.

IV.5.4 Flexing moment (resistance) design:

$$M_{sd} \leq M_{pl,rd}$$

$$M_{sd} = \frac{q_u \times l^2}{8} = \frac{3.875 \times 6^2}{8} = 17,437 \text{ KN.m}$$

$$M_{pl,rd} = \frac{w_{ply} \times f_y}{\gamma_{m0}} = \frac{221 \times 275}{1.1} = 55.25 \text{ KN.m}$$

Therefore :

$$M_{sd} = 17,437 \text{ KN.m} < M_{pl,rd} = 55.25 \text{ KN.m}$$

✓ So the condition is checked

IV.6. Verification at final stage :

IV.6.1. Valuation of charges:

➤ Permanent Loads G:

• Weight of the slab of concrete reinforced:

$$G_1 = \rho \cdot E = 2500 \times 0.1 \quad G_1 = 2.5 \text{ kN/m}^2$$

• Sheet weight (TN40) + accessory: $G_2 = 0,15 \text{ kN/m}^2$

• Olive weight IPE180: $G_3 = 0.224 \text{ kN/m}^2$

• Tile Coating Weight+Sand + Mortar : $G_4 = 1.2 \text{ kN/m}^2$

• False ceiling weight: $G_5 = 0.1 \text{ kN/m}^2$

$$G_s = (G_1 + G_2 + G_4 + G_5) \cdot D + G_3$$

$$G_s = (2.5 + 0.15 + 1.2 + 0.1) \times 1 + 0.224 = 4.174 \text{ kN/m}$$

$$G_s = 4.174 \text{ KN/m}$$

➤ Les charges variables Q:

Operating overload: $P = 2.5 \text{ kN/m}^2$

$$Q = P \times d = 2.5 \times 1 = 2.5 \text{ kN/m}$$

$$Q = 2.5 \text{ KN/m}$$

IV.6.2 Load Combinations:

➤ at the SLS:

$$\Sigma G + Q = 7.138 + 2.5 = 9.674 \text{ kN/m} \quad \rightarrow \quad q_s = 9.674 \text{ kN/m}$$

➤ to the ULS:

$$\Sigma 1.35 \cdot G + 1.5 \cdot Q = (1.35 \times 7.174) + (1.5 \times 2.5) = 13.4349 \text{ kN/m}$$

$$q_u = 13.4349 \text{ kN/m}$$

IV.6.3 Calculation of the effective width of concrete:

$$B_{\text{eff}} = \min \left(2 \frac{l_0}{8} l_0 \times; b \right) \text{ with } = 6\text{m and } b = 1\text{m}$$

$$B_{\text{eff}} = \min \left(2 \times \frac{6}{8} ; 1 \right) = \min (1.5 ; 1)$$

$$B_{\text{eff}} = 1\text{m}$$

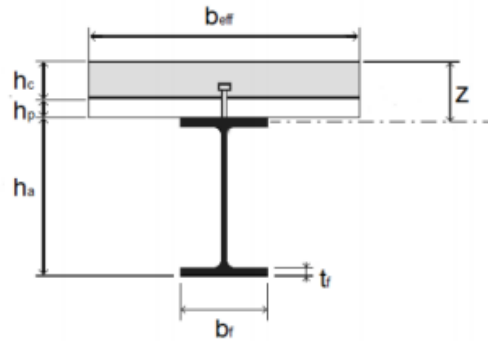


FIGURE IV 2 Representation of the mixed section

IV.6.4 The position of the neuter axis:

The position of the neuter axis (z) of the mixed section is given by the following formula

$$Z = \frac{t+h}{2 \times S} \times \frac{B_{\text{eff}} \times t}{n}$$

➤ Equivalence coefficient n :

$$n = \frac{E_a}{E_{cm}}$$

With:

-Effective modulus of elasticity of concrete $E_c' = \frac{E_{cm}}{3}$

•Secant modulus of elasticity for long-term E_{cm} loading

$$E_{cm} = 30.5 \text{ KN/mm}^2$$

Thus:

$$n = \frac{210000}{30500} = 20.65$$

$$n = 21$$

$$\triangleright S = A + \frac{B}{n}$$

$$\text{With: } B = b \cdot t \rightarrow B = 100 \times 10 = 1000 \text{ cm}^2$$

$$S = 28.5 + \frac{1000}{21}$$

$$S = 76.119 \text{ cm}^2$$

Then:

$$Z = \frac{100 \times 10}{21} \times \frac{10 + 20}{2 \times 76,119} = 9.383 \Rightarrow z = 9,383 < 10 \text{ cm}$$

✓ So, the axis is positioned in the flange of the beam.

IV.6.6 Shear force verification:

➤ Demanding Shearing Effort

$$V_{zed} = \frac{q_u \times l}{2} = \frac{13,4349 \times 6}{2} = 40.3 \text{ KN}$$

$$V_{zed} = 40.3 \text{ KN}$$

Resistant Shear Force:

$$V_{Z,rd} = \frac{A_{vz} \cdot \left(\frac{f_y}{\sqrt{3}}\right)}{\gamma_{M0}}$$

$$A_{vz} = d \times h = 159 \times 5.6 = 890.4 \text{ mm}^2$$

$$V_{Z,rd} = \frac{890,4 \times \left(\frac{275}{\sqrt{3}}\right)}{1,1} = 128.51 \text{ KN}$$

$$\frac{V_{zed}}{V_{Z,rd}} = 0.313 < 0.5$$

✓ So the shear force is checked

IV.6.7 Deflection Verification:

Checking the arrow at the SLS by verifying this relationship

$$f_{SER} < f_{ADM}$$

$$f_{SER} = \frac{5 \times q \times l^4}{384 \times E \times I} = \frac{5 \times 9.674 \times 6^4}{384 \times 2,1 \times 10^5 \times 4997,782} = 1,619 \text{ cm}$$

$$\text{And: } f_{adm} = \frac{L}{250} = \frac{600}{250} = 2,4 \text{ cm ; } f_{ser} = 1,619 \text{ cm}$$

$$f_{ser} < f_{adm}$$

✓ The arrow is checked

IV.6.8 Checking the bending moment:

For this verification the following condition is used

$$M_{sd} \leq M_{pl,rd}$$

$$M_{sd} = \frac{q_u \times l^2}{8} = 43.54 \text{ KN.m}$$

$$M_{pl,rd} = \frac{w_{ply} \times f_y}{\gamma_{m0}} = \frac{221 \times 275}{1.1} = 55.25 \text{ KN.m}$$

$$\text{Thus: } M_{sd} = 43,54 \text{ KN.m} < M_{pl,rd} = 55.25 \text{ KN.m}$$

✓ So the condition is checked

IV.7 Calculation and sizing of connectors:

IV.7.1 The number of connectors:

$$N_c \geq \frac{V_l}{P_{Rd}}$$

IV.7.2 Calculation of the longitudinal shear force V_l :

$$V_l = \min F_{cf} = \left\{ \begin{array}{l} \frac{A_a \times f_y}{\gamma_a} \\ \frac{0,085 \times A_c \times F_{ck}}{\gamma_c} + \frac{A_{se} \times f_{sk}}{\gamma_s} \end{array} \right\}$$

By designating :

$$A_a = 28.5 \text{ cm}^2$$

The area of the steel structural member

$$A_c = b_{eff} \times t = 100 \times 10 = 1000 \text{ cm}^2 . \text{ The area of the effective concrete cross-section}$$

$$f_{ck} = 25 \text{ N/mm}^2$$

$$A_{se} = 0 \text{ (no reinforcement)}$$

$$f_y = 275 \text{ N/mm}$$

$$V_l = \min \rightarrow F_{cf} = \left\{ \begin{array}{l} \frac{28,5 \times 275}{1,1} = 712.5 \text{ Kn} \\ \frac{0,085 \times 1000 \times 25}{1,5} + 0 = 1416,67 \text{ Kn} \end{array} \right\} V_l = 712.5 \text{ kN}$$

IV.7.3 Design Shear Strength P_{Rd} :

According to the Eurocode we have :

$$P_{Rd} = \min \left\{ \begin{array}{l} 0,8 f_u \frac{\pi \times d^2}{4} \times \frac{1}{\gamma_v} \\ 0,29 \alpha d^2 \times \sqrt{F_{ck} \times E_{cm}} \times \frac{1}{\gamma_v} \end{array} \right\}$$
$$P_{Rd} = \min \left\{ \begin{array}{l} 0,8 \times 430 \times \frac{\pi \times 18^2}{4} \times \frac{1}{1,25} = 70.03 \text{ kN} \\ 0,29 \times 1 \times 18^2 \times \sqrt{25 \times 30,5} \times \frac{1}{1,25} = 65,63 \text{ kN} \end{array} \right\}$$

$$P_{Rd} = 65.63 \text{ kN}$$

Therefore:

$$N \geq 10.85 \frac{V_L}{P_{Rd}} = \frac{712.5}{65.63}$$

$$N = 11 \text{ connectors}$$

We have n=11 connectors spread over the half length of the beam, which is equivalent to 22 connectors over the total length of the beam.

