الجـــــــ

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Firstly, I thank Almighty Allah for making it possible and giving me the strength and patience to carry out this work.

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Dedication

I wish to dedicate this work to my parents Alhaji Ibrahim Yaro Nuhu and Hajiya Jummai T. Ibrahim-Nuhu who have been my pillars of support right from birth till this moment. They laid a solid foundation for my educational achievements. May The Almighty God reward them abundantly.

I also wish to dedicate this work to Mr Hamdaoui Karim, who had been a father figure and a mentor to me here in Algeria. May The Almighty God reward him abundantly.

ABSTRACT

This project consists of studying the 9 floors of an accommodation building located in REMCHI, wilaya of TLEMCEN.

This project is developed in several stages, after the presentation of the project, the decent load and climate study (wind and snow) were carried out according to «RNV 2013» regulations. We proceeded then to the fundamental step for the pre-dimensioning of load-bearing elements which are subsequently checked to validate the use or the replace of the profiles to ensure the structural safety according to the Algerian code «CCM97». The seismic study was conducted according to «RPA99 v 2003» regulations, then the study of the mixed floor according to the «EUROCODE 4». The connections study was performed according to the «EUROCODE 3», and at the end, foundations were dimensioned according to the «BAEL91» code.

Keywords: steel structure, mixed floor, study of an accommodation building, climatic study.

RESUME

Ce projet consiste à étudier un bâtiment R+8 en charpente métallique situé à REMCHI, wilaya de TLEMCEN.

Ce projet est élaboré en plusieurs étapes ; après la présentation du projet on a effectué la décente de charge et l'étude climatique (vent et neige) selon le règlement « RNV v 2013 ». On a procédé ensuite à l'étape fondamentale de pré dimensionnement d'éléments porteurs qui ont été vérifiés par la suite pour valider l'utilisation des profilés ou leurs remplacements pour assurer la sécurité structurale, et ceci selon le code algérien « CCM97 ». L'étude sismique était effectuée selon le règlement en vigueur, le « RPA99 v 2003 », ensuite l'étude du plancher mixte selon « l'EUROCODE 4 ». L'étude des assemblages était élaborée en utilisant «l'EUROCODE 3» et par la fin l'étude de l'infrastructure selon le code «BAEL91».

Mots clés : charpente métallique, plancher mixte, étude d'un bâtiment, calcul neige et vent.

ملخص

يهدف هذا المشروع لدراسة مبنى سكني ذو 9 طوابق يقع في مدينة الرمشي بولاية تلمسان

انجز هذا المشروع و تطور في عدة مراحل، فبعد تقديمه نفذت نسب نزول الأثقال والدراسة المناخية (رياح وثلوج) وفق المنشور«RNVv2013 ». بعد ذلك تطرقنا للمرحلة الأساسية لتحديد أبعاد عناصر الأحمال و التي تم التحقق فيها فيما بعد واستعمالها أو استبدالها للتأمين الهيكلي وقفا للمنشور الجزائري «CCM97 ». تمت الدراسة الزلزالية وفق للمنشور «RPA99v2003 » وبعدها دراسة الأرضية المركبة وفقا لـ« EUROCODE4 » . تمت دراسة الوصلات والتجميع باستعمال «EUROCODE3 » وفي الأخير البنية التحتية وفق المنشور «BAEL91» .

الكلمات المفتاحية: هيكل معدني، أرضية مزدوجة، دراسة مبنى سكني، حساب أحمال الثلوج والرياح.

LIST OF NOTATIONS

Loads :

G : Dead Load.

Q : Live Load.

N : Snow load.

V : Wind load.

The characteristic of an I or H section:

b : flange width.

d : height of the web.

h : section depth.

t_w : web thickness.

t_f : flange thickness.

Latin capital letters:

A : gross section of a member.

A_{net} : net section of a member.

A_w : web section.

A_v : shear area.

C_t : topographic coefficient.

C_r : roughness coefficient.

C_e : operating coefficient.

C_d : dynamic coefficient.

E : Young's modulus.

F : Force.

G : transverse modulus of elasticity.

I : moment of inertia

M : bending moment.

M_{sd} : Applied bending moment.

$M_{pl, Rd}$: Plastic moment of resistance.

$M_{b, Rd}$:lateral-torsional buckling resistance moment.

$N_{pl, Rd}$: normal force of the plastic resistance of the gross cross-section.

$N_{b, Rd}$: effort normal d'un élément comprimé au flambement.

N_{sd} : Applied normal force.

$N_{t, sd}$: Normal tensile stress.

$N_{c, Rd}$: calculation value of the cross-sectional compressive strength.

R : compression coefficient of the structure.

T : Period.

V_{sd} : calculation value of transverse shear force.

W_{pl} : plastic resistance modulus.

W : weight of the structure.

Latin lowercase:

f **:** Deflection.

f_y : Yield strength.

h : height of the member.

L : length of the member (column –beam).

L_f : buckling length.

t : thickness of the member.

Z : height above sol.

Z_0 : roughness parameter.

$Z_{éq}$: Equivalent height.

Greek lowercase:

χ : Reduction coefficient for appropriate buckling mode.

β : The effective damping of the FPS isolator.

β_w : Correlation factor.

λ : slenderness.

γ_M : factor of safety.

λ_{LT} : lateral-torsional buckling slenderness.

α : Imperfection factor.

ε : Elastic reduction coefficient of steel.

σ_c : Steel stress.

σ_b : Concrete stress.

ξ : Percentage of critical attenuation.

η : Attenuation correction factor.

μ : Coefficient of shape of the snow load.

Index:

adm : admissible.

eff : effective.

m : average.

fr : friction.

réf : reference.

max : maximum.

min : minimum.

corresp : corresponding.

e :external.

i :internal.

x : x direction.

y : y direction.

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General introduction

General Introduction

A metal structure is a construction where its main and secondary elements are made of steel. The latter make it possible to cross large ranges, which is due to their resistances and ductilities. They also resist seismic forces.

In this end of study project which focuses on the study and sizing of an accommodation in metal frame G+8 located in REMCHI in the wilaya of TLEMCEN. We will apply the knowledge and information acquired during our training, using the rules of construction currently in force in Algeria, as well as the means of computer calculation.

The project is subdivided into several chapters each presenting a specific part : in the first and second chapters we briefly presented our project and the different materials used as well as the different technical regulations, while in the third we evaluated the different loads and overloads of the building that will later be used for the pre-dimensioning of the elements in the fourth chapter.

The behavior of the structure vis-à-vis the earthquake was elaborated by the static method in the sixth chapter followed by the verification of the load-bearing elements vis-à-vis the various instabilities and the study of the assemblies, preceded by the study of the composite floor, the study is ended by the calculation of the infrastructure.

The thesis is concluded with a general conclusion.

Chapter I:
Generality

I.1. Introduction :

I.2. Project Presentation :

The project consists of dimensioning multistory building (G+8) in mixed construction intended for residential use. It is situated in **Remchi**, Wilaya of **Tlemcen**.

The building occupies a ground area of 664.56m^2 with a height of 28.04m consisting of a ground floor, eight other levels with inaccessible terrace.

Figure I. 5 : Project site location

I.2.1. Geometric Characteristics:

The geometric characteristics of the building are:

- Area occupied by the building: **664.56m^2**
- Total length of the building: **23.4m**
- Total breadth of the building: **28.4m**
- Total height of the building: **28.04m**
- Total height of the building without the parapet: **27.54m**
- Height of the ground floor: **3.06m**

- Height of the other floors: **3.06m**

I.2.2. Location and data concerning the project site:

The building is in **Remchi** in the Wilaya of **Tlemcen** classified by the **RPA 99 version 2003 [2]** as a zone with a low seismicity: **Zone 1**

- Altitude of Remchi (Wilaya of Tlemcen): **213m**
- Wind zone: **Zone II**
- Snow zone: **Zone A** according to the **RNV v 2003 [1]** classification
- Site classification: **S2** (Firm soil)
- Allowable bearing capacity of the soil: $\overline{\sigma_{sol}} = 2bars$
- Usage group of structure: structures of average importance

I.3. Technical Regulations used:

- **DTR B C 2 48**: Règles parasismiques Algériennes **RPA 99 / Version 2003**): Algerian Paraseismic calculation rules
- **D.T.R.-B.C.-2.44** Règles de conception et de calcul des structures en acier "CCM 97 »: Rules for design and calculation of steel structures
- **D.T.R. C 2-4.7** Règlement neige et vent “R.N.V. 1999”: Snow and wind regulations
- **BAEL 91 v 99** (Béton armé aux états limites) : Reinforced concrete in ‘limit states’
- **DTR B.C. 2.2:** Charges permanentes et d’exploitation) : Dead and Live Loads
- **Eurocode 4**: Design of composite steel and concrete structures
- **Eurocode 3** : Design of steel structures

I.4. Softwares used:

- Auto CAD 2019.
- AUTODESK ROBOT 2014.

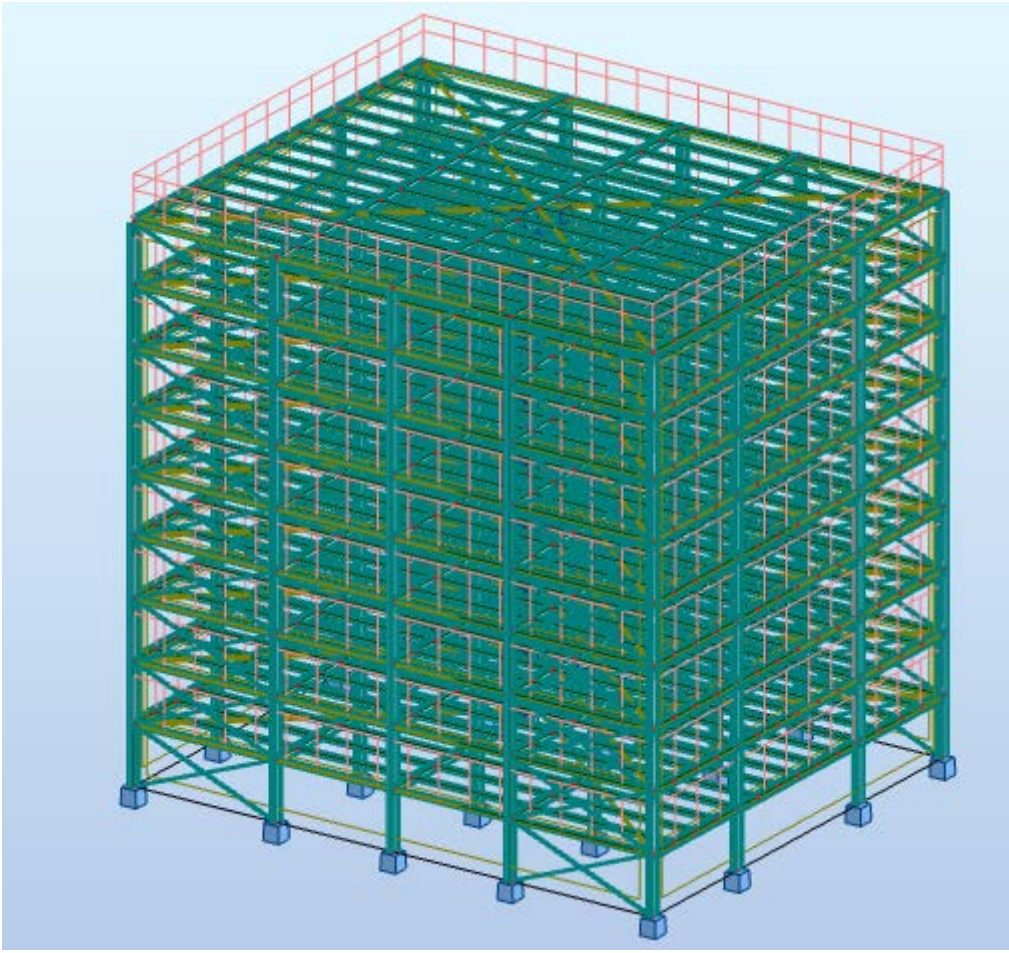


Figure I. 6 : Modelisation of the building (REMCHI -TLEMCEN).

I.5. Materials used :

I.5.1. Structural Steel:

I.5.1.1. Steels used for the load-bearing elements of the structure:

The steels used in buildings are of different types and grades in relation to the mechanical characteristics it represents. The mechanical characteristics of different steel grades (see Table [3]) are as follows :

Table I. 2 : Nominal values of fu and fy.

Steel Grade	Thichkness (mm)			
	t≤40 mm		40 mm≤t≤100 mm	
	F_y(N/mm^2)	F_u(N/mm^2)	F_y(N/mm^2)	F_u(N/mm^2)
Fe 360	235	360	215	340
Fe 430	275	430	255	410
Fe 510	355	510	355	490

The mechanical characteristics of the steel are:

- Résistance limite d'élasticité (Yielding stress) : $f_y = 235MPa$
- Tensile strength : $f_u = 360\,\text{MPa}$
- Young's modulus : $E = 210000\text{MPa}$
- Poisson ration : $\vartheta = 0.3$
- Shear modulus : $G = E/2(1+\vartheta) = 80769 \approx 81000MPa$

I.5.1.2. Reinforcement Steel:

- Sollicitations under normal actions :

$$\sigma_s = \frac{f_e}{\gamma_s} = \frac{400}{1{,}15} = 348\,\text{MPa}$$

- Sollicitations under accidental actions :

$$\sigma_s = 400MPa$$

I.5.2. Reinforced Concrete:

The concrete used for the realisation of the substructures and the slabs has a dosage of 350 kg/m^3 and 150kg/m^3 for the blinding concrete.

➢ Characteristics of the concrete :

The concrete is mechanically defined by:

- Compressive strength at 28 days for concrete class C25/30: fc_{28}=25Mpa
- tensile strength at 28 days is deduced from that of compression by the relationship: $f_{t28} = 0.06f_{c28} + 0.6 = 2.1MPa$

I.6. Conclusion :

In this chapter we determined the characteristics of the materials used which will serve us in the chapters to follow.

Chapter II:
Design of the structure

II.1. Introduction :

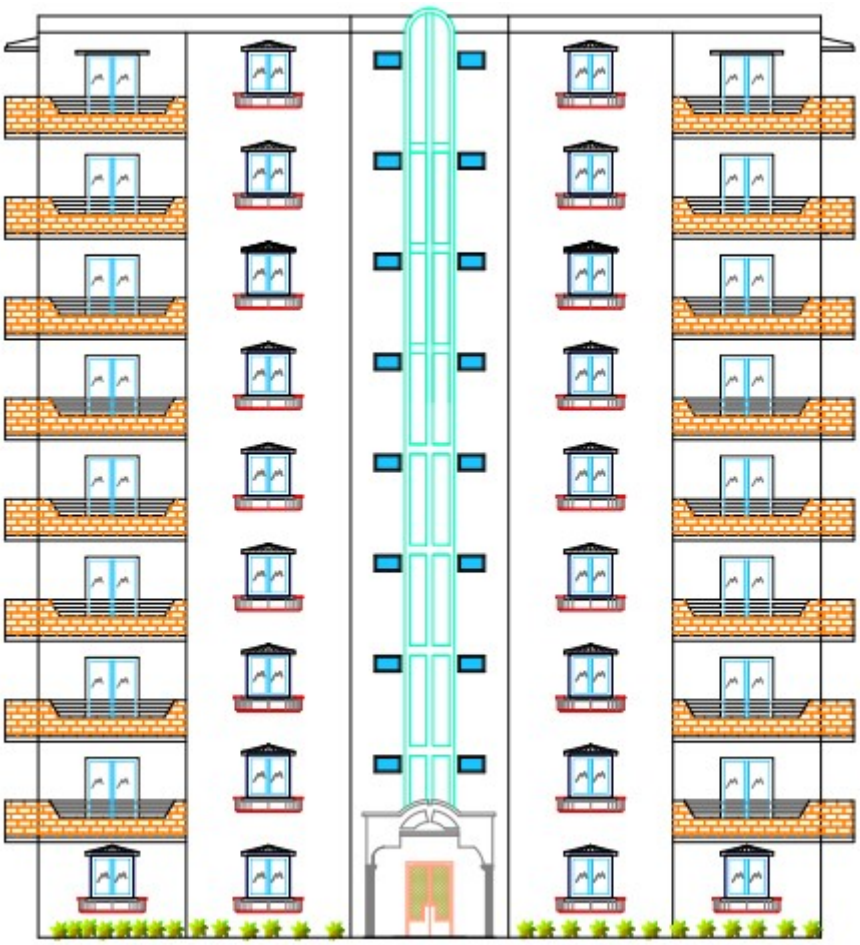
The design and modeling phase of a structure is among the most essential phases in a construction project; thus it must be considered as a priority in the study of a project.

Functional and structural aspects are considered for the design of the project, due to this, it is necessary to know the different viable solutions with their particular suggestions, limitations and costs.

II.2. Architectural design :

The building is rectangular in shape and composed of a ground floor and eight floors (see ANEXE A) :

- All floors are for use as accommodation
- Inaccessible Terrace.



