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Department of: Civil and Hydraulic Engineering

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Sector: civil engineering

Specialty: structures

Theme

Design and study of a metal frame hangar

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Finally, we would like to thank all those who helpedand supported us and were aid for us to complete this project successful.

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Dedication

I would love to dedicate this work

To my parents, loving and protective parents one could ever have; they are my source of inspiration and perseverance. There are not enough words in the world to describe how grateful I am for you both have helped turn me into the man I am today.

To all my family,'' FEKHAR and BABOUHOUN ''

I am lucky to have you by my side

To my brothers, 'Abdou, djaber, Ahmed' you are my source of strength. To my lovely sisters, 'Suomi' how lucky I am to be surrounded by you, you have always been my strongest supporters.

To all my loyal friends out there, and to my best friendSoufiane B, Mouhamed B, Hamou T, Zouhir CH, Idriss B,

And to friends in my scout family and mechanic club.

TOUFIk .FE

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To all my family,'' KAIDAR '' I am lucky to have you by my side

To my brothers, you are my source of strength.

To my lovely sisters, how lucky I am to be surrounded by you,

you have always been my strongest supporters.

To all my loyal friends out there, and to my best friend

Habib, charef, slimane, chaker.

You are the only brothers Whose take my hand so thatI can always be strong

AHMED . k

إن الذين نحبهم ونعزهم مكانتهم ليست بين الأسطر
والصفحات لأن مقامهم أجل وأعلى فالقلب سكناهم والذكرى
ذكراهم والقلب لن ينساهم

Summary

Tables summary

Figure summary

SYMBOLS USED

G : Permanent loads.

P : Maintenance overloads.

W : Climatic wind load.

F_e : Driving force.

Solicitations:

$Q_{y.sd}$: Load applied in plane ⊥ to blade.

$Q_{z.sd}$: Load applied in the plane of the web.

$M_{y.Sd}$: Design bending moment around the yy axis caused by the $Q_{z.sd}$ load.

$M_{z.Sd}$: Design bending moment around the zz axis caused by the $Q_{y.sd}$ load.

M_{cr} : Elastic critical moment of lateral buckling.

N_{sd} : normal effort.

$V_{y.sd}$: Shearing force in the plane of the soles.

$V_{z.sd}$: Cutting effort in the plane of the soul.

$N_{t.Rd}$: Design resistance of the tensile section.

$N_{pl.Rd}$: Plastic resistance of the raw section.

$N_{u.Rd}$: Ultimate resistance of the net section in line with the fixing holes.

$N_{c.Rd}$: Compressive strength force.

$V_{pl.Rd}$: Resistance of the section to the shearing force,

$M_{c.Rd}$: Bending moment of resistance.

$M_{ely.}$: Bending moment of elastic resistance following yy.

$M_{elz.Rd}$: Bending moment of elastic resistance according to zz.

$M_{ply.}$: Bending moment of plastic resistance following yy.

$M_{plz.Rd}$: Bending moment of plastic resistance according to zz.

$M_{b.Rd}$: Bending moment of resistance to buckling.

$M_{v.Rd}$: Bending moment of reduced plastic resistance counts given the shear effort.

$M_{Ny.Rd}$: Bending moment of plastic resistance according to yy reduced under the effect ofnormal effort.

$M_{Nz.Rd}$: Bending moment of plastic resistance according to zz reduced under the effect ofnormal effort.

Material characteristics:

E : Longitudinal modulus of elasticity.

f_y : elastic limit of the material,

f_u : Material breaking limit or tensile strength specified minimum.

v : Poisson's ratio.

Geometric characteristics of the sections:

A : Area of the butt section,

A_{net} : Area of the net section to the right of the fixing holes.

I_y : Moment of inertia along the yy axis.

I_z : Moment of inertia along the zz axis.

d_0 : Diameter of the hole,

$\emptyset$: Bolt diameter.

t : Sheet thickness.

t_f : Thickness of the sole.

t_w : Thickness of the core.

b : Width of the sole.

h : Height of the section.

r : Core / flange connection radius.

i_y : Radius of gyration along the yy axis.

i_z : Radius of gyration along the zz axis.

yy : Axis parallel to the soles (strong axis).

zz : Axis perpendicular to the flanges (Weak axis).

, l_0 : Length of the element.

l_y : Buckling length along the yy axis.

l_z : Buckling length along the zz axis.

L : Lateral buckling length (for lateral torsional buckling)

W_{ely}: Modulus of elastic resistance of the section along the yy axis.

W_{elz}: Elastic modulus of resistance of the section along the zz axis.

W_{ply} : Plastic modulus of resistance of the section along the yy axis

W_{plz}: Plastic modulus of resistance of the section along the zz axis

W_{eff}: Elastic modulus of resistance of the effective section.

P : Self-weight.

Other symbols:

γ_M: Partial safety factor of the material.

γ_F: Partial safety factor for the action considered.

ψ: Combination coefficient.

Δ: Displacement.

λ: Stretching.

λ_1: Eulerian stretch.

$\bar{\lambda}$: Slenderness reduced with respect to buckling.

$\bar{\lambda}$: Slenderness reduced with respect to the lateral discharge.

χ : Reduction factor with respect to buckling.

χ_{LT}: Reduction factor with respect to the discharge.

α: Imperfection factor for buckling.

α_{LT}: Imperfection factor for the discharge.

f_y : Arrow along the yy axis.

f_z : Arrow along the zz axis.

f_{ad} : Allowable deflection.

λ_{lim}: The limit slenderness.

Post base:

β_j: Coefficient of the sealing material.

c : Additional support width for the base plates.

f_{ck} : Resistance of concrete to compression.

f_{jd} : Design resistance to crushing of the sealing material.

f_{cd} : Design resistance to concrete crushing.

$F_{v.Rd}$: Design shear resistance of the sealing of the post base plate.

Abstract

This project aims to design and study a metal structure building in the EL-MERK living base located in the municipality of EL Borma Wilaya Ouargla. It consists of several porticoes, stabilized by bracing, and covered by a roof with symmetrical hips. This project must be studied in several stages, the first of which is the evaluation of weights, including excess, as well as the effects of the climate (snow and wind), and this according to Algerian regulations, “RNV 99” version 2013, then, size the basic and secondary elements of the building after a dynamic study according to Algerian regulations. For the earthquake, “RPA 99” version 2003 then calculate the connections and assemblies accoding to the “CCM 97” regulation Finally, the infrastructure was studied according to the rules of “BAEL 91” and for the programs on which we relied “ROBOT”, the work ends with conclusions.

Keywords: Steel structure – Shed for storing – sizing – Assembly – Earthquake.

ملخص

يهدف هذا المشروع إلى تصميم ودراسة مبنى بهيكل معدني في مسكن الميرك الواقع في بلدية البرمة ولاية ورقلة. وتتكون من عدة أروقة مثبتة بدعامة ومغطاة بسقف بأرداف متناظرة. يجب دراسة هذا المشروع على عدة مراحل ، أولها تقييم الأوزان بما في ذلك الزائدة وكذلك تأثيرات المناخ (الثلج والرياح) وهذا حسب اللوائح الجزائرية " RNV 99" إصدار 2013 ، ثم قياس العناصر الأساسية والثانوية للمبنى بعد دراسة ديناميكية حسب اللوائح الجزائرية. بالنسبة للزلزال ، "RPA 99" إصدار 2003 ثم حساب التوصيلات والتجمعات وفقًا للائحة "CCM 97" أخيرًا ، تمت دراسة البنية التحتية وفقًا لقواعد "BAEL 91" وللبرامج التي اعتمدنا عليها "ROBOT" ، ينتهي العمل باستنتاجات.

الكلمات المفتاحية: الهياكل المعدنية – مبنى للتخزين – القياس – التجميع – الزلازل.

Résumé

Ce projet vise à concevoir et à étudier un bâtiment en structure métallique dans la base de vie EL-MERK situé à la commune de EL Borma Wilaya Ouargla. Il est constitué de plusieurs portiques, stabilisés par des contreventements, et couvert par une toiture à quatre versants symétrique. Ce projet doit être étudié en plusieurs étapes, dont la première est l'évaluation des poids, y compris les excès, ainsi que des effets du climat (neige et vent), et ceci selon la réglementation algérienne, « RNV 99 » version 2013, en suit, dimensionner les éléments de base et secondaires du bâtiment après une étude dynamique selon la réglementation algérienne. Pour le séisme, « RPA 99 » version 2003 puis calculer les connexions et assemblages selon la réglementation « CCM 97 » Enfin, l'infrastructure a été étudiée selon les règles de « BAEL 91 » et pour les programmes sur lesquels nous nous sommes appuyés « ROBOT », le travail se termine par des conclusions.

Mots clés : Charpente métallique – Hangar de stockage –Dimensionnement – Assemblage –Séisme.

GENERAL INTRODUCTION

As part of our **Maste**r in Civil Engineering training specializing in «Metal Construction» at the **University of kasdi Merbah**, «**Ouargla**», we are brought, at the end of our course, to carry out an end of studies project (ESP), The aim of this project is to be confronted with a scientific and technical professional situation It therefore brings together all of the qualities that an engineer must have in his daily work. This is a hangar in metal frame in the Wilaya of **Ouargla**.

Metal frames are distinguished by certain advantages such as:

The lightness, the quick assembly and the ease of transformation, this is the reason why this hall was designed with a metal frame. This pendant, this material, also has some of the drawbacks, which are mainly corrosion and its low resistance to fire, so the protection of the entire structure is essential.

In this end of studies project, we will apply and supplement the knowledge and information acquired during our training, using the construction rules currently in force in Algeria, as well as the means of computer calculation.

I.1 Introduction

The metal construction allows for quick and efficient installation, a long service life and environmentally friendly demolition. Considering its total lifespan, a steel structure stands the comparison with other modes of construction.

A good knowledge of the materials used in metal construction is essential for the realization of a structural. In our case, we have chosen steel (S235, S257) as the basic material for the technical study and design of a metal building for its physical and mechanical characteristics (rigidity, ductility, etc.) which allow us to meet the requirements.

I.2 Presentation of the project

As part of our end-of-study project, our work consists of the design and calculation of an industrial building in a metal frame which will be used as a telecom maintenance building for the benefit of the EL-MERK base at the EL Borma site in Wilaya of Ouargla.

I.2.1 Location of the project site

The project is located in the EL-MERK site in the municipality of El Barma in the Wilaya of Ouargla, 343 km from Ouargla, in Algeria.

I.2.2 Geometric data

length of the structure: **21m**

Width of the structure: **11m**

Total height: **5.20m**

-we have five frames.

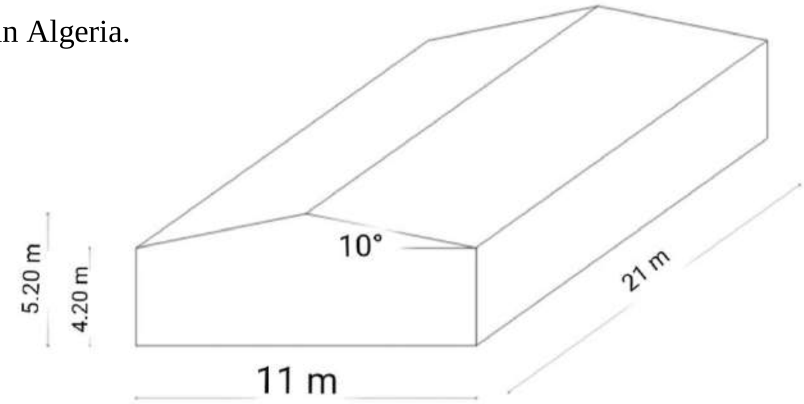


Figure I.1: Diagram showing the structure

- Spacing between frame is (**5.50**, **5.50**, **5,00**, **5,00**) **m**

I.2.3 site data

- Seismicity zone 0, according to the classification established by the (**RPA 99 amends 2003).**

- Zone **lll** wind according to the (**DTR C2-47 RNV version 2013).**

Allowable soil stress, **δ**=2 bar (sand geotechnical report).

Area of sand: **zone D.**

I.2.4 Regulation used

Table I.1: Technical regulations used**.**

types of regulation	Definition
RNV99-V2013	Rules defining the effects of snow and wind
RPA99-V2003	Algerian seismic rules version 2003
CCM97	Design and calculation of steel structures
BAEL91	Reinforced concrete at limit states
DTR BC 2 .2	Loads and overloads
EUROCODE 3	calculation of steel structures

I.2.5 Units used

Table I.2: units used**.**

Units	Use
Metre **m**	Dimensions of buildings, spans and dimension of elements.
Squares metre $\mathbf{m^2}$	For steel sections
daN/m^2	For applied loads
daN.m	For the flexing moments.
daN	For concentrated loads.

I.2.6. Choosing structural

- The metal frame is embedded at the base.
- In the transverse direction vertical stability is ensured by ordinary self-stable gantries.
- In the longitudinal direction, vertical stability is ensured by stabilization steps in X.

I.3 structural elements

Column: **HEA**. Rafter: **IPE** Post: **HEA**

Purlin: **UPN**. strut purlin: **HEA.**

Bracing: **L 60×60×6** Stabilities: **2*L 60×60×6**

I.4 Materials used

For metal frame (profiles):

For our project, we chose the following construction materials:

I.4.1 structural steels for structural elements

Steel is a material made up mainly of iron and a little carbon, which are extracted from natural raw materials taken from the sub sand (iron and coal mines).

Carbon is involved in the composition only to a very small extent (generally less than 1%).

In addition to iron and carbon, steel can contain other elements associated with it, such as:

Unintentionally like phosphorus and sulfur which are the impurities which alter the properties of steels.

Voluntarily like silicon, manganese, nickel, chromium… etc. the latter have the properties of improving the mechanical characteristics of steels (tensile strength, hardness, ductility, resistance to corrosion).

- Steel **E28 (S257JR)** for structural elements.
- Steel **E24 (S235 JR)** for flat irons.
- Steel **E28 (S257 JR)** for anchoring bolts.

preloaded bolts according to the standard **NF EN 14399-3**.

welds must comply with the standar **dNF P 22-470** ou **CM66(80)**.

For reinforcing steel, we use **FeE 400**.

I.4.2 concrete

Concrete is a building material Composed of aggregates, sand, cement, water and possibly additive to modify its properties.

Concrete has excellent compressive strength up to **450daN / cm²** but 10 times less in tension or in shear.

unit weight **ρ= 25 KN/m³**

- The concrete used is defined from a mechanical point of view by:

Compressive strength at 28 days: $f_{c28} = 25\ MPa.$

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

II.1 Introduction:

The effect of climatic actions on a metal construction is very important. So, an in-depth study must be developed for the determination of the different actions due to wind and snow in all possible directions, depending on the **RNV99- version 2013.**

II.2 Wind calculations:

The wind is a very important horizontal action which acts directly on the structure in these two main directions. For this, an in-depth study is taken into account when sizing the metal framework, this study is carried out based on the characteristics depending on the structure as well as the installation site. In our case, these characteristics are as follows:

Wilaya of Ouargla belongs to wind zone III **[Wind map- RNV/2013]**

The industrial zone is classified as **Category I** land.

[Table2.4-RNV/2013]

The implantation site is a flat site.

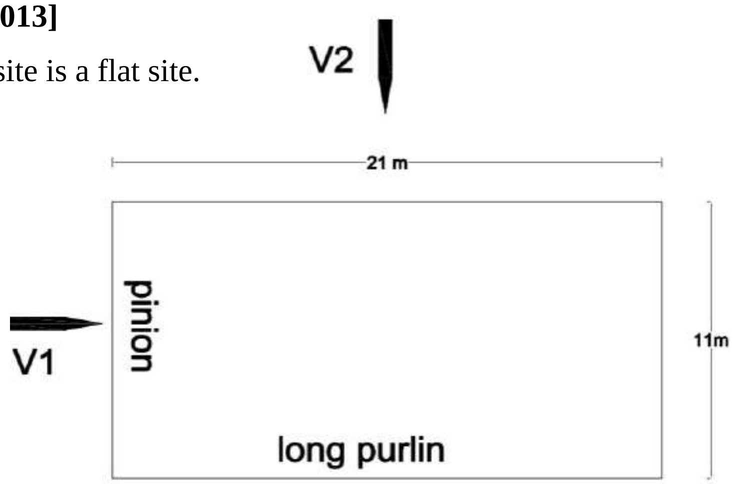


Figure II.1: the main wind direction

II.2.1 Determination of peak dynamic pressure:

$q_{\mathrm{p}}(\mathrm{z_e}) = q_{\mathrm{réf}} \times \mathrm{Ce(ze)}$ [Formula 2.1 RNV/2013]

The structure is a permanent construction located in zone III therefore:

$q_{\mathrm{réf}} = 500\ \mathrm{N/m^2}$ [Table2.2 – RNV/2013]

II.2.1.1 Reference height Ze:

For windward walls of buildings with vertical walls, Ze is determined as shown in **figure 2.1 de RNVA 2013**.

As in our case, the height of the walls $h = 4.20\,m < b = 11\,m$; Ze = h = 4.20m.

For roofs, Ze is taken equal to the maximum height of buildings; (According to RNVA 2013 Chap 2item 2.3.2) → $Ze = H = 5.20\ m$.

II.2.1.2 Expo sure coefficient C_e :

$C_e(Z) = C_t^2(Z) \times C_r^2(Z) \times (1 + 7I_v(Z))$ [Formula 2.2 RNV/2013]

II.2.1.3 Topography coefficient C_t:

The structure is located in a flat site (Ø<0.05) therefore:

$C_t(Z) = 1$ [Formula 2.1 RNV/2013]

II.2.1.4 Coefficient of roughness C_r:

The structure is located in an area therefore: [Table2.4 – RNV/2013]

Category III land

$K_T = 0.17$

$Z_{min} = 1m$

we have**:** $Z_{min} = 1m < Z < 200m$

$C_r(Z) = K_T \times \ln(\frac{Z}{Z0})$

$Z_0 = 0.01$

Roofing: $Z_e = 5.20\ m$ $C_r(5.20) = 0.783$

Vertical walls: $Z_e = 4.20\ m$ $C_r(4.20) = 1.027$

II.2.1.5 Turbulence intensity I_v:

we have: $Z > Z_{min} = 1m$

$$I_v(Z) = \frac{1}{C_t(Z) \times \ln(\frac{Z}{Z_0})}$$ [Formula 2.5 RNV/2013]

Roofing: $Z_e = 5.20m$ $I_v(5.20) = 0.160$

Vertical walls: $Z_e = 4.20\ m$ $I_v(4.20) = 0.166$

Finally, the dynamic pressure values are summarized, including the following table:

Table II.1: dynamic pressure values

Coefficient	$Z_e(m)$	$C_t(Z)$	$C_r(Z)$	I_v	C_e	$q_{réf}$ (N/m^2)	$q_p(z_e)$(N/m^2)
Vertical walls	4.20	1	1.027	0.166	2.276	500	1138
Roofing	5.20	1	0.783	0.160	1.3	500	649.5

II.2.2 Determination of dynamic coefficient C_d

The dynamic coefficient C_d is given in chapter 3 of **RNV/2013**. In the case of our project, the total height of the structure H = 5.20 m is strictly less than 15 m so we can take the simplified value of C_d.

$C_d = 1$ [§3.2 − RNV/2013]

II.2.3 Determination of external pressure coefficients C_{pe} and internal C_{pi} :

II.2.3.1 The values of C_{pe}:

II.2.3.1.1 Wind perpendicular to the long-side (direction V1):

- Vertical walls:

b = $11m$; d = 21m ; h = 4.20m.

$e = Min(b; 2 \times h) = Min(11; 2 \times 4.20) = 8.40m.$

$d = 21\ m > e = 8.40m.$

The following table gives the areas and values of C_{pe} for each zone:

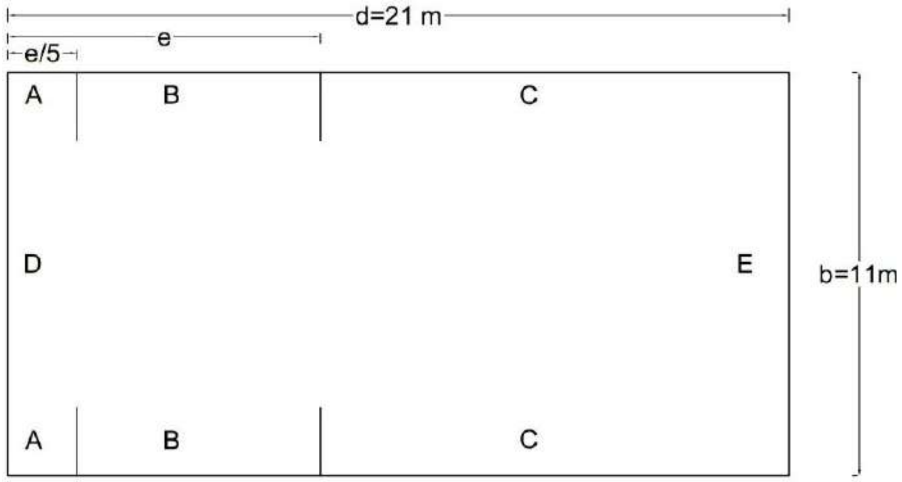


Figure II.2: legend for vertical walls

Table II.2: value of the results of the coefficient of external pressure for the walls in the direction V1, ϴ = 90°.

Zone	A	B	C	D	E
Surface (m^2)	7.10	28.2	52.9	46.2	46.2
C_{pe}	-1.05	-0.80	-0.50	0.80+	-0.30

The surface of each zone is> 10m² therefore:

$C_{pe} = C_{pe.10}$ [§5.1.1.2 − RNV/2013]

- Roofing:

The wind whose direction is perpendicular to the generators, we will take the values of C_{pe} of two-sided roofing or the wind direction $\Theta = 90°$ [§5.1.8.1 $- RNV/2013$]

We have: $\Theta = 90°$; $\propto = 10°$; $b = 11m$; $d = 21m$; $h = 5.20m$.

$e = M_{min}\ (b\,; 2h) = M_{min}\ (11\,; 2 \times 5.20) = 10.4\ m$

In this case we have five zones as follows:

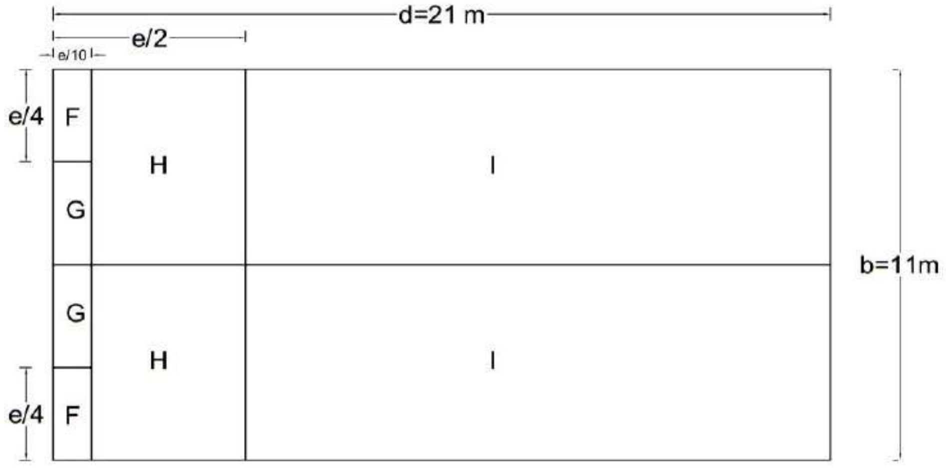


Figure II.3: Legend for the roof (derection V1)

Table II.3: The values of the surfaces of the wind zones of the roof in direction V1.

zone	F	G	H	I
Surface (m^2)	2.70	3.00	22.9	86.9

The surface of each zone is> 10m² therefore:

$C_{pe} = C_{pe.10}$ [§5.1.1.2 $- RNV/2013$]

Since $\propto$ = 10 °, therefore the values of C_{pe} are determined by the linear interpolation between the two values of C_{pe} (5 °) and C_{pe} (15 °) by the following formula:

$$C_{pe}(10°) = C_{pe}(5°) + [\frac{10-5}{15-5}\ (C_{pe}(15°) - C_{pe}(5°))]$$

The following table gives the values of C_{pe} for each zone:

Table II.4: The values of the Cpe on the roof in the V1 direction.

zone	F	G	H	I
C_{pe}	1.82-	-1.66	-0.65	-0.55

II.2.3.1.2 Wind perpendicular to the long-side (direction V2):

- Vertical walls:

b $=$ 21m ; $d = 11m$; $h = 4.20m$.
$e = Min(b; 2 \times h) = Min(21; 2 \times 4.20) = 8.40m$.
$d = 21\ m > e = 8.40m$.
The following table gives the areas and values of Cpe for each zone:

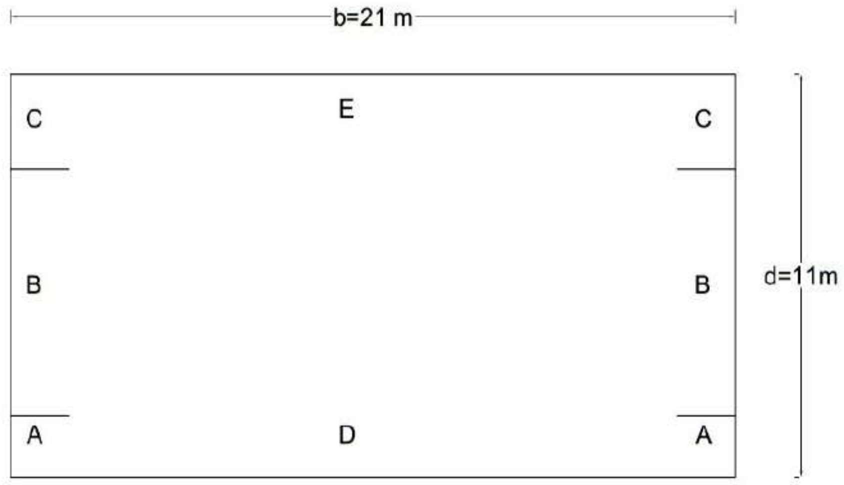


Figure II.4: Legend for vertical walls (derection V2)

Table II.5: value of the results of the coefficient of external pressure for the walls in the direction V2, ϴ = 0°.

Zone	A	B	C	D	E
Surface (m^2)	7.10	28.2	10.9	88.2	88.2
C_{pe}	-1.05	-0.80	-0.50	0.80+	-0.30

The surface of each zone is > 10m² therefore:

$C_{pe} = C_{pe.10}$ [§5.1.1.2 $- RNV/2013$]

- Roofing:

The wind whose direction is perpendicular to the generators, we will take the values of C_{pe} of two-sided roofing or the wind direction ϴ $= 0°$

We have: ϴ $= 0°$; $\propto = 10°$; $b = 21m$; $d = 11m$; $h = 5.20m$.

$e = M_{min}\ (b\ ; 2h) = M_{min}\ (11\ ; 2 \times 5.20\ ) = 10.4\ m$

In this case we have five zones as follows:

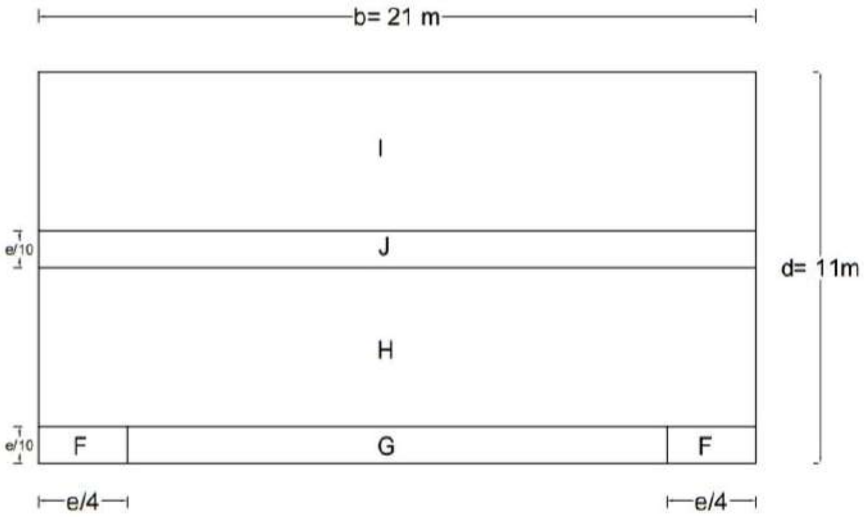


Figure II.5: Legend for the roof (derection V2)

Table II.6: The values of the surfaces of the wind zones of the roof in direction V2

Zone	F	G	H	I	J
Surface (m^2)	2.70	16.40	93.70	93.70	21.84

The surface of each zone is $> 10m^2$ therefore:

$C_{pe} = C_{pe.10}$ [$\S 5.1.1.2 - RNV/2013$]

Since $\propto$ = 10°, therefore the values of C_{pe} are determined by the linear interpolation between the two values of C_{pe} (5 °) and C_{pe} (15 °) by the following formula:

$$C_{pe}(10°) = C_{pe}(5°) + [\frac{10-5}{15-5} \ (C_{pe}(15°) - C_{pe}(5°))]$$

The following table gives the values of C_{pe} for each zone:

Table II.7: The values Cpe for each zone

Zone	F	G	H	I	J
$C_{pe}(Depression)$	-1.33	-1.00	-0.45	-0.50	-0.40
$C_{pe}(surpression)$	0.10	0.10	0.10	-0.30	-0.30

II.2.3.2 The values of Cpi:

II.2.3.2.1 the area of the openings in the faces (long side):

Face BC: three openings of 0.8 × 1.20 and three openings of 0.8 × 2.10.
Face AD: three openings of 0.8 × 1.20, one opening 1.6 × 2.10, one opening of 0.6 × 0.6 and one opening of 4.0 × 3.50.

