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Presented by: Abdelkarim Khelfaoui

Subject:

Study Of a Metal Shed For Parking

In front of the jury composed of :

DJIREB SAMIR.	MCB (UKM Ouargla)	President
KHELASSI AMAR	MAA (UKM Ouargla)	Examiner
LOKMANE ABDELDJOUAD	MCB (UKM Ouargla)	Supervisor

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Thanks

First of all, I thank ALLAH, the almighty for giving me the strength, courage, and the will to carry out this modest work.

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Dedication

I dedicate this modest work to the beings who are the

most expensive to my heart:

- *My dear parents, symbols of courage and will, who have dedicated and sacrificed their lives for our wellbeing. may God protect them.*
- *My dear brothers*
- *All my families big and small.*
- *All my friends*
- *All STEACHER.Ltd engineers*
- *All HAOUD BERKAOUI engineers*
- *I also dedicate to the term recognition To all my Class of civil engineering, Structure and VOA, and specially for ECBR 2023*

Abstract

The end of studies project represents the last phase of my training, it allowed me to know the used regulations in force. The complexity of calculations in civil engineering inevitably calls on the services of digital tools such as AUTODESK ROBOT STRUCTURAL ANALYSIS, it saves time, and gives precision, and reliability, the metal hangars Are a metal frame, sizing of the secondary and main elements is done according to climatic forces, most often winds, so that we rely on the following calculation rules: RNV99/2013 and C.C.M97, La Structure stability also depends on the correct assembly so that the building can withstand the forces that affect it.

This study allowed me to conclusions that the modeling must be as close as possible to reality, to approach the real behavior of the structure and obtain better results. In metallic structures, the wind's effects on metallic structures are frequently the worst.

Résumé

Le projet de fin d'études représente la dernière phase de ma formation, il m'a permis de connaître les réglementations en vigueur utilisées. La complexité des calculs en génie civil fait inévitablement appel aux services d'outils numériques tels que AUTODESK ROBOT STRUCTURAL ANALYSIS, cela fait gagner du temps, et donne précision, et fiabilité, les hangars métalliques sont à ossature métallique, le dimensionnement des éléments secondaires et principaux se fait en fonction des forces climatiques, le plus souvent des vents, de sorte que l'on s'appuie sur les règles de calcul suivantes : RNV99/2013 et C.C.M97, La stabilité de la structure dépend également du bon assemblage pour que le Structure puisse résister aux forces qui l'affectent.

Cette étude m'a permis de conclure que la modélisation doit être la plus proche possible de la réalité, pour se rapprocher du comportement réel de la structure et obtenir de meilleurs résultats. Dans les structures métalliques, les effets du vent sur les structures métalliques sont souvent les pires.

ملخص

يمثل مشروع نهاية الدراسة المرحلة الأخيرة من تدريبي ، فقد تعرفت على الأنظمة المعمول بها ، وهي DTR و RPA99 الإصدار 2003 وRNVA2013 ، واستخدمت برنامج AUTODESK ROBOT STRUCTURAL ANALYSIS ، حيث يوفر الوقت ويعطي الدقة والموثوقية، ويتم تحديد حجم العناصر الثانوية والرئيسية وفقًا للقوى المناخية ، غالبًا الرياح ، بحيث نعتمد

على قواعد الحساب التالية: RNV99 / 2013 و CCM97 ، يعتمد استقرار الهيكل أيضًا على التجميع الصحيح بحيث يمكن للمبنى تحمل القوى التي تؤثر عليه.

سمحت لي هذه الدراسة باستنتاجات مفادها أن النمذجة يجب أن تكون أقرب ما يمكن إلى الواقع ، لمقاربة السلوك الحقيقي للهيكل والحصول على نتائج أفضل، كما انه غالبًا ما تكون تأثيرات الرياح على الهياكل المعدنية هي الأسوأ ، ومع ذلك ، فإن الدراسة الزلزالية ليست مهملة ، والتحقق من الإطار ضد عدم الاستقرار هو خطوة ضرورية لتحديد حجم العناصر. يعد التصميم الجيد للتجميع ضروريًا لاستقرار الهياكل المعدنية.

Summary

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Notation and symbols

Sollicitations-Contraintes-Déformations :

E_a : Module d'élasticité longitudinale de l'acier (MPa).

ν : Coefficient de poisson pour l'acier.

G : Module d'élasticité transversale de l'acier (MPa).

F_p : Effort de précontrainte dans un boulon (kN).

M_{sd} : Moment sollicitant maximum (kN.m).

M_{rd} : Moment résistant (KN.m).

M_{st} : Moment stabilisateur

M_{cr} : Moment critique élastique de déversement.

N_{sd} : Effort normal due aux charges verticales (KN).

N_u : Effort normal pondéré (KN).

N_{rd} : Effort normal résistant (KN).

N_{pl} : Effort normal de plastification (KN).

$W_{ply.Z}$: Module plastique de la section

$W_{ely.Z}$: Module élastique de la section.

W_{eff} : Module élastique efficace de la section.

M_{ply} : Moment résistant plastique de la section.

$M_{ely.Z}$: Moment résistant élastique de la section.

V_{sd} : Effort tranchant sollicitant (KN).

V_{pl} : Effort tranchant de plastification (KN).

V_u : Effort tranchant de calcul ultime.

F : Flèche d'une poutre (mm).

F_{adm} : Flèche admissible (mm).

f_y : Contrainte limite d'élasticité d'un acier (MPa).

f_u : Contrainte de rupture d'une pièce (MPa).

f_{ub} : Contrainte de rupture d'un boulon (MPa).

ξ_y : Déformation correspondant à la contrainte limite d'élasticité (%).

ξ **:** allongement relatif (déformation %).

f_{yb} : Résistance limite d'élasticité d'un boulon (MPa).

σ **:** Contrainte normale (MPa).

τ : Contrainte tangentielle ou de cisaillement (MPa).

Caractéristiques Géométriques

A : Section brute d'une pièce (cm^2).

A_{net} : Section nette d'une pièce (cm^2).

A_V : Aire de cisaillement (cm^2).

I_Y : Moment d'inertie de flexion maximal (cm^4).

a : Épaisseur utile (ou gorge) d'un cordon de soudure (mm).

b : Largeur d'une semelle d'une poutre (mm).

h : Hauteur d'une pièce en générale (mm).

A_s : Section d'armature de béton (cm^2).

L : Longueur, ou portée d'une poutre(m).

L_{cr} : Longueur critique (m).

t : Épaisseur d'une pièce ou d'une tôle (mm).

t_f : Épaisseur d'une semelle de poutre (mm).

t_w : Épaisseur d'une âme de poutre (mm).

i : Rayon de giration d'une section (mm).

L_f : Longueur de flambement (mm).

Φ : Diamètre d'une armature transversale.

G : Action permanente.

Q : Action d'exploitation.

e : L'excentricité de l'effort normal.

Coefficients et grandeurs sans dimensions

n : Nombre de connecteur répartir sur une longueur critique.

P : Nombre de plans de cisaillement ou de frottement.

n : Coefficient d'équivalence acier-béton.

C_{pi} : Coefficient de pression intérieur.

C_{pe} : Coefficient de pression extérieur.

K : Coefficient de flambement.

K_y, K_z : Coefficient de flambement-flexion.

β_M : Facteur de moment uniforme équivalent (flambement).

λ : Élancement de l'élément.

$\lambda_{y.z}$ **:** Elancement géométrique pour le mode de flambement.

λ_{cr} **:** Elancement critique d'Euler.

μ **:** Coefficient de frottement entre deux pièces en contact.

χ **:** Coefficient de réduction de flambement.

$\chi_{y.z}$ **:** Coefficient de réduction pour le mode de flambement considéré.

λ_{LT} **:** Elancement réduit pour le déversement.

χ_{LT} **:** Coefficient de réduction pour le déversement.

a_{LT} **:** Facteur d'imperfection pour le déversement.

$Q_{My.z}$ **:** Facteur de moment uniforme équivalent pour le flambement.

Q_{MLT} **:** Facteur de moment uniforme équivalent pour le déversement.

y **:** Coefficient partiel de sécurité.

Ψ_s **:** Coefficient de scellement relatif a une armature (psi).

GENERAL INTRODUCTION

General Introduction

The aim of civil engineering studies is to design structures that can withstand various natural phenomena such as earthquakes and extreme winds. This involves creating structural systems that effectively absorb forces and transfer them to the foundations. Current structures are complex, with variable shapes, multiple curvatures, and large spans. Metal constructions have become popular due to their advantages, including reliability, speedy execution, high steel resistance, earthquake resistance due to steel's ductility, and the ability to create extensive architectural structures. Metal constructions are also lightweight, requiring less significant foundations and reducing costs by enabling more efficient site utilization.

However, metal constructions have their drawbacks. They are susceptible to elastic instability due to the thinness of profiles, have poor fire resistance, and require regular maintenance of protective coatings against corrosion for durability. When calculating a structure, it is crucial to ensure elastic stability under all possible combinations of actions defined by regulations (CCM97 or EUROCOD 03). This stability can be assessed at both the overall structure level and the individual element level (e.g., posts, beams).

The end-of-study project at hand involves a comprehensive technical study to dimension and verify a steel construction shed for storage. The study will adhere to relevant regulations and recommendations, including RPA99/2003, BAEL, CCM97, and RNVA2013.

At the beginning of the design, we must determine the nature of the floor to be built on and identify all the forces affecting the structure so that we can design the elements of the structure to resist those forces.

CHAPTER ONE

PROJECT PRESENTATION

I.1. Introduction

The steel structure is a branch of building construction, which remains to our day the most suitable solution, for the construction of industrial structures, or buildings with large spaces generally.

The study of a project is established by taking into consideration its functional aspects, which obligate the civil engineer to be familiar with the mechanical properties and behavior of the materials employed by considering the following parameters:

- Technical advantages
- Aesthetic benefits
- Cost effectiveness

This chapter is devoted to providing some reminders of calculations about the materials used, loads and frameworks.

I.2. Introducing and implementing the project

The project of this study is “steel structure shed” which localed in “In-Amenas”, wilaya of “Illizi”. The shed is on the ground floor only and the roof is composed of a double slope. There are two entrances through gates on the main facade.

The structure has a surface area of 192.00 m^2, and is 12.00 m width, and 16.00 m length with a spacing between main frames of 4.00 m, which makes a total of five (05) main frames.

I.3. Geometric characteristics of the structure

The structure has a rectangular plane characterized by the dimensions as follows:

Total height: H1= 5.70 m.

Column height: H2= 4.50 m.

Roof slope: 20%

Shed length: L1= 16.00 m.

Shed width: L2= 12.00 m.

Main facade cuttings: 2 doors (4.00x3.20 m)

Sidewall cuttings: 8 windows (2.00x0.80 m)

I.4. Elements of the structure

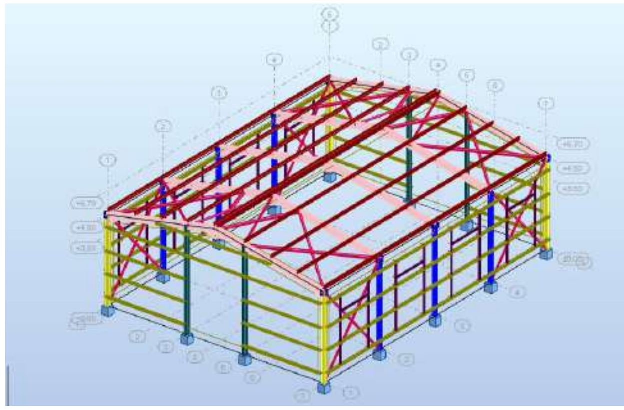

Figure I.4:I View of the building elements

I.4.1. Cladding and Roofing:

The cover is made of ribbed sheet type sandwich panel 30mm thick, length 6 m and width 0.726 m, it will be arranged to use its maximum resistance module.

- Thickness: e = 30mm
- Self-weight: P = 0.20 KN/m2
- Failure stress: f_u = 400 N/mm^2
- Elastic stress: f_y = 160 N/mm2
- Admissible deflection: δ_{max}= l/200
- Modulus of resistance: w = 9.24 cm3/ml
- Moment of inertia: I = 27.21 cm4/ml

I.4.2. Columns and rafters:

Columns and rafters are steel profiles with a large section and they hold all the structure together.

I.4.3. Purlins and girts

Purlins are metal profiles with a constant section and there roll to hold the roofing and the cladding.

I.4.4. Bracings

The bracings in (x) shape stabilize the shed in both directions, ensure the verticality of the columns and take the forces due to the earthquake and the wind by transmitting it to the foundations.

I.5. Soil

Bearing Capacity of Soil: 2.2 bars (gravelly sand).

I.6. Materials used

I.6.1. Concrete

For the construction of the infrastructure and the slabs, the type of cement used is the CPA characterized by the dosage of 350Kg / m^3 whose characteristics are as follows:

Unit weight: $\gamma_{concrete} = 25\ KN/m^3$

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

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

I.6.2. Steel

Table 1 Steel Mechanical properties

Steel grade	S235
Tensile strength f_u	360 MPa
Yield strength f_y	235 MPa
Elasticity modulus E	210000 MPa
Shear modulus G	81000 MPa
Unit weight γ_{steel}	78.50 KN/m^3
Poisson's ratio μ	0.3

I.7. Loads

- Dead loads (G): are the permanent loads of the structure. Dead loads in buildings consist of the weight of the building itself, plus the weight of all fixed items like walls, roof, and carpeting.
- Live loads (Q): are the transient or temporary additional loads imposing additional force on a structure and its foundation.
- Climatic loads (S and W): are the different environmental construction loads taken into inconsideration, such as seismic, snow, and wind loads.

I.8. Rules used

For the study of this project the technical frameworks used are:

- Règles de conception et de calcul des structures en acier (CCM97), document technique réglementaire D.T.R-B.C-2.44.
- Règles parasismique algériennes (RPA99 version 2003) D.T.R-B.C-2.48.
- Charges permanentes et surcharges d'exploitation D.T.R-B.C-22.
- Règles neige et vent (RNVA2013) D.T.R-C2.47.
- Eurocode 1, Eurocode 2 et Eurocode 3.
- Règles de calcul des fondations superficielles D.T.R-B.C-2.33.
- Règle de L'étude de l'infrastructure selon le «BAEL91»

CHAPTER TWO

LOAD ASSESSMENT

II.1. Introduction

In this chapter, we will define the different loads acting on our structure, which are summed up in the action of permanent and operating loads And climatic effects. These have a great influence on the stability of the structure. For this, an in-depth study must be developed to determine of these different actions.

II.2. Permanent load

It includes not only the own weight of the main and secondary structural elements, but also the weight of the elements incorporated into the load-bearing elements such as the roofing and the cladding.

Metal Covers: G1 = 0,2 $KN/_{m^2}$

Dropped ceiling: G2 = 0,17 $KN/_{m^2}$

II.3. operating overload

Gutter: $Q_1 = 0.04\ KN/_{m}$

Sand load: $Q_2 = 0.4\ kN/_{m^2}$

Maintenance: $Q_3 = 2$ agents of 1 kN

maintenance loads which are defined as, point loads of 100 kg at 1/3 and 2/3 of the span of a beam.

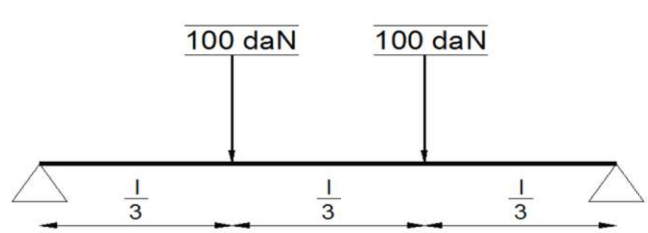

Figure II.3:I Point Live Loads of Beams

The operating costs are determined according to the regulatory technical document loads and overloads (DTR B.C.22).

II.4. Climatic load

Snow: neglected in zone D

II.4.1. Wind load

II.4.1.1. Calculating the dynamic coefficient

The dynamic coefficient: C_d = 1 (Construction less than 15 m)

q_{ref} = 0,58 kN/m^2 (Zone IV) Table 2-2 of RNVA2013 (chapter II-calculation bases)

II.4.1.2. Site effect

The topography coefficient: C_t = 1 (Site plat) (Chapitre II RNVA2013).

Land category: III (Table 2-4 of RNVA2013 (chapiter II- bases de calcul))

Table 2 Land property

K_T	Z_0	Z_{min}	ε
0.215	0.3	5	0.61

II.4.1.3. Determination of the roughness coefficient

$C_r = K_T \times \ln\left(\frac{Z}{Z_0}\right)$ Chapitre II § 2. 4.5 RNV2013 P53.

$(Z_{min} = 5\ m \leq Z = 5.7\ m \leq 200\ m)$

$$C_r = 0.215 \times \ln\left(\frac{5.7}{0.3}\right) = 0.4417$$

II.4.1.4. Wind exposure coefficient

(Formula 2.2 of RNVA2013 (chapiter II- bases de calcul))

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

$I_v = \frac{1}{C_t \times \ln\left(\frac{Z}{Z_0}\right)}$ $(Z = 5.7\ m > Z_{min} = 5\ m)$

$$C_e = 1^2 \times (0.4417)^2 \times \left(1 + 7\left(\frac{1}{1 \times \ln\left(\frac{5.7}{0.3}\right)}\right)\right) = 1 \times 0.400 \times 3.377 = 1.35$$

II.4.1.5. Calculating the pressure

$q_p(z) = q_{ref} \times C_e(z)$ Chapitre II § 2. 3 p50 RNVA2013

$q_p = 0.58 \times 1.35 = 0.783\ kN/m^2$

II.4.1.6. Determination of external pressure coefficient

$C_{Pe} = C_{Pe.10}\ (S \geq 10\ m^2)$

$e = Min(b; 2h) = Min(12; 2 \times 5.7) = Min(12; 11.4) = 11.4\ m$

therefor $d = 16\ m > e = 11.4\ m$

Table 3 The values of the surfaces of the wind zones of the walls in the direction V1

Zone	A		B		C	
Geometric dimension (m)	e/5	h	4e/5	h	d-e	h
	2.28	4.5	9.12	4.5	4.6	4.5
Area (m2)	10.26		41.04		20.7	

Table 4 The values of the surfaces of the wind zones of the roof in the direction V1

Zone	F		G		H		I	
Geometric dimension (m)	e/4	e/10	b'-e/4	e/10	b/2	4e/10	b/2	d-(e/2)
	2.85	1.14	3.15	1.14	6	4.56	6	10.3
Area (m^2)	3.249		3.59		27.36		61.8	

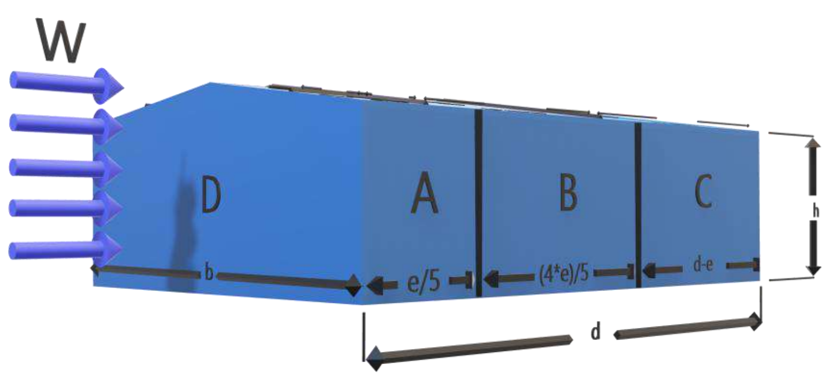

Figure II.4:I presentation of wind directions on the structure V1

II.4.1.7. Determines the permeability index

$$\mu_p = \frac{\sum opening\ surfaces\ \ or\ C_{pe} \leq 0}{\sum surfaces\ of\ all\ openings}$$

$$\mu_P = \frac{(3.2 \times 4.5) + (2 \times 0.8 \times 4 \times 2)}{(3.2 \times 4.5 \times 2) + (2 \times 0.8 \times 4 \times 2)} = \frac{27.2}{41.6} = 0.65$$

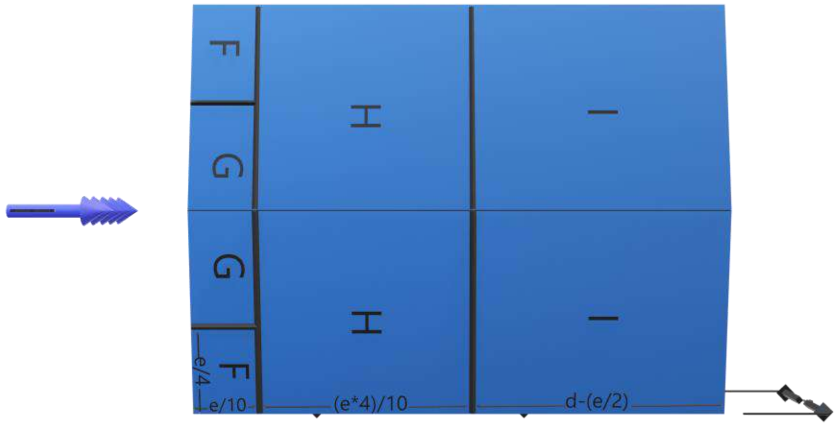

Figure II.4:II wind directions and legend for roofs V1.

II.4.1.8. Determination of internal pressure coefficient

The most unfavorable combination of external and internal pressure, considered to act simultaneously. Should be considered simultaneously for each potential combination of openings and other sources of air leakage.

Is the pressure coefficient C_{pi} a function of the permeability index and h/d ratio (RNVA2013)

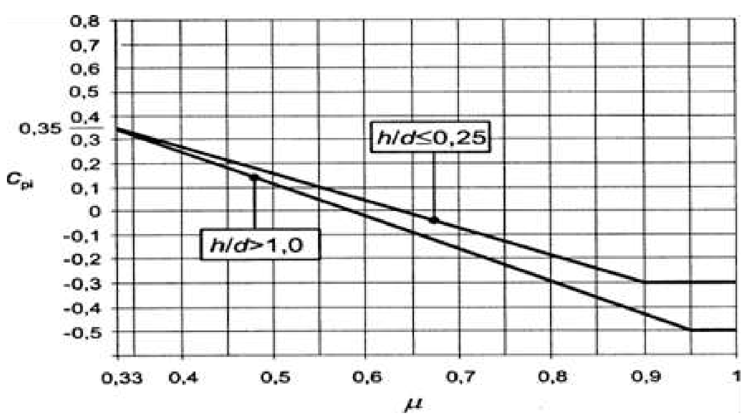

Figure II.4:III Internal pressure coefficients applicable for openings

$(\frac{h}{d} = \frac{5.7}{16} = 0.36)$ By interpolation

$$C_{pi}(\frac{h}{d}) = \left[\frac{C_{pi}(1)-C_{pi}(0.25)}{1-0.25} * \left(\frac{h}{d} - 0.25\right)\right] + C_{pi}(0.25)\ ;\ C_{Pi}(\frac{5.7}{16}) = -0.03$$

$$C_{pe\ moyenne} = C_{pe\ average} = \frac{\sum zone\ surface * C_{pe\ zone}}{\sum surfaces\ of\ zones}$$

$$W_j = C_d \times q_p \times \ (C_{Pe} - C_{Pi})$$

Table 5 The values of C_{pe} in the direction V1 and V3

Zone	End wall "Front"	End wall "Back"	Sidewall			Roof			
	D	E	A	B	C	F	G	H	I
C_d	1								
$Q_p\ kn/m_2$	0.783								
C_{pe}	+0.8	-0.3	-1	-0.8	-0.5	-1.23	-1.33	-0.67	-0.5
$C_{pe\ average}$			-0.74			-0.6			
C_{pi}	-0.03								
$C_{pe} - C_{pi}$	+0.83	-0.27	-0.71			-0.57			
$W_j\ kn/m^2$	+0.649	-0.211	-0.556			-0.446			

$$C_{Pe} = C_{Pe.10}\ (S \geq 10\ m^2)$$

$$e = Min(b; 2h) = Min(16; 2 \times 5.7) = Min(16; 11.4) = 11.4\ m$$

therefor $d = 12\ m > e = 11.4\ m$

$$\mu_P = \frac{(3.2\times4.5\times2)+(2\times0.8\times4)}{(3.2\times4.5\times2)+(2\times0.8\times4\times2)} = \frac{35.2}{41.6} = 0.85$$

By interpolation $C_{Pi}(\frac{5.7}{12}) = -0.28$

Table 6 The values of the surfaces of the wind zones of the walls and the roof in the direction V2

Zone	A	B	C	F	G	H	J	I
Area (m2)	10.78	47.09	2.74	6.50	11.74	77.76	18.24	77.76

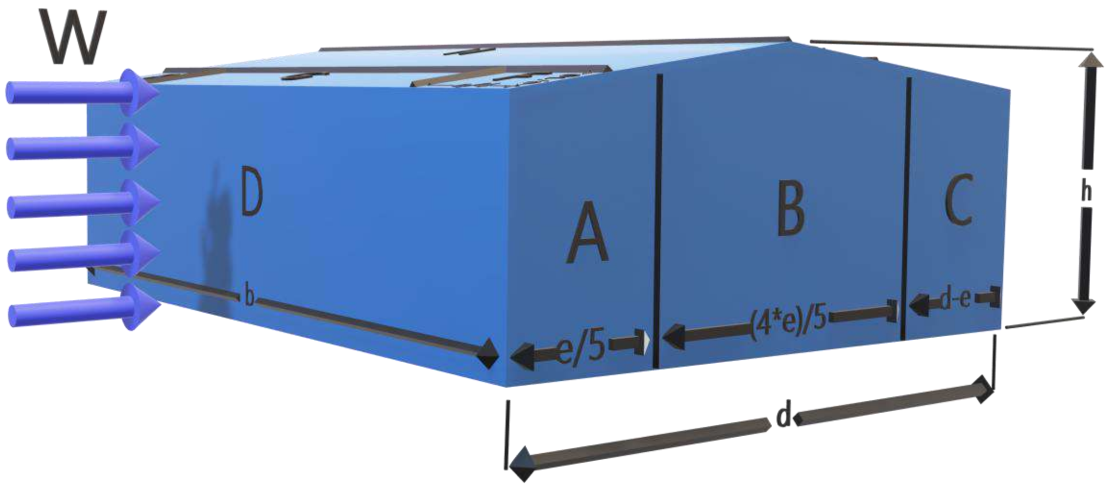

Figure II.4:IV presentation of wind directions on the structure V2

Table 7 The values of C_{pe} in the direction V2 (variant 1)

Zone	End wall "Front"	End wall "Back"	Sidewall			Roof (Slope in the wind)			Roof (Leeward slope)	
	D	E	A	B	C	F	G	H	J	I
C_d	1									
$Q_p\ kn/m_2$	0.783									
C_{pe}	+0.8	-0.3	-1	-0.8	-0.5	-0.76	-0.7	-0.26	-0.4	-0.83
$C_{pe\ average}$			-0.82			-0.35			-0.75	
C_{pi}	-0.26									
$C_{pe}-C_{pi}$	+1.06	-0.04	-0.56			-0.09			-0.49	
$W_j\ kn/m^2$	+0.830	-0.03	-0.438			-0.07			-0.384	

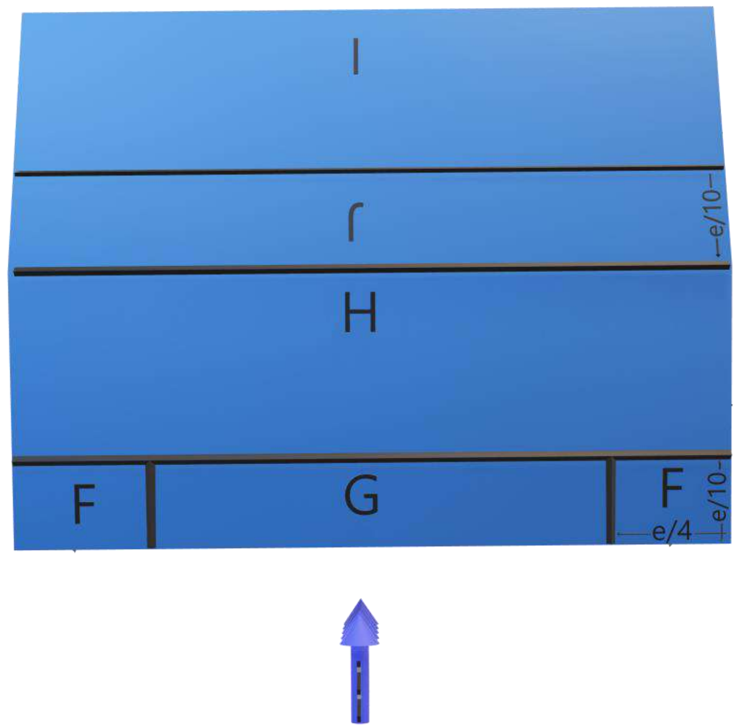

Figure II.4:V presentation of wind directions on the structure V2

Table 8 The values of C_{pe} in the direction V2 (variant 2)

Zone	End wall "Front"	End wall "Back"	Sidewall			Roof (Slope in the wind)			Roof (Leeward slope)	
	D	E	A	B	C	F	G	H	J	I
C_d	1									
$Q_p\ kn/m_2$	0.783									
C_{pe}	+0.8	-0.3	-1	-0.8	-0.5	+0.36	+0.36	+0.26	0	0
$C_{pe\ average}$			-0.82			+0.28			0	
C_{pi}	-0.26									
$C_{pe}-C_{pi}$	+1.06	-0.04	-0.56			+0.54			+0.26	
$W_j\ kn/m^2$	+0.830	-0.03	-0.438			+0.422			+0.204	

II.4.2. Friction force

Wind friction effects on the surface can be overlooked. When the total area of all the surfaces parallel to the wind (or weakly inclined in relation to the direction of the wind) is less than or equal to 4 times the total area of all the external surfaces perpendicular to the wind (in the sub -wind)

(According to RNVA 2013, chapter 2, article 3.6.3)

- In this case the direction of the wind it is perpendicular to the side wall (V2)
  - Calculating surfaces parallel to wind:

The end walls surfaces:

$$\big((12+4.5)+(6*1.2)\big)*2=122.4\ m^2$$

The roof which is weakly tilted in relation to the direction of the wind:

$$\left(\left(\frac{1.5}{\sin(11.31)}\right)*16*2\right)=197.91\ m^2$$

⇒ the total surfaces in parallel with the wind:

$$197.91+122.=320.31\ m^2$$

- Calculation of surfaces perpendicular to the wind (and downwind)

The surfaces of the two side walls:

$$(4.5*16)*2=108\ m^2$$

Verification of the condition (article 2.6.3 RNVA2013):

The total surface area parallel to the wind ≤ 4 × (the total area of surfaces perpendicular to the wind) $320.31\ \leq 4\times 108=432\ m^2$ ………………...Verified.