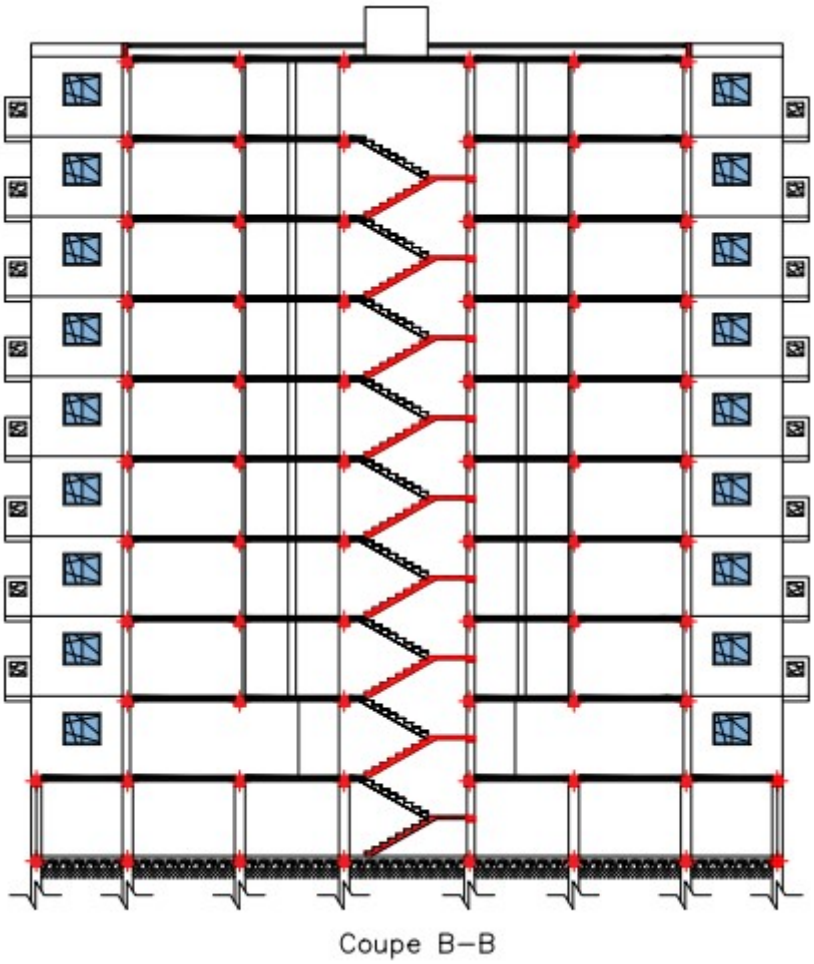


Figure II. 8 : Architectural modelisation of the building**.**

II.3. Structural conception:

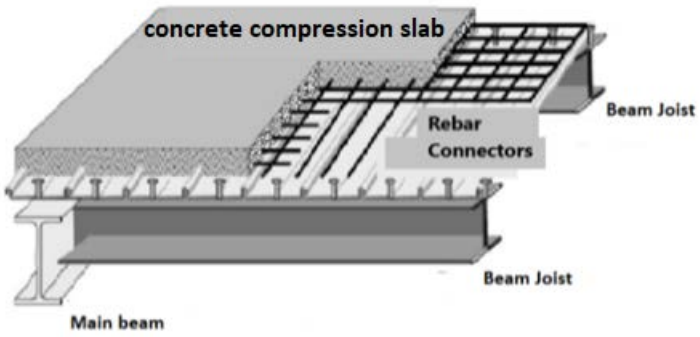
II.3.1. Horizontal structure:

It refers to the standard and terrace floors:

II.3.1.1. Standard floors:

They are composite floors suitable for use in all levels of our building. It is a light and quick alternative to implement as it allows a long range. It consists of :

- Corrugated sheet steel tray made of galvanized cold-formed profiled steel: COFRASTRA 40.
- Concrete compression slab of 10cm thickness.
- Connectors that provide the connection between the slab and the steel tank and the load-bearing structure
- It rests on metallic beams and joists.



. **Figure II. 9 :** Composition of a mixed floor

II.3.1.2. Terrace floor:

It is identical to common floors with an upper part that includes additional layers (as shown below):

- Multilayer waterproofing
- Gravel protection
- Thermal insulation.

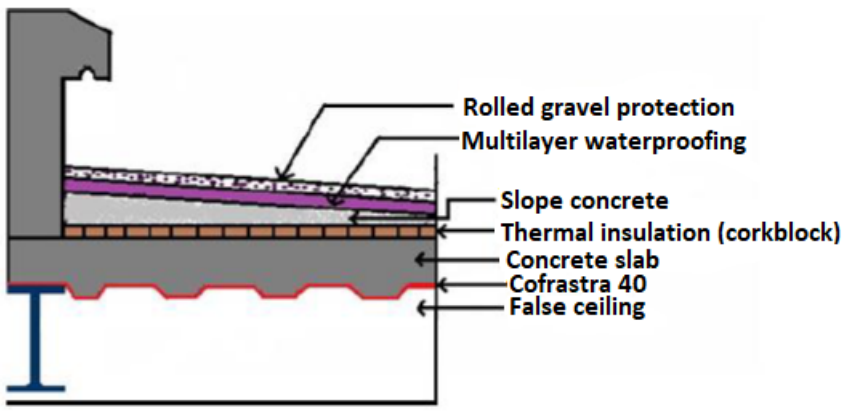


Figure II. 10 : Composition of a Terrace floor.

II.3.2. Vertical Structures:

II.3.2.1. Bracing:

To ensure the stability of the structure in the face of horizontal forces (wind forces and earthquake especially)

A X-bracing system was made available to obtain a self-stable post-beam framework.

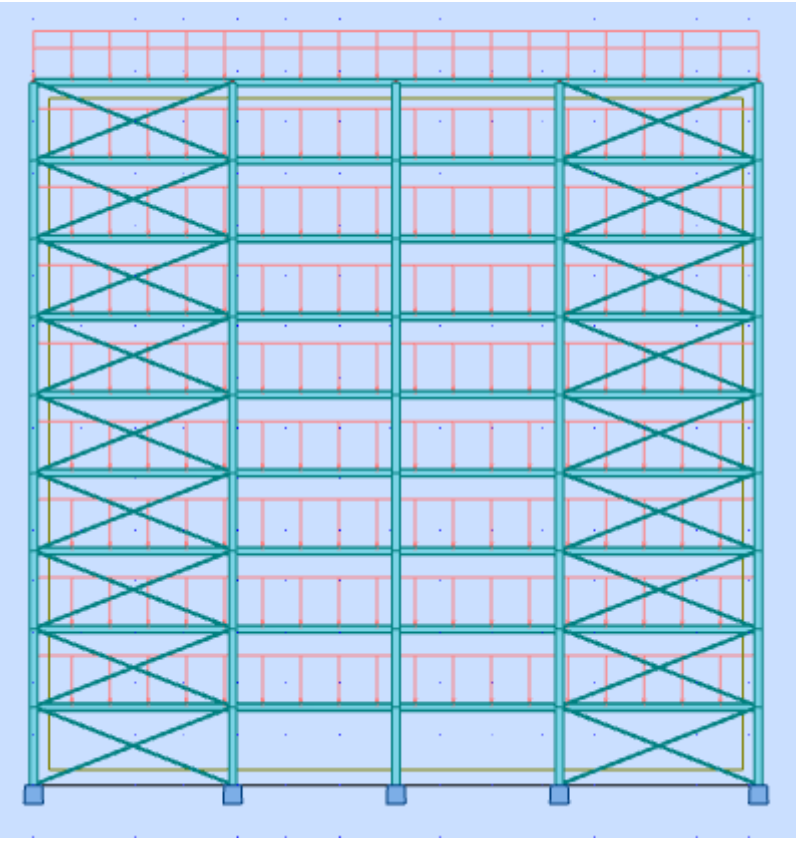


Figure II. 11 : Numerical modeling of the braced structure (facade).

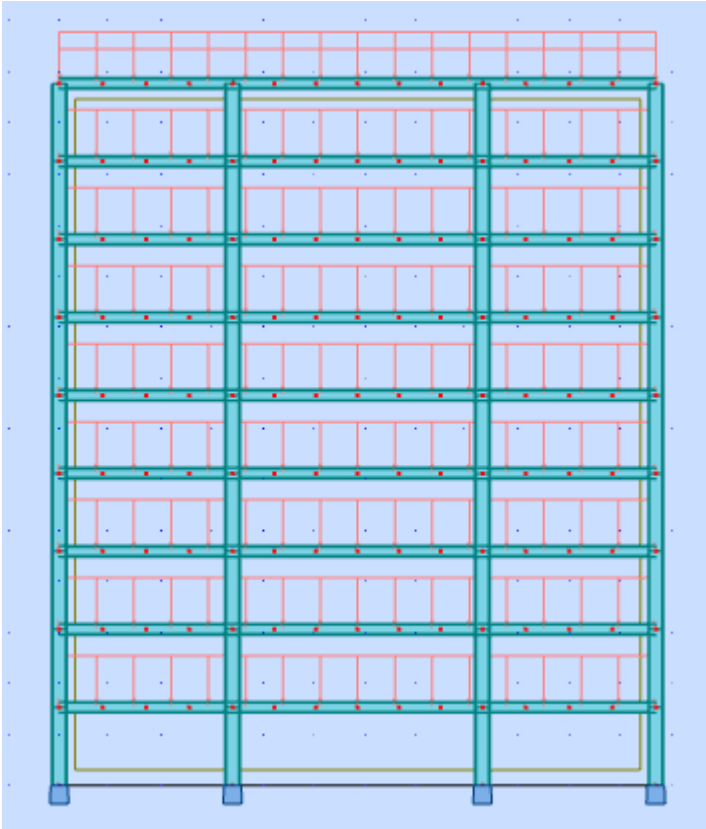


Figure II. 12 : Numerical modeling of a non-braced structure (long side).

II.3.2.2. Stairs:

Stairs are secondary structures that allow access to the different levels of the building. They are metallic structures, the metal sheet steps coated with laying mortar and tiles. The stairs of each floor are composed of 2 flights of steps carried by a stringer. The landing is made up of composite floor with composite slab.

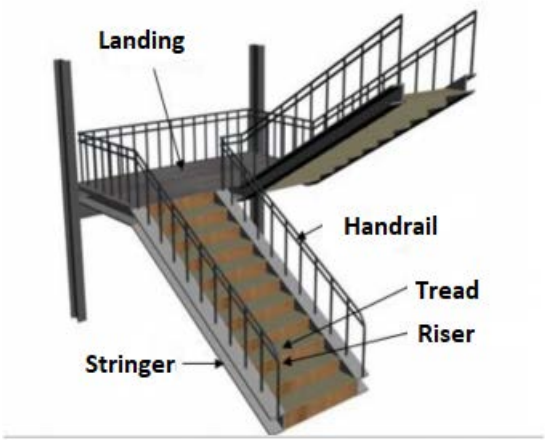


Figure II. 13 : Design of a staircase.

II.3.3. Connections:

The main connections of the structural systems used to ensure stability are:

- High resistance bolts.
- Welding whose mechanical characteristic is at least equivalent to those of the steel grades used.

II.4. Conclusion :

In this chapter we presented the components of the elements in the design of this building that have a total influence on the loads considered.

Chapter III:
Evaluation of Loads and Overloads

III.1. Introduction:

The evaluation of the Loads and Overloads applied on a structure is a fundamental step from which the dimensioning of the different elements of the structure begins.

For this study, the following charges will be considered:

- Dead Loads (Self-weight of the constituent elements)
- Live Loads (Loads due to the use of the structure)
- Climatic Actions (from snow and wind)

For this, there are norms that gives the values of the loads and are found in the **DTR BC 2.2 (charge permanent et charge d'exploitation)** Technical Regulation.

III.2. Dead Loads G:

Dead Loads are the charges which don't change with time. It is the intrinsic (self-weight) of the structure as well as permanent attachments or accessories. They are given in the **DTR BC 2.2 (charges permanents et charge d'exploitation)** Technical Regulation

III.2.1 Floors:

III.2.1.1 Inaccessible Terrace Floor

Table III. 8 : Dead load of the inaccessible terrace floor

Type	Value of Dead Load (daN/m^2)
Rolled gravel protection (5cm)	0.05x1700=85
Multilayer waterproofing (5cm)	0.05x600=30
Slope Concrete (10cm)	0.1x220=220
Thermal insulation (cork block); 4cm	0.4x400=16
Concrete slab (8cm)	0.08x2500=200
Ribbed sheet (Cofrastra 40)	11
False ceiling	10
Total	G=572daN/m^2

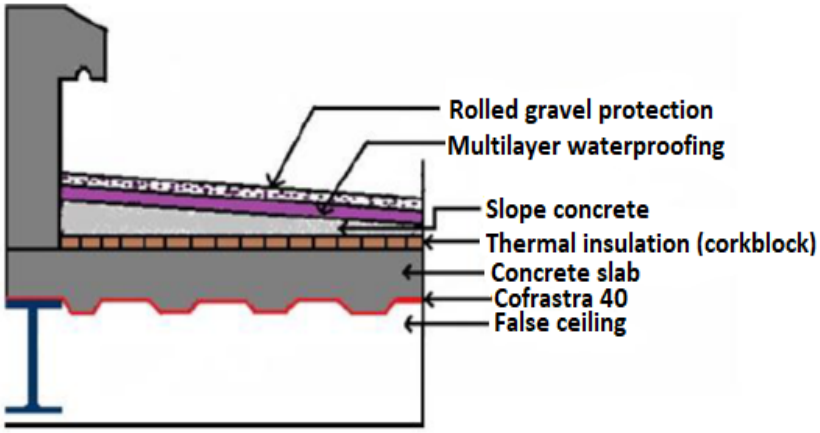


Figure III. 9 : Inaccesible terrace floor.

III.2.1.2 Standard Floors:

Table III.2: Dead load of standard floor

Type	Value of Dead Load (daN/m^2)
Tiled Covering (2cm)	0.02x20x100=40
Laying Mortar (2cm)	0.02x20x100=40
Slab weight (10cm)	0.1x2500=250
Sand bed (2cm)	0.02x1800=36
Cofrastra 40	11
False ceiling (1cm)	10
Total	G=387daN/m^2

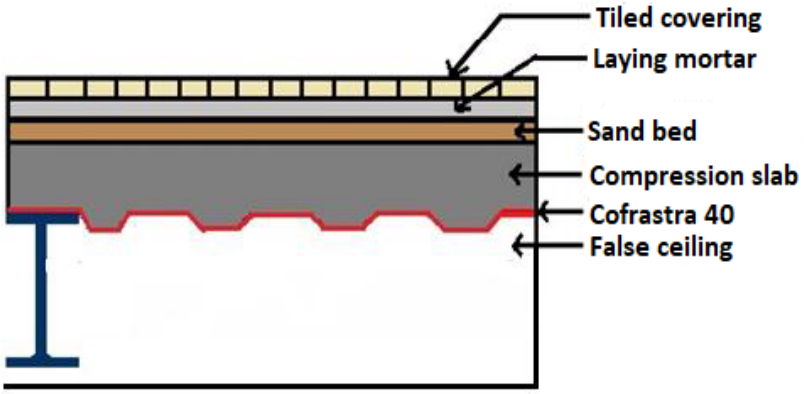


Figure III. 2: Standard floor

III.2.2. Stairs:

III.2.2.1. Stair Flight:

Table III. 3: Dead load of stair flight

Type	Value of Dead Load (daN/m^2)
Checkered sheets (thickness :5mm)	45
Laying Mortar (2cm)	40
Tiling (thickness: 2cm)	40
Total	G= 125daN/m^2

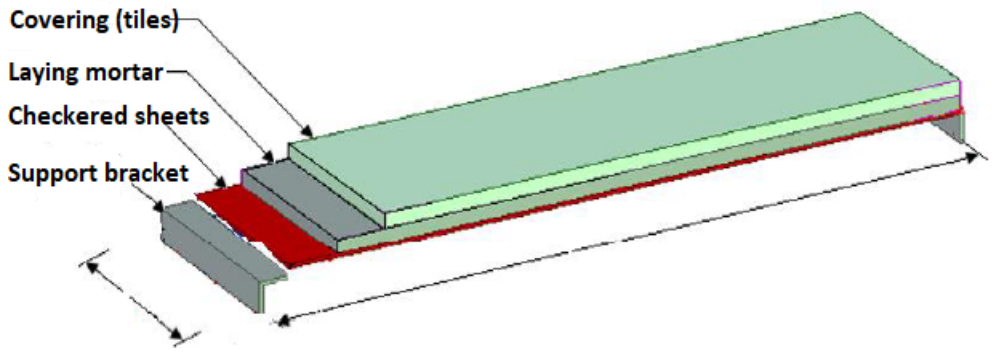


Figure III. 3: Components of a tread

III.2.2.2. Stair Landing:

Table III. 4 : Dead load of landing

Type	Value of Dead Load (daN/m^2)
Cofrastra 40	11
Concrete Slab (thickness: 8cm)	200
Laying mortar (thickness: 2cm)	40
Tiling (thickness 2cm)	40
Total	G=291daN/m^2

III.2.3. Parapet:

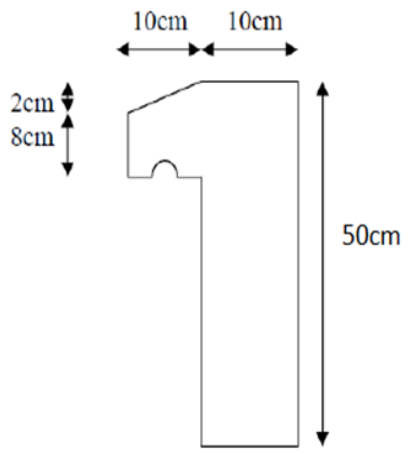


Figure III.4: Diagram of the parapet

$$Area = \frac{0.02\ x\ 0.1}{2} + (0.08x0.1) + (0.1x0.5) = 0.059m^2$$

$$Weight\ of\ the\ parapet = 0.05x1x2500 = 147.5daN/ml$$

III.3. Live Loads Q:

The live loads denoted by Q is due to the furnitures and people who live in or frequent the building. There are norms that set the values of the loads according to the intended usage of the structure which are inscribed in DTR BC 2.2.

Table III. 10 : Live Loads.

Level	Live Load (daN/m²)
Standard floors	150
Inaccessible Terrace floor	100
Stairs	250

III.4. Climatic actions:

III.4.1. Snow effect:

III.4.1.1. Introduction :

The accumulation of snow on the roof produces an overload that must be taken into account for the verification of the elements of this structure. The RNV regulation version 2013 applies

to all constructions in Algeria located at an altitude of less than 2000 meters. Our project is located at an altitude of 213m.

III.4.1.2. Calculation of the snow load:

$$S = \mu \times S_k \text{ [kN/m²]} \qquad \text{(§3.1.1. RNV99 version 2013) [1]}$$

- S_k : (en KN/m²) is the snow load on the ground, depending on the altitude and snow area.
- μ : is a load adjustment coefficient, a function of the shape of the roof, called the shape coefficient

The studied building is located in Remchi, Tlemcen which corresponds to zone A according to the classification of RNV99 with an altitude of 213m.

The value of S_K in KN/m² is determined by the following distribution of variation, as a function of the altitude H in m of the site under consideration:

$$S_K = \frac{(0{,}07 \times H) + 15}{100} \qquad \text{(§4.2RNV99) [1]}$$

Where : H=213m

S_K=0.2991KN/m^2

The structure has a flat roof i.e., a slope of

$\mathbf{0 \leq \alpha \leq 30°}$ according to table III.6 (c.f.6.2.1.[1]), the coefficient of shape of the roofs μ=0.8.