II.2.3.2.2 the area of the openings in the other faces:

- We have (pinion):

Face AB: one opening of 0.8 × 2.10.

Face CD: three openings of 0.8 × 1.20.

- **Calculation the coefficient of permeability μp:**

$$\mu p = \frac{\sum of\ the\ surfaces\ of\ the\ openings\ or\ Cpe \leq 0}{\sum surfaces\ of\ all\ openings}$$

Table II.8: value of the results of the coefficient of permeability

Face	Surface(m2)	μp	observation
AB	1.68	0.95	no dominant face
BC	20.6	0.38	no dominant face
CD	2.88	0.91	no dominant face
AD	7.92	0.76	no dominant face

- **Calculation of the internal pressure coefficient C_{pi}:**

Table II.9: value of the results of the coefficient of the internal pressure

Face	h/d	Cpi
AB	0.25	-0.30
BC	0.25	0.27
CD	0.25	-0.30
AD	0.25	-0.15

II.2.4 Calculation of the different pressures on the structure

The pressure Wzj acting on a surface element of the structure as a function of the height is given as follows: $[Formule2.6 - RNV/2013]$

$$\mathrm{W(zj)} = q_{\mathrm{p}}(\mathrm{Ze}) \times [\mathrm{C_{pe}} - \mathrm{C_{pi}}] \qquad [\mathrm{N/m^2}]$$

we have $q_j = C_d \times W(zj)$

so $q_j = C_d \times q_{\mathrm{p}}(ze) \times [\mathrm{C_{pe}} - \mathrm{C_{pi}}] \qquad [N/m^2]$

- Vertical walls:

The following table gives the values of the pressure on the vertical walls for the wind direction V1 in the case of the internal depression.

Table II.10: Pressure values on the walls in the v1 direction ϴ=90°, facing the wind AB, CD.

Zone	$q_{\mathrm{p}}[N/m^2]$	C_{pe}	C_{pi}	$(C_{pe} - C_{pi})$	$W_{zj}[N/m^2]$
A	1138	-1.05	-0.30	-0.75	-854
B	1138	-0.80	-0.30	-0.50	-569
C	1138	-0.50	-0.30	-0.20	-228
D	1138	0.80	-0.30	1.1	1252
E	1138	-0.30	-0.30	0.00	0

- Roofing:

The following table gives the values of the pressure on the roof for the wind direction V1 in the case of the interior depression.

Table II.11: Values of the dynamic pressure on the roof, direction V1 ϴ = 90 °, facing the wind AB, CD.

Zone	$q_{\mathrm{p}}[N/m^2]$	C_{pe}	C_{pi}	$(C_{pe} - C_{pi})$	$W_{zj}[N/m^2]$
F	649.5	-1.82	-0.30	-1.52	-987
G	649.5	-1.66	-0.30	-1.36	-883
H	649.5	-0.65	-0.30	-0.35	-227
I	649.5	-0.55	-0.30	-0.25	-162

- Vertical walls:

The following table gives the values of the pressure on the vertical walls for the wind direction V2 in the case of the internal depression.

Table II.12: Pressure values on the walls in the v2 direction ϴ =0°, facing the wind **BC.**

Zone	$q_{\mathrm{p}}[N/m^2]$	C_{pe}	C_{pi}	$(C_{pe} - C_{pi})$	$W_{zj}[N/m^2]$
A	1138	-1.05	0.27	-1.32	-1502
B	1138	-0.80	0.27	-1.07	-1218
C	1138	-0.50	0.27	-0.77	-876
D	1138	0.80	0.27	0.53	603
E	1138	-0.30	0.27	-0.57	-649

Table II.13: Pressure values on the walls in the v2 direction Θ=0°, facing the wind **AD.**

Zone	$q_{\mathrm{p}}[N/m^2]$	C_{pe}	C_{pi}	$(C_{pe} - C_{pi})$	$W_{\mathrm{zj}}[N/m^2]$
A	1138	-1.05	-0.15	-0.90	-1024
B	1138	-0.80	-0.15	-0.65	-740
C	1138	-0.50	-0.15	-0.35	399
D	1138	0.80	-0.15	0.95	1081
E	1138	-0.30	-0.15	-0.15	-171

- Roofing:

The following table gives the values of the pressure on the roof for the wind direction V2 in the case of the interior depression.

Table II.14: Values of the dynamic pressure on the roof, direction V2 Θ =0 °, facing the wind BC.

Zone	$q_{\mathrm{p}}[N/m^2]$	C_{pe}	C_{pi}	$(C_{pe} - C_{pi})$	$W_{\mathrm{zj}}[N/m^2]$
Depression					
F	649.5	-1.33	0.27	-1.6	-1039
G	649.5	-1.00	0.27	-1.27	-825
H	649.5	-0.45	0.27	-0.72	-468
I	649.5	-0.50	0.27	-0.77	-500
J	649.5	-0.40	0.27	-0.67	-435
Surpression					
F	649.5	0.10	0.27	-0.17	-110
G	649.5	0.10	0.27	-0.17	-110
H	649.5	0.10	0.27	-0.17	-110
I	649.5	-0.30	0.27	-0.57	-370
J	649.5	-0.30	0.27	-0.57	-370

Table II.15: Values of the dynamic pressure on the roof, direction V2 ϴ =0 °, facing the wind AD.

Zone	$q_{\mathrm{p}}[N/m^2]$	C_{pe}	C_{pi}	$(C_{pe} - C_{pi})$	$W_{zj}[N/m^2]$
Depression					
F	649.5	-1.33	-0.15	-1.18	-1039
G	649.5	-1.00	-0.15	-0.85	-825
H	649.5	-0.45	-0.15	-0.30	-468
I	649.5	-0.50	-0.15	-0.35	-500
J	649.5	-0.40	-0.15	-0.25	-435
Surpression					
F	649.5	0.10	-0.15	0.25	-110
G	649.5	0.10	-0.15	0.25	-110
H	649.5	0.10	-0.15	0.25	-110
I	649.5	-0.30	-0.15	-0.15	-370
J	649.5	-0.30	-0.15	-0.15	-370

II.2.5 Determination of friction force:

$$\frac{d}{b} = \frac{21}{11} = 1.9 < 3$$

$$\frac{d}{h} = \frac{21}{5.20} = 4.04 > 3$$

One of the conditions is verified. it is necessary to consider the frictional forces.

The friction force F_{fr} is given by the following formula:

$$F_{fr} = \sum (q_h \times C_{fr} \times S_{fr})$$

Or:

q_h: (in daN / m^2) is the dynamic pressure of the wind at the height h considered.

S_{fr}: (in m^2) is aire of element of the considered surface.

C_{fr}: is the coefficient of friction for element of surface considered.

In our case, we will take a cladding on the roof and at the level of the vertical walls, the undulations of which are perpendicular to the direction of the wind. (table 2.8 RNVA 2013). the friction force is therefore:

- Roofing:

$F_{fr} = 0.65 \times 0.04 \times (21 \times 2 \times 5.59) = 6.10$ KN.

- vertical walls:

$F_{fr} = 1.138 \times 0.04 \times (21 \times 2 \times 4.20) = 8.03$ KN

The total frictional force. $F_{fr} = 7.25 + 5.45 = 12.7$ KN

Note:

The friction area for the roof is determined by entering the length of the developed roof, namely: $5.5/\cos 10 = 5.58$

III.1 purlin study

III.1.1 INTRODUCTION

The secondary elements represent the framework necessary to support the roofing and the cladding. Therefore, the calculation of the procedure is as follows:

1. Evaluate the loads applied to the elements considered, then find the combination with the most unfavorable effect.

2. Pre-sizing of the elements according to the deflection limitation condition.

3. Verification of resistance, stability and rigidity based on the formulas of the resistance of materials (RDM) and the various rules.

The secondary elements that make up our structure are as follows:

- PURLIN.
- WALL GIRT.
- POSTS.

III.1.2 Definition

Purlins are beams intended to support the roof and to transmit the loads and overloads applied to the latter to the cross member or to the farm. They are arranged parallel to the ridgeline, and they are calculated in deflected bending, under the effect of dead loads, operations and climatic. They either are made in hot-formed sections in (I), or else in (U), or in cold-formed sections in (Z), (U), (Σ) or in a lattice for spans greater than 5.5m. In our structure, we will use UPN.

we take a spacing between purlins of 1.25 m (4 times) and at both ends 0.35 m and 0.24 m.

III.1.3 Determination of stresses

III.1.3.1 Evaluation of loads and overloads

III.1.3.1.1 Dead load (G):

TL.80 sandwich purlin cladding weight with accessories……. 60 kg/m^2.

Estimated self-weight of the purlin (UPN 140) ……. 16kg/m

$G = (W_{\text{blanket}} + A_{\text{ccessory}}) \times s + W_{\text{purlin}}$

s: spacing between purlins (s = 1.25 m).

$G = 60 \times 1.25 + 16 = 91\ \text{kg/m} = 0.91\ \text{KN/m}.$

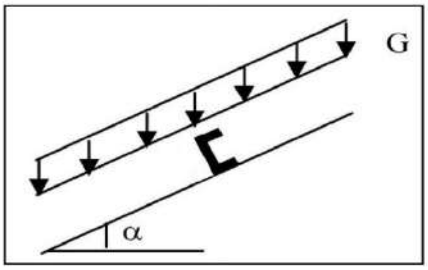


Figure III.1: Static diagram of the permanent loads G on the purlins

III.1.3.1.2 Live load:

Concentrated load of 100 Kg, each located at 1/3 and 2/3 of the reach. The uniformly distributed load **q** is obtained by equalizing the two maximum moments due to **Q'**.

$$M_{max} = \frac{Q' \times L}{3} = \frac{q \times L^2}{8} \quad \mapsto \quad q = \frac{8 \times Q'}{3 \times L} = 48.48 \text{ Kg/m} = 0.48 \text{ KN/m}$$

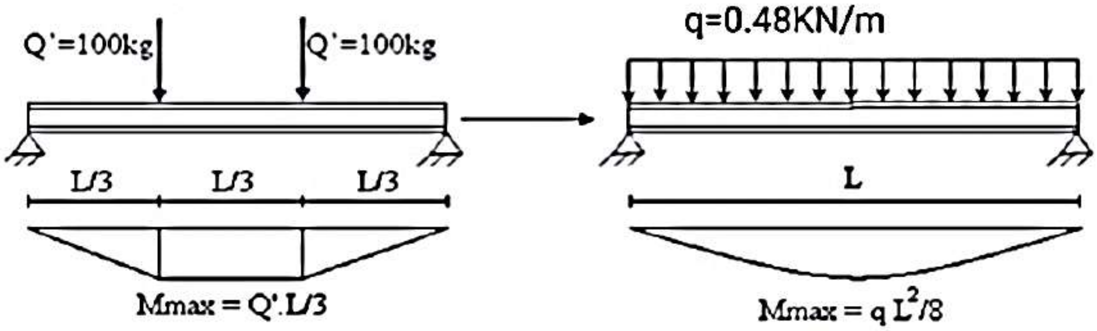


Figure III.2: Distribution of point loads over the scope of the purlin

III.1.3.1.3 Climatic wind load (W):

$W = -0.5 \times 1.25 = -0.625 \text{ KN/m}$

III.1.3.1.4 Climatic sand load (S):

$S = 0.25 \times 1.23 = 0.31 \text{ KN/m}$

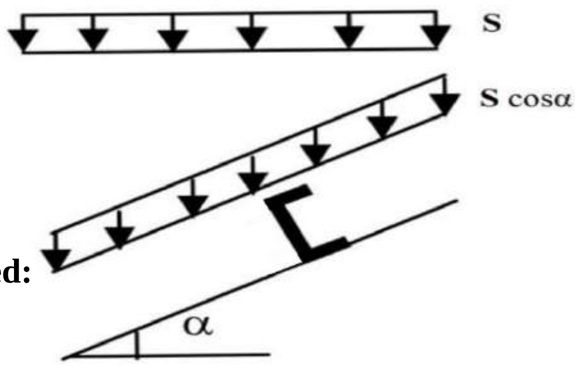


Figure III.3: Climatic sand load S

III.1.3.1.5 The loads and live loads applied:

$G = 0.91 \text{ KN/m}$

$W = -0.625 \text{ KN/m}$

$S = 0.31 \text{ KN/m}$

$q = 0.4848 \text{ KN/m}$

III.1.3.2 Most unfavorable load combinations:

III.1.3.2.1 Down action: ↓

$Q_{sd1} = 1.35\, G + 1.5\, q = 1.35 \times 0.91 + 1.5 \times 0.48 = 1.95 \text{ KN/m}$

$Q_{sd2} = 1.35 \times G + 1.5 \times S = 1.35 \times 0.91 + 1.5 \times 0.31 = 1.694 \text{ KN/m}$

III.1.3.2.2 Action up: ↑

$Q_{z.sd} = G \cos\alpha - 1.5\, W = 0.91 \cos(10) - 1.5 \times 0.625 = -0.041 \text{ KN/m}$

$Q_{y.sd} = 1.35\, G \sin\alpha = 1.35 \times 0.91 \times \sin(10) = 0.213 \text{ KN/m}$

The most unfavorable combinations to be used to calculate them:

III.1.3.2.3 Section resistance:

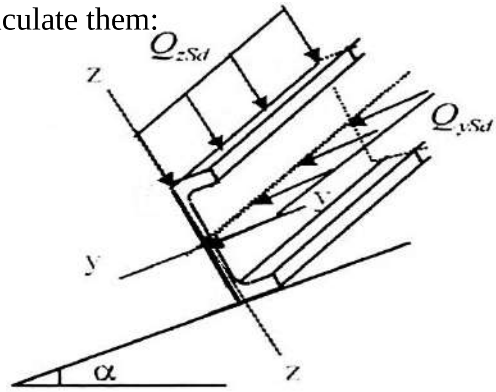


Figure III.4: Section resistance

$Q_{sd} = 1.95\ \mathrm{KN/m}$

$Q_{z.sd} = Q_{sd} \cos\alpha = 1.92\ \mathrm{KN/m}$

$M_{y.sd} = \frac{Q_{z.sd} \times l^2}{8} = \frac{1.92 \times 5.5^2}{8} = 7.26 \mathrm{KN.m}$

$Q_{y.sd} = Q_{sd} \sin\alpha = 0.34\ \mathrm{K\ /m}$

$M_{z.sd} = \frac{Q_{y.sd} \times (l/2)^2}{8} = \frac{0.34 \times 2.75^2}{8} = 0.32 \mathrm{KN.m}$

III.1.3.2.4 Element spill: compressed lower sole not laterally retained.

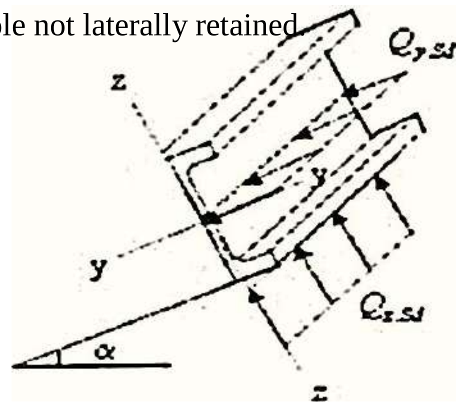


Figure III.5: compressive force.

$Q_{z.sd} = -\ 0.041\ \mathrm{KN/m}$

$Q_{y.sd} = 0.213\ \mathrm{KN/m}$

$M_{y.sd} = \frac{Q_{z.sd} \times l^2}{8} = \frac{0.041 \times 5.5^2}{8} = 0.155 \mathrm{KN.m}$

$M_{z.sd} = \frac{Q_{y.sd} \times (l/2)^2}{8} = \frac{0.213 \times 2.75^2}{8} = 0.81 \mathrm{KN.m}$

III.1.4 Principle of pre-sizing.

III.1.4.1 Condition of the resistance (ULS).

III.1.4.1.1 flexion verification:

III.1.4.1.1.1 Calculation in plasticity: (Sections of class 1 and 2)

$$\left(\frac{M_{y.sd}}{M_{pl.y.Rd}}\right)^{\alpha} + \left(\frac{M_{z.sd}}{M_{pl.z.Rd}}\right)^{\beta} \leq 1.0$$

where α and β are constants which place in security if they are taken equal to the unit, but which can take the following values:

sections in **I** and **H**: $\alpha = 2$ and $\beta = 5n \geq 1.$

with: $n = N_{sd} / N_{pl.Rd} = 0 \mapsto \beta = 1.$

by trial and error, we choose the following profile **UPN140**.

III.1.4.1.1.2 section class:

sole class: (compressed sole)

$$\frac{c}{t_f} = \frac{b/2}{t_f} \leq 10\varepsilon$$

$$\varepsilon = \sqrt{\frac{235}{fy}} = \sqrt{\frac{235}{275}} = 0.92$$

$\frac{b/2}{t_f} = \frac{60/2}{10} = 3.00 \leq 9.2$ …………………………………………………………… Ok

Soul check: (flexed)

$$\frac{c}{t_f} = \frac{d}{t_w} \leq 72\varepsilon$$

$\frac{d}{t_W} = \frac{98}{7} = 14 \leq 66.24$ …………….…………………………………………… Ok

The section is class 1

Note:

Rolled sections with gauges less than or equal to **UPN 140**, are generally of a class 1 section.

III.1.4.1.1.3 geometric characteristics of l' UPN 140:

$Wel.y = 86.4\ cm^3$; $Wel.z = 14.8\ cm^3$

$Wpl.y = 103\ cm^3$; $Wpl.z = 28.3\ cm^3$

$$M_{pl.y.Rd} = \frac{Wpl.y \times fy}{\gamma_{M1}} = \frac{103 \times 2750 \times 10^{-2}}{1.1} = 2575\ dan.m$$

$$M_{pl.z.Rd} = \frac{Wpl.z \times fy}{\gamma_{M1}} = \frac{28.3 \times 2750 \times 10^{-2}}{1.1} = 707.5\ dan.m$$

$$\left(\frac{M_{y.sd}}{M_{pl.y.Rd}}\right)^{\alpha} + \left(\frac{M_{z.sd}}{M_{pl.z.Rd}}\right)^{\beta} = \left(\frac{7.26}{25.75}\right)^{2} + \left(\frac{0.32}{7.075}\right)^{1} = 0.13 \leq 1.0 \text{ …………….. Verified.}$$

III.1.4.1.2 Shear verification:

The shear verification is given by the following formulas:

$V_{z.sd} \leq V_{pl.z.Rd}$; $V_{y.sd} \leq V_{pl.y.Rd}$

$$V_{pl.z.Rd} = \frac{A_{vz} \times \left(f_y/\sqrt{3}\right)}{\gamma_{M0}} \quad ; \quad V_{pl.y.Rd} = \frac{A_{vy} \times \left(f_y/\sqrt{3}\right)}{\gamma_{M0}}$$

UPN 140: $A_{vz} = 10.4\ cm^2$; $A_{vy} = 12\ cm^2$

$$V_{z.sd} = \frac{Q_{z.sd} \times l}{2} = \frac{1.92 \times 5.5}{2} = 5.28\ KN$$

$$V_{y.sd} = 0.625 \times Q_{y.sd} \times (l/2) = 0.625 \times 0.34 \times (5.5/2) = 0.58\ KN$$

$$V_{pl.z.Rd} = \frac{A_{vz} \times \left(f_y/\sqrt{3}\right)}{\gamma_{M0}} = \frac{10.4 \times \left(2750/\sqrt{3}\right)}{1.1} = 150.11\ KN$$

$$V_{pl.y.Rd} = \frac{A_{vy} \times (f_y/\sqrt{3})}{\gamma_{M0}} = \frac{12 \times (2750/\sqrt{3})}{1.1} = 173.21 \text{ KN}$$

$V_{\text{z.sd}} = 5.28 \text{ KN} \leq V_{\text{pl.z.Rd}} = 150.11 \text{ KN}$ …………………… OK

$V_{\text{y.sd}} = 0.58 \text{ KN} \leq V_{\text{pl.y.Rd}} = 173.21 \text{ KN}$ …………………… OK

Note:

In most cases the shear verification is verified for rolled sections as soon as the bending moment verification is satisfied.

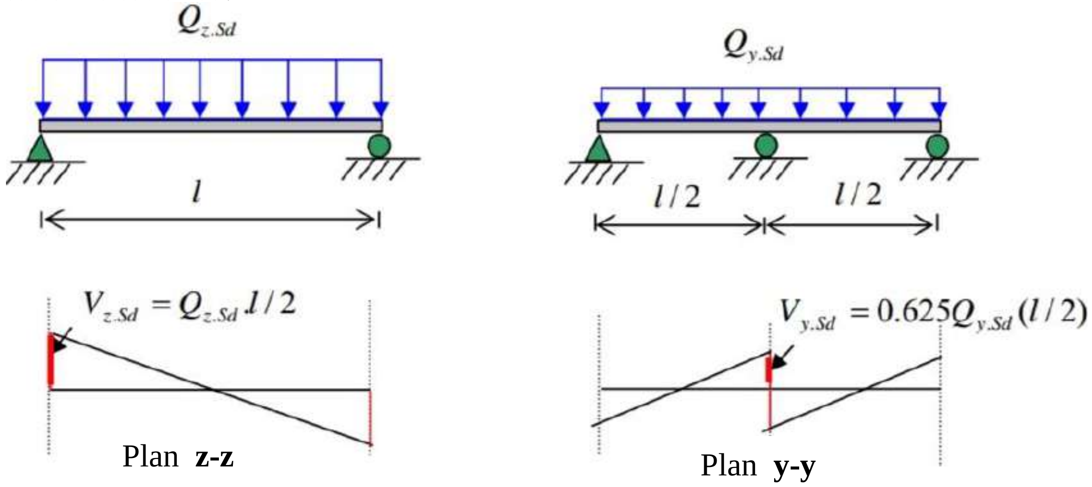


Figure III.6: Shear forces.

III.1.4.2 verification at SLS

III.1.4.2.1 flexion verification:

The calculation of the deflection is made by the combination of loads and overloads of services (unweighted).

$\text{Q}_{\text{Sd1}} = \text{G} + \text{S} = 0.91 + 0.31 = 1.22 \text{ KN/m}$

$\text{Q}_{\text{Sd1}} = \text{G} + \text{q} = 0.91 + 0.48 = 1.39 \text{ KN/m}$

$\text{Q}_{\text{Sd2}} = \text{Gcos} \propto - \text{W} = 0.91 \times \cos 10 - 0.625 = 0.271 \text{ KN/m}$

$\text{Q}_{\text{Sd}} = \text{Max}(\text{Q}_{\text{Sd1}}, \text{Q}_{\text{Sd2}}) = 1.39 \text{ KN/m}$

$\text{Q}_{\text{z.sd}} = \text{Q}_{\text{Sd1}} \cos \propto = 1.369 \text{ KN/m}$

$$\text{Q}_{\text{y.sd}} = \text{Q}_{\text{Sd1}} \sin \propto = 0.24 \text{ KN/m}$$

III.1.4.2.2 Verification condition:

$f \leq f_{ad}$ with $f_{ad} = \frac{l}{200}$

- Vertical arrow (according to zz'): on two support

$$f_{ad} = \frac{l}{200} = \frac{550}{200} = 2.75\ cm$$

$$f_z = \frac{5}{384} \times \frac{Q_{z.Sd} \times l^4}{E \times I_y}$$

$$f_z = \frac{5}{384} \times \frac{1.369 \times (550)^4}{2.1 \times 10^5 \times 605} = 1.28\ cm < f_{ad} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{ OK}$$

- Lateral arrow (according to yy'): on three supports

$$f_{ad} = \frac{l/2}{200} = \frac{275}{200} = 1.375\ cm$$

$$f_y = \frac{2.05}{384} \times \frac{Q_{y.sd} \times (l/2)^4}{E \times I_z}$$

$$f_y = \frac{2.05}{384} \times \frac{0.24 \times (275)^4}{2.1 \times 10^5 \times 62.7} \approx 0.1 \text{ cm} < f_{ad} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{OK}$$

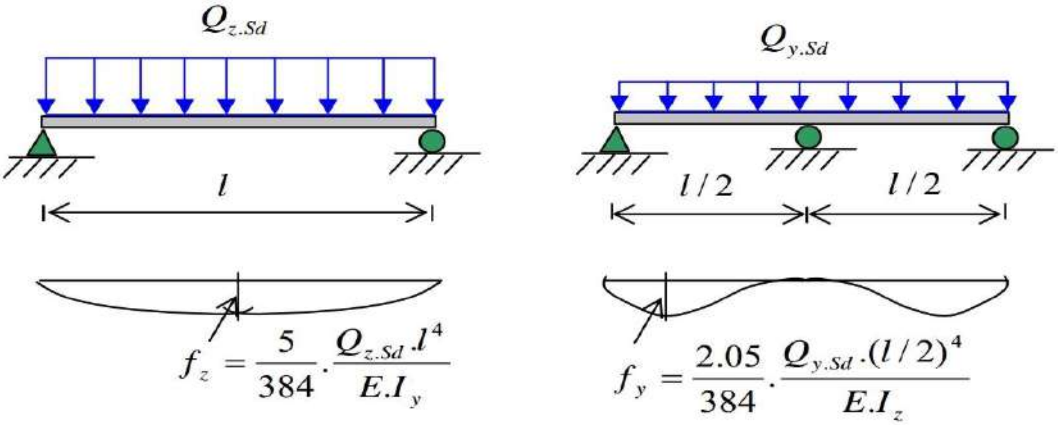


Figure III.7: flexion extension.

III.1.4.3 Verification of the instability element:

III.1.4.3.1 buckling verification:

III.1.4.3.1.1 Calculation of the ultimate moment:

$Q_{z.sd} = G \cos\alpha - 1.5\ W = 0.91 \cos(10) - 1.5 \times 0.625 = -0.041\ KN/m$

$$M_{y.sd} = \frac{Q_{z.sd} \times l^2}{8} = \frac{0.041 \times 5.5^2}{8} = 0.155\ KN.m$$

$Q_{y.sd} = 1.35\ G \sin\alpha = 1.35 \times 0.91 \times \sin(10) = 0.213\ KN/m$

$$M_{z.sd} = \frac{Q_{y.sd} \times (l/2)^2}{8} = \frac{0.213 \times 2.75^2}{8} = 0.81 KN.m$$

$$M_{ply.Rd} = \frac{W_{pl.y} \times fy}{\gamma_{M1}} = \frac{103 \times 2750 \times 10^{-2}}{1.1} = 2575\ dan.m$$

$$M_{plz.Rd} = \frac{W_{pl.z} \times fy}{\gamma_{M1}} = \frac{28.3 \times 2750 \times 10^{-2}}{1.1} = 707.5\ dan.m$$

III.1.4.3.1.2 Calculation of spill resistance moment:

$$M_{b.Rd} = \chi_{LT} \times \beta_w \times \frac{W_{pl.y} \times f_y}{\gamma_{M1}}$$

$\beta_w = 1.0$ for class 1 and class 2 sections.

The reduced slenderness $\bar{\lambda}_{LT}$ is determined by the following formula: (Annex F to the Eurocode, §F.2).

$$\bar{\lambda}_{LT} = \left[\frac{\beta_w \times W_{pl.y} \times f_y}{M_{cr}}\right]^{0.5} = \left[\frac{\lambda_{LT}}{\lambda_1}\right] \times [\beta_w]^{0.5}$$

Where $\lambda_1 = \pi\sqrt{\frac{E}{f_y}} = 86.81$

For beams with constant section and doubly symmetrical (rolled I and H section) the slenderness $\lambda_{LT}$is:

$$\lambda_{LT} = \frac{L/i_z}{C_1^{0.5}\left[1 + \frac{1}{20}\left(\frac{L/i_z}{h/t_f}\right)^2\right]^{0.25}}$$

UPN140: $i_z = 1.75cm$; $h = 14cm$; $t_f = 1cm$; $C1 = 1.132$; $L = 275$ mm

$I_y = 605\ cm^4$; $I_z = 62.7\ cm^4$

$$\lambda_{LT} = \frac{275/1.75}{1.132^{\ 0.5}\left[1 + \frac{1}{20}\left(\frac{275/1.75}{14/1}\right)^2\right]^{0.25}} = 89.86$$

$$\bar{\lambda}_{LT} = \left[\frac{\lambda_{LT}}{\lambda_1}\right] \times [\beta_w]^{0.5} = \frac{89.86}{86.81} = 1.04$$

$$\varphi_{LT} = 0.5\left[1 + \alpha_{LT}\left(\bar{\lambda}_{LT} - 0.2\right) + \bar{\lambda}_{LT}^2\right] = 0.5[1 + 0.49(1.04 - 0.2) + 1.04^2] = 1.25$$

$$\chi_{LT} = \frac{1}{\varphi_{LT} + \left[\varphi_{LT}^2 - \bar{\lambda}_{LT}^2\right]^{0.5}} = \frac{1}{1.25 + [1.25^2 - 1.04^2]^{0.5}} = 0.51$$

III.1.4.3.1.3 Calculation of χ_{LT} using Table 5.5.2 of Eurocode 3.