⇒ So, we must neglect the friction effect case of wind perpendicular to the side walls.

- In this case the direction of the wind it is perpendicular to the end wall (V1)
  - Calculating surfaces parallel to wind:

The side walls surfaces:

$$(4.5*16)*2=108\ \ m^2$$

The roof which is weakly tilted in relation to the direction of the wind:

$$\left(\left(\frac{1.5}{\sin(11.31)}\right)*16*2\right)=197.91\ m^2$$

⇒ the total surfaces in parallel with the wind:

$197.91 + 108 = 305.31\ m^2$

- Calculation of surfaces perpendicular to the wind (and downwind)

The surfaces of the two end walls:

$\big((12 + 4.5) + (6 * 1.2)\big) * 2 = 122.4\ m^2$

Verification of the condition (article 2.6.3 RNVA2013):

The total surface area parallel to the wind ≤ 4 × (the total area of surfaces perpendicular to the wind) $305.31\ \leq 4 \times 122 = 489.6\ m^2$ ………………...Verified.

⇒ So we must neglect the friction effect case of wind perpendicular to the end walls.

CHAPTER THREE

SECONDARY ELEMENTS

III.1. Introduction

The characteristics of our structure having been defined; we move on in this chapter to the pre-sizing of the load-bearing elements of the building.

III.2. Permanent load

It includes not only the own weight of the main and secondary structural elements, but also the weight of the elements incorporated into the load-bearing elements such as the roofing and the cladding.

III.3. Calculation of roof purlin

- ✓ Definition

The most stressed purlin is studied, which is the intermediate purlin with a span L=4m, inclined at an angle α = 11.31° and in the center distance "e" equal to 1.5 m.

Calculation of the spacing between purlins

$$\cos 11.31° = \frac{6}{x} \Rightarrow x = 6.12$$

We take a spacing between purlins of 1.5m (4 times) and at both ends 0.06m.

- the profile estimated as purlin is: IPE120

- there is at 5 purlins

III.3.1. Determination of solicitations

Evaluation of loads and overloads

a) Permanent loads (G)

- Own weight of the cladding with attachment accessory …………….20Kg/m^2
- Own weight of dropped celling……………………………………17 Kg/m^2
- Dead weight of the estimated purlin (IPE120) ………………………10.4 Kg/m

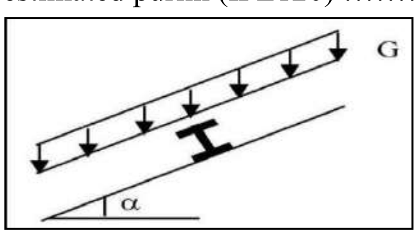

Figure III.3:I Static diagram of permanent loads G on purlins.

$G = \left(P_{cover} + P_{droped\ selling}\right) * e + \ P_{purline}\ ; e = 1.5$

$G = (0.2 + 0.17) * 1.5 + 0.104 = 0.659\ kn/m$

$G = 0.659\ kn/m$

b) Exploitation overload (Q)

a. <u>Maintenance agents</u>

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

The uniformly distributed load P due to the maintenance overloads is obtained by equalizing the two maximum moments due to P and to the point loads P'.

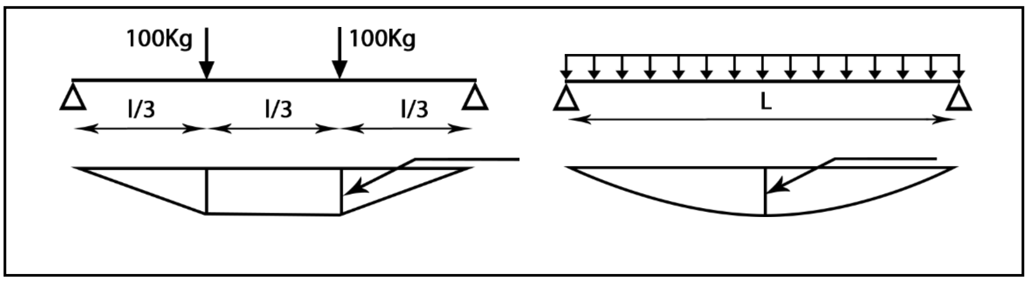

Figure III.3:II Static diagram of the equivalent beam.

$$M_{max} = \frac{PL}{3} = \frac{Q_{eq}L^2}{8} \Rightarrow Q_{eq} = \frac{8}{3} \times \frac{P}{L} = \frac{8}{3} \times \frac{1}{4}$$

$Q_{eq} = 0.667KN/m$

b. Sand load

$Q_1 = 0.4\ KN/m^2$

$Q_T = (Q_1 * e) + Q_{eq} = ((0.4) * 1.5) + 0.677 = 1.267\ KN/m$

c) Climatic load (Wind)

$w = -0.446\ KN/m$

d) Climatic Load (Sand)

$S = 0.2\ KN/m^2$

$S = 0.2 * 1.5 = 0.3\ KN/m$

➢ Loads and overloads applied

G = 0.659 KN/m

Q = 1.267 KN/m

W = -0.446 KN/m

- Breakdown of loads
  - Along the Z-Z axis:

    $G_{ZZ} = G \cos\alpha = 0.646\ KN/m$

    $Q_{ZZ} = Q \cos\alpha = 1.242\ KN/m$

    $W_{ZZ} = W = -0.446\ KN/m$

    $S_{ZZ} = S \cos\alpha = 0.294\ KN/m$
  - Along the Y-Y axis:

    $G_{YY} = G \sin\alpha = 0.137\ KN/m$

    $Q_{YY} = Q \sin\alpha = 0.264\ KN/m$

    $W_{YY} = 0 = 0\ KN/m$

    $S_{ZZ} = S \sin\alpha = 0.059\ KN/m$
- The combinations of actions

1. ULS (ELU)
   - Along the Z-Z axis:

     Comb 1 = $1.35G_z + 1.5Q_z = \mathbf{2.736\ KN/m}$

     Comb 2 = $G_z + 1.5W_z = -\ 0.357\ KN/m$

     Comb 3 = $1.35G_z + 1.5S_z = 1.314\ KN/m$
   - Along the Y-Y axis:

     Comb 1 = $1.35G_Y + 1.5Q_Y = \mathbf{0.547\ KN/m}$

     Comb 2 = $1.35G_Y = 0.174\ KN/m$

     Comb 3 = $1.35G_y + 1.5S_y = 0.263\ KN/m$

   According to the two axes y and z, the maximum loads at ULS (ELU) /ml returning to the most stressed purlin is:

$q_{uz} = 1.35G_z + 1.5Q_z = \mathbf{2.74\ KN/m}$ | $q_{uy} = 1.35G_Y + 1.5Q_Y = \mathbf{0.55\ KN/m}$

2. SLS (ELS)
   - Along the Z-Z axis:

     Comb 1 = $G_z + Q_z = \mathbf{1.889\ KN/m}$

     Comb 2 = $G_z + W_z = -0.025\ KN/m$

Comb 3 = $G_z + S_z = 0.94\ KN/m$

- Along the Y-Y axis:

Comb 1 = $G_Y + Q_Y = \mathbf{0.378\ KN/m}$

Comb 2 = $G_Y = 0.129\ KN/m$

Comb 3 = $G_y + S_y = 0.188\ KN/m$

According to the two axes y and z, the maximum loads at SLS (ELS) /ml returning to the most stressed purlin is:

$q_{uz} = G_z + Q_z = \mathbf{1.889\ KN/m}$ | $q_{uy} = G_Y + Q_Y = \mathbf{0.378\ KN/m}$

III.3.2. Principle of pre-sizing

The purlins are subjected to deviated bending (biaxial bending).

They must meet the following two conditions:

Deflection condition (at the SLS).

Resistance condition (the ULS).

Generally, the purlins are pre-dimensioned by using the deflection condition, then the resistance condition is checked.

III.3.6.1. Verification at SLS (Deflection)

The deflection at the limit state $F_z = \frac{5 \times q_z \times l^4}{384 \times E \times I_y} \leq \frac{l}{200} \Rightarrow I_y \geq \frac{5 \times q_Z \times 200 \times l^3}{384 \times E}$

For axe Z-Z

$$I_y \geq \frac{1000 \times 1.915 \times 10^{-2} \times 400^3}{384 \times 21000} = 152\ cm^4$$

What gives us $I_y \geq 112.61\ cm^4$

For axe Y-Y

$I_y \geq= 29.98\ cm^4$

What gives us $I_z \geq 29.98\ cm^4$ So we choose IPE120 with sag rods (bridges) is done with service loads and overloads (unweighted) $F \leq F_{allowable}$

For a beam on two supports uniformly loaded (axis Z-Z):

The unfavorable load combination

ULS

$q_{uz} = 1.35 G_z + 1.5 Q_z = \mathbf{2.74\ KN/m}$ | $q_{uy} = 1.35 G_Y + 1.5 Q_Y = \mathbf{0.55\ KN/m}$

SLS

$q_{uz} = G_z + Q_z = \mathbf{1.889\ KN/m}$ | $q_{uy} = G_Y + Q_Y = \mathbf{0.38\ KN/m}$

Table 9 Characteristics of the IPE 120

Profile	h (cm)	b (cm)	t_w (cm)	t_f (cm)	r (cm)	P (Kg/m)	d (cm)
IPE120	120	64	4.4	6.3	7	10.4	93.4
	A (cm2)	I_y (cm4)	i_y (cm)	W_{ply} cm3)	I_z (cm4)	W_{plz} (cm3)	i_z (cm)
	13.2	318	4.9	60.7	27.7	13.6	1.45

III.3.6.2. Verification of the Deflection

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

$$F_z = \frac{5 \times q_z \times l^4}{384 \times E \times I_y} \leq \frac{l}{200}\ beam\ on\ 02\ supports$$

$$F_z = \frac{5 \times 1.92 \times 10^{-2} \times 400^4}{384 \times 21000 \times 318} = 0.943 \leq F_{allow} \frac{400}{200} = 2$$

$F_z = 0.943 \leq F_{allow} = 2$ ……………………..Condition checked

b) Calculation of the deflection along the Y-Y axis:

$$F_y = \frac{5 \times q_y \times {}^{l}/_{2}{}^{4}}{384 \times E \times I_y} \leq \frac{{}^{l}/_{2}}{200}\ beam\ on\ 03\ supports$$

$$F_y = \frac{5 \times 0.38 \times 10^{-2} \times 200^4}{384 \times 21000 \times 318} = 0.135 \leq F_{allow} \frac{200}{200} = 1$$

$F_z = 0.135 \leq F_{allow} = 1$ ……………………..Condition checked

So the deflection condition is checked for both axes.

III.3.6.3. Resistance condition (ULS)

In the condition of resistance to ULS, the following checks must be made:

a) Deviated bending verification:

According to Euro Code 3, the biaxial bending resistance of the profile is verified if the following condition is met:

$$\left(\frac{M_{Y.sd}}{M_{PLY}}\right)^{\alpha} + \left(\frac{M_{Z.sd}}{M_{PLZ}}\right)^{\beta} \leq 1$$

With: $\alpha = 2 \ldots\ldots\ldots for\ profile\ I.;\ \beta = 5n \geq 1, n = \frac{N}{N_{pl}} = 0 \ \Rightarrow\ \beta = 1$

- Axe Z-Z

$$M_{ysd} = \frac{q_z \times l^2}{8} = \frac{2.74 \times 4^2}{8} = 5.472\ KN * m$$

- Axe Y-Y

$$M_{zsd} = \frac{q_y \times l/2^2}{8} = \frac{0.55 \times 2^2}{8} = 0.275\ KN * m$$

Determination of the profile class: $F_y = 235\ MPa$

Web: $\frac{d}{t_w} = \frac{93.4}{4.4} = 21.23 \ \leq 72\varepsilon$

in which $\varepsilon = \sqrt{235/235} = 1 \ \Rightarrow\ \varepsilon = 1$ therefor ⇒ web is class 1

Flange: $\frac{c}{t_f} = \frac{(64-4.4)/2}{6.3} = 4.73 \leq 9\varepsilon\ ;\ \varepsilon = 1$

therefor ⇒ flange is class 1

So, the section is class 1 ⇒ $\gamma_{m0} = 1.1$

$$M_{plyrd} = \frac{W_{ply} \times f_y}{\gamma_{m0}} = \frac{60.7 \times 10^{-6} \times 235 \times 10^3}{1.1} = 12.968 KN * m$$

$$M_{plzrd} = \frac{W_{plz} \times f_y}{\gamma_{m0}} = \frac{13.6 \times 10^{-6} \times 235 \times 10^3}{1.1} = 2.905\ KN * m$$

And the condition will be:

$$\left[\frac{5.472}{12.968}\right]^2 + \left[\frac{0.275}{2.905}\right]^1 = 0.273 \leq 1\ ;\ So\ the\ biaxial\ bending\ is\ verified$$

b) Shear verification:

For the shear check, the following condition is used:

$V_{Zsd} \leq V_{pl,Rd}$………………………….. [EC .3 p158]

$V_{ysd} \leq V_{pl,Rd}$………………………….. [EC .3 p158]

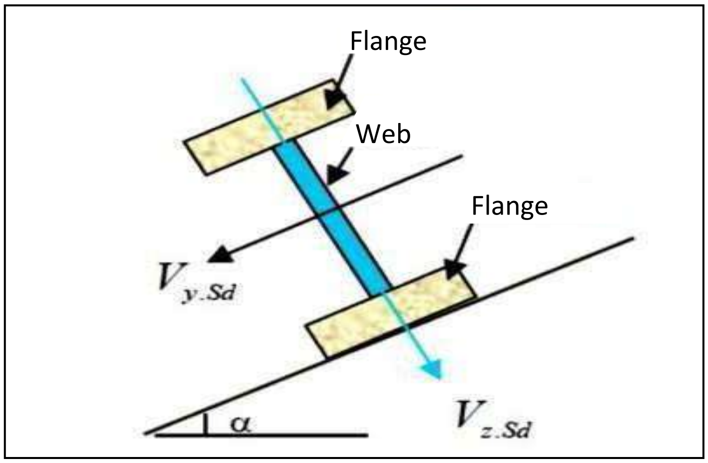

Figure III.3:III Shear representation of the purlin

$$V_{Zsd} = \frac{q_z \times l}{2} = \frac{2.74*4}{2} = 5.472\ KN$$

$$V_{pl,Rd} = \frac{A_{vy} \times f_y}{\sqrt{3} \times \gamma_{m0}} = \frac{8.6 \times 10^{-4} \times 235 \times 10^3}{\sqrt{3} \times 1.1} = 106.075\ KN$$

So

$V_{zsd} = 5.472 \leq V_{pl,Rd} = 106.075$

$V_{zsd} = 5.472 \leq 0.5V_{pl,Rd} = 53.037$ it's verified

$$V_{ysd} = \frac{q_y \times l}{2} = \frac{0.55 * 4}{2} = 1.094\ KN$$

So

$$V_{ysd} = 1.094 \leq V_{pl,Rd} = \frac{6.31 \times 10^{-4} \times 235 \times 10^3}{\sqrt{3} \times 1.1} = 77.83$$

$V_{ysd} = 1.094 \leq 0.5V_{pl,Rd} = 38.92$ it's verified

So, the resistance of the purlins to shearing is checked.

c) Lateral-Torsional buckling verification

Lateral-Torsional buckling which is lateral buckling + cross-section rotation

The top flange which is compressed under the action of the downward loads is susceptible to buckling. Since it is fixed to the roof, there is therefore no risk of Bend-twist coupling, unlike the lower flange which is compressed under the action of the uplift wind and which is itself likely to Lateral-Torsional buckling as long as it is free throughout its range.

We recall the unfavorable load combination for the risk of overturning. This is the first combination in which the wind acts alone and risks causing the purlin to Lateral-Torsional buckling (bending upwards) and compressing the bottom flange at mid-span (see figure)

According to Eurocode 3, the lateral torsional buckling resistance of the profile is verified if the following condition is Satisfied:

$$M_{ysd} = \frac{q_z \times l^2}{8} \leq M_{bRd} = \frac{X_{lt} \times \beta_W \times W_{ply} \times f_y}{\gamma_{m1}}$$

For the calculation of X_{lt} we have the formula: $X_{lt} = \frac{1}{\phi_{LT}+(\phi_{LT}^2-\bar{\lambda}_{LT}^2)^{0.5}}$

$\beta_w = 1$ for class sections (1) and (2)

M_{bRd}: Lateral-Torsional buckling resistant moment.

M_{ysd}: Soliciting moment

X_{lt}: Reduction coefficient for Bend-twist coupling.

$\gamma_{M1} = 1.1$

$\bar{\lambda}_{LT}$: It is the slenderness of variously $\bar{\lambda}_{LT} = \frac{\lambda_{LT}}{\lambda_1}\sqrt{\beta_W}$

$$\lambda_1 = \pi\sqrt{\frac{E}{F_y}} = 93.9\varepsilon \;\; ; \;\; and\ \varepsilon = \sqrt{\frac{235}{F_y}} = 1 \;\; therefor \;\Rightarrow\; \lambda_1 = 93.9$$

For beams with a constant and doubly symmetrical section, the approximate formula below can be used, which places it in safety.

And $\lambda_{LT} = \frac{l/i_z}{\sqrt{C_1}\left[1+\frac{1}{20}\left(\frac{l/i_z}{h/t_{fs}}\right)^2\right]^{0.25}}$ EC3 par 1-1

l: lateral support length (distance between the girder and the gantry: L=4m).

$C_1 = 1.132$

$$\lambda_{LT} = \frac{400/1.24}{\sqrt{1.132}\left[1+\frac{1}{20}\left(\frac{400/1.24}{12/0.63}\right)^2\right]^{0.25}} = 140.836$$

$$\bar{\lambda}_{LT} = \frac{140.836}{93.9}\sqrt{1} = 1.5 >> 0.4$$

There is a risk of Bend-twist coupling

$$X_{lt} = \frac{1}{\phi_{LT} + \left(\phi_{LT}^2 - \bar{\lambda}_{LT}^2\right)^{0.5}}, ; and \quad \phi_{LT} = 0.5[1 + \alpha_{LT}(\bar{\lambda}_{LT} - 0.2) + \bar{\lambda}_{LT}{}^2]$$

With $\alpha_{LT} = 0.21$ for rolled profiles

$\phi_{LT} = 0.5[1 + 0.21(1.5 - 0.2) + 1.5^2] = 1.761$

The reduction coefficient for Bend-twist coupling

$$X_{lt} = \frac{1}{1.761 + (1.761^2 - 1.5^2)^{0.5}} = 0.373$$

$$M_{bRd} = \frac{X_{lt} \cdot \beta_u \cdot W_{ply} \cdot f_y}{\gamma_{m1}} = \frac{0.373 \times 1 \times 60.7 \times 10^{-3} \times 235}{1.1} = 4.831\ KN * m$$

$M_{ysd} = 5.472\ KN * m > M_{bRd} = 4.831\ KN * m$

Therefore: The Bend-twist coupling stability of the purlin is not checked, so we upscale to IPE140 with sag rods.

Table 10 Characteristics of the IPE 140

Profile	**h (cm)**	**b (cm)**	**t_w (cm)**	**t_f (cm)**	**r (cm)**	**P (Kg/m)**	**d (cm)**
IPE140	140	73	4.7	6.9	7	12.9	112.2
	A (cm2)	**I_y (cm4)**	**i_y (cm)**	**W_{ply} cm3)**	**I_z (cm4)**	**W_{plz} (cm3)**	**i_z (cm)**
	16.4	541	5.74	88.3	44.9	19.3	1.65

$$\lambda_{LT} = \frac{400/1.65}{\sqrt{1.132}\left[1 + \frac{1}{20}\left(\frac{400/1.65}{14/0.69}\right)^2\right]^{0.25}} = 140.33$$

$$\bar{\lambda}_{LT} = \frac{140.33}{93.9}\sqrt{1} = 1.494 >> 0.4$$

$\phi_{LT} = 0.5[1 + 0.21(1.5 - 0.2) + 1.5^2] = 1.753$

The reduction coefficient for Bend-twist coupling

$$X_{lt} = \frac{1}{1.753 + (1.753^2 - 1.494^2)^{0.5}} = 0.375$$

$$M_{bRd} = \frac{X_{lt} \cdot \beta_u \cdot W_{ply} \cdot f_y}{\gamma_{m1}} = \frac{0.375 \times 1 \times 88.3 \times 10^{-3} \times 235}{1.1} = 7.07\ KN * m$$

$M_{ysd} = 5.801\ KN * m < M_{bRd} = 7.07\ KN * m$ Verified

The purlin stability to overturning is checked and we stick with IPE 140 section

III.3.6.4. *The Autodesk ROBOT analysis results*

STEEL DESIGN

CODE: *NF EN 1993-1:2005/NA:2007/AC:2009, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 268 Beamroof_268 **POINT:** 4 **COORDINATE:** x = 0.50 L = 2.00 m

---

SECTION PARAMETERS: IPE 140

h=14.0 cm	gM0=1.00	gM1=1.00	
b=7.3 cm	Ay=11.16 cm2	Az=7.65 cm2	Ax=16.43 cm2
tw=0.5 cm	Iy=541.22 cm4	Iz=44.92 cm4	Ix=2.46 cm4
tf=0.7 cm	Wply=88.34 cm3	Wplz=19.25 cm3	

INTERNAL FORCES AND CAPACITIES:

N,Ed = 0.24 kN	My,Ed = 4.99 kN*m	Mz,Ed = -0.76 kN*m	
Nc,Rd = 386.11 kN	My,Ed,max = 4.99 kN*m	Mz,Ed,max = -0.76 kN*m	
Nb,Rd = 386.11 kN	My,c,Rd = 20.76 kN*m	Mz,c,Rd = 4.52 kN*m	
	MN,y,Rd = 20.76 kN*m	MN,z,Rd = 4.52 kN*m	
	Mb,Rd = 9.13 kN*m		
			Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 10.96 kN*m	Curve,LT -	XLT = 0.44
Lcr,upp=4.00 m	Lam_LT = 1.38	fi,LT = 1.56	XLT,mod = 0.44

VERIFICATION FORMULAS:
Section strength check:
N,Ed/Nc,Rd = 0.00 < 1.00 (6.2.4.(1))
My,Ed/MN,y,Rd = 0.24 < 1.00 (6.2.9.1.(2))
Mz,Ed/MN,z,Rd = 0.17 < 1.00 (6.2.9.1.(2))
(My,Ed/MN,y,Rd)^ 2.00 + (Mz,Ed/MN,z,Rd)^1.00 = 0.23 < 1.00 (6.2.9.1.(6))
Global stability check of member:
My,Ed,max/Mb,Rd = 0.55 < 1.00 (6.3.2.1.(1))
N,Ed/(Xy*N,Rk/gM1) + kyy*My,Ed,max/(XLT*My,Rk/gM1) + kyz*Mz,Ed,max/(Mz,Rk/gM1) = 0.72 < 1.00 (6.3.3.(4))
N,Ed/(Xz*N,Rk/gM1) + kzy*My,Ed,max/(XLT*My,Rk/gM1) + kzz*Mz,Ed,max/(Mz,Rk/gM1) = 0.72 < 1.00 (6.3.3.(4))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):
uy = 0.9 cm < uy max = L/200.00 = 2.0 cm Verified
uz = 0.5 cm < uz max = L/200.00 = 2.0 cm Verified

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

III.4. The sag rods (bridges)

$q_{ue} = 0.56\ KN/m$

$l = \frac{L}{2} = 2\ m$

$R = 0.554 * 2 =$1.108 KN

III.4.1. Tensile forces in the section of the L1 line coming from the eave strut:

$T_1 = \frac{R}{2} = 0.55\ KN$

$L_2 : T_2 = T_1 + R = 1.66\ KN$

$L_3 : T_3 = T_2 + R = 2.77\ KN$

$l_4 : T_4 = \frac{T_4}{2 * \sin(\alpha)}\ \ with\ \ \alpha = \mathrm{atan}\left(\frac{1.5}{2}\right) = 36.9\,°$

$T_4 = 2.308\ KN$

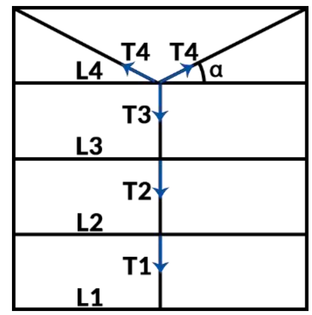

Figure III.4:I Sag Rods

III.4.2. The sg rods sizing

The most stressed section if the one with the maximum effort $N_{t,sd} = 2.286$ The condition of plastic resistance of the raw section imposes that:

$$N_{t,sd} \leq N_{pl} = A \times \frac{f_y}{\gamma_{m0}} \Longrightarrow A \geq \frac{N_{t,sd} * \gamma_{m0}}{f_y} = \frac{2.308 * 1.1}{23.5} = 0.108\ cm^2$$

$$A \geq \frac{\pi * \emptyset^2}{4} \Longrightarrow \emptyset \geq \sqrt{\frac{4 * 0.108}{3.14}} \Longrightarrow \emptyset \geq 0.371\ cm$$

Therefore, we choose a rod with circular cross section of $\emptyset_{10}$

III.5. Calculation of purlin shoe

The purlin shoe is a fixing device for attaching purlins to crossbeams.

The main resistance effort of the purlin shoe is the overturning moment due to the loading (especially under the action of wind uplift), The eccentricity "t" is limited by the following condition:

$$2 * \left(\frac{b}{2}\right) \leq t \leq 3 * \left(\frac{b}{2}\right) \quad ; \quad b = 7.3\ cm \quad ; \quad h = 140$$

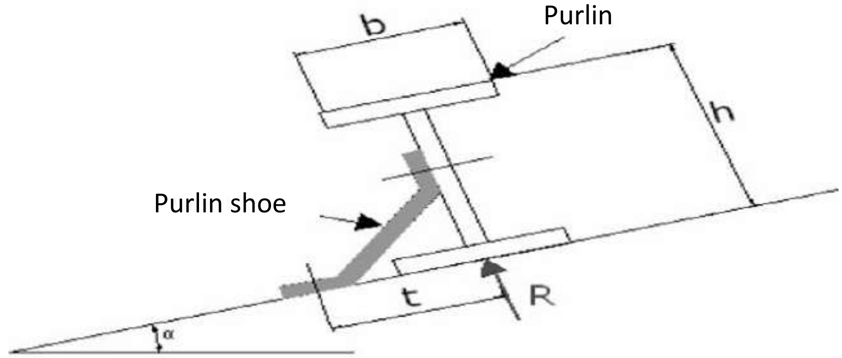

Figure III.5:I Provision of the purlin shoe on the purlin.