IV.8 Stud Spacing:

We have:

$$S = \frac{L_{cr}}{n}$$

$$\text{With } L_{cr} = 0.5 \times W = 0.5 \times 6 = 3 \text{ m}$$

S: The spacing between 2 successive studs

L_{cr} : The length between successive critical cross-sections

n Number of studs

$$S = \frac{L_{cr}}{n} = \frac{300}{11} = 27.27 \text{ cm}$$

$$S = 27.27 \text{ cm}$$

✓ Thus, for every 27.27 cm, one stud will be present throughout the mixed section.

IV.9 Deduction:

➤ Through this study, we demonstrated the design of a composite floor composed of a 10 cm thick concrete slab resting on IPE 200.

➤ The connection between the slab and the joists is made using 22 connectors distributed every 27.27 cm

CHAPTER V

STUDY OF STAIRS

V.1 Introduction:

The staircase is an architectural structure composed of a regular series of steps that facilitates the transition between different levels of a building composed of landing, flight and stringer



FIGURE V 1 The elements of a staircase.

V.2 Dimensioning of the stairs :

Floor height: $H = 3.91 \text{ m}$

Step = 1.3 m

Step height: $H = 16.5 \text{ cm} \leq H 18.5 \text{ cm}$

The width of the step: $g = 27 \text{ cm} \leq g 30 \text{ cm}$

(BLONDEL formula) $2h + g = 60 \text{ cm} \leq 64 \text{ cm}$

We accept a step height $h = 17 \text{ cm}$

$n = \frac{H}{h} = \frac{391}{17} = 23$ counter steps Number of steps is **22 steps**

Width of the step: $g = 28 \text{ cm}$

Verification according to BLONDEL :

We can check that $60 \text{ cm} \leq g + 2.h \leq 66 \text{ cm}$

$\rightarrow 60 < 28 + 2 \cdot 17 = 62 \text{ cm} < 66 \quad \times \Rightarrow$ the condition is checked.

:

- The figure below summarizes the different dimensions of the stairs:

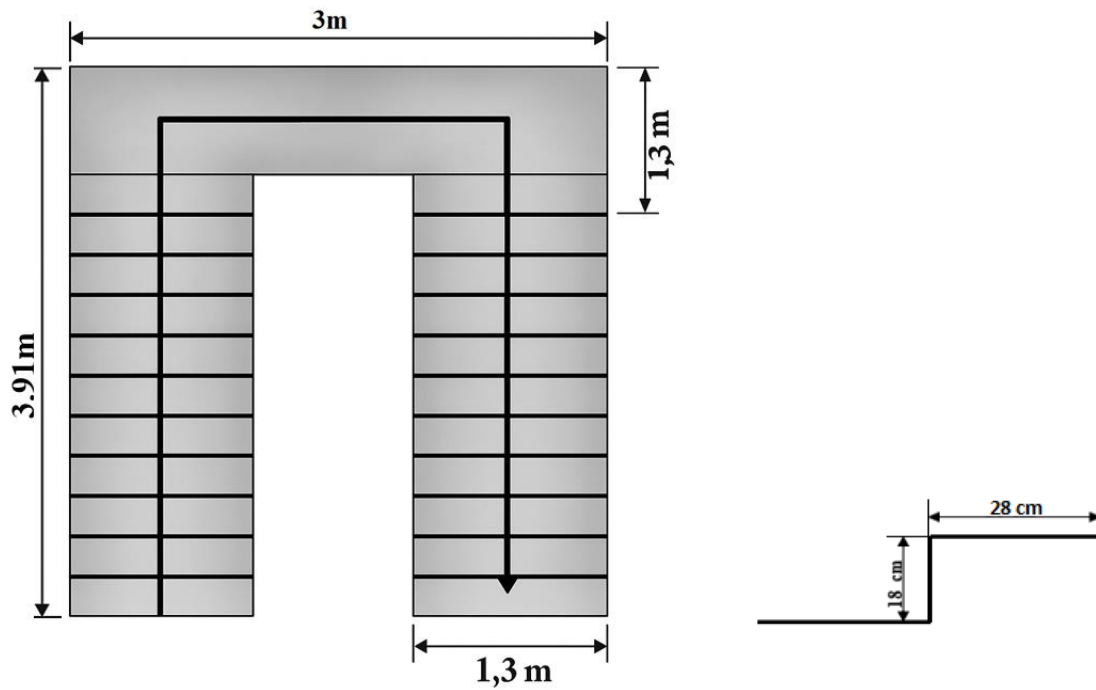


FIGURE V 2 Dimensions of the staircase

V.3 Design of step supports:

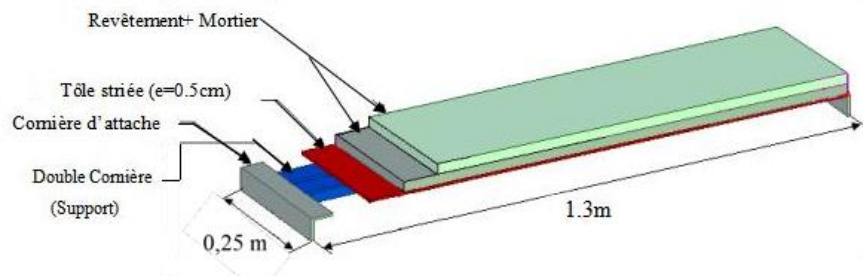


FIGURE V 3 Constructive elements of the step

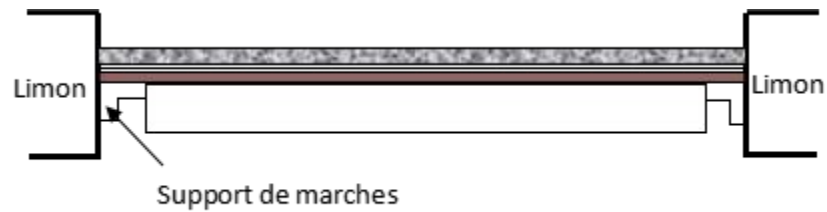


FIGURE V 4 positions of step support

Step length: $L = 1.3\text{m}$

Width of the step: $l = 0.28\text{ m}$

V.3.1 Stress determinations:

- Ribbed sheet thickness = 5 mm $G1 = 0.4\text{KN/m}^2$

- Laying mortar + G2 coatings = 0.6KN/m^2

- Permanent load:

$$G = (G1 + G2) \times l = (0.4 + 0.6) \times 0.25 = 0.25\text{ KN/m}$$

- Operating overload:

$$q = 2.50\text{ KN/m}^2$$

$$Q = q \times l = 2.50 \times 0.25 = 0.625\text{ KN/m}$$

Combination at the SLS:

$$q_{ser} = G + Q = 0.25 + 0.625 = 0.875\text{ KN/m}$$

Combination at the ULS:

$$q_u = 1.35G + 1.5Q = 1.35 \times 0.25 + 1.5 \times 0.625 = 1.275\text{ KN/m} \rightarrow 1.275\text{KN/m}$$

V.3.2 Pre-dimensioning of support angles:

The supports of the steps are sized under the condition of the following deflection

$$f_{adm} = \frac{L}{250} = \frac{130}{250} = 5,2\text{ mm}$$

$$f_y = \frac{5q_s l^4}{384El_y} \leq f_{adm} = \frac{L}{250}$$

$$I_y \geq \frac{5 \times 0.875 \times 250 \times 1.3^3}{384 \times 2.1 \times 10^5}$$

$$I_y \geq 29798.719\text{ mm}^4$$

The 40×40×4 angle is adopted as the step support with inertia equal to $I_y = 4.47 \times 10^4 \text{ mm}^4$

V.3.3 Deflection Verification at SLS:

Permanent load :

$$G = (G1 + G2) \times l + G$$

$$L40 \times 40 \times 4 = 0.4 + 0.6 \times 0.25 + 0.242 = 0.492 \text{ KN}$$

Combination at the SLS :

$$G + Q = 0.492 + 0.625 = 1.117 \text{ KN/m}$$

Suit at the ULS:

$$1.35G + 1.5Q = 1.35 \times 0.492 + 1.5 \times 0.625 = 1.6017 \text{ KN/m}$$

$$f_y = \frac{5qsl4384El_y FADM = L250 f_y = \frac{5qsl^4}{384El_y} \leq f_{adm} = \frac{L}{250}$$

$$f_y = \frac{5 \times 1.117 \times 1.3^4}{384 \times 2.1 \times 10^5 \times 4.47 \times 10^4} \leq f_{adm} = \frac{L}{250}$$

$$f_y = 4.42 \text{ mm} \leq f_{adm} = 5.2 \text{ mm}$$

The arrow is checked

V.3.4 Verification of ULS Resistance:

For this verification we use the following condition:

$$M_{sd} \leq M_{pl. Rd}$$

Class 3 Angle Cross-Section

It is necessary to check that $M_{sd} \leq M_{el.y. Rd}$

$$M_{sd} = \frac{q \times L^2}{8} = \frac{1.275 \times 1.3^2}{8} = 0.269 \text{ KN.m}$$

$$M_{el.y. Rd} = \frac{W_{ely} \times f_y}{\gamma_{Mo}} = \frac{1.55 \times 10^3 \times 275}{1.1} = 0.387 \text{ KN.m}$$

$$\text{So: } M_{sd} \leq M_{pl. Rd}$$

The resistance condition is verified.

V.3.5 Shear force design:

For this verification the following condition is used :

$$V_{sd} < V_{pl, rd}$$

$$V_{sd} = \frac{q_u \times L}{2} = \frac{1.275 \times 1.3}{2} = 0.828 \text{ KN}$$

$$V_{pl,rd} = A_v \times \frac{f_y}{\gamma_{M0} \times \sqrt{3}} = 12.24 \times \frac{0.275}{1.1 \times \sqrt{3}} = 1.76 \text{ KN}$$

Thus: $V_{sd} = 1.76 \text{ KN} < V_{pl,rd} = 44.45 \text{ KN}$

The condition is checked.

V.4 Silt Sizing:

V.4.1 Determination of the Silt Section:

It is necessary to calculate the minimum height that meets the slope condition and the step width to have the minimum UPN profile to be adopted

In the ABC triangle:

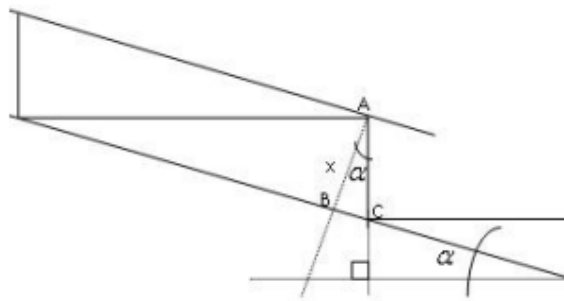


FIGURE V 5 Static pattern of a staircase

$$\text{What} = \frac{x}{AC} = \frac{x}{h}$$

$$\text{TGA} = \frac{18}{28} = \frac{x}{h} = 32.70^\circ$$

With:

$$x = h \times \cos \alpha = 18 \times \cos (32.70) = 151.47 \text{ mm}$$

So, we adopt at least a **UPN160** of $h=160 \text{ mm}$

V.4.2 Load assessment:

- Flock
- Dead loads
- Poids de 2 lemons (UPN160) $G1 = 2 \times 0.189 = 0.378 \text{ kN/m}$.
- Angle weight (40×40×4) $G2 = 0.0242 \text{ kN/m}$
- Sheet metal formwork with a thickness (2mm) $G3 = 0.15 \text{ kN/m}^2$
- Laying mortar (2cm) $G4 = 0.4 \text{ kN/m}^2$
- Tile coverings $G5 = 0.6 \text{ kN/m}^2$
- Railing $G6 = 0.15 \text{ kN/m}$

Total load:

$$G_v = (G_3 + G_4 + G_5 + G_6) \times L + G_1 + G_2 = (0.15 + 0.4 + 0.6 + 0.15) \times 1.3 + 0.378 + 0.0242$$

$$G_v = 2.09 \text{ kN/m}$$

Total load for one stringer:

$$G_{v1} = \frac{G_v}{2} = \frac{2.09}{2} = 1.045 \text{ KN/m}$$

Operating Expense:

$$P = 2.5 \text{ kN/m}^2$$

$$Q_v = P \cdot L = 2.5 \times 1.3 = 3.25 \text{ kN/m}$$

$$Q_v = 3.25 \text{ kN/m}$$

Total load for one stringer:

$$Q_{v1} = \frac{Q_v}{2} = \frac{3.25}{2} = 1.62 \text{ kN/m}$$

$$Q_{v1} = 1.62 \text{ kN/m}$$

-Bearing:**Dead loads:**

- Concrete slab: (ep=8cm) $G_1 = 2 \text{ kN/m}^2$

- Weight of TN40: $G_2 = 0.12 \text{ kN/m}^2$

- Laying mortar (2cm): $G_3 = 0.4 \text{ kN/m}^2$

- Tile coverings: $G_4 = 0.6 \text{ kN/m}^2$

$$G_p = (G_1 + G_2 + G_3 + G_4) \times L = (2 + 0.12 + 0.4 + 0.6) \times 1.3$$

$$G_p = 4.056 \text{ KN/m}$$

Operating Expense:

$$P = 2.5 \text{ kN/m}^2$$

$$Q_p = P \cdot L = 2.5 \times 1.3 = 3.25 \text{ kN/m}$$

$$Q_p = 3.25 \text{ KN/m}$$

V.4.3 Combination of Loads:**- UIS:**

- Stringer: $S = 1.35 \cdot G_{v1} + 1.5 \cdot Q_{v1} = 1.35 \times 1.045 + 1.5 \times 1.62 = 3.84 \text{ kN/m}$

- Landing: $S = 1.35 \cdot G_p + 1.5 \cdot Q_p = 1.35 \times 4.056 + 1.5 \times 3.25 = 10.35 \text{ kN/m}$

- SLS:

- Stringer: $GV + QV = 1.045 + 1.62 = 2.665 \text{ kN/m}$

- Landing: $Gp + Qp = 4.056 + 3.25 = 7.306 \text{ kN/m}$

V_{\max} , M_{\max} and N_{\max} are calculated using RDM6 software:

$$V_{\max} = 10.6 \text{ KN}$$

$$M_{\max} = 6.61 \text{ KN.m}$$

$$N_{\max} = 5.75 \text{ KN}$$

V.4.4 Deflection Verification at SLS:

The condition to be checked is the following:

$$f_{\max} < f_{\text{adm}}$$

With:

$$f_{\text{adm}} = L/250 = 391/250 = 1.564 \text{ cm}$$

$$f_{\max} = \frac{5q_s L^4}{384EI_y} = \frac{5 \times 730.6 \times 130^4}{384 \times 2.1 \times 10^6 \times 925} = 1.39 \text{ cm}$$

$$f_{\max} < f_{\text{adm}} \quad \rightarrow \text{then the arrow is checked}$$

V.4.5 Strength Design (ULS):

For this verification the following condition is used:

$$M_{\text{sd}} \leq M_{\text{pl, Rd}}$$

Class 3 Angle Cross-Section

With:

$$M_{\text{ysd}} = 6.61 \text{ KN.m}$$

$$M_{\text{el, Rd}} = \frac{W_{\text{el}} \times f_y}{\gamma_{M0}} = \frac{116 \times 0.275}{1.1} = 29 \text{ KN.m}$$

$$M_{\text{sd}} = 6.61 \text{ KN.m} \leq M_{\text{pl, Rd}} = 29 \text{ KN.m}$$

\rightarrow Then the condition of the resistance is verified.