The values of the reduction coefficient χ_{LT} for the appropriate reduced slenderness $\bar{\lambda}_{LT}$can be obtained from Table 5.5.2 OF EC3 with $\bar{\lambda} = \bar{\lambda}_{LT}$ and $\chi = \chi_{LT}$, using:

- A curve for rolled profiles.
- Curve c for welded profiles.

$\bar{\lambda}_{LT} = 1.04$

By linear interpolation between the values of $\bar{\lambda}_{LT} = 1.00$ and $\bar{\lambda}_{LT} = 1.10$

$$f(x) = f(x_1) - \frac{(x - x_1)}{x_2 - x_1}.f(x_1) - f(x_2)$$

$$f(x) = 0.6656 - \frac{(1.04 - 1.00)}{(1.10 - 1.00)}.(0.6656 - 0.5960) = 0.638$$

$M_{b.Rd} = \chi_{LT} \times \mathrm{M_{ply.Rd}} = 0.638 \times 2575 = 1642.85\ \mathrm{kg.m}$

$\mathrm{M_{y.sd}} = 15.5\ \mathrm{kg.m} < \mathrm{M_{b.Rd}} = 1642.85\ \mathrm{kg.m}$ … … … … … … … … … … … … … … . . Verified.

Conclusion: The profile chooses **UPN 140** suitable for purlins.

III.1.5 Calculation of liernes

Liernes are tie rods that work in traction. They are generally formed of round bars. Their main role is to prevent lateral deformation of the purlins.

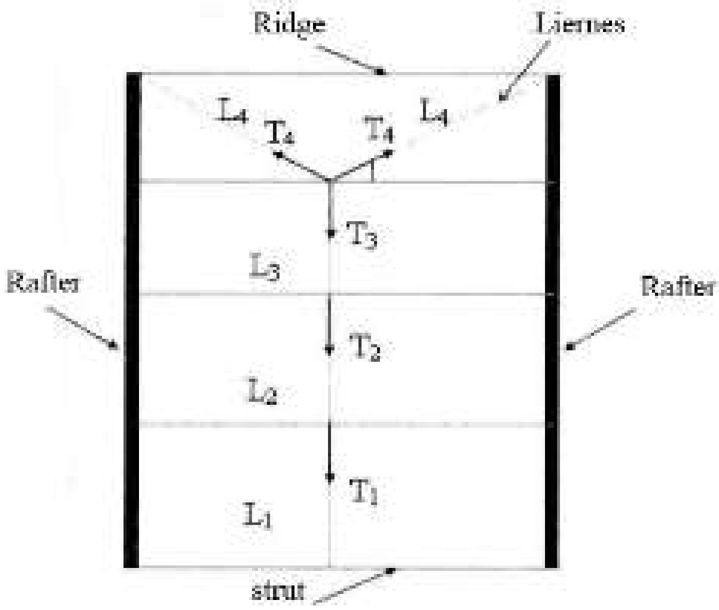


Figure III.8: The arrangement of the lines on the purlins

III.1.5.1 Calculation of the maximum effort due to the liernes:

The R reaction at the level of the lierne:

$\mathrm{R} = 1.25\mathrm{Q_y} \times \mathrm{l}/2 = 1.25 \times 0.34\ \times 2.75 = 1.17\mathrm{KN}$

Tractive force in the section of lierne L1 coming from the sandstone purlin:

$$\mathrm{T_1} = \frac{\mathrm{R}}{2} = \frac{1.17}{2} = 0.59\mathrm{KN}$$

Effort in the section $\mathrm{L_2}$: $T_2 = T_1 + R = 0.59 + 1.17 = 1.76$ KN

Effort in the section $\mathrm{L_3}$: $T_3 = T_2 + R = 1.76 + 1.17 = 2.93$ KN

Effort in the section $\mathrm{L_4}$: $2T_4.\sin\theta = T_3$

$\theta = \mathrm{arctg}\left(\frac{2.75}{0.9}\right) = 71.88°$; $\mathrm{T_4} = \frac{\mathrm{T_3}}{2\sin\theta} = \frac{2.93}{2\times\sin 71.88} = 1.54\mathrm{KN}$

III.1.5.2 Sizing of the liernes:

The most used section is L3.

- Tension element:

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

$N_{pl.Rd} = \dfrac{A.f_y}{\gamma_{M0}}$: Plastic resistance of the row section

$N_{sd} = T_3 \leq \dfrac{A.f_y}{\gamma_{M0}} \quad \mapsto \quad A \geq \dfrac{T_3.\gamma_{M0}}{f_y} = \dfrac{293 \times 1.1}{2750} = 0.12\ cm^2$

$A = \frac{\pi.\emptyset^2}{4} \geq 0.12\ cm^2 \ \mapsto \quad \emptyset = \sqrt{\frac{4\times0.12}{\pi}} = 0.39\ cm$

Either a round bar of diameter: $\emptyset = 0.39\ cm\ = 3.9\ mm$

For practical reasons and for greater safety, we opt for a round bar with a diameter of φ = 6 mm.

III.1.6 Calculation of the sample:

The sampler is a fastening device for attaching purlins to trusses.

The main resistance force of the sample is the overturning moment due to the loading (especially under the action of uplifting the wind).

III.1.6.1 Calculation of the loads accruing to the sample:

Lifting effort:

$Q_{z.sd} = G\cos\alpha - 1.5\ W = 0.91\cos(10) - 1.5 \times 0.625 = -0.041\ KN/m$

Crawling next effort

$Q_{y.sd} = 1.35\ G\ Sin\alpha = 1.35 \times 0.91 \times Sin\ (10) = 0.213\ KN/m$

The eccentricity "t" is limited by the following condition:

$$2(b/2) \leq t \leq 3(b/2)$$

For UPN 140: $b = 6\ cm\ ; h = 14\ cm$

$6 \leq t \leq 9\ cm \quad \mapsto \quad$ Either: $\quad t = 70\ mm$

III.1.6.1.1 Shore sample:

$R_z = Q_{z.Sd}.l/2 = 0.041\ .5.5/2 = 0.113\ KN$

$R_y = Q_{y.Sd}.l/2 = 0.213.5.5/2 = 0.59\ KN$

III.1.6.1.2 Intermediate sample

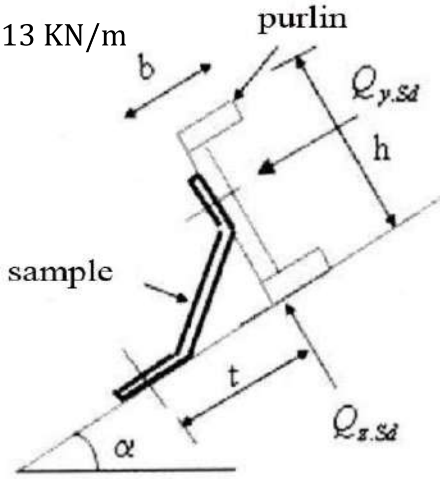


Figure III.9: Sample representation.

$R_z = 2 \times 0.113 = 0.226\ KN\ ; \quad R_y = 2 \times 0.59 = 1.18\ KN$

III.1.6.1.3 Calculation of overturning moment:

$M_R = R_z \times t + R_y \times h/2 = 22.6 \times 7 + 118 \times 7 = 984.2\ daN.cm$

III.1.6.2 Sizing of the sample:

Flexion simple

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

$M_{el.Rd} = \frac{W_{el} \cdot f_y}{\gamma_{M0}}$: (Moment of plastic risistance of the row secton)

$$M_{sd} = M_r \leq \frac{W_{el} . f_y}{\gamma_{M0}}$$

III.1.6.3 Calculation of the thickness of the sample:

$$W_{el} \geq \frac{M_r . \gamma_{M0}}{f_y} = \frac{984.2 \times 1.1}{2750} = 0.394\ cm^2$$

$$W_{el} = \frac{b . e^2}{6} \quad \text{for rectangular sections}$$

$$e \geq \sqrt{\frac{6 \times W_{el}}{b}} = \sqrt{\frac{6 \times 0.394}{15}} = 0.397\ cm \ \ ; \text{either} \ \ \mathrm{e} = 0.397\ \mathrm{cm}$$

Note:

The width of the sample (b= 15 cm) is calculated after dimensioning the top chord of the truss. L70 × 70 × 7 (see CH.6 calculation of trusses).

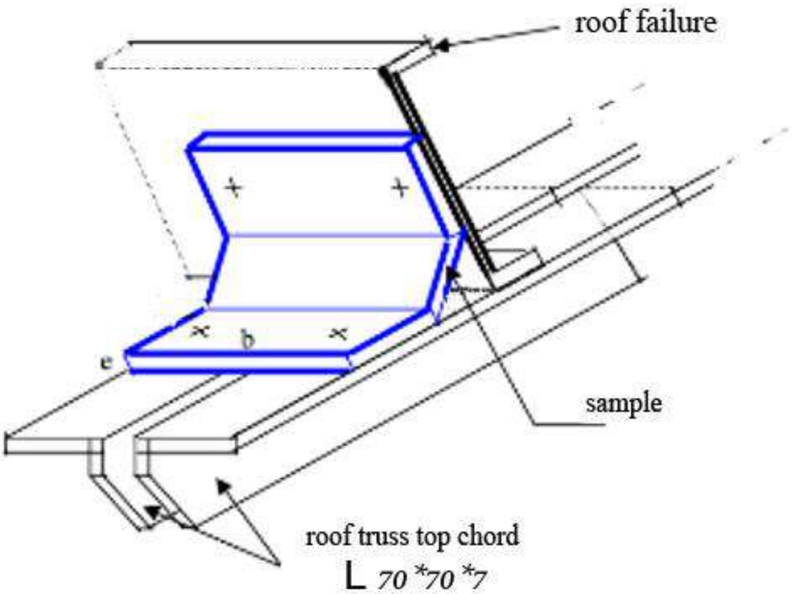


Figure III.10: link the sample between roof failure and roof Struss top chord

III.2. Calculation of the side wall girts:

The side wall girts are elements of rolled section which consist of joists (IPE, UAP, UPE) or thin bent profiles. Being arranged horizontally, they are carried either by the gantry posts or by the intermediate posts. they are intended to take up the forces of the wind on the cladding. The center line of the girts is determined by the allowable span of the cladding boxes.

III.2.1 Calculation data:

Each beam rests on 2 supports at a distance:

- L = 5.5 m on the long side.
- L = 4 m on the gable.
- The span between beam axis e = 1.20 m.
- There are 3 heddle lines on each wall.

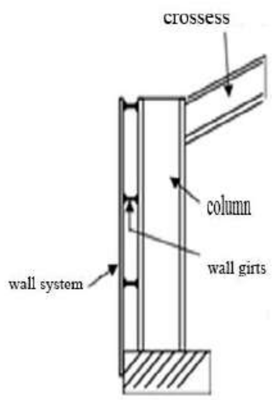


Figure III.11: Arrangement of the beam on the post.

III.2.2 Determination of loads and overloads:

III.2.2.1 verification of the long purlin:

III.2.2.1.1 Permanent loads:

Self-weight of the TL.80 sandwich panel cladding with accessories ...... 20 Kg/m2.

Estimated self-weight of the boom (UPN140) … ... 16 Kg/m.

$$G = (20 \times 1.20) + 16 = 40 \text{ kg/m} = 0.4 \text{ KN/m}.$$

III.2.2.1.2 Climatic wind load:

$W = 1.081 \text{ KN/m2}$

$W = 1.081 \times 1.2 = 1.297 \text{ KN/m}$

III.2.2.1.3 Most unfavorable load combination: $1.35\,G + 1.5\,W$

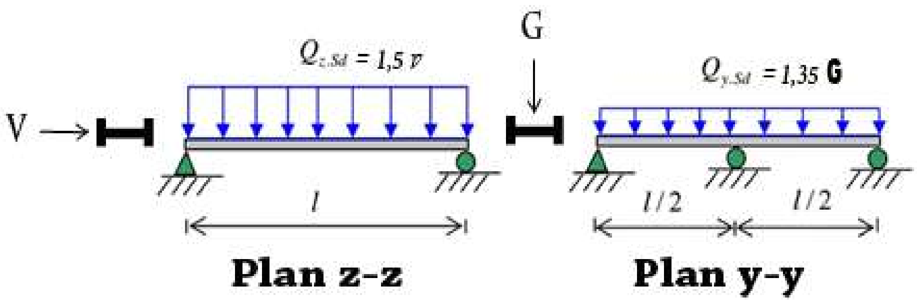


Figure III.12: representation of loads and overloads

- beam on two supports:

$$M_{y.Sd} = \frac{Q_{z.sd} \times l^2}{8} = \frac{(1.5.W) \times l^2}{8} = \frac{(1.5 \times 1.297) \times 5.5^2}{8} = 7.36 \text{ KN.m}$$

- beam on three supports:

$$M_{z.Sd} = \frac{Q_{y.sd} \times (l/2)^2}{8} = \frac{(1.35.G) \times (l/2)^2}{8} = \frac{(1.35 \times 0.4) \times (5.5/2)^2}{8} = 0.51 \text{ KN.m}$$

by trial and error, we choose L' **UPN 140**.

III.2.2.1.4 verification of l' UPN 140 in section:

III.2.2.1.4.1 ultimate limit state verification:

- **flexion verification:**

Calculation in plasticity: (Sections of class 1 and 2)

$$\left(\frac{M_{y.sd}}{M_{pl.y.Rd}}\right)^{\alpha} + \left(\frac{M_{z.sd}}{M_{pl.z.Rd}}\right)^{\beta} \leq 1.0$$

by trial and error, we choose the following profile UPN140.

- **See page 18**

The section is class 1

Note:

Rolled sections with gauges less than or equal to UPN 140, are generally of a class 1 section.

- geometric characteristics of l' UPN 140:

$Wel.y = 86.4 \text{ cm}^3$; $Wel.z = 14.8 \text{ cm}^3$

$Wpl.y = 103 \text{ cm}^3$; $Wpl.z = 28.3 \text{ cm}^3$

$M_{pl.y.Rd} = 2575 \text{ dan.m}$; $M_{pl.z.Rd} = 707.5 \text{ dan.m}$

$$\left(\frac{M_{y.sd}}{M_{pl.y.Rd}}\right)^{\alpha} + \left(\frac{M_{z.sd}}{M_{pl.z.Rd}}\right)^{\beta} = \left(\frac{7.36}{25.75}\right)^{2} + \left(\frac{0.51}{7.075}\right)^{1} = 0.15 \leq 1.0 \ldots\ldots\ldots\ldots\ldots\ldots \text{Verified.}$$

- **Shear verification:**

UPN 140: $A_{vz} = 10.4\ cm^2$; $A_{vy} = 12\ cm^2$

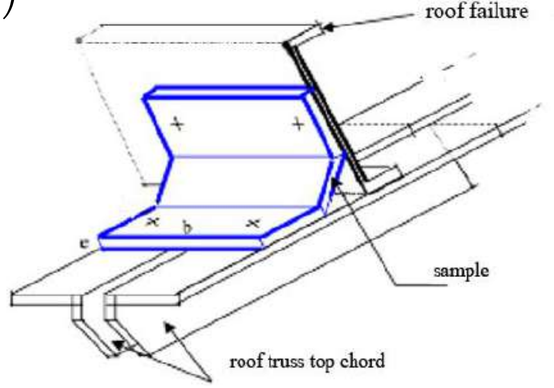


Figure III.13: representation of the roof failure

$$V_{z.sd} = \frac{1.5\ W \times l}{2} = \frac{1.5 \times 1.297 \times 5.5}{2} = 5.35 \text{ KN}$$

$V_{y.sd} = 0.625 \times 1.35.\, G \times (l/2) = 0.625 \times 1.35 \times 0.4 \times (5.5/2) = 0.93 \text{ KN}$

$$V_{pl.z.Rd} = \frac{A_{vz} \times \left(f_y/\sqrt{3}\right)}{\gamma_{M0}} = \frac{10.4 \times \left(2750/\sqrt{3}\right)}{1.1} = 150.11 \text{ KN}$$

$$V_{pl.y.Rd} = \frac{A_{vy} \times \left(f_y/\sqrt{3}\right)}{\gamma_{M0}} = \frac{12 \times \left(2750/\sqrt{3}\right)}{1.1} = 173.21 \text{ KN}$$

$V_{z.sd} = 5.35 \text{ KN} \leq V_{pl.z.Rd} = 150.11 \text{ KN}$ … … … … … … … … OK

$V_{y.sd} = 0.93 \text{ KN} \leq V_{pl.y.Rd} = 173.21 \text{ KN}$ … … … … … … … … OK

Note:

In most cases the shear verification is verified for rolled sections as soon as the bending moment verification is satisfied.

- **Verification of the spill element:**

there is no risk of the beam spilling as long as the compressed sole is supported laterally over its entire length.

III.2.2.1.4.2 Verification at SLS:

- **Flexion verification:**

The calculation of the deflection is made by the combination of loads and overloads of services (unweighted). $Q = G + W$

Verification condition**:** $f \leq f_{ad}$ with $f_{ad} = \frac{l}{200}$

- Vertical arrow (according to zz'): on two support

$$f_{ad} = \frac{l}{200} = \frac{550}{200} = 2.75\ cm$$

$$f_z = \frac{5}{384} \times \frac{W \times l^4}{E \times I_y}$$

$$f_z = \frac{5}{384} \times \frac{1.297 \times (550)^4}{2.1 \times 10^5 \times 605} = 1.22\ cm < f_{ad}$$ … … … … … … … … … … … … OK

- Lateral arrow (according to yy'): on three supports

$$f_{ad} = \frac{l/2}{200} = \frac{275}{200} = 1.375\ cm$$

$$f_y = \frac{2.05}{384} \times \frac{G \times (l/2)^4}{E \times I_z}$$

$$f_y = \frac{2.05}{384} \times \frac{0.4 \times (275)^4}{2.1 \times 10^5 \times 62.7} \approx 0.1 \text{ cm} < f_{ad}$$ … … … … … … … … … … … … … OK

III.2.2.2 pinion verification:

the gable runner is subjected to a negative pressure wind of:

$W = -0.569$ KN/m2

$W = 0.569 \times 1.2 = 0.68$ KN/m

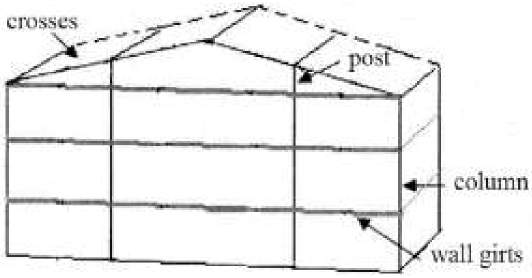


Figure III.14: representation of the pinion.

III.2.2.2.1 ultimate limit state verification:

- **flexion verification:**

l' **UPN 140** of the class 1:

$$\left(\frac{M_{y.sd}}{M_{pl.y.Rd}}\right)^{\alpha} + \left(\frac{M_{z.sd}}{M_{pl.z.Rd}}\right)^{\beta} \leq 1.0$$

$M_{pl.y.Rd} = 2575$ dan.m ; $M_{pl.z.Rd} = 707.5$ dan.m

- beam on two supports:

$$\mathrm{M_{y.Sd}} = \frac{\mathrm{Q_{z.sd}} \times l^2}{8} = \frac{(1.5.W)\times l^2}{8} = \frac{(1.5\times 0.68)\times 5.5^2}{8} = 3.86 \text{ KN.m}$$

- beam on three supports:

$$\mathrm{M_{z.Sd}} = \frac{\mathrm{Q_{y.sd}} \times (\mathrm{l}/2)^2}{8} = \frac{(1.35.\mathrm{G})\times(\mathrm{l}/2)^2}{8} = \frac{(1.35\times 0.4)\times(5.5/2)^2}{8} = 0.51 \text{ KN.m}$$

$$\left(\frac{M_{y.sd}}{M_{pl.y.Rd}}\right)^{\alpha} + \left(\frac{M_{z.sd}}{M_{pl.z.Rd}}\right)^{\beta} = \left(\frac{3.86}{25.75}\right)^{2} + \left(\frac{0.51}{7.075}\right)^{1} = 0.1 \leq 1.0 \ldots\ldots\ldots\ldots\ldots.. \text{Verified.}$$

- **Shear verification:**

UPN 140: $A_{vz} = 10.4\ cm^2$; $A_{vy} = 12\ cm^2$

$$V_{z.sd} = \frac{1.5\ \mathrm{W}\ \times \mathrm{l}}{2} = \frac{1.5 \times\ 1.297 \times 4}{2} = 3.89 \text{ KN}$$

$$\mathrm{V_{y.sd}} = 0.625 \times 1.35.\mathrm{G} \times (\mathrm{l}/2) = 0.625 \times 1.35 \times 0.4 \times (4/2) = 0.675 \text{ KN}$$

$$V_{pl.z.Rd} = \frac{A_{vz} \times \left(f_y/\sqrt{3}\right)}{\gamma_{M0}} = \frac{10.4 \times \left(2750/\sqrt{3}\right)}{1.1} = 15011.12 \text{ daN} = 150.11 \text{ KN}$$

$$V_{pl.y.Rd} = \frac{A_{vy} \times \left(f_y/\sqrt{3}\right)}{\gamma_{M0}} = \frac{12 \times \left(2750/\sqrt{3}\right)}{1.1} = 17320.51 \text{ daN} = 173.21 \text{ KN}$$

$V_{z.sd} = 3.89 \text{ KN} \leq V_{pl.z.Rd} = 150.11$ KN … … … … … … … … OK

$V_{y.sd} = 0.675 \text{ KN} \leq V_{pl.y.Rd} = 173.21$ KN … … … … … … … … OK

- **verification of the spill element:**

the sole, compressed under the action of the negative pressure wind, is liable to discharge as long as it is free over its entire length.

the discharge verification formula is given as follows: $\frac{M_{y.sd}}{M_{b.Rd}}+\frac{M_{z.sd}}{M_{plz.Rd}} \leq 1.0$

- **Calculation of spill resistance moment:**

$M_{b.Rd} = \chi_{LT} \times \beta_w \times \frac{W_{pl.y} \times f_y}{\gamma_{M1}}$

$\beta_w = 1.0$ for class 1 and class 2 sections.

The reduced slenderness $\bar{\lambda}_{LT}$ is determined by the following formula: (Annex F to the Eurocode, §F.2).

$$\bar{\lambda}_{LT} = \left[\frac{\beta_w \times W_{pl.y} \times f_y}{M_{cr}}\right]^{0.5} = \left[\frac{\lambda_{LT}}{\lambda_1}\right] \times [\beta_w]^{0.5}$$

$\lambda_1 = 86.80$

For beams with constant section and doubly symmetrical (rolled I and H section) the slenderness $\lambda_{LT}$is:

$$\lambda_{LT} = \frac{L/i_z}{C_1^{0.5}\left[1+\frac{1}{20}\left(\frac{L/i_z}{h/t_f}\right)^2\right]^{0.25}}$$

UPN140: $i_z = 1.75cm$; $h = 14cm$; $t_f = 1cm$; $C1 = 1.132$; $L = 200$ mm

$I_y = 605\ cm^4$; $I_z = 62.7\ cm^4$

$$\lambda_{LT} = \frac{200/1.75}{1.132^{\ 0.5}\left[1+\frac{1}{20}\left(\frac{200/1.75}{14/1}\right)^2\right]^{0.25}} = 74.46$$

$$\bar{\lambda}_{LT} = \left[\frac{\lambda_{LT}}{\lambda_1}\right] \times [\beta_w]^{0.5} = \frac{74.46}{86.80} = 0.86$$

$$\varphi_{LT} = 0.5\left[1+\alpha_{LT}(\bar{\lambda}_{LT}-0.2)+\bar{\lambda}_{LT}^2\right] = 0.5[1+0.49(0.86-0.2)+0.86^2] = 1.03$$

$$\chi_{LT} = \frac{1}{\varphi_{LT}+\left[\varphi_{LT}^2-\bar{\lambda}_{LT}^2\right]^{0.5}} = \frac{1}{1.03+[1.03^2-0.86^2]^{0.5}} = 0.63$$

$$M_{b.Rd} = \chi_{LT} \times M_{pl.y.Rd} = 0.63 \times 25.75 = 16.22 KN.m$$

$$\frac{M_{y.sd}}{M_{b.Rd}}+\frac{M_{z.sd}}{M_{plz.Rd}} = \frac{3.86}{16.22}+\frac{0.51}{7.075} = 0.31 \leq 1.0$$

III.2.2.2.2 verification at SLS

- **flexion verification:**

- Vertical arrow (according to zz'): on two support

$$f_{ad} = \frac{l}{200} = \frac{400}{200} = 2\ cm$$

$$f_z = \frac{5}{384} \times \frac{W \times l^4}{E \times I_y}$$

$$f_z = \frac{5}{384} \times \frac{0.68 \times (400)^4}{2.1 \times 10^5 \times 605} = 0.2 \text{ cm} < f_{ad} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{ OK}$$

- Lateral arrow (according to yy'): on three supports

$$f_{ad} = \frac{l/2}{200} = \frac{200}{200} = 1\ cm$$

$$f_y = \frac{2.05}{384} \times \frac{G \times (l/2)^4}{E \times I_z}$$

$$f_y = \frac{2.05}{384} \times \frac{0.4 \times (200)^4}{2.1 \times 10^5 \times 62.7} = 0.03 \text{ cm} < f_{ad} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{OK}$$

Conclusion:

The profile chooses UPN 140 suitable for purlins.

III.3 post:

III.3.1 Introduction:

The posts are elements made of rolled sections and intended to stiffen the cladding, having the role of transmitting the various horizontal forces (due to the wind). The posts are arranged vertically on the gable with different heights, the intermediate post is placed resting on the two e1dge posts, the latter are subjected to compound bending in the forces are expressed after:

-Normal force produced by the self-weight of the post, the cladding smooth and the cladding.

-Bending force produced by the action of wind on the pinion.

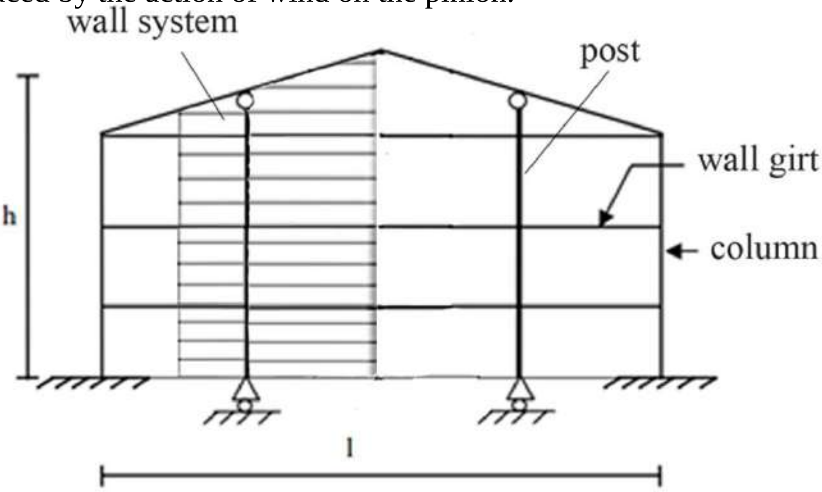


Figure III.15: representation of the post.

III.3.2 Determination of stresses:

The Post works on bending under the action of the force of the wind coming from the cladding and the rails, and on compression under the effect of its own weight, the weight of the cladding and the rails which is associated with it, and therefore works in compound flexion.