Table III. 11 : Shape coefficient– one-sided roof

(α) Angle of the slope to the horizontal (en °)	**0 ≤ α ≤ 30 °**	**30 ° ≤ α ≤ 60 °**	**α ≥ 60 °**
Coefficient μ	0.8	$0{,}8\left(\frac{60-\alpha}{30}\right)$	0.0

$$S = \mu x S_k = 0.8x0.2991$$

$$S = 0.2392 KN/m^2$$

III.4.2. Wind effect:

III.4.2.1. Introduction :

The effect of the wind on a construction is quite preponderant and has a great influence on the stability of the structure, because of this, an in-depth study must be done for the determination of the different actions due to the wind and this should be done for all possible directions. The calculation will be carried out in accordance with the Snow and Wind Regulation. This regulatory technical document (DTR) provides the procedures and general principles for the determination of wind actions over the whole construction and its different parts and applies to constructions whose height is less than 200m.

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

- The direction.
- The intensity.
- The region.
- The site of implementation of the structure and the surrounding environment.
- The geometric shape and openings of the structure.

The calculation must be carried out separately for the two wind directions, which are Perpendicular to the walls of the construction (Figure III.5).

- The V1 direction of the wind: perpendicular to the main facade.
- The direction V1' of the wind: perpendicular to the main façade but the opposite side of V1.
- The V2 direction of the wind: parallel to the main facade.
- The direction V2' of the wind: parallel to the main facade but the opposite side of V2.

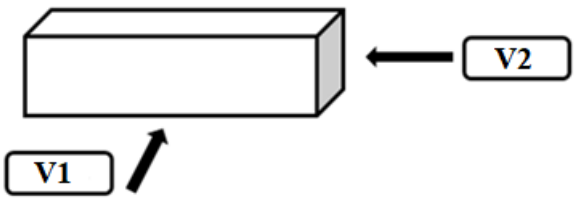


Figure III. 6 : Wind on the sides of the structure.

III.4.2.2. Data relating to the site in accordance with the rules of RNV99 [1]:

✓ **Zone de vent III :**

$q_{réf}$ = 435 N/m² (cf.chapter2, table2.[1])

✓ **Terrain Category: I**

K_T = 0,215 (field factor) (cf.Table 2.4.[1])

z_0 = 0,3m (roughness parameter) (cf.Table 2.4.[1])

z_{min} =5m (minimum heights). (cf.Table 2.4.[1])

ε = 0,61 (cf.Table 2.4.[1])

III.4.2.3. Determination of aerodynamic pressure W (zj):

$W(z_j) = C_d \times q_p(z_e) \times (C_{pe} - C_{pi})$ [N/m²] (cf.§2.5.2[1])

➢ **Reference height z_e:**

According to the RNV99 version 2003 (see chapter 2 § 2.3.2.[1]).

For vertical walls z_e is determined according to the maximum height h of the building, and the dimension of the building measured in the direction perpendicular to the direction of the wind.

- For the walls in the face of the wind and parallel to the wind, ze is taken equal to the maximum height of the building.
- For the roofs ze is taken equal to the maximum height of the building.

Table III.12 : Reference height.

Wind direction	h(m)	b(m)	d(m)
V_1 and V'_1	28.04	23.4	28.4
V_2 and V'_2	28.04	28.4	23.4

For V1, b≤h≤2b and for V2, h≤b→ze=h

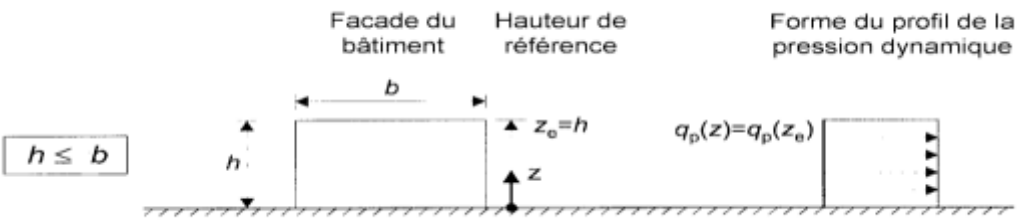


Figure III. 7 : Subdivision of the building for defining the reference height

➢ Determination of dynamic pressure q_p:

The dynamic pressure $\mathbf{q_p}$ (z_e) exerted on a surface element j is given by:

$q_p(z_e)=q_{réf}\times C_e(z)$ (cf.§2.3.1[2])

C_e : wind exposure coefficient.

➢ Determination of exposure coefficient Ce(z):

In the case where the structure is not very sensitive to dynamic excitations the exposure coefficient is given by the formula below:

$$\mathbf{C_e(z)=C_t^2(z) \times C_r^2(z) \times [1+7\ Iv(z)]} \qquad \text{(cf.§2.4.2[1])}$$

For V1 and V2

Where:

Ct(z): topography coefficient.

Cr(z): roughness coefficient.

K_T: field factor.

I_v : turbulence intensity.

➢ Roughness coefficient:

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

$$\begin{cases} C_r(z) = K_T \times ln\left(\frac{z}{z_0}\right) & For: z_{min} \leq z \leq 200 \\ C_r(z) = K_T \times ln\left(\frac{z_{min}}{z_0}\right) & For: z \leq z_{min} \end{cases} \text{(cf.§2.4.4[1])}$$

➢ Topographie coefficient :

The topography coefficient takes into account the increase in wind speed when it blows over obstacles such as hills, isolated elevations, etc.

$C_t(Z) = 1$ For: θ≤0.05

$C_t(Z) = 1 + S_{max} \times \left(1 - \frac{|X|}{K_{red}}\right) \times e - \alpha\left(\frac{\theta}{L}\right)$ For: θ≥0.05 (cf.§2.4.5[1])

- **Flat site** : topography coefficient Ct=1.

➢ Turbulence intensity coefficient:

The turbulence intensity coeffcient is defined as the standard deviation of turbulence as divided by the mean wind speed and is given by the equation:

$$I_v = \frac{1}{C_r(z) \times ln\left(\frac{z}{z_0}\right)}$$ for $z \geq z_{min}$

$$I_v = \frac{1}{C_r(z) \times ln\left(\frac{z_{min}}{z_0}\right)}$$ for $z \leq z_{min}$ (cf.§2.4.6[1])

The results are shown in the following table:

Table III. 13 : Dynamic pressure values.

H(m)	$C_t(z)$	$C_r(z)$	Iv(z)	Ce(z)	q_p (daN/m^2)
12,14	1	0.975	0,220	2,414	105.009

➢ Calculation of Pressure Coefficients :

- **External Pressure coessficient C_{Pe} :**

The external pressure coefficient depends on the geometric form of the base of the structure (rectangular in our case) and the dimension(area) of the charged surface.

Where:

b: dimension perpendicular to the wind direction.

d: the dimension parallel to the wind direction.

For the **C_{Pe}** values, we refer to (cf. chapter5.§1.1.2.[1]) :

$C_{pe} = C_{pe.1}$ if: $S \leq 1$ m

$C_{pe} = C_{pe.1} + (C_{pe.10} + C_{pe.1}) \times \log10(S)$ if: 1 m^2 < S < 10 m^2

$Cpe = Cpe.10$ if: $S \geq 10$ m

Where S represents the charged area of the wall under consideration.

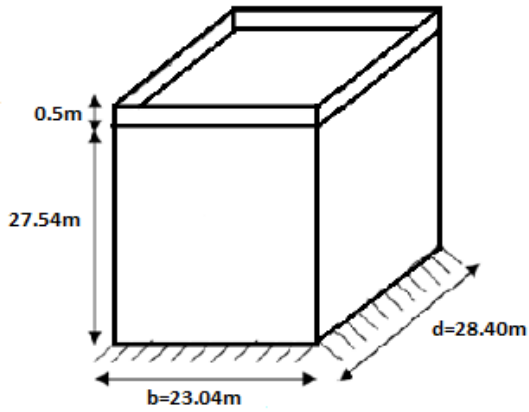


Figure III. 8 : Simplified model of the structure

In our case**: S ≥ 10 m2**

Therefore: Cpe = $\mathbf{C_{pe.10}}$

Wind direction V_1 (perpendicular to the facade) and V_1':

For the V_1 et V_1' wind directions, the wind pressure coefficients are as follows:

(Cf. tableau7.[1])

For vertical walls:

We divide the wall as indicated in figure 5.1 (cf. figure5.1.[1]).

e = min [b ; 2h]

b = 23.04m

d = 28.40 m alors : e = min [23.04 **;** 56.08]

h = 28.04m e= 23.04m

d>e: therefore the wall is divided in 3 pressure zones A, B, C, D et E which are illustrated in (Figure III.8) below:

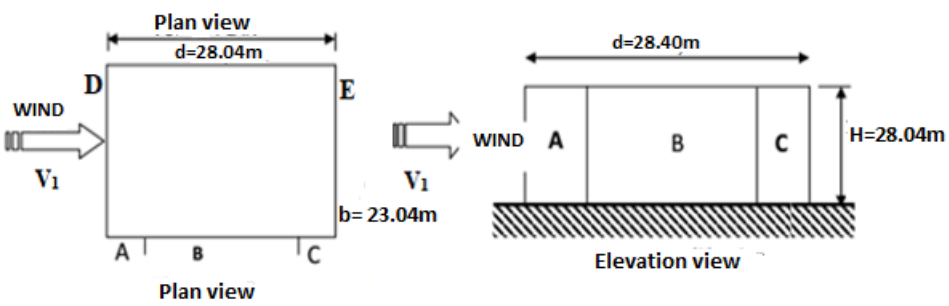


Figure III. 9 : Vertical walls for the V1 direction.

Table III.14 : Cpe values for the vertical walls of buildings with rectangular base for V1 direction (cf.Tableau.5.1 [1])

A	B	C	D	E
$C_{pe,10}$	$C_{pe,10}$	$C_{pe,10}$	$C_{pe,10}$	$C_{pe,10}$
-1.0	-0.8	-0.5	+0.8	-0.3

Wind direction V_2 (perpendicular to the facade) and V_2':

We divide the wall as indicated in figure 5.1 (cf. figure5.1.[1]).

e = min [b ; 2h]

b = 28.40m

d = 23.04 m alors : e = min [28.04 **;** 56.08]

h = 28.04m e= 28.04m

d<e: therefor the wall is divided in 2 pressure zones A and B which are illustrated in the table below:

Table III.10 : Cpe values for the vertical walls of buildings with rectangular base for V2 direction (cf.Tableau.5.1 [1])

A'	B'	C	D
$C_{pe,10}$	$C_{pe,10}$	$C_{pe,10}$	$C_{pe,10}$
-1.0	-0.8	+0.8	-0.3

- **Coefficient of interior pressure Cpi :**

The coefficient of interior pressure Cpi takes into consideration the influence of the openings on the wind load applied on the structure. It is function of the indice of permeability μp and the ratio h/d.

Sens V1 :

$$\mu = \frac{36.8+71.06+71.06}{(36.8\times2)+(71.06\times2)}$$

μ= 0.829

h/d= 28.04/28.40=1.14>1 (cf.§5.2.2.2[1])

The openings considered here are the orifices of all kinds leading to the outside and through which air can circulate.

Cp_i= -0.35

Sens V2:

$$\mu = \frac{71.06+36.8+36.8}{(71.06\times2)+(36.8\times2)}$$

μ= 0.670

h/d= 28.04/23.04=1.21>1

Cp_i= -0.13

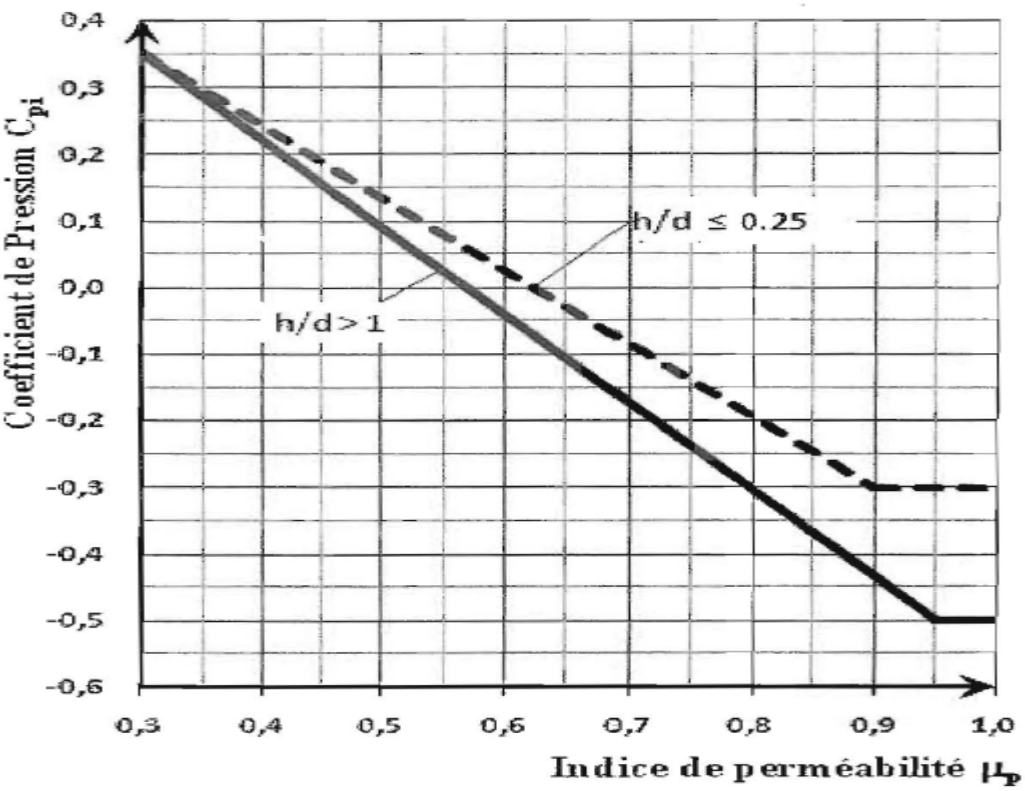


Figure III. 10 : Interior pressure coefficients Cpi of buildings without a dominant face. (see figure.5.14.[1]).

From the tabove we find the following values of C_{pi} :

Table III.11 : Détermination du coefficient de pression interne Cpi (V1 et V'1).

Side	Surface area of openings (m²)	μ	C_{pi}
V1	36.8	0.829	-0.35
V 1'	36.8	0.829	-0.35
V2	36.8	0.670	-0.13
V2'	36.8	0.670	-0.13

- **Determination of the dynamique coeffcient Cd**

$$Cd = \frac{1 + 2 \times g \times Iv(zeq) \times \sqrt{Q^2 + R^2}}{1 + 7 \times Iv(zeq)}$$

According to RNV.v.2013 (cf.3.2 [1]) , Zeq=0.6h ≥ zmin for the calculation of Cd

Turbulence scale:

$Li(zeq) = 300x\left(\frac{z}{200}\right)^{\varepsilon}$ ……. (zmin<z<200m)

$$Li(zeq) = 300 \times (\frac{16.824}{200})^{0.61} = 66.268$$

Where :

Zeq=0.6h≥ zmin

Zeq=0.6x28.04=16.824m

Quasistatic part:

$$Q^2 = \frac{1}{1+0.9\times(\frac{b+h}{Li(zeq)})^{0.63}} = \frac{1}{1+0.9\times(\frac{23.04+28.40}{66.268})^{0.63}} = 00.565$$

Average speed

Vm=Cr(z)xCt(z)xVref=0.975x1x27=26.32m/s

Admissional frequency

$$Nx = \frac{n1x \times Li(zeq)}{Vm(zeq)}$$

$$n1x = \frac{46}{h}$$

$$n1x = \frac{46}{h} = \frac{46}{28.04} = 1.640Hz$$

$$Nx = \frac{1.640 \times 66.268}{26.32} \qquad Nx = 4.129$$

Admission function RN

$$RN = \frac{6.8 \times Nx}{(1 + 10.2 \times Nx)^{\frac{5}{3}}} = \frac{6.8 \times 4.129}{(1 + (10.2 \times 4.129))^{\frac{5}{3}}} = 0.0529$$

Aerodynamic admittance function Rh and Rb

$$\eta h = \frac{4.6 \times Nx \times h}{Li(zeq)} = \frac{4.6 \times 4.129 \times 28.04}{66.268} = 8.036$$

$$\eta b = \frac{4.6 \times Nx \times b}{Li(zeq)} = \frac{4.6 \times 4.129 \times 23.04}{66.268} = 6.603$$

$$Rh = \left(\frac{1}{\eta h}\right) - \left(\frac{1}{2 \times \eta^2 h}\right) \times (1 - e^{-2 \times \eta h}) = 0.116$$

$$Rb = \left(\frac{1}{\eta b}\right) - \left(\frac{1}{2 \times \eta^2 b}\right) \times (1 - e^{-2 \times \eta b}) = 0.139$$

Log decrement of depreciation

δ = δs +δa

δs = 0.05 (table 3.1)

δa=0

where

δ =0.05

Resonant part:

$$R^2 = \frac{\pi^2}{2 \times \delta} \times RN \times Rh \times Rb$$

$$R^2 = \frac{\pi^2}{2 \times 0.05} \times 0.0529 \times 0.116 \times 0.139$$

$$R^2 = 0.0841$$

Average frequency

$$\mathrm{V} = \mathrm{n1x} \times \sqrt{\frac{R^2}{Q^2 + R^2}} \geq 0.8$$

$$V = 1.640 \times \sqrt{\frac{0.0841}{0.565 + 0.0841}}$$

$$V = 0.590HZ \ > 0.8$$

Peak factor

$$g = \sqrt{2 \times Ln(600 \times v)} + \frac{0.6}{\sqrt{2} \times Ln(600 \times v)} \geq 3$$

$$g = \sqrt{2 \times Ln(600 \times 0.590)} + \frac{0.6}{\sqrt{2} \times Ln(600 \times 0.590)}$$

G=3.601>3

Turbulence intensity

$$Iv(zeq) = \frac{1}{Ct(z) \times Ln\left(\frac{z}{z0}\right)} = \frac{1}{1 \times Ln\left(\frac{16.824}{0.3}\right)} = 0.248$$

Therefore :

$$Cd = \frac{1 + 2 \times 3.601 \times 0.248 \times \sqrt{0.565 + 0.0841}}{1 + 7 \times 0.248}$$

Cd=0.891

The results of the aerodynamic pressures $W(z_j)$ are given in the following table:

Table III.12 : Aerodynamic pressures for **V1 and V1'**

Zone	c_d	q_p(N/m^2)	C_{pe}	C_{pi}	C_{pe}-C_{pi}	W (z_j) (N/m^2)
A	0.891	105.009	-1.0	-0.35	-0.65	-60.81
B	0.891	105.009	-0.8	-0.35	-0.45	-42.10
C	0.891	105.009	-0.5	-0.35	-0.15	-14.03
D	0.891	105.009	+0.8	-0.35	+1.15	+107.59
E	0.891	105.009	-0.3	-0.35	+0.05	+4.67

Table III.13 : Aerodynamic pressures for **V2 and V2'**

Zone	c_d	q_p(N/m^2)	C_{pe}	VC_{pi}	C_{pe}-C_{pi}	W (z_j) (N/m^2)
A'	0.891	105.009	-1.0	-0.13	-0,87	-81.39
B'	0.891	105.009	-0.8	-0.13	-0.67	-62.68
D	0.891	105.009	+0.8	-0.13	+0.93	+87.01
E	0.891	105.009	-0.3	-0.13	-0.17	-15.90

III.5. Conclusion:

In this chapter we calculated the Dead Loads, Live Loads and also loads due to climatic actions on the studied structure, we also used the general rules and procedures for the calculation of the loads.