V.4.6 Trench force verification:

For this verification the following condition is used:

$$V_{\text{sd}} \leq V_{\text{pl, Rd}}$$

$$V_{\text{ysd}} = 10.6 \text{ KN.m}$$

$$V_{\text{pl, Rd}} = A_v \frac{f_y}{\gamma_{M0}}$$

$$A_v = A - 2b t_f + (t_{w+r}) t_f = 24 - 2 \times 6.5 \times 1.05 + (0.75 + 1.05) \times 1.05 = 12.24 \text{ cm}^2$$

$$V_{pl, Rd} = \frac{f_y}{\gamma_{M0}} \frac{0.275}{1.1}$$

$$V_{pl, Rd} = 306 \text{ KN}$$

$$V_{sd} \leq V_{pl, Rd}$$

→ Then the condition is verified.

V.4.7 Verification a la compression:

For this verification the following condition is used:

$$N_{sd} \leq N_{rd}$$

$$N_{rd} = A_v \frac{f_y}{\gamma_{M0}} = \times \frac{27.5}{1.1} = 29 \text{ KN}$$

$$N_{sd} = 5.75 \text{ KN}$$

$$N_{sd} \leq N_{rd} \rightarrow \text{Then the condition is true.}$$

V.5 Study of the landing beam:

Distributed Loads:

Permanent load:

- Concrete slab (ep=8cm): $G1 = 2 \text{ kN/m}^2$
- Weight of TN40: $G2 = 0.12 \text{ kN/m}^2$
- Laying mortar (2cm) : $G3 = 0.4 \text{ kN/m}^2$
- tile coverings : $G4 = 0.6 \text{ kN/m}^2$
- Bulkhead weight : $G5 = 1.75 \text{ kN/m}^2$

$$G = (G1 + G2 + G3 + G4) \times 0,65 + G5 \times 2 = (2 + 0,12 + 0,4 + 0,6) \times 0,65 + 1,75 \times 2$$

$$G = 5.52 \text{ kN/m}$$

Operating Expense:

$$P = 2.5 \text{ kN/m}^2$$

$$Q_p = P \times 0.65 = 2.5 \times 0.65 = 1.62 \text{ kN/m}$$

Load Combination:

ULS:

$$1.35G + 1.5Q = 1.35 \times 5.52 + 1.5 \times 1.62 = 9.49 \text{ kN/m}$$

SLS:

$$G + Q = 5.52 + 1.62 = 7.12 \text{ kN/m}$$

The following results are given by the calculation code 'Robot Structural Analysis' 2024:

V.5.1 Deflection check at the SLS:

The condition to be checked is the following:

$$f_{\max} < f_{\text{adm}}$$

With:

$$f_{\text{adm}} = L/250 = 300/250 = 1.2 \text{ cm}$$

$$f_{\max} = \frac{5q_s L^4}{384EI_y} = \frac{5 \times 730,6 \times 130^4}{384 \times 2.1 \times 10^6 \times 925} = 1.39 \text{ cm}$$

$$f_{\max} < f_{\text{adm}} \rightarrow \text{then the arrow is checked.}$$

V.5.2 Strength Design (ULS):

For this verification the following condition is used:

$$M_{sd} \leq M_{pl, Rd}$$

Cross-sectional class :

Class 3 Angle Protector

With:

$$M_{ysd} = 6.61 \text{ KN.m}$$

$$M_{el, Rd} = \frac{W_{pl} \times f_y}{\gamma_{M0}} = \frac{138 \times 0.275}{1.1} = 34.5 \text{ KN.m}$$

$$M_{sd} = 6.61 \text{ KN.m} \leq M_{pl, Rd} = 34.5 \text{ KN.m}$$

→ Then the condition of the resistance is verified.

V.5.3 Verification Force Shear:

For this verification, the following condition is used:

$$V_{sd} \leq V_{pl, Rd}$$

$$V_{ysd} = 10.6 \text{ KN.m}$$

$$V_{pl, Rd} = A_v \frac{f_y}{\gamma_{M0}}$$

$$A_v = A - 2b t_f + (t_{w+r}) t_f = 24 - 2 \times 6.5 \times 1.05 + (0.75 + 1.05) \times 1.05 = 12.24 \text{ cm}^2$$

$$V_{pl, Rd} = \frac{f_y}{\gamma_{M0}} \frac{0.275}{1.1}$$

$$V_{pl, Rd} = 306 \text{ KN}$$

$$V_{sd} \leq V_{pl, Rd}$$

→ Then the condition is verified.

V.6 Conclusion :

For the study of the stairs, the choice of dimensions that was planned was justified according to the dimensions of the stairwell:

- Support angle L 40×40×4
- UPN 160 Lemon
- IPE 220 landing beam

CHAPTER VI

SEISMIC STUDY

VI.1 Introduction :

In order to avoid any risk of ruin that may occur in the event of an earthquake, we have considered that the seismic study is important in the calculation of the structures.

$$V = \lambda \times \frac{S_{ad}}{g}(T_0) \times W$$

As :

λ : Correction coefficient

$$\lambda = \begin{cases} 0.85 & \text{si } T_0 \leq (2 \times T_2) \text{ et si le bâtiment a plus de 2 niveaux} \\ 1 & \text{autrement} \end{cases}$$

$\frac{S_{ad}}{g}(T_0)$: calculation spectrum for the period T_0 .

W : weight total of the structure

TABULAR VI 1 List of mass contribution

Cas/Mode	Période [sec]	Masse Modale UX[%]	Masse Modale UY[%]	Masses Cumulées UX[%]	Masses Cumulées UY[%]
6/ 1	0,56	3,29	0	3,29	0
6/ 2	0,56	3,29	0	6,59	0
6/ 3	0,54	8,82	0	15,4	0
6/ 4	0,34	72,18	0	87,58	0
6/ 5	0,16	7,17	0	94,75	0
6/ 6	0,14	0	2,52	94,75	2,52
6/ 7	0,13	0	49,9	94,75	52,42
6/ 8	0,13	0	0,78	94,75	53,19
6/ 9	0,12	0	15,09	94,75	68,28
6/ 10	0,1	0,01	4,58	94,77	72,86
6/ 11	0,09	0,09	0,35	94,85	73,21
6/ 12	0,09	0	0,1	94,85	73,31
6/ 13	0,09	0,62	3,74	95,47	77,05
6/ 14	0,09	0,04	0,13	95,51	77,18
6/ 15	0,09	0,13	15,24	95,64	92,41

Calculation of the fundamental period of the structure :

$$T_{\text{empirique}} = C_t \times (h_N)^{\frac{3}{4}}$$

With:

$\begin{cases} h_N & \text{hauteur de la structure} \\ C_t & \text{Coefficient, fonction du systeme de contreventement, du type de remplissage,} \\ & \text{donne sur le tableau "4.3 RPA 2024 " } \end{cases}$

TABULAR VI 2 Calculation of the fundamental period of the structure

	Ct	Ht	Value	1.3×T0
T0	0.085	11.8	0.54	0.7

Coefficient de correction:

Site S3: $T_1 = 0.1 \text{ s}$; $T_2 = 0.4 \text{ s}$; $T_3 = 1.2 \text{ s}$; $S = 1.55$