III.3.3 Evaluation of loads and overloads:

III.3.3.1 permanent loads (G): (concentrated load)

G = self-weight of the post + self-weight of the rails + self-weight of the cladding.

self-weight of the post ……………………………………………….24.7 Kg/m

self-weight of the beams ………………………………...…..…….16 Kg/m

Self-weight of the cladding (TL 80) with Accessory……… 20 Kg/m

Self-weight of the post: (to be determined)

weight of the post: $24.7 \times 5.1 = 125.97$ daN $= 1.26$ KN

weight of the beams: $16 \times (3 + (4 \times 2)) = 176$ daN $= 1.76$ KN weight of the cladding (TL 80) with Accessory: $20 \times 5.1 \times 3.5 = 3.57$ KN

$G = 1.26 + 1.76 + 3.57 = 6.59$ KN

III.3.3.2 Climatic overloads V: (horizontal along the plane of the core)

wind: ……………………………………………………………………. ….. -0.569 KN/m^2

$W = 0.569 \times 3.5 = 1.99$ KN/m

III.3.4 Sizing of the post:

III.3.4.1 Under the arrow condition:

Verification of deflection is performed under loads (unweighted).

$$f_z = \frac{5}{384} \times \frac{W_n \times l^4}{E \times I_y} \leq f_{ad} = \frac{l}{200}$$

$l = 5$ m :length of the most heavily loaded post (middle post).

$$I_y \geq \frac{1000}{384} \times \frac{W.l^3}{E} = \frac{1000 \times 1.99 \times 500^3}{384 \times 2.1 \times 10^6} = 327.35\ cm^4$$

The section of the profile is chosen from the tables having at least the value **I_y** greater than or equal to the value found.

What corresponds to a profile **HEA 140** ($I_y = 1033$ cm^4)

III.3.4.2 Geometric characteristics of HEA 140:

$h = 133\ mm$; $b = 140 mm$; $t_w = 5.5\ mm$; $t_f = 8.5\ mm$; $d = 92\ mm$;

$W_{el,y} = 155.4\ cm^3$; $W_{el.z} = 55.62\ cm^3$; $W_{pl.y} = 173.5$ cm^3 ; $W_{pl.z} = 84.85$ cm^3 ;

$I_y = 1033$ cm^4 ; $I_z = 389.3$ cm^4

III.3.5 verification the section at resistance:

III.3.5.1 Impact of shear force:

If: $V_{Sd} \leq 0.5 V_{pl.Rd} \rightarrow$ There is no interaction between the bending moment and the shearing force.

$Q_{z.sd} = 1.5V = 1.5 \times 1.99 = 2.99$ KN/m

$$V_{z.sd} = \frac{Q_{z.sd} \times l}{2} = \frac{2.99 \times 5}{2} = 7.48 \text{ KN}$$

$A_{vz} = 10.12\ cm^2$

$$V_{pl.z.Rd} = \frac{A_{vz} \times \left(f_y/\sqrt{3}\right)}{\gamma_{M0}} = \frac{10.12 \times \left(2750/\sqrt{3}\right)}{1.1} = 14606.96 \text{ daN} = 146.07 \text{ KN}$$

$$\frac{V_{z.sd}}{V_{pl.z.Rd}} = \frac{7.48}{146.07} = 0.05 < 0.5$$

$\rightarrow$ The effect of the shear force on the resistance moment can be neglected.

Note: In the case of simply supported beams, there is no effect of the shear force (zero value at mid-span) on the resistance moment.

III.3.5.2 Incidence of normal exertion:

If $N_{Sd} \leq Min\left(0.25\ \mathrm{N}_{pl.Rd}, 0.5A_w f_y/\gamma_{M0}\right)$: There is no interaction between moment of resistance and normal stress.

$N_{Sd} = 1.35G = 1.35 \times 6.59 = 8.897$ KN

$$N_{pl.Rd} = \frac{A.f_y}{\gamma_{M0}} = \frac{31.4 \times 2750}{1.1} = 78500 \text{ daN} = 785 \text{ KN}$$

$0.25\ \mathrm{N}_{pl.Rd} = 0.25 \times 785 = 196.25$ KN

$A_w = A - 2b.t_f = 31.4 - 2 \times 14 \times 0.85 = 7.5\ cm^2$

$0.5A_w f_y/\gamma_{M0} = 0.5 \times 7.5 \times 2750/1.1 = 9375$ daN $= 93.75$ KN

$N_{Sd} = 8.897$ KN $\leq Min(196.25\ , 93.75) = 93.75$ KN

→ The effect of normal stress on the resistance moment can be neglected.

- Verification the section at resistance: $\mathrm{M_{y.sd}} \leq \mathrm{M_{c.Rd}}$

Section class:

Sole class: (compressed sole)

$$\frac{\mathrm{c}}{\mathrm{t_f}} = \frac{b/2}{\mathrm{t_f}} = \frac{140/2}{8.5} = 8.24\ \leq 10\varepsilon = 9.2\ \rightarrow \text{Class 1}$$

Soul class: (compressed core)

$$\frac{\mathrm{c}}{\mathrm{t_f}} = \frac{d}{\mathrm{t_w}} = \frac{92}{5.5} = 16.73\ \leq 33\varepsilon = 30.36 \rightarrow \text{Class 1}$$

$$\varepsilon = \sqrt{\frac{235}{fy}} = \sqrt{\frac{235}{275}} = 0.92$$

The section is class 1.

$\mathrm{M_{c.Rd}} = \mathrm{M}_{ply.Rd} = \mathrm{Wpl.y} \times fy/\gamma_{M0} = 173.5 \times 10^{-2} \times 2750/1.1 = 4337.5$ daNm

$$\mathrm{M_{y.sd}} = \frac{\mathrm{Q_{z.sd}} \times l^2}{8} = \frac{2.99 \times 5^2}{8} = 9.34 \text{ KN}m$$

$\mathrm{M_{y.sd}} = 9.34$ KN.$m < \mathrm{M}_{ply.Rd} = 43.38$ KN.m … … … … … … … … … … … … … OK

III.3.6 Verification of the instability element:

The post is stressed by bending (due to the wind) and compression (due to its own weight, the weight of the cladding boxes and the rails). In any case, it does not support the roof (it is secured to the gantry by sliding support). He works in compound flexion.

The instability check is given by the following formulas:

- Compound bending with risk of buckling:

$$\frac{N_{Sd}}{\chi_{min}.N_{pl.Rd}}+\frac{k_y.\mathrm{M_{y.sd}}}{\mathrm{M}_{ply.Rd}}\leq 1.0$$

- Compound bending with risk of overturning:

$$\frac{N_{Sd}}{\chi_z.N_{pl.Rd}}+\frac{k_{LT}.\mathrm{M_{y.sd}}}{\chi_{LT}.\mathrm{M}_{ply.Rd}}\leq 1.0$$

III.3.7 Calculation of the minimum reduction coefficient for buckling χ_{min}:

$\chi_{min} = Min(\chi_y;\chi_z)$

- Buckling with respect to the strong yy axis (in the plane of the gantry)

$$\chi_y=\frac{1}{\varphi_y+\left[\varphi_y^2-\bar{\lambda}_y^2\right]^{0.5}}$$

$$\varphi_y=0.5\left[1+\alpha_y\left(\bar{\lambda}_y-0.2\right)+\bar{\lambda}_y^2\right]$$

$\bar{\lambda}_y = \left[\frac{\lambda_y}{\lambda_1}\right]\times[\beta_A]^{0.5}$; $\beta_A = 1.0$ for class 1 and class 2 sections.

$\lambda_1 = \sqrt{\frac{235}{f_y}}\times 93.9 = 86.81$: Eulerian slenderness

α: imperfection factor corresponding to the appropriate buckling curve, given by Table 5.5.1 of Eurocode 3.

$$\lambda_y=\frac{l_y}{i_y}=\frac{500}{5.73}=87.26$$

$$\bar{\lambda}_y=\left[\frac{\lambda_y}{\lambda_1}\right]\times[\beta_A]^{0.5}=\left[\frac{87.26}{86.81}\right]\times 1.0=1.01$$

Buckling curve: $h/b = 133/140 = 0.95 \leq 1.2$

Buckling axis $y-y$ → buckling curve b ; $\propto = 0.34$

$$\varphi_y = 0.5[1+0.34\times(1.01-0.2)+1.01^2]=1.15$$

$$\chi_y=\frac{1}{1.15+[1.15^2-1.01^2]^{0.5}}=0.59$$

- Buckling with respect to the weak zz axis (outside the gantry plane):

$$\chi_z = \frac{1}{\varphi_z + [\varphi_z^2 - \bar{\lambda}_z^2]^{0.5}}$$

$$\varphi_z = 0.5[1 + \alpha_z(\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2]$$

$$\lambda_z = \frac{l_z}{i_z} = \frac{120}{3.52} = 34.09\ ; \qquad l_z = 1.2\ m \text{ (between beam axis)}$$

$$\bar{\lambda}_z = \left[\frac{\lambda_z}{\lambda_1}\right] \times [\beta_A]^{0.5} = \left[\frac{34.09}{86.81}\right] \times 1.0 = 0.39$$

Buckling curve: $h/b = 133/140 = 0.95 \leq 1.2$

Buckling axis $Z - Z$ $\rightarrow$ buckling curve **c** ; $\propto = 0.49$

$$\varphi_z = 0.5[1 + 0.49 \times (0.39 - 0.2) + 0.39^2] = 0.62$$

$$\chi_z = \frac{1}{0.62 + [0.62^2 - 0.39^2]^{0.5}} = 0.91$$

$$\chi_{min} = Min(\chi_y; \chi_z) = Min(0.59; 0.91) = 0.59$$

III.3.8 Calculation of reduced slenderness vis-à-vis the lateral buckling $\bar{\lambda}_{LT}$:

$$\bar{\lambda}_{LT} = \left[\frac{\lambda_{LT}}{\lambda_1}\right] \times [\beta_w]^{0.5} \quad \text{with: } \lambda_1 = 86.80$$

$\bar{\lambda}_{LT}$: Slenderness of the element with respect to the lateral torsional buckling for rolled **I** or **H** sections.

$$\lambda_{LT} = \frac{L_z/i_z}{C_1^{0.5}\left[1 + \frac{1}{20}\left(\frac{L/i_z}{h/t_f}\right)^2\right]^{0.25}} = \frac{120/3.52}{1.132^{\ 0.5}\left[1 + \frac{1}{20}\left(\frac{120/3.52}{13.3/0.85}\right)^2\right]^{0.25}} = 30.38$$

Simply supported beam with an evenly distributed load: C1= 1.132

$$\bar{\lambda}_{LT} = \left[\frac{\lambda_{LT}}{\lambda_1}\right] \times [\beta_w]^{0.5} = \left[\frac{30.38}{86.80}\right] \times 1.0 = 0.35$$

$\bar{\lambda}_{LT} = 0.35 < 4$ → there is no risk of spillage.

$$\varphi_{LT} = 0.5[1 + \alpha_{LT}(\bar{\lambda}_{LT} - 0.2) + \bar{\lambda}_{LT}^2] = 0.5[1 + 0.34 \times (0.35 - 0.2) + 0.35^2] = 0.59$$

$$\chi_{LT} = \frac{1}{\varphi_{LT} + [\varphi_{LT}^2 - \bar{\lambda}_{LT}^2]^{0.5}} = \frac{1}{0.59 + [0.59^2 - 0.35^2]^{0.5}} = 0.94 < 1.0$$

$$\alpha_{LT} = 0.34$$

III.3.9 Calculation of the k coefficients:

$$\mu_y = \bar{\lambda}_y(2\beta_{My} - 4) + \frac{W_{ply} - W_{ely}}{W_{ely}} = 1.01 \times (2 \times 1.3 - 4) + \frac{173.5 - 155.4}{155.4} = -1.298$$

With: $\mu_y \leq 0.9$

$$k_y = 1 - \frac{\mu_y . N_{Sd}}{\chi_y . Af_y} = 1 - \frac{-1.298 \times 889.7}{0.59 \times 31.4 \times 2750} = 1.02 \quad with \quad k_y \leq 1.5$$

β_{My} is an equivalent uniform moment factor for buckling

Beam simply supported with an evenly distributed load: $\beta_{My} = 1.3$

$$\mu_{LT} = 0.15 \times \bar{\lambda}_z . \beta_{MLT} - 0.15 = 0.15 \times 0.39 \times 1.3 - 0.15 = -0.074 < 0.9$$

$$k_{LT} = 1 - \frac{\mu_{LT} . N_{Sd}}{\chi_z . Af_y} = 1 - \frac{-0.074 \times 889.7}{0.91 \times 31.4 \times 2750} = 1.0$$

β_{MLT} is an equivalent uniform moment factor for lateral torsional buckling

Beam simply supported with an evenly distributed load: $\beta_{MLT} = 1.3$

$N_{Sd} = 889.7$ daN

$$\mathrm{M_{y.Sd}} = \frac{(1.5.W) \times l^2}{8} = \frac{(1.5 \times 0.68) \times 5^2}{8} = 3.19 \text{ KN.m}$$

$$\mathrm{N_{pl.Rd}} = \frac{\mathrm{A} \times fy}{\gamma_{M1}} = \frac{31.4 \times 2750}{1.1} = 78500 \text{ dan}$$

$$\mathrm{M_{ply.Rd}} = \frac{\mathrm{W_{pl.y}} \times fy}{\gamma_{M1}} = \frac{173.5 \times 2750 \times 10^{-2}}{1.1} = 4337.5 \text{ dan.m}$$

III.3.10 Buckling verification:

$$\frac{N_{Sd}}{\chi_{min} . N_{pl.Rd}} + \frac{k_y . \mathrm{M_{y.sd}}}{\mathrm{M}_{ply.Rd}} \leq 1.0$$

$$\frac{889.7}{0.59 \times 78500} + \frac{1.02 \times 319}{4337.5} = 0.094 \leq 1 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{ok}$$

III.3.11 Spill verification:

$$\frac{N_{Sd}}{\chi_z . N_{pl.Rd}} + \frac{k_{LT} . \mathrm{M_{y.sd}}}{\chi_{LT} . \mathrm{M}_{ply.Rd}} \leq 1.0$$

$$\frac{889.7}{0.91 \times 78500} + \frac{1 \times 319}{4337.5} = 0.086 \leq 1 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{ok}$$

Conclusion:

HEA 140 is suitable as a post

III.4. Calculation of bracing:

III.4.1 Introduction:

Bracings are elements intended to ensure the stability of the framework by opposing the action of horizontal forces such as the wind, the braking of overhead cranes, seismic action, etc. They are intended to guarantee the correct path of horizontal forces towards the foundations. They are placed on the roof in the plane of the slopes: windward beam "horizontal bracing" and on the facades: stability bearing "vertical bracing", and must take up the horizontal forces applied both on the gable and on the long sides.

III.4.2 Calculation of the gable wind beam:

It will be calculated as a lattice beam resting on two supports and subjected to the upper horizontal reactions of the posts to which we add the driving force.

Note:

1. Compressed diagonals are not taken into account when determining the forces in the bars as long as they buckle at the slightest force.

2. The problem is reduced to an isostatic calculation and to determine these forces, one uses the method of the sections.

III.4.3 Evaluation of horizontal forces:

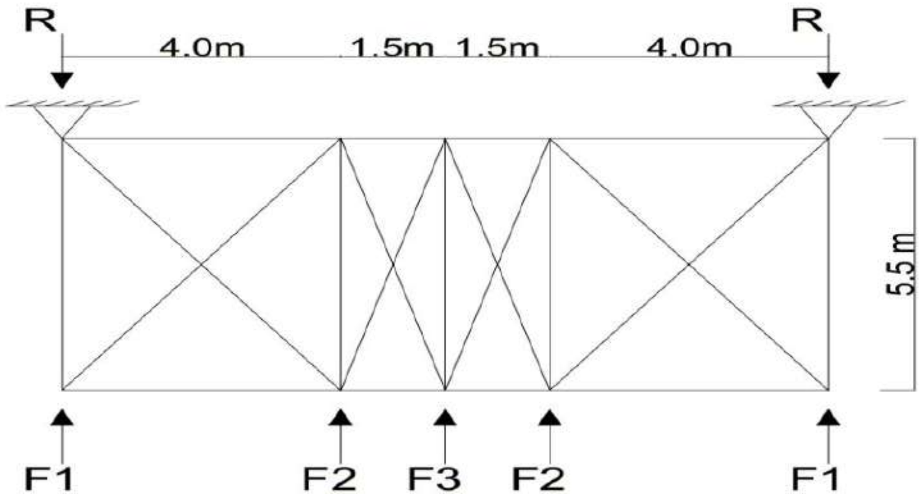


Figure III.16: Wind beam

$$F_i = 1.5 \times \left[(W \times S_i) + \frac{F_e}{n}\right] \quad ; \quad W = W(z) = q_p(z) \times \sum(C_{pe} - C_{pi})$$

From the wind study, the value of W = W (Z) is given below. (see CH2):

$$\sum(\mathrm{C_{pe}} - \mathrm{C_{pi}}) = \mathrm{C_{peD}} - \mathrm{C_{peE}} = 0.8 + 0.3 = 1.1$$

$$q_p(z) = q_{ref} \times C_{ex} = 500 \times 2.276 = 1138\ \mathrm{N/m^2}$$

$$W = W(z) = q_p(z) \times \sum(C_{pe} - C_{pi}) = 113.8 \times 1.1 = 125.18\ \mathrm{daN/m^2}$$

The driving force F_e is the friction force for the roof, and is given by: (see CH.2)

$$F_e = F_{fr} = \sum(q_p(z) \times C_{fr} \times S_{fr}) \qquad ; \ q_p(z) = 1138\ \mathrm{N/m^2}$$

$C_{fr} = 0.04$ coefficient of friction.

$S_{fr} = (21 \times 2 \times 4.20) = 176.4\ \mathrm{m^2}$ roof friction surface.

$F_{fr} = 1.138 \times 0.04 \times (21 \times 2 \times 4.20) = 8.03\ \mathrm{KN}$

III.4.3.1 Evaluation of the horizontal forces at the head of the posts:

Table III.1 : the results of Fi

	1	2	3
$H_i(m)$	4.20	5.00	5.20
$L_i(m)$	2.00	2.75	1.50
$S_i(m)$	4.20	6.88	3.90
$W \times S_i$(KN)	5.25	8.60	4.88
$\frac{F_e}{n}$ (KN)	1.61	1.61	1.61
F_i(KN)	10.29	15.32	9.74

III.4.3.2 Tractive effort in the diagonals:

We only work the taut diagonals and it is considered that the compressed diagonals do not take up any effort, because due to their great slenderness, they tend to buckle under low forces. Depending on the direction of the wind (wall D or wall E), one or the other of the diagonals is stretched.

The slope bracing is a truss girder assumed horizontal.

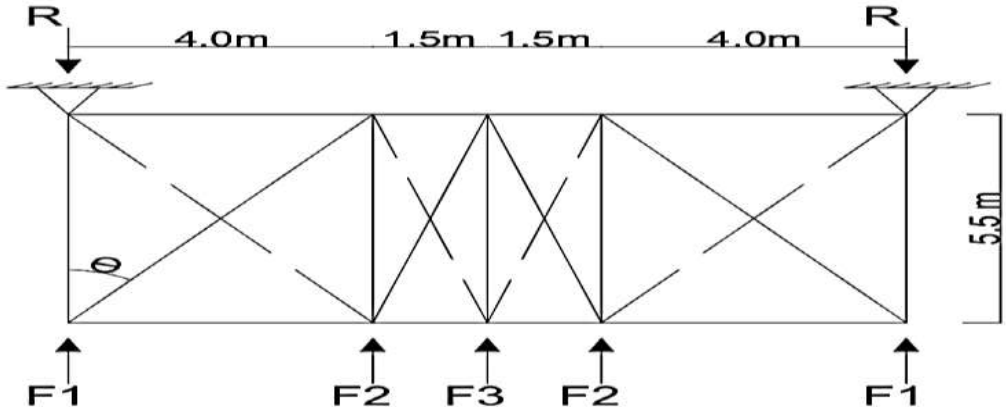


Figure III.17: Wind beam

By the method of cuts, it is established that the force $\mathrm{F_d}$ in the end diagonals (the most stressed) is given as follows:

$\mathrm{F_d}\cos\theta + \mathrm{F_1} = \mathrm{R}$

With:

$$R = \frac{2\mathrm{F_1} + 2\mathrm{F_2} + \mathrm{F_3}}{2} = \frac{2 \times 10.29 + 2 \times 15.32 + 9.74}{2} = 30.48\ \mathrm{KN}$$

$$\tan\theta = \frac{4}{5.5} \rightarrow \tan^{-1}\frac{4}{5.5} = 36.03^{\circ}$$

from where $\mathrm{F_d} = \frac{R - \mathrm{F_1}}{\cos\theta} = \frac{30.48 - 10.29}{\cos 36.03^{\circ}} = 24.97\ \mathrm{KN}$

$N_{Sd} = \mathrm{F_d} = 24.97\ \mathrm{KN}$

III.4.4 Diagonal of the section:

Calculation of the gross section A

$$N_{Sd} \leq \mathrm{N_{pl.Rd}} = \frac{\mathrm{A} \times fy}{\gamma_{M0}} \quad \rightarrow A \geq \frac{N_{Sd} \times \gamma_{M0}}{fy} = \frac{24.97 \times 1.1}{27.5} = 0.998\ cm^2$$

we adopt a cornier: L. $60 \times 60 \times 6$ ($\mathrm{A} = 691 mm^2$)

Net section : $A_{net} = A - e \times d_0 = 6.91 - 0.6 \times 1.3 = 6.13\ cm^2$

III.4.5 Section resistance verification:

$N_{Sd} \leq N_{u.Rd}$

$$N_{u.Rd} = \frac{\beta . A_{net} . f_u}{\gamma_{M2}} = \frac{0.7 \times 6.13 \times 36}{1.25} = 123.58\ \mathrm{KN}$$

$N_{Sd} = 24.97\ \mathrm{KN} \leq N_{u.Rd} = 123.58\ \mathrm{KN}$ ………………………………..OK

Conclusion:

we adopt a cornier: L.60 × 60 × 6.

III.4.6 Verification of the uprights of the wind beam

The upwind girder uprights are purlins that work in deflected bending under the action of vertical loads, and in addition to compression under (F), so the purlin must be checked at compound deflection. The verification formulas are as follows:

III.4.6.1 Deviated compound flexion (biaxial):

III.4.6.1.1 verification the section at resistance:

Class 1 and 2 section:

$$\left(\frac{M_{y.sd}}{M_{Ny.Rd}}\right)^{\alpha} + \left(\frac{M_{z.sd}}{M_{Nz.Rd}}\right)^{\beta} \leq 1.0$$

$\alpha = 2$ and $\beta = 5n \geq 1$.

$$M_{Ny.Rd} = M_{pl.y.Rd}\left[\frac{1-n}{1-0.5a}\right] \quad ; \quad M_{Nz.Rd} = M_{pl.z.Rd}\left[1-\left(\frac{n-a}{1-a}\right)^2\right]$$

$$A_w = A - 2b.t_f \quad (soul\ area) \quad ; \quad a = min(A_w/A; 0.5)$$

$$n = \frac{N_{Sd}}{N_{pl.Rd}} \quad ; N_{pl.Rd} = \frac{A.f_y}{\gamma_{M0}} \quad ; \mathrm{M_{ply.Rd}} = \frac{\mathrm{W_{pl.y}} \times fy}{\gamma_{M0}} \quad ; \mathrm{M_{plz.Rd}} = \frac{\mathrm{W_{pl.z}} \times fy}{\gamma_{M0}}$$

III.4.6.1.2 Loads and overloads attributable to the intermediate failure:

- Deflected bending: (see calculation of purlins)

$G = 0.91$ KN/m

$S = 0.31$ KN/m

- Compression: (see calculation of bracing)

$V = \mathrm{F_2} = 15.32$ KN

III.4.6.1.3 Load combination: (Two and more variable actions)

$1.35\ G + 1.35\ S + 1.35W$

$Q_{Sd} = 1.35\ G + 1.5\ S = 1.35 \times 0.91 + 1.5 \times 0.31 = 1.694$ KN/m

$N_{Sd} = 1.35\ V = 1.35 \times 15.32 = 20.68$ KN

$$Q_{z.Sd} = Q_{Sd}.\cos\alpha = 1.668\ \mathrm{KN/m} \quad ; \mathrm{M_{y.sd}} = \frac{\mathrm{Q_{z.sd}} \times l^2}{8} = 5.213\ \mathrm{KN.m}$$

$$Q_{y.sd} = Q_{sd}.\sin\alpha = 0.294\ \mathrm{KN/m} \quad ; \mathrm{M_{z.sd}} = \frac{\mathrm{Q_{y.sd}} \times (l/2)^2}{8} = 0.229\ \mathrm{KN.m}$$

- **Geometric characteristics of the UPN140:**

$W_{el,y} = 86.4\ cm^3$; $W_{el.z} = 14.8\ cm^3$

$W_{pl.y} = 103\ \mathrm{cm^3}$; $W_{pl.z} = 28.3\ \mathrm{cm^3}$

$$M_{ply.Rd} = \frac{W_{pl.y} \times f_y}{\gamma_{M1}} = \frac{103 \times 2750 \times 10^{-2}}{1.1} = 2575\ \mathrm{dan.m}$$

$M_{plz.Rd} = \frac{W_{pl.z} \times f_y}{\gamma_{M1}} = \frac{28.3 \times 2750 \times 10^{-2}}{1.1} = 707.5 \text{ dan.m}$

$N_{pl.Rd} = \frac{A.f_y}{\gamma_{M0}} = \frac{20.4 \times 2750}{1.1} = 510 \text{ KN}$

- **Incidence of shear force:**

If: $V_{Sd} \leq 0.5V_{pl.Rd} \rightarrow$ There is no interaction between the bending moment and the shearing force.

Note:

At mid-span the value of the bending moment is maximum and the value of the shearing force is zero, so there is no interaction between the bending moment and the shearing force.

- **Incidence of normal exertion:**

If $N_{Sd} \leq Min\left(0.25 \text{ N}_{pl.Rd}, 0.5A_w f_y/\gamma_{M0}\right)$: There is no interaction between moment of resistance and normal stress.

$0.25\text{N}_{pl.Rd} = 0.25 \times 510 = 127.5 \text{ KN}$

$A_w = A - 2b.t_f = 20.4 - 2 \times 6.0 \times 1.0 = 8.4\ cm^2$

$0.5A_w f_y/\gamma_{M0} = 0.5 \times 8.4 \times 2750/1.1 = 105 \text{ KN}$

$N_{Sd} = 20.68 \text{ KN} \leq Min(127.5 \text{ KN}\ ;\ 105 \text{ KN}) = 105 \text{ KN}$

→ The effect of normal stress on the resistance moment can be neglected.

No reduction in plastic moments of resistance:

$M_{Ny.Rd} = M_{ply.Rd} \quad ; \quad M_{Nz.Rd} = M_{plz.Rd}$

The verification formula is as follows:

$$\left(\frac{M_{y.sd}}{M_{pl.y.Rd}}\right)^{\alpha} + \left(\frac{M_{z.sd}}{M_{pl.z.Rd}}\right)^{\beta} \leq 1.0$$

Where α = 2 and $\beta = 1$

$$\left(\frac{5.213}{25.75}\right)^{2} + \left(\frac{0.229}{7.075}\right)^{1} = 0.1 \leq 1.0 \ldots\ldots\ldots\ldots\ldots.. \text{Verified}$$

III.4.6.2 Verification of the element with instabilities (Spillage and Buckling):

Dump = Lateral buckling + Rotation of the cross section.

Upper sole:

The upper sole which is compressed under the action of downward vertical loads is susceptible to dumping. Since it is fixed to the roof, there is therefore no risk of spillage.

Bottom sole:

The lower sole which is compressed under the action of the uplift wind is liable to discharge as long as it is free throughout its span.

III .4.6.2.1 Combination at the ULS:

$G + 1.5W$

$Q_{z.Sd} = G.\cos\alpha - 1.5W$; $Q_{y.sd} = 1.35G.\sin\alpha$; $N_{Sd} = 1.5$

With:

$G = 0.91$ KN/m Permanent charge.

$W = -0.625$ KN/m Uplift wind.

$V = 15.32$ KN Wind compressive force returning to the intermediate purlin.