$7.3cm \ \leq t \leq 10.95\ cm$, we adopt t = 9

The dimensioning principle is to determine the overturning moment due to the loading especially with the uplift wind force.

The most unfavorable combination:

a) Lifting force:

➢ $G_Z - 1.5W_Z$

$q_{uz} = 1.65$

b) Crawling following force:

➢ $1.35G_y + 1.5Q_y$

$q_{uy} = 0.56$

Purlin shoe forces

$$R_Z = e * q_{uz} \times \frac{l}{2} = 1.5 * 1.67 \times \frac{4}{2} = 5.01KN$$

$$R_Y = e * q_{uz} \times \frac{l}{2} = 1.5 * 0.56 \times \frac{4}{2} = 1.661\ KN$$

- **Calculation of the overturning moment**

The force R risks causing the purlin shoe to bend. To prevent this risk, it is necessary to check that the overturning moment M_r does not exceed the bending moment.

$$\Rightarrow M_r = R_z \times t + R_y \times \frac{h}{2} \leq \frac{W_{ply} \times f_y}{\gamma_{M0}} \Rightarrow W_{ply} \geq \frac{\gamma_{M0} \times M_r}{f_y} \quad ; \quad W_{ply} = \frac{I_{y..purlin\ shoe}}{\frac{e}{2}}$$

$$M_r = 5.01 \times 9 + 1.661 \times \frac{14}{2} = 56.72\ KN * cm$$

- **Sizing the purlin shoe**

We take b=12cm

$$\Rightarrow e \geq \sqrt{\frac{6 \times M_r \times \gamma_{M0}}{f_y \times b}} = \sqrt{\frac{6 \times 56.72 \times 1.1}{23.5 \times 12}} = 13.82\ mm\ \cong 20\ mm$$

Note: The purlin shoe width (b) is taken after sizing the rafters (crosshead).

So: we adopt a 25 mm thick purlin shoe

III.6. Calculation of the girt

III.6.1. Definition

Cladding stringers are made up of beams (IPE, UAP, UPE) or thin folded sections. Being arranged horizontally, they are carried either by the portal posts or by the intermediate posts. The between axes of the rails is determined by the allowable span of the cladding trays.

III.6.2. Determination of loads and overloads

a) permanent loads

- Own weight of the cover attachment accessory … …………..…20.0 kg/m^2
- Dead weight of the estimated girt (UPN140) ……………………16 Kg/m
- Dead weight of the window and window jamb and headers……..25.0 kg/m

  Window jamb is an UPN80 with weight of….……………………..8.64 kg/m

$G = [(P_{cover}) \times e] + P_{girt} + P_{window}$

e: spacing between the girts e= 1.125 m

$G = (0.2) * 1.125 + 0.16 + 0.26 = 0.635\ K/m$

b) Climatic wind loads

The cladding rails are calculated with the value obtained: W=0.827KN/m² (chapter II)

From where: $W = 0.827 * e = 0.846 * 1.125 = 0.952\ KN/m$

$W = 0.952\ KN/m$

1) Loads applied in the ULS (ELU):

$q_{uy} = 1.35G = 0.857\ KN/m$

$q_{uz} = 1.5W = 1.428\ KN/m$

2) Loads applied in the SLS (ELS):

$q_{uy} = G = 0.635\ KN/m$

$q_{uz} = W = 0.952\ KN/m$

III.6.3. Pre dimensioning

Verification at the serviceability limit state is carried out with the service loads and overloads (unweighted): $f \leq f_{allowable}$

$$f_Z = \frac{5 \times q_Z \times l^4}{384 \times E \times I_y} \leq \frac{l}{200} \Rightarrow I_y \geq \frac{5 \times 200 \times q_Z \times l^3}{384 \times E}$$

$$I_y \geq \frac{5 \times 200 \times 0.952 \times 10^{-2} \times 400^3}{384 \times 2.1 \times 10^5 \times 10^{-1}} = 75.54\ cm^4$$

$I_Z \geq 50.39\ cm^4$

Which gives us $I_y \geq 75.54\ cm^4$ So we adopt a UPN 140

Table 11 Characteristics of the UPN 140

Profile	h (cm)	b (cm)	t_w (cm)	t_f (cm)	r (cm)	P (Kg/m)	d (cm)
UPN140	14	6	0.7	1	1	16	8.2
	A (cm2)	**I_y (cm4)**	**i_y (cm)**	**W_{ply} cm3)**	**I_z (cm4)**	**W_{plz} (cm3)**	**i_z (cm)**
	20	605	5.45	103	62.7	28.3	1.75

Verification of the Deflection

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

$$F_z = \frac{5 \times q_z \times l^4}{384 \times E \times I_y} \leq \frac{l}{200}\ beam\ on\ 02\ supports$$

$$F_z = \frac{5 \times 0.952 \times 10^{-2} \times 400^4}{384 \times 21000 \times 318} = 0.25 \leq F_{allow} \frac{400}{200} = 2$$

$F_z = 0.25 \leq F_{allow} = 2$ ……………………….Condition checked

b) Calculation of the deflection along the Y-Y axis:

$$F_y = \frac{5 \times q_y \times l^4}{384 \times E \times I_y} \leq \frac{l}{200}\ beam\ on\ 02\ supports$$

$$F_y = \frac{5 \times 0.635 \times 10^{-2} \times 400^4}{384 \times 21000 \times 318} = 1.608 \ \leq F_{allow}\frac{400}{200} = 2$$

$F_z = 1.608 \ \leq F_{allow} = 1$ ……………………….Condition checked

So, the deflection condition is checked for both axes.

III.6.4. Resistance condition ULS (ELU):

In the condition of resistance to ULS, the following checks must be made

For this verification, the following condition is used: ………… [EC.3 p163]

a) Bi-axial bending verification:

$$\left(\frac{M_{Y.sd}}{M_{PLY}}\right)^{\alpha} + \left(\frac{M_{Z.sd}}{M_{PLZ}}\right)^{\beta} \leq 1$$

With $\alpha = 2\,;\ \beta = 1\ for\ class\ 1\ sections$

$$M_{ysd} = \frac{q_z \cdot l^2}{8} = \frac{1.428 * 4^2}{8} = 2.855\ KN/m$$

$$M_{zsd} = \frac{q_y \cdot l^2}{8} = \frac{0.857 * 4^2}{8} = 1.72\ KN/m$$

Determination of the profile class: $F_y = 235\ MPa$

$$\varepsilon = \sqrt{235/235} = 1$$

- Web

  $\frac{d}{t_w} = \frac{8.2}{0.7} = 11.71 < 72\varepsilon$ therefor the web is class 1

- Flange

  $\frac{c}{t_f} = \frac{5.5-0.8-0.9}{0.9} = 4.33 < 9\varepsilon$ therefor the flange class is 1

  So, the section class is 1 and $\gamma_{m0} = 1.1$

$$M_{plyrd} = \frac{w_{ply} \cdot f_y}{\gamma_{m0}} = \frac{103 * 23.5 \cdot 10^{-2}}{1.1} = 22.01\ KN * m$$

$$M_{plzrd} = \frac{w_{plz} \cdot f_y}{\gamma_{m0}} = \frac{28.3 \cdot 23.5 \cdot 10^{-2}}{1.1} = 6.05\ KN * m$$

$$\left[\frac{2.855}{22.01}\right]^2 + \left[\frac{1.72}{6.05}\right]^1 = 0.3\ < 1 \text{ So the bi} - \text{axial bending is verified}$$

b) Shear verification:

For the shear check, the following condition is used:

$V_{zsd} \leq V_{plzrd}$………………………..[EC .3 p158]

$V_{ysd} \leq V_{plyrd}$………………………..[EC .3 p158]

$$V_{ysd} = \frac{q_y \cdot l}{2} = \frac{1.428 * 4}{2} = 2.855\ KN \qquad V_{zsd} = \frac{q_z \cdot l}{2} = \frac{0.857 * 4}{2} = 1.715\ KN$$

$$V_{plyrd} = \frac{A_{vy} \cdot F_y}{\sqrt{3} \cdot \gamma_{m0}} = \frac{13.353 * 23.5}{\sqrt{3} * 1.1} = 164.694 \text{ KN}$$

$$V_{plzrd} = \frac{A_{vz} \cdot F_y}{\sqrt{3} \cdot \gamma_{m0}} = \frac{10.4 * 23.5}{\sqrt{3} * 1.1} = 128.277\ KN$$

$$V_{zsd} = 1.715\ KN < V_{plzrd} = 128.277\ KN$$

$$V_{yd} = 2.855 < V_{plyrd} = 164.694\ KN$$

And

$$V_{zsd} = 1.385\ KN < 0.5V_{plzrd} = 64.14\ KN$$

$$V_{ysd} = 2.855 < 0.5V_{plyrd} = 82.35\ KN$$

So, the shear resistance of the Girts is checked.

c) Lateral-Torsional buckling verification:

- Calculation of reduced slenderness:

$$\lambda_{LT} = \frac{l/i_z}{\sqrt{C_1}\left[1+\frac{1}{20}\left(\frac{l/i_z}{h/t_{fs}}\right)^2\right]^{0.25}}$$

In the case of beams subjected to the end moment

$$C_1 = 1.88 - 1.40\psi + 0.52\psi^2 \leq 2.7 \quad ; \quad \psi = \frac{M_a}{M_b} = 0$$

So $C_1 = 1.88$

$$\lambda_{LT} = \frac{400/1.75}{\sqrt{1.88}\left[1 + \frac{1}{20}\left(\frac{400/1.75}{14/1}\right)^2\right]^{0.25}} = 85.68$$

$$\varepsilon = 1 \ ; \ \lambda_1 = \pi\sqrt{\frac{E}{F_y}} = 93.9\varepsilon = 93.9$$

$$\bar{\lambda}_{LT} = \frac{85.68}{93.9}\sqrt{1} = 0.912$$

$$\bar{\lambda}_{LT} = 0.912 > 0.4\ \textit{there is a risk of spillage}$$

$$X_{lt}=\frac{1}{\phi_{LT}+\left(\phi_{LT}^{2}-\bar{\lambda}_{LT}^{2}\right)^{0.5}}, \qquad and \quad \phi_{LT}=0.5[1+\alpha_{LT}\left(\bar{\lambda}_{LT}-0.2\right)+\bar{\lambda}_{LT}{}^{2}]$$

$\alpha_{LT}=0.21\, for\ rolled\ profiles$

$\phi_{LT}=0.5[1+0.21(0.912-0.2)+0.912^{2}]=0.991$

$$X_{lt}=\frac{1}{0.991+(0.991^{2}-0.912^{2})^{0.5}}=0.726$$

$$M_{ysd}=\frac{q_z*l^2}{8}=\frac{1.428*4^2}{8}=4.064\ KN/m$$

$$M_{brd}=\frac{X_{lt}*\beta_w*W_{ply}*F_y}{\gamma_{m1}}=\frac{0.726*1*103*235*10^{-3}}{1.1}=15.67\frac{KN}{m}$$

$M_{ysd}=4.064\ KN/m\ <M_{brd}=15.67\ KN/m$

Therefore: the UPN 140 profile torsional lateral torsional stability is verified,

III.7. Calculation of Columns (Potelets)

III.7.1. Introduction:

The columns are most often I or H sections intended to stiffen the fence (Cover) and resist the horizontal forces of the wind, their characteristics vary according to the nature of the cover (in masonry or corrugated iron) and the height. construction.

They are considered articulated in both ends.

We have

Every end wall has two posts

The columns are made of S235 steel

III.7.2. Calculation of loads and overloads:

a) Permanent loads G:

Dead weight of a girt (UPN140): 0.16 KN/m.

Dead weight of the cover: 0.2 KN/m^2

Dead weight of the door: 0.26 KN/m^2

The distance e = $\frac{l_1}{2}+\frac{l_2}{2}=\frac{4}{2}+\frac{4}{2}=4$

e: The actual distance carried by the post

G= dead weight of the post + dead weight of the girts + dead weight of the cover + dead weight of the door + dropped ceiling

b) Climatic overload W

The most unfavorable wind load on the gable is:

W=0.827 KN/m^2

V=W*e=3.308 KN/m

III.7.3. The pre-sizing of the column:

load along the Z-Z axis

$$f_Z = \frac{5 \times q_Z \times l^4}{384 \times E \times I_y} \leq \frac{l}{200} \Rightarrow I_y \geq \frac{5 \times 200 \times q_Z \times l^3}{384 \times E}$$

$$I_y \geq \frac{5 \times 200 \times 3.308 \times 10^{-2} \times 530^3}{384 \times 2.1 \times 10^5 \times 10^{-1}} = 610.72\ cm^4$$

What gives us $I_y \geq 620.798$ so we take for an IPE220

Table 12 Characteristics of the IPE 220

Profile	h (cm)	b (cm)	t_w (cm)	t_f (cm)	r (cm)	P (Kg/m)	d (cm)
IPE220	22	11	0.59	0.92	1.2	26.2	17.76
	A (cm2)	**I_y (cm4)**	**i_y (cm)**	**W_{ply} cm3)**	**I_z (cm4)**	**W_{plz} (cm3)**	**i_z (cm)**
	33.4	2772	9.11	285	205	58.1	2.48

III.7.4. Verification of section resistance:

If: $V_{sd} \leq 0.5V_{plRd}$. → There is no interaction between the bending moment and the shear force.

$$Q_{zsd} = 1.5V = 1.5 * 3.308 = 4.962\ KN/m$$

$$V_{zsd} = \frac{Q_{zsd} * l}{2} = \frac{4.962 * 5.3}{2} = 13.15\ KN$$

$$V_{plzRd} = \frac{A_{vz}\frac{F_y}{\sqrt{3}}}{\gamma_{m0}} = \frac{15.9 * \frac{23.5}{\sqrt{3}}}{1.1} = 196.115\ KN$$

$$\frac{V_{zsd}}{V_{plzRd}} = \frac{13.15}{196.115} = 0.067 < 0.5$$

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

III.7.5. Incidence of normal effort:

If $N_{sd} \leq$ min (0.25NplRd; $0.5A_wF_y/\gamma_{m0}$)): There is no interaction between the resistant moment and the normal force.

Girt weight: (0.16*4*5) = 3.2 KN

Cover weight and accessories: (0.2) *$\left((6*4.5*0.2)+(\frac{6*1.2}{2}*0.2)\right) = 6.12\ KN$

Door weight: $0.26*(3.2*4) = 1.66\ KN$

Post weight: $0.262*5.38 = 1.41\ KN$

Dropped ceiling 0.17*2*3.8 = 1.292 KN

$G = 1.66+6.12+1.29+3.2+1.41 = 13.68\ KN$

$N_{sd} = 1.35G = 1.35*13.68 = 18.468\ KN$

$$N_{plRd} = \frac{A*F_y}{\gamma_{m1}} = \frac{33.4*23.5}{1.1} = 713.55\ KN$$

$$0.25N_{plRd} = 178.39\ KN$$

$A_w = A - 2b*t_f = 33.4 - 2*11*0.92 = 20.42\ cm^2$

$$\frac{0.5A_wF_y}{\gamma_{m0}} = 0.5*20.42*\frac{23.5}{1.1} = 218.123\ KN$$

$N_{sd} = 18.468\ KN < \min(178.39\ ; 218.123) = 178.39\ KN$

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

Section Classification

$\varepsilon = \sqrt{235/235} = 1$

- Web

  $\frac{d}{t_w} = \frac{17.76}{0.59} = 30.1 < 72\varepsilon\ and\ 33\varepsilon$; therefor the web is class 1

- Flange

  $\frac{c}{t_f} = \frac{(11/2)}{0.92} = 5.98 < 10\varepsilon$ therefor the flange class is 1

  So, the section class is 1 and $\gamma_{m0} = 1.1$

$$M_{crd} = M_{plyrd} = W_{ply}*\frac{F_y}{\gamma_{m0}} = 285*10^{-2}*\frac{23.5}{1.1} = 60.886\ KN*m$$

$$M_{ysd} = \frac{Q_{zsd}l^2}{8} = \frac{4.962*5.3^2}{8} = 16.932\ KN*m$$

$M_{ysd} = 16.932\ KN*m < M_{crd} = 60.886\ KN*m$

So, the bending condition is checked.

III.7.6. Verification of the element for instabilities:

Compound bending with risk of buckling:

$$\frac{N_{sd}}{X_{min}N_{plRd}}+\frac{K_yM_{ysd}}{M_{plRd}}\leq 1$$

Compound bending with risk of Bend-twist coupling:

$$\frac{N_{sd}}{X_zN_{plRd}}+\frac{K_yM_{ysd}}{M_{plRd}}\leq 1$$

III.7.6.1. Buckling in relation to the axis Y-Y

$$\lambda_1=\pi\left(\frac{E}{F_y}\right)^{0.5}=\pi\left(\frac{21000}{23.5}\right)^{0.5}=93.9\ ;\ \beta_A=1\quad For\ sections\ of\ classes\ 1,2,3.$$

$$\lambda_y=\frac{l_y}{i_y}=\frac{530}{9.11}=58.178\ ;\ \bar{\lambda}_y=\frac{\lambda_y}{\lambda_1}=\frac{58.178}{93.9}=0.62$$

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

Buckling curve $\frac{h}{b}=\frac{220}{110}=2\ >1.2:\ t_f=9.2\ mm<40\ mm$

Buckling axis Y-Y → buckling curve $\alpha_y=0.21$

$$\phi_y=0.5\left[1+\alpha_y\left(\bar{\lambda}_y-0.2\right)+\bar{\lambda}_y{}^2\right]=0.5(1+0.21(0.62-0.2)+0.62^2)=0.736$$

$$X_y=\frac{1}{\phi_y+(\phi_y{}^2-\bar{\lambda}_y{}^2)^{0.5}}=\frac{1}{0.736+(0.736^2-0.62^2)^{0.5}}=0.882$$

III.7.6.2. Buckling in relation to the axis Z-Z

$$\lambda_z=\frac{l_z}{i_z}=\frac{112.5}{2.48}=45.363\ ;\ \bar{\lambda}_z=\frac{\lambda_z}{\lambda_1}=\frac{45.363}{93.9}=0.483$$

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

Buckling axis Y-Y → buckling curve $\alpha_y=0.34$

$$\phi_z=0.5\left[1+\alpha_z\left(\bar{\lambda}_z-0.2\right)+\bar{\lambda}_z{}^2\right]=0.5(1+0.34(0.48-0.2)+0.48^2)=0.665$$

$$X_z=\frac{1}{\phi_z+(\phi_z{}^2-\bar{\lambda}_z{}^2)^{0.5}}=\frac{1}{0.665+(0.665^2-0.483^2)^{0.5}}=0.892$$

$$X_{min} = \min(X_y; X_z) = 0.882$$

Calculation of reduced slenderness with respect to lateral buckling λ_{LT}

$$\lambda_{LT} = \frac{l/i_z}{\sqrt{C_1}\left[1+\frac{1}{20}\left(\frac{l/i_z}{h/t_{fs}}\right)^2\right]^{0.25}} = \frac{530/9.11}{\sqrt{1.132}\left[1+\frac{1}{20}\left(\frac{530/9.11}{22/0.92}\right)^2\right]^{0.25}} = 134.37$$

$$\bar{\lambda}_{LT} = \frac{\lambda_{LT}}{\lambda_1}\beta_a{}^{0.5} = \frac{134.37}{93.9}1^{0.5} = 1.431 > 0.4$$

$\bar{\lambda}_{LT} = 1.444 > 0.4$ there is a risk of Bend-twist coupling

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

$\alpha_{LT} = 0.21$ for rolled sections.

$$\phi_{LT} = 0.5\left[1+\alpha_{LT}(\bar{\lambda}_{LT}-0.2)+\bar{\lambda}_{LT}{}^2\right] = 0.5(1+0.21(1.431-0.2)+1.431^2)$$

$$= 1.653$$

$$X_{LT} = \frac{1}{\phi_{LT}+(\phi_{LT}{}^2-\bar{\lambda}_{LT}{}^2)^{0.5}} = \frac{1}{1.653+(1.653^2-1.431^2)^{0.5}} = 0.403$$

Calculation of the K coefficients:

$$\mu_y = \bar{\lambda}_y(2\beta_{My}-4)+\frac{W_{ply}-W_{ely}}{W_{ely}} = 0.62*(2*1.3-4)+\frac{285-252}{252} = -0.736$$

$$K_y = 1-\left(\frac{\mu_y}{x_y}*\frac{N_{sd}}{A*F_y}\right) = 1-\left(\frac{-0.736}{0.882}*\frac{18.468}{33.4*23.5}\right) = 1.017$$

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

Simply supported beam with uniformly distributed load: $\beta_{My} = 1.3$ Table 4

$$\mu_{LT} = 0.15\bar{\lambda}_Z\beta_{MLT}-0.15 = 0.15*0.483*1.3-0.15 = -0.056$$

$$K_{LT} = 1-\left(\frac{\mu_{LT}}{x_z}*\frac{N_{sd}}{A*F_y}\right) = 1-\left(\frac{-0.056}{0.892}*\frac{14.43}{33.4*23.5}\right) = 1.001$$

$N_{plRd} = 713.545\ KN$	$M_{plyrd} = 60.89\ KN\cdot m$	$M_{ysd} = 16.93\ KN\cdot m$	$N_{sd} = 18.47\ KN$

a) Buckling verification

$$\frac{N_{sd}}{X_{min}N_{plRd}}+\frac{K_yM_{ysd}}{M_{plRd}} \le 1$$

$$\frac{18.468}{0.879 * 713.545} + \frac{1.017 * 17.447}{60.886} = 0.309 \leq 1 \ldots\ldots\ldots\ldots\ldots\ldots\ldots \; verified$$

b) Lateral-Torsional buckling verification

$$\frac{N_{sd}}{X_z N_{plRd}} + \frac{K_{LT} M_{ysd}}{X_{LT} M_{plRd}} \leq 1$$

$$\frac{18.468}{0.892 * 713.545} + \frac{1.001 * 16.932}{0.397 * 60.886} = 0.717 \leq 1 \ldots\ldots\ldots\ldots\ldots\ldots\ldots \; verified$$

So, the section IPE220 is suitable as a post.

III.7.7. Autodesk ROBOT analysis result

STEEL DESIGN

CODE: *NF EN 1993-1:2005/NA:2007/AC:2009, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 91 Posts_91 **POINT:** 1 **COORDINATE:** x = 0.00 L = 0.00 m

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SECTION PARAMETERS: IPE 220

h=22.0 cm	gM0=1.00	gM1=1.00	
b=11.0 cm	Ay=22.89 cm2	Az=15.88 cm2	Ax=33.37 cm2
tw=0.6 cm	Iy=2771.84 cm4	Iz=204.89 cm4	Ix=8.86 cm4
tf=0.9 cm	Wply=285.43 cm3	Wplz=58.11 cm3	

INTERNAL FORCES AND CAPACITIES:

N,Ed = 22.74 kN	My,Ed = 14.54 kN*m	Mz,Ed = 0.07 kN*m	Vy,Ed = 0.02 kN
Nc,Rd = 784.21 kN	My,Ed,max = 14.54 kN*m	Mz,Ed,max = -0.07 kN*m	Vy,T,Rd = 310.40 kN
Nb,Rd = 236.91 kN	My,c,Rd = 67.08 kN*m	Mz,c,Rd = 13.66 kN*m	Vz,Ed = -10.41 kN
	MN,y,Rd = 67.08 kN*m	MN,z,Rd = 13.66 kN*m	Vz,T,Rd = 215.39 kN
	Mb,Rd = 45.75 kN*m		Tt,Ed = 0.00 kN*m
			Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 0.00	Mcr = 62.98 kN*m	Curve,LT -	XLT = 0.61
Lcr,upp=5.38 m	Lam_LT = 1.03	fi,LT = 1.15	XLT,mod = 0.68

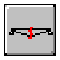
BUCKLING PARAMETERS:

About y axis:		About z axis:	
Ly = 5.38 m	Lam_y = 0.44	Lz = 5.38 m	Lam_z = 1.62
Lcr,y = 3.77 m	Xy = 0.94	Lcr,z = 3.77 m	Xz = 0.30
Lamy = 41.32	kyy = 0.96	Lamz = 151.99	kyz = 0.81

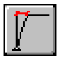
Torsional buckling:		Flexural-torsional buckling	
Curve,T=b	alfa,T=0.34	Curve,TF=b	alfa,TF=0.34
Lt=5.38 m	fi,T=1.02	Ncr,y=4050.67 kN	fi,TF=0.64

Ncr,T=984.95 kN	X,T=0.67	Ncr,TF=4050.67 kN	X,TF=0.91
Lam_T=0.89	Nb,T,Rd=522.39 kN	Lam_TF=0.44	Nb,TF,Rd=713.53 kN

LIMIT DISPLACEMENTS



Deflections (LOCAL SYSTEM): *Not analyzed*



Displacements (GLOBAL SYSTEM):

vx = 0.6 cm < vx max = L/150.00 = 3.6 cm Verified

vy = 0.3 cm < vy max = L/150.00 = 3.6 cm Verified

Section OK !!!

III.8. Horizontal braces

III.8.1. Charges to consider

$$F_1 = \left(W \times \frac{h_1}{2} \times \frac{b}{8}\right) + \frac{F_{fr}}{8} = \left(0.462 \times \frac{4.5}{2} \times \frac{16}{8}\right) + \frac{0}{8} = 2.08\ KN$$

$$F_2 = \left(W \times \frac{h_2}{2} \times \frac{b}{4}\right) + \frac{F_{fr}}{4} = \left(0.462 \times \frac{5.3}{2} \times \frac{16}{4}\right) + \frac{0}{4} = 4.97\ KN$$

III.8.2. Calculation of reactions

$$R_A = R_B = F_1 + F_2 = 7.05\ KN$$

$$\tan\theta = \frac{4.4}{4} = 1.1 \rightarrow \theta = 47.73°$$

$$F_d = \frac{R - F_1}{\cos\theta} = \frac{7.05 - 2.08}{\cos 47.73} = \frac{4.97}{0.67} = 7.42\ KN$$

$$N_{Sd} = 1.5\ F_d = 1.5 \times 7.42 = 11.13\ KN$$

$$N_{Sd} \leq N_{PL,Rd} = \frac{Af_y}{\gamma M_0} \rightarrow A \geq \frac{N_{Sd} \times \gamma M_0}{f_y} = \frac{11.13 \times 1.1}{235 \times 10^{-1}} = 0.52\ cm^2$$

For practical reasons, we choose an insulated angle iron of $70 \times 70 \times 7$ With a 10 mm bolt and 11 mm holes

$$A_{net} = 9.4 - 0.5 * 1 = 8.75\ cm^2$$

$$N_{U,Rd} = \frac{\beta \times A_{net} \times f_u}{\gamma M_2} = \frac{0.7 \times 8.75 \times 360 \times 10^{-1}}{1.25} = 176.4\ KN$$

$N_{Sd} = 11.13\ KN\ < N_{U,Rd} = 176.4\ KN$

III.8.3. Verification of resistance to ULS (ELU)

Deviated Compound Bending Verification

$$\left(\frac{M_{Y,Sd}}{M_{NY,Rd}}\right)^{\alpha} + \left(\frac{M_{Z,Sd}}{M_{NZ,Rd}}\right)^{\beta} \leq 1$$

$\alpha = 2$ et $\beta = 1$

$Q_{sd} = q_{uz} = 1.89\ KN/m$

$N_{sd} = q_{uy} = 0.38\ KN/m$

III.8.4. Verification of resistance section

$V_{Z,Sd} \leq 0.5\ V_{PL,Rd}$

$$V_{PL,Rd} = \frac{A_{PLZ} f_y}{\sqrt{3}\gamma M_0} = \frac{7.62 \times 235 \times 10^{-1}}{\sqrt{3} \times 1.1} = \frac{118.44}{1.90} = 179\ KN$$

$A_{PLZ} = A - \left(2b \times t_f\right) + \left[(t_w + 2r)t_f\right]$

$\rightarrow A_{PLZ} = 16.4 - (2 \times 7.3 \times 0.69) + [(0.47 + 2 \times 0.7)0.69] = 5.04\ cm^2$

$V_{Z,Sd} = \frac{q_z l}{2} = \frac{1.89 \times 4}{2} = 3.78\ KN < 0.5\ V_{PL,Rd} = 89.5$ Verified

III.8.5. Verification of the section at normal effort

$$N_{Sd} \leq \min\left(0.25\ N_{PL,Rd}\ ;\ \frac{0.5 A_w f_y}{\gamma M_1}\right)$$

$$N_{PL,Rd} = \frac{A f_y}{\gamma M_1} = \frac{16.4 \times 235 \times 10^{-1}}{1.1} = 350.3\ KN$$

$\rightarrow 0.25\ N_{PL,Rd} = 0.25 \times 350 = 87.6\ KN$

$$\frac{0.5 A_w f_y}{\gamma M_1} = \frac{0.5 \times 6.33 \times 235 \times 10^{-1}}{1.1} = 67.6\ KN$$

$A_w = A - \left(2b \times t_f\right) = 16.4 - (2 \times 7.3 \times 0.69) = 6.33\ cm^2$

$N_{Sd} = 3.70\ KN < \min(87.6\ ;\ 67.6)$ Verified

III.8.6. Autodesk ROBOT analysis result

STEEL DESIGN

CODE: *NF EN 1993-1:2005/NA:2007/AC:2009, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 290 Simple bar_290 **POINT:** 2 **COORDINATE:** x = 0.17 L = 1.01 m

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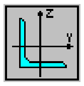


SECTION PARAMETERS: CAE 100x7

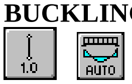
h=10.0 cm	gM0=1.00	gM1=1.00	
b=10.0 cm	Ay=7.00 cm2	Az=7.00 cm2	Ax=13.66 cm2
tw=0.7 cm	Iy=128.20 cm4	Iz=128.20 cm4	Ix=2.21 cm4
tf=0.7 cm	Wely=17.54 cm3	Welz=17.54 cm3	
	Weff,y=17.54 cm3		Aeff=13.66 cm2

Attention: Section of the 4 class! The program does not perform a full analysis of the 4 class for these section types; they are treated as the 3 class sections.