TABULAR VI 3 Calculation of correction coefficient

	T0	type	T2	2T× ₂	L
L	0.54	Type2	0.4	0.8	0.85

VI.4 Design Spectrum:

$$\frac{S_{ad}}{g}(T_0) = \begin{cases} A \times I \times S \times \left[\frac{2}{3} + \frac{T_0}{T_1} \times \left(2.5 \times \frac{Q_F}{R} - \frac{2}{3} \right) \right] & \text{SI } 0 \leq T_0 \leq T_1 \\ A \times I \times S \times \left[2.5 \times \frac{Q_F}{R} \right] & \text{SI } T_1 \leq T_0 \leq T_2 \\ A \times I \times S \times \left[2.5 \times \frac{Q_F}{R} \right] \times \left[\frac{T_2}{T_0} \right] & \text{SI } T_2 \leq T_0 \leq T_3 \\ A \times I \times S \times \left[2.5 \times \frac{Q_F}{R} \right] \times \left[\frac{T_2 * T_3}{T_0^2} \right] & \text{SI } T_3 \leq T_0 \leq 4 \text{ s} \end{cases}$$

(3.15) de South Africa 2024

on a $T_2 < T_0 < T_3$

Coefficient of behaviour R :

In our case we have a gantry frame with bar braces centered in X so **R = 4.5^(b)**

Determination of the QF quality factor :

$$Q_F = 1 * \sum_{1}^n P_q$$

TABULAR VI 4 Determination of the QF quality factor

						SANS "X"	SANS "Y"
Catégorie	Critère, q					q	q
a	Régularité en plan					<input type="checkbox"/> FAUX	<input type="checkbox"/> FAUX
	Régularité en élévation					<input type="checkbox"/> FAUX	<input type="checkbox"/> FAUX
	Conditions minimales sur le nombre étage					<input type="checkbox"/> FAUX	<input type="checkbox"/> FAUX
	Conditions minimales sur les travées					<input type="checkbox"/> FAUX	<input type="checkbox"/> FAUX
b	Régularité en plan					<input type="checkbox"/> FAUX	<input type="checkbox"/> FAUX
	Régularité en élévation					<input type="checkbox"/> FAUX	<input type="checkbox"/> FAUX
	Redondance en plan					<input type="checkbox"/> FAUX	<input type="checkbox"/> FAUX

Note for past table :

- TRUE: Observed

- FAUX :Non observe

$$Q_F = 1 + (0 + 0 + 0)$$

The most important group is group 2 because the hangar houses important equipment for production or installations requiring continuity of service.

Based on the table (3.11 EPS 2024)

$$I = 1$$

Acceleration coefficient A :

Based on the table (3.3 RPP 2024)

$$A = 0.15$$

Calculating the force:

TABULAR VI 5 Calculation of force

	λ	Spectre	In (kN)	Strength (kN)
X	0.085	0.096	3178,89	259,4
And	0.085	0.096	3178,89	259,4

RPA audits :

$$0.8 \times V_{\text{calcul}} \leq V_{\text{dynamique}}$$

$$0.8 \times 259,4 < V_x$$

$$207.5 \text{ kN} < 303.4 \text{ kN Verified}$$

$$0.8 \times 259.4 < V_y$$

$$207.5 \text{ kN} < 254.5 \text{ kN Verified}$$

Displacement Verification :

Le déplacement horizontal :

$$\delta_k = \frac{R}{Q_F} \times \delta_{ek}$$

Relative displacement :

$$\Delta_k = \delta_k - \delta_{k-1}$$

TABULAR VI 6 Verification of displacement

Sense	Longitudinal X		Transversal Y	
Level	δ_k	Δ_k	δ_k	Δ_k
1	0.29	0.29	0.02	0.02
2	0.53	0.24	0.04	0.02

From the table above, we have seen that the relative displacements do not exceed 1% of the floor height.

Verification of the P- Δ effect:

All this is taken directly from the software "ROBOT" :

The P- Δ effect:

$$\theta_k = \frac{P_k \times \Delta_k}{V_k \times h_k} \leq 0.10$$

Sense -X-:

TABULAR VI 7 Verification of the P- Δ effect (direction X)

Floor	W (Kn)	P_k m	V_k m	h_k m	Δ_k m	θ_k	Observation
1	2215,643	3640,64	303,43	4,08	0.0029	0,008	Verified

Sense -Y-:

TABULAR VI 8 Verification of the P- Δ effect (Y-direction)

Floor	W (Kn)	P_k m	V_k m	h_k m	Δ_k m	θ_k	Observation
1	2215,643	3640,64	254,63	4,08	0,0002	0,0007	Verified
2	321,87	934,28	133,41	4,07	0.0004	0,0006	Verified

CHAPTER VII

CHECKING ELEMENTS

VII.1 Verification of the elements:

According to the analyses on the ROBOT-STRUCTURAL-ANALYSES2024 software which is verifying the conditions of eurocode03 we have the following results:

TABULAR VII 1 Verification of elements via ROBOT 2024

Element	Section	Déversement	Flambement	Flèche	Observation
Column	HEA300	Vérifié	Vérifié	Vérifié	Vérifié
Traverse	IPE 360	Vérifié	Vérifié	Vérifié	Vérifié
Post	HEA 320	Vérifié	Vérifié	Vérifié	Vérifié
Purlin	IPE 160	Vérifié	Vérifié	Vérifié	Vérifié
Cladding rail	UPN 180	Vérifié	Vérifié	Vérifié	Vérifié
CV Stability	2 UPN 160	Vérifié	Vérifié	Vérifié	Vérifié
Sand beam	HEA 140	Vérifié	Vérifié	Vérifié	Vérifié

Forms used for verification:

Spill Verification :

$$M_{sd} \leq M_{b,Rd} = X_{LT} \beta_w W_{ply} \frac{f_y}{\gamma_{M1}}$$

$$\chi_{LT} = \frac{1}{\phi_{LT} + \sqrt{\phi_{LT}^2 - \lambda_{LT}^2}} \leq 1,0$$

$$\phi_{LT} = 0,5 \left[1 + \alpha_{LT} (\bar{\lambda}_{LT} - 0,2) + \bar{\lambda}_{LT}^2 \right]$$

Buckling Verification:

$$\frac{N_{sd}}{x_{min} \cdot A \cdot f_y / y_{M1}} + \frac{k_y \cdot M_{y, sd}}{W_{pl, y} \cdot f_y / y_{M1}} + \frac{k_z \cdot M_{z, sd}}{W_{pl, z} \cdot f_y / y_{M1}} \leq 1 \dots\dots\dots(5.51)$$

Where:

$k_y = 1 - (\mu_y \cdot N_{sd} / X_y \cdot A \cdot f_y)$	$k_y \leq 1.50$
$\mu_y = \lambda_y (2\beta_{M, y} - 4) + ((W_{pl, y} - W_{el, y}) / (W_{el, y}))$	$\mu_y \leq 0.90$
$k_z = 1 - (\mu_z \cdot N_{sd} / X_z \cdot A \cdot f_y)$	$k_z \leq 1.50$
$\mu_z = \lambda_z (2\beta_{M, z} - 4) + ((W_{pl, z} - W_{el, z}) / (W_{el, z}))$	$\mu_z \leq 0.90$

CHAPTER VIII

CALCULATION

OF

ASSEMBLIES

VIII.1 Introduction:

This chapter studies the assemblies of a metal shed and the analysis of their design, their mechanical behavior and their impact on structural stability, with the aim of optimizing their performance according to standards.

VIII.2 Role of Assemblies:

The assemblies ensure the transmission of forces between structural elements, guaranteeing rigidity, stability and overall strength of the shed.

VIII.3 The main methods of assembly:

- Bolting .
- Welding.
- Riveting.

VIII.4 Types of End Plate Bolted Connections:

- Beam-column connections.
- Beam-beam.
- foot of the post.

VIII.5 Design of Assemblies:

Following the verification of the selected profiles with the software

ROBOT-STRUCTURAL-ANALYSES2024 we find the following results:

TABULAR VIII 1 Calculation of assemblies of elements

Elements	Bolts or anchor rod				Distance longitudinal between bolts (mm)	Distance transversal between bolts (mm)
	Diameter (mm)	Class	N ^{br} of lines	N ^{br} of columns		
Assembly of foot of post	33	4.6	4	2	430	150
Assembly column-traverse	22	HR 10.9	6	2	100	100
Assembly of column-traverse (roof)	20	HR 10.9	7	2	80	100
Assembly beam - beam (web)	16	8.8	2	2	65	60
Assembly Traverse-Traverse (ridge)	20	HR 10.9	7	2	90	100

Checking formulas :

Bolt slippage :

$$f_s = k_s \times \mu \times m \times f_p$$

shear bolt verification :

$$V/n \leq f_s$$

diametrical pressure :

$$\frac{U}{n_b} \leq k_t \times \alpha_b \times d \times t \times \frac{f_e}{\gamma_{m2}}$$

CHAPTER XI

INFRASTRUCTURE STUDY

XI.1 Introduction:

The foundation is the section of a building or public infrastructure that ensures the transfer of loads to the ground.