- **Bending load: see calculation of purlins**

$$Q_{z.Sd} = G.\cos\alpha - 1.5W = -0.041\text{KN/m} \quad ; \; \mathrm{M_{y.sd}} = \frac{\mathrm{Q_{z.sd}} \times l^2}{8} = 0.155 \text{ KN.m}$$

$$Q_{y.sd} = 1.35G.\sin\alpha = 0.213\text{KN/m} \quad ; \; \mathrm{M_{z.sd}} = \frac{\mathrm{Q_{y.sd}} \times (l/2)^2}{8} = 0.201 \text{ KN.m}$$

- **Compressive load: (see calculation of bracing)**

$N_{Sd} = 1.5\,V = 1.5 \times 15.32 = 22.98$ KN

III .4.6.2.2 The instability verification formulas are as follows:

- compound deviated bending with risk of buckling:

$$\frac{N_{Sd}}{\chi_{min}.N_{pl.Rd}} + \frac{k_y.\mathrm{M_{y.sd}}}{\mathrm{M}_{ply.Rd}} + \frac{k_z.\mathrm{M_{z.sd}}}{\mathrm{M}_{plz.Rd}} \leq 1.0$$

- compound deflection with risk of buckling:

$$\frac{N_{Sd}}{\chi_z.N_{pl.Rd}} + \frac{k_{LT}.\mathrm{M_{y.sd}}}{\chi_{LT}.\mathrm{M}_{ply.Rd}} + \frac{k_z.\mathrm{M_{z.sd}}}{\mathrm{M}_{plz.Rd}} \leq 1.0$$

Where

$$N_{pl.Rd} = \frac{A.f_y}{\gamma_{M0}} \quad ; \quad M_{ply.Rd} = \frac{W_{pl.y} \times f_y}{\gamma_{M1}} \quad ; \quad M_{plz.Rd} = \frac{W_{pl.z} \times f_y}{\gamma_{M1}}$$

III .4.6.2.3 Calculation of the reduction coefficient $\chi_z; \chi_y$:

- Buckling with respect to the strong yy axis

$$\chi_y = \frac{1}{\varphi_y + \left[\varphi_y^2 - \bar{\lambda}_y^2\right]^{0.5}} \quad ; \quad \varphi_y = 0.5\left[1 + \alpha_y\left(\bar{\lambda}_y - 0.2\right) + \bar{\lambda}_y^2\right]$$

$$\bar{\lambda}_y = \left[\frac{\lambda_y}{\lambda_1}\right] \times [\beta_A]^{0.5}$$

α: imperfection factor corresponding to the appropriate buckling curve, given by Table 5.5.1 of Eurocode 3.

- Buckling with respect to the weak zz axis

$$\chi_z = \frac{1}{\varphi_z + \left[\varphi_z^2 - \bar{\lambda}_z^2\right]^{0.5}} \quad ; \quad \varphi_z = 0.5\left[1 + \alpha_z\left(\bar{\lambda}_z - 0.2\right) + \bar{\lambda}_z^2\right]$$

$\bar{\lambda}_z = \left[\frac{\lambda_z}{\lambda_1}\right] \times [\beta_A]^{0.5}$; $\beta_A = 1.0$ for class 1 and class 2 sections.

$$\lambda_1 = \sqrt{\frac{235}{f_y}} \times 93.9 = 86.81$$

$$\lambda_y = \frac{l_y}{i_y} = \frac{500}{5.45} = 91.74 \quad ; \quad \lambda_z = \frac{l_z}{i_z} = \frac{250}{1.75} = 142.857$$

$$\bar{\lambda}_y = \left[\frac{\lambda_y}{\lambda_1}\right] \times [\beta_A]^{0.5} = 1.06 \quad ; \quad \bar{\lambda}_z = \left[\frac{\lambda_z}{\lambda_1}\right] \times [\beta_A]^{0.5} = 1.65$$

Buckling curve: $h/b = 140/60 = 2.33 > 1.2$

Buckling axis $y - y$ $\rightarrow$ buckling curve **a**; $\propto = 0.21$

Buckling axis $Z - Z$ $\rightarrow$ buckling curve **b**; $\propto = 0.34$

$$\varphi_y = 0.5[1 + 0.21 \times (1.16 - 0.2) + 1.16^2] = 1.27$$

$$\chi_y = \frac{1}{1.27 + [1.27^2 - 1.16^2]^{0.5}} = 0.56$$

$$\varphi_z = 0.5[1 + 0.34 \times (1.81 - 0.2) + 1.81^2] = 2.41$$

$$\chi_z = \frac{1}{2.41 + [2.41^2 - 1.81^2]^{0.5}} = 0.25$$

$$\chi_{min} = Min\left(\chi_y; \chi_z\right) = Min(0.56; 0.25) = 0.25$$

III .4.6.2.4 Calculation of the reduction coefficient for the discharge χ_{LT}:

UPN140: $i_z = 1.75\ cm$; $h = 14\ cm$; $t_f = 1\ cm$; $L = 250\ cm$

$$\lambda_{LT} = \frac{L_z/i_z}{C_1^{0.5}\left[1 + \frac{1}{20}\left(\frac{L/i_z}{h/t_f}\right)^2\right]^{0.25}} = \frac{250/1.75}{1.132^{0.5}\left[1 + \frac{1}{20}\left(\frac{250/1.75}{14/1}\right)^2\right]^{0.25}} = 85.07$$

$$\bar{\lambda}_{LT} = \left[\frac{\lambda_{LT}}{\lambda_1}\right] \times [\beta_w]^{0.5} = \left[\frac{85.07}{86.81}\right] \times 1.0 = 0.979 > 0.4$$

$$\varphi_{LT} = 0.5\left[1 + \alpha_{LT}\left(\bar{\lambda}_{LT} - 0.2\right) + \bar{\lambda}_{LT}^2\right] = 0.5[1 + 0.21 \times (0.979 - 0.2) + 0.979^2] = 1.06$$

$$\chi_{LT} = \frac{1}{\varphi_{LT} + \left[\varphi_{LT}^2 - \bar{\lambda}_{LT}^2\right]^{0.5}} = \frac{1}{1.06 + [1.06^2 - 0.979^2]^{0.5}} = 0.68 < 1.0$$

III .4.6.2.5 Calculation of coefficients *k*:

$\mu_{LT} = 0.15 \times \bar{\lambda}_z . \beta_{MLT} - 0.15$ and $\mu_{LT} \leq 0.9$

$$k_{LT} = 1 - \frac{\mu_{LT}.N_{Sd}}{\chi_z.Af_y} \quad \text{and} \quad k_{LT} \leq 1.0$$

$\beta_{MLT} = 1.3$ is an equivalent uniform moment factor for lateral torsional buckling

$$\mu_{LT} = 0.15 \times \bar{\lambda}_z.\beta_{MLT} - 0.15 = 0.15 \times 1.65 \times 1.3 - 0.15 = 0.172$$

$$k_{LT} = 1 - \frac{\mu_{LT}.N_{Sd}}{\chi_z.Af_y} = 1 - \frac{0.172 \times 22.98}{0.25 \times 20.4 \times 27.5} = 0.972$$

$$\mu_y = \bar{\lambda}_y(2\beta_{My} - 4) + \frac{W_{ply} - W_{ely}}{W_{ely}} \quad \text{with} \quad \mu_y \leq 0.9$$

$$k_y = 1 - \frac{\mu_y.N_{Sd}}{\chi_y.Af_y} \quad \text{with} \quad k_y \leq 1.5$$

$$\mu_y = 1.06 \times (2 \times 1.3 - 4) + \frac{103 - 86.4}{86.4} = -1.292 \leq 0.9$$

$$k_y = 1 - \frac{-1.292 \times 22.98}{0.56 \times 20.4 \times 27.5} = 1.095 \leq 1.5$$

$$\mu_z = \bar{\lambda}_z(2\beta_{Mz} - 4) + \frac{W_{plz} - W_{elz}}{W_{elz}} \quad \text{with} \quad \mu_z \leq 0.9$$

$$k_z = 1 - \frac{\mu_z.N_{Sd}}{\chi_z.Af_y} \quad \text{with} \quad k_z \leq 1.5$$

$$\mu_z = 1.65 \times (2 \times 1.3 - 4) + \frac{28.3 - 14.8}{14.8} = -1.398 \leq 0.9$$

$$k_z = 1 - \frac{-1.398 \times 22.98}{0.25 \times 20.4 \times 27.5} = 1.229 \leq 1.5$$

III .4.6.2.6 Buckling verification:

$$\frac{N_{Sd}}{\chi_{min}.N_{pl.Rd}} + \frac{k_y.\mathrm{M}_{\mathrm{y.sd}}}{\mathrm{M}_{ply.Rd}} + \frac{k_z.\mathrm{M}_{\mathrm{z.sd}}}{\mathrm{M}_{plz.Rd}} \leq 1.0$$

$$\frac{22.98}{0.25 \times 510} + \frac{1.095 \times 0.155}{25.75} + \frac{1.229 \times 0.201}{7.075} = 0.222 < 1 \ldots\ldots\ldots..\text{Verified}$$

III .4.6.2.7 Spill verification:

$$\frac{N_{Sd}}{\chi_z.N_{pl.Rd}} + \frac{k_{LT}.\mathrm{M}_{\mathrm{y.sd}}}{\chi_{LT}.\mathrm{M}_{ply.Rd}} + \frac{k_z.\mathrm{M}_{\mathrm{z.sd}}}{\mathrm{M}_{plz.Rd}} \leq 1.0$$

$$\frac{22.98}{0.25 \times 510} + \frac{0.972 \times 0.155}{0.68 \times 25.75} + \frac{1.229 \times 0.201}{7.075} = 0.224 < 1 \ldots\ldots\ldots..\text{Verified}$$

conclusion:

The selected profile **UPN 140** is suitable as a roof purlin.

III .4.7 Calculation of the long section of stability:

The stability brackets must take up the forces of the wind on the gables transmitted by the bracing of the slopes (upwind beam). We only work the taut diagonals, as in the case of the windward beam.

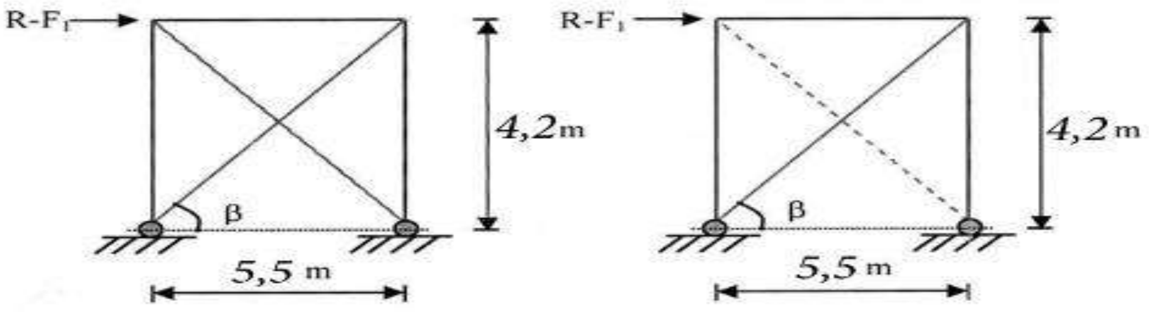


Figure III.18: Long section of the bracing

- **By the cut method:**

III .4.7.1 Tractive effort in the stretched diagonal:

$$\tan\beta = \frac{4.20}{5} = 0.84$$

$$\beta = \tan^{-1} 0.84 = 40.03^{\circ}$$

$$N = \frac{R - F_1}{\cos\beta} = \frac{30.48 - 10.29}{\cos 40.03^{\circ}} = 26.367 \text{ KN}$$

III .4.7.2 Diagonal of the section:

Calculation of the gross section A

$$N_{Sd} \leq \mathrm{N_{pl.Rd}} = \frac{\mathrm{A} \times fy}{\gamma_{M0}} \quad \Rightarrow \quad N_{Sd} = N = 26.367 \text{ KN}$$

$$A \geq \frac{N_{Sd} \times \gamma_{M0}}{fy} = \frac{26.367 \times 1.1}{27.5} = 1.05\ cm^2$$

III .4.7.3 we adopt a cornier: $2 \times \mathrm{L}.60 \times 60 \times 6$

Net section : $A_{net} = A - e \times d_0 = 6.91 - 0.6 \times 1.3 = 6.13\ cm^2$

III .4.7.4 Section resistance verification:

$$N_{Sd} \leq N_{u.Rd}$$

$$N_{u.Rd} = \frac{\beta . A_{net} . f_u}{\gamma_{M2}} = \frac{0.7 \times 6.13 \times 36}{1.25} = 123.58 \text{ KN}$$

$$N_{Sd} = 26.367 \text{ KN} \leq N_{u.Rd} = 123.58 \text{ KN} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{OK}$$

Conclusion:

we adopt a cornier: $2 \times \mathrm{L}.60 \times 60 \times 6$.

III .5 calculation of the eave strut:

The eave strut is considered to be a vertical bracing bar; therefore, it is subjected to a horizontal force and its own weight, from which the verification will be made in compound bending.

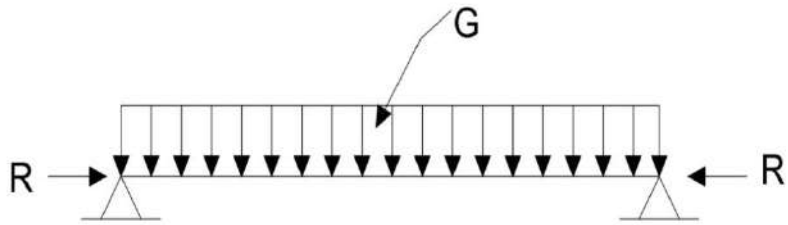


Fig III.19: statistical diagram of the eave strut

The intermediate longitudinal portal gantry beam receives two reactions of the beam to the gable wind, calculated previously, which are considered to be a compressive force with:

$N_{sd} = \mathrm{R} = 30.48\ \mathrm{KN}$

III .5.1 Pre-sizing:

The pre-sizing is done in simple compression:

$$N_{sd} \leq \mathrm{N_{pl.Rd}} = \frac{\mathrm{A} \times fy}{\gamma_{M0}} \quad \rightarrow\ A \geq \frac{N_{sd} \times \gamma_{M0}}{fy} = \frac{30.48 \times 1.1}{27.5} = 1.219\ cm^2$$

we adopt a cornier: **HEA 120** With $\mathrm{A} =\ 25.3\ mm^2$ and $G = 19.9\ \mathrm{Kg/m}$

III .5.2 Verification of the strut at buckling:

If $\lambda_{max} \geq 0.2$ must take into account the risk of buckling, and the verification to be done is as follows:

$$N_{sd} \leq \chi_{\mathrm{LT}} \times \beta_A \times \frac{A.f_y}{\gamma_{M1}} \quad with\ \ \bar{\lambda}_{max} = Max(\bar{\lambda}_y; \bar{\lambda}_z)\ \ ;\ \ \beta_A = 1\ Class\ 1\ section$$

$\mathrm{L_K}$: Is the buckling length of the strut beam with $\mathrm{L_K} = 5.50$.

$$\lambda_y = \frac{L_y}{i_y} = \frac{550}{4.89} = 112.47 \quad ; \quad \lambda_z = \frac{L_z}{i_z} = \frac{550}{3.02} = 182.12$$

$$\bar{\lambda}_y = \left[\frac{\lambda_y}{\lambda_1}\right] \times [\beta_\mathrm{A}]^{0.5} = 1.296 \quad ; \quad \bar{\lambda}_z = \left[\frac{\lambda_z}{\lambda_1}\right] \times [\beta_\mathrm{A}]^{0.5} = 2.098$$

$\bar{\lambda}_{max} = Max(1.296; 2.098) = 2.098 > 0.2$ So, there is the risk of buckling.

III .5.3 Calculation of χ_{LT} :

χ_{LT}: Reduction coefficient as a function of $\bar{\lambda}_{LT}$.

$\bar{\lambda}_{LT}$: Is the slenderness reduced vis-à-vis the spill.

$\bar{\lambda}_{LT} = \left[\frac{\lambda_{LT}}{\lambda_1}\right] \times [\beta_w]^{0.5}$; $\lambda_{LT} = \dfrac{L_z/i_z}{C_1^{0.5}\left[1+\frac{1}{20}\left(\frac{L/i_z}{h/t_f}\right)^2\right]^{0.25}}$

With: $\chi_{LT} = \dfrac{1}{\varphi_{LT} + [\varphi_{LT}^2 - \bar{\lambda}_{LT}^2]^{0.5}}$; OR $\varphi_{LT} = 0.5[1 + \alpha_{LT}(\bar{\lambda}_{LT} - 0.2) + \bar{\lambda}_{LT}^2]$

Therefore: $\lambda_{LT} = 98.37$; $\bar{\lambda}_{LT} = 1.133$; $\varphi_{LT} = 1.239$; $\chi_{LT} = 0.575$

Finally:

$$N_{sd} = 30.48 \text{ KN} \leq 0.575 \times 1 \times \frac{25.3 \times 27.5}{1.1} = 363.688 \text{ KN} \ldots\ldots\text{Verified}$$

III .5.4 Verification of compound bending

The verification to be done is as follows:

$$\frac{N_{sd}}{A.f_y/\gamma_{m0}} + \frac{M_{sd.y}}{M_{Pl.y}} \leq 1$$

$M_{sd.y}$: Bending moment around the y-y' axis:

$$M_{sd.y} = \frac{(1.35 \times G_{HEA120}) \times L^2}{8} = 1.02\ KN.m$$

Therefore:

$$\frac{30.48}{25.3 \times 27.5/1} + \frac{1.02}{29.88} = 0.078 \leq 1 \ldots\ldots\text{Verified}$$

We can see that the condition is checked so we opt for a **HEA 120** for the beam strut.

IV. Introduction:

Gantry cranes made up of posts and ties are generally the most used nowadays for reasons of simplicity in comparison with porticos (post-trusses). Their uses are however limited because of their ranges. ($l \leq 40$ml). For gantries with long spans, it is preferable for economic reasons to opt for truss girders (trusses). Rolled I or H sections are most commonly used as a structural steel post. The sleepers are generally made of rolled I-sections.

IV.1 Effect of vertical loads on a gantry:

IV.1.1 Permanent loads:

TL.80 sandwich purlin cladding weight with accessories……. 60 kg/m^2.

Estimated self-weight of the purlin (UPN 140) ……. $16\ \text{kg/m} \approx 8\ \text{kg/m}^2$

Eave strut (IPE 270) ………………………………$36.1\ \text{kg/m} \approx 18.05\ \text{kg/m}^2$

Total weight: $G = 60 + 8 + 18.05 = 86.05\ \text{kg/m}^2$

Between axis of the gantries is 5.5m: $G = 86.05 \times 5.5 = 473.275\ \text{kg/m}$

IV.1.2 Effect of sand: (see CH.3)

$$S = 0.25 \times 5.5 = 1.375\ KN/m$$

IV.1.3 Effect of the wind: (see CH.2)

Table IV.1: Values for Effect of the wind

Zone	Cpe	Cpi	Wzj[N/m2]	Wzj[N/m]
A	-1.05	0.27	-1502	-8261
B	-0.80	0.27	-1218	-6699
C	-0.50	0.27	-876	-4818
D	0.80	0.27	603	3316.5
E	-0.30	0.27	-649	-3569.5
F	-1.33	0.27	-1039	-5714.5
G	-1.00	0.27	-825	-4537.5
H	-0.45	0.27	-468	-2574
I	-0.50	0.27	-500	-2750
J	-0.40	0.27	-435	-2392.5

IV.1.4 Equivalent pressure coefficient:

The equivalent uniformly distributed wind load is reduced in the same way:

Left side: $\dfrac{4537.5 \times 1.04 + 2574 \times (5.5 - 1.04)}{5.5} = 2945.28\ \mathrm{kg/m}$

Right side: $\dfrac{2392.5 \times 1.04 + 2750 \times (5.5 - 1.04)}{5.5} = 2682.4\ \mathrm{kg/m}$

Given that the actions of the wind on the two sides are comparable, and for reasons of simplicity, one can admit a single equivalent value on the two sides.

IV.1.5 Equivalent wind load:

$$\frac{4537.5 \times 1.04}{11} + \frac{2574 \times 4.46}{11} + \frac{2392.5 \times 1.04}{11} + \frac{2750 \times 4.46}{11} = \frac{w \times 11}{11}$$

$w = 2813.84\ \mathrm{kg/m}$

Or : $w = \dfrac{2945.28 + 2682.4}{2} = 2813.84\ \mathrm{kg/m}$

IV.2 Calculation of internal forces:

We assume $I_2 \approx I_1$

$$k = \frac{stiffness\_crawling}{stiffness\_crutch} = \frac{I_2.h}{I_1.s} = \frac{h}{s} = \frac{4.20}{5.5/\cos 10} = 0.752$$

$$\varphi = \frac{f}{h} = \frac{1.0}{4.20} = 0.238$$

$$\Delta = K + 3 + 3\varphi + \varphi^2 = 0.752 + 3 + 3 \times 0.238 + 0.238^2 = 4.523$$

IV.2.1 Downward vertical loads: (Permanent loads and snow load)

Calculation under unit load: $q = 1.0 kg/m$

$$\beta = \frac{8 + 5\varphi}{4\Delta} = \frac{8 + 5 \times 0.238}{4 \times 4.523} = 0.507$$

$$\gamma = 1 - \beta.(1 + \varphi) = 1 - 0.507.(1 + 0.238) = 0.372$$

$$H_A = H_E = \beta \times \frac{q \times l^2}{8.h} = 0.507 \times \frac{1.0 \times 11^2}{8 \times 4.2} = 1.826\ \mathrm{Kg}$$

$$V_A = V_E = \frac{q \times l}{2} = \frac{1.0 \times 11}{2} = 5.5\ \mathrm{Kg}$$

$$\frac{q \times l^2}{8} = \frac{1.0 \times 11^2}{8} = 15.125 \text{ Kg. m}$$

$$M_B = M_D = -\beta \times \frac{q \times l^2}{8} = -0.507 \times 15.125 = -7.668 \text{ Kg. m}$$

$$M_C = \gamma \times \frac{q \times l^2}{8} = 0.372 \times 15.125 = 5.627 \text{ Kg. m}$$

IV.2.2 Vertical loads upwards: (Uplift wind)

Calculation under unit load: $q = 1.0 \text{ kg/m}$

$$H_A = H_E = \beta \times \frac{q \times l^2}{8.h} = 0.507 \times \frac{1.0 \times 11^2}{8 \times 4.2} = 1.826 \text{ Kg}$$

$$V_A = V_E = \frac{q \times l}{2} = \frac{1.0 \times 11}{2} = 5.5 \text{ Kg}$$

$$M_B = M_D = +\beta \times \frac{q \times l^2}{8} = +0.507 \times 15.125 = +7.668 \text{ Kg. m}$$

$$M_C = -\gamma \times \frac{q \times l^2}{8} = -0.372 \times 15.125 = -5.627 \text{ Kg. m}$$

IV.2.3 Horizontal wind: (pressure)

Unit charge: $q = 1.0 \text{ kg/m}$

$$H_E = \delta \times \frac{q.h}{2} \quad ; \quad H_A = q.h - H_E \quad ; V_A = -V_E = \frac{q.h^2}{2.l}$$

$$M_B = \beta \times \frac{q.h^2}{2} \quad ; \quad M_D = -\delta \times \frac{q.h^2}{2} \quad ; M_C = -\gamma \times \frac{q.h^2}{2}$$

$$\delta = \frac{5K + 12 + 6\varphi}{8\Delta} = \frac{5 \times 0.752 + 12 + 6 \times 0.238}{8 \times 4.523} = 0.475$$

$$\beta = 1 - \delta = 1 - 0.475 = 0.525$$

$$\gamma = \delta.(1 + \varphi) - \frac{1}{2} = 0.475.(1 + 0.238) - \frac{1}{2} = 0.09$$

$$H_E = \delta \times \frac{q.h}{2} = 0.475 \times \frac{1.0 \times 4.20}{2} = 0.998 \text{ Kg}$$

$$H_A = q.h - H_E = 1.0 \times 4.20 - 0.998 = 3.202 \text{ Kg}$$

$$V_A = -V_E = \frac{1.0 \times 4.20^2}{2 \times 11} = 0.802 \text{ Kg}$$

$$\frac{q \times h^2}{2} = \frac{1.0 \times 4.20^2}{2} = 8.82 \text{ Kg. m}$$

$$M_B = \beta \times \frac{q.h^2}{2} = 0.525 \times 8.82 = 4.631 \text{ Kg. m}$$

$$M_D = -\delta \times \frac{q.h^2}{2} = -0.475 \times 8.82 = -4.189 \text{ Kg. m}$$

$$M_C = -\gamma \times \frac{q.h^2}{2} = -0.09 \times 8.82 = -0.794 \text{ Kg. m}$$

IV.2.4 Horizontal wind: (depression)

$$H_A = \delta \times \frac{q.h}{2} = 0.475 \times \frac{1.0 \times 4.20}{2} = 0.998 \text{ Kg}$$

$$H_E = q.h - H_A = 1.0 \times 4.20 - 0.998 = 3.202 \text{ Kg}$$

$$V_E = -V_A = \frac{1.0 \times 4.20^2}{2 \times 11} = 0.802 \text{ Kg}$$

$$M_D = -\beta \times \frac{q.h^2}{2} = -0.525 \times 8.82 = -4.631 \text{ Kg. m}$$

$$M_B = \delta \times \frac{q.h^2}{2} = 0.475 \times 8.82 = 4.189 \text{ Kg. m}$$

$$M_C = \gamma \times \frac{q.h^2}{2} = 0.09 \times 8.82 = 0.794 \text{ Kg. m}$$

IV.2.5 Summary tables:

IV.2.5.1 Internal forces under unit load $q = 1.0 \text{ kg/m}$

Tabale IV.2: Internal forces under unit

		Reaction d' support (Kg)				Moments (Kg.m)		
Action	q(Kg/m)	H_A	H_E	V_A	V_E	M_B	M_C	M_D
G	1.0	1.826	-1.826	5.5	5.5	-7.668	5.627	-7.668
S	1.0	1.826	-1.826	5.5	5.5	-7.668	5.627	-7.668
V1(horizontal)	1.0	-0.998	-3.202	-0.802	0.802	4.189	0.794	-4.631
V2(uprising)	1.0	-1.826	1.826	-5.5	-5.5	7.668	-5.627	7.668

IV.2.5.2 Internal forces under current loads:

Table IV.3: Values the Internal forces under current loads

		Reaction d' support (Kg)				Moments (Kg.m)		
Action	q(Kg/m)	H_A	H_E	V_A	V_E	M_B	M_C	M_D
G	473.275	864.2	-864.2	2603.01	2603.01	-3629.07	2663.118	-3629.07
S	137.5	251.075	-251.075	756.25	756.25	-1054.35	773.7125	-1054.35
V1(horizontal)	3569.5	-3562.36	-11429.54	-2862.74	2862.74	14952.64	2834.183	-16530.4
V2(uprising)	2813.84	-5138.07	5138.07	-15476.12	-15476.12	21576.53	-15833.5	21576.53
V3 = V1 + V2		8700.43	-6291.47	-18338.86	-12613.38	36529.16	-12999.3	5046.171

IV.2.5.3 Combinations at ULS:

Table IV.4: Values Combinations at ULS

	Reaction d' support (Kg)				Moments (Kg.m)		
Combination	H_A	H_E	V_A	V_E	M_B	M_C	M_D
1.35G + 1.5 S	1543.283	-1543.28	4648.439	4648.439	-6480.77	4755.778	-6480.77
1.35G + 1.35 S + 1.35 V3	13251.2	-9999.11	-20222.5	-12493.1	42991.75	-12909.3	489.7139
G + 1.5 V3	13914.85	-10301.4	-24905.3	-16317.1	51164.67	-16835.8	3940.187

IV.3 Calculation of the global geometric imperfection:

They are taken into account when the sum of the horizontal forces is less than 15% of the sum of the vertical forces. They can be replaced by a system of equivalent forces calculated for each column.

$H_{eq} = \emptyset N_{sd}$

with:

H_{eq}- equivalent horizontal force applied at the head of each column.

N_{sd}- normal compression force in the post.

$\emptyset = \emptyset_0 \times \alpha_h \times \alpha_m$- initial defect of plumb.

$\emptyset_0 = 1/200$- is the base value.

$\alpha_h = 2/\sqrt{h}$ -is the reduction coefficient which takes into account the height h applicable to the column.

$\alpha_m = \sqrt{0.5(1+1/m)}$ - is the reduction coefficient which takes into account the number of columns in a row.

$h = 5.20\ m$: is the height of the structure in meters.

$m = 2$: number of posts in a row.

$\alpha_h = 2/\sqrt{5.20} = 0.877$

$\alpha_m = \sqrt{0.5(1+1/2)} = 0.866$

$\emptyset = \emptyset_0 \times \alpha_h \times \alpha_m = 1/200 \times 0.877 \times 0.866 = 0.0038$

IV.3.1 Modeling with the imperfections:

IV.3.1.1 Efforts at the base of columns at ULS:

Table IV.5: Efforts at the base of columns at ULS

Combination ELU	Column1	Column1	sum					
	H_A(KN)	V_A(KN)	H_E(KN)	V_E(KN)	H(KN)	V(KN)	0.15 V	$\lvert H\rvert \geq 0.15\lvert V\rvert$
Comb 1	15.43	46.48	-15.43	46.48	0.00	92.96	13.944	No

If: $\lvert H\rvert \geq 0.15\lvert V\rvert$ The defects of plumb are not considered.

Note:

Plumbing faults are not to be considered for combinations 2 and 3 because the following condition: $\lvert H\rvert \geq 0.15\lvert V\rvert$ is verified.