INTERNAL FORCES AND CAPACITIES:

N,Ed = 17.45 kN	My,Ed = 0.36 kN*m	
Nc,Rd = 321.01 kN	My,Ed,max = 0.65 kN*m	
Nb,Rd = 61.20 kN	My,c,Rd = 4.12 kN*m	Vz,Ed = 0.28 kN
		Vz,c,Rd = 94.97 kN
		Class of section = 4

BUCKLING PARAMETERS:



About y axis:

About z axis:

Ly = 6.06 m	Lam_y = 2.11	Lz = 6.06 m	Lam_z = 2.11
Lcr,y = 6.06 m	Xy = 0.19	Lcr,z = 6.06 m	Xz = 0.19
Lamy = 197.97	kyy = 1.06	Lamz = 197.97	kzy = 1.06

VERIFICATION FORMULAS:
Section strength check:
My,Ed/My,c,Rd = 0.09 < 1.00 (6.2.5.(1))
N,Ed/Nc,Rd + My,Ed/My,c,Rd = 0.14 < 1.00 (6.2.1(7))
Vz,Ed/Vz,c,Rd = 0.00 < 1.00 (6.2.6.(1))
Global stability check of member:
Lambda,y = 197.97 < Lambda,max = 210.00 Lambda,z = 197.97 < Lambda,max = 210.00 STABLE
N,Ed/(Xmin*N,Rk/gM1) + kyy*My,Ed,max/(XLT*My,Rk/gM1) = 0.45 < 1.00 (6.3.3.(4))
N,Ed/(Xmin*N,Rk/gM1) + kzy*My,Ed,max/(XLT*My,Rk/gM1) = 0.45 < 1.00 (6.3.3.(4))

Section OK !!!

III.9. Vertical braces

III.9.1. Tractive effort in the mowed diagonal

$N\cos\alpha = R - F_1$

$\tan\alpha = \dfrac{4.5}{4} = 1.125\ ;\ \alpha = \tan^{-1} 1.125 = 48.39°$

$N = \dfrac{R - F_1}{\cos\alpha} = \dfrac{9.69 - 2.08}{\cos 48.39} = 11.45\ KN$

III.9.2. Diagonal section

$N_{Sd} = N = 11.45\ KN$

$N_{Sd} \leq N_{PL,Rd} = \dfrac{Af_y}{\gamma M_0} \rightarrow A \geq \dfrac{N_{Sd} \times \gamma M_0}{f_y} = \dfrac{11.45 \times 1.1}{235 \times 10^{-1}} = 0.54\ cm^2$

$N_{U,Rd} = \dfrac{\beta \times A_{net} \times f_u}{\gamma M_2} = \dfrac{0.7 \times 8.75 \times 360 \times 10^{-1}}{1.25} = 176.4\ KN$

$N_{Sd} = 11.45\ KN\ < N_{U,Rd} = 176.4\ KN$

III.9.3. Verification of resistance to ULS (ELU)

$Q_{sd} = q_{uz} = 0.38\ KN/m$

$N_{sd} = q_{uy} = 0.107\ KN/m$

- Verification of resistance section

$V_{Z,Sd} \leq 0.5\ V_{PL,Rd}$

$V_{PL,Rd} = \dfrac{A_{PLZ} f_y}{\sqrt{3}\gamma M_0} = \dfrac{5.04 \times 235 \times 10^{-1}}{\sqrt{3} \times 1.1} = \dfrac{118.44}{1.90} = 62.34\ KN$

$A_{PLZ} = A - (2b \times t_f) + [(t_w + 2r)t_f]$

$\rightarrow A_{PLZ} = 16.4 - (2 \times 7.3 \times 0.69) + [(0.47 + 2 \times 0.7)0.69] = 5.04\ cm^2$

$V_{Z,Sd} = \frac{q_z l}{2} = \frac{0.38 \times 4}{2} = 0.76\ KN < 0.5\ V_{PL,Rd} = 31.17$ verified

- Verification of the section at normal effort

$N_{Sd} \leq \min\left(0.25\ N_{PL,Rd}\ ;\ \dfrac{0.5 A_w f_y}{\gamma M_1}\right)$

$$N_{PL,Rd} = \frac{Af_y}{\gamma M_1} = \frac{16.4 \times 235 \times 10^{-1}}{1.1} = 350.3\ KN$$

$$\rightarrow 0.25\ N_{PL,Rd} = 0.25 \times 350 = 87.6\ KN$$

$$\frac{0.5 A_w f_y}{\gamma M_1} = \frac{0.5 \times 6.33 \times 235 \times 10^{-1}}{1.1} = 67.6\ KN$$

$$A_w = A - \left(2b \times t_f\right) = 16.4 - (2 \times 7.3 \times 0.69) = 6.33\ cm^2$$

$N_{Sd} = 1.35\ G = 1.35 \times 0.38 = 0.51\ KN < \min(87.6\ ;\ 67.6)$ Verified

- Deviated Compound Bending Verification

$$\left(\frac{M_{Y,Sd}}{M_{NY,Rd}}\right)^{\alpha} + \left(\frac{M_{Z,Sd}}{M_{NZ,Rd}}\right)^{\beta} \leq 1$$

$\alpha = 2$ et $\beta = 1$

$$M_{Y,Sd} = \frac{q_z l^2}{8} = \frac{0.38 \times 4^2}{8} = 0.21\ KN.m$$

$$M_{Z,Sd} = \frac{q_y l^2}{8} = \frac{0.107 \times 4^2}{8} = 0.76\ KN.m$$

$$M_{NY,Rd} = M_{PLY,Rd} = \frac{W_{PLY} \times f_y}{\gamma M_0} = \frac{83.3 \times 235 \times 10^{-3}}{1.1} = 17.80\ KN.m$$

$$M_{NZ,Rd} = M_{PLZ,Rd} = \frac{W_{PLZ} \times f_y}{\gamma M_0} = \frac{19.2 \times 235 \times 10^{-3}}{1.1} = 4.10\ KN.m$$

$$\left(\frac{M_{Y,Sd}}{M_{NY,Rd}}\right)^{\alpha} + \left(\frac{M_{Z,Sd}}{M_{NZ,Rd}}\right)^{\beta} = \left(\frac{0.21}{17.80}\right)^{2} + \left(\frac{0.76}{4.10}\right)^{1} = 0.0009 + 0.126 = 0.13 < 1 \text{ verified}$$

After calculating in ROBOT, ROBOT suggests an insulated angle iron of
$100 \times 100 \times 7$

So, we take an insulated angle iron of $100 \times 100 \times 7$ With a 10 mm bolt and 11 mm holes

III.9.4. Autodesk ROBOT analysis result

STEEL DESIGN

CODE: *NF EN 1993-1:2005/NA:2007/AC:2009, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 284 sidebracing_284 **POINT:** 2 **COORDINATE:** x = 0.17 L = 1.00 m

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LOADS:
Governing Load Case: 16 ULS /105/ 1*1.35 + 2*1.35 + 3*1.35 + 4*1.35 + 5*1.35 + 6*1.35 + 7*1.05 + 8*1.05 + 9*1.05 + 14*1.50

MATERIAL:
ACIER fy = 235.00 MPa

SECTION PARAMETERS: CAE 100x7

h=10.0 cm	gM0=1.00	gM1=1.00	
b=10.0 cm	Ay=7.00 cm2	Az=7.00 cm2	Ax=13.66 cm2
tw=0.7 cm	Iy=128.20 cm4	Iz=128.20 cm4	Ix=2.21 cm4
tf=0.7 cm	Wely=17.54 cm3	Welz=17.54 cm3	
	Weff,y=17.54 cm3		Aeff=13.66 cm2

Attention: Section of the 4 class! The program does not perform a full analysis of the 4 class for these section types; they are treated as the 3 class sections.

INTERNAL FORCES AND CAPACITIES:

N,Ed = 13.75 kN	My,Ed = 0.24 kN*m	
Nc,Rd = 321.01 kN	My,Ed,max = 0.43 kN*m	
Nb,Rd = 62.01 kN	My,c,Rd = 4.12 kN*m	Vz,Ed = 0.19 kN
		Vz,c,Rd = 94.97 kN
		Class of section = 4

BUCKLING PARAMETERS:

About y axis:		About z axis:	
Ly = 6.02 m	Lam_y = 2.09	Lz = 6.02 m	Lam_z = 2.09
Lcr,y = 6.02 m	Xy = 0.19	Lcr,z = 6.02 m	Xz = 0.19
Lamy = 196.53	kyy = 1.04	Lamz = 196.53	kzy = 1.04

VERIFICATION FORMULAS:
Section strength check:
My,Ed/My,c,Rd = 0.06 < 1.00 (6.2.5.(1))
N,Ed/Nc,Rd + My,Ed/My,c,Rd = 0.10 < 1.00 (6.2.1(7))
Vz,Ed/Vz,c,Rd = 0.00 < 1.00 (6.2.6.(1))
Global stability check of member:
Lambda,y = 196.53 < Lambda,max = 210.00 Lambda,z = 196.53 < Lambda,max = 210.00 STABLE
N,Ed/(Xmin*N,Rk/gM1) + kyy*My,Ed,max/(XLT*My,Rk/gM1) = 0.33 < 1.00 (6.3.3.(4))
N,Ed/(Xmin*N,Rk/gM1) + kzy*My,Ed,max/(XLT*My,Rk/gM1) = 0.33 < 1.00 (6.3.3.(4))

Section OK !!!

CHAPTER FOUR

STRUCTURAL ELEMENTS

IV.1. The Rafters

IV.1.1. Introduction

Rafters are an essential part on metal roofs. they help to support the purlin and roof deck. The rafters are a series of metal beams that serve as sloped structural pieces, and there is two types, to know what system we use a single beam or truss system we must check the following condition

$$\frac{h}{l} = \frac{1.2}{12} = 0.1$$

So we choose a single beam system

Table 13 Characteristic of Section IPE 300

Profile	h (cm)	b (cm)	t_w (cm)	t_f (cm)	r (cm)	P (Kg/m)	d (cm)
IPE300	30	15	0.71	1.27	1.5	42.2	24.86
	A (cm2)	**I_y (cm4)**	**i_y (cm)**	**W_{ply} cm3)**	**I_z (cm4)**	**W_{plz} (cm3)**	**i_z (cm)**
	53.8	8356	12.5	628	604	125	3.35

IV.1.2. Loads

a) Dead load
Rafters weight: 42.2 kg/m
Purlin weight: 12.9 kg/m
Dropped Ceiling: 17 kg/m^2
Cover weight: 20 kg/m^2

b) Live load
Sand load: 20 kg/m^2
Maintenance agents: 66.7 kg/m

c) Live load
Wind load: 0.446 KN/m^2

IV.1.3. Verification of deflection at SLS

$q = q_{uz} * 5 = 1.89 * 5 = 9.45\ KN/m$

$$f = \frac{5ql^4}{384EI} < f_{adm} = \frac{l}{200} = \frac{612}{200} = 2\ cm$$

$$I_y > \frac{5ql^3}{2 \times 384E} = \frac{5 \times 200 \times 9.45 \times 10^{-2} \times 612^3}{2 \times 384 \times 21000} = 1343.09\ cm^4$$

IV.1.4. Verification of resistance to ULS

Deviated bending verification

$$\left(\frac{M_{Y,Sd}}{M_{PL,Y}}\right)^{\alpha} + \left(\frac{M_{Z,Sd}}{M_{PL,Z}}\right)^{\beta} \leq 1$$

$\alpha = 2$ (For profiles in I Shape) et $\beta = 1$

$$M_{Y,Sd} = \frac{q_z l^2}{8} = \frac{13.7 \times 4^2}{8} = 27.4\ KN.m$$

$$M_{Z,Sd} = \frac{q_y l^2}{8} = \frac{2.75 \times 4^2}{8} = 5.5\ KN.m$$

$$M_{PL} = \frac{W_{PL} \times f_y}{\gamma M_0}$$

For γM_0 we must determine the class of the profile

Web: $\frac{d}{t_w} = \frac{248.6}{7.1} = 35.01 \leq 72\varepsilon$ et $\varepsilon = \sqrt{\frac{235}{235}} = 1$ So: the class of the Web is 1

Flange: $\frac{b - t_w}{2t_f} = \frac{150 - 7.1}{2 \times 12.7} = 5.63 \leq 9\varepsilon$ et $\varepsilon = \sqrt{\frac{235}{235}} = 1$ So: the class of the Flange is 1

So, the class section is 1 $\rightarrow \gamma M_0 = 1.1$

$$M_{PL,Y} = \frac{W_{PLY} \times f_y}{\gamma M_0} = \frac{628 \times 235 \times 10^{-3}}{1.1} = 134.16\ KN.m$$

$$M_{PL,Z} = \frac{W_{PLZ} \times f_y}{\gamma M_0} = \frac{125 \times 235 \times 10^{-3}}{1.1} = 26.7\ KN.m$$

$$\left(\frac{M_{Y,Sd}}{M_{PL,Y}}\right)^{\alpha} + \left(\frac{M_{Z,Sd}}{M_{PL,Z}}\right)^{\beta} = \left(\frac{27.4}{134.16}\right)^2 + \left(\frac{5.5}{26.7}\right)^1 = 0.042 + 0.206 = 0.248 < 1$$

IV.1.5. Shear verification

$V_{Z,Sd} \leq V_{PL,Rd}$ and $V_{Y,Sd} \leq V_{PL,Rd}$

$$V_{PL,Rd} = \frac{A_{PLZ} f_y}{\sqrt{3}\gamma M_0} = \frac{12.03 \times 235 \times 10^{-1}}{\sqrt{3} \times 1.1} = \frac{282.76}{1.90} = 135.53\ KN$$

$$A_{PLZ} = A - \left(2b \times t_f\right) + \left[(t_w + 2r)t_f\right]$$

$$\rightarrow A_{PLZ} = 16.4 - (2 \times 12 \times 0.98) + [(0.62 + 2 \times 1.5)0.98] = 12.03\ cm^2$$

$$V_{Z,Sd} = \frac{q_z l}{2} = \frac{13.7 \times 4}{2} = 27.4\ KN < V_{PL,Rd} = 135.53\ KN$$

$$V_{Y,Sd} = \frac{q_y l}{2} = \frac{2.75 \times 4}{2} = 3.8\ KN < V_{PL,Rd} = 135.53\ KN$$

IV.1.6. Spill verification

$M_{Y,Sd} \leq M_{b,Rd}$

The section class 1 → $\gamma M_1 = 1.1$	$\beta_w = 1$ class 1

$$M_{b,Rd} = \frac{X_{lt} \times \beta_w \times W_{PLY} \times f_y}{\gamma M_1}$$

$$X_{lt} = \frac{1}{\phi_{LT} + ({\phi_{LT}}^2 - {\overline{\lambda_{LT}}}^2)^{0.5}}$$

$$\overline{\lambda_{LT}} = \frac{\lambda_{LT}}{\lambda_1}\sqrt{\beta_w}$$

$$\lambda_1 = \pi\sqrt{\frac{E}{f_y}} = 93.3\varepsilon \text{ et } \varepsilon = \sqrt{\frac{235}{f_y}} = 1 \text{ donc } \lambda_1 = 93.3$$

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

$C_1 = 1.132$

$$\lambda_{LT} = \frac{\frac{400}{3.35}}{\sqrt{1.132}\left[1+\frac{1}{20}\left(\frac{\frac{400}{3.35}}{\frac{30}{1.27}}\right)^2\right]^{0.25}} = 91.35$$

$$\overline{\lambda_{LT}} = \frac{91.35}{93.3}\sqrt{1} = 0.98 > 0.4$$

There is a risk of spillage

$$\phi_{LT} = 0.5\left[1+\alpha_{LT}\left(\overline{\lambda_{LT}}-0.2\right)+\overline{\lambda_{LT}}^2\right] \text{ With } \alpha_{LT} = 0.21 \text{ for rolled profiles}$$

$$\phi_{LT} = 0.5[1+0.21(0.98-0.2)+0.98^2] = 1.06$$

$$X_{lt} = \frac{1}{1.06+(1.06^2-0.98^2)^{0.5}} = 0.68$$

$$M_{b,Rd} = \frac{0.68 \times 1 \times 628 \times 235 \times 10^{-3}}{1.1} = 91.25\ KN.m$$

Therefor

$$M_{Y,Sd} = 27.4\ KN*m < M_{b,Rd} = 91.25\ KN*m$$

The stability of the rafters to overturning is checked, an IPE 300 is adopted.

IV.1.7. The results of Autodesk ROBOT

The most effective combination "16 ULS /7"

STEEL DESIGN

CODE: *NF EN 1993-1:2005/NA:2007/AC:2009, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 38 rafter_38 **POINT:** 7 **COORDINATE:** x = 0.01 L = 0.06 m

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LOADS:
Governing Load Case: 16 ULS /7/ 1*1.35 + 2*1.35 + 3*1.35 + 4*1.35 + 5*1.35 + 6*1.35 + 7*1.50 + 8*1.50 + 9*1.50 + 15*0.90

MATERIAL:

ACIER fy = 235.00 MPa

SECTION PARAMETERS: IPE 300

h=57.1 cm	gM0=1.00	gM1=1.00	
b=15.0 cm	Ay=32.10 cm2	Az=38.99 cm2	Ax=88.31 cm2
tw=0.7 cm	Iy=35682.49 cm4	Iz=905.49 cm4	Ix=29.11 cm4
tf=1.1 cm	Wely=1244.91 cm3	Welz=120.73 cm3	

INTERNAL FORCES AND CAPACITIES:

N,Ed = 57.68 kN	My,Ed = 27.55 kN*m	Mz,Ed = -10.29 kN*m	Vy,Ed = 0.30 kN
Nc,Rd = 2075.32 kN	My,el,Rd = 292.55 kN*m	Mz,el,Rd = 28.37 kN*m	Vy,T,Rd = 242.69 kN
Nb,Rd = 2046.98 kN	My,c,Rd = 292.55 kN*m	Mz,c,Rd = 28.37 kN*m	Vz,Ed = 9.68 kN
			Vz,T,Rd = 389.65 kN
	Mb,Rd = 292.55 kN*m		Tt,Ed = 2.05 kN*m
			Class of section = 3

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 616431.98 kN*m	Curve,LT - d	XLT = 1.00
Lcr,upp=0.06 m	Lam_LT = 0.02	fi,LT = 0.43	XLT,mod = 1.00

BUCKLING PARAMETERS:

About y axis:		About z axis:	
Ly = 6.12 m	Lam_y = 0.24	Lz = 6.12 m	Lam_z = 0.02
Lcr,y = 3.06 m	Xy = 0.99	Lcr,z = 0.06 m	Xz = 1.00
Lamy = 22.39	kzy = 1.00	Lamz = 1.79	kzz = 1.00

Torsional buckling:		Flexural-torsional buckling	
Curve,T=c	alfa,T=0.49	Curve,TF=c	alfa,TF=0.49
Lt=3.06 m	fi,T=1.12	Ncr,y=3823118.01 kN	fi,TF=1.12
Ncr,T=2366.00 kN	X,T=0.58	Ncr,TF=2366.00 kN	X,TF=0.58
Lam_T=0.94	Nb,T,Rd=1198.59 kN	Lam_TF=0.94	Nb,TF,Rd=1198.59 kN

VERIFICATION FORMULAS:

Section strength check:

N,Ed/Nc,Rd + My,Ed/My,c,Rd + Mz,Ed/Mz,c,Rd = 0.48 < 1.00 (6.2.1(7))
sqrt(Sig,x,Ed^2 + 3*(Tau,ty,Ed)^2)/(fy/gM0) = 0.99 < 1.00 (6.2.1.(5))
Vy,Ed/Vy,T,Rd = 0.00 < 1.00 (6.2.6-7)
Vz,Ed/Vz,T,Rd = 0.02 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.86 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.57 < 1.00 (6.2.6)

Global stability check of member:

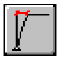
Lambda,y = 22.39 < Lambda,max = 210.00 Lambda,z = 1.79 < Lambda,max = 210.00 STABLE
N,Ed/Min(Nb,Rd,Nb,T,Rd,Nb,TF,Rd) = 0.05 < 1.00 (6.3.1)
My,Ed/Mb,Rd = 0.09 < 1.00 (6.3.2.1.(1))
N,Ed/(Xmin*N,Rk/gM1) + kyy*My,Ed/(XLT*My,Rk/gM1) + kyz*Mz,Ed/(Mz,Rk/gM1) = 0.50 < 1.00 (6.3.3.(4))
N,Ed/(Xmin*N,Rk/gM1) + kzy*My,Ed/(XLT*My,Rk/gM1) + kzz*Mz,Ed/(Mz,Rk/gM1) = 0.50 < 1.00 (6.3.3.(4))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):

uy = 0.1 cm < uy max = L/200.00 = 3.1 cm Verified

Governing Load Case: 19 SLS /7/ 1*1.00 + 2*1.00 + 3*1.00 + 4*1.00 + 5*1.00 + 6*1.00 + 7*1.00 + 8*1.00 + 9*1.00 + 15*0.60
uz = 0.5 cm < uz max = L/200.00 = 3.1 cm Verified
Governing Load Case: 19 SLS /91/ 1*1.00 + 2*1.00 + 3*1.00 + 4*1.00 + 5*1.00 + 6*1.00 + 7*0.70 + 9*0.70 + 14*1.00



Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

IV.2. The Columns

IV.2.1. Charges to consider

$H_{Post} = 4.5\ m$ (Column Height)

$e_{Post} = 4.0\ m$ (The distance between the Columns)

action of the wind (W)

$W = 0.846 \times 4.0 = 3.38\ KN/m$

Permanent loads (G)

$$G = \left(P_{Cladding} \times e_{Post} \times \frac{B}{2}\right) + \left(P_{Dropped\ ceiling} \times e_{Post} \times \frac{B}{2}\right) + (P_{Post} \times H_{Post}) + \left(P_{rafter} * L_{rafter}\right) + (P_{putlin} * N_{purlin} * e_{Post})$$

IV.2.2. Column pre-sizing

$$f_z = \frac{5Wl^4}{384EI} < \frac{l}{200} = \frac{450}{200} = 2.25\ cm$$

$$I_y > \frac{5Wl^4}{2.25 \times 384E} = \frac{5 \times 0.0338 \times 450^4}{2.25 \times 384 \times 21000} = 381.95\ cm^4$$

So, we chose section HEA180

Table 14 Characteristic of HEA 180

Profile	h (cm)	b (cm)	t_w (cm)	t_f (cm)	r (cm)	P (Kg/m)	d (cm)
HEA180	17.1	18	0.6	0.95	1.5	35.5	12.2

	A (cm2)	I_y (cm4)	i_y (cm)	W_{ply} cm3)	I_z (cm4)	W_{plz} (cm3)	i_z (cm)
	45.3	2510	7.45	324.9	924.6	156.5	4.52

$$G = \left(0.2 \times 4.0 \times \frac{12}{2}\right) + \left(0.17 \times 4.0 \times \frac{12}{2}\right) + (0.355 \times 4.5) + (0422 * 6.12) + (2.74 * 5 * 4)$$

$$\rightarrow G = 67.86\ KN$$

$$B = 12\ m$$

IV.2.3. Verification of resistance section

$$V_{Z,Sd} \leq 0.5\ V_{PL,Rd}$$

$$V_{PL,Rd} = \frac{A_{PLZ} f_y}{\sqrt{3}\gamma M_0} = \frac{14.52 \times 235 \times 10^{-1}}{\sqrt{3} \times 1.1} = \frac{341.22}{1.90} = 17.09\ KN$$

$$A_{PLZ} = A - \left(2b \times t_f\right) + \left[(t_w + 2r)t_f\right]$$

$$\rightarrow A_{PLZ} = 45.3 - (2 \times 18 \times 0.95) + [(0.6 + 2 \times 1.5)0.95] = 14.52\ cm^2$$

$$V_{Z,Sd} = \frac{q_z l}{2} = \frac{3.38 \times 4.5}{2} = 7.6\ KN < 0.5\ V_{PL,Rd} = 89.55\ KN$$

IV.2.4. Verification of the section at normal force

$$N_{Sd} \leq \min\left(0.25\ N_{PL,Rd}\ ;\ \frac{0.5 A_w f_y}{\gamma M_1}\right)$$

$$N_{PL,Rd} = \frac{A f_y}{\gamma M_1} = \frac{45.3 \times 235 \times 10^{-1}}{1.1} = 967.77\ KN$$

$$\rightarrow 0.25\ N_{PL,Rd} = 0.25 \times 967.77 = 241.94\ KN$$

$$\frac{0.5 A_w f_y}{\gamma M_1} = \frac{0.5 \times 11.1 \times 235 \times 10^{-1}}{1.1} = 118.57\ KN$$

$$A_w = A - \left(2b \times t_f\right) = 45.3 - (2 \times 18 \times 0.95) = 11.1\ cm^2$$

$$N_{Sd} = 1.35\ G = 1.35 \times 67.86 = 101.79K\ N < \min(967.77\ ;\ 118.57)$$

IV.2.5. Checking the resistance element

Compound bending verification with risk of buckling

$$\frac{N_{Sd}}{X_{min} \times N_{PL,Rd}} + \frac{K_Y \times M_{Y,Sd}}{M_{PL,Rd}} \leq 1$$

Compound bending verification with risk of overturning

$$\frac{N_{Sd}}{X_Z \times N_{PL,Rd}} + \frac{K_{LT} \times M_{Y,Sd}}{X_{LT} \times M_{PL,Rd}} \leq 1$$

$K_Y = 1 \rightarrow$ class 1 ; $K_{LT} = 1 \rightarrow$ class 1

$$X_{min} = min(X_Y; X_Z)$$

$$X_Y = \frac{1}{\phi_Y + \left(\phi_Y{}^2 - \overline{\lambda_Y}^2\right)^{0.5}} \qquad \overline{\lambda_Y} = \frac{\lambda_Y}{\lambda_1}\sqrt{\beta_w} \qquad \beta_w = 1 \text{ class } 1$$