XI.2 Choice of foundations:

The choice of the type of foundation is made according to 3 parameters

Nature and weight of the superstructure, the applied loads, soil quality.

XI.3 Sizing of the sole:

The dimensions of the seat plate from the ROBOT 2024 software :

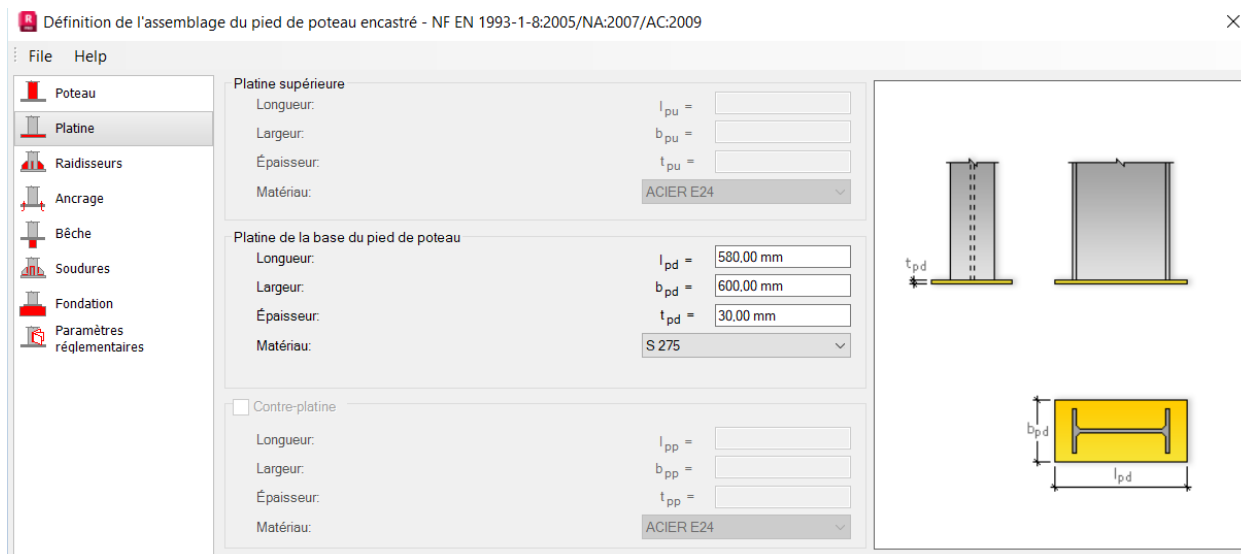


FIGURE XI 1 Dimensions of the foot post plate

Therefore, we have:

$$\begin{cases} a = 580 \text{ mm} \\ b = 600 \text{ mm} \end{cases}$$

Based on the geotechnical report carried out:

$$\overline{\sigma}_{sol} = 1.5 \text{ bars}$$

We will size for insulated insoles subjected to normal stress.

Considering the maximum loads under the most unfavourable combination.

$$N_{sd} = 32056.5 \text{ daN}$$

$$\frac{A}{B} = \frac{a}{b} \rightarrow A = 0.96B$$

It must be verified that $\sigma_{cal} < \overline{\sigma_{sol}}$

With:

$$\overline{\sigma_{sol}} = \frac{N_s}{A \times B}$$

$$A \times B \leq \frac{N_s}{\overline{\sigma_{sol}}}$$

$$0.96 \times B \geq \sqrt{\frac{N_s}{\overline{\sigma_{sol}}}}$$

$$B \geq \sqrt{\frac{32056.5}{15000}} \times \frac{1}{0.96}$$

$$B \geq 1.52 \text{ m}$$

We take $\begin{cases} A = 1.6 \text{ m} \\ B = 1.6 \text{ m} \end{cases}$

XI.3.1 Sole Height Calculation:

$$d \geq \frac{A - a}{4} = \frac{160 - 58}{4}$$

$$d \geq 25.5 \text{ cm}$$

We take $d = 30 \text{ cm}$

Then $h = d + 5 = 40 + 5 = 45 \text{ cm}$

XI.3.2 Calculation of the reinforcement of the foundation:

By the connecting rod method:

Calculation of A_u :

ULS $1.35G + 1.5Q$:

$$N_{sd} = 31515.7 \text{ daN}$$

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

$$\sigma_{st} = \frac{f_e}{\gamma_s} = \frac{400}{1.15} = 348 \text{ MPa}$$

$$A_u = \frac{31515,7 \times 1,02}{8 \times 0,4 \times 348 \times 10^5}$$

$$A_u = 2.88 \text{ cm}^4$$

AT SLS G+Q:

$$N_{sd} = 22567 \text{ daN}$$

$$A_s = \frac{N_s \times (A - a)}{8 \times d \times \overline{\sigma_{sol}}}$$

$$\overline{\sigma_{sol}} = \min \left(\frac{2}{3} f_e ; 110 \sqrt{\eta \times f_{t28}} \right) = 201.63 \text{ MPa}$$

$$A_s = \frac{22567 \times 1,02}{8 \times 0,4 \times 201,63 \times 10^5}$$

$$A_s = 3.57 \text{ cm}^4$$

Calculation of A_b :

TO THE ULS 1.35G+1.5Q:

$$N_{sd} = 31515.7 \text{ daN}$$

$$A_u = \frac{N_u \times (B - b)}{8 \times d \times \sigma_{st}}$$

$$\sigma_{st} = \frac{f_e}{\gamma_s} = \frac{400}{1,15} = 348 \text{ MPa}$$

$$A_u = \frac{31515,7 \times 1}{8 \times 0,4 \times 348 \times 10^5}$$

$$A_u = 2.83 \text{ cm}^4$$

AT SLS G+Q:

$$N_{sd} = 22567 \text{ daN}$$

$$A_s = \frac{N_s \times (B - b)}{8 \times d \times \overline{\sigma_{sol}}}$$

$$\overline{\sigma_{sol}} = \min \left(\frac{2}{3} f_e ; 110 \sqrt{\eta \times f_{t28}} \right) = 201.63 \text{ MPa}$$

$$A_s = \frac{22567 \times 1}{8 \times 0,4 \times 201,63 \times 10^5}$$

$$A_s = 3.49 \text{ cm}^4$$

We take 8T12 with $A_{st} = 9.05 \text{ cm}^2$

XI.3.3 Verification of non-brittleness condition:

$$A_{st} \leq 0.23 \times b \times d \times \frac{f_t}{f_e}$$

$$A_{st} = 5,8 \text{ cm}^2 < 0.23 \times 1.6 \times 0.3 \times \frac{2,1}{400} = 9,05 \text{ cm}^2$$

So, the condition is checked

XI.3.4 Spacing Calculation:

$$S_t \leq \min (15\phi_{\min} ; 40 \text{ cm}) = \min (18 ; 40 \text{ cm})$$

(A.8.1, 3/BAEL91).