IV.3.1.2 Equivalent force at the head of the posts:

Table IV.6: Equivalent force at the head of the posts

Combination	Column1		Column1	
	N_{sd}(Kg)	$H_{eq} = \emptyset N_{sd}$	N_{sd}(Kg)	$H_{eq} = \emptyset N_{sd}$
Comb 1 : 1.35G + 1.5 S	46.48	0.18	46.48	0.18

IV.3.2 Calculation of additional internal forces:

Horizontal force at the top of the column:

$P = 2 \times 0.18 = 0.36$ KN

$$H_A = \frac{P}{2}\left[1 + \frac{\emptyset(3+2\emptyset)}{2\Delta}\right] = \frac{0.36}{2}\left[1 + \frac{0.238(3 + 2 \times 0.238)}{2 \times 4.523}\right] = 0.196 \text{ KN}$$

$$H_E = P - H_A = 0.36 - 0.196 = 0.164 \text{ KN}$$

$$V_A = -V_E = -\frac{Ph}{l} = -\frac{0.36 \times 4.20}{11} = -0.137 \text{ KN}$$

$$\beta = \frac{1}{2}\left[1 + \frac{\emptyset(3+2\emptyset)}{2\Delta}\right] = \frac{1}{2}\left[1 + \frac{0.238(3 + 2 \times 0.238)}{2 \times 4.523}\right] = 0.5457$$

$$\delta = \frac{1}{2}\left[1 - \frac{\emptyset(3+2\emptyset)}{2\Delta}\right] = \frac{1}{2}\left[1 - \frac{0.238(3 + 2 \times 0.238)}{2 \times 4.523}\right] = 0.4543$$

$$\delta = \frac{\emptyset}{2}\left[1 - \frac{(1+\emptyset)(3+2\emptyset)}{2\Delta}\right] = \frac{0.238}{2}\left[1 - \frac{(1 + 0.238)(3 + 2 \times 0.238)}{2 \times 4.523}\right] = 0.062$$

$$M_B = +\beta \times Ph = 0.5457 \times 5$$

$4.20 = -0.094$ KN. m

IV.3.3 ULS combinations with H_{eq} taken into account:

Table IV.7: ULS combinations with H_{eq} taken into account

	Reaction d' support (KN)				Moments (kN. m)		
Combination	H_A	H_E	V_A	V_E	M_B	M_C	M_D
1.35G + 1.5 S	15.43	-15.43	46.48	46.48	-64.81	47.56	-64.81
P	0.196	0.164	-0.137	0.137	0.825	-0.094	-0.687
1.35G + 1.5 S + P	15.626	-15.266	46.343	46.617	-63.985	47.466	-65.497
1.35G + 1.35 S + 1.35 V3	13.25	-99.99	-20.22	-12.49	42.99	-12.91	4.89
G + 1.5 V3	13.91	-10.3	-24.91	-16.32	511.65	-16.84	39.40

IV.4 Choice of the analysis method:

The choice of the analysis method is conditioned by the value of the critical distance coefficient α_{cr} .

if $\alpha_{cr} \geq 10$ Rigid structure: elastic analysis at 1^{st} order.

if $\alpha_{cr} < 10$ Flexible structure: elastic analysis taking into account the effects of 2^{eme} order.

if $\alpha_{cr} \geq 15$ Rigid structure: plastic analysis.

IV.4.1 Determination of the minimum critical distance factor α_{cr}:

In the case of low slope portal frames, the critical distance coefficient α_{cr} can be calculated with the following approximate formula for the combination of action considered.

$$\alpha_{cr} = \frac{H}{\delta_H} \times \frac{h}{V}$$

With:

H: Total horizontal action

V:Total vertical action

δ_H:horizontal displacement

$h = 4.20m$ post height

$$\alpha_{cr} = \frac{H}{\delta_H} \times \frac{h}{V} = 11$$

Or by the following relation:

$$R = \frac{I_1 . s}{I_2 . h} = \frac{s}{h} = \frac{5.5/\cos 10}{4.20} = 1.329$$

$$N_{cr.p} = \frac{\pi^2 EI}{h^2} = \frac{\pi^2 \times 2.1 \times 10^4 \times 5410}{420^2} = 6356.495 \text{ KN}$$

$$N_{cr.t} = \frac{\pi^2 EI}{s^2} = \frac{\pi^2 \times 2.1 \times 10^4 \times 5410}{558.5^2} = 3594.76 \text{ KN}$$

Under the combination 1.35G + 1.5 S :

$$N_{sd.t} = 46.48 \times \sin 10 + 15.43 \times \cos 10 = 23.267 \text{ KN}$$

$$N_{sd.p} = 46.48 \text{ KN}$$

$$\frac{1}{\alpha_{cr}} = \frac{V_{sd}}{V_{cr}} = \left[\frac{23.267}{6356.495} + (4 + 3.3 \times 1.329)\left(\frac{46.48}{3594.76}\right)\right] = 0.1 \leq 0.1$$

→Rigid structure

We opt for the elastic method to the 1st order.

IV.5 Pre- size of the cross Strut:

IV.5.1 The maximum moments requesting the cross strut:

IV.5.1.1 Downward actions: gravity loads

Under the combination: $1.35G + 1.5\,S + P$

- Support: $M_D = -65.497\ \mathrm{KN.m}$
- At the ridge: $M_C = 47.466\ \mathrm{KN.m}$

IV.5.1.2 Upward actions: uplift wind

Under the combination: $G + 1.5\,V3$

- Support: $M_B = 511.65\ \mathrm{KN.m}$
- At the ridge: $M_C = -16.84\ \mathrm{KN.m}$

IV.5.1.3 Preliminary calculation:

$$\mathrm{M_{y.sd}} \leq \mathrm{M}_{ply.Rd} = \frac{\mathrm{Wpl.y} \times fy}{\gamma_{M0}}$$

$$\mathrm{Wpl.y} \geq \frac{\mathrm{M_{y.sd}} \times \gamma_{M0}}{fy} = \frac{65.497 \times 1.1 \times 10^2}{27.50} = 261.988\ cm^3$$

is IPE 270 ; $\mathrm{Wpl.y} = 484 cm^3$

Note:

The selected sections are underestimated to take into account the effects of buckling, lateral-torsional buckling and deflection.

IV.5.2 Checking the cross at the SLS:

IV.5.2.1 Arrow check:

The deflection is calculated at the ridge of the cross member, at C, under the unweighted combined action of: $G + S$

The maximum deflection at the ridge is given by the following formula:

$$y_{max} = \frac{1}{384 \times 2.1 \times 10^4 \times 5790}(5 \times 6.105 \times 10^{-2} \times 1100^4 - 48 \times 46.83 \times 10^2 \times 1100^2) = 3.746\ cm$$

$y_{max} = 3.746\ cm < l/200 = 5.5cm$ … … … … … … … … … … … …. Verified.

IV.5.3 Verification of the cross at the SLS:

IV.5.3.1 Checking the resistance section:

Assessment of efforts:

$\mathrm{M_{y.sd}} = 65.497\ \mathrm{KN.m}$

$N_{sd} = 46.617 \times \sin 10 + 15.266 \times \cos 10 = 23.13\ \mathrm{KN}$

$V_{z.sd} = 46.617 \times \cos 10 - 15.266 \times \sin 10 = 43.26\ \mathrm{KN}$

IV.5.3.1.1 Sections of class:

Sole class: (compressed)

$$\frac{\mathrm{c}}{\mathrm{t_f}} = \frac{b/2}{\mathrm{t_f}} = \frac{135/2}{10.2} = 6.618 \leq 10\varepsilon = 9.2\ \rightarrow \text{Class 1}$$

Soul class: (compound flexion)

$$d_c = \frac{N_{sd}}{t_w . f_y} = \frac{23.13}{0.66 \times 27.5} = 1.274\ cm$$

$$\alpha = \frac{1}{d}\left(\frac{d + d_c}{2}\right) = \frac{1}{21.96}\left(\frac{21.96 + 1.274}{2}\right) = 0.529 \qquad \alpha < 1$$

For class 1 section:

$$\frac{d}{\mathrm{t_w}} = \frac{219.6}{6.6} = 33.27 \qquad ; \qquad \frac{396.\varepsilon}{(13\alpha - 1)} = \frac{396 \times 0.92}{(13 \times 0.529 - 1)} = 61.99$$

$33.27 < 61.99$ (class 1 soul)

The section in IPE 270 is class 1.

IPE 270 : $\mathrm{A} = 45.9\ \mathrm{cm^2}$; $\mathrm{Wpl.y} = 484\ \mathrm{cm^3}$; $\gamma_{M0} = 1.1$; $f_y = 27.5\ \mathrm{KN/cm^2}$

IV.5.3.1.2 Incidence of shear force:

If: $V_{Sd} \leq 0.5 V_{pl.Rd} \rightarrow$ There is no interaction between the bending moment and the shearing force.

$V_{\mathrm{z.sd}} = 43.26\ \ \mathrm{KN}$

$A_{vz} = 22.1\ cm^2$

$$V_{pl.z.Rd} = \frac{A_{vz} \times \left(f_y/\sqrt{3}\right)}{\gamma_{M0}} = \frac{22.1 \times \left(27.5/\sqrt{3}\right)}{1.1} = 318.99\ \mathrm{KN}$$

$$\frac{V_{z.sd}}{V_{pl.z.Rd}} = \frac{43.26}{318.99} = 0.136 < 0.5$$

→ The effect of the shear force on the resistance moment can be neglected.

IV.5.3.1.3 Incidence of normal exertion:

If $N_{Sd} \leq Min\left(0.25\ N_{pl.Rd}, 0.5A_w f_y/\gamma_{M0}\right)$: There is no interaction between moment of resistance and normal stress.

$N_{Sd} = 23.13$ KN

$$N_{pl.Rd} = \frac{A.f_y}{\gamma_{M0}} = \frac{45.9 \times 27.5}{1.1} = 1147.5 \text{ KN}$$

$0.25\ N_{pl.Rd} = 0.25 \times 1147.5 = 286.875$ KN

$A_w = A - 2b.t_f = 45.9 - 2 \times 13.5 \times 1.02 = 18.36\ cm^2$

$0.5A_w f_y/\gamma_{M0} = 0.5 \times 18.36 \times 27.5/1.1 = 229.5$ KN

$N_{Sd} = 23.13 \text{ KN} \leq Min(286.875\ ; 229.5) = 229.5$ KN

→ The effect of normal stress on the resistance moment can be neglected.

The resistance check formula is given as follows:

$M_{y.sd} \leq M_{c.Rd}$ The section is class 1.

$M_{c.Rd} = M_{ply.Rd} = Wpl.y \times fy/\gamma_{M0} = 484 \times 27.5/1.1 = 12100$ KN. m

$M_{y.sd} = 65.497 \text{ KN. m} < M_{ply.Rd} = 12100$ KN. m … … … … … … … … … … … … OK

IV.5.3.2 Verification of the instability element:

IV.5.3.2.1 Downward action:

Spill verification:

Spillage = Lateral buckling of the compressed part + Rotation of the cross section.

IV.5.3.2.2 Upper sole:

The upper sole which is compressed under the action of downward vertical loads is liable to dump between the lateral support points.

The instability element verification formulas are as follows:

Compound bending with risk of buckling:

$$\frac{N_{Sd}}{\chi_{min}.N_{pl.Rd}}+\frac{k_y.\mathrm{M_{y.sd}}}{\mathrm{M}_{ply.Rd}}\leq 1.0$$

Compound bending with risk of overturning:

$$\frac{N_{Sd}}{\chi_z.N_{pl.Rd}}+\frac{k_{LT}.\mathrm{M_{y.sd}}}{\chi_{LT}.\mathrm{M}_{ply.Rd}}\leq 1.0$$

IV.5.3.3 Calculation of the reduction coefficient for buckling χ_{min}:

IV.5.3.3.1 Buckling lengths:

$l_y=\frac{550}{\cos 10}=558.5\ cm$ (Half of the crossbar)

$l_y=\frac{275}{\cos 10}=279.24\ cm$(Maintained by the purlins connected to the wind beam)

IV.5.3.3.2 slenderness:

$\lambda_y=\frac{l_y}{i_y}=\frac{558.5}{11.12}=50.22$; $\lambda_z=\frac{l_z}{i_z}=\frac{279.24}{3.02}=92.46$

IV.5.3.3.3 Reduced slenderness:

$\bar{\lambda}_y=\left[\frac{\lambda_y}{\lambda_1}\right]\times[\beta_A]^{0.5}=0.579$; $\bar{\lambda}_z=\left[\frac{\lambda_z}{\lambda_1}\right]\times[\beta_A]^{0.5}=1.065$

IV.5.3.3.4 Buckling curves: $h/b=270/135=2>1.2$

Buckling axis $\mathrm{y-y}$ → buckling curve **a** ; $\alpha_y=0.21$

Buckling axis $\mathrm{Z-Z}$ → buckling curve **b** ; $\alpha_z=0.34$

$\varphi_y=0.5[1+0.21\times(0.579-0.2)+0.579^2]=0.707$

$$\chi_y=\frac{1}{0.707+[0.707^2-0.579^2]^{0.5}}=0.899$$

$\varphi_z=0.5[1+0.34\times(1.065-0.2)+1.065^2]=1.214$

$$\chi_z=\frac{1}{1.214+[1.214^2-1.065^2]^{0.5}}=0.557$$

$\chi_{min}=Min(\chi_y;\chi_z)=Min(0.899;0.557)=0.557$

IV.5.3.4 Calculation of the reduction coefficient for the lateral discharge χ_{LT}:

IPE 270 : $i_z=3.02\ cm$; $h=27\ cm$; $t_f=1.02\ cm$

$L = \frac{275}{\cos 10} = 279.24\ cm$ Maintained by purlins connected to the wind beam

$C_1 = 1.88 - 1.4\Psi + 0.52\Psi^2 \leq 2.7$

With $M_a < M_b$ Moments at the ends of the section.

$$-1.0 \leq \Psi \leq 1.0$$

IV.5.3.5 Calculation of the moment at any point P:

$$M_p = 46.617.x - 65.497 - 15.266 \times 0.176.x - \frac{8.45.x^2}{2}$$

$$M_p = 43.93.x - 65.497 - 4.225.x^2$$

$$M_\alpha = M_p(x = 2.79m) = 43.93 \times 2.79 - 65.497 - 4.225 \times 2.79^2 = 24.18\ KN.m$$

$$\Psi = \frac{M_\alpha}{M_b} = \frac{24.18}{-65.497} = -0.369$$

$$C_1 = 1.88 - 1.4 \times (-0.369) + 0.52 \times (-0.369)^2 = 2.47 \leq 2.7$$

$$\lambda_{LT} = \frac{L_z/i_z}{C_1^{0.5}\left[1 + \frac{1}{20}\left(\frac{L/i_z}{h/t_f}\right)^2\right]^{0.25}} = \frac{279.24\ /3.02}{2.47^{0.5}\left[1 + \frac{1}{20}\left(\frac{279.24\ /3.02}{27/1.02}\right)^2\right]^{0.25}} = 52.23$$

$$\bar{\lambda}_{LT} = \left[\frac{\lambda_{LT}}{\lambda_1}\right] \times [\beta_w]^{0.5} = \left[\frac{52.23}{86.81}\right] \times 1.0 = 0.602 > 0.4$$

$$\varphi_{LT} = 0.5[1 + 0.21 \times (0.602 - 0.2) + 0.602^2] = 0.723$$

$$\chi_{LT} = \frac{1}{\varphi_{LT} + [\varphi_{LT}^2 - \bar{\lambda}_{LT}^2]^{0.5}} = \frac{1}{0.723 + [0.723^2 - 0.602^2]^{0.5}} = 0.89 < 1.0$$

IV.5.3.6 Calculation of the k coefficients:

$\beta_{MLT} = 1.8 - 0.7\Psi$ equivalent uniform moment factor for lateral torsional buckling.

$$\beta_{MLT} = 1.8 - 0.7\Psi = 1.8 - 0.7 \times (-0.369) = 2.06$$

$$\mu_{LT} = 0.15 \times \bar{\lambda}_z.\beta_{MLT} - 0.15 = 0.15 \times 1.065 \times 2.06 - 0.15 = 0.179$$

$$k_{LT} = 1 - \frac{\mu_{LT}.N_{Sd}}{\chi_z.Af_y} = 1 - \frac{0.179 \times 23.13}{0.557 \times 45.9 \times 27.5} = 0.994$$

Calculation of the equivalent uniform moment factor for the following bending buckling yy.

$$\beta_{My} = \beta_{M\Psi} + \frac{M_Q}{\Delta M}\left(\beta_{MQ} - \beta_{M\Psi}\right) \qquad ;\ \beta_{M\Psi} = 1.8 - 0.7\Psi$$

$$\Psi = \frac{\mathrm{M}_\alpha}{\mathrm{M}_\mathrm{b}} = \frac{47.466}{-65.497} = -0.725 \qquad ;\ \beta_{M\Psi} = 1.8 - 0.7 \times (-0.725) = 2.308$$

$$\Delta M = 65.497 + 47.466 = 112.963\ \ \mathrm{KN.m}$$

$$M_Q = \frac{ql^2}{8} = \frac{8.45 \times 5.5^2}{8} = 31.95\ \mathrm{KN.m}$$

$\beta_{MQ} = 1.3$ evenly distributed load.

$$\beta_{My} = \beta_{M\Psi} + \frac{M_Q}{\Delta M}\left(\beta_{MQ} - \beta_{M\Psi}\right) = 2.308 + \frac{31.95}{112.963}(1.3 - 2.308) = 2.0$$

$$\mu_y = 0.579 \times (2 \times 2.0 - 4) + \frac{484-429}{429} = 0.128 \leq 0.9$$

$$k_y = 1 - \frac{0.128 \times 23.13}{0.899 \times 45.9 \times 27.5} = 0.997 \leq 1.5$$

IV.5.3.7 Buckling verification:

$$\frac{N_{Sd}}{\chi_{min}.N_{pl.Rd}} + \frac{k_y.\mathrm{M}_{\mathrm{y.sd}}}{\mathrm{M}_{ply.Rd}} \leq 1.0$$

$$\frac{23.13}{0.557 \times 1147.5} + \frac{0.997 \times 65.497}{121} = 0.576 < 1 \ldots\ldots\ldots..\mathrm{Verified}$$

IV.5.3.8 Spill verification:

$$\frac{N_{Sd}}{\chi_z.N_{pl.Rd}} + \frac{k_{LT}.\mathrm{M}_{\mathrm{y.sd}}}{\chi_{LT}.\mathrm{M}_{ply.Rd}} \leq 1.0$$

$$\frac{23.13}{0.557 \times 1147.5} + \frac{0.994 \times 65.497}{0.89 \times 121} = 0.641 < 1 \ldots\ldots\ldots..\mathrm{Verified}$$

IV.5.4 Upward action: ↑

IV.5.4.1 Bottom sole:

The lower sole which is compressed under the action of the uplift wind is liable to discharge as long as it is free over its entire length.

Assessment of efforts:

$$\mathrm{M}_{\mathrm{y.sd}} = 511.65\ \mathrm{KN.m}$$

$N_{sd} = -24.91 \times \sin 10 + 13.91 \times \cos 10 = 9.37 \text{ KN}$

$V_{z.sd} = -24.91 \times \cos 10 - 13.91 \times \sin 10 = -26.947 \text{ KN}$

IV.5.4.2 Calculation of the reduction coefficient for the lateral discharge χ_{LT}:

$$\Psi = \frac{\mathrm{M}_\alpha}{\mathrm{M}_\mathrm{b}} = \frac{-16.84}{511.65} = -0.033$$

$$C_1 = 1.88 - 1.4 \times (-0.033) + 0.52 \times (-0.033)^2 = 1.927 \leq 2.7$$

$$\lambda_{LT} = \frac{279.24\ /3.02}{1.927^{0.5}\left[1 + \frac{1}{20}\left(\frac{279.24\ /3.02}{27/1.02}\right)^2\right]^{0.25}} = 59.131$$

$$\bar{\lambda}_{LT} = \left[\frac{59.131}{86.81}\right] \times 1.0 = 0.681 > 0.4$$

$$\varphi_{\mathrm{LT}} = 0.5[1 + 0.21 \times (0.681 - 0.2) + 0.681^2] = 0.782$$

$$\chi_{\mathrm{LT}} = \frac{1}{0.782 + [0.782^2 - 0.681^2]^{0.5}} = 0.86 < 1.0$$

Conclusion:

The chosen profile IPE 270 is suitable as a cross member.

IV.6 Checking the posts:

Assessment of efforts:

$\mathrm{M}_{\mathrm{y.sd}} = 65.497 \text{ KN.m}$

$N_{sd} = 46.617 \text{ KN}$

$V_{z.sd} = 15.266 \text{ KN}$

IV.6.1 Calculation of the reduction coefficient for buckling χ_{min}:

$$\chi_{min} = Min(\chi_y; \chi_z)$$

IV.6.1.1 Buckling with respect to the strong yy axis (in the plane of the gantry):

$$\lambda_y = \frac{l_y}{i_y} = \frac{420}{9.17} = 45.802 \quad ; \quad \bar{\lambda}_y = \left[\frac{\lambda_y}{\lambda_1}\right] \times [\beta_\mathrm{A}]^{0.5} = 0.528$$

Buckling curves: $h/b = 210/220 = 0.95 < 1.2$

Buckling axis $y - y \quad \rightarrow$ buckling curve **b** ; $\alpha_y = 0.34$

$$\varphi_y = 0.5[1 + 0.34 \times (0.528 - 0.2) + 0.528^2] = 0.695$$

$$\chi_y = \frac{1}{0.695 + [0.695^2 - 0.528^2]^{0.5}} = 0.872$$

IV.6.1.2 Buckling with respect to the weak zz axis (outside the gantry plane):

$$\lambda_z = \frac{l_z}{i_z} = \frac{120}{5.51} = 21.78 \quad ; \qquad \bar{\lambda}_z = \left[\frac{\lambda_z}{\lambda_1}\right] \times [\beta_A]^{0.5} = 0.251$$

Buckling curves: $h/b = 210/220 = 0.95 < 1.2$

Buckling axis $Z - Z \quad \rightarrow$ buckling curve **c** ; $\alpha_z = 0.49$

$$\varphi_z = 0.5[1 + 0.49 \times (0.251 - 0.2) + 0.251^2] = 0.544$$

$$\chi_z = \frac{1}{0.544 + [0.544^2 - 0.251^2]^{0.5}} = 0.97$$

$$\chi_{min} = Min(\chi_y; \chi_z) = Min(0.872; 0.97) = 0.872$$

IV.6.2 Calculation of the reduction coefficient for the lateral discharge χ_{LT}:

HEA 220 : $i_z = 5.51\ cm$; $h = 21\ cm$; $t_f = 1.1\ cm$ $\qquad L = 120\ cm$

$$C_1 = 1.88 - 1.4\Psi + 0.52\Psi^2 \leq 2.7$$

$$\Psi = \frac{M_\alpha}{M_b}$$

With $M_a < M_b$ Moments at the ends of the most loaded section.

$$-1.0 \leq \Psi \leq 1.0$$

$$M_b = 65.497\ KN.m$$

$$M_\alpha = M_{y.sd}(h = 3m) = \frac{65.497 \times 3}{4.20} = 46.78\ KN.m$$

$$\Psi = \frac{M_\alpha}{M_b} = \frac{46.78}{65.497} = 0.714$$

$$C_1 = 1.88 - 1.4 \times 0.714 + 0.52 \times 0.714^2 = 1.145 \leq 2.7$$

We take $C_1 = 1.145$

$$\lambda_{LT} = \frac{120\ /5.51}{1.145^{0.5}\left[1+\frac{1}{20}\left(\frac{120\ /5.51}{21/1.1}\right)^2\right]^{0.25}} = 20.035$$

$$\bar{\lambda}_{LT} = \left[\frac{20.035}{86.81}\right] \times 1.0 = 0.23 > 0.4$$

$$\varphi_{LT} = 0.5[1 + 0.21 \times (0.23 - 0.2) + 0.23^2] = 0.529$$

$$\chi_{LT} = \frac{1}{0.529 + [0.529^2 - 0.23^2]^{0.5}} = 0.995 < 1.0$$

IV.6.3 Calculation of the k coefficients:

IV.6.3.1 Calculation of the coefficient k_{LT}:

Calculation of the equivalent uniform moment factor β_{MLT}:

End moment case:

$$\Psi = \frac{M_\alpha}{M_b} = \frac{46.78}{65.497} = 0.714$$

$$\beta_{MLT} = \beta_{M\Psi} = 1.8 - 0.7\Psi = 1.8 - 0.7 \times 0.714 = 1.3$$

$$\mu_{LT} = 0.15 \times \bar{\lambda}_z.\beta_{MLT} - 0.15 = 0.15 \times 0.251 \times 1.3 - 0.15 = -0.1$$

$$k_{LT} = 1 - \frac{\mu_{LT}.N_{Sd}}{\chi_z.Af_y} = 1 - \frac{-0.1 \times 46.617}{0.97 \times 64.3 \times 27.5} = 1.002$$

We take $k_{LT} = 1.0$

IV.6.3.2 Calculation of the coefficient k_y:

Calculation of the equivalent uniform moment factor β_{My}:

End moment case:

$$\Psi = \frac{M_\alpha}{M_b} = \frac{0}{65.497} = 0$$

$$\beta_{My} = \beta_{M\Psi} = 1.8$$

$$\mu_y = 0.528 \times (2 \times 1.8 - 4) + \frac{568.5 - 515.2}{515.2} = -0.11 \leq 0.9$$

$$k_y = 1 - \frac{-0.11 \times 46.617}{0.872 \times 64.3 \times 27.5} = 1.0 \leq 1.5$$

$$\mathrm{N_{pl.Rd}} = \frac{\mathrm{A} \times fy}{\gamma_{M0}} = \frac{64.3 \times 27.5}{1.1} = 1607.5\ KN$$

$$M_{ply.Rd} = \frac{W_{pl.y} \times f_y}{\gamma_{M0}} = \frac{568.5 \times 27.5}{1.1} = 14212.5KN.cm = 142.12KN.m$$

IV.6.4 Buckling verification:

$$\frac{N_{Sd}}{\chi_{min}.N_{pl.Rd}} + \frac{k_y.\mathrm{M_{y.sd}}}{\mathrm{M}_{ply.Rd}} \leq 1.0$$

$$\frac{46.617}{0.872 \times 1607.5} + \frac{1.0 \times 65.497}{142.12} = 0.494 < 1 \dots\dots\dots..\text{Verified}$$

IV.6.5 Spill verification:

$$\frac{N_{Sd}}{\chi_z.N_{pl.Rd}} + \frac{k_{LT}.\mathrm{M_{y.sd}}}{\chi_{LT}.\mathrm{M}_{ply.Rd}} \leq 1.0$$

$$\frac{46.617}{0.97 \times 1607.5} + \frac{1.0 \times 65.497}{0.995 \times 142.12} = 0.49 < 1 \dots\dots\dots..\text{Verified}$$

Conclusion:

The chosen profile HEA 220 is suitable as a post

V. Introduction:

An assembly is a device which makes it possible to join together and secure several parts together, ensuring the transmission and distribution of the various stresses between the parts. without generating parasitic solicitations, in particular torsions.

V.1. Assembly Column Rafter:

V.1.1. Introduction:

- the column - rafter assembly is made using a plate bolted to the transom and the column.
- the assembly is solicited by a bending moment, shearing force and a Normal force.

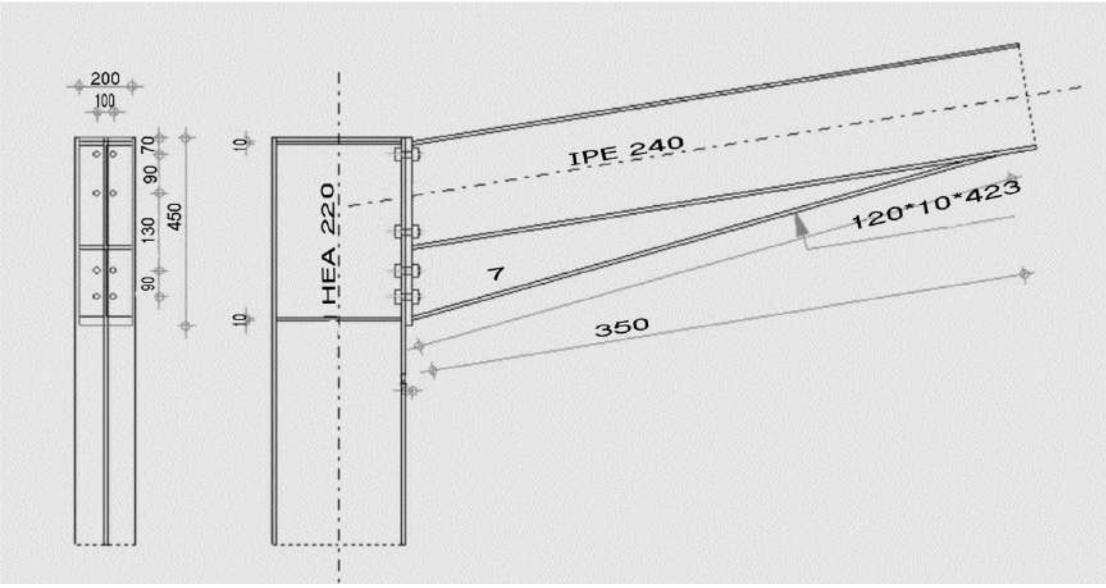


Figure V.1: Representation of the Column – Rafter assembly.