The class section 1 $\rightarrow \gamma M_1 = 1.1$

$$\lambda_1 = \pi\sqrt{\frac{E}{f_y}} = 93.3\varepsilon \text{ et } \varepsilon = \sqrt{\frac{235}{f_y}} = 1 \text{ donc } \lambda_1 = 93.3$$

$$\lambda_Y = \frac{l_y}{i_y} = \frac{450}{7.45} = 60.40 \qquad \overline{\lambda_Y} = \frac{60.40}{93.3}\sqrt{1} = 0.65$$

$$\phi_Y = 0.5\left[1 + \alpha_Y\left(\overline{\lambda_Y} - 0.2\right) + \overline{\lambda_Y}^2\right] \text{ With } \alpha_Y = 0.21$$

$$\phi_Y = 0.5[1 + 0.21(0.65 - 0.2) + 0.65^2] = 0.76$$

$$X_Y = \frac{1}{0.76 + (0.76^2 - 0.39^2)^{0.5}} = 1.05$$

$$X_Z = \frac{1}{\phi_Z + (\phi_Z{}^2 - \overline{\lambda_Z}^2)^{0.5}} \qquad \overline{\lambda_Z} = \frac{\lambda_Z}{\lambda_1}\sqrt{\beta_w}$$

$\beta_w = 1 \rightarrow$ class 1

The class section 1 → $\gamma M_1 = 1.1$

$$\lambda_1 = \pi\sqrt{\frac{E}{f_y}} = 93.3\varepsilon \text{ et } \varepsilon = \sqrt{\frac{235}{f_y}} = 1 \text{ donc } \lambda_1 = 93.3$$

$$\lambda_Z = \frac{l_Z}{i_Z} = \frac{450}{4.52} = 99.56 \qquad \overline{\lambda_Z} = \frac{99.56}{93.3}\sqrt{1} = 1.07$$

$$\phi_Z = 0.5\left[1 + \alpha_Z\left(\overline{\lambda_Z} - 0.2\right) + \overline{\lambda_Z}^2\right] \text{ with } \alpha_Z = 0.21$$

$$\phi_Z = 0.5[1 + 0.21(1.07 - 0.2) + 1.07^2] = 1.16$$

$$X_Z = \frac{1}{1.16 + (1.16^2 - 1.07^2)^{0.5}} = 0.62$$

$$X_{min} = min(1.05; 0.62) = 0.62$$

$$X_{lt} = \frac{1}{\phi_{LT} + (\phi_{LT}^2 - \overline{\lambda_{LT}}^2)^{0.5}} \qquad \overline{\lambda_{LT}} = \frac{\lambda_{LT}}{\lambda_1}\sqrt{\beta_w}$$

$$\lambda_1 = 93.3\,;\ C_1 = 1.132$$

$$\lambda_{LT} = \frac{\frac{l}{i_z}}{\sqrt{C_1}\left[1 + \frac{1}{20}\left(\frac{\frac{l}{i_z}}{\frac{h}{t_f}}\right)^2\right]^{0.25}} = \frac{\frac{450}{4.52}}{\sqrt{1.132}\left[1 + \frac{1}{20}\left(\frac{\frac{450}{4.52}}{\frac{17}{0.95}}\right)^2\right]^{0.25}} = 74.20$$

$$\overline{\lambda_{LT}} = \frac{74.20}{93.3}\sqrt{1} = 0.8 > 0.4\, There\ is\ a\ risk\ of\ spillage$$

$$\phi_{LT} = 0.5\left[1 + \alpha_{LT}\left(\overline{\lambda_{LT}} - 0.2\right) + \overline{\lambda_{LT}}^2\right] \text{ with } \alpha_{LT} = 0.21$$

$$\phi_{LT} = 0.5[1 + 0.21(0.8 - 0.2) + 0.8^2] = 0.88$$

$$X_{lt} = \frac{1}{0.88 + (0.88^2 - 0.8^2)^{0.5}} = 0.8$$

$$M_{Y,Sd} = \frac{q_{z,sd}l^2}{8} = \frac{3.38 \times 4.5^2}{8} = 8.55\ KN.m$$

$$M_{PL,Rd} = \frac{W_{PLY} \times f_y}{\gamma M_1} = \frac{69.41 \times 235 \times 10^{-3}}{1.1} = 69.41\ KN.m$$

Buckling check

$$\frac{101.79}{0.62 \times 967.77} + \frac{1 \times 8.55}{69.41} = 0.169 + 0.123 = 0.29 < 1$$

Spill verification

$$\frac{101.79}{0.62 \times 967.77} + \frac{1 \times 8.55}{0.8 \times 69.41} = 0.169 + 0.154 = 0.32 < 1$$

Section HEA180 is verified

IV.2.6. The results of Autodesk ROBOT

The most effective combination “16 ULS /105”

STEEL DESIGN

CODE: *NF EN 1993-1:2005/NA:2007/AC:2009, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:

MEMBER: 40 琥╟n **POINT:** 7 **COORDINATE:** x = 1.00 L = 4.50 m

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LOADS:
Governing Load Case: 16 ULS /105/ 1*1.35 + 2*1.35 + 3*1.35 + 4*1.35 + 5*1.35 + 6*1.35 + 7*1.05 + 8*1.05 + 9*1.05 + 14*1.50

MATERIAL:
ACIER fy = 235.00 MPa

SECTION PARAMETERS: HEA 180

h=17.1 cm	gM0=1.00	gM1=1.00	
b=18.0 cm	Ay=37.93 cm2	Az=14.47 cm2	Ax=45.25 cm2
tw=0.6 cm	Iy=2510.29 cm4	Iz=924.61 cm4	Ix=14.86 cm4
tf=0.9 cm	Wply=324.85 cm3	Wplz=156.49 cm3	

INTERNAL FORCES AND CAPACITIES:

N,Ed = 43.17 kN	My,Ed = -58.29 kN*m	Mz,Ed = -0.00 kN*m	Vy,Ed = 0.00 kN
Nc,Rd = 1063.38 kN	My,Ed,max = -58.29 kN*m		Mz,Ed,max = -0.00 kN*m
	Vy,T,Rd = 514.61 kN		
Nb,Rd = 742.80 kN	My,c,Rd = 76.34 kN*m	Mz,c,Rd = 36.78 kN*m	Vz,Ed = -22.21 kN
	MN,y,Rd = 76.34 kN*m	MN,z,Rd = 36.78 kN*m	Vz,T,Rd = 196.32 kN
	Mb,Rd = 76.34 kN*m		Tt,Ed = -0.00 kN*m
			Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 0.00	Mcr = 825.93 kN*m	Curve,LT -	XLT = 1.00
Lcr,low=2.25 m	Lam_LT = 0.30	fi,LT = 0.55	XLT,mod = 1.00

BUCKLING PARAMETERS:

About y axis: About z axis:

Ly = 4.50 m	Lam_y = 0.68	Lz = 4.50 m	Lam_z = 0.74
Lcr,y = 4.76 m	Xy = 0.79	Lcr,z = 3.15 m	Xz = 0.70
Lamy = 63.91	kyy = 0.63	Lamz = 69.69	kyz = 0.59

Torsional buckling:		Flexural-torsional buckling	
Curve,T=c	alfa,T=0.49	Curve,TF=c	alfa,TF=0.49
Lt=2.25 m	fi,T=0.68	Ncr,y=2295.96 kN	fi,TF=0.85
Ncr,T=4832.77 kN	X,T=0.86	Ncr,TF=2295.96 kN	X,TF=0.74
Lam_T=0.47	Nb,T,Rd=914.63 kN	Lam_TF=0.68	Nb,TF,Rd=783.38 kN

VERIFICATION FORMULAS:
Section strength check:

N,Ed/Nc,Rd = 0.04 < 1.00   (6.2.4.(1))
My,Ed/MN,y,Rd = 0.76 < 1.00   (6.2.9.1.(2))
Mz,Ed/MN,z,Rd = 0.00 < 1.00   (6.2.9.1.(2))
(My,Ed/MN,y,Rd)^ 2.00 + (Mz,Ed/MN,z,Rd)^1.00 = 0.58 < 1.00   (6.2.9.1.(6))
Vy,Ed/Vy,T,Rd = 0.00 < 1.00   (6.2.6-7)
Vz,Ed/Vz,T,Rd = 0.11 < 1.00   (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00   (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00   (6.2.6)

Global stability check of member:

Lambda,y = 63.91 < Lambda,max = 210.00   Lambda,z = 69.69 < Lambda,max = 210.00   STABLE
N,Ed/Min(Nb,Rd,Nb,T,Rd,Nb,TF,Rd) = 0.06 < 1.00   (6.3.1)
My,Ed,max/Mb,Rd = 0.76 < 1.00   (6.3.2.1.(1))
N,Ed/(Xy*N,Rk/gM1) + kyy*My,Ed,max/(XLT*My,Rk/gM1) + kyz*Mz,Ed,max/(Mz,Rk/gM1) = 0.53 < 1.00 (6.3.3.(4))
N,Ed/(Xz*N,Rk/gM1) + kzy*My,Ed,max/(XLT*My,Rk/gM1) + kzz*Mz,Ed,max/(Mz,Rk/gM1) = 0.31 < 1.00 (6.3.3.(4))

Section OK !!!

CHAPTER FIVE

ASSEMBLY STUDY

V.1. Introduction

Assemblies play a crucial role in uniting and joining multiple parts together in a construction. They ensure the transmission and distribution of stresses between the parts without creating additional unwanted stresses, especially torsional stresses. In metal construction, the design and calculation of assemblies are as important as sizing individual parts to ensure overall safety. Since frames typically lack significant redundancies, assemblies become critical points for handling the stresses present in different structural components. If an assembly fails, it jeopardizes the overall functioning of the entire structure.

V.1.1. Column-Rafter connection

V.1.1.1. Soliciting Efforts

$V_{sd} = 37.52\ KN$

$N_{sd} = 69.31\ KN$

$M_{sd} = 61.85\ KN * m$

V.1.1.2. End plate

The plate thickness t_p

We take t_p equal to t_f of the column, So $t_p = 9\ mm$

V.1.1.3. Welding of the end plate

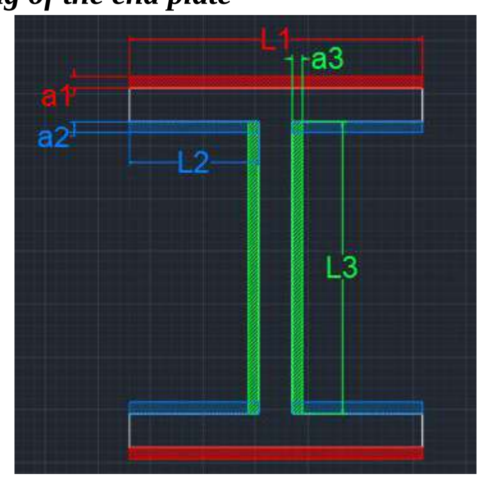


Figure V.1.1:I Different welding areas

- Welding the "web"

We must fulfill the following condition

$$\frac{f_u}{\beta_u * \gamma_{ma}} \geq \sqrt{\left(\frac{N}{\sqrt{2} * a_3 * l_3}\right)^2 + 3 * \left(\frac{V}{2 * a_3 * l_3}\right)^2}$$

- Welding the "flange"

We must fulfill the following condition

$$\frac{f_u}{\beta_u * \gamma_{ma}} \geq \sqrt{\left(\frac{N}{\sqrt{2} * \sum al}\right)^2 + 3 * \left(\frac{N'}{\sqrt{2} * \sum(al)'}\right)^2}$$

$$\sum al = 2 * a_1 * l_1 + 4 * a_1 * l_1 \qquad \sum (al)' = \frac{\sum al}{2}$$

f_y	f_u	β_u	γ_{ma}
235	360	1.25	0.8

Finding the welding thickness of the web a_3

$$\rightarrow a_3 \geq \sqrt{\frac{l_3^2 * \frac{f_u}{\beta_u * \gamma_{ma}}^2}{\frac{N^2}{2} + \frac{3 * V^2}{4}}} = \sqrt{\frac{(274.6^2) * \frac{360 * 10^{-3}}{1.25 * 0.8}^2}{\frac{69.31^2}{2} + \frac{3 * 37.52^2}{4}}} = 1.68\, mm$$

a_3 most be greater than 1.68 mm

Finding the welding thickness of the flange a_1 & a_2

We spouse $a_1 = a_2$ and we get:

$$N' = \frac{M_{sd}}{h_p} = \frac{61.85 * 10^{-3}}{300} = 107.64\, KN$$

h_p : $the\ rafter\ height$

$$\rightarrow a_1 = a_2 = \frac{\sqrt{\frac{\frac{N^2}{2} + \frac{3 * N'^2}{(2/4)}}{(f_u/(\beta_u * \gamma_{ma}))}}}{2 * l_1 + 4 * l_2} = \frac{\sqrt{\frac{\frac{69.31^2}{2} + \frac{3 * 107.64^2}{(2/4)}}{(360 * 10^{-3}/(1.25 * 0.8))}}}{2 * 150 + 4 * 71.45} 0.91\, mm$$

a_3 most be greater than 0.91 mm

So we take $a_1 = a_2 = a_3 = 5\, mm$

Checking the web welding

$$\frac{360 * 10^{-1}}{1.25 * 0.8} \geq \sqrt{\left(\frac{69.31}{\sqrt{2} * 0.5 * 27.46}\right)^2 + 3 * \left(\frac{37.52}{2 * 0.5 * 27.64}\right)^2}$$

$36\ KN/cm^2 \geq 4.28\ KN/cm^2$ verified

Checking the flange welding

$$\sum al = 2 * 0.5 * 15 + 8 * 0.5 * 7.145 = 43.58\ cm^2$$

$$\sum (al)' = \frac{43.58}{2} = 21.79\ cm^2$$

$$\frac{360 * 10^{-1}}{1.25 * 0.8} \geq \sqrt{\left(\frac{69.31}{\sqrt{2} * 43.58}\right)^2 + 3 * \left(\frac{107.64}{\sqrt{2} * 21.79}\right)^2}$$

$36\ KN/cm^2 \geq 6.15\ KN/cm^2$ verified

The rafter welding to the end plate is secure

V.1.1.4. Calculation of bolts:

- Resistance of the assembly to the shear force V_{sd}

$$V_1 = \frac{V_{sd}}{2 * n} \leq F_s = k_a * m * \mu * \frac{F_p}{\gamma_{ms}} \quad ; \quad F_p = 0.7 * f_{ub} * A_s$$

A_s: resistant section of the bolts $A_s = 84.3\ mm^2$

F_s: slip resistance.

K_s: coefficient depending on the size of the drill holes $K_s = 1$

m : friction interface number $m = 1$

μ : coefficient of friction of parts $\mu = 0.3$

F_p: allowable prestressing force in the bolts

γ_{ms}: partial safety factor $\gamma_{ms} = 1.25$

$f_{ub} = 800\ N/mm^2$

$$So, F_s = 1 * 1 * 0.3 * \frac{0.7 * 800 * 10^{-3} * 84.3}{1.25} = 11.33\ KN$$

$$n \geq \frac{V_{sd}}{2 * F_s} = \frac{37.52}{2 * 11.33} = 1.66 \approx 2$$

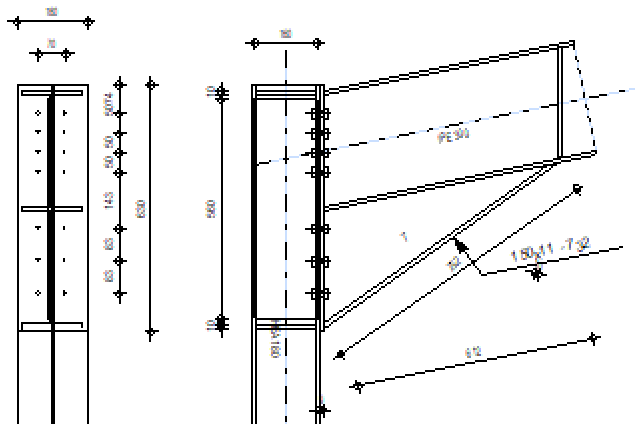


Figure V.1.1:II View of the overall assembly

For a design purpose we take 7 bolts in line

- Arrangement of bolts

$2.2 * d_0 \leq P_1 \leq \min\,(12t_p\,,150\;mm)$

$d_0 = \emptyset + 2 = 12 + 2 = 14\;mm$ | $t_p = 9\;mm$

$2.2 * 10 \leq P_1 \leq min(12 * 9\,,150)$

$22\;mm \leq P_1 \leq \min\,(108\;mm\,,150\;mm)$

For a design purpose we choose $P_1 = 50\;mm$

$2.2 * 10 \leq P_2 \leq \min\,(14 * 9\,,120\;mm)$

$22\;mm \leq P_2 \leq \min\,(126\;mm\,,120mm)$

For a design purpose we choose $P_2 = 70\;mm$

- Resistance of the connection to bending moment

We must fulfill the following condition

$$M_{sd} \leq M_{rd} = \frac{N_1 * \sum d_i^2}{d_1} \Longrightarrow N_1 = \frac{M_{sd} * d_1}{\sum d_i^2}$$

N_1 : The internal effort of the most loaded bolt

$d_1 = 536.3\;mm$	$d_2 = 486.3\;mm$	$d_3 = 436.3\;mm$
$d_4 = 386.3\;mm$	$d_5 = 243.3\;mm$	$d_6 = 160.3\;mm$
	$d_7 = 77.3\;mm$	

$$N_1 = \frac{63.2 * 10^3 * 536.3}{536.3^2 + 486.3^2 + 436.3^2 + 386.3^2 + 243.3^2 + 160.3^2 + 77.3^2}$$

$N_1 = 34.75\ KN$

n : number of bolts par row

$N_1 \leq n * F_p = n * (0.7 * f_{ub} * A_s)$

$N_1 = 34.75\ KN \leq n * F_p = 7 * (0.7 * 800 * 10^{-3} * 36.6) = 330.46\ KN\ verified$

- Verification of resistance of the assembly to shear force by bolt

We must fulfill the following condition

$V_1 < F_s$

$$V_1 = \frac{V_{sd}}{2 * n} = \frac{37.52}{2 * 7} = 2.68\ KN$$

$V_1 = 2.68\ KN < F_s = 11.33\ KN\ verified$

The assembly resists perfectly, so we adopt 6 class bolts

- Resistance of the column web in the tension zone "unstiffened web"

We must fulfill the following condition

$$F_t = f_y * t_{\omega c} * \frac{b_{eff}}{\gamma_{m0}} \geq F_v = \frac{M_{sd}}{h - t_f}$$

$b_{eff} = t_{fB} + 2 * t_p + 5 * \left(t_{fC} + r_C\right)$

$b_{eff} = 12.7 + (2 * 9) + 5 * (9.5 + 15) = 153.2\ mm$

t_{fB} : the flange thickness of the rafter

t_p : the plate thicknesses

t_{fC} : the flange thickness of the post (column)

r_C : the radios between the flange and the web of the post

$$F_t = 235 * 10^{-3} * 6 * \frac{153.2}{1.1} = 196.37\ KN$$

$$F_v = \frac{61.85 * 10^3}{300 - 12.7} = 215.28\ KN$$

$F_t = 196.37\ KN < F_v = 215.28\ KN\ Not\ verified$

We most add stiffeners to increase the column web resistance.

- Resistance of the column web in the compression zone "unstiffened web"

$$F_c = f_y * t_{\omega C} * \left(\frac{1.25 + 0.5 * \gamma_{m0} * \sigma_n}{f_y}\right) * \frac{b_{eff}}{\gamma_{m0}}$$

σ_n : normal compressive stress in the column web due to compressive force and bending moment.

$$\sigma_n = \frac{V_{sd}}{A} + \frac{M_{sd}}{W_{ply}} = \frac{36.74}{45.3} + \frac{63.2 * 10^2}{324.9} = 19.86\ KN/cm^2$$

$$F_c = 235 * 10^{-1} * 0.6 * \left(1.25 + \left(\frac{0.5 * 1.1 * 19.86}{235 * 10^{-1}}\right)\right) * \frac{153.2 * 10^{-1}}{1.1}$$

$F_c = 336.77\ KN > F_v = 215.28\ KN\ verified$

the compression zone of the column web is stable but we'll add stiffeners for a security major

- Web resistance in the sheared zone

$$V_r = \frac{0.58 * f_y * h_C * t_{wC}}{\gamma_{m0}} = \frac{0.58 * 235 * 10^{-3} * 152 * 6}{1.1} = 113\ KN$$

$V_r = 113\ KN < F_v = 274.54\ KN\ Not\ verified$

We most add the diagonal stiffeners.

V.1.2. Assembly stability

V.1.2.1. Positioning the bolt holes

$N = 35.28\ KN$ from ROBOT calculation software

We take type 8.8 bolts

$t = max(7{,}7{,}10) \rightarrow t = 10\ mm$ we take a bolt with a diameter of Ø10

d_0 : bolt hole diameter $d_0 = d + 2\ mm = 12\ mm$

$e_1 \geq 1.2d_0 \rightarrow e_1 \geq 14.4\ mm \rightarrow e_1 = 50\ mm$

$p_1 \geq 2.2d_0 \rightarrow p_1 \geq 26.4\ mm \rightarrow p_1 = 60\ mm$

V.1.2.2. Calculation of the number of bolts

Shear strength of a bolt

f_{ub} : Bolt tensile strength value $\rightarrow f_{ub} = 800\ N/mm^2$

γ_{m1} : Partial safety factor $\rightarrow \gamma_{m1} = 1.25$

$So,$

$$F_{v,RD} = \frac{0.9 * f_{ub} * A_s}{\gamma_{m1}} = \frac{0.9 * 800 * 10^{-3} * 58}{1.25} = 33.41\ KN$$

$$n = \frac{N}{F_{v,RD}} = \frac{35.28}{33.41} = 1.06$$

So, we take 2 bolts

V.1.2.3. Diametric pressure check

We must fulfill the following condition

$$F_{b,Rd} = \frac{f_{ub} * 2.5 * d * t * a}{\gamma_{m1}} \geq F_{v,Rd} = 33.41\ KN$$

$$a = \min\left(\frac{e_1}{3 * d_0}\ ;\ \frac{(3 * p_1/d_0) - 1}{4}\ ;\ \frac{f_{ub}}{f_u}\ ; 1\right) = \min(1.38\ ;\ 3.5\ ; 2.2\ ; 1)$$

$$a = 1$$

$$F_{b,Rd} = \frac{800 * 10^{-3} * 2.5 * 10 * 10 * 1}{1.25} = 160\ KN$$

$$F_{b,Rd} = 160\ KN \geq F_{v,Rd} = 33.41\ KN\ Verified$$

V.1.3. Bracing assembly

V.1.3.1. Positioning the bolt holes

$N = 15.95\ KN$ from ROBOT calculation software

We take type 8.8 bolts

$t = max(7,7,10) \rightarrow t = 10\ mm$ we take a bolt with a diameter of Ø10

d_0 : bolt hole diameter $d_0 = d + 2\ mm = 12\ mm$

$e_1 \geq 1.2d_0 \rightarrow e_1 \geq 14.4\ mm\ \rightarrow e_1 = 50\ mm$

$p_1 \geq 2.2d_0 \rightarrow p_1 \geq 26.4\ mm\ \rightarrow p_1 = 60\ mm$

V.1.3.2. Calculation of the number of bolts

Shear strength of a bolt

f_{ub} : Bolt tensile strength value $\rightarrow\ f_{ub} = 800\ N/mm^2$

γ_{m1} : Partial safety factor $\rightarrow\ \gamma_{m1} = 1.25$

$So,$

$$F_{v,RD} = \frac{0.9 * f_{ub} * A_s}{\gamma_{m1}} = \frac{0.9 * 800 * 10^{-3} * 58}{1.25} = 33.41\ KN$$

$$n = \frac{N}{F_{v,RD}} = \frac{15.95}{33.41} = 0.5$$

So, for design purpose we take 2 bolts

V.1.3.3. Diametric pressure check

We must fulfill the following condition

$$F_{b,Rd} = \frac{f_{ub} * 2.5 * d * t * a}{\gamma_{m1}} \geq F_{v,Rd} = 33.41\ KN$$

$$a = \min\left(\frac{e_1}{3 * d_0}\ ;\ \frac{(3 * p_1/d_0) - 1}{4}\ ;\ \frac{f_{ub}}{f_u}\ ; 1\right) = \min(1.38\ ;\ 3.5\ ; 2.2\ ; 1)$$

$$a = 1$$

$$F_{b,Rd} = \frac{800 * 10^{-3} * 2.5 * 10 * 10 * 1}{1.25} = 160\ KN$$

$$F_{b,Rd} = 160\ KN \geq F_{v,Rd} = 33.41\ KN\ Verified$$

CHAPTER SIX

FOUNDATION CALCULATION

VI.1. The Column bases

VI.1.1. Efforts soliciting

$N_{sd} = 53.79\ KN$

$M_{sd} = 47.13\ KN * m$

$V_{sd} = 24.02\ KN$

VI.1.2. Dimensioning of the base plate

a. Weld beads

- Flange HEA 180

$a_s = 0.7 * t_f = 0.7 * 9.5 = 6.65 \implies$ We take $a_s = 8\ mm$

- Web HEA 180

$a_s = 0.7 * t_w = 0.7 * 6 = 4.2 \implies$ We take $a_s = 8\ mm$

b. Plate surface

$$\sigma = \frac{N}{h_p * b_p} \leq f_{ub} = 14.2\ MPa$$

$$we\ take\ h_p = b_p\ and\ we\ get\ \rightarrow h_p > \sqrt{\frac{N}{f_{ub}}} = \sqrt{\frac{55.55 * 10^3}{14.2}} = 62.55\ mm$$

So, for design purpose we take $h_p = 360\ mm$ and $b_p = 330\ mm$

c. Plate thickness

$$\sigma = \frac{N}{h_p * b_p} \leq \frac{f_y}{3} * \left(\frac{t}{u}\right)^2 \Rightarrow t = \sqrt{\frac{3 * \sigma * u^2}{f_y}}$$

$$\sigma = \frac{55.55 * 10^3}{360 * 330} = 0.47\ MPa \quad ; \quad u = 90\ mm$$

$$t = \sqrt{\frac{3 * 0.47 * 90^2}{235}} = 6.95\ mm$$

For design purpose we take $t = 25\ mm$

d. Verification of the compressive stress on the foundation

$\sigma = \frac{N}{h_p * b_p} = \frac{55.55*10^3}{360*330} = 0.47\ MPa \leq f_{ub} = 14.2\ MPa$ Verified

VI.1.3. Anchor Rod Dimension

The anchor rods are dimensioned for simple traction, under a tensile force N_t

$$\frac{N_t}{2} \leq \frac{\pi * \emptyset^2}{4} * f_y$$

With $N_t = 55.55\ KN$

$$\emptyset \geq \sqrt{\frac{2 * N_t}{\pi * f_y}} \rightarrow \emptyset \geq \sqrt{\frac{2 * 55.55}{3.14 * 235 * 10^{-3}}} = 12.22\ mm$$

So, We choose $\emptyset = 18\ mm$

Verification of the anchor rod

$$N_a = 0.1 * \left(1 + \frac{7 * g_c}{1000}\right) * \frac{\theta}{\left(1 + \frac{\theta}{d_1}\right)^2} * (l_1 + 3.5 * l_2 + 6.4 * r) \geq \frac{N}{6}$$

$g_c = 3.5\ KN/m^3$	$r = \emptyset * 3 = 54\ mm$
$l_1 = \emptyset * 20 = 360\ mm$	$l_2 = \emptyset * 2 = 36\ mm$

d_1 : the smallest distance from the axis of the rod to the exterior wall of the concrete block

$d_1 = 130\ mm$

$$N_a = 0.1 * \left(1 + \frac{7 * 350}{1000}\right) * 10^{-2} * \frac{18}{\left(1 + \frac{18}{130}\right)^2} * (360 + 3.5 * 36 + 6.4 * 54)$$

$N_a = 39.84\ KN$

$$\frac{N}{6} = \frac{55.55}{6} = 9.26\ KN$$

$N_a = 39.84\ KN \geq 9.18\ KN$ Verified

$So,$ the anchor rod $\emptyset = 18\ mm$ is secure

VI.1.4. Stress verification in steel

We must fulfill the following conditions

$$\sigma_a = \frac{N}{A_r} * \frac{L - h + \frac{h'}{3}}{h - \frac{h'}{3}} \leq f_y \quad ; \quad \sigma_b = \frac{2 * N * L}{b * h' * \left(h - \frac{h'}{3}\right)} \leq f_{ub}$$