We take $S_t = 15 \text{ cm}$

Reinforcement diagram:

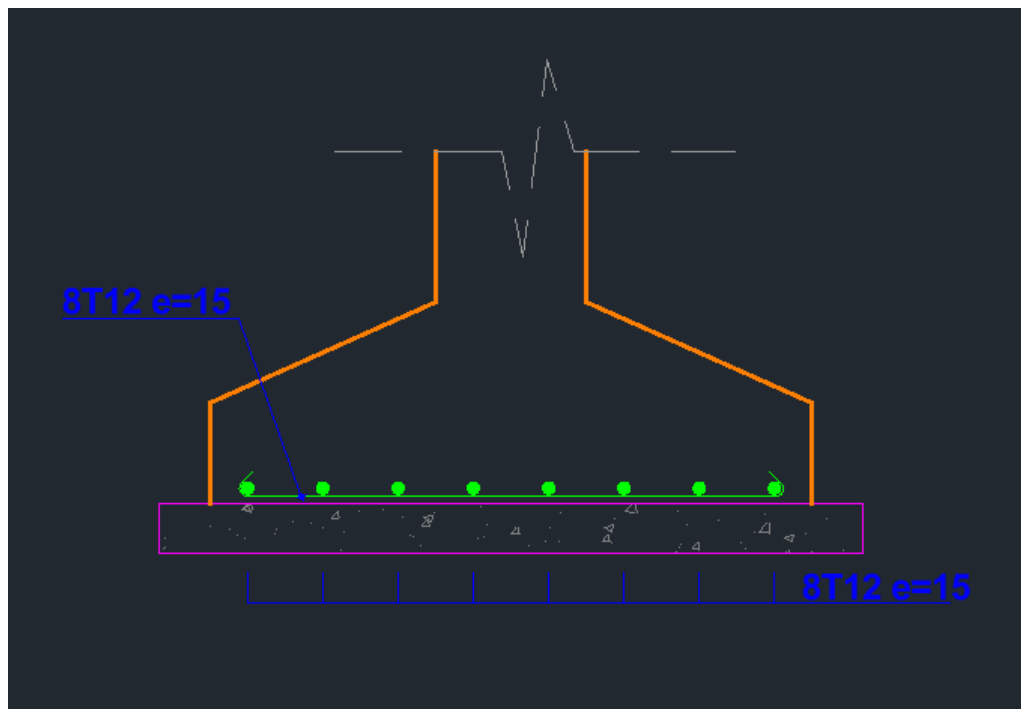


FIGURE XI 2 Sole reinforcement stake

CHAPTER X

TEMPERATURE STUDY

X.1 Introduction:

In arid climate regions such as Biskra, industrial buildings are highly exposed to extreme temperature variations, especially in summer. Metal sheds, which are often poorly insulated, can reach very high interior temperatures, which impacts thermal comfort, product conservation and energy consumption

X.2 Thermal transmittance calculation in a metal wall :

Structure of the wall considered (from the outside to the inside)

TABULAR X 1 Thermal layer composition table

Layer	Thickness 'e'	Thermal conductivity λ [W/m·K]
Ribbed steel sheet	0.001 m (1 mm)	50 W/m·K
Layer of still air (film)	-	Tabulated value (see thermal resistance)
Rock wool insulation	0.05 m	0.040 W/m·K
Air Void + Interior Cladding	-	Overlooked or counted via Rsi resistance

General Formula (ISO 6946) :

$$U = \frac{1}{R_{tot}} = \frac{1}{R_{si} + \sum \left(\frac{e_i}{\lambda_i} \right) + R_{se}}$$

With :

- $R_{si}=0.13 \text{ m}^2 \cdot \text{K/W}$ (normalized internal strength)
- $R_{se}=0.04 \text{ m}^2 \cdot \text{K/W}$ (Normalized External Resistance)
- e_i = layer thickness i
- λ_i = thermal conductivity of the i-layer

X.3 Digital Application:

$$\text{Steel: } \frac{0,001}{50} = 0,00002 \text{ m}^2 \cdot \text{K/W (neglected)}$$

$$\text{Insulating : } \frac{0,05}{0,04} = 1.25 \text{ m}^2 \cdot \text{K/W}$$

$$R_{tot} = 0.13 + 1.25 + 0.04 = 1.42 \text{ m}^2 \cdot \text{K/W}$$

$$U = \frac{1}{1,42} = 0.70 \text{ W/m}^2 \cdot \text{K}$$

The transmission coefficient U of the wall is:

$$U = 0.70 \text{ W/m}^2 \cdot \text{K}$$

Interpretation :

- A **U-value < 0.6 W/m² · K** is generally required for insulated buildings in warm regions.
- Here, with 5 cm of insulation, we are close but not yet compliant. The insulation could be **increased to 8 cm** to improve thermal comfort.

Fire and fire protection for the pole

Objective :

Test the fire resistance of an unprotected steel stud, subjected to the temperature of an ISO 834 standard fire, for 30 minutes.

Steel temperature after 30 min (without protection)

According to **EN 1993-1-2 Annex E or tabulated curves**

- For an unprotected profile, the estimated temperature of the steel can reach

$$\theta_a = 630 \text{ }^{\circ}\text{C}$$

Reduction of mechanical properties to $\theta_a = 630 \text{ }^{\circ}\text{C}$

Based on **Table 3.1 of EN 1993-1-2**

TABULAR X 2 Reduction factors for steel properties at elevated temperatures 1

Temperature θ ($^{\circ}\text{C}$)	$k_{y,\theta}$	$k_{E,\theta}$
630	0,47	0,31

Therefore :

- **Reduced resistance :**

$$f_{y,\theta} = k_{y,\theta} \cdot f_y = 0,47 \cdot 275 = 129,25 \text{ MPa}$$

- **Reduced modulus of elasticity :**

$$E_{\theta} = k_{E,\theta} \cdot E = 0,31 \cdot 210000 = 65100 \text{ MPa}$$

HEA300 Profile Property

$$\text{Section A (HEA 300): } 115.1 \text{ cm}^2 = 11\,510 \text{ mm}^2$$

Turning radius: $i = 73.6 \text{ mm}$

Calculation of reduced slenderness :

$$\bar{\lambda}_{\theta} = \frac{L}{i} \cdot \sqrt{\frac{f_y}{\pi^2 E_{\theta}}} = \frac{10000}{73,6} \cdot \sqrt{\frac{275}{\pi^2 \cdot 65100}}$$

$$\bar{\lambda}_{\theta} = 1.18$$

Determination of buckling coefficient χ_{θ} :

Buckling curve type b (HEA)

$$\text{Pour } \bar{\lambda}_{\theta} = 1.18 \Rightarrow \chi_{\theta} = 0.41$$

Fire resistance of the Pole

$$N_{b,Rd,\theta} = \frac{\chi_{\theta} \cdot A \cdot f_{y,\theta}}{\gamma_{M,\theta}} = \frac{0.41 \cdot 11510 \cdot 129.25}{1,1 \cdot 1000}$$

$$N_{b,Rd,\theta} = 554.94 \text{ kN}$$

Axial Load of Pole $N_{ed} = 120.47 \text{ kN}$ (extracted by robot2024)

Verification :

$$N_{ed} = 120.47 \text{ kN} \leq N_{b,Rd,\theta} = 554.94 \text{ kN}$$

X.3 Conclusion:

The HEA 300 pole subjected to a load of 120.47 kN is fire-resistant for 30 minutes without additional protection. It meets the requirements of **Eurocode 3 – Part 1-2** concerning the stability of structures in the event of fire.

Fire and fire protection for the sleeper:

We have :

TABULAR X 3 Reduction factors for steel properties at elevated temperature 2

Temperature θ (°C)	$k_{y,\theta}$	$k_{E,\theta}$
630	0,44	0,29

Therefore :

• **Reduced resistance :**

$$f_{y,\theta} = k_{y,\theta} \cdot f_y = 0,44 \cdot 275 = 121 \text{ MPa}$$

- **Reduced modulus of elasticity :**

$$E_{\theta} = k_{E,\theta} \cdot E = 0.29 \cdot 210000 = 60900 \text{ MPa}$$

HEA300 Profile Property

Section A (IPE360): $68.4 \text{ cm}^2 = 6840 \text{ mm}^2$

Turning radius: $i = 73.6 \text{ mm}$

Calculation of reduced slenderness:

$$\bar{\lambda}_{\theta} = \frac{L}{i} \cdot \sqrt{\frac{f_y}{\pi^2 E_{\theta}}} = \frac{12000}{74,9} \cdot \sqrt{\frac{275}{\pi^2 \cdot 60900}}$$

$$\bar{\lambda}_{\theta} = 1.49$$

Buckling curve:

Buckling curve type b :

$$\text{For: } \bar{\lambda}_{\theta} = 1.18 \Rightarrow \chi_{\theta} = 0.41$$

Fire resistance of the crossbar:

$$N_{b,Rd,\theta} = \frac{\chi_{\theta} \cdot A \cdot f_{y,\theta}}{\gamma_{M,\theta}} = \frac{0.34 \cdot 6840 \cdot 121}{1,1 \cdot 1000}$$

$$N_{b,Rd,\theta} = 281.1 \text{ kN}$$

Axial Load of Traverse $N_{ed} = 316 \text{ kN}$ (extracted by robot2024)

Verification:

$$N_{Ed} = 316 \text{ kN} > N_{b,Rd,\theta} = 281.1 \text{ kN}$$

The **IPE 360** sleeper fails without fire protection after 30 minutes of fire. Passive protection (intumescent paint, flocking, etc.) is required to achieve the required stability.