V.1.2. The demanding effort:

$\mathrm{M_{y.sd}} = 65.497\ \mathrm{KN.m}$

$N_{sd} = 46.617\ \mathrm{KN}$

$V_{z.sd} = 15.266\ \mathrm{KN}$

We choose bolts of class HR 10.5

Bolt diameter $\mathrm{d} = 20\ \mathrm{mm}$

Number of bolts = 8

Number of queues = 2

Column HEA 220

Rafter IPE 240

Plate height $h_p = 450$ mm

Plate width $b_p = 200$ mm

Plate thickness $t_p = 15$ mm

V.1.3. calculation of the height of the compressed part:

$$x = t_{fb}\sqrt{\frac{b_b}{t_{wb}}}$$

$t_{wb} = 6\ mm$; $t_{fb} = 10\ mm$; $b_b = 120$ mm

$$x = 10 \times \sqrt{\frac{120}{6}} = 44.72\ mm$$

$d_1 = 365$ mm ; $d_2 = 275$ mm ; $d_3 = 145$ mm

V.1.4. Calculation force of prestressing authorized in the bolts is worth:

$F_p = 0.7 \times f_{ub} \times A_s$

Diameter bolt 20 mm

$A_s = 245\ mm^2$; $f_{ub} = 1000\ N/mm^2$

$F_p = 0.7 \times 1000 \times 10^{-3} \times 245 = 171.5$ KN For a bolt

V.1.5. the effective moment of resistance of the assembly:

$$M_{Rd} = \frac{N_1 . \sum d_i^2}{d_1} = \frac{n . F_p . \sum d_i^2}{d_1}$$

Or:

n: is the number of bolts in a horizontal row.

V.1.6. verification of the resistance of the assembly:

$$M_{Sd} \leq M_{Rd}$$

$\Sigma d_i^2 = (145^2 + 275^2 + 365^2) = 229875\ \text{mm}^2$

$$M_{Rd} = \frac{n . F_p . \sum d_i^2}{d_1} = \frac{2 \times 171.5 \times 229875}{365} \times 10^{-3} = 216.02\ KN.m$$

$M_{Sd} = 65.497\ \text{KN.m} \leq M_{Rd} = 216.02\ KN.m$ … … … … … … … … … … … … OK

V.1.7. assembly resistance under shear force:

By bolt $\frac{V_{Sd}}{n} = \frac{15.266}{8} = 1.91\ KN$

It is necessary to verify that $V_{Sd}/n \leq V_{Rd} = k_s.m.\mu.F_p/\gamma_{M1}$

$k_s = 1.0$ normal hole. (Eurocode 3 § 6.5.8.1).

$m = 1$ a friction plane.

$\mu = 0.3$ Coefficient of friction. (Eurocode 3 § 6.5.8.3).

F_p : Calculation of prestressing. (Eurocode 3 § 6.5.8.3).

$1.91\ KN \leq V_{Rd} = 0.3 \times 171.5/1.25 = 41.16\ KN$

V.1.8. verification of the resistance of the column web in the tensile zone:

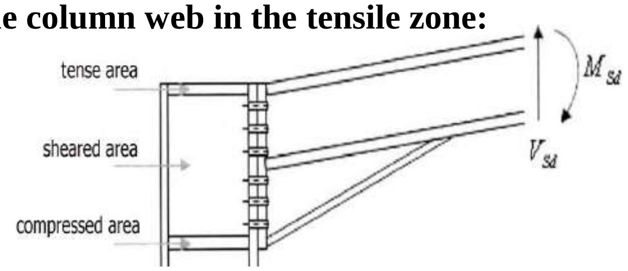


Figure V.2: Representation of the resistance of the column.

$F_v \leq F_{t.Rd}$

With: $F_{t.Rd} = t_{wc} \times b_{eff} \times \dfrac{f_y}{\gamma_{M0}}$

Or:

$F_{t.Rd}$ Tensile strength of the column core.

t_{wc} thickness of the web of the column.

$b_{eff} = p$ center distance of bolt rows (p = 100 mm).

$F_{t.Rd} = 27.5 \times 0.7 \times 10/1.1 = 175\ KN$

shear force is worth:

$$F_V = \frac{M_{Sd}}{h - t_f} = \frac{65.497}{0.21 - 0.011} = 329.13\ KN$$

$F_V = 329.13\ KN > F_{t.Rd} = 175\ KN \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots..$unverified

Hence the need for stiffening: (stiffener thickness 10 mm).

V.1.9. verification of the resistance of the pole core in the compressed area.

$N_{Sd} \leq F_{c.Rd}$

$$\delta_{c.Sd} = \frac{V_{Sd}}{A} + \frac{M_{Sd}.z_{max}}{I_y}$$

With:

$\delta_{c.Sd}$: normal compressive stress in the core of the column to the compressive stress and at the same time.

$$\delta_{c.Sd} = \frac{V_{Sd}}{A} + \frac{M_{Sd}.z_{max}}{I_y} = \frac{15.266}{64.3} + \frac{65.497 \times 10^2 \times 18}{5410} = 22.03\,KN/cm^2$$

$$\delta_{c.Sd} = 22.03\,KN/cm^2 > 0.7f_y = 19.25\,KN/cm^2 \quad \rightarrow \quad k_c = 1.7 - \delta_{c.Sd}/f_y = 0.7$$

$t_p = 15\,mm$ thickness of the end plate.

$$b_{eff} = 10 + 2 \times 4\sqrt{2} + 5(11 + 18) + 2 \times 15 = 196.31\,mm$$

t_{fb} : beam sole thickness.

t_{fc} : column sole thickness.

t_p : end plate thickness.

r_c : column core / flange connection radius.

a_p : core throat thickness (estimated 4.0 mm).

if $\bar{\lambda}_p \leq 0.72 \quad \rightarrow \quad \rho = 1.0$

if $\bar{\lambda}_p > 0.72 \quad \rightarrow \quad \rho = \left(\bar{\lambda}_p - 0.2\right)/\bar{\lambda}_p{}^2$

$\bar{\lambda}_p = 0.932\sqrt{\dfrac{b_{eff}.d_{wc}.f_y}{E.t_{wc}^2}}$ reduced slenderness of the effective part of the core.

$$\bar{\lambda}_p = 0.932\sqrt{\frac{b_{eff}.d_{wc}.f_y}{E.t_{wc}^2}} = 0.932\sqrt{\frac{19.631 \times 15.2 \times 27.5}{2.1 \times 10^4 \times 0.7^2}} = 0.83 > 0.72$$

$$\rho = \left(\bar{\lambda}_p - 0.2\right)/\bar{\lambda}_p^2 = (0.83 - 0.2)/0.83^2 = 0.91$$

$$F_{c.Rd} = \frac{k_c.\rho b_{eff}.t_{wc}.f_y}{\gamma_{M1}\sqrt{\left(1 + 1.3 \times \left(b_{eff}/h\right)^2\right)}} = \frac{0.7 \times 0.91 \times 19.631 \times 0.7 \times 27.5}{1.1\sqrt{(1 + 1.3 \times (19.631/21)^2)}} = 149.73\,KN$$

$$N_{sd} = \Sigma N_i$$

ΣN_i : the sum of the forces in the tensioned bolts.

$$N_i = \frac{M_{Sd}.d_i}{\Sigma d_i} \qquad \mathrm{M_{sd}} = 65.497\ \mathrm{KN.m}$$

$$N_1 = \frac{M_{Sd}.d_1}{\Sigma d_i^2} = \frac{65.497 \times 365 \times 10^{-3}}{229875 \times 10^{-6}} = 103.99\ KN$$

$$N_2 = \frac{M_{Sd}.d_2}{\Sigma d_i^2} = \frac{65.497 \times 275 \times 10^{-3}}{229875 \times 10^{-6}} = 78.35\ KN$$

$$N_3 = \frac{M_{Sd}.d_3}{\Sigma d_i^2} = \frac{65.497 \times 145 \times 10^{-3}}{229875 \times 10^{-6}} = 41.31\ KN$$

$$N_{sd} = \Sigma N_i = 103.99 + 78.35 + 41.31 = 223.65\ KN$$

$$N_{sd} = 223.65KN > F_{c.Rd} = 149.73\ KN \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots..\mathrm{unverified}$$

the resistance of the column web in compression is low in comparison with the acting force. A stiffener must therefore be provided (stiffener thickness 10 mm).

V.1.10. verification of the resistance of the column core in the sheared zone:

$$F_v \leq V_{Rd}$$

$$V_{Rd} = 0.58 \times f_y \times .t_w = 0.58 \times 27.5 \times 21 \times 0.7/1.1 = 213.15\ KN$$

shear force is worth:

$$F_v = \frac{M_{Sd}}{\mathrm{h} - t_f} = \frac{65.497}{0.21 - 0.011} = 329.13\ KN$$

$$F_v = 329.13\ KN \geq V_{Rd} = 213.15\ KN \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots..\mathrm{unverified}$$

- **Note:**

it is not necessary to check the stiffened web of the column for resistance core the stiffeners have a thickness equal to those of the flanges of the beam.

V.2. Assembly Rafter – Rafter:

• The rafter - rafter is made using a bolted plate.

• If the span of the gantry does not exceed certain limits for transport (approximately 11m), assembly of the ridge can be carried out in the factory, off site, thus saving money.

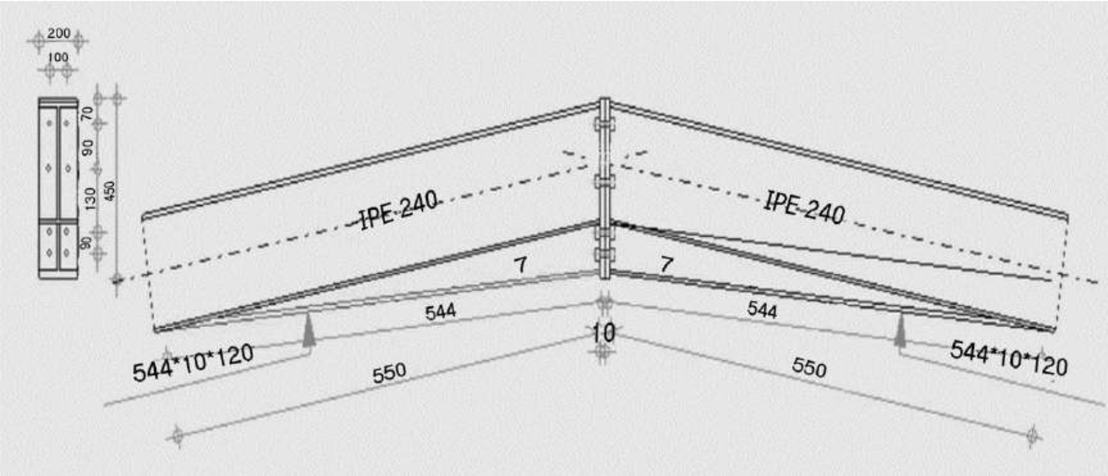


Figure V.3: representation of the rafter - rafter.

$M_{y.sd} = 47.466 \text{ KN.m}$

$N_{sd} = 23.13 \text{ KN}$

$V_{z.sd} = 43.26 \text{ KN}$

V.2.1. the effective moment of resistance of the assembly:

$$M_{Rd} = \frac{N_i.\Sigma d_i^2}{d_1} = \frac{n.F_p.\Sigma d_i^2}{d_1}$$

V.2.1.1. verification of the resistance of the assembly:

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

$F_p = 0.7 \times 1000 \times 10^{-3} \times 245 = 171.5 \text{ KN}$ For a bolt

$\Sigma d_i^2 = (145^2 + 275^2 + 365^2) = 229875 \text{ mm}^2$

$$M_{Rd} = \frac{n.F_p.\Sigma d_i^2}{d_1} = \frac{2 \times 171.5 \times 229875}{365} \times 10^{-3} = 216.01\ KN.m$$

$M_{sd} = 47.466 \text{ KN.m} \leq M_{Rd} = 216.01\ KN.m \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots..ok.$

V.2.1.2. resistance of the assembly under the shearing force:

By bolt $\frac{V_{Sd}}{n} = \frac{43.26}{8} = 5.41\ KN$

It is necessary to verify that $V_{Sd}/n \leq V_{Rd} = k_s.m.\mu.F_p/\gamma_{M2}$

$5.41\ KN \leq V_{Rd} = 0.3 \times 171.5/1.25 = 41.16\ KN$

V.3. Calculation of column bases:

The bases of the posts and the anchor rods are elements of continuity which ensure the transmission of forces from the superstructure to the foundations, they are connecting devices.

These consist of a base plate called a plate for reducing the pressure in the concrete, welded to the post resting on the foundation and attached by nuts to the anchor rods which are embedded in the concrete.

V.3.1. Sizing of the column anchor rod:

- Axial compressive load: $N_{sd} = 46.617$ KN
- Corresponding shear force: $V_{z.sd} = 15.266$ KN
- Lifting effort: $N_{sd} = 23.13$ KN
- Corresponding shear force: $V_{z.sd} = 43.26$ KN

after the results from chapter 4.

V.3.2. basic data:

Grade steel seat plate S235: $f_y = 235\ N/mm^2$

Class concrete foundation C25/30: $f_{ck} = 25\ N/mm^2$

Partial safety factors:

Steel: $\gamma_{M0} = 1.1$; $\gamma_{M1} = 1.25$

Concrete: $\gamma_c = 1.5$

V.3.3. compressive strength of concrete:

$f_{cd} = \alpha_{cc} f_{ck}/\gamma_c$

Or: $f_{ck} = 25\, N/mm^2$

the value of α_{cc} is given in the national annex.

its recommended value is:

$\alpha_{cc} = 1.0$

the design resistance of the concrete becomes:

$f_{cd} = \alpha_{cc} f_{ck}/\gamma_c = 1 \times 25/1.5 = 16.7\, N/mm^2$

V.3.4. design crushing resistance of the sealing material:

the value of the coefficient of the sealing material is: $\beta_j = 2/3$

the dimensions of the foundation being unknown, take $(A_{c1}/A_{c0})^{0.5} = \alpha = 1.5$

the design crushing resistance of the sealing material:

$f_{jd} = \alpha . \beta_j . f_{cd} = f_{cd} = 16.7\,\mathrm{N/mm^2}$

V.3.5. estimate of area of the seat plate:

an estimate of the required area of the seat plate is given by the greater of the following two values:

$$A_{C0} = \frac{1}{\mathrm{h}_c b_{fc}} \left(\frac{N_{Sd}}{f_{cd}}\right)^2 = \frac{1}{210 \times 220} \left(\frac{46617}{16.7}\right)^2 = 168.66\ \mathrm{mm^2}$$

$A_{C0} = \dfrac{N_{Sd}}{f_{cd}} = \dfrac{46617}{16.7} = 2791.44\ \mathrm{mm^2}$, who is the biggest.

V.3.6. Choice of the type of the base plate:

As an estimate for:

$A_{C0} = 2791.44\ \mathrm{mm^2} < 0.95 \times 210 \times 220 = 43890\ \mathrm{mm^2}$

A short throw plate is satisfactory.

The correct plan dimensions for the short throw seat plate are chosen as follows:

$b_p = 500\ \mathrm{mm} > b_{fc} + 2t_{fc} = 220 + 2 \times 11 = 242\ \mathrm{mm}$

$h_p = 500 \text{ mm} > h_c + 2t_{fc} = 210 + 2 \times 11 = 232 \text{ mm}$

Which give: $A_{C0} = 500 \times 500 = 250000 \text{ mm}^2 > 2791.44 \text{ mm}^2$

V.3.7. Verification of the design resistance of the base plate:

V.3.7.1. Calculation of the additional support width c:

Where:

$A = +2$

$B = -(b_{fc} - t_{wc} + h_c) = -(220 - 7 + 210) = -423$

$$C = \frac{0.5 \times N_{Sd}}{f_{jd}} - \left(2b_{fc}t_{fc} + 4t_{fc}^2 + 0.5h_c t_{wc} - t_{fc}t_{wc}\right)$$

$C = 0.5 \times 46617/16.7 - (2 \times 220 \times 11 + 4 \times 11^2 + 0.5 \times 210 \times 7 - 11 \times 7)$

$= -4586.28 \text{ mm}^2$

The additional width is:

$$c = \frac{-B - \sqrt{B^2 - 4AC}}{2A} = \frac{423 - \sqrt{423^2 - 4 \times 2 \times (-4586.28)}}{2 \times 2} = -10.33 \text{ mm}$$

Note:

Since the compressive force Ns is low, which gives us the negative value of the additional width c.

For the calculation of additional width c in the case where the compressive force requesting the column is low, one proceeds as follows:

Calculation of the additional support width c:

By setting: $t = 25$ mm as the thickness of the base plate.

$$c = t\left(\frac{f_{yp}}{3f_{jd}\gamma_{M0}}\right)^{0.5} = 25 \times \left(\frac{235}{3 \times 16.7 \times 1.1}\right)^{0.5} = 51.62 \text{ mm}$$

$c = 51.62 \text{ mm} \leq (h_c - 2t_{fc})/2 = (210 - 2 \times 11)/2 = 94 \text{ mm}$

There is no overlap of the areas in compression for the sections of the two soles.

Note:

In the case of articulated assemblies, the overhang of the base plate is generally taken equal to 25 mm.

So: $\beta_c = 25 \text{ mm} < c = 51.62 \text{ mm}$ → the plate is of short projection.

V.3.7.2. Calculation of the cross section A_{eff} :

The base plate is of short projection.

$$A_{eff} = 2(b_{fc} + 2\beta_c)(c + \beta_c + t_{fc}) + (\mathrm{h}_c - 2c - 2t_{fc})(2c + t_{wc})$$

$$A_{eff} = 2(220 + 2 \times 25)(51.62 + 25 + 11) + (210 - 2 \times 51.62 - 2 \times 11)(2 \times 51.62 + 7)$$

$$A_{eff} = 56658.74 \text{ mm}^2$$

V.3.7.3. Calculation of resistance to axial force N_{Sd} :

$$N_{Sd} \leq N_{Rd}$$

With: $N_{Rd} = A_{eff}.f_{jd}$

$$N_{Rd} = 56658.74 \times 16.7 \times 10^{-3} = 946.2\ KN$$

$$N_{Sd} = 46.617\ KN \leq N_{Rd} = 946.2\ KN \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots. ok$$

V.3.8. Calculation of the resistance of the base plate at bending moment:

V.3.8.1. Calculation of the moment of resistance $M_{R.d}$:

$$M_{R.d} = \frac{t^2 f_y}{6\gamma_{M0}} = \frac{25^2 \times 235}{6 \times 1.1} = 22253.79\ Nmm/mm = 22.25\ KNmm/mm$$

V.3.8.2. Calculation of bending moment M_{Sd} :

$$M_{Sd} = \frac{(c^2/2)N_{Sd}}{A_{eff}} = \frac{(51.62^2/2) \times 46.617}{56658.74} = 1.09\ KNmm/mm$$

$$M_{Sd} = 1.09\ KNmm/mm < M_{R.d} = 22.25\ KNmm/mm \ \ldots.\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots. ok$$

V.3.8.3. verification the shear strength of the sealant base plate: $V_{Sd} \leq F_{v.Rd}$

With:

$$F_{v.Rd} = F_{f.Rd} = C_{f.d}N_{Sd} = 0.3 \times 46.617 = 13.99\ KN$$

N_{Sd} : is the computational force of the column compression.

$C_{f.d}$: is the coefficient of friction between the base plate and the sealing layer. A value of 0.3 is specified for the cement and sand backing mortar.

$V_{Sd} = 15.266 \text{ KN} > F_{v.Rd} = 13.99 \text{ KN} \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots$ unverified

Note:

The shear strength of the base plate seal is not verifier, the anchor rods will therefore take up the shear force which must be verifier accordingly.

V.3.9. anchor rods:

They are straight but generally curved at one end, which makes it possible to take up a greater tensile force and to block the rod and therefore the whole of the frame during the assembly of the structure. These tensile forces are generated by the uplift wind in general and by the moments at the base in the case of embedded posts.

V.3.9.1. Shear resistance of anchor rods:

EN1993-1-8 §6.2.2 gives the following formula for shear resistance:

$$F_{v.Rd} = F_{f.Rd} + n_b . F_{vb.Rd}$$

Or:

$F_{f.Rd}$: design resistance by friction in the presence of an axial compressive force N_{Sd} in the column.

$$F_{f.Rd} = 0.2 N_{Sd}$$

$F_{vb.Rd}$: design resistance of an anchor rod to shear.

$$F_{vb.Rd} = \frac{\alpha_{cb} . f_{ub} . A_s}{\gamma_{M2}}$$

$$\alpha_{cb} = 0.44 - 0.0003 . f_{yb} \qquad and \qquad 235\ N/mm^2 \leq f_{yb} \leq 640\ N/mm^2$$

n_b : number of rods located in the assembly.

It can be seen that this rule makes it possible to add the design resistance to shear of the anchor rods to that by friction, this latter resistance only existing for an axial force of compression in the post.

We verifier that the following condition is satisfied:

$$V_{Sd} \leq F_{v.Rd}$$

For two M27 rods in class 4.6.

$A_s = 245mm^2$ $\quad ; f_{ub} = 400\, N/mm^2$ $\quad ; f_{yb} = 240\, N/mm^2$

$F_{f.Rd} = 0.2N_{Sd} = 0.2 \times 46.617 = 9.323$ KN

$$F_{vb.Rd} = \frac{(0.44 - 0.0003 \times 240\,) \times 400 \times 245}{1.25} \times 10^{-3} = 29\, KN$$

$F_{v.Rd} = 9.323 + 2 \times 29 \approx 67$KN

$V_{Sd} = 15.266$ KN $< F_{v.Rd} = 67$ KN … … … … … … … … … … … … … … … … ok

For greater safety, it is common practice to provide shear spades to relieve the anchor rods in the event of great shear forces such as an earthquake.

We choose a spade having dimensions satisfying the following conditions:

- Effective depth: $60\, mm \leq L_{eff\,(\text{spade})} \leq 1.5\, \text{h}_{\text{spade}}$
- Height of the spade: $\text{h}_{\text{spade}} \leq 0.4\, \text{h}_c$
- Maximum slenderness of the wings: $b_{\text{spade}}/t_{\text{spade}} \leq 20$

V.3.9.2. Resistance of the anchor rods to the lifting force:

In the case where the force N_{sd} at the base of the column is a lifting force, the anchor rods must transmit this force as well as the entire concomitant shearing force $V_{z.sd}$ to the foundation.

According to the results of chapter 4.

Combination: G + 1.5V3

$N_{sd} = \text{V}_\text{A} = 23.13$ KN $\quad$ *and* $\quad V_{z.sd} = \text{H}_\text{A} = 43.26$ KN

V.3.9.3. Verification the resistance anchor rod:

By placing oneself in safety, one checks for an anchor rod that the following condition is satisfied:

$$\frac{V_{sd}/n_b}{F_{vb.Rd}} + \frac{N_{Sd}/n_b}{N_{t.Rd}} \leq 1$$

With:

$$N_{t.Rd} = \frac{0.9.f_{ub}.A_s}{\gamma_{M2}} = \frac{0.9 \times 400 \times 245}{1.25} = 70.6\, KN$$

$$\frac{V_{sd}/n_b}{F_{vb.Rd}} + \frac{N_{Sd}/n_b}{N_{t.Rd}} = \frac{43.26/2}{29} + \frac{23.13/2}{70.6} = 0.91 < 1 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots..ok$$

For two M27 rods in class 4.6.

$A_s = 245mm^2$; $f_{ub} = 400\ N/mm^2$; $f_{yb} = 240\ N/mm^2$; $d = 27$

V.3.9.4. Verification the anchor rod for adhesion:

For an anchor bolt:

$$N_{Sd}/2 \leq F_{anc.Rd}$$

The tensile anchoring resistance of an anchor rod is:

$$F_{anc.Rd} = \pi.d.f_{bd}(l_1 + 6.4r + 3.5l_2)$$

The current values are given as follows:

$r = 3d$; $l_2 = 2d$; $l_1 = 20d$

$r = 3d = 3 \times 27 = 81\ mm$

$l_1 = 20d = 20 \times 27 = 540\ mm$

$l_2 = 2d = 2 \times 27 = 54\ mm$

The total length of the rod:

$l_b = l_1 + 6.4r + 3.5l_2 = 540 + 6.4 \times 81 + 3.5 \times 54 = 1247.4\ mm$

Using the following formula given in the CTICM Eurocode guide [1].

The total length of the rod required is:

$$l_{b.rqd} = 0.144d\frac{f_{ub}}{f_{bd}}$$

f_{ub} : ultimate strength of the anchor bolt.

f_{bd} : computational bond stress.

d : diameter of the anchor bolt.

Calculation of the adhesion stress f_{bd} :

Class concrete foundation C25/30:

$f_{ck} = 25\ N/mm^2$: compressive strength of concrete.

$\gamma_c = 1.15$: partial safety factor.

$$f_{bd} = \frac{0.36\sqrt{f_{ck}}}{\gamma_c} = \frac{0.36\sqrt{25}}{1.15} = 1.57\ N/mm^2$$

$$l_{b.rqd} = 0.144 \times 27 \times \frac{400}{1.57} = 990.57\ mm$$

The tensile anchoring resistance of an anchor rod is:

$$F_{anc.Rd} = \pi.d.l_{b.rqd}.f_{bd}$$

$$F_{anc.Rd} = \pi \times 27 \times 990.57 \times 1.57 = 131916N \approx 131.19KN$$

$$N_{Sd}/2 = 23.13/2 = 11.56 < F_{anc.Rd} = 131.19KN \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots..ok$$

IV.1 CALCULATION OF FOUNDATIONS:

The foundations of a construction are the parts of the structure which are in direct contact with the ground. They transmit the loads from the superstructure to the ground, which is why they constitute a very important part since their good design and construction results in the good resistance of the entire structure. The sizing of the foundations is made according to the Reinforced Concrete at Limit States RCLS 91 regulation.

IV.1.1 Load to be taken into consideration:

- **Ultimate limit states ULS**

$\mathrm{M_u} = 65.497\ \mathrm{KN.m}$

$N_u = 46.617 \times \sin 10 + 15.266 \times \cos 10 = 23.13\ \mathrm{KN}$

- **Service limit states SLS**

$\mathrm{M_s} = 16.84\ \mathrm{KN.m}$

$N_s = 9.37\ \mathrm{KN}$

$\bar{\sigma}_{sol} = 2\ \mathrm{bar} = 0.2\ \mathrm{MPa} = 20000\ \mathrm{daN/m^2}$

IV.1.2 Choice of the type of foundation:

- **Insulated sole:**

The sole is dimensioned under the force "N" and the value of the allowable soil stress.

$$\frac{N_u}{S} \leq \sigma_{sol}$$

With:

N_u : normal effort in the ultimate state

S : surface of the sole in contact with the ground.

Σsol : admissible stress of the soil.

A : small dimension of the sole.

B : large dimension of the sole.

$$\frac{N_u}{S} \leq \sigma_{sol} \quad \mapsto \quad \frac{N_u}{\sigma_{sol}} \leq S$$

According to the relation relating to the homothetic of the dimensions of the plate of the sole we have:

$$\frac{a}{b} = \frac{A}{B} \quad \mapsto \quad A.b = B.a \quad \mapsto \quad A = \frac{B.a}{b}$$

$$S = A.B$$

According to the inequality of the justification of the ultimate state of resistance vis-à-vis the ground.

$$\frac{N_u}{S} \leq \sigma_{sol} \quad \mapsto \quad \frac{N_u}{\sigma_{sol}} \leq B^2 \quad \mapsto \quad B \geq \sqrt{\frac{N_u}{\sigma_{sol}}}$$

The sizing of the sole section will be done according to ROBOT software.