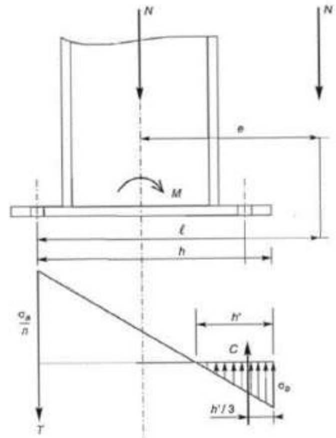


Figure VI.1.4:I presentation of column plate

$e = \frac{M}{N} = \frac{47.62}{55.55} = 0.86\ m$	$L = e + \frac{h_p}{2} - u = 997.25\ mm$
$h = h_p - u = 320\ mm$	$A_r = n * A_s = 4 * 192 = 768\ mm^2$

h' : the length of the compression tringle σ_b

$$h'^3 + 3(L - h)h'^2 + 90 * A_r * \frac{L}{b_p} * h' - 90 * A_r * L = 0$$

After solving the quadratic equation, we find $h' = 136.5\ mm$

$$\sigma_a = \frac{55.55}{768} * \frac{997.25 - 320 + \frac{136.5}{3}}{320 - \frac{136.5}{3}} = 0.19 \frac{KN}{mm^2} \leq f_y = 0.235 \frac{KN}{mm^2}$$

$\sigma_a = 0.19\ KN/mm^2 \ \leq f_y = 0.235\ KN/mm^2$ Verified

$$\sigma_b = \frac{2 * 55.55 * 997.25}{330 * 136.5 * \left(320 - \frac{136.5}{3}\right)} = 8.96\ Mpa \ \leq f_{ub} = 14.2\ Mpa$$

$\sigma_b = 8.96\ Mpa \ \leq f_{ub} = 14.2\ MPa$ Verified

VI.1.5. Checking the studs

We must fulfill the following condition

$N' \leq N_a$

From previous calculation $N_a = 39.84\ KN$

$$N' = \frac{V}{2} = \frac{22.24}{2} = 11.12\ KN$$

$N' = 11.12\ KN \leq N_a = 39.84\ KN$ Verified

VI.1.6. BAEL equilibrium condition

$$\frac{N}{4} \leq F_A = \pi * T_{SU} * \emptyset * L_1$$

With :

$L_1 = 20 * \emptyset = 360\ mm$	$T_{SU} = 0.6 * \varphi S^2 * f_{tj} = 0.6 * 1 * 2.1 = 1.26$
$\varphi S = 1$	$f_{tj} = 0.06 * F_{c28} + 0.6 = 2.1\ MPa$

$F_a = 3.14 * 1.26 * 18 * 360 = 25.64\ KN$

$$\frac{N}{4} = \frac{55.5}{4} = 13.89\ N$$

$\frac{N}{4} = 13.89\ KN \leq F_a = 25.64\ KN$ Not Verified

VI.2. Calculation of foundations

VI.2.1. Parameters to consider

The allowable stress of our site soil $\sigma_{soil} = 2.2\ bar$

The anchorage depth $D = 2.0\ m$

VI.2.2. Charge to consider

Table 15 The loads on foundation

	Effort	ULS	SLS
Sole	$N_{sd}(KN)$	54.8	38.84
	$M_{sd}(KN*m)$	51.51	35.55
$M_{sd}(KN)$	$\sigma_{soil} = 2.2\ bar = 0.22\ MPa$		

VI.2.3. Dimensioning of the Column pad footing

h & b : the dimensions of the post plate

H & B : the dimensions of pad footing

$h_t = d + c\,; with\ c = 10\ cm$ (Concrete of cleanliness)

a. Determination of A and B

We take $a = 0.66\ m$; $b = 0.49\ m$

$$\sigma_{soil} = \frac{N_s}{A*B} \rightarrow A*B \geq \frac{N_s}{\sigma_{soil}} \rightarrow A*B \geq \frac{55.1}{0.22*10^{-3}} = 250454.55\ mm^2$$

$$A*B \geq \frac{55.1}{0.22*10^{-3}} = 0.25\ m^2$$

So, we choose a pad footing with $A = 2.2\ m$ & $B = 1.4\ m$

b. Determination

$h_t = d + 0.1\ m$

$$\frac{B-b}{4} \leq d \leq A - a \implies \frac{1.5-0.49}{4} \leq d \leq 2.2 - 0.66$$

$0.25\ m \leq d \leq 1.54\ m$

We take $d = 1.1\ m$

$h_t = 1.1 + 0.1 = 1.2\ m$

c. Overturning stability check

$$e_0 = \frac{M_s}{N_s} \leq \frac{B}{4}$$

$$e_0 = \frac{35.55}{38.84} = 0.92\ m > \frac{1.4}{4} = 0.35\ m \quad \text{not verified}$$

Triangle diagram

We must check: $\sigma_2 = \frac{2*N}{3*\left(\frac{B}{2}-e_0\right)*B} \leq \sigma_{soil}$

$$\sigma_2 = \frac{2*38.84}{3*\left(\frac{1.4}{2}-0.92\right)*1.4} \leq \sigma_{soil}$$

$\sigma_2 = 85.91\ KN/m^2 \leq \sigma_{soil} = 220\ KN/m^2$ Verified

VI.2.4. Reinforcement calculation

The calculation is done at the ELU and the verification at the ELS: For (A’), we will use the “console” method.

$$\sigma_d = \frac{B + 0.35*b - 3*e_0}{3*(0.5*B - e_0)} * \sigma_2$$

$$\sigma_d = \frac{1.4 + 0.35*0.49 - 3*0.92}{3*(0.5*1.4 - 0.92)} * 85.91 = 156.2\ KN/m^2$$

$$M_d = B * \left(\frac{B}{2}*0.35*0.4\right)^2 * \left(\frac{\sigma_d + 2*\sigma_2}{6}\right)$$

$$M_d = 1.4 * \left(\frac{1.4}{2}*0.35*0.4\right)^2 * \left(\frac{156.2 + 2*85.91}{6}\right) = 0.74\ KN*m$$

$A' = \frac{M_d}{Z*\sigma_{bc}} = \frac{0.74}{0.9*1.1*25*10^3} * 10^{-4} = 0.297\ cm^2$ So, for design purpose we choose

$A' = 9T10 = 7.07\ cm^2$ with spacing $e = 16\ cm$

VI.2.5. Reinforcement verification

$$e_0 = \frac{M_s}{N_s} \leq \frac{B}{4}$$

$$e_0 = \frac{47.62}{55.55} = 0.86\ m > \frac{1.4}{4} = 0.35\ m \quad \text{not verified}$$

Triangle diagram

$$\sigma_2 = \frac{2*55.55}{3*\left(\frac{1.4}{2}-0.86\right)*1.4} = 168.22\ KN/m^2 \le \sigma_{soil} = 220\ KN/m^2 \text{ Verified}$$

$$\sigma_d = \frac{B + 0.35 * b - 3 * e_0}{3 * (0.5 * B - e_0)} * \sigma_2$$

$$\sigma_d = \frac{1.4 + 0.35 * 0.49 - 3 * 0.86}{3 * (0.5 * 1.4 - 0.86)} * 168.22 = 505.47\ KN/m^2$$

$$M_d = B * \left(\frac{B}{2} * 0.35 * 0.4\right)^2 * \left(\frac{\sigma_d + 2 * \sigma_2}{6}\right)$$

$$M_d = 1.4 * \left(\frac{1.4}{2} * 0.35 * 0.4\right)^2 * \left(\frac{505.47 + 2 * 168.22}{6}\right) = 1.89\ KN * m$$

$$A' = \frac{M_d}{Z * \sigma_{bc}} = \frac{1.89}{0.9 * 1.1 * 25 * 10^3} * 10^4 = 0.76\ cm^2 < 7.07\ cm^2\ Verified$$

$e \geq 6\emptyset + 6 = 12\ cm$ So, we take $e = 16\ cm$

: spacing between steel bars

GENERAL CONCLUSION

This modest work gave me an opportunity to apply and deepen all my knowledge acquired during the course of my master's degree in civil engineering.

The design of a metal structure is based on the dimensioning at the ultimate limit states, taking into account the harshest surrounding actions such as operating overloads, snow, wind and earthquake.

This experience also allowed me to better understand the field of structural steel construction which allowed me 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 works in this field, and developed the ideas thanks to the reading of the various bibliographical references and especially with the help of the team of professors of the department of civil engineering (Faculty of science and technology of the center of Ouargla university).

At the end of this project which constitutes for me the first experience in this vast field, it acquired us very important magnitudes to take the first step in my future professional life.

Bibliographic references

I. MINISTRY OF HOUSING AND URBAN PLANNING (Algeria), Rules for the design and calculation of steel structures “CCM97”.

II. MINISTRY OF HOUSING AND URBAN PLANNING, (Algeria), Snow and Wind Regulation "R.N.V 99 version 2013" (D.T.R.C 2-4.7)

III. MINISTRY OF HOUSING AND URBAN PLANNING, (Algeria), Algerian seismic rules "RPA 99 version 2003"

IV. EUROCODE 3 “Design of steel structures, national application document part 1-1”.

V. MINISTRY OF HOUSING AND URBAN PLANNING, Ed: CSTB 2000, (Algeria), Technical rules for the design and calculation of reinforced concrete structures and constructions according to the "BAEL 91 revised 99" limit state method

VI. UNIVERSITY PUBLICATIONS OFFICE (Algeria), practical calculation of metal structures

Appendix 1

Column-Rafter Connection

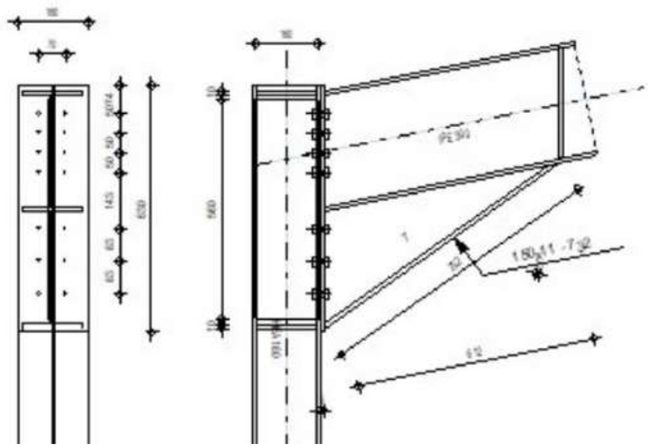
Autodesk Robot Structural Analysis Professional 2020

Design of fixed beam-to-column connection

NF EN 1993-1-8:2005/NA:2007/AC:2009

OK

Ratio
0.59



General

Connection no.: `7`

Connection name: `Frame knee`

Structure node: `38`

Structure bars: `39, 42`

Results

Beam resistances

COMPRESSION

A_b = 53.81 [cm^2] Area EN1993-1-1:[6.2.4]

$N_{cb,Rd} = A_b \, f_{yb} / \gamma_{M0}$

$N_{cb,Rd}$ = 1264.54 [kN] Design compressive resistance of the section EN1993-1-1:[6.2.4]

SHEAR

A_{vb} = 46.98 [cm^2] Shear area EN1993-1-1:[6.2.6.(3)]

$V_{cb,Rd} = A_{vb} \, (f_{yb} / \sqrt{3}) / \gamma_{M0}$

$V_{cb,Rd}$ =	637.41	[kN]	Design sectional resistance for shear	EN1993-1-1:[6.2.6.(2)]
$V_{b1,Ed} / V_{cb,Rd} \le 1,0$	0.07 < 1.00		verified	(0.07)

Column resistances

WEB PANEL - SHEAR

$M_{b1,Ed}$ =	58.22	[kN*m]	Bending moment (right beam)	[5.3.(3)]
$M_{b2,Ed}$ =	0.00	[kN*m]	Bending moment (left beam)	[5.3.(3)]
$V_{c1,Ed}$ =	-22.18	[kN]	Shear force (lower column)	[5.3.(3)]
$V_{c2,Ed}$ =	0.00	[kN]	Shear force (upper column)	[5.3.(3)]
z =	515	[mm]	Lever arm	[6.2.5]

$V_{wp,Ed} = (M_{b1,Ed} - M_{b2,Ed}) / z - (V_{c1,Ed} - V_{c2,Ed}) / 2$

$V_{wp,Ed}$ =	124.06	[kN]	Shear force acting on the web panel	[5.3.(3)]
A_{vs} =	14.47	[cm^2]	Shear area of the column web	EN1993-1-1:[6.2.6.(3)]
A_{vp} =	9.60	[cm^2]	Area of the web stiffening plate	EN1993-1-1:[6.2.6.(3)]
A_{vc} =	24.07	[cm^2]	Shear area	EN1993-1-1:[6.2.6.(3)]
d_s =	590	[mm]	Distance between the centroids of stiffeners	[6.2.6.1.(4)]
$M_{pl,fc,Rd}$ =	0.95	[kN*m]	Plastic resistance of the column flange for bending	[6.2.6.1.(4)]
$M_{pl,stu,Rd}$ =	1.06	[kN*m]	Plastic resistance of the upper transverse stiffener for bending	[6.2.6.1.(4)]
$M_{pl,stl,Rd}$ =	1.06	[kN*m]	Plastic resistance of the lower transverse stiffener for bending	[6.2.6.1.(4)]

$V_{wp,Rd} = 0.9 (A_{vs}*f_{y,wc}+A_{vp}*f_{ya}) / (\sqrt{3}\ \gamma_{M0}) + \text{Min}(4 M_{pl,fc,Rd} / d_s , (2 M_{pl,fc,Rd} + M_{pl,stu,Rd} + M_{pl,stl,Rd}) / d_s)$

$V_{wp,Rd}$ =	300.39	[kN]	Resistance of the column web panel for shear	[6.2.6.1]
$V_{wp,Ed} / V_{wp,Rd} \le 1,0$	0.41 < 1.00		verified	(0.41)

WEB - TRANSVERSE COMPRESSION - LEVEL OF THE BEAM BOTTOM FLANGE

Bearing:

t_{wc} =	9	[mm]	Effective thickness of the column web	[6.2.6.2.(6)]
$b_{eff,c,wc}$ =	168	[mm]	Effective width of the web for compression	[6.2.6.2.(1)]
A_{vc} =	24.07	[cm^2]	Shear area	EN1993-1-1:[6.2.6.(3)]
ω =	0.81		Reduction factor for interaction with shear	[6.2.6.2.(1)]
$\sigma_{com,Ed}$ =	151.02	[MPa]	Maximum compressive stress in web	[6.2.6.2.(2)]
k_{wc} =	1.00		Reduction factor conditioned by compressive stresses	[6.2.6.2.(2)]

t_{wc} =	9	[mm]	Effective thickness of the column web	[6.2.6.2.(6)]
A_s =	17.40	[cm^2]	Area of the web stiffener	EN1993-1-1:[6.2.4]

$F_{c,wc,Rd1} = \omega\ k_{wc}\ b_{eff,c,wc}\ t_{wc}\ f_{yc} / \gamma_{M0} + A_s\ f_{ys} / \gamma_{M0}$

$F_{c,wc,Rd1}$ =	697.43	[kN]	Column web resistance	[6.2.6.2.(1)]

Buckling:

d_{wc} =	122	[mm]	Height of compressed web	[6.2.6.2.(1)]
λ_p =	0.50		Plate slenderness of an element	[6.2.6.2.(1)]
ρ =	1.00		Reduction factor for element buckling	[6.2.6.2.(1)]
λ_s =	2.31		Stiffener slenderness	EN1993-1-1:[6.3.1.2]
χ_s =	1.00		Buckling coefficient of the stiffener	EN1993-1-1:[6.3.1.2]

$F_{c,wc,Rd2} = \omega\ k_{wc}\ \rho\ b_{eff,c,wc}\ t_{wc}\ f_{yc} / \gamma_{M1} + A_s\ \chi_s\ f_{ys} / \gamma_{M1}$

$F_{c,wc,Rd2}$ =	697.43	[kN]	Column web resistance	[6.2.6.2.(1)]

Final resistance:

$F_{c,wc,Rd,low} = \text{Min}\ (F_{c,wc,Rd1}\ ,\ F_{c,wc,Rd2})$

$F_{c,wc,Rd}$ =	697.43	[kN]	Column web resistance	[6.2.6.2.(1)]

WEB - TRANSVERSE COMPRESSION - LEVEL OF THE BEAM TOP FLANGE

Bearing:

t_{wc} =	9	[mm]	Effective thickness of the column web	[6.2.6.2.(6)]
$b_{eff,c,wc}$ =	166	[mm]	Effective width of the web for compression	[6.2.6.2.(1)]
A_{vc} =	24.07	[cm^2]	Shear area	EN1993-1-1:[6.2.6.(3)]
ω =	0.82		Reduction factor for interaction with shear	[6.2.6.2.(1)]
$\sigma_{com,Ed}$ =	151.02	[MPa]	Maximum compressive stress in web	[6.2.6.2.(2)]
k_{wc} =	1.00		Reduction factor conditioned by compressive stresses	[6.2.6.2.(2)]
A_s =	17.40	[cm^2]	Area of the web stiffener	EN1993-1-1:[6.2.4]

$F_{c,wc,Rd1} = \omega\ k_{wc}\ b_{eff,c,wc}\ t_{wc}\ f_{yc} / \gamma_{M0} + A_s\ f_{ys} / \gamma_{M0}$

$F_{c,wc,Rd1}$ =	694.97	[kN]	Column web resistance	[6.2.6.2.(1)]

Buckling:

d_{wc} =	122	[mm]	Height of compressed web	[6.2.6.2.(1)]
λ_p =	0.49		Plate slenderness of an element	[6.2.6.2.(1)]

d_{wc} = 122 [mm] Height of compressed web [6.2.6.2.(1)]

ρ = 1.00 Reduction factor for element buckling [6.2.6.2.(1)]

λ_s = 2.31 Stiffener slenderness EN1993-1-1:[6.3.1.2]

χ_s = 1.00 Buckling coefficient of the stiffener EN1993-1-1:[6.3.1.2]

$F_{c,wc,Rd2} = \omega\ k_{wc}\ \rho\ b_{eff,c,wc}\ t_{wc}\ f_{yc} / \gamma_{M1} + A_s\ \chi_s\ f_{ys} / \gamma_{M1}$

$F_{c,wc,Rd2}$ = 694.97 [kN] Column web resistance [6.2.6.2.(1)]

Final resistance:

$F_{c,wc,Rd,upp} = \mathrm{Min}\ (F_{c,wc,Rd1}\ ,\ F_{c,wc,Rd2})$

$F_{c,wc,Rd,upp}$ = 694.97 [kN] Column web resistance [6.2.6.2.(1)]

CONNECTION RESISTANCE FOR COMPRESSION

$N_{j,Rd} = \mathrm{Min}\ (\ N_{cb,Rd} 2\ F_{c,wb,Rd,low}\ ,\ 2\ F_{c,wc,Rd,low}\ ,\ 2\ F_{c,wc,Rd,upp}\ )$

$N_{j,Rd}$ = 942.94 [kN] Connection resistance for compression [6.2]

$N_{b1,Ed} / N_{j,Rd} \leq 1{,}0$	0.02 < 1.00	verified	(0.02)

CONNECTION RESISTANCE FOR BENDING

$F_{t,Rd}$ = 48.38 [kN] Bolt resistance for tension [Table 3.4]

$B_{p,Rd}$ = 89.17 [kN] Punching shear resistance of a bolt [Table 3.4]

$F_{t,fc,Rd}$ – column flange resistance due to bending

$F_{t,wc,Rd}$ – column web resistance due to tension

$F_{t,ep,Rd}$ – resistance of the front plate due to bending

$F_{t,wb,Rd}$ – resistance of the web in tension

$F_{t,fc,Rd} = \mathrm{Min}\ (F_{T,1,fc,Rd}\ ,\ F_{T,2,fc,Rd}\ ,\ F_{T,3,fc,Rd})$ [6.2.6.4] , [Tab.6.2]

$F_{t,wc,Rd} = \omega\ b_{eff,t,wc}\ t_{wc}\ f_{yc} / \gamma_{M0}$ [6.2.6.3.(1)]

$F_{t,ep,Rd} = \mathrm{Min}\ (F_{T,1,ep,Rd}\ ,\ F_{T,2,ep,Rd}\ ,\ F_{T,3,ep,Rd})$ [6.2.6.5] , [Tab.6.2]

$F_{t,wb,Rd} = b_{eff,t,wb}\ t_{wb}\ f_{yb} / \gamma_{M0}$ [6.2.6.8.(1)]

Additional reduction of the bolt row resistance

$F_{t4,Rd} = F_{t1,Rd}\ h_4/h_1$

$F_{t4,Rd}$ = 59.71 [kN] Reduced bolt row resistance [6.2.7.2.(9)]

$F_{t4,Rd} = F_{t2,Rd}\ h_4/h_2$

$F_{t4,Rd}$ =	31.26	[kN]	Reduced bolt row resistance	[6.2.7.2.(9)]FRA

Additional reduction of the bolt row resistance

$F_{t5,Rd} = F_{t1,Rd}\ h_5/h_1$

$F_{t5,Rd}$ =	37.84	[kN]	Reduced bolt row resistance	[6.2.7.2.(9)]

$F_{t5,Rd} = F_{t2,Rd}\ h_5/h_2$

$F_{t5,Rd}$ =	19.81	[kN]	Reduced bolt row resistance	[6.2.7.2.(9)]FRA

Additional reduction of the bolt row resistance

$F_{t6,Rd} = F_{t1,Rd}\ h_6/h_1$

$F_{t6,Rd}$ =	25.15	[kN]	Reduced bolt row resistance	[6.2.7.2.(9)]

$F_{t6,Rd} = F_{t2,Rd}\ h_6/h_2$

$F_{t6,Rd}$ =	13.17	[kN]	Reduced bolt row resistance	[6.2.7.2.(9)]FRA

Additional reduction of the bolt row resistance

$F_{t7,Rd} = F_{t1,Rd}\ h_7/h_1$

$F_{t7,Rd}$ =	12.45	[kN]	Reduced bolt row resistance	[6.2.7.2.(9)]

$F_{t7,Rd} = F_{t2,Rd}\ h_7/h_2$

$F_{t7,Rd}$ =	6.52	[kN]	Reduced bolt row resistance	[6.2.7.2.(9)]FRA

CONNECTION RESISTANCE FOR BENDING $M_{j,Rd}$

$M_{j,Rd} = \sum h_j\ F_{tj,Rd}$

$M_{j,Rd}$ =	98.93	[kN*m]	Connection resistance for bending	[6.2]

$M_{b1,Ed} / M_{j,Rd} \leq 1,0$	0.59 < 1.00	verified	(0.59)

CONNECTION RESISTANCE FOR SHEAR

α_v =	0.60		Coefficient for calculation of $F_{v,Rd}$	[Table 3.4]
β_{Lf} =	0.88		Reduction factor for long connections	[3.8]
$F_{v,Rd}$ =	38.38	[kN]	Shear resistance of a single bolt	[Table 3.4]
$F_{t,Rd,max}$ =	48.38	[kN]	Tensile resistance of a single bolt	[Table 3.4]
$F_{b,Rd,int}$ =	78.84	[kN]	Bearing resistance of an intermediate bolt	[Table 3.4]
$F_{b,Rd,ext}$ =	78.84	[kN]	Bearing resistance of an outermost bolt	[Table 3.4]

$F_{tj,Rd,N}$ – Bolt row resistance for simple tension

$F_{tj,Ed,N}$ – Force due to axial force in a bolt row

$F_{tj,Rd,N}$ – Bolt row resistance for simple tension

$F_{tj,Rd,M}$ – Bolt row resistance for simple bending

$F_{tj,Ed,M}$ – Force due to moment in a bolt row

$F_{tj,Ed}$ – Maximum tensile force in a bolt row

$F_{vj,Rd}$ – Reduced bolt row resistance

$F_{tj,Ed,N} = N_{j,Ed}\, F_{tj,Rd,N} / N_{j,Rd}$

$F_{tj,Ed,M} = M_{j,Ed}\, F_{tj,Rd,M} / M_{j,Rd}$

$F_{tj,Ed} = F_{tj,Ed,N} + F_{tj,Ed,M}$

$F_{vj,Rd} = \text{Min}\,(n_h\, F_{v,Ed}\,(1 - F_{tj,Ed} / (1.4\, n_h\, F_{t,Rd,max}),\, n_h\, F_{v,Rd}\,,\, n_h\, F_{b,Rd}))$

$V_{j,Rd} = n_h \sum_1^n F_{vj,Rd}$				[Table 3.4]
$V_{j,Rd} =$	474.12	[kN]	Connection resistance for shear	[Table 3.4]

$V_{b1,Ed} / V_{j,Rd} \leq 1,0$	0.09 < 1.00	verified	(0.09)

WELD RESISTANCE

$A_w =$	93.63	[cm^2]	Area of all welds	[4.5.3.2(2)]
$A_{wy} =$	39.58	[cm^2]	Area of horizontal welds	[4.5.3.2(2)]
$A_{wz} =$	54.05	[cm^2]	Area of vertical welds	[4.5.3.2(2)]
$I_{wy} =$	39366.74	[cm^4]	Moment of inertia of the weld arrangement with respect to the hor. axis	[4.5.3.2(5)]
$\sigma_{\perp max}=\tau_{\perp max} =$	-34.76	[MPa]	Normal stress in a weld	[4.5.3.2(6)]
$\sigma_{\perp}=\tau_{\perp} =$	-33.03	[MPa]	Stress in a vertical weld	[4.5.3.2(5)]
$\tau_{II} =$	7.83	[MPa]	Tangent stress	[4.5.3.2(5)]
$\beta_w =$	0.80		Correlation coefficient	[4.5.3.2(7)]

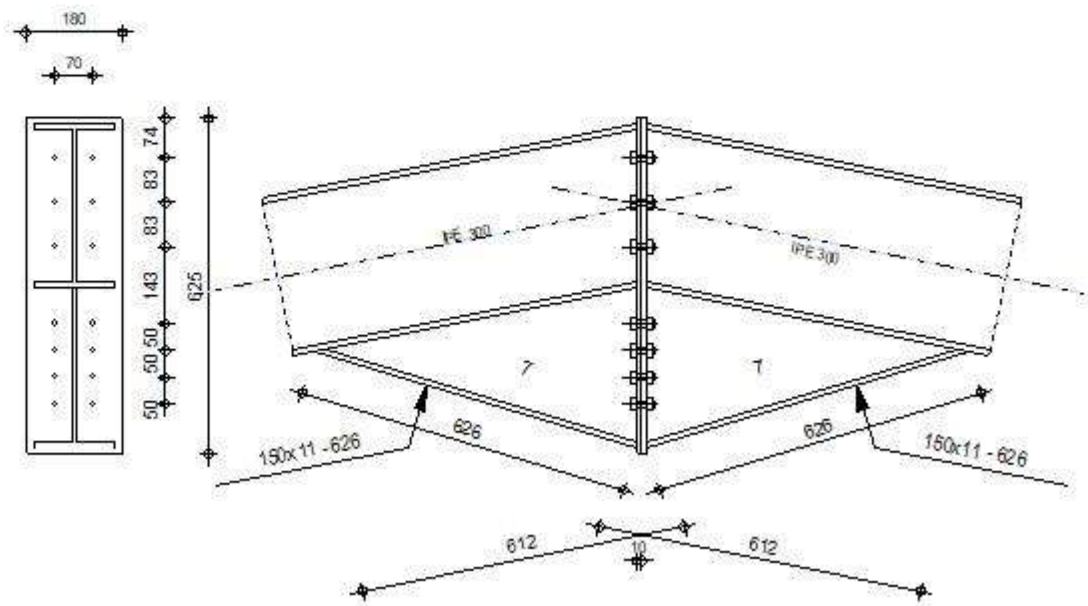
$\sqrt{[\sigma_{\perp max}^2 + 3*(\tau_{\perp max}^2)]} \leq f_u/(\beta_w*\gamma_{M2})$	69.52 < 365.00	verified	(0.19)
$\sqrt{[\sigma_{\perp}^2 + 3*(\tau_{\perp}^2+\tau_{II}^2)]} \leq f_u/(\beta_w*\gamma_{M2})$	67.44 < 365.00	verified	(0.18)
$\sigma_{\perp} \leq 0.9*f_u/\gamma_{M2}$	34.76 < 262.80	verified	(0.13)