OVERALL CONCLUSION

This project aimed to study and size a conventional storage shed using the ROBOT software, specifically for seismic calculation. It provided a better understanding of the construction of metal structures, their foundations and the order of magnitude of their efforts. It also facilitated the assumption of the different design techniques, the use of specialized software and the regulations in force in the field. The consultation of various bibliographical references also enriched the work. The project aimed to improve the accuracy of the calculations, verifications and definitions of each construction element.

bibliography references:

D.T.R C 2.4.7 : Règlement Neige et Vent (RNV 2013)

D.T.R.B.C 2.44 : Règles de Conception et de Calcul des Structures en Acier (CCM97)

D.T.R.B.C 2.48 : Règles Parasismiques Algériennes (RPA 2024)

D.T.R.B.C 2.2 : Charge Permanentes et Charges d'exploitation

Calcul des Eléments de construction Métallique selon l'Eurocode 03

EUROCODE 3 : Calcul des éléments résistants d'une construction métallique


Calcul des Structures Métallique selon l'Eurocode3

Softwares used:

Auto CAD 2024

Robot structural analysis 2024


Annexes:



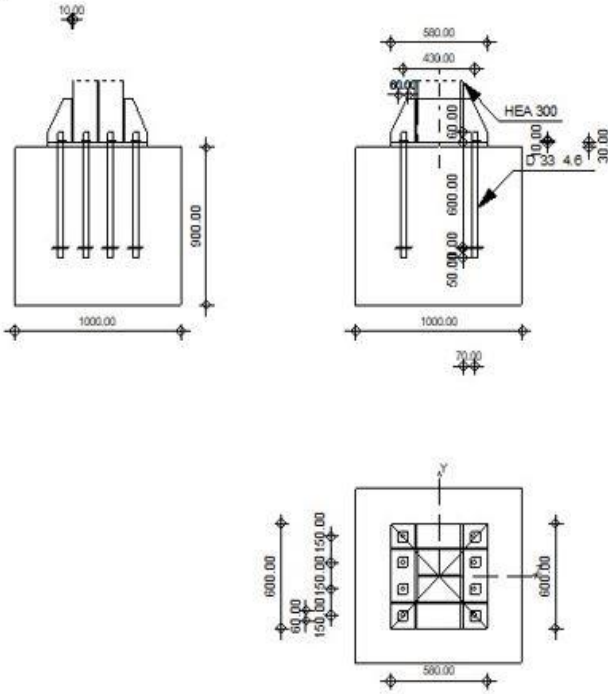
Autodesk Robot Structural Analysis Professional 2020


Calcul du Pied de Poteau encastré

Eurocode 3 : NF EN 1993-1-8 :2005/NA :2007/AC :2009



Ratio
0.77






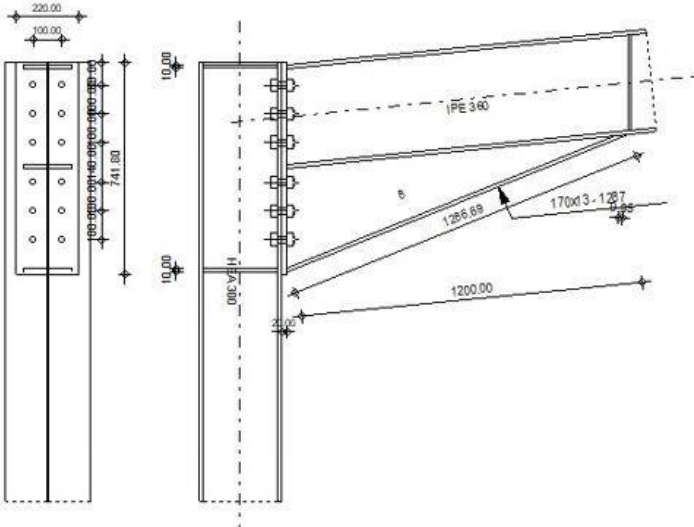
Autodesk Robot Structural Analysis Professional 2024

Calcul de l'Encastrement Traverse-Poteau

NF EN 1993-1-8:2005/NA:2007/AC:2009



Ratio
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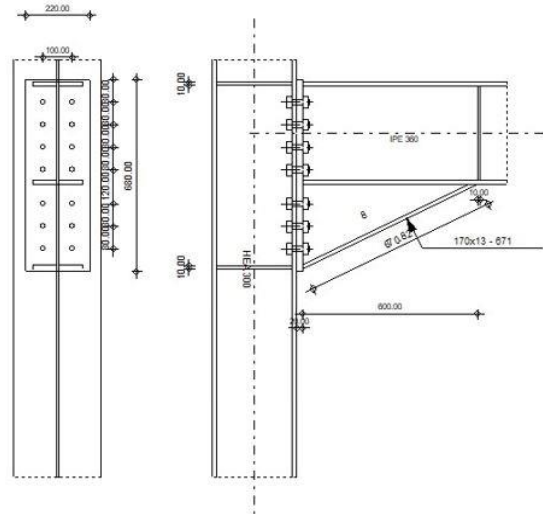
Autodesk Robot Structural Analysis Professional 2020

Calculs de l'assemblage poutre-poutre (âme)

NF EN 1993-1-8:2005/NA:2007/AC:2009

OK

Ratio
0.49



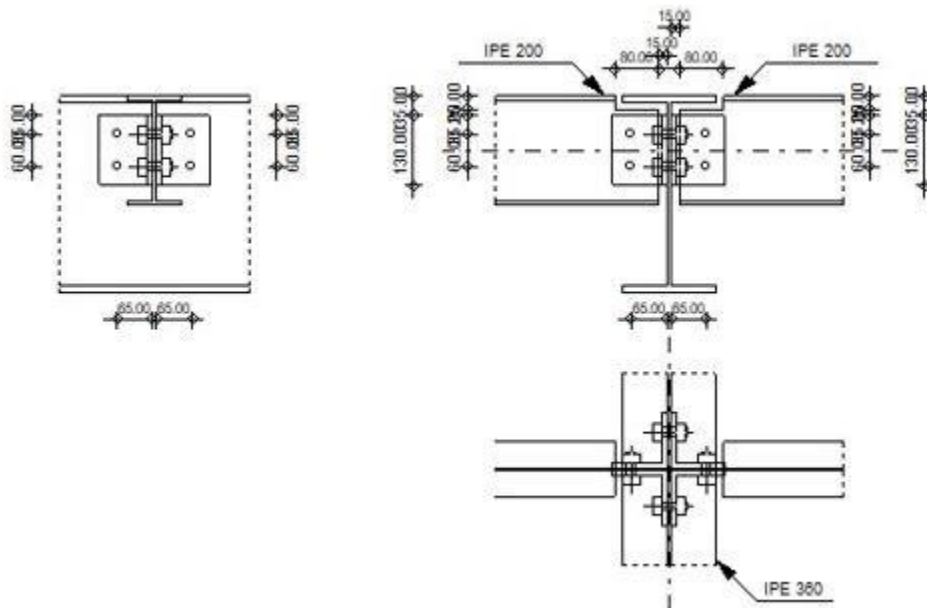
Autodesk Robot Structural Analysis Professional 2020

Calculs de l'assemblage poutre-poutre (âme)

NF EN 1993-1-8:2005/NA:2007/AC:2009

OK

Ratio
0.49





Autodesk Robot Structural Analysis Professional 2020

Calcul de l'Encastrement Poutre-Poutre

NF EN 1993-1-8:2005/NA:2007/AC:2009



Ratio
0.09