The results that we obtained are necessary for the sizing of the different elements of our structure.

Chapter 4:
Pre-sizing of Elements

IV.1. Introduction:

Each structure must withstand the different loads and overloads that are applied to it, thus the pre-sizing step is essential.

In this chapter, we will therefore proceed to the pre-sizing and calculation of the secondary elements and the load-bearing elements (beams and columns), the latter is done according to CCM97 [3]. It is noted that the goal is to find a better compromise between cost and safety.

IV.2. Some notions related to the CCM97 regulation:

IV.2.1. Objective:

The CCM97 regulation's [3] objective is the codification of the sizing by calculation and verification of the structures of steel-frame buildings.

IV.2.2. Scope:

This document contains principles, rules and commentaries applicable primarily to common buildings that respect the limits imposed in the sections below.

Common buildings are by convention those in which Live Loads are moderate (buildings for residential or accommodation use, for office use, school and hospital buildings, buildings for commercial use such as shops).

The structures are made from hot-rolled steel construction products with the exception of high-strength steel grades.

IV.2.3. Classification of cross-sections according to CCM97:

For calculation purposes CCM97 has proposed four classes of cross-sections which are defined as follows (see § 5.3.2 [3]):

- Class 1: Cross-sections that can form a plastic ball joint with the rotational capacity required for plastic analysis.
- Class 2: Cross-sections that can develop their plastic moment resistance, but with a limited rotational capacity.
- Class 3: Cross-sections whose stress calculated in the compressed extreme fibre of the steel element may reach the yield strength, but whose local buckling is likely to prevent the development of the moment of plastic resistance.

- Class 4: Cross-sections whose resistance to the bending moment or to the compression has to be determined while taking into consideration the effects of local buckling explicitly.

IV.2.4. Partial safety coefficient:

The partial safety coefficient for materials shall be taken equal to the following values according to (see § 5.0.1 [3]):

- Class section (1, 2,3) → γ_{M0} =1,1.
- Class section (4) → γ_{M1} =1,1.
- Clean sections to the right of the holes → γ_{M2} =1 ,25.
- Case of the ultimate limit states of the elements → γ_{M1} =1.1.

IV.2.5. Limit values of deflections:

Steel structures must be dimensioned in such a way that the deflection remain within the limits appropriate to the intended use and occupation of the building and the nature of the filling materials to be supported. The recommended limit values for vertical deflections are given in the table below (c.f.Table 4.1 [3]).

Table IV. 5 : Recommended limit values for vertical deflections.

Conditions δ_{Vmax}	Deflection in the final state
Roofs in general	L/200
Roof supporting personnel other than maintenance personnel	L/250
Floors in general	L/250
Floors and roofs supporting partitions made of plaster or other fragile or rigid materials	L/250
Floors supporting the columns (unless the deflection has been included in the overall analysis of the ultimate limit state	L/400
Case where δ_{Vmax} may affect the appearance of the building	L/250

IV.3. Presizing:

IV.3.1. Joists:

Joists are beams usually made of IPE or IPN which work under simple flexion and rest on the main beams, the distance between joists is practically determined by the following equation: **0,7 m ≤ L ≤ 1,5 m.**

IV.3.1.1. Pre-sizing of the joists of the terrace floor:

- Length of the joists……L=7.8 m.
- Distance between the joists……e=1,5 m.
- S=7.8x1.5=11.7m^2.
- The joist is solicited by the following loads and overloads:

G=572 kg/m^2.

Q = 100 kg /m^2.

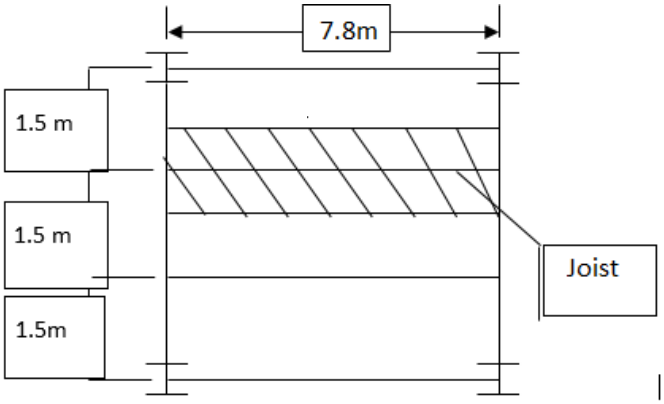


Figure IV. 4 : Joists diagram.

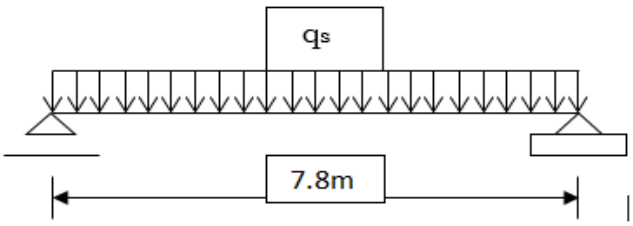


Figure IV. 5 : Static diagram of the most stressed joist.

Table IV. 6 : Load on the joist.

Types of Load	Load Value
Dead load of the floor	G= 5, 72 KN/m^2
Live Load of the floor	Q= 1 KN/ m^2

ELS :

➢ Load combinations :

The calculation is done in l'ELS, for this we take the unweighted charges:

Load on the joist (unweighted load):

q_s= (G+Q)× distance between the joists

q_s= (5,72+1)×1,5

q_s =10,08 KN/m

➢ Deflection condition :

The value of the maximum deflection is:

$$f^{max} = \frac{5}{384}\frac{q_s.L^4}{E\,I_y}$$

The value of the allowable deflection is:

$$\bar{f} = \frac{L}{250}$$

In the case of a bi-articulated beam the formula for calculating the deflection for pre-sizing is:

$$f^{max} = \frac{5}{384}\frac{q_s.l^4}{E\,I_y} \leq \bar{f} = \frac{l}{250}$$

where:

l : length of the most stressed joist 7.8 m.

So:

$I_y = \frac{5\times250\times q\times l^3}{384E}$ → Iy≥ 7414x10^{4}mm^4 We chose **IPE 300.**

IV.3.1.2. Pre-sizing of the joists of the Standard floor:

Since the load q_s=(1,5+3,87)x1.5=8.055 KN/m^2 and the length of the joists of the standard floors are the same as that of the terrace floor, we will use the same pre sizing of the terrace floor for the standard floors.

IV.3.2. Presizing of the Principal Beams :

IV.3.2.1. Presizing of the inaccessible main beams :

- Distance between the joists: 1.5 m.
- Length of the beam: 9.8m.
- Distance between the beams: 7.1m.

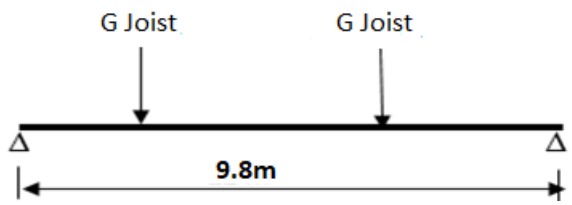


Figure IV. 6 : Static diagram of the main beam.

The loads that are supported by the principal beams and the inaccesible terrace floor are as follows:

Table IV. 7 : Load of the main beam.

Load type	Load
Dead load of floor	G=5,72 kN /m^2
Live load of floor	Q=1 KN/m^2
Self weight of the joist	G=0,422 kN/m

ELS :

Load combinations :

$q_{ELS\ floor}$ =(G_{floor} +Q_{floor}) ×distance between the beams =(5,72 +1)×7.1= 47.71KN/m

$q_{ELS.joist}$= G_{joist}× length of joist × number of joist =0,422×7.1× (9.8/1.5)=19.57 KN/m

$q_{ELS\ beam} = q_{ELS\ floor} + q_{ELS\ joist}$

$q_{ELS\ beam}$ =67.28KN/m

Deflection condition :

The maximum value of the deflection is :

$$f_q + f_p = \frac{5q_{ELS.plancher}.L^4}{384E\ I_{y.poutre}} + \frac{q_{ELSs.solive}.l^3}{76{,}8\times E\times I_{y.solive}}$$

The value of the allowable deflection is:

$$\bar{f} = \frac{L}{250}$$

In the case of a bi-articulated beam the formula for calculating the deflection for pre-sizing is:

$$f^{max} \leq \bar{f}$$

$$I_y \geq \frac{250}{L}\left(\frac{5\times q_{ELS.floor}\times L^4}{384E} + \frac{q_{ELS.joist}\times L^3}{76.8\ E}\right)$$

$$I_y \geq \frac{250}{9.8}\left(\frac{5\times 47.71\times 9.8^4}{384\times 2.1\times 10^8} + \frac{19.57\times 7.8^3}{76.8\ \times 2.1\times 10^8}\right)$$

$I_y \geq 7.10750\times 10^{-4}m^4 = 71075.0\times 10^4 mm^4$→We choose **HEB 450.**

IV.3.2.2. Presizing of the Standard floor beams:

- Distance between the solives: 1.5 m.
- Length of the beam: 9.8m.
- Distance between the beams: 7.1m.

The loads that are supported by the principal beams and the inaccesible terrace floor are as follows:

Table IV. 4 : Loads of the standard main beams.

Load type	Load
Dead load of floor	G=3.87KN /m^2
Live load of floor	Q=1.50 KN/m^2
Self-weight of the joist	G=0,422 kN/m

- **ELS:**

- **Load combinations :**

$q_{ELS\ floor}$ =(G_{floor} +Q_{floor}) ×entre axe des poutres =(3.87+1.5)×7.1= 38.127KN/m

$q_{ELS.joist}$= G_{joist}× length of joist × number of joist = $0.422 \times 7.1 \times \frac{9.8}{1.5} = 19.57 KN/m$

$q_{ELS\ beam} = q_{ELS\ floor} + q_{ELS\ joist}$

$q_{ELS\ beam}$ =57.69KN/m

- **Deflection condition :**

The maximum value of the deflection is :

$$f_q + f_p = \frac{5 q_{ELS.plancher} . L^4}{384 E\ I_{y.poutre}} + \frac{q_{ELSs.solive} . l^3}{76{,}8 \times E \times I_{y.solive}}$$

The value of the allowable deflection is:

$$\bar{f} = \frac{L}{250}$$

In the case of a bi-articulated beam the formula for calculating the deflection for pre-sizing is:

$$f^{max} \leq \bar{f}$$

$$I_y \geq \frac{250}{L}\left(\frac{5 \times q_{ELS.floor} \times L^4}{384E} + \frac{q_{ELS.joist} \times L^3}{76.8\,E}\right)$$

$$I_y \geq \frac{250}{9.8}\left(\frac{5 \times 38.12 \times 9.8^4}{384 \times 2.1 \times 10^8} + \frac{19.57 \times 7.8^3}{76.8 \times 2.1 \times 10^8}\right)$$

$I_y \geq 5.70837 \times 10^{-4} m^4 = 5.7083.70 \times 10^4 mm^4$→We choose **HEB 450.**

IV.3.3. Presizing of Columns:

The Columns are vertical elements which transfer the exterieur loads coming from the dead, Climatic and Live Loads to the foundation.

We commence by choosing the most solicited column and consider all it's dead and live loads. The height of the column is h=3.06m and it supports an area of 58.93m^2

The Compression effort the column is solicited by:

l'E.L.U :

N_{sd} = 1,35 G + 1,5Q

G: Self-weight of the elements that solicits the studied column with self-weight not included.

Q: Live Load

The loads applied to the column are shown in the table below:

Table IV. 8 : Loads supported by the column

Type of charges (KN)	Terrace	Standard floor
Floor Dead Load G	5.72	5,72
Beam Dead Load G	1.71	1.71
Joist Dead Load G	0.422	0,422
Live Load Q	1	1,50

N_{sd}=1.35 [(G $_{\text{terrace floor}}$ + G $_{\text{terrace joist}}$ +G $_{\text{terrace beam}}$) + 8×(G $_{\text{standard floor}}$ + G $_{\text{standard floor joist}}$ +$G_{\text{standard floor beam}}$) +1.5($Q_{\text{terrace}}$+8($Q_{\text{standard floor}}$] x surface area supported by the column

N_{sd} = 5593.6945 KN

The columns are elements sollicited by axial compression, the calculation value Nsd of the compression force in each cross-section must satisfy the condition:

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

$$N_{pl,Rd} \equiv \frac{A \times f_y}{\gamma_{M0}}$$

A≥261.83x10^2mm^2 We choose **HEB 600.**

IV.4. Conclusion :

This chapter allowed us to initially determine the sections of the structural elements, to be able to model our building.

Chapter 5:
Structural Analysis

V.1. Introduction:

Modeling is the establishment of a model from the actual structure. This will be followed by some modifications, in order to approximate the behavior of the model as close as possible to that of the original structure.

Several types of approaches are possible to be able to appreciate the behavior of the model:

- Finite element modeling.
- Concentrated mass and equivalent stiffness modelling.

In our project, the finite element approach using ROBOT software is adopted.

V.2. Robot Software Overview:

The ROBOT software, one of the products of ROBOT BAT, is a software for calculating and optimizing structures. It is based on the finite element analysis method to study flat and spatial structures such as lattice, gantries suitable for steel, wood, reinforced concrete or mixed concrete construction. It includes modules for sizing, verification, assembly, and reinforcement according to the various existing national standards.

V.3. Principal phase of modeling by ROBOT:

- Choice of units (length and force) (KN/m) of the calculation regulations.
- Defining the basic geometry.
- Defining the different sections of the structure (column, beam,.. etc.).
- Design the different elements.
- Defining the different types of supports.
- Defining the type of floor and its width
- Defining the different loads (Dead load, Live load, Wind load etc.) and applying them.
- Modal Analysis
- Defining the different load combinations.
- Start calculation.
- Open the file result of checking the displacements and the period of the structure.
- Find the solicitations (M, N, V) to in order to verify the elements.

V.4. The results given by software:

After modeling our structure, the results in terms of stresses (M, N, V) and displacements are given by the tables below :

V.4.1. Principal beams:

V.4.1.1. Standard and terrace floor principal beams:

The results given by the ROBOT of the different solicitations is shown in the figure below.

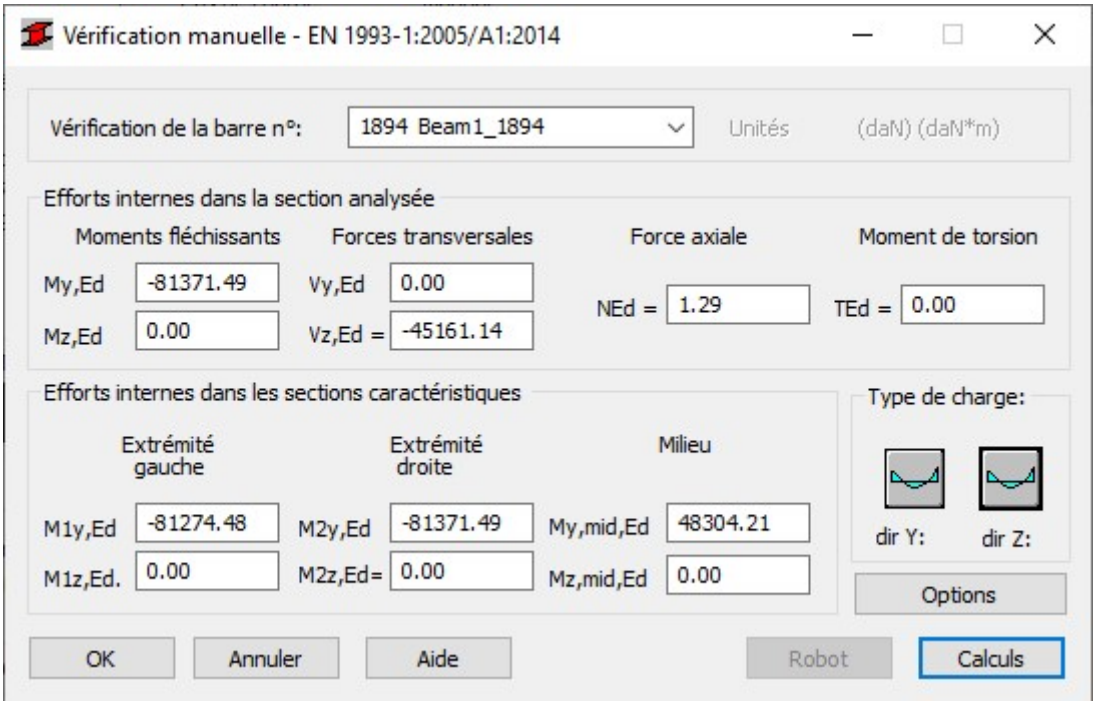


Figure V. 4 : M N T forces of the standard floor main beams.