Connection conforms to the code	Ratio	0.59

Appendix 2

Rafter-Rafter Connection

	Autodesk Robot Structural Analysis Professional 2020 **Design of fixed beam-to-beam connection** NF EN 1993-1-8:2005/NA:2007/AC:2009	OK
		Ratio **0.49**



General

Connection no.: 28

Connection name: Beam-Beam

Structure node: 41

Structure bars: 41, 42

Geometry

<u>LEFT SIDE</u>

<u>BEAM</u>

Section: IPE 300

Bar no.: 41

α =	-168.7	[Deg]	Inclination angle
h_{bl} =	300	[mm]	Height of beam section
b_{fbl} =	150	[mm]	Width of beam section
t_{wbl} =	7	[mm]	Thickness of the web of beam section
t_{fbl} =	11	[mm]	Thickness of the flange of beam section
r_{bl} =	15	[mm]	Radius of beam section fillet
A_{bl} =	53.81	[cm^2]	Cross-sectional area of a beam
I_{xbl} =	8356.11	[cm^4]	Moment of inertia of the beam section

Material: ACIER

f_{yb} =	235.00	[MPa]	Resistance

RIGHT SIDE

BEAM

Section: IPE 300

Bar no.: 42

α =	-11.3	[Deg]	Inclination angle
h_{br} =	300	[mm]	Height of beam section
b_{fbr} =	150	[mm]	Width of beam section
t_{wbr} =	7	[mm]	Thickness of the web of beam section
t_{fbr} =	11	[mm]	Thickness of the flange of beam section
r_{br} =	15	[mm]	Radius of beam section fillet
A_{br} =	53.81	[cm^2]	Cross-sectional area of a beam
I_{xbr} =	8356.11	[cm^4]	Moment of inertia of the beam section

Material: ACIER

f_{yb} =	235.00	[MPa]	Resistance

BOLTS

The shear plane passes through the UNTHREADED portion of the bolt.

d =	12	[mm]	Bolt diameter
Class =	8.8		Bolt class
F_{tRd} =	48.38	[kN]	Tensile resistance of a bolt

d =	12	[mm]	Bolt diameter
n_h =	2		Number of bolt columns
n_v =	7		Number of bolt rows
h_1 =	74	[mm]	Distance between first bolt and upper edge of front plate

Horizontal spacing e_i = 70 [mm]

Vertical spacing p_i = 83;83;143;50;50;50 [mm]

PLATE

h_{pr} =	625	[mm]	Plate height
b_{pr} =	180	[mm]	Plate width
t_{pr} =	10	[mm]	Plate thickness

Material: ACIER

f_{ypr} =	235.00	[MPa]	Resistance

LOWER STIFFENER

w_{rd} =	150	[mm]	Plate width
t_{frd} =	11	[mm]	Flange thickness
h_{rd} =	300	[mm]	Plate height
t_{wrd} =	7	[mm]	Web thickness
l_{rd} =	612	[mm]	Plate length
α_d =	16.7	[Deg]	Inclination angle

Material: ACIER

f_{ybu} =	235.00	[MPa]	Resistance

FILLET WELDS

a_w =	5	[mm]	Web weld
a_f =	8	[mm]	Flange weld
a_{fd} =	5	[mm]	Horizontal weld

MATERIAL FACTORS

γ_{M0} =	1.00	Partial safety factor	[2.2]
γ_{M1} =	1.00	Partial safety factor	[2.2]

γ_{M0} =	1.00	Partial safety factor	[2.2]
γ_{M2} =	1.25	Partial safety factor	[2.2]
γ_{M3} =	1.10	Partial safety factor	[2.2]

Loads

Ultimate limit state

Case: 16: ULS /43/ 1*1.35 + 2*1.35 + 3*1.35 + 4*1.35 + 5*1.35 + 6*1.35 + 7*1.50 + 9*1.50 + 15*0.90

$M_{b1,Ed}$ =	-53.59	[kN*m]	Bending moment in the right beam
$V_{b1,Ed}$ =	-2.17	[kN]	Shear force in the right beam
$N_{b1,Ed}$ =	-21.22	[kN]	Axial force in the right beam

Results

BEAM RESISTANCES

COMPRESSION

A_b =	53.81	[cm^2]	Area	EN1993-1-1:[6.2.4]

$N_{cb,Rd} = A_b f_{yb} / \gamma_{M0}$

$N_{cb,Rd}$ =	1264.54	[kN]	Design compressive resistance of the section	EN1993-1-1:[6.2.4]

SHEAR

A_{vb} =	46.98	[cm^2]	Shear area	EN1993-1-1:[6.2.6.(3)]

$V_{cb,Rd} = A_{vb} (f_{yb} / \sqrt{3}) / \gamma_{M0}$

$V_{cb,Rd}$ =	637.41	[kN]	Design sectional resistance for shear	EN1993-1-1:[6.2.6.(2)]

$V_{b1,Ed} / V_{cb,Rd} \leq 1,0$	0.00 < 1.00	verified	(0.00)

BENDING - PLASTIC MOMENT (WITHOUT BRACKETS)

W_{plb} =	628.36	[cm^3]	Plastic section modulus	EN1993-1-1:[6.2.5.(2)]

$M_{b,pl,Rd} = W_{plb} f_{yb} / \gamma_{M0}$

$M_{b,pl,Rd}$ =	147.66	[kN*m]	Plastic resistance of the section for bending (without stiffeners)	EN1993-1-1:[6.2.5.(2)]

BENDING ON THE CONTACT SURFACE WITH PLATE OR CONNECTED ELEMENT

W_{pl} =	1372.44	[cm^3]	Plastic section modulus	EN1993-1-1:[6.2.5]

$M_{cb,Rd} = W_{pl} f_{yb} / \gamma_{M0}$

$M_{cb,Rd}$ =	322.52	[kN*m]	Design resistance of the section for bending	EN1993-1-1:[6.2.5]

FLANGE AND WEB - COMPRESSION

$M_{cb,Rd}$ =	322.52	[kN*m]	Design resistance of the section for bending	EN1993-1-1:[6.2.5]
h_f =	595	[mm]	Distance between the centroids of flanges	[6.2.6.7.(1)]

$F_{c,fb,Rd} = M_{cb,Rd} / h_f$

$F_{c,fb,Rd}$ =	542.15	[kN]	Resistance of the compressed flange and web	[6.2.6.7.(1)]

WEB OR BRACKET FLANGE - COMPRESSION - LEVEL OF THE BEAM BOTTOM FLANGE

Bearing:

β =	11.3	[Deg]	Angle between the front plate and the beam	
γ =	16.7	[Deg]	Inclination angle of the bracket plate	
$b_{eff,c,wb}$ =	174	[mm]	Effective width of the web for compression	[6.2.6.2.(1)]
A_{vb} =	25.68	[cm^2]	Shear area	EN1993-1-1:[6.2.6.(3)]
ω =	0.88		Reduction factor for interaction with shear	[6.2.6.2.(1)]
$\sigma_{com,Ed}$ =	0.00	[MPa]	Maximum compressive stress in web	[6.2.6.2.(2)]
k_{wc} =	1.00		Reduction factor conditioned by compressive stresses	[6.2.6.2.(2)]

$F_{c,wb,Rd1} = [\omega\ k_{wc}\ b_{eff,c,wb}\ t_{wb}\ f_{yb} / \gamma_{M0}] \cos(\gamma) / \sin(\gamma - \beta)$

$F_{c,wb,Rd1}$ =	518.96	[kN]	Beam web resistance	[6.2.6.2.(1)]

Buckling:

d_{wb} =	249	[mm]	Height of compressed web	[6.2.6.2.(1)]
λ_p =	0.91		Plate slenderness of an element	[6.2.6.2.(1)]
ρ =	0.86		Reduction factor for element buckling	[6.2.6.2.(1)]

$F_{c,wb,Rd2} = [\omega\ k_{wc}\ \rho\ b_{eff,c,wb}\ t_{wb}\ f_{yb} / \gamma_{M1}] \cos(\gamma) / \sin(\gamma - \beta)$

$F_{c,wb,Rd2}$ =	443.88	[kN]	Beam web resistance	[6.2.6.2.(1)]

Resistance of the bracket flange

$F_{c,wb,Rd3} = b_b\ t_b\ f_{yb} / (0.8 * \gamma_{M0})$

$F_{c,wb,Rd3}$ =	471.47	[kN]	Resistance of the bracket flange	[6.2.6.7.(1)]

Final resistance:

$F_{c,wb,Rd,low} = \mathrm{Min}\ (F_{c,wb,Rd1}\ ,\ F_{c,wb,Rd2}\ ,\ F_{c,wb,Rd3})$

$F_{c,wb,Rd,low}$ =	443.88	[kN]	Beam web resistance	[6.2.6.2.(1)]

GEOMETRICAL PARAMETERS OF A CONNECTION

EFFECTIVE LENGTHS AND PARAMETERS - FRONT PLATE

Nr	m	m_x	e	e_x	p	$l_{eff,cp}$	$l_{eff,nc}$	$l_{eff,1}$	$l_{eff,2}$	$l_{eff,cp,g}$	$l_{eff,nc,g}$	$l_{eff,1,g}$	$l_{eff,2,g}$
1	26	–	55	–	50	162	172	162	172	131	111	111	111
2	26	–	55	–	50	162	172	162	172	100	50	50	50
3	26	–	55	–	50	162	172	162	172	100	50	50	50
4	26	–	55	–	97	162	172	162	172	193	97	97	97
5	26	–	55	–	113	162	172	162	172	226	113	113	113
6	26	–	55	–	83	162	172	162	172	166	83	83	83
7	26	–	55	–	83	162	172	162	172	164	127	127	127

m – Bolt distance from the web

m_x – Bolt distance from the beam flange

e – Bolt distance from the outer edge

e_x – Bolt distance from the horizontal outer edge

p – Distance between bolts

$l_{eff,cp}$ – Effective length for a single bolt in the circular failure mode

$l_{eff,nc}$ – Effective length for a single bolt in the non-circular failure mode

$l_{eff,1}$ – Effective length for a single bolt for mode 1

$l_{eff,2}$ – Effective length for a single bolt for mode 2

$l_{eff,cp,g}$ – Effective length for a group of bolts in the circular failure mode

$l_{eff,nc,g}$ – Effective length for a group of bolts in the non-circular failure mode

$l_{eff,1,g}$ – Effective length for a group of bolts for mode 1

$l_{eff,2,g}$ – Effective length for a group of bolts for mode 2

CONNECTION RESISTANCE FOR COMPRESSION

$N_{j,Rd}$ = Min ($N_{cb,Rd}$2 $F_{c,wb,Rd,low}$)

$N_{j,Rd}$ = 887.76 [kN] Connection resistance for compression [6.2]

$N_{b1,Ed}$ / $N_{j,Rd} \leq 1,0$ 0.02 < 1.00 verified (0.02)

CONNECTION RESISTANCE FOR BENDING

CONNECTION RESISTANCE FOR BENDING $M_{j,Rd}$

$M_{j,Rd} = \sum h_j F_{tj,Rd}$

$M_{j,Rd}$ = 109.48 [kN*m] Connection resistance for bending [6.2]

$M_{b1,Ed} / M_{j,Rd} \leq 1,0$ 0.49 < 1.00 verified (0.49)

CONNECTION RESISTANCE FOR SHEAR

α_v = 0.60 Coefficient for calculation of $F_{v,Rd}$ [Table 3.4]

β_{Lf} = 0.88 Reduction factor for long connections [3.8]

$F_{v,Rd}$ = 38.38 [kN] Shear resistance of a single bolt [Table 3.4]

$F_{t,Rd,max}$ = 48.38 [kN] Tensile resistance of a single bolt [Table 3.4]

$F_{b,Rd,int}$ = 87.60 [kN] Bearing resistance of an intermediate bolt [Table 3.4]

$F_{b,Rd,ext}$ = 87.60 [kN] Bearing resistance of an outermost bolt [Table 3.4]

Nr	$F_{tj,Rd,N}$	$F_{tj,Ed,N}$	$F_{tj,Rd,M}$	$F_{tj,Ed,M}$	$F_{tj,Ed}$	$F_{vj,Rd}$
1	96.77	-3.03	88.55	43.34	40.31	53.92
2	96.77	-3.03	51.54	25.23	22.20	64.18
3	96.77	-3.03	39.57	19.37	16.34	67.50
4	96.77	-3.03	40.52	19.83	16.80	67.24
5	96.77	-3.03	24.75	12.12	9.08	71.61
6	96.77	-3.03	15.60	7.64	4.61	74.15
7	96.77	-3.03	6.45	3.16	0.13	76.69

$F_{tj,Rd,N}$ – Bolt row resistance for simple tension

$F_{tj,Ed,N}$ – Force due to axial force in a bolt row

$F_{tj,Rd,M}$ – Bolt row resistance for simple bending

$F_{tj,Ed,M}$ – Force due to moment in a bolt row

$F_{tj,Ed}$ – Maximum tensile force in a bolt row

$F_{vj,Rd}$ – Reduced bolt row resistance

$F_{tj,Ed,N} = N_{j,Ed} F_{tj,Rd,N} / N_{j,Rd}$

$F_{tj,Ed,M} = M_{j,Ed} F_{tj,Rd,M} / M_{j,Rd}$

$F_{tj,Ed} = F_{tj,Ed,N} + F_{tj,Ed,M}$

$F_{vj,Rd} = \mathrm{Min}\,(n_h F_{v,Ed} (1 - F_{tj,Ed}/ (1.4\ n_h F_{t,Rd,max}),\ n_h F_{v,Rd}\ ,\ n_h F_{b,Rd}))$

$V_{j,Rd} = n_h \sum_1^n F_{vj,Rd}$ [Table 3.4]

$V_{j,Rd}$ = 475.31 [kN] Connection resistance for shear [Table 3.4]

$V_{b1,Ed} / V_{j,Rd} \leq 1,0$	0.00 < 1.00	verified	(0.00)

WELD RESISTANCE

A_w =	117.56	[cm^2]	Area of all welds	[4.5.3.2(2)]
A_{wy} =	63.33	[cm^2]	Area of horizontal welds	[4.5.3.2(2)]
A_{wz} =	54.24	[cm^2]	Area of vertical welds	[4.5.3.2(2)]
I_{wy} =	54216.21	[cm^4]	Moment of inertia of the weld arrangement with respect to the hor. axis	[4.5.3.2(5)]
$\sigma_{\perp max}=\tau_{\perp max}$ =	-21.88	[MPa]	Normal stress in a weld	[4.5.3.2(6)]
$\sigma_{\perp}=\tau_{\perp}$ =	-19.90	[MPa]	Stress in a vertical weld	[4.5.3.2(5)]
τ_{II} =	-0.40	[MPa]	Tangent stress	[4.5.3.2(5)]
β_w =	0.80		Correlation coefficient	[4.5.3.2(7)]

$\sqrt{[\sigma_{\perp max}^2 + 3*(\tau_{\perp max}^2)]} \leq f_u/(\beta_w*\gamma_{M2})$	43.77 < 365.00	verified	(0.12)
$\sqrt{[\sigma_{\perp}^2 + 3*(\tau_{\perp}^2+\tau_{II}^2)]} \leq f_u/(\beta_w*\gamma_{M2})$	39.81 < 365.00	verified	(0.11)
$\sigma_{\perp} \leq 0.9*f_u/\gamma_{M2}$	21.88 < 262.80	verified	(0.08)

Connection conforms to the code	Ratio	0.49

Appendix 3

Column base

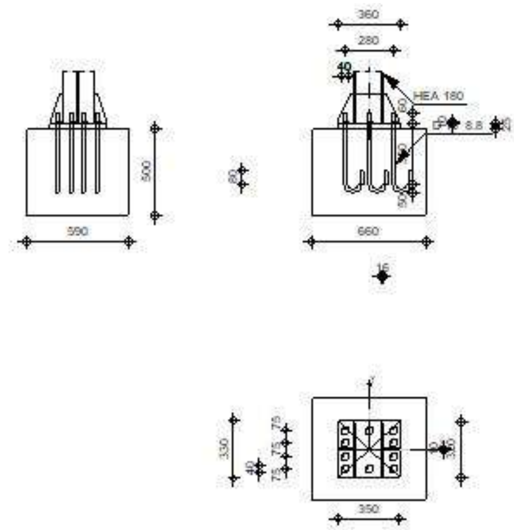
Autodesk Robot Structural Analysis Professional 2020

Fixed column base design

Eurocode 3: NF EN 1993-1-8:2005/NA:2007/AC:2009 + CEB Design Guide: Design of fastenings in concrete

OK

Ratio
0.83



GENERAL

Connection no.: 14

Connection name: Fixed column base

Structure node: 37

Structure bars: 39

GEOMETRY

COLUMN

Section: HEA 180

Bar no.: 39

L_c =	4.50	[m]	Column length
α =	0.0	[Deg]	Inclination angle
h_c =	171	[mm]	Height of column section

L_c = 4.50 [m] Column length

b_{fc} = 180 [mm] Width of column section

t_{wc} = 6 [mm] Thickness of the web of column section

t_{fc} = 10 [mm] Thickness of the flange of column section

r_c = 15 [mm] Radius of column section fillet

A_c = 45.25 [cm^2] Cross-sectional area of a column

I_{yc} = 2510.29 [cm^4] Moment of inertia of the column section

Material: ACIER

f_{yc} = 235.00 [MPa] Resistance

f_{uc} = 365.00 [MPa] Yield strength of a material

COLUMN BASE

l_{pd} = 360 [mm] Length

b_{pd} = 330 [mm] Width

t_{pd} = 25 [mm] Thickness

Material: ACIER E24

f_{ypd} = 235.00 [MPa] Resistance

f_{upd} = 365.00 [MPa] Yield strength of a material

ANCHORAGE

The shear plane passes through the UNTHREADED portion of the bolt.

Class = 8.8 Anchor class

f_{yb} = 550.00 [MPa] Yield strength of the anchor material

f_{ub} = 800.00 [MPa] Tensile strength of the anchor material

d = 18 [mm] Bolt diameter

A_s = 1.92 [cm^2] Effective section area of a bolt

A_v = 2.54 [cm^2] Area of bolt section

n_H = 3 Number of bolt columns

n_V = 4 Number of bolt rows

Horizontal spacing e_{Hi} = 140 [mm]

Vertical spacing e_{Vi} = 75;75 [mm]

Anchor dimensions

L_1 =	60	[mm]
L_2 =	350	[mm]
L_3 =	100	[mm]
L_4 =	80	[mm]

STIFFENER

l_s =	350	[mm]	Length
w_s =	320	[mm]	Width
h_s =	171	[mm]	Height
t_s =	10	[mm]	Thickness
d_1 =	20	[mm]	Cut
d_2 =	20	[mm]	Cut

LOADS

Case: 16: ULS /142/ 1*1.35 + 2*1.35 + 3*1.35 + 4*1.35 + 5*1.35 + 6*1.35 + 7*1.05 + 9*1.05 + 15*1.50

$N_{j,Ed}$ =	-54.82	[kN]	Axial force
$V_{j,Ed,y}$ =	0.00	[kN]	Shear force
$V_{j,Ed,z}$ =	-22.59	[kN]	Shear force
$M_{j,Ed,y}$ =	43.94	[kN*m]	Bending moment
$M_{j,Ed,z}$ =	-0.00	[kN*m]	Bending moment

RESULTS

CONNECTION CAPACITY CHECK

$N_{j,Ed} / N_{j,Rd} \leq 1,0$ (6.24)		0.02 < 1.00	verified	(0.02)
e_y =	802 [mm]	Axial force eccentricity		[6.2.8.3]
$z_{c,y}$ =	90 [mm]	Lever arm $F_{C,Rd,y}$		[6.2.8.1.(2)]
$z_{t,y}$ =	140 [mm]	Lever arm $F_{T,Rd,y}$		[6.2.8.1.(3)]
$M_{j,Rd,y}$ =	55.24 [kN*m]	Connection resistance for bending		[6.2.8.3]
$M_{j,Ed,y} / M_{j,Rd,y} \leq 1,0$ (6.23)		0.80 < 1.00	verified	(0.80)

e_z =	0	[mm]	Axial force eccentricity	[6.2.8.3]
$z_{c,z}$ =	83	[mm]	Lever arm $F_{C,Rd,z}$	[6.2.8.1.(2)]
$z_{t,z}$ =	113	[mm]	Lever arm $F_{T,Rd,z}$	[6.2.8.1.(3)]
$M_{j,Rd,z}$ =	0.00	[kN*m]	Connection resistance for bending	[6.2.8.3]

$M_{j,Ed,z} / M_{j,Rd,z} \leq 1{,}0$ (6.23)	0.04 < 1.00	verified	(0.04)
$M_{j,Ed,y} / M_{j,Rd,y} + M_{j,Ed,z} / M_{j,Rd,z} \leq 1{,}0$	0.83 < 1.00	verified	(0.83)

SHEAR

SHEAR CHECK

$V_{j,Rd,y} = n_b{*}\min(F_{1,vb,Rd,y}, F_{2,vb,Rd}, F_{v,Rd,cp}, F_{v,Rd,c,y}) + F_{f,Rd}$

$V_{j,Rd,y}$ =	206.31	[kN]	Connection resistance for shear	CEB [9.3.1]

$V_{j,Ed,y} / V_{j,Rd,y} \leq 1{,}0$	0.00 < 1.00	verified	(0.00)

$V_{j,Rd,z} = n_b{*}\min(F_{1,vb,Rd,z}, F_{2,vb,Rd}, F_{v,Rd,cp}, F_{v,Rd,c,z}) + F_{f,Rd}$

$V_{j,Rd,z}$ =	206.31	[kN]	Connection resistance for shear	CEB [9.3.1]

$V_{j,Ed,z} / V_{j,Rd,z} \leq 1{,}0$	0.11 < 1.00	verified	(0.11)
$V_{j,Ed,y} / V_{j,Rd,y} + V_{j,Ed,z} / V_{j,Rd,z} \leq 1{,}0$	0.11 < 1.00	verified	(0.11)

STIFFENER CHECK

Stiffener parallel to the web (along the extension of the column web)

M_1 =	9.24	[kN*m]	Bending moment acting on a stiffener	
Q_1 =	169.61	[kN]	Shear force acting on a stiffener	
z_s =	29	[mm]	Location of the neutral axis (from the plate base)	
I_s =	1819.98	[cm^4]	Moment of inertia of a stiffener	
σ_d =	2.20	[MPa]	Normal stress on the contact surface between stiffener and plate	EN 1993-1-1:[6.2.1.(5)]
σ_g =	84.65	[MPa]	Normal stress in upper fibers	EN 1993-1-1:[6.2.1.(5)]
τ =	99.19	[MPa]	Tangent stress in a stiffener	EN 1993-1-1:[6.2.1.(5)]
σ_z =	171.81	[MPa]	Equivalent stress on the contact surface between stiffener and plate	EN 1993-1-1:[6.2.1.(5)]

$\max(\sigma_g, \tau / (0.58), \sigma_z) / (f_{yp}/\gamma_{M0}) \leq 1.0$ (6.1)	0.73 < 1.00	verified	(0.73)

Stiffener perpendicular to the web (along the extension of the column flanges)

M_1 =	1.73	[kN*m]	Bending moment acting on a stiffener	
Q_1 =	46.07	[kN]	Shear force acting on a stiffener	
z_s =	39	[mm]	Location of the neutral axis (from the plate base)	
I_s =	1630.18	[cm^4]	Moment of inertia of a stiffener	
σ_d =	1.54	[MPa]	Normal stress on the contact surface between stiffener and plate	EN 1993-1-1:[6.2.1.(5)]
σ_g =	16.59	[MPa]	Normal stress in upper fibers	EN 1993-1-1:[6.2.1.(5)]
τ =	26.94	[MPa]	Tangent stress in a stiffener	EN 1993-1-1:[6.2.1.(5)]
σ_z =	46.69	[MPa]	Equivalent stress on the contact surface between stiffener and plate	EN 1993-1-1:[6.2.1.(5)]

max (σ_g, τ / (0.58), σ_z) / (f_{yp}/γ_{M0}) ≤ 1.0 (6.1)	0.20 < 1.00	verified	(0.20)

WELDS BETWEEN THE COLUMN AND THE BASE PLATE

$\sigma_\perp$ =	25.64	[MPa]	Normal stress in a weld	[4.5.3.(7)]
$\tau_\perp$ =	25.64	[MPa]	Perpendicular tangent stress	[4.5.3.(7)]
τ_{yII} =	0.00	[MPa]	Tangent stress parallel to $V_{j,Ed,y}$	[4.5.3.(7)]
τ_{zII} =	-4.27	[MPa]	Tangent stress parallel to $V_{j,Ed,z}$	[4.5.3.(7)]
β_W =	0.85		Resistance-dependent coefficient	[4.5.3.(7)]

$\sigma_\perp$ / (0.9*f_u/γ_{M2})) ≤ 1.0 (4.1)	0.10 < 1.00	verified	(0.10)
$\sqrt{}(\sigma_\perp^2 + 3.0\ (\tau_{yII}^2 + \tau_\perp^2))$ / ($f_u/(\beta_W*\gamma_{M2})$)) ≤ 1.0 (4.1)	0.15 < 1.00	verified	(0.15)
$\sqrt{}(\sigma_\perp^2 + 3.0\ (\tau_{zII}^2 + \tau_\perp^2))$ / ($f_u/(\beta_W*\gamma_{M2})$)) ≤ 1.0 (4.1)	0.13 < 1.00	verified	(0.13)

VERTICAL WELDS OF STIFFENERS

Stiffener parallel to the web (along the extension of the column web)

$\sigma_\perp$ =	83.82	[MPa]	Normal stress in a weld	[4.5.3.(7)]
$\tau_\perp$ =	83.82	[MPa]	Perpendicular tangent stress	[4.5.3.(7)]
τ_{II} =	61.99	[MPa]	Parallel tangent stress	[4.5.3.(7)]
σ_z =	199.08	[MPa]	Total equivalent stress	[4.5.3.(7)]
β_W =	0.85		Resistance-dependent coefficient	[4.5.3.(7)]

max ($\sigma_\perp$, τ_{II} * √3, σ_z) / ($f_u/(\beta_W*\gamma_{M2})$) ≤ 1.0 (4.1)	0.58 < 1.00	verified	(0.58)

Stiffener perpendicular to the web (along the extension of the column flanges)

$\sigma_{\perp}$ =	15.67	[MPa]	Normal stress in a weld	[4.5.3.(7)]
$\tau_{\perp}$ =	15.67	[MPa]	Perpendicular tangent stress	[4.5.3.(7)]
τ_{II} =	16.84	[MPa]	Parallel tangent stress	[4.5.3.(7)]
σ_z =	42.81	[MPa]	Total equivalent stress	[4.5.3.(7)]
β_W =	0.85		Resistance-dependent coefficient	[4.5.3.(7)]

max ($\sigma_{\perp}$, τ_{II} * √3, σ_z) / (f_u/(β_W*γ_{M2})) ≤ 1.0 (4.1) 0.12 < 1.00 verified (0.12)

TRANSVERSAL WELDS OF STIFFENERS

Stiffener parallel to the web (along the extension of the column web)

$\sigma_{\perp}$ =	83.75	[MPa]	Normal stress in a weld	[4.5.3.(7)]
$\tau_{\perp}$ =	83.75	[MPa]	Perpendicular tangent stress	[4.5.3.(7)]
τ_{II} =	80.85	[MPa]	Parallel tangent stress	[4.5.3.(7)]
σ_z =	218.33	[MPa]	Total equivalent stress	[4.5.3.(7)]
β_W =	0.85		Resistance-dependent coefficient	[4.5.3.(7)]

max ($\sigma_{\perp}$, τ_{II} * √3, σ_z) / (f_u/(β_W*γ_{M2})) ≤ 1.0 (4.1) 0.64 < 1.00 verified (0.64)

Stiffener perpendicular to the web (along the extension of the column flanges)

$\sigma_{\perp}$ =	29.09	[MPa]	Normal stress in a weld	[4.5.3.(7)]
$\tau_{\perp}$ =	29.09	[MPa]	Perpendicular tangent stress	[4.5.3.(7)]
τ_{II} =	17.18	[MPa]	Parallel tangent stress	[4.5.3.(7)]
σ_z =	65.35	[MPa]	Total equivalent stress	[4.5.3.(7)]
β_W =	0.85		Resistance-dependent coefficient	[4.5.3.(7)]

max ($\sigma_{\perp}$, τ_{II} * √3, σ_z) / (f_u/(β_W*γ_{M2})) ≤ 1.0 (4.1) 0.19 < 1.00 verified (0.19)