$a = b = 45$ cm $\mapsto$ $A = B$ $\mapsto$ square sole

- $\mathrm{M}_{\mathrm{u\,max}} = 65.497$ KN. m
- $N_{u\,max} = 23.13$ KN

Calculation of the length (B) of the sole:

$$B \geq \sqrt{\frac{N_u}{\sigma_{sol}}} \quad \mapsto \quad \sqrt{\frac{2313}{20000}} = 0.34$$

We adopt: $B = 1.60$ m

By homothetic:

$$\frac{a}{b} = \frac{A}{B} \quad \mapsto \quad A.b = B.a \quad \mapsto \quad A = \frac{B.a}{b} = \frac{1.60 \times 0.45}{0.45} = 1.60 \text{ m}$$

We adopt: $A = 1.60$ m

IV.1.3 Calculation of the sole height (h):

$$d = \frac{B - b}{4} \quad \mapsto \quad \frac{160 - 45}{4} = 28.75\ cm$$

We adopt: $d = 30\ cm$

$$h = d + d' = 30 + 5 = 35 \text{ cm}$$

$$h - d' < B - b \quad \mapsto \quad 35 - 5 = 40 \text{ cm} < B - b = 160 - 45 = 115 \text{ cm} \,.\,.\text{CV}$$

$$e_0 = \frac{M_U}{N_U} \leq \frac{B}{6} \quad \mapsto \quad e_0 = \frac{65.497}{23.13} = 2.83 \text{ m } > \frac{1.6}{6} = 0.27 \text{ m}$$

→ Triangular diagram

So, we verifier:

$$\sigma_2 = \frac{2N}{3\left(\frac{B}{2} - e_0\right) \times B} \leq \sigma_{sol}$$

$$\frac{2 \times 2313}{3\left(\frac{1.6}{2} - 2.83\right) \times 1.6} = 474.75 \text{ daN/m}^2 \leq \sigma_{sol} = 20000 \text{ daN/m}^2 \text{ ….……….. ok}$$

IV.1.4 Calculation of reinforcement

The calculation is done at ULS and verification at SLS:

For (A’), we will use the" console "method.

$$\sigma_d = \frac{B + 0.35b - 3e_0}{3 \times (0.5B - e_0)} \times \sigma_2$$

$$\sigma_d = \frac{1.6 + 0.35 \times 0.45 - 3 \times 2.83}{3 \times (0.5 \times 1.6 - 2.83)} \times 474.75 = 524.84 \text{ daN/m}^2$$

$$M_d = B\left(\frac{B}{2} - 0.35 \times b\right)^2 \times \left(\frac{\sigma_d + 2\sigma_2}{6}\right)$$

$$M_d = 1.6\left(\frac{1.6}{2} - 0.35 \times 0.45\right)^2 \times \left(\frac{524.84 + 2 \times 474.75}{6}\right) = 162.29 \; daN.m$$

$$A' = \frac{M_d}{z.\sigma_{bc}}$$

with $z = 0.9 \times d = 0.9 \times 30 = 27 \text{ cm} = 0.27 \text{ m}$

$$A' = \frac{389.37 \times 10^2}{27 \times 3478.2} = 4.14 \; cm^2$$ so we adopte $A' = 9 \; HA \; 10 = 7.07cm^2$

For (A), we will use the connecting rod method with a fictitious load… (Q)

$$\emptyset = N_u\left(1 + \frac{3e_0}{B}\right) = 2313 \times \left(1 + \frac{3 \times 2.83}{1.6}\right) = 14586.35 \text{ daN}$$

$$A = \frac{\emptyset(A-a)}{8.\mathrm{d}.\sigma_{bc}} = \frac{14586.35 \times (160-45)}{8 \times 30 \times 3478.2} = 20.09\ cm^2$$

so we adopte $A^{'} = 10\ HA\ 16 = 20.1\ cm^2$

IV.1.4.1 Verification of reinforcement:

$$e_0 = \frac{M_s}{N_s} \leq \frac{B}{6} \quad \Rightarrow\ e_0 = \frac{16.84}{9.37} = 1.79\ \mathrm{m} \geq \frac{1.6}{6} = 0.26\ \mathrm{m}$$

→ Triangular diagram

So we verifier:

$$\sigma_2 = \frac{2N}{3\left(\frac{B}{2} - e_0\right) \times B} \leq \sigma_{sol}$$

$$\frac{2 \times 937}{3\left(\frac{1.6}{2} - 1.79\right) \times 1.6} = 394.36\ \mathrm{daN/m^2} \leq \sigma_{sol} = 20000\ \mathrm{daN/m^2}\ \ldots\ldots\ldots\ldots \mathrm{ok}$$

$$\sigma_d = \frac{B + 0.35b - 3e_0}{3 \times (0.5B - e_0)} \times \sigma_2$$

$$\sigma_d = \frac{1.6 + 0.35 \times 0.45 - 3 \times 1.79}{3 \times (0.5 \times 1.6 - 1.79)} \times 394.36 = 479.67\ \mathrm{daN/m^2}$$

$$M_d = B\left(\frac{B}{2} - 0.35 \times b\right)^2 \times \left(\frac{\sigma_d + 2\sigma_2}{6}\right)$$

$$M_d = 1.6\left(\frac{1.6}{2} - 0.35 \times 0.45\right)^2 \times \left(\frac{479.67 + 2 \times 394.36}{6}\right) = 139.62\ daN.m$$

$$A^{'}_{ser} = \frac{M_d}{z.\sigma_s}$$

$$A^{'}_{ser} = \frac{139.62 \times 10^2}{27 \times 3478.2} = 1.48\ cm^2 \leq 7.07 cm^2\ \ldots\ldots\ldots\ldots CV$$

$$\emptyset = N_s\left(1 + \frac{3e_0}{B}\right) = 937 \times \left(1 + \frac{3 \times 1.79}{1.6}\right) = 4081.81\ \mathrm{daN}$$

$$A_{ser} = \frac{\emptyset(A-a)}{8.\mathrm{d}.\sigma_s} = \frac{4081.81 \times (160-45)}{8 \times 30 \times 3478.2} = 5.62\ cm^2 \leq 20.1\ cm^2$$

IV.2 CALCULATION OF LONGRINS:

The role of the outriggers is to connect the soles together, they are subjected to a tensile force. A outrigger is placed directly on a clean concrete to prevent pollution of the fresh concrete of the outrigger by the support soil during the pouring of the concrete. The clean concrete also provides uniform support for the outrigger.

- **Sizing of outriggers:**

According to RPA99, for a type S3 floor the minimum dimensions of the cross section of the outriggers are: 45 cm x 30 cm.

IV.2.1 Calculation of reinforcement:

The outriggers must be designed to resist traction under the action of a force equal to

$$F = \max(N/a\,; 20\ KN)$$

With:

N: Equal to the maximum value of the vertical gravity loads brought by the points solidified support.

a: Coefficient depending on the seismic zone and the category of site considered, for soils S3 and seismic zone 0 (a = 12).

- **ULS:**

$N_u = 23.13$ KN

$F = \max(N/a\,; 20\ KN) = 20\ KN$

IV.2.1.1 Reinforcement of stringers

RPA99 requires a minimum section

$A_s = 0.6\%\ B = (0.6/100)(45 \times 30) = 8.10\ cm^2$

we adopt: 6 T16=12.06

IV.2.1.2 Fragility name condition:

$A_s \geq 0.23(f_t/f_e)bd$

$A_s \geq 0.23 \times (2.1/400) \times 30 \times 43 = 1.55cm^2$

we adopt: $A_s \geq 1.55cm^2$ … ok

IV.2.1.3 Frame spacing:

$S_t \leq \min(20\ cm\ , 15\emptyset cm)\ \mapsto\ \ S_t \leq \min(20\ cm\ , 15 \times 1.6cm) = 20cm$

we adopt: $S_t = 15\ cm$

IV.2.1.4 The transverse reinforcements:

We choose flat-rate: $\emptyset_t\ = 8$ mm

$A_s = 2.01cm^2$

GENERAL CONCLUSION

This modest work gave us an opportunity to apply and deepen all our knowledge acquired during the course of our master's degree in civil engineering.

The design of a metallic structure is based on the sizing at ultimate limit states taking into account the most severe surrounding actions such as operating overloads, snow, wind and earthquake.

This work consists in studying and dimensioning a metal frame hangar with an overhead crane, designed in regular form. After having defined the loads acting on the structure, the columns, sand beams, trusses, bracings, stabilities, purlins, posts and cladding rails as elements of the structure were dimensioned.

This dimensioning concerns each element, assembly, connection or sensitive part of the construction. Precision and rigor in the calculations and verification on the one hand and the exact definition of the various construction details are required.

This experience also allowed us to better understand the field of steel frame construction, which allowed us on the one hand to assimilate the different techniques and calculation software as well as the regulations governing the principles of design and calculation of structures. in this field, and developed the ideas thanks to the reading of the various bibliographical references.

At the end of this project, which constitutes for us a first experience in this vast field, it acquires us very important values to put the first step in my future professional life

Table of results

Geometric data	• length of the structure: **21m** • Width of the structure: **11m** • Total height: **5.20m**
The loads and live loads applied	• **G** = 0.91 KN/m • **W** = -0.625 KN/m • **S** = 0.31 KN/m
purlin study	• The profile chooses UPN 140 suitable for purlins.
Calculation of the side wall girts	• The profile chooses UPN 140 suitable for purlins.
Calculation of the Post	• **HEA 140** is suitable as a post.
Calculation of bracing	• we adopt a cornier: L.60 × 60 × 6
calculation of the eave strut	• we opt for a **HEA 120** for the beam strut
Calculation of the cross Strut	• The chosen profile IPE 270 is suitable as a cross member.
Calculation of the column	• The chosen profile HEA 220 is suitable as a column.
Assembly Column Rafter	• We choose bolts of class HR 10.5 • Bolt diameter d = 20 mm (Number of bolts = 8; Number of queues = 2) • Plate (h = 450 mm; b = 200 mm; t = 15 mm)
Assembly Rafter – Rafter	• **Assembly Rafter – Rafter:** • We choose bolts of class HR 10.5 • Bolt diameter d = 20 mm (Number of bolts = 8; Number of queues = 2) • Plate (h = 450 mm; b = 200 mm; t = 10 mm)
anchor rods	• For two M27 rods in class 4.6. • A_s= 425 mm² / f_{ub}= 400 N/mm²/ f_{yb}=240 N/mm² • h = 27
Calculation of Foundations	• (A=1.60m ; B=1.60m ; a= 45 cm ; b=45cm) • A= 10 HA 16 ; A’= 9 HA 10
Calculation of Longrins	• a*b = (45*30) cm² • A= 6T16 ; ø=8

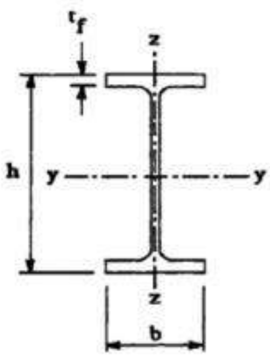
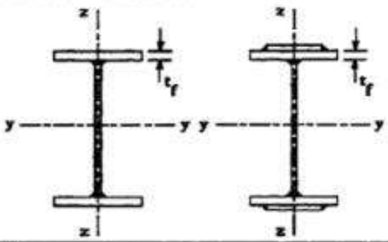
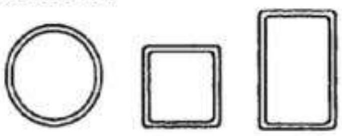
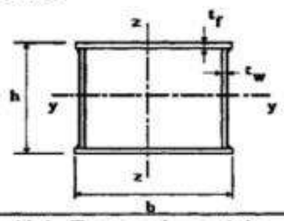
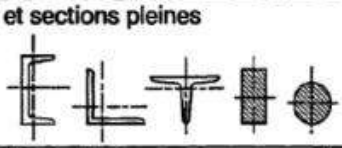
APPENDICES 1

Reinforcement table

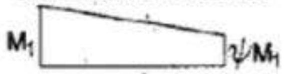
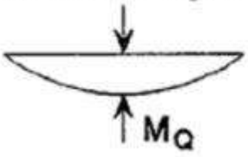
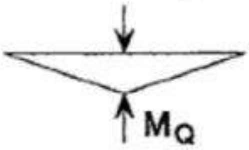
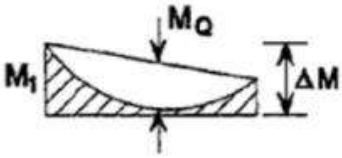
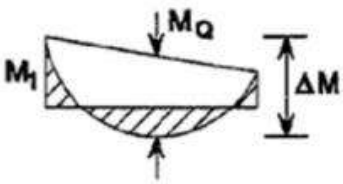
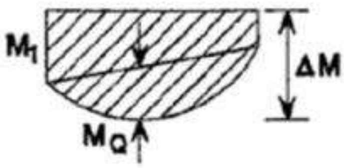
Φ (mm)	5	6	8	10	12	14	16	20	25	32	40
1	0,20	0,28	0,50	0,79	1,13	1,54	2,01	3,14	4,91	8,04	12 , 57
2	0,39	0,57	1,01	1,57	2,26	3,08	4,02	6,28	9,82	16,08	25 , 13
3	0,59	0,85	1,51	2,36	3,39	4,62	6,03	9,42	14,73	24,13	37 , 7
4	0,79	1,13	2,01	3,14	4,52	6,16	8,04	12,57	19,64	32,17	50 , 27
5	0,98	1,41	2,51	3,93	5,65	7,72	10,05	15,71	24,54	40,21	62 , 83
6	1,18	1,70	3,02	4,71	6,79	9,24	12,06	18,85	29,45	48,25	75 , 40
7	1,37	1,98	3,52	5,50	7,92	10,78	14,07	21,99	34,36	56,30	87 , 96
8	1,57	2,26	4,02	6,28	9,05	12,32	16,08	25,13	39,27	64,34	100,53
9	1,77	2,54	4,52	7,07	10,18	13,85	18,10	28,27	44,18	72,38	113,10
10	1,96	2,83	5,03	7,85	11,31	15,39	20,11	31,42	49,09	80,42	125,66
11	2,16	3,11	5,53	8,64	12,44	16,93	22,12	34,56	54,00	88,47	138,23
12	2,36	3,39	6,03	9,42	13,57	18,47	24,13	37,70	58,91	96,51	150,80
13	2,55	3,68	6,53	10,21	14,70	20,01	26,14	40,84	63,81	104,55	163,36
14	2,75	3,96	7,04	11,00	15,38	21,55	28,15	43,98	68,72	112,59	175,93
15	2,95	4,24	7,54	11,78	16,96	23,09	30,16	47,12	73,63	120,64	188,50
16	3,14	4,52	8,04	12,57	18,10	24,63	32,17	50,27	78,54	128,68	201,06
17	3,34	4,81	8,55	13,35	19,23	26,17	34,18	53,41	83,45	136,72	213,63
18	3,53	5,09	9,05	14,14	20,36	27,71	36,19	56,55	88,36	144,76	226,20
19	3,73	5,37	9,55	14,92	21,49	29,25	38,20	59,69	93,27	152,81	238,76
20	3,93	5,65	10,05	15,71	22,62	30,79	40,21	62,83	98,17	160,85	251,33

APPENDICES 2

Tableau 5.5.3 Choix de la courbe de flambement correspondant à une section

Type de Section	limites	axe de flambement	courbe de flambement
Sections en I laminées 	$h / b > 1,2$: $t_f \leq 40$ mm	y - y z - z	a b
	$40 \text{ mm} < t_f \leq 100$ mm	y - y z - z	b c
	$h / b \leq 1,2$: $t_f \leq 100$ mm	y - y z - z	b c
	$t_f > 100$ mm	y - y z - z	d d
Sections en I soudées 	$t_f \leq 40$ mm	y - y z - z	b c
	$t_f > 40$ mm	y - y z - z	c d
Sections creuses 	laminées à chaud	quel qu'il soit	a
	formées à froid - en utilisant f_{yb} *)	quel qu'il soit	b
	formées à froid - en utilisant f_{ya} *)	quel qu'il soit	c
Caissons soudés 	d'une manière générale (sauf ci-dessous)	quel qu'il soit	b
	Soudures épaisses et $b / t_f < 30$, $h / t_w < 30$	y - y z - z	c c
Sections en U, L, T et sections pleines 		quel qu'il soit	c

*) Voir 5.5.1.4 (4) et figure 5.5.2

Diagramme des moments	Facteur de moment uniforme équivalent β_M
Moments d'extrémité  $-1 \le \psi \le 1$	$\beta_{M,\psi} = 1{,}8 - 0{,}7\psi$
Moment crée par des forces latérales dans le plan  	$\beta_{M,Q} = 1{,}3$ $\beta_{M,Q} = 1{,}4$
Moment créé par des forces latérales dans le plan et des moments d'extrémité   	$\beta_M = \beta_{m,\psi} + \frac{M_Q}{\Delta M}\left(\beta_{M,Q} - \beta_{M,\psi}\right)$ $M_Q = \lvert MaxM \rvert$ dû aux charges transversales seulement $\Delta M = \begin{cases} \lvert \max M \rvert & \text{pour diagrammes de moment sans changement de signe} \\ \lvert \max M \rvert + \lvert \min M \rvert & \text{pour diagrammes de moment avec changement de signe} \end{cases}$

APPENDICES 3

profile tables

Compression		$(N_{Sd} / N_{pl\,Rd}) = 0.25$								$(N_{Sd} / N_{pl\,Rd}) = 0.5$							
Profil Poteau	Résistance	Effort Axial	Platine (mm)				Fondation (mm)			Effort Axial	Platine (mm)				Fondation (mm)		
	$N_{pl\,Rd}$ (kN)	N_{Sd} (kN)	h_p	b_p	t_p	Proj.	h_r	b_r	d_r	N_{Sd} (kN)	h_p	b_p	t_p	Proj.	h_r	b_r	d_r
HEA100	499	125	115	120	8	C	175	180	100	250	140	140	10	E	210	210	100
HEA120	595	149	130	140	8	C	195	210	100	298	155	160	10	E	235	240	100
HEA140	738	185	150	160	8	C	225	240	100	369	180	185	10	E	270	280	100
HEA160	911	228	170	180	8	C	255	270	100	456	200	210	12	E	300	315	100
HEA180	1063	266	190	200	8	C	285	300	100	532	220	230	12	E	330	345	110
HEA200	1265	316	210	220	8	C	315	330	105	633	245	255	12	E	370	385	125
HEA220	1512	378	235	245	8	C	355	370	120	756	270	280	14	E	405	420	135
HEA240	1806	451	255	265	8	C	385	400	130	903	295	305	16	E	445	460	150
HEA260	2040	510	275	285	8	C	415	430	140	1020	315	325	16	E	475	490	160
HEA280	2286	571	300	310	8	C	450	465	150	1143	300	310	28	C	450	465	150
HEA300	2644	661	320	330	8	C	480	495	160	1322	320	330	30	C	480	495	160
HEA320	2923	731	345	335	10	C	520	505	175	1461	390	380	18	E	585	570	195
HEA340	3127	784	365	335	10	C	550	505	185	1568	415	385	20	E	.625	580	210
HEA360	3355	839	385	335	10	C	580	505	195	1677	435	385	20	E	655	580	220
HEA400	3736	934	430	340	10	C	645	510	215	1868	485	395	22	E	730	595	245
HEA450	4184	1046	485	345	10	C	730	520	245	2092	540	400	24	E	810	600	270
HEA500	4642	1161	540	350	12	C	810	525	270	2321	595	405	24	E	895	610	300
HEA550	4976	1244	590	350	12	C	885	525	295	2488	590	350	38	C	885	525	295
HEA600	5322	1330	640	350	12	C	960	525	320	2661	640	350	38	C	960	525	320
HEA650	5678	1420	695	355	12	C	1045	535	350	2839	695	355	38	C	1045	535	350
HEA700	6121	1530	745	355	12	C	1120	535	375	3061	745	355	40	C	1120	535	375
HEA800	6717	1679	850	360	12	C	1275	540	425	3358	850	360	38	C	1275	540-	425
HEA900	7532	1883	950	360	12	C	1425	550	475	3766	950	360	40	C	1425	540	475

Compression		$(N_{Sd} / N_{pl\,Rd}) = 0.25$								$(N_{Sd} / N_{pl\,Rd}) = 0.5$							
Profil Poteau	Résistance	Effort Axial	Platine (mm)				Fondation (mm)			Effort Axial	Platine (mm)				Fondation (mm)		
	$N_{pl\,Rd}$ (kN)	N_{Sd} (kN)	h_p	b_p	t_p	Proj.	h_r	b_r	d_r	N_{Sd} (kN)	h_p	b_p	t_p	Proj.	h_r	b_r	d_r
IPE80	180	45	95	60	8	C	145	90	100	90	105	75	8	E	160	115	100
IPE100	243	61	115	70	8	C	175	105	100	121	130	85	8	E	195	130	100
IPE120	310	78	135	80	8	C	205	120	100	155	150	95	8	E	225	145	100
IPE140	386	97	155	90	8	C	235	135	100	193	175	105	8	E	265	160	100
IPE160	472	118	175	100	8	C	265	150	100	236	195	120	8	E	295	180	100
IPE180	563	141	200	110	8	C	300	165	100	261	220	130	10	E	330	195	110
IPE200	669	167	220	120	8	C	330	180	110	335	240	140	10	E	360	210	120
IPE220	784	196	240	130	8	C	360	195	120	392	265	155	10	E	400	235	135
IPE240	919	230	260	140	8	C	390	210	130	460	290	170	12	E	435	255	145
IPE270	1080	270	295	160	8	C	445	240	150	540	295	160	18	C	445	240	150
IPE300	1265	316	325	175	8	C	490	265	165	632	325	175	20	C	490	265	165
IPE330	1471	368	355	185	8	C	535	280	180	736	355	185	20	C	535	280	180
IPE360	1709	427	390	200	8	C	585	300	195	855	390	200	22	C	585	300	195
IPE400	1985	496	430	210	8	C	645	315	215	992	430	210	22	C	645	315	215
IPE450	2322	581	480	220	8	C	720	330	240	1161	480	220	24	C	720	330	240
IPE500	2715	679	535	235	8	C	805	355	270	1357	535	235	26	C	805	355	270
IPE550	3159	790	585	245	8	C	880	370	295	1579	585	245	28	C	880	370	295
IPE600	3666	916	640	260	10	C	960	390	320	1833	640	260	28	C	960	390	320

APPENDICES 4

reduction coefficient value χ_{ksi}

buckling curve a:

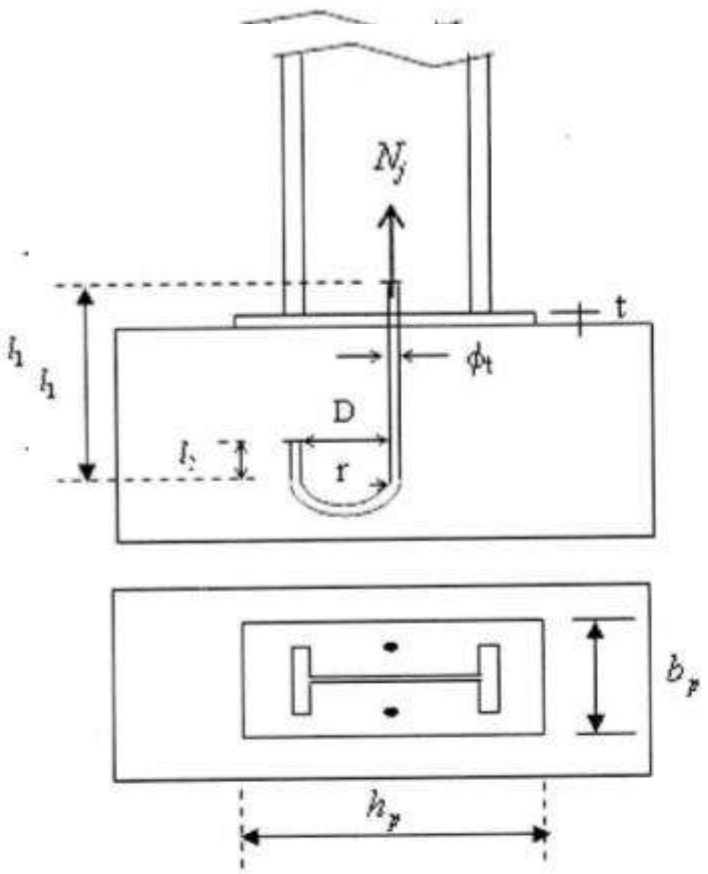
Reduction factors χ for buckling curve a (α = 0.21)											
$\bar{\lambda}$	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09	
0.00	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	0.00
0.10	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	0.10
0.20	1.0000	0.9978	0.9956	0.9934	0.9912	0.9889	0.9867	0.9844	0.9821	0.9798	0.20
0.30	0.9775	0.9751	0.9728	0.9704	0.9680	0.9655	0.9630	0.9605	0.9580	0.9554	0.30
0.40	0.9528	0.9501	0.9474	0.9447	0.9419	0.9391	0.9363	0.9333	0.9304	0.9273	0.40
0.50	0.9243	0.9211	0.9179	0.9147	0.9114	0.9080	0.9045	0.9010	0.8974	0.8937	0.50
0.60	0.8900	0.8862	0.8823	0.8783	0.8742	0.8700	0.8657	0.8614	0.8569	0.8524	0.60
0.70	0.8477	0.8430	0.8382	0.8332	0.8282	0.8230	0.8178	0.8124	0.8069	0.8014	0.70
0.80	0.7957	0.7899	0.7841	0.7781	0.7721	0.7659	0.7597	0.7534	0.7470	0.7405	0.80
0.90	0.7339	0.7273	0.7206	0.7139	0.7071	0.7003	0.6934	0.6865	0.6796	0.6726	0.90
1.00	0.6656	0.6586	0.6516	0.6446	0.6376	0.6306	0.6236	0.6167	0.6098	0.6029	1.00
1.10	0.5960	0.5892	0.5824	0.5757	0.5690	0.5623	0.5557	0.5492	0.5427	0.5363	1.10
1.20	0.5300	0.5237	0.5175	0.5114	0.5053	0.4993	0.4934	0.4875	0.4817	0.4760	1.20
1.30	0.4703	0.4648	0.4593	0.4538	0.4485	0.4432	0.4380	0.4329	0.4278	0.4228	1.30
1.40	0.4179	0.4130	0.4083	0.4036	0.3989	0.3943	0.3898	0.3854	0.3810	0.3767	1.40
1.50	0.3724	0.3682	0.3641	0.3601	0.3561	0.3521	0.3482	0.3444	0.3406	0.3369	1.50
1.60	0.3332	0.3296	0.3261	0.3226	0.3191	0.3157	0.3124	0.3091	0.3058	0.3026	1.60
1.70	0.2994	0.2963	0.2933	0.2902	0.2872	0.2843	0.2814	0.2786	0.2757	0.2730	1.70
1.80	0.2702	0.2675	0.2649	0.2623	0.2597	0.2571	0.2546	0.2522	0.2497	0.2473	1.80
1.90	0.2449	0.2426	0.2403	0.2380	0.2358	0.2335	0.2314	0.2292	0.2271	0.2250	1.90
2.00	0.2229	0.2209	0.2188	0.2168	0.2149	0.2129	0.2110	0.2091	0.2073	0.2054	2.00
2.10	0.2036	0.2018	0.2001	0.1983	0.1966	0.1949	0.1932	0.1915	0.1899	0.1883	2.10
2.20	0.1867	0.1851	0.1836	0.1820	0.1805	0.1790	0.1775	0.1760	0.1746	0.1732	2.20
2.30	0.1717	0.1704	0.1690	0.1676	0.1663	0.1649	0.1636	0.1623	0.1610	0.1598	2.30
2.40	0.1585	0.1573	0.1560	0.1548	0.1536	0.1524	0.1513	0.1501	0.1490	0.1478	2.40
2.50	0.1467	0.1456	0.1445	0.1434	0.1424	0.1413	0.1403	0.1392	0.1382	0.1372	2.50
2.60	0.1362	0.1352	0.1342	0.1332	0.1323	0.1313	0.1304	0.1295	0.1285	0.1276	2.60
2.70	0.1267	0.1258	0.1250	0.1241	0.1232	0.1224	0.1215	0.1207	0.1198	0.1190	2.70
2.80	0.1182	0.1174	0.1166	0.1158	0.1150	0.1143	0.1135	0.1128	0.1120	0.1113	2.80
2.90	0.1105	0.1098	0.1091	0.1084	0.1077	0.1070	0.1063	0.1056	0.1049	0.1042	2.90
3.00	0.1036	0.1029	0.1022	0.1016	0.1010	0.1003	0.0997	0.0991	0.0985	0.0978	3.00
3.10	0.0972	0.0966	0.0960	0.0954	0.0949	0.0943	0.0937	0.0931	0.0926	0.0920	3.10
3.20	0.0915	0.0909	0.0904	0.0898	0.0893	0.0888	0.0882	0.0877	0.0872	0.0867	3.20
3.30	0.0862	0.0857	0.0852	0.0847	0.0842	0.0837	0.0832	0.0828	0.0823	0.0818	3.30
3.40	0.0814	0.0809	0.0804	0.0800	0.0795	0.0791	0.0786	0.0782	0.0778	0.0773	3.40
3.50	0.0769	0.0765	0.0761	0.0757	0.0752	0.0748	0.0744	0.0740	0.0736	0.0732	3.50
3.60	0.0728	0.0724	0.0721	0.0717	0.0713	0.0709	0.0705	0.0702	0.0698	0.0694	3.60

APPENDICES 5

values of the normal admissible forces, N_j^{max} (daN) per anchoring rod and their diameters given according to the characteristics of the rod and the anchoring depth:

ϕ_t	D	l_1	l_2	l_f	N_j^{max}
16	40	280	25	120	2170
20	50	280	32	120	3040
20	50	480	32	120	4420
24	70	500	40	160	6070
30	90	500	50	160	8580
33	100	700	55	160	12260

l_f : thread length.



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