V.4.2. Columns: The results given by the ROBOT of different solicitations are represented in the figure below

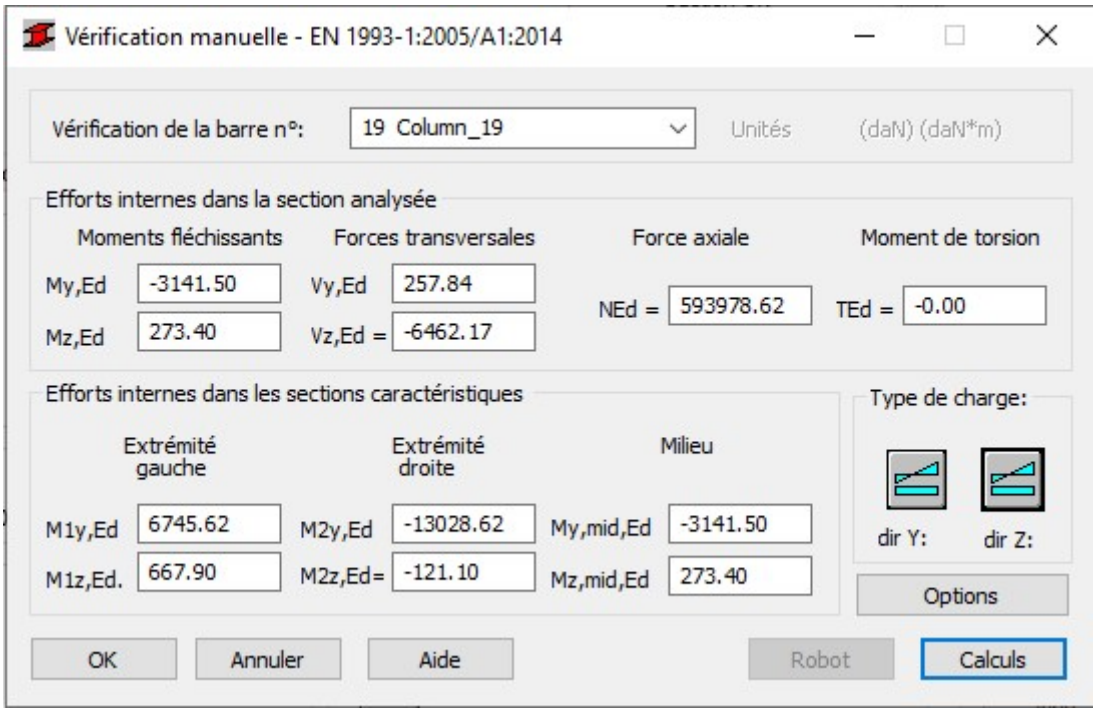


Figure V. 5 : M N T forces of the column.

V.4.3. Bracing: The results given by the ROBOT of different stresses shown in the figure below.

	FX (daN)
MAX	26940.75
Bar	5
Node	8
Case	30 (C) (CQC)
Mode	
MIN	-19698.89
Bar	11
Node	979
Case	24 (C) (CQC)
Mode	

Figure V. 6 : Stresses on the bracing.

V.5. Conclusion : In this chapter, we used the ROBOT software to model, calculate, verify and size different elements of our structure. This last step allows us to define calculation values used in the following chapters.

Chapter 6:
Seismic Study

VI.1. Introduction:

An earthquake is a sudden release of the potential energy accumulated in the rocks because of the relative movements of different parts of the Earth's crust. When the stresses exceed a certain threshold, an equilibrium break occurs and gives rise to seismic waves that propagate in all directions and reach the surface of the ground.

The vibrations of the ground (horizontal and vertical) caused by the passage of seismic waves drive the constructions in their movements.

The displacements at the base of the construction caused by the latter generate inertial forces that are proportional to the weight of the construction.

It is therefore necessary to build earthquake-resistant structures in seismic zones.

The evolution of seismic action is based on the new version of the Algerian seismic regulation, the RPA99 version~~s~~ 2003.

This document proposes calculation methods and seismic recommendations. Its application to metal construction is one of the essential objectives of our end-of-study project.

According to the RPA99 version 2003 regulation [2], the determination of the response of a structure and its dimensioning can be done by three calculation methods:

- Equivalent Static analysis method
- Spectral analysis method
- Dynamic analysis method by accelerograph.

VI.2. Classification criteria by RPA99 version 2003:

VI.2.1. Classification of seismic zones:

The national territory is divided into four zones of increasing seismicity, defined on the map of seismicity zones and the associated table which specifies this distribution by wilaya and by municipality.

- ZONE 0 : negligible seismicity.
- ZONE I : low seismicity.
- ZONE II_a et II_b : medium seismicity.
- ZONE III : high seismicity.

VI.2.2. Classification of the building:

For our case, and according to the map and table cited above: Remchi is located in a zone of low seismicity; Zone I

The classification of buildings is also done based on the criterion of the importance of the structure in relation to the safety, economic and social level.

- Group 1A: structures of vital importance.
- Group 1B: structures of great importance.
- Group 2: common structures or those of medium importance.
- Group 3: structures of low importance.

Our structure is a building intended for use as accommodation, hence it is considered as a structure of medium importance: Group 2.

VI.3. Equivalent Static analysis Method:

It consists of replacing the real dynamic forces that develop within the structure with a system of fictitious static forces whose effects are assumed to be equivalent to those of the seismic action.

VI.3.1. Conditions for the application of the equivalent static method

The equivalent static method may be used under the following conditions:

a) The building or block studied, which meets the conditions of regularity in plan and elevation prescribed in Chapter III, paragraph 3.5 with a height not exceeding 65m in zones I and II and 30m in zones III.

b) The building or block studied has an irregular configuration while respecting, in addition to the height conditions set out in a), the following additional conditions:

- **Zone I** : All groups.
- **Zone II** : Groupe d'usage 3.

  Usage group 2, if the height is less than or equal to 7 floors or 23m.

  Usage group 1B, if the height is less than or equal to 5 floors or 17m.

  Usage group 1A, if the height is less than or equal to 3 floors or 10m.
- **Zone III** :

  Usage group 3 and 2, if the height is less than or equal to 5 floors or 17m.

  Usage group 1B if the height is less than or equal to 3 floors or 10m.

  Usage group 1A if the height is less than or equal to 2 floors or 8m.

VI.3.2. Concept:

The real dynamic forces that develop in construction are replaced by a system of fictitious static forces whose effects are considered equivalent to those of seismic action.

The movement of the ground can be in any direction in the horizontal plane. The equivalent horizontal seismic forces will be considered as applied successively according to two characteristic orthogonal directions chosen by the designer. In the general case, these two directions are the main axes of the horizontal plane of the structure.

It should be noted, however, that the forces and deformations obtained for the element from the static analysis methods for the recommended design loads are lower than the forces and deformations that would be observed on the structure under the effects of a major earthquake for which the loads have been specified. This overshoot of forces is balanced by the ductile behavior that is provided by the construction details of the element. (See 4.2.1. [2]).

- **Data relating to the structure:**

The acceleration coefficient is given by the table (see 4.1 [2]) according to the seismic zone and the group of the building.

In our case we are in zone 1 and it is a building for residential use group 2 so:

A = 0.08.

The percentage of critical damping depends on the constituent material (%), we have a steel gantry with dense filling and according to the table (see 4.2 [2])ξ

Thus**:** ξ **=5%; then η= 1.**

The overall behaviour coefficient of the structure R is given by the table (see 4.3[2]) as a function of the bracing system as defined in 3.4.

Dans notre cas nous avons :

A metal structure braced in X following the x and y axis therefore: Rx=4 and Ry=4

Q: Quality factor is in function of:

- ✓ The redundancy and geometry of the elements of the structure.
- ✓ Regularity in plan and in elevation.
- ✓ The quality of construction control.
- ✓ The value of **Q** is determined by the following formula: $\mathbf{Q = 1 + \sum_1^5 P_q}$.

P_q is the penalty to be retained depending on whether the quality criterion (q) is met or not, its value is given from the table below (see Table 4.4, 4.2.3 [2]).

	P_q	
	X direction	**Y direction**
Minimum conditions on bracing lines	0	0
Plan redundancy	0	0
Regularity in plan	0	0
Regularity in elevation	0	0
Material quality control	0.05	0.05
Quality control of exécution	0.1	0.1
Total	0,15	0,15
	Q_x =1,15	Q_y=1,15

VI.4. Verification of the seismic study under the conditions of RPA99/2003:

The evaluation of seismic actions can be done according to the spectral modal analysis method.

VI.4.1. Calculation of the total seismic force:

The total seismic force V applied to the base of the structure shall be calculated successively in two orthogonal horizontal directions according to the following formula:

$$V = \frac{A \times D \times Q}{R} \times W \qquad \text{(cf.4.1. [2])}$$

The value of the fundamental period (T) of the structure can be estimated from the empirical formula:

$$T = C_T \times h_n^{3/4} \qquad \text{(cf.4.6. [2])}$$

$\boldsymbol{h_n}$**:** Height measured in meters from the base of the structure to the last level (N) : h_N=27.54m

$\boldsymbol{C_T}$: is a coefficient, a function of the bracing system and the type of filling.

Self-stable steel gantries with masonry filling C_T=0.085

$$T = 0.085 \times 27.54^{\frac{3}{4}} = 1.021 sec$$

The characteristic periods associated with the site category are given in Table 4.7 (cf. Table 4.7[2])

So : T1= 0,15 sec ; T2= 0,40 sec.

D: is the average dynamic amplification factor that is based on the site category of the damping factor (**η**) and the fundamental period of the structure(T)

$$D=\begin{cases} 2{,}5\eta \rightarrow 0 \le T \le T_2 \\ 2{,}5\eta\left(\frac{T_2}{T}\right)^{\frac{2}{3}} \rightarrow T_2 \le T \le 3{,}0\,s \\ 2{,}5\eta\left(\frac{T_2}{T}\right)^{\frac{2}{3}}\left(\frac{3}{T}\right)^{\frac{5}{3}} \rightarrow T \le 3{,}0 \end{cases} \quad \text{(cf.4.2.[2])}$$

$D = 2.5 \times \eta = 2.5 \times 1 = 2.5$

VI.4.2. Calculation of the weight of the structure :

W is equal to the sum of the Wi weights, calculated for each floor (i) :

W = Σ avec Wi= $W_{Gi} + \beta\ W_{Qi}$ (cf.4.5[2])

W_{Gi}: weight due to dead loads and those of any fixed equipment, attached to the structure.

W_{Qi}: Live Load.

β : Weighting coefficient, depending on the nature and duration of the operating load is given in Table (4.5).

Pour notre cas β = 0,20**.**

Wi= 458950.64kg

$$Vx = \frac{0.8\times1.33\times1.15}{4} \times 458950.64 \times 10^{-2} = 140.39KN$$

$$Vy = \frac{0.8 \times 1.33 \times 1.15}{4} \times 458950.64 \times 10^{-2} = 140.39KN$$

According to the article of RPA99/2003 (see 4.3.6[2]), the result of the seismic forces at base Vt obtained by combining modal values must not be less than 80% of the resultant of the seismic forces determined by the spectral method V for a value of the fundamental period given by the appropriate empirical formula. Let Vt > 0.8 V.

Following the application of the computational spectrum in both directions of the structure, the results are as follows:

- Seismic force in the X direction: Vt= 784.67KN.
- Seismic force in the Y direction: Vt= 842.40KN.

Table VI. 6 : Resulting from seismic forces at the base.

	Vt (KN)	80%V (KN)	Observation
Sens X	784.67	112.312	Verified
Sens Y	842.80	112.312	Verified

VI.4.3. Period:

Taking into account (cf. §4.2.4 [2]) the value of T calculated from the Rayleigh formula or the results obtained by the software must not exceed those estimated from the empirical formulas: **T = 1.021sec**, appropriate by more than 30%.

The fundamental period of the structure obtained by the software is: T=1.24sec

This period is considered acceptable compared to that formulated by RPA99/2003 which is:

1.021×1,3=1,372sec

T numeric =1.24sec < T empiric= 1.327 sec……condition verified.

VI.5. Dynamic study:

The dynamic study of a structure in reality is often very complex, which is why modelling is used.

The latter, amounts to representing a physical problem with an infinite degree of freedom (DDL) number by a model having a finite DDL number while respecting the precision of the parameters of the original system namely (mass, rigidity and damping), in other words a simplified model that brings us as close as possible to the real behavior of the structure, with a finite number of nodal elements and displacements.

VI.5.1. Dynamic methods

(a) The spectral modal analysis method may be used in all cases, and in particular, in cases where the equivalent static method is not permitted.

(b) The dynamic analysis method by accelerogram may be used on a case-by-case basis by qualified personnel who have previously justified the choice of calculation earthquakes and the laws of behaviour used, as well as the method of interpretation of the results and the safety criteria to be met.

The objective of this chapter is to determine a model that meets the requirements of seismic design.

VI.5.1.1. Number of modes considered (RPA99/2003):

According to **RPA99**/2003 (cf. 4.3.4 [2]): for structures represented by plane models in two orthogonal directions, the number of vibration modes to be retained in each of the two directions the excitation must be such that:

- The sum of the actual modal masses for the selected modes is equal to at least 90% of the total mass of the structure.
- Or that all modes with an effective modal mass greater than 5% of the total structural mass are used for the determination of the total response of the structure.

The minimum number of modes to be used is three in each direction considered.

VI.5.1.2. Modeling of the structure studied:

During a dynamic analysis of a structure, it is essential to find the appropriate modeling of the latter. Given the complexity and volume of calculation required to analyze the building, the use of the computer tool is essential.

As part of this study we have adopted for a calculation software that has existed for a few years and that is within our reach: it is the ROBOT software.

VI.5.1.3. Modal Analysis:

Modal analysis makes it possible to determine the proper modes and frequencies of the structure in the absence of external forces.

For dynamic analysis, the number of modes to be considered shall be such that the sum of the effective modal masses for the selected modes is equal to at least 90% of the total mass of the structure, or that all modes having an effective modal mass greater than 5% of the total mass of the structure are used for the determination of the total response of the structure. .

This type of analysis can be applied to all types of structure with more accurate and often satisfactory results provided that good modeling is done.

For our study the response spectrum is given by the software (spectrum RPA99), with the following data:

According to the geotechnical report relating to this structure, the **soil is of category S2.**

It is characterized by the following data:

- Seismicity zone: ZONE 1
- Usage group: GROUP 1B.

- Percentage of critical depreciation: ξ = 5%.
- Site classification : S2.

X direction :

- Behaviour coefficient: $R_x = 4$.
- Quality factor: Q_x= 1,15.

Y direction :

- Behaviour coefficient : $R_y = 4$.
- Quality factor : Q_y= 1,15.
- **Initial model:**

  Our structure is a self-stable frame, a three-dimensional set of rigidly bound beams and posts.
- **Rigidity modelling:**

  The modelling of the elements constituting the bracing (stiffness) is carried out as follows:

  Each beam and column was modeled by a two-node beam finite element (6 ddl per node).
- **The modal responses of the structure**

The deformations of the structure according to the most predominant modes of vibration are:

1st mode:

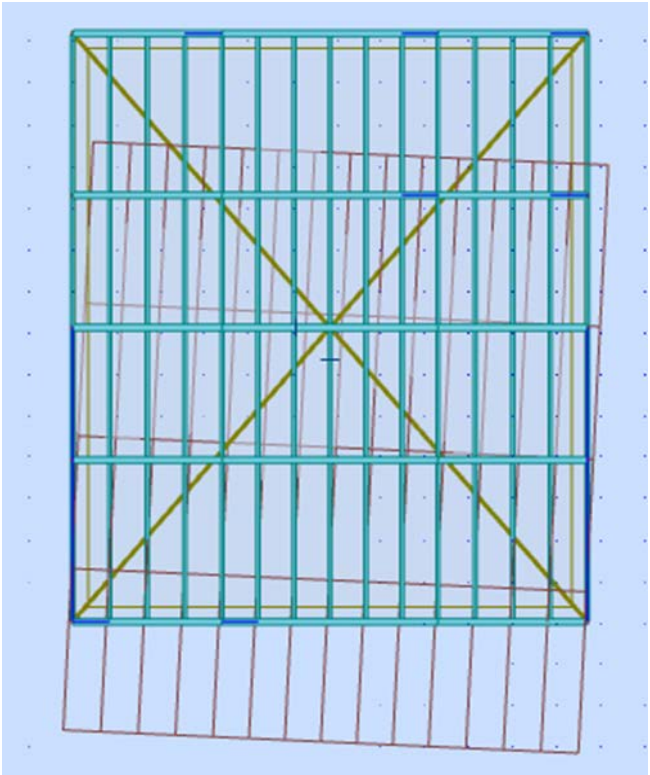


Figure VI. 4: 1st translation mode of the structure at T=1.24 s.

2nd mode:

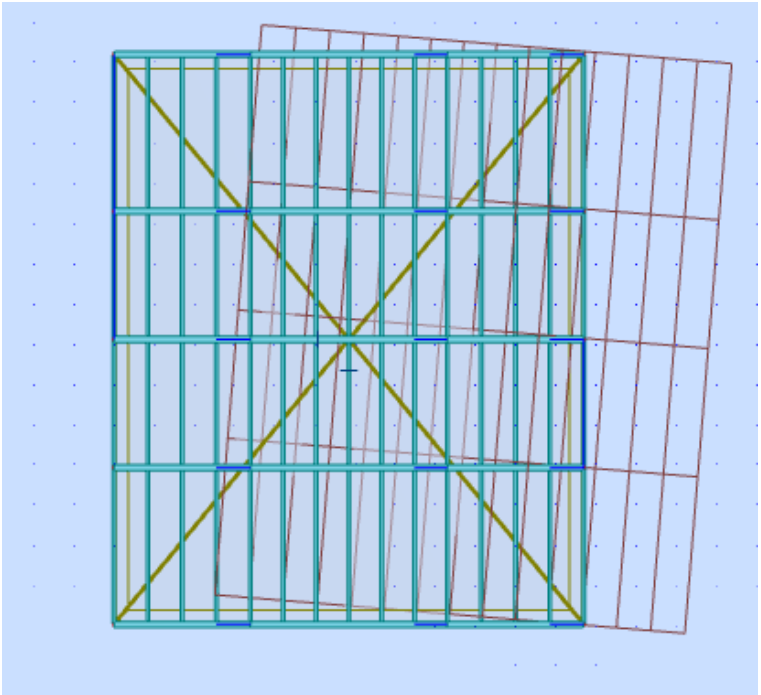


Figure VI. 5 : 2nd translation mode of the structure at T = 1.09s.

3rd mode:

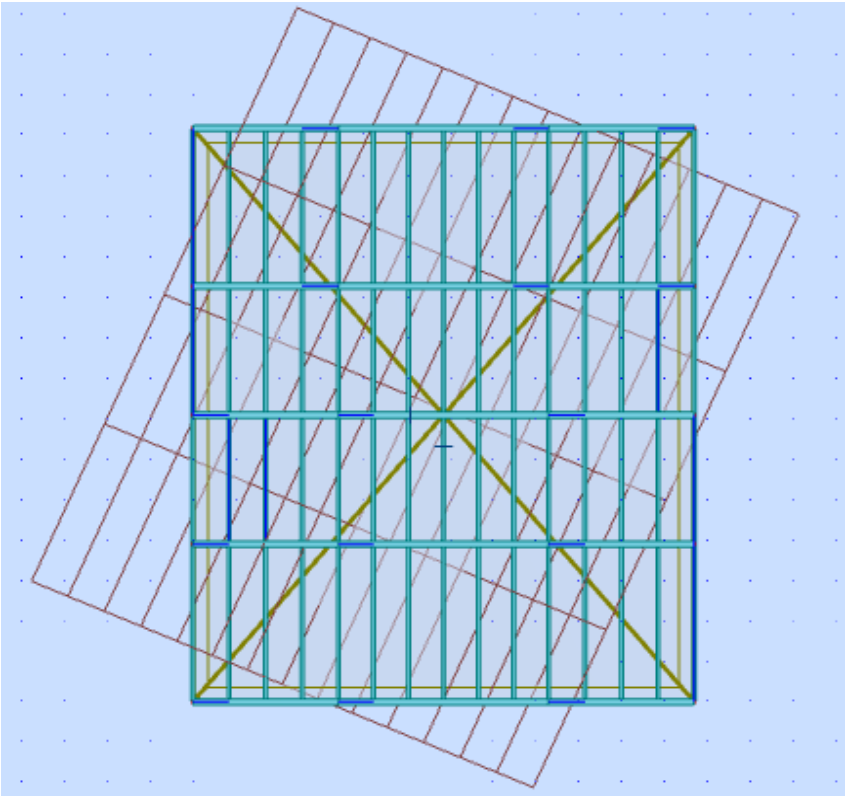


Figure VI. 6 : 3rd mode of rotation of the structure at T=0.93s.

VI.5.1.4. Result of the dynamic analysis:

- **Percentage of mass participation:**

The results in the following table correspond to the structure without bracing.

Table VI. 7 : Period and factors of mass participation of the initial model.

Case/Mode	Frequency (Hz)	Period (sec)	Rel.mas.UX (%)	Rel.mas.UY (%)	Rel.mas.UZ (%)	Cur.mas.UX (%)	Cur.mas.UY (%)	Cur.mas.UZ (%)	Total mass UX (kg)	Total mass UY (kg)	Total mass UZ (kg)
10/ 1	0.04	26.24	0.00	65.21	0.0	0.00	65.21	0.0	4443593.24	4443593.24	0.0
10/ 2	0.24	4.09	0.00	84.93	0.0	0.00	19.72	0.0	4443593.24	4443593.24	0.0
10/ 3	0.67	1.48	0.00	84.93	0.0	0.00	0.00	0.0	4443593.24	4443593.24	0.0
10/ 4	0.70	1.44	0.00	91.69	0.0	0.00	6.76	0.0	4443593.24	4443593.24	0.0
10/ 5	0.76	1.32	79.07	91.69	0.0	79.07	0.00	0.0	4443593.24	4443593.24	0.0
10/ 6	1.38	0.72	79.07	95.11	0.0	0.00	3.42	0.0	4443593.24	4443593.24	0.0
10/ 7	2.16	0.46	79.07	95.11	0.0	0.00	0.00	0.0	4443593.24	4443593.24	0.0
10/ 8	2.30	0.44	79.07	97.15	0.0	0.00	2.04	0.0	4443593.24	4443593.24	0.0
10/ 9	2.43	0.41	89.64	97.15	0.0	10.56	0.00	0.0	4443593.24	4443593.24	0.0
10/ 10	3.42	0.29	89.64	98.47	0.0	0.00	1.32	0.0	4443593.24	4443593.24	0.0

The results in the following table correspond to the structure with bracing.

TableVI. 8 : Periods and factors of mass participation for the final model.

Case/Mode	Frequency (Hz)	Period (sec)	Rel.mas.UX (%)	Rel.mas.UY (%)	Rel.mas.UZ (%)
9/ 1	0.74	1.24	5.44	72.88	0.0
9/ 2	0.75	1.09	77.82	78.89	0.0
9/ 3	1.07	0.93	79.07	79.60	0.0
9/ 4	2.31	0.43	79.33	92.45	0.0
9/ 5	2.41	0.42	89.46	92.84	0.0
9/ 6	3.37	0.30	89.62	92.98	0.0
9/ 7	4.15	0.24	89.66	96.40	0.0
9/ 8	4.47	0.22	93.74	96.44	0.0
9/ 9	5.83	0.17	93.75	98.07	0.0
9/ 10	6.06	0.17	93.81	98.08	0.0

From this table, we notice that the participation rate of the mass exceeds the threshold of 90% from mode 4 which are sufficient to represent a good behavior of the structure.

VI.5.1.5. Verification of displacements:

According to (cf.§ 4.43 [2]), the horizontal displacement at each level 'K' of the structure is calculated as follows: :

$\delta_k = R\ \delta_{ek}$ (cf.4.19[2])

δ_{ek} : Displacement due to Fi seismic forces (including torsion effect).

R : Behavior coefficient.

The relative displacement of level "k" with respect to level "k-1" is equal to :

$\Delta k = \delta_k - \delta_{k-1}$

The relative lateral displacements of a floor in relation to the adjacent floor must satisfy (cf. §5.10[2]), i.e. they must not exceed 1% of the floor height $\Delta_k < 1\%$ H floor.

- **Before bracing**:

Table VI. 9 : Resulting displacements according to façade wall (front bracing).

Height	Δx (cm)	Δy(cm)	$H_{étage}$/100 (cm)	Observation
Total 28.04m	11.8	162.9	28.04	Not verified

➢ **After bracing**:

Table VI. 10 : Resulting displacements following X (after bracing).

Niveau	Δx (cm)	Δy(cm)	$H_{étage}$ /100 (cm)	Observation
Total 28.04m	10.3	11.6	28.04	Verified

VI.6. Additional torsional effect:

The additional torsional effect was not taken into consideration because our structure is symmetrical.

VI.7. Conclusion :

In this chapter we analyzed the behavior of our structure under the action of the earthquake according to the Algerian seismic regulation RPA99 version 2003. Two methods were used: the equivalent static method and the modal spectral method by which the Shear force at the base was determined, following the bracing of the structure by X-blades. The latter were best suited for the proper functioning of the building vis-à-vis the openings.

Chapter 7:
Sizing of Structural and Secondary Elements

VII.1. Introduction:

Structural elements must be sized based on CCM 97 rules and the verification is done using the combinations of seismic force and vertical loads under the most unfavorable combinations retrieved from the **Autodesk Robot software.**

The simple frame is composed of beams and columns which are only subjected to vertical loads. The secondary elements are the joists, bracing and the staircase, the latter is the part of a structure that serves to ensure the connection between the different levels of a construction.

All elements of the structure must be determined and verified in such a way that they withstand both the service limit state and the ultimate limit state, and the loads applied.

VII.2. Sizing of Principal elements:

VII.2.1. Principal beams:

The main and secondary beams are structural elements, which make it possible to support the loads of the floors and transmit them to the posts, they are solicited mainly by a bending moment.

VII.2.1.1.: Principal beams for both the terrace and standard floors:

HEB 450 was chosen as the principal beams according to the results of the calculation software. The results of the solliciting moment and shear force obtained under the most unfavorable combination are as follows:

$M_{sd\ support}$ =-813.71KN.m.

$M_{sd\ span}$ =483.04 KN.m.

V_{sd} =-451.61 KN

a-Classification of the Principal beams HEB 450:

Table VII.1: Characteristics of the HEB 450 Section.

Weight	Area	Dimension		Characteristics			
G Kg/m	A cm^2	h mm	b mm	I_y cm^4	I_z cm^4	$W_{pl.y}$ cm^3	$W_{pl.z}$ cm^3
171	218×10^2	450	300	79890×10^4	11720×10^4	3982×10^3	1198×10^3

$$\sqrt{\frac{235}{f_y}}=\sqrt{\frac{235}{235}}=1$$

tw=14mm tf=26mm r=27mm d=344mm

Compressed flange:

$$\frac{c}{t_f}=\frac{300/2}{26}=5{,}76<10\,\varepsilon$$

Flange of Class 1

Webs (deflected):

$$\frac{d}{t_w}=\frac{344}{14}=24{,}57<72\,\varepsilon$$

Webs of class 1

The Global Section is of Class 1

b-Verification of shear:

We have to verify that: $V_{sd}\leq 0{,}5\,V_{pl.Rd}$

$$V_{pl.Rd}=A_V\frac{f_y/\sqrt{3}}{\gamma_{M0}}$$

Where : A_v : shear area

$A_v=A-2b\times tf+(t_w+2r)\times t_f=79{,}66\times10^2$ mm^2

$$V_{plrd}=79.68\times10^2\times\frac{235\times\sqrt{3}}{1.1}=982.550KN$$

V_{sd}=451.611≤ 0.5V_{plrd}=491.275KN.....condition verified.

c-Verification of the bending moment:

We have to verify that : $M_{sd} \leq M_{crd}$

(M_{crd}= M_{plrd} for classes 1 and 2)

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

$$M_{pl.y.Rd} = \frac{w_{ply} \times f_y}{\gamma_{M0}} = \frac{3982 \times 10^3 \times 235}{1,1} = 850,7\,KN.m$$

Msd_{span}=813.71KN ≤M_{plrd}=840.70KN……condition verified.

d- Verification of the deflection:

We have to verify that:

$$f\max = \frac{5}{384} \times \frac{qELS \times l^4}{EI_y} \leq \frac{l}{250}$$

Under the combination :

Self-weight of HEB 450 : p_p= 171 kg /m

$$f\max = \frac{5}{384} \times \frac{39.83 \times 9.8^4}{2.1 \times 10^5 \times 79890 \times 10^4 \times 10^{12}} = 28.5$$

$$\bar{f} = \frac{9800}{250} = 39.2$$

Where:

q_{ELS}=(G+Pp)+Q=((3.87x7.1)+1.17)+(1.50x7.1)=39.83KN

$fmax = 28.5KN \leq \bar{f} = 39.2KN$ ……..Condition verified.

e-Verification of lateral-torsional buckling:

The verification of lateral-torsional buckling is not necaessary because we used composite floors in our structure which prevents lateral-torsional buckling.

The profile met all the CCM 97 conditions regarding the verification of resistance, hence it is suitable to be uses as the principal beam.

VII.2.2. Column:

Generally, the vertical load-bearing elements of the building frames are subjected to compression, and eventually flexion according to the static system adopted, the latter by the loads of the floor and the horizontal forces.

Thus the columns are sized under compound bending. Our columns are profile HEB 650 with heights of 3.06m. According to the Autodesk Robot Software, the load values under the most unfavorable combination are:

N_{sd} = 593978.62daN=5939.786 KN

$M_{y.sd}$ = -3141.50daNm=-34.415 KNm $M_{z.sd}$ =273.40daNm= 2.73 KN.m

V_{ysd} = 257.84daN=2.578 KN V_{zsd}=-6462.17daN=-64.621KN

VII.2.2.1. Classification of Column HEB 600 :

Table VII.2 : Characteristics of HEB 600 section.

Weight	Area	Dimension		Characteristics			
G Kg/m	A cm^2	H mm	b mm	I_y cm^4	I_z cm^4	$W_{pl.y}$ cm^3	$W_{pl.z}$ cm^3
212	270×10^2	600	300	171000×10^4	135030×10^4	6425×10^3	1391×10^3

$$\sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{235}} = 1$$

Compressed flange:

$$\frac{c}{t_f} = \frac{300/2}{30} = 5 < 10\,\varepsilon$$

Flange of Class 1

Webs (deflected):

$$\frac{d}{t_w} = \frac{486}{15,5} = 31,35 < 72\,\varepsilon$$

Webs of class 1

The Global section is of Class 1

VII.2.2.2. Verification for resistance to shear:

We have to verify that**:**

$$V_{sd} \leq 0,5\,V_{pl.Rd}$$

$$V_{pl.Rd} = A_V \frac{f_y/\sqrt{3}}{\gamma_{M0}}$$

Where: A_v : Shear area

$A_v = A - 2b \times tf + (t_w + 2r) \times t_f = 110,8 \times 10^2$ mm^2

$$V_{pl.Rd} = 110,8 \times 10^2 \times \frac{235/\sqrt{3}}{1,1} = 1366,64\,KN$$

V_{sd}=2.748< 0.5V_{plrd}=683.32.....condition verified.

VII.2.2.3. Verification of the resistance to compound bending:

$$\left[\frac{M_{y.sd}}{M_{Ny.Rd}}\right]^{\alpha} + \left[\frac{M_{z.sd}}{M_{Ny.Rd}}\right]^{\beta} \leq 1$$

N_{sd}=5939.78KN

$$Nplrd = \frac{A \times f_y}{1.1} = \frac{270 \times 10^2}{1.1} = 5768.181KN$$

$n = \frac{Nsd}{Nplrd} = \frac{5939.78}{5768.181} = 1.029 > 1$ proposed solution : we increase the section of the column hence we choose HEB 650

Table VII.3: Characteristics of HEB 650 section.

Weight	**Area**	**Dimension**		**Characteristics**			
G Kg/m	A cm^2	H Mm	B Mm	I_y cm^4	I_z cm^4	$W_{pl.y}$ cm^3	$W_{pl.z}$ cm^3
225	286.3×10^2	650	300	210600×10^4	13980×10^4	7320×10^3	14411×10^3

$$\sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{235}} = 1$$

Compressed Flange:

$\frac{c}{tf} = \frac{300/2}{31} = 4.8 < 10\ \varepsilon$

Flange of Class 1

Webs (deflected):

$\frac{d}{tw} = \frac{534}{16} = 33.37 < 72\ \varepsilon$

Webs of Class 1

The Global section is of Class 1.

Verification of shear:

We have to verify that**:**

$$V_{sd} \leq 0{,}5\, V_{pl.Rd}$$

$$V_{pl.Rd} = A_V \frac{f_y/\sqrt{3}}{\gamma_{M0}}$$

Where : A_v : Shear area

$A_v = A - 2b\times tf + (t_w + 2r) \times t_f = 122.00\times10^2$ mm

$$V_{plrd} = 122 \times 10^2 \times \frac{235/\sqrt{3}}{1.1} = 1504.784KN$$

V_{sd}=2.748< 0.5V_{plrd}=752.395.....condition verified.

Verification of the resistance to compound bending:

$$\left[\frac{M_{y.sd}}{M_{Ny.Rd}}\right]^{\alpha} + \left[\frac{M_{z.sd}}{M_{Ny.Rd}}\right]^{\beta} \leq 1$$

N_{sd}=5939.78KN

$Nplrd = \frac{A \times f_y}{1.1} = \frac{286.3 \times 10^2}{1.1}$=6116.409KN

$$n = \frac{Nsd}{Nplrd} = \frac{5939.78}{6116.409} = 0.97$$

We have:

α =2

ß=5n=5x 0.97=4.85 >1

$$a = \frac{A - 2b \times tf}{A} = \frac{286.3 \times 10^2 - 2 \times 300 \times 31}{286.3 \times 10^2} = 0.35$$

$$M_{pl.y.Rd} = \frac{WplyRd \times fy}{\gamma M0} = \frac{7320 \times 10^3 \times 235}{1.1} = 1563.818KNm$$

$$M_{Ny.Rd} = 1.33M_{pl.y.Rd}(1 - n) = 1.33 \times 1563.818 \times (1 - 0.97) = 62.39KNm$$

Therefore :

$$M_{Ny.Rd} = M_{pl.y.Rd} = 62.39KNm$$

$M_{y.sd}$=-31.4150KNm

$$M_{pl.z.Rd} = \frac{Wplz \times fy}{\gamma M0} = \frac{1441 \times 10^3 \times 235}{1.1} = 307.8KNm$$

$$M_{N.z.Rd} = M_{pl.z.Rd} \times \left(\frac{1-n}{\frac{0.5 + h \times t}{A}} \right) = 307.8 \times \left(\frac{1-0.97}{\frac{0.5 + 650 \times 16}{286.3 \times 10^2}} \right) = 50.84KNm$$

$M_{Nsd} = 2.73KNm$

$(\frac{M_{y.sd}}{M_{Ny.Rd}})^{\alpha} + (\frac{M_{z.sd}}{M_{Ny.Rd}})^{\beta} = (\frac{-31.4150}{62.39})^2 + (\frac{2.73}{50.84})^{4.85} = 0.25 < 1$ …….Condition verified

VII.2.2.4. Verification against instabilities:

Verification of the strength of the compressed and flexed element (Paragraph 5.5.1. CCM97):

- Compound bending with risk of buckling:

$$\frac{N_{sd}}{\chi_{min} \times \frac{A \times f_y}{\gamma_{M1}}} + K_y \frac{M_{sd,y}}{w_{pl,y} \times \frac{f_y}{\gamma_{M1}}} + K_z \frac{M_{sd,z}}{w_{pl,z} \times \frac{f_y}{\gamma_{M1}}} \leq 1$$

- Compound bending with risk of lateral torsional buckling:

$$\frac{N_{sd}}{\chi_{min} \times \frac{A \times f_y}{\gamma_{M1}}} + K_{LT} \frac{M_{sd,y}}{\chi_{LT} \times w_{pl,y} \times \frac{f_y}{\gamma_{M1}}} + K_z \frac{M_{sd,z}}{w_{pl,z} \times \frac{f_y}{\gamma_{M1}}} \leq 1$$

a-Verification of buckling:

$$\frac{N_{sd}}{\chi_{min} \times \frac{A \times f_y}{\gamma_{M1}}} + K_y \frac{M_{sd,y}}{w_{pl,y} \times \frac{f_y}{\gamma_{M1}}} + K_z \frac{M_{sd,z}}{w_{pl,z} \times \frac{f_y}{\gamma_{M1}}} \leq 1$$

- **Buckling length:**

$L_f = 0.5 l_{f0} = 0.5 x 3.06 = 1.53m$ (fixed-fixed)

- **y-y axis:**

$$\bar{\lambda} = \left(\frac{\lambda}{\lambda_1} \right) \times [\beta A]^{0,5}$$

$\beta_A = 1$ …….section of class 1.

$$\lambda_y = \frac{l_f}{i_y} = \frac{1500}{271.2} = 5.53$$

$$\lambda_1 = \pi\sqrt{\frac{E}{f_y}} = 93{,}9$$

$$\overline{\lambda_y} = \frac{l_f}{i_y} = \frac{5.53}{93.9} = 0.058$$

$\overline{\lambda}_y$ =0,058<0,2 there in no risk of buckling in the y-y axis.

- **Following the z-z axis:**

$$\overline{\lambda} = \left(\frac{\lambda}{\lambda_1}\right) \times [\beta A]^{0,5}$$

$\beta_A = 1$ …….profile of classe 1.

$$\lambda_y = \frac{l_f}{i_y} = \frac{1500}{69.9} = 21.45$$

$$\lambda_1 = \pi\sqrt{\frac{E}{f_y}} = 93{,}9$$

$$\overline{\lambda_z} = \frac{l_f}{i_y} = \frac{21.45}{93.9} = 0.22$$

$\overline{\lambda}_z$ =0,22>0,2 there is a risk of buckling in axe z-z.

➢ **Calculation of χ_z :**

Choix de la courbe de flambement :

$$\begin{cases} \dfrac{h}{b} = \dfrac{450}{300} = 1{,}5 \succ 1{,}2 \\ t_f = 31 < 40 \rightarrow \text{donc : buckling curve b} \rightarrow \alpha = 0{,}34 \text{ (cf. Tableau 55.1 [3])} \end{cases}$$

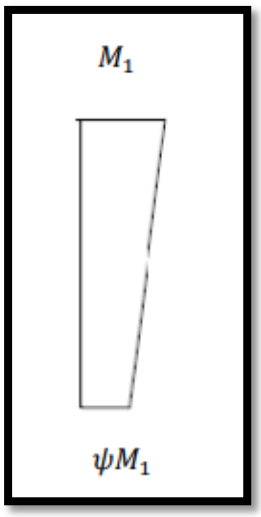
$$\phi_z = 0.5\left[1 + \alpha\left(\bar{\lambda}_z - 0.2\right) + \lambda_z{}^2\right] = 0.52$$

$$\chi_z = \frac{1}{\varphi_z + (\varphi_z{}^2 - \overline{\lambda_z}^2)^{0.5}} = \frac{1}{0.52 + (0.52^2 - 0.22^2)^{0.5}} = 1.0 \leq 1 \text{…….. Condition verified}$$

Calculation of K_z :

$$K_z = 1 - \frac{\mu_z . N_{sd}}{\chi_z . A . f_y}$$

- where : $K_z \leq 1,5$



$$\mu_z = \overline{\lambda_z}(2\beta_{M.Z} - 4) + \left(\frac{W_{pl.z} - W_{el.z}}{W_{el.z}}\right)$$

- where : $\mu_z \leq 0,9$

$$\beta_{M_z} = 1,8 - 0,7\,\Psi_z$$

$$\Psi_z = \frac{M_{z.2}}{M_{z.1}} = \frac{-121.10}{667.90} = -0.18$$

$$\beta_{M_z} = 1,8 - (0,7 \times 0.18) = 1,926$$

Thus :

$$\mu_z = \overline{\lambda_z}(2\beta_{M.Z} - 4) + \left(\frac{W_{pl.z} - W_{el.z}}{W_{el.z}}\right) = 0,22\ (2 \times 1.926 - 4) + \left(\frac{1441 \times 10^3 - 932.3 \times 10^3}{932.3 \times 10^3}\right) = 0.39$$

$\mu_z \leq 0,9$ ……..Condition verfifed.

$$K_z = 1 - \frac{\mu_z . N_{sd}}{\chi_z . A . f_y} = 1 - \frac{0.39 \times 5939.7862 \times 10^3}{1 \times 286.3 \times 10^2 \times 235} = 0.655$$

$K_z \leq 1,5$ ……condition verified.

$$\frac{N_{sd}}{\chi\ min \times \frac{A \times fy}{\gamma_{M1}}} + K_z \frac{M_{sd.z}}{W_{pl.z} \times \frac{fy}{\gamma_{M1}}} = \frac{5939.38 \times 10^3}{1 \times \frac{286.3 \times 10^2}{1.1}} + \frac{0.655 \times 2.73 \times 10^6}{1441 \times 10^3 \times \frac{235}{1.1}} = 0.97 < 1$$ …. Condition verified

so there is no risk of buckling in z-z.

b-Verification of lateral-torsional buckling:

The phenomenon of lateral-torsional buckling is to be taken into account only for reduced slenderness:

$$\overline{\lambda_{LT}} > 0,4$$

Where:

$\overline{\lambda_{LT}}$ is given by the following formula:

$$\overline{\lambda_{LT}} = \left[\frac{\lambda_{LT}}{\lambda_1}\right].(\beta_w)^{0,5}$$

$$\beta_w = 1$$

$$\lambda_1 = \pi\sqrt{\frac{E}{f_y}} = 93,9$$

$$\lambda_{LT} = \frac{k.L\left(\frac{w_{pl.y}}{I_w.I_z}\right)^{0,25}}{\sqrt{C_1}\left[\left(\frac{k}{k_w}\right)^2 + \frac{(kL)^2 \times G \times t_f}{\pi^2 \times E \times I_w}\right]^{0,25}}$$

With: C1 = 3.093 according to the moment diagram and the effective length factor K (see Table B.1.1.[3]).

Since the column are fixed at both ends, the factors of effective length K_w and K are taken equal to 0.5 so the formula is of form λ_{LT}

$$\lambda_{LT} = \frac{0,5\times3600.\left(\frac{(7320\times10^3)^2}{13360\times10^9\times13980\times10^4}\right)^{0,25}}{\sqrt{3,093}\left[1+\frac{(0;5.3600)^2\times225\times31}{\pi^2\times210000\times13360\times10^9}\right]^{0,25}} = 13,97$$

$$\overline{\lambda_{LT}} = \left[\frac{\lambda_{LT}}{\lambda_1}\right].(\beta_w)^{0,5}$$

$\overline{\lambda_{LT}}$ = 0,14 < 0,4 → We do not have to take into consideration the lateral-torsional buckling.

VII.3. Sizing of secondary elements:

VII.3.1. Joists :

For the floor joists we chose IPE 300 profiles.

a- Joist classification IPE 300:

Table VII. 4 : Characteristics of IPE 300 section.

Weight	Area	Dimension		Characteristics			
G Kg/m	A cm^2	h mm	b mm	I_y cm^4	I_z cm^4	$W_{pl.y}$ cm^3	$W_{pl.z}$ cm^3
42.2	53.8×10^2	300	150	8356×10^4	603.8×10^4	628.4×10^3	125.2×10^3

$$\varepsilon = \sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{235}} = 1$$

Compressed Flange :

$$\frac{c}{tf} = \frac{150/2}{10.7} = 7.00 < 10\ \varepsilon$$

Flange of Class 1

Webs (deflected) :

$$\frac{d}{tw} = \frac{248.6}{7.1} = 35.01 < 72\ \varepsilon$$

Webs of class 1

Globale section is of Class 1.

b-Combinaition of charge:

- **ELU :**

q_{ELS}=((G+p_p) + Q) x distance between the joists

q_{ELS}=**[(**5,72+1)x1.5=10.08KN/ml

- **ELS:**

q_{ELU1}=1,35(G+p_p) +1,5Q

q_{ELU1}=((1,35×5,72) +(1,5×1))×1,5)=13,83 KN/ml

q_{ELU2}=1,35(G+p_p) + 1,5Q

q_{ELU2} =[(1,35×3.87) +(1,5×1.5)] × 1,5=11.21 KN/ml

c-Shear Verification :

We have to verify that: $V_{sd} \leq 0{,}5\, V_{pl.Rd}$

$$V_{pl.Rd} = A_V \frac{f_y/\sqrt{3}}{\gamma_{M0}}$$

Where: A_v :shear area.

A_v=A-2b×tf + (t_w+2r) × t_f=25.66×10^2 mm^2

$$V_{pl.Rd} = 25.66 \times 10^2 \times \frac{235/\sqrt{3}}{1.1} = 316.49KN$$

$$V_{sd} = \frac{13.8 \times 7.8}{2} = 53.93KN$$

$V_{sd} = 53.93 \leq 0.5\, V_{plrd} = 158.24KN$ …….. condition verified

d-Verification of the bending moment:

It must be verified that: $M_{sd} \leq M_{pl.Rd}$

$$M_{pl.y.Rd} = \frac{W_{ply} \times f_y}{\gamma_{M0}} = \frac{628.4 \times 10^3 \times 235}{1.1} = 134.24KNm$$

$$M_{sd} = \frac{q_{ELS} \times l^2}{8} = \frac{13.83 \times 7.8^2}{8} = 105.17KNm$$

$M_{sd} = 105.17KNm \leq M_{pl.y.Rd} = 134.24KNm$……condition verified

e-Verification of the deflection:

we have to verify that:

$$f_{max} = \frac{5}{384}\frac{q_{ELS}.l^4}{E\,I_y} \leq \bar{f} = \frac{l}{250}$$

Under the combination :

$$f\max = \frac{5}{384} \times \frac{10.08 \times 7.8^4}{2.1 \times 10^8 \times 8356 \times 10^4 \times 10^{-12}} = 27.68mm$$

$$\bar{f} = \frac{7.8 \times 10^3}{250} = 31.2mm$$

$f\max = 27.68\text{mm} < \bar{f} = 31.2mm$ ……..Condition verified

f-Verification of lateral buckling:

Verification of the lateral buckling is not necessary because we used the composite floors that prevent lateral buckling

The IPE 300 profile meets all the conditions of CCM97 regarding the resistance check hence it is suitable as the floor joists.

VII.3.2. Stairs:

The staircase is a work consisting of a series of horizontal plans allowing access to the different floors of the building, the staircase is determined by its height, its step, its tread and it's step height, In our project, we have a single type of staircase in metal frame which is composed of two flights linked to a landing, the flights consists of two stringers in UPN profile, and the steps consist of support angle iron and checkered sheet metal.

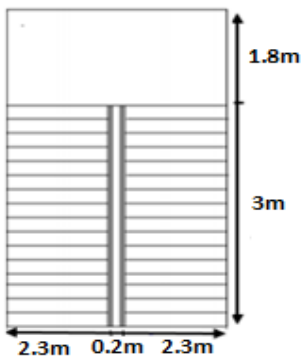


Figure VII.1 : Diagram of the staircase with two flights.

Figure VII. 2: Constituents of a stairs.

VII.3.2.1. Dimensioning of the stairs:

To size the step and riser, we use the BLONDEL formula. The condition assures the comfort of the stairs:

$$60 \text{ cm} \leq (g+2h) \leq 65 \text{ cm}$$

Where:

- h : Height of the riser given by : $14cm \leqslant$ $cm \leq 20$
- g : Length of the step (tread) given by : $22cm \leq g \leq 30\ cm$.
- n : Number of the risers.
- m : Number of steps.
- H : Height of a flight of stairs H=1.53m.

a-Height of the riser et the length of the step:

We take: $\begin{cases} h = 16 \text{ cm} \\ g = 30 \text{ cm} \end{cases}$

(g+2h)= (30+2×16) =62 cm

60m≤ 62 ≤ 65 cm ……..condition verified.

b-Length of steps(m) :

In order to calculate the length of the steps we have to calculate the length of the counter steps by the following equation:

$$n = \frac{153}{16} = 9.6$$

We take: n=10

m=n-1=10-1=9

we have 10 steps and 9 counter steps

The total length of the stairs would be : L=g×n =30×10=300cm.

VII.3.2.2. Dimensioning of the load-bearing elements:
a-sizing of the step angle iron:

The steps are made by checkered sheets and angle irons whose length L = 2.3 m.

The modeling of a step is done like that of a beam simply supported

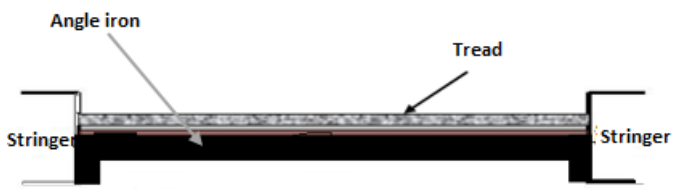


Figure VII.3: Flight cross-section.

- **Deflection condition:**

The value of the maximum deflection is:

$$f^{max} = \frac{5}{384}\frac{q_{ELS}.L^4}{E\,I_y}$$

The value of the allowable deflection is:

$$\bar{f} = \frac{L}{250}$$

In the case of a bi-articulated beam the formula of the deflection for pre-sizing is:

$$f^{max} = \frac{5}{384}\frac{q_{ELS}.l^4}{E\,I_y} \leq \bar{f} = \frac{l}{250}$$

where: q_{ELS}=(G+Q)g=(1,25+2.5)× $\frac{0,3}{2}$=0,56KN/m

therefore:

$I_y \geq \frac{5\times0.56\times2.3^4\times250}{384\times2.1\times10^8} \rightarrow I_y \geq 10.5\times10^4 mm^4$ → We choose **L 50×50×5.**

- **Verification of the deflection (self-weight included):**

q_{ELS}=q_{ELS}+p_p=0,56+0,044= 0,604KN/m

We have to verify that:

$$f_{max} \leq \bar{f}$$

$I_y = \frac{5\times0,827\times2.3^4}{384\times2,1\times10^8\times12.84\times10^4\times10^{-12}} = 8.16\times10^{-3}m \leq \frac{2.3}{250} = 9,2\times10^{-3}m....$ Condition verified

- **Class of Angle Iron L 50×50×5:**

$$\varepsilon = \sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{235}} = 1$$

$\frac{h}{t} = \frac{50}{5} = 10 < 15\,\varepsilon$ $\frac{b+h}{2t} = \frac{50+50}{2\times5} = 10 < 11.5\,\varepsilon$

Class 3 Class 2

Hence **L 50×50×5** is of class 3.

➢ Verification of shear:

We have to verify that:

$V_{sd} \leq 0.5V_{pl.Rd}$

$$V_{pl.Rd} = A_V \frac{f_y / \sqrt{3}}{\gamma_{M0}}$$

Where: A_v : Shear Area.

$A_v = L \times t = 50 \times 5 = 250mm^2$

$$V_{pl.Rd} = 250 \times \frac{235 / \sqrt{3}}{1,1} = 30,83\,KN$$

$$V_{sd} = \frac{qELU \times l}{2} = \frac{0.87 \times 2.3}{2} = 1.00KN$$

Where q_{ELU}=1,35(G+p_p) + 1,5Q=[1,35 (1,25)+1,5(2.5)]x0,15 =0.87KN/ml

$V_{sd} = 1.00KN \leq 0.5V_{plrd} = 22.5KN$.....condition verified.

➢ Verification of the bending moment:

We have to verify that: $M_{sd} \leq M_{pl.Rd}$

$$M_{pl.y.rd} = \frac{W_{ply} \times f_y}{\gamma_{M0}} = \frac{3.61 \times 10^3 \times 235}{1.1} = 0.771KNm$$

$$M_{sd} = \frac{q_{ELU} \times l}{2} = \frac{0.87 \times 2.34^2}{8} = 0.595KNm$$

$M_{sd} = 0.595KNm \leq M_{pl.y.rd} = 0.771KNm$.....condition verified.

b-Sizing of Stringer:

Stringer inclination:$\alpha = tan^{-1}\left(\frac{H}{L}\right) = tan^{-1}\left(\frac{153}{300}\right) = 27°$

Length of the Stringer: L= $\left(\frac{200}{\sin(26^{\circ})}\right)$ =456,23cm We take L= 456 cm

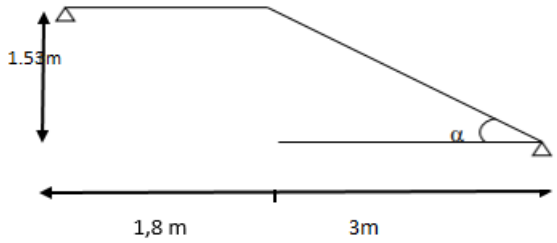


Figure VII.4 : Stringer dimension.

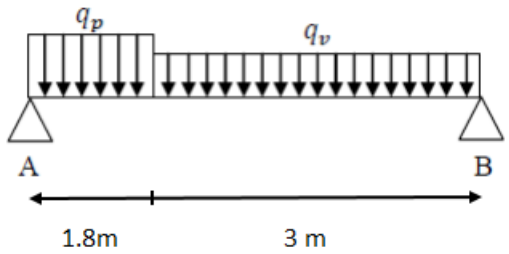
➢ **Loads :**

Dead Load: G_{flight}=1,25 KN/ml

$G_{landing}$ =2,91 KN/ml

Live Load: Q= 2.5 KN/ml

Load combination:



ELS :

flight: $q_{ELS\,1} = \frac{1,8}{2}(1,25 + 2.5) = 3.37 KN/ml$

landing: $q_{ELS\,2} = \frac{1,8}{2}(2,91 + 2.5) = 4.869 KN/ml$

ELU :

flight: $q_{ELU\,1}$ = [1.35(1.25×$\frac{1,8}{2}$+0 .128)+1.5(2.5×$\frac{1,8}{2}$)]=7.09 KN /ml

landing: $q_{ELU\,2}$ =[1.35(2.91×$\frac{1,8}{2}$+0 .128)+1.5(4×$\frac{1,8}{2}$)]= 9.10 KN/ml

The equivalent load:

- **ELS :**

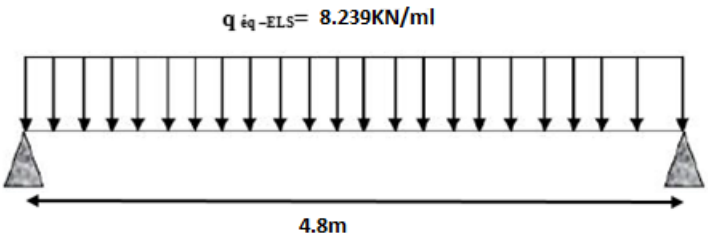


Figure VII.5 : Equivalent load in l'ELS.

- **ELU :**

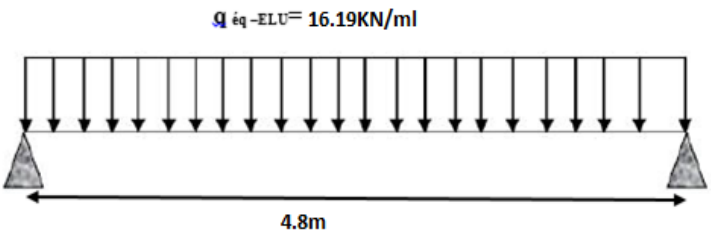


Figure VII.6 : Equivalent charge in L'ELU.

- **Deflection condition:**

The value of the maximum deflection is:

$$f^{max} = \frac{5}{384}\frac{q_{éq-ELS}.L^4}{E\,I_y}$$

The value of the allowable deflection is

$$\bar{f} = \frac{L}{250}$$

In the case of a bi-articulated beam the boom formula for pre-sizing is used:

$$f^{max} = \frac{5}{384}\frac{q_{éq\text{-}ELS}.l^4}{E\,I_y} \leq \bar{f} = \frac{l}{250}$$

Hence:

$I_y = \frac{5\times3.37\times3^3\times250}{384\times2,1\times10^8} \rightarrow I_y \geq 141.04\times10^4 mm^4$……. we choose **UPN 100**

- **Verification of the deflection (self-weight included):**

$q_{ELS}=q_{ELS}+p_p=3.37+0,106=3.47KN/m$

We have to verify that:

$$f_{max} \leq \bar{f}$$

$\frac{5\times3.47\times3.37^4}{384\times2,1\times10^8\times206\times10^4\times10^{-12}} = 13.47\,m \leq \frac{3370}{250} = 13.48m$……condition verified.

- **Verification of resistance:**

The most unfavorable soliciting efforts calculated by the ROBOT software are:

M_{ysd}=55.4476KN.m.

N_{sd}= 13.46KN.

$V_{z.sd}$=3.96KN.

- **Verification of shear:**

We have to verify that**:**

$V_{sd} \leq 0.5V_{pl.Rd}$

$$V_{pl.Rd} = A_V\frac{f_y/\sqrt{3}}{\gamma_{M0}}$$

Where: A_v : Shear Area.

$A_v = 6.95\times10^2 mm^2$

$$V_{pl.Rd} = 6.95 \times 10^2 \times \frac{235/\sqrt{3}}{1.1} = 85.72KN$$

$V_{sd} = 55.47 \leq 0.5\, V_{plrd} = 42.86KN$ …….. condition not verified

Solution: we choose a section with a bigger area: UPN 120

- **Verification of shear for UPN 120:**

We have to verify that: $V_{sd} \leq 0{,}5\, V_{pl.Rd}$

$$V_{pl.Rd} = A_V \frac{f_y / \sqrt{3}}{\gamma_{M0}}$$

Where: A_v : Shear area.

$A_v = 9.35 \times 10^2 mm^2$

$$V_{pl.Rd} = 9.35 \times 10^2 \times \frac{235/\sqrt{3}}{1.1} = 114.709KN$$

$V_{sd} = 55.47 \leq 0.5\, V_{plrd} = 57.35KN$ …….. condition verified

- **Verification of the resistance to compound bending:**

The stair stringer is solicited in compound bending, so it must satisfy the following condition of resistance:

$$M_{sd} \leq M_{N.y.Rd} = M_{ply.Rd}\left(\frac{1-n}{1-0{,}5a}\right)$$

Ou : $n = \frac{N_{sd}}{N_{pl.Rd}}$ et : $a = (A - 2bt_f)/A$ where : $a \leq 0.5$

N_{sd}=55.44KN

$$N_{pl.Rd} = \frac{A \times f_y}{\gamma_{M0}} = \frac{17 \times 10^3 \times 235}{1.1} = 363.181KN \qquad a = \frac{A - 2 \times b \times t_f}{A} = 0{,}45$$

$$n = \frac{N_{sd}}{N_{plrd}} = \frac{55.44}{363.181} = 0.152$$

$$M_{pl.y.rd} = \frac{W_{ply} \times f_y}{\gamma_{M0}} = \frac{72.6 \times 10^3 \times 235}{1.1} = 15.51KNm$$

$$M_{N.y.Rd} = 15.51 \times (\frac{1 - 0.152}{1 - 0.5 \times 0.41}) = 16.54KNm$$

$M_{N.y.Rd} = 16.54 \geq M_{ysd} = 3.96KNm$…… condition verified.

- **Verification of Buckling:**

Class of the section: according to the table of profiles → class 1

Since $M_{.z.Sd}$ is null, we have:

$$\frac{N_{sd}}{\chi_{min} \times \frac{A \times f_y}{\gamma_{M1}}} + K_y \frac{M_{sd,y}}{w_{pl,y} \times \frac{f_y}{\gamma_{M1}}} \leq 1$$

- **Buckling length:**

$$l_f = l_0 = 3.37 \text{ m}$$

- **Following the y-y axis:**

$$\bar{\lambda} = \left(\frac{\lambda}{\lambda_1}\right) \times [\beta A]^{0,5}$$

$\beta_A = 1$ ……Class 1.

$$\lambda_y = \frac{l_f}{i_y} = \frac{3370}{4.62 \times 10} = 72.94 \qquad \lambda_1 = \pi\sqrt{\frac{E}{f_y}} = 93.9$$

$$\bar{\lambda}_y = \frac{\lambda_y}{\lambda_1} = \frac{72.94}{93.9} = 0.776$$

$\bar{\lambda}_y$ =0.776>0.2…… There is risk of buckling in the y-y axis.

- **Following the z-z axis:**

$$\bar{\lambda} = \left(\frac{\lambda}{\lambda_1}\right) \times [\beta A]^{0,5}$$

$\beta_A = 1$ …… section of class 1.

$$\lambda_y = \frac{l_f}{i_y} = \frac{3370}{1.59 \times 10} = 211.94 \qquad \lambda_1 = \pi\sqrt{\frac{E}{f_y}} = 93,9$$

$$\bar{\lambda}_y = \frac{\lambda_y}{\lambda_1} = \frac{211.94}{93,9} = 2.25$$

$\bar{\lambda}_z$ =2.25>0,2……there is a risk of buckling along the z-z axis.

Since: $\lambda_y < \lambda_z$

$\chi_{\min}$ Will be taken equal to our profile is U-shaped so buckling along the weak axis then the curve (c). χ_y

According to curve (c) → $\chi_{\min} = 0,7247$

β_{M-Y} =1.3

Thus:

$$\mu_y = \overline{\lambda_y}(2\beta_{M.y} - 4) + \left(\frac{W_{pl.y} - W_{el.y}}{W_{el.y}}\right) = 0,698(2 \times 1,3 - 4) + \left(\frac{179 - 150}{150}\right) = -0,783$$

$\mu_y \leq 0,9$……Condition verified.

$$K_y = 1 - \frac{\mu_y . N_{sd}}{\chi_y . A . f_y} = 1 - \frac{-0,783 \times 7,56 \times 10^3}{0,7247 \times 28 \times 10^2 \times 235} = 1,01$$

$K_y \leq 1.5$ ……Condition verified.

$$\frac{N_{sd}}{\chi_{min}\times\frac{A\times f_y}{\gamma_{M1}}}+K_z\frac{M_{sd,y}}{w_{pl,y}\times\frac{f_y}{\gamma_{M1}}}=\frac{7{,}56\times10^3}{0{,}7247\times\frac{28\times10^2\times235}{1{,}1}}+1{,}01\frac{18{,}46\times10^6}{179\times10^3\times\frac{235}{1{,}1}}=0{,}45$$

0.459 < 1……. ……condition verified.

- **Verification of lateral-torsional buckling:**

The phenomenon of lateral-torsional buckling is only taken into account for reduced slenderness.

$\overline{\lambda_{LT}} > 0.4$

$\overline{\lambda_{LT}}$ is given by the following formula:

$$\overline{\lambda_{LT}}=\left[\frac{\lambda_{LT}}{\lambda_1}\right].(\beta_w)^{0,5}$$

$\beta_w=1$

$$\lambda_1=\pi\sqrt{\frac{E}{f_y}}=93{,}9$$

$$\lambda_{LT}=\frac{l/i_z}{\left[1+\frac{1}{20}\left(\frac{l/i_z}{h/t_f}\right)^2\right]^{0,25}\sqrt{C_1}}$$

Where: C1 = 1 according to the moment diagram and the effective length factor K.

Since: the columns are fixed at both ends, the factors of effective length K and K are taken equal to 0.5 so the formula of is of the form: λ_{LT}

$$\lambda_{LT}=\frac{4560/20{,}2}{\left[1+\frac{1}{20}\left(\frac{4560/20{,}2}{180/11}\right)^2\right]^{0,25}\sqrt{1}}=166{,}50$$

$$\overline{\lambda_{LT}} = \left[\frac{\lambda_{LT}}{\lambda_1}\right].(\beta_w)^{0,5}$$

$\overline{\lambda_{LT}}$ = 1,77> 0,4 so it has the risk of lateral-torsional buckling.

$$\alpha_{LT} = 0,21$$

$$\varphi_{LT} = 0,5\left[1+\alpha\left(\overline{\lambda}_{LT} - 0,2\right)+\overline{\lambda}_{LT}^{\,2}\right] = 2,23$$

$$\chi_{LT} = \frac{1}{\varphi_{LT} + \left[\varphi_{LT}^{\,2} - \overline{\lambda_{LT}}^{\,2}\right]^{0,5}} = 0,53$$

$$\mu_{LT} = 0,15.\overline{\lambda_z}.\beta_{M.LT} - 0,15 = 0,318$$

- **Calculation of K_{LT} :**

Buckling curve a : $\chi_z = 0,2994$

$$K_{LT} = 1 - \frac{\mu_{LT}.N_{sd}}{\chi_z.A.f_y} = 0,98$$

$$\frac{N_{sd}}{\chi_z \times A \times f_y} + K_{LT}\frac{M_{sd,y}}{w_{pl,y} \times \chi_{LT} \times f_y} = \frac{7,56\times10^3}{0,2994\times2800\times235} + 0,98\frac{18,46\times10^6}{179\times10^3\times0,53\times235} = 0,84$$

Lateral-torsional buckling verified.

The UPN 180 profile is suitable as the stringer of the flight.

c-Landing beam Joist:

Spacing between the joist is 0.95 m.

- **Load:**

$G_{landing}$ =2.90 KN/m^2

$Q_{landing}$= 4KN/m^2

q_{ELS}= (2,9 +4)×0,95 =6,55 KN/ml

- **Deflection condition:**

The value of the maximal deflection is:

$$f^{max} = \frac{5}{384}\frac{q_{éq-ELS}.L^4}{E\,I_y}$$

The value of the allowable deflection is:

$$\bar{f} = \frac{L}{250}$$

In the case of a bi-articulated beam the formula of the deflection for pre-sizing used is:

$$f^{max} = \frac{5}{384}\frac{q_{éq\text{-}ELS}.l^4}{E\,I_y} \leq \bar{f} = \frac{l}{250}$$

So: $I_y \geq \frac{5\times 6{,}55\times 1{,}8^3\times 250}{384\times 2{,}1\times 10^8} \rightarrow I_y \geq 59{,}21\times 10^4\ mm^4$ So we choose **IPE 80.**

d- Landing beam:

- **Load:**

$G_{landing}$ =2,90 KN/m^2

$Q_{landing}$= 4 KN/m^2

$P_{p\ Joist}$ =0,06 KN/m

q_{ELS}= $(2{,}9+4)\times\frac{1{,}8}{2}+0{,}06 = 6{,}27$ KN/ml

➢ Deflection condition:

The value of the maximum deflection is:

$$f^{max} = \frac{5}{384}\frac{q_{éq-ELS}.L^4}{E\,I_y}$$

The value of the allowable deflection is:

$$\bar{f} = \frac{L}{250}$$

In the case of a bi-articulated beam the formula of the deflection for pre-sizing used is:

$$f^{max} = \frac{5}{384}\frac{q_{éq\text{-}ELS}.l^4}{E\,I_y} \leq \bar{f} = \frac{l}{250}$$

So : $I_y \geq \frac{5\times 6{,}27\times 3{,}8^3\times 250}{384\times 2{,}1\times 10^8} \Rightarrow I_y \geq 533{,}30\times 10^4\ mm^4$ We choose **IPE 160.**

➢ Verification of the deflection (self-weight included):

$q_{ELS}=q_{ELS}+p_p=6{,}27+0{,}158=6{,}42KN/m$

We have to verify that:

$$f_{max} \leq \bar{f}$$

$$\frac{5\times 6{,}42\times 3{,}8^4}{384\times 2{,}1\times 10^8\times 869\times 10^4\times 10^{-12}} = 9{,}55\times 10^{-3}\ m \leq \frac{3{,}8}{250} = 0{,}015\,m$$ ...Condition verified.

➢ Class of IPE 160 :

$$\varepsilon = \sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{235}} = 1$$

Compressed Flange:

$$\frac{c}{t_f} = \frac{82/2}{7,4} = 5,54 < 10\,\varepsilon$$

Flange of class 1

Webs (deflected):

$$\frac{d}{t_w} = \frac{127,2}{5} = 25,44 < 72\,\varepsilon$$

Webs of class 1

So the IPE 160 section is of class 1.

- **Verification of shear:**

We have to verify that: $V_{sd} \leq 0,5\,V_{pl.Rd}$

$$V_{pl.Rd} = A_V \frac{f_y/\sqrt{3}}{\gamma_{M0}}$$

Where: A_v : Shear Area.

$A_v = 9,66\times10^{2}\,mm^2$

$$V_{pl.Rd} = 9,66\times10^{2} \times \frac{235/\sqrt{3}}{1,1} = 119,14\,KN$$

q_{ELU}=1,35(G+p_p)+1,5Q=(1,35×2,9 +1,5×4)×$\frac{1,8}{2}$+ (1,35×0,158)=9 ,14 KN/ml

$$V_{sd} = \frac{q_{ELU}\times L}{2} = \frac{9,14\times3,8}{2} = 17,37\,KN$$

$V_{sd} = 17,37\,KN \leq 0,5\,V_{pl.Rd} = 59,57\,KN$ .....condition verified.

- **Verification of the bending moment:**

We have to verify that: $M_{sd} \leq M_{pl.Rd}$

$$M_{pl.y.Rd} = \frac{w_{ply}\times f_y}{\gamma_{M0}} = \frac{124\times10^{3}\times235}{1,1} = 26,49\,KN.m$$

$$M_{sd} = \frac{q_{ELU}\times l^2}{8} = \frac{9,14\times3,8^2}{8} = 16,49\,KN.m$$

$M_{sd} = 16{,}49\,KN.m \leq M_{pl.Rd} = 26{,}49\,KN.m$ ……condition verified.

The IPE 160 meets all CCM97 requirements for resistance verification.

VII.3.3. Bracing:

Bracing is intended to ensure the overall stability of the structure vis-à-vis the horizontal effects resulting from any actions on it such as wind and earthquake. They take up these actions and help prevent, or at least restrict, lateral movements in the frames.

For our structure X bracing was used.

For this type of bracing according to **AUTODESK ROBOT** software the most unfavorable loads are:

$N_{traction} = 269.40$ KN

$N_{compression} = -196.98$ KN

VII.3.3.1. Verification of Traction:

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

$$N_{pl\,Rd} = \frac{A.f_y}{\gamma_{M0}} = \frac{12.3\times10^2\times235}{1.1} = 262{,}272KN$$

$N_{traction} = 269.40 < 2\times N_{pl\,Rd} = 524.544$ KN…….condition verified.

VII.3.3.2.Verification of compression:

$N_{compression} \leq N_{pl\,Rd}$

$$N_{pl\,Rd} = \frac{A.f_y}{\gamma_{M0}} = \frac{12.3\times10^2\times235}{1.1} = 262.272\ KN$$

$N_{compression} = 196.98\ KN < 2\times N_{pl\,Rd} = 524.54$ KN……...condition verified.

VII.4. Conclusion:

The sizing of the main and secondary metal structural elements was done under the CMM97 code.

Good sizing takes into account the maximum stresses applied to the ultimate limit states such that each constructive detail composing the structure is calculated, dimensioned and verified according to the standards.

In this chapter we summarize the results of the sizing of the structural and secondary elements (see the table below).

Element	**Profile Section**	
Joist	Standard floor	Terrace
	IPE 300	IPE 300
Principal beam	HEB 450	HEB450
Column	HEB 650	
Staircase beam	IPE 180	
Stair Joist	IPE 80	

Chapter 8 :
Composite Floor Study

VIII.1. Introduction:

Mixed construction is the mechanical combination of reinforced concrete and metal profiles, in order to form perfectly monolithic elements contributing to the better functioning of a civil engineering structure. It is a question of making the most of the constituent materials (concrete, reinforcements and metal profiles) from the point of view of strength, durability, protection, aesthetics and speed of execution. The mechanical adhesion between metal profiles and reinforced concrete does not exist in its natural state, this solidarization is obtained through the connecting organs called the shear connectors.

VIII.2. Definition of a composite slab:

A composite slab is a building element consisting of a cold profiled steel sheet covered with compression concrete, a reinforcement mesh resting on the supporting structure of the building composed of metal beams.

In this type of slab, the profiled sheet has several functions, it serves in particular:

- Work platform during the construction of the structure.
- Formwork during the putting of the concrete of the slab.
- lower reinforcement of the mixed slab.

To ensure the steel-concrete bond, the sheet must be able to transmit the shear flow to the interface between the sheet and the concrete.

This mechanical connection can be achieved in the following ways:

- By the re-entrant shape of the ribs, creating a bond by friction.
- By anchors put in the ends of the slab, constituted of welded shear connectors