Connection conforms to the code	Ratio	0.83

Appendix 4

Foundation

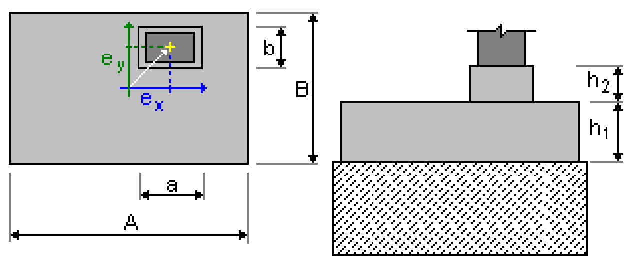
1 Spread footing: Foundation37 Number of identical elements: 1

1.1 Basic data

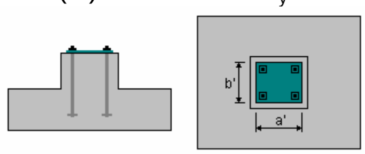
1.1.1 Assumptions

- Geotechnic calculations according to : DTU 13.12
- Concrete calculations according to : BAEL 91 mod. 99
- Foundation with lean concrete
- Shape selection : without limits

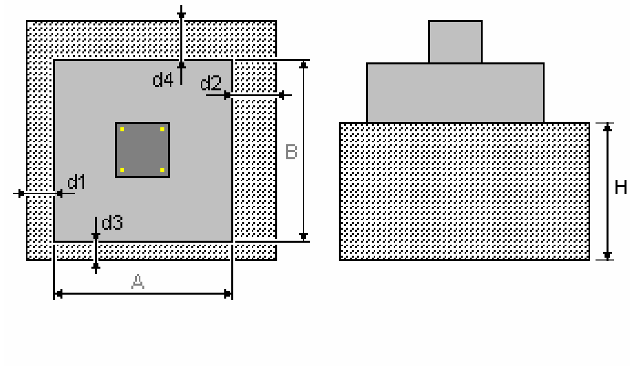
1.1.2 Geometry:



A	= 2.30 (m)	a	= 0.66 (m)
B	= 1.40 (m)	b	= 0.59 (m)
h1	= 0.50 (m)	e_x	= 0.00 (m)
h2	= 0.50 (m)	e_y	= 0.00 (m)



a'	= 36.0 (cm)
b'	= 33.0 (cm)
c1	= 5.0 (cm)
c2	= 3.0 (cm)



H = 0.10 (m)
d1 = 0.10 (m)
d2 = 0.10 (m)
d3 = 0.10 (m)
d4 = 0.10 (m)

1.1.3 Materials

- Concrete : BETON25; Characteristic strength = 25.00 MPa
  Unit weight = 2501.36 (kG/m3)
- Lean concrete : BETON20; Characteristic strength = 20.00 MPa
  Unit weight = 2501.36 (kG/m3)
- Longitudinal reinforcement : type HA 400 Characteristic strength = 400.00 MPa
- Transversal reinforcement : type HA 400 Characteristic strength = 400.00 MPa
- Additional reinforcement: : type HA 400 Characteristic strength = 400.00 MPa

1.1.4 Loads:

Foundation loads:

Case	Nature	Group	N (kN)	Fx (kN)	Fy (kN)	Mx (kN*m)	My (kN*m)
DL1	dead load(Structural)	37	10.49	1.99	-0.00	0.00	3.58
1	dead load(Structural)	37	4.16	1.71	-0.00	0.00	2.91
11	dead load(Structural)	37	0.37	0.15	-0.00	0.00	0.26
111	dead load(Structural)	37	8.49	1.90	-0.00	0.00	3.44
1111	dead load(Structural)	37	0.27	-0.01	-0.00	0.00	-0.00
11111	dead load(Structural)	37	0.00	0.00	0.00	0.00	0.00
111111	live load(Category A)	37	9.78	4.03	-0.00	0.00	6.85
1111111	live load(Category A)	37	0.00	-0.00	0.00	-0.00	-0.00
2	live load(Category A)	37	0.98	0.40	-0.00	0.00	0.69
WIND1	wind	37	-10.77	1.95	0.15	-0.21	-1.43
WIND2	wind	37	-10.77	1.95	-0.15	0.21	-1.43
WIND3	wind	37	-4.34	-15.29	-0.00	0.00	-22.53
WIND4	wind	37	-5.81	1.73	0.00	-0.00	6.55
WIND5	wind	37	8.12	-9.78	-0.00	0.00	-12.90
WIND6	wind	37	7.62	6.78	0.00	-0.00	14.85

Backfill loads:

Case	Nature	Q1

(kN/m2)

1.3 RC design

1.3.1 Assumptions

- Cracking : limited
- Exposure : mild
- Include element sensitivity condition for fragile failure : yes

1.3.2 Analysis of punching and shear

Shear

Design combination **ULS :**
1.35DL1+1.351+1.3511+1.35111+1.351111+1.3511111+1.00111111+1.002+1.80WIND6
Load factors: **1.00** * Foundation weight
1.00 * Soil weight
Design load:
Nr = 129.15 (kN) Mx = 0.00 (kN*m) My = 72.43 (kN*m)
Length of critical circumference: 1.40 (m)
Shear force: 36.93 (kN)
Section effective height h_{eff} = 0.44 (m)
Shear area: A = 0.62 (m2)
Shear stress: 0.06 (MPa)
Admissible shear stress: 1.17 (MPa)
Safety factor: 19.46 > 1

1.3.3 Required reinforcement

Spread footing:

bottom:

ULS :
1.35DL1+1.351+1.3511+1.35111+1.351111+1.3511111+1.00111111+1.002+1.80WIND6
My = 36.60 (kN*m) A_{sx} = 4.84 (cm2/m)

ULS :
1.35DL1+1.351+1.3511+1.35111+1.351111+1.3511111+1.50111111+1.501111111+1.502+1.20WIND5
Mx = 5.22 (kN*m) A_{sy} = 4.84 (cm2/m)

$A_{s\ min}$ = 4.40 (cm2/m)

top:
A'_{sx} = 0.00 (cm2/m)
A'_{sy} = 0.00 (cm2/m)

$A_{s\ min}$ = 0.00 (cm2/m)

Maximum code-specified spacing e_{max} = 0.25 (m)

Column pier:
Longitudinal reinforcement A = 10.00 (cm2) $A_{min.}$ = 10.00 (cm2)
A = 2 * (Asx + Asy)

Asx = 4.63 (cm2) Asy = 0.37 (cm2)

Splice reinforcement (foundation to lean concrete): A = 2.91 (cm2)

1.3.4 Provided reinforcement

2.3.1 Spread footing:

Bottom:

Along X axis:

9 HA 400 10 l = 2.20 (m) e = 1*-0.59 + 8*0.15

Along Y axis:

15 HA 400 10 l = 1.30 (m) e = 1*-1.05 + 14*0.15

Top:

2.3.2 Pier

Longitudinal reinforcement

Along Y axis:

12 HA 400 12 l = 1.08 (m) e = 1*-0.25 + 5*0.10

Transversal reinforcement

6 HA 400 10 l = 2.46 (m) e = 1*0.18 + 3*0.20 + 2*0.09

Splice reinforcement

12 HA 400 12 l = 0.68 (m) e_x = 0.65 e_y = 1*-0.66 + 1*0.66 + 1*0.65

2 Material survey:

- Concrete volume = 1.80 (m3)
- Lean concrete volume = 0.40 (m3)
- Formwork = 4.95 (m2)

- Steel HA 400
  - Total weight = 52.06 (kG)
  - Density = 28.85 (kG/m3)
  - Average diameter = 10.6 (mm)
  - Survey according to diameters:

Diameter	Length (m)	Weight (kG)
10	54.03	33.33
12	21.09	18.73

Appendix 5

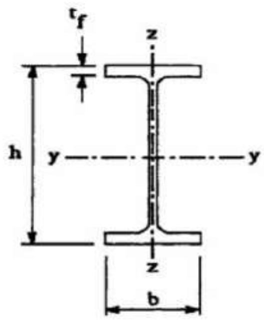
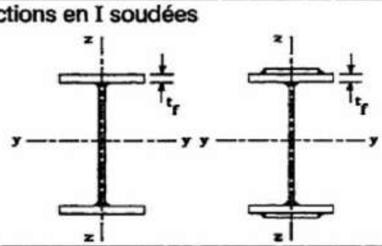
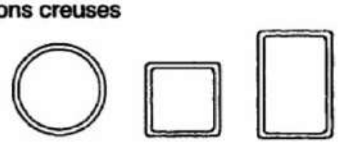
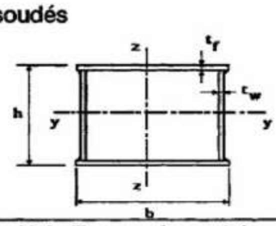
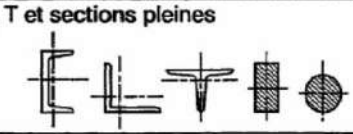
Reinforcement table

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

Section en cm² de N armatures de diamètre Φ (mm)

Appendix 6

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 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

Appendix 7

Désignation Designation Bezeichnung		Dimensions Abmessungen						Dimensions de construction Dimensions for detailing Konstruktionsmaße					Surface Oberfläche	
	G kg/m	h mm	b mm	t_w mm	t_f mm	r mm	A mm^2 $x10^2$	h_i mm	d mm	Ø	p_{min} mm	p_{max} mm	A_L m^2/m	A_G m^2/t
IPE AA 140*	10,1	136,6	73	3,8	5,2	7,0	12,8	126,2	112,2	-	-	-	0,546	54,26
IPE A 140•	10,5	137,4	73	3,8	5,6	7,0	13,4	126,2	112,2	-	-	-	0,547	52,05
IPE 140	12,9	140	73	4,7	6,9	7,0	16,4	126,2	112,2	-	-	-	0,551	42,70
IPE AA 220*	21,2	216,4	110	4,7	7,4	12,0	27,0	201,6	177,6	M 12	60	62	0,843	39,78
IPE A 220•	22,2	217	110	5,0	7,7	12,0	28,3	201,6	177,6	M 12	60	62	0,843	38,02
IPE 220	26,2	220	110	5,9	9,2	12,0	33,4	201,6	177,6	M 12	60	62	0,848	32,36
IPE O 220+	29,4	222	112	6,6	10,2	12,0	37,4	201,6	177,6	M 10	58	66	0,858	29,24
IPE A 300•	36,5	297	150	6,1	9,2	15,0	46,5	278,6	248,6	M 16	72	86	1,156	31,65
IPE 300	42,2	300	150	7,1	10,7	15,0	53,8	278,6	248,6	M 16	72	86	1,160	27,46
IPE O 300+	49,3	304	152	8,0	12,7	15,0	62,8	278,6	248,6	M 16	74	88	1,174	23,81

Désignation Designation Bezeichnung		Valeurs statiques / Section properties / Statische Kennwerte axe fort y-y strong axis y-y starke Achse y-y					axe faible z-z weak axis z-z schwache Achse z-z							Classification EN 1993-1-1: 2005 Pure bending y-y			Pure compression			EN 10025-2: 2004	EN 10025-4: 2004	EN 10225:2001
	G kg/m	I_y mm^4 $x10^4$	$W_{el.y}$ mm^3 $x10^3$	$W_{pl.y}$♦ mm^3 $x10^3$	i_y mm x10	A_{vz} mm^2 $x10^2$	I_z mm^4 $x10^4$	$W_{el.z}$ mm^3 $x10^3$	$W_{pl.z}$♦ mm^3 $x10^3$	i_z mm x10	s_s mm	I_t mm^4 $x10^4$	I_w mm^6 $x10^9$	S235	S355	S460	S235	S355	S460			
IPE AA 140	10,1	407	59,7	67,6	5,64	6,14	33,8	9,27	14,5	1,63	22,4	1,19	1,46	1	1	-	1	2	-	✓		
IPE A 140	10,5	435	63,3	71,6	5,70	6,21	36,4	10,0	15,5	1,65	23,2	1,36	1,58	1	1	1	1	2	3	✓	✓	✓
IPE 140	12,9	541	77,3	88,3	5,74	7,64	44,9	12,3	19,3	1,65	26,7	2,45	1,98	1	1	1	1	1	2	✓	✓	✓
IPE AA 220	21,2	2219	205	230	9,07	12,8	165	29,9	46,5	2,47	33,6	5,02	17,9	1	1	-	2	4	-	✓		
IPE A 220	22,2	2317	214	240	9,05	13,6	171	31,2	48,5	2,46	34,5	5,69	18,7	1	1	1	2	4	4	✓	✓	✓
IPE 220	26,2	2772	252	285	9,11	15,9	205	37,3	58,1	2,48	38,4	9,07	22,7	1	1	1	1	2	4	✓	✓	✓
IPE O 220	29,4	3134	282	321	9,16	17,7	240	42,8	66,9	2,53	41,1	12,3	26,8	1	1	1	1	2	2	✓	✓	✓
IPE A 300	36,5	7173	483	542	12,4	22,3	519	69,2	107	3,34	42,1	13,4	107	1	1	1	3	4	4	✓	✓	✓
IPE 300	42,2	8356	557	628	12,5	25,7	604	80,5	125	3,35	46,1	20,1	126	1	1	1	2	4	4	✓	✓	✓
IPE O 300	49,3	9994	658	744	12,6	29,1	746	98,1	153	3,45	51,0	31,1	158	1	1	1	1	3	4	✓	✓	✓

Désignation Designation Bezeichnung		Dimensions Abmessungen						Dimensions de construction Dimensions for detailing Konstruktionsmaße					Surface Oberfläche	
	G kg/m	h mm	b mm	t_w mm	t_f mm	r mm	A mm^2 $x10^2$	h_i mm	d mm	Ø	p_{min} mm	p_{max} mm	A_L m^2/m	A_G m^2/t
HE 180 AA*	28,7	167	180	5	7,5	15	36,5	152	122	M 24	84	92	1,018	35,51
HE 180 A	35,5	171	180	6	9,5	15	45,3	152	122	M 24	86	92	1,024	28,83
HE 180 B	51,2	180	180	8,5	14	15	65,3	152	122	M 24	88	92	1,037	20,25
HE 180 C*	69,8	190	183	11,5	19	15	89,0	152	122	M 27	92	96	1,063	15,22
HE 180 M	88,9	200	186	14,5	24	15	113,3	152	122	M 24	94	98	1,089	12,25

Désignation / Designation / Bezeichnung	Valeurs statiques / Section properties / Statische Kennwerte													Classification EN 1993-1-1: 2005						EN 10025-2: 2004	EN 10025-4: 2004	EN 10225:2001
	axe fort y-y / strong axis y-y / starke Achse y-y					axe faible z-z / weak axis z-z / schwache Achse z-z								Pure bending y-y			Pure compression					
G kg/m	I_y mm^4 $x10^4$	$W_{el.y}$ mm^3 $x10^3$	$W_{pl.y}$♦ mm^3 $x10^3$	i_y mm x10	A_{vz} mm^2 $x10^2$	I_z mm^4 $x10^4$	$W_{el.z}$ mm^3 $x10^3$	$W_{pl.z}$♦ mm^3 $x10^3$	i_z mm x10	s_s mm	I_t mm^4 $x10^4$	I_w mm^6 $x10^9$		S235	S355	S460	S235	S355	S460			
HE 180 AA	28,7	1967	235,6	258,2	7,34	12,16	730,0	81,11	123,6	4,47	37,57	8,33	46,36	2	3	3	2	3	3	✓	✓	✓
HE 180 A	35,5	2510	293,6	324,9	7,45	14,47	924,6	102,7	156,5	4,52	42,57	14,80	60,21	1	2	3	1	2	3	✓	✓	✓
HE 180 B	51,2	3831	425,7	481,4	7,66	20,24	1363	151,4	231,0	4,57	54,07	42,16	93,75	1	1	1	1	1	1	✓	✓	✓
HE 180 C	69,8	5543	583,5	675,0	7,89	27,30	1944	212,5	324,9	4,68	67,07	102,1	141,9	1	1	-	1	1	-	✓		
HE 180 M	88,9	7483	748,3	883,4	8,13	34,65	2580	277,4	425,2	4,77	80,07	203,3	199,3	1	1	1	1	1	1	✓	✓	✓

Désignation / Designation / Bezeichnung		Dimensions / Abmessungen							Dimensions de construction / Dimensions for detailing / Konstruktionsmaße				Surface / Oberfläche	
	G kg/m	h mm	b mm	t_w mm	t_f mm	r_1 mm	r_2 mm	A mm^2 $x10^2$	d mm	Ø	e_{min} mm	e_{max} mm	A_L m^2/m	A_G m^2/t
UPN 80*	8,64	80	45	6,0	8,0	8,0	4,0	11,0	47	-	-	-	0,312	37,10
UPN 140	16,0	140	60	7,0	10,0	10,0	5,0	20,4	98	M 12	33	37	0,489	30,54

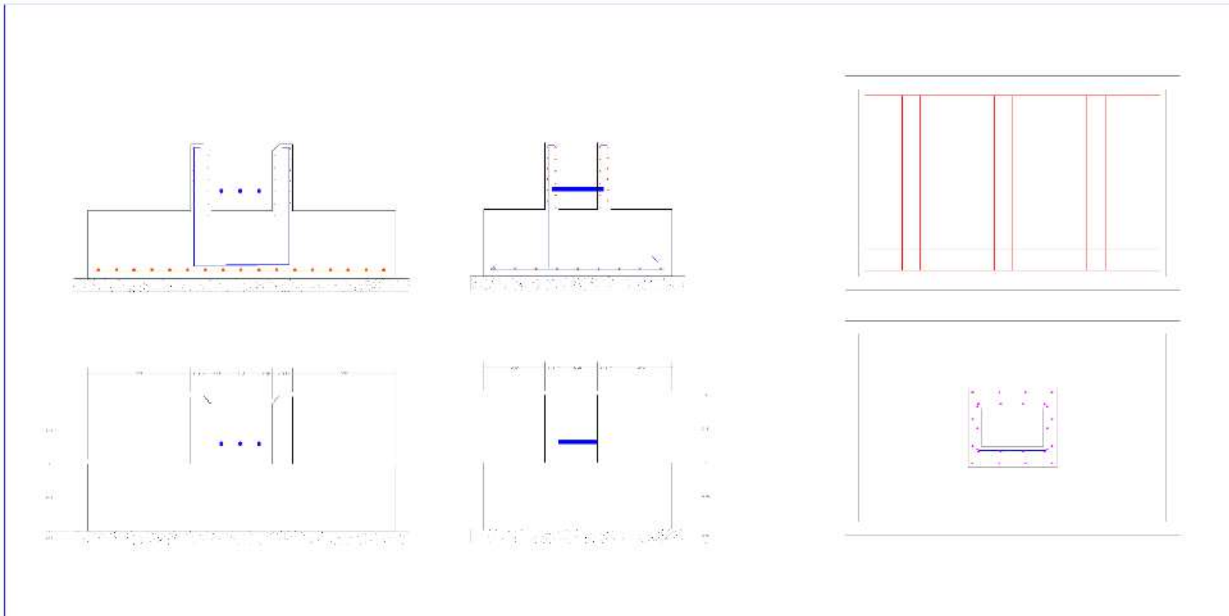
Désignation / Designation / Bezeichnung		Valeurs statiques / Section properties / Statische Kennwerte															Classification EN 1993-1-1: 2005				EN 10025-2: 2004	EN 10025-4: 2004	EN 10225:2001
		axe fort y-y / strong axis y-y / starke Achse y-y					axe faible z-z / weak axis z-z / schwache Achse z-z										Pure bending y-y		Pure compression				
	G kg/m	I_y mm^4 $x10^4$	$W_{el.y}$ mm^3 $x10^3$	$W_{pl.y}$■ mm^3 $x10^3$	i_y mm x10	A_{vz} mm^2 $x10^2$	I_z mm^4 $x10^4$	$W_{el.z}$ mm^3 $x10^3$	$W_{pl.z'}$ mm^3 $x10^3$	i_z mm x10	s_s mm	I_t mm^4 $x10^4$	I_w mm^6 $x10^9$	y_s mm x10	y_m mm x10		S235	S355	S235	S355			
UPN 80	8,64	106	26,5	32,3	3,10	4,90	19,4	6,36	11,9	1,33	19,4	2,20	0,18	1,45	2,67		1	1	1	1	✓		
UPN 140	16,0	605	86,4	103	5,45	10,4	62,7	14,8	28,3	1,75	23,9	5,68	1,80	1,75	3,37		1	1	1	1	✓		

Désignation / Designation / Bezeichnung		Dimensions / Abmessungen					Position des axes / Position of axes / Lage der Achsen				Surface / Oberfläche	
	G kg/m	h=b mm	t mm	r_1 mm	r_2 mm	A mm^2 $x10^2$	z_s=y_s mm x10	v mm x10	u_1 mm x10	u_2 mm x10	A_L m^2/m	A_G m^2/t
L 100 x 100 x 7*	10,7	100	7	12	6,0	13,7	2,69	7,07	3,81	3,51	0,390	36,33

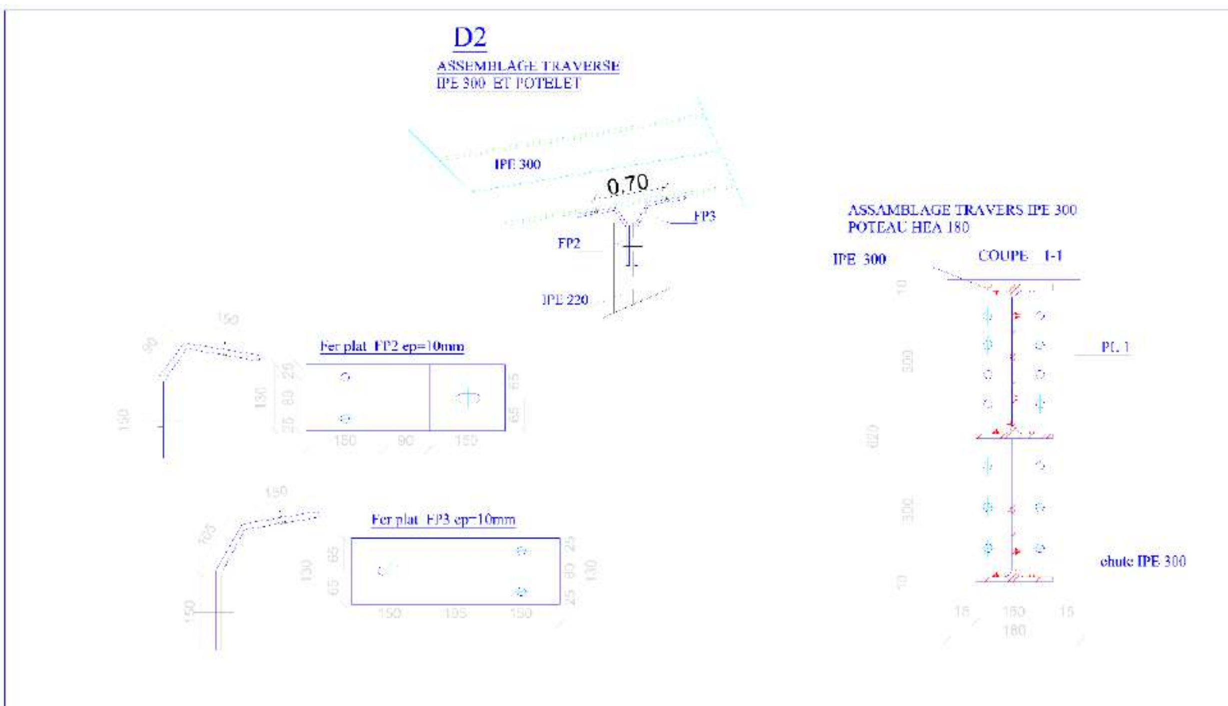
Désignation / Designation / Bezeichnung		Dimensions / Abmessungen					Dimensions de construction / Dimensions for detailing / Konstruktionsmaße			
	G kg/m	h=b mm	t mm	r_1 mm	r_2 mm	A mm^2 $x10^2$	Ø	e_{min} mm	e_{max} mm	A_{net} mm^2 $x10^2$
L 100 x 100 x 7*	10,7	100	7	12	6,0	13,7	M 27	47	53	11,6

Désignation / Designation / Bezeichnung		Valeurs statiques / Section properties / Statische Kennwerte								Classification EN 1993-1-1: 2005		EN 10025-2: 2004	EN 10025-4: 2004	EN 10225:2001
		axe y-y / axe z-z / axis y-y / axis z-z / Achse y-y / Achse z-z			axe u-u / axis u-u / Achse u-u		axe v-v / axis v-v / Achse v-v			Pure compression				
	G kg/m	I_y= I_z mm^4 $x10^4$	$W_{el.y}$= $W_{el.z}$ mm^3 $x10^3$	i_y= i_z mm x10	I_u mm^4 $x10^4$	i_u mm x10	I_v mm^4 $x10^4$	i_v mm x10	I_{yz} mm^4 $x10^4$	S235	S355			
L 100 x 100 x 7	10,7	128,2	17,54	3,06	203,7	3,86	52,72	1,96	-75,48	3	3	✓		

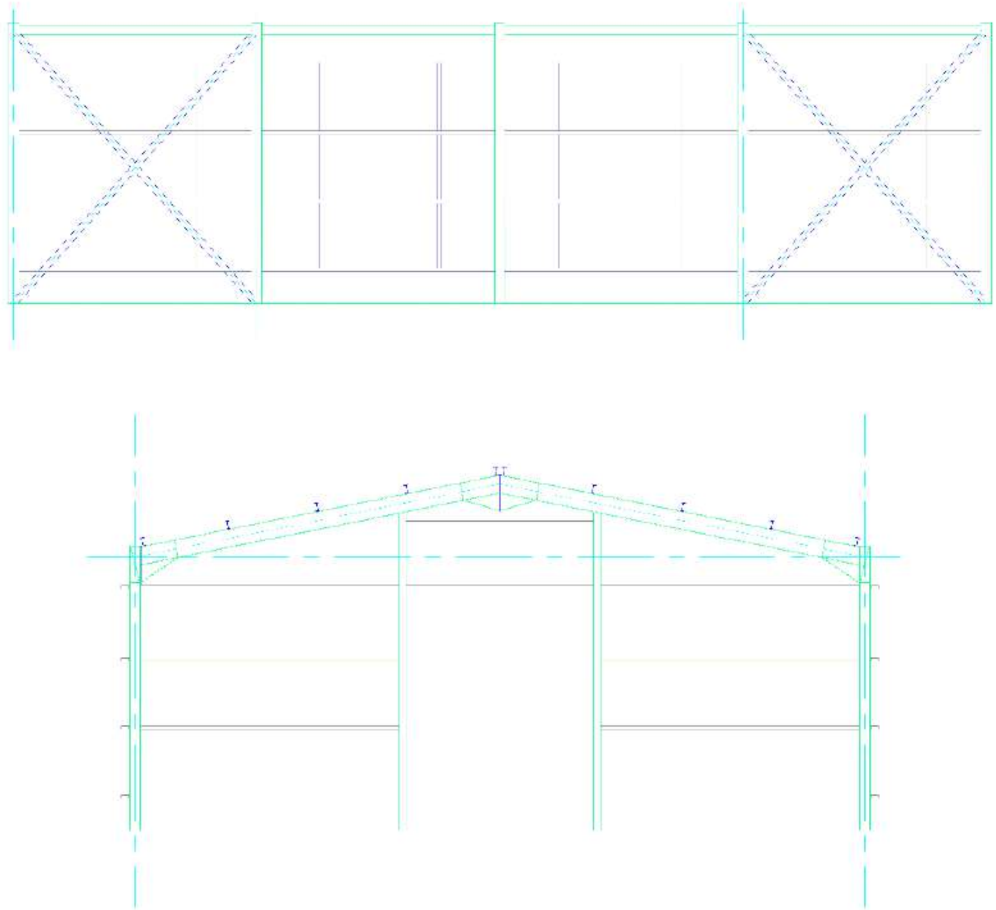
Appendix 8



PLAN COFFRAGE ET FERRAILLGE SEMELLES



PLAN ASSAMBLAGE POTEAU- TRAVERSE et POTELLET - TRAVERSE



PLAN DE REPERAGE PIGNON et LONG-PAN

For more Calculation information or project drawings, visit the link below.

https://mirakski.github.io/PFE/

or scan the code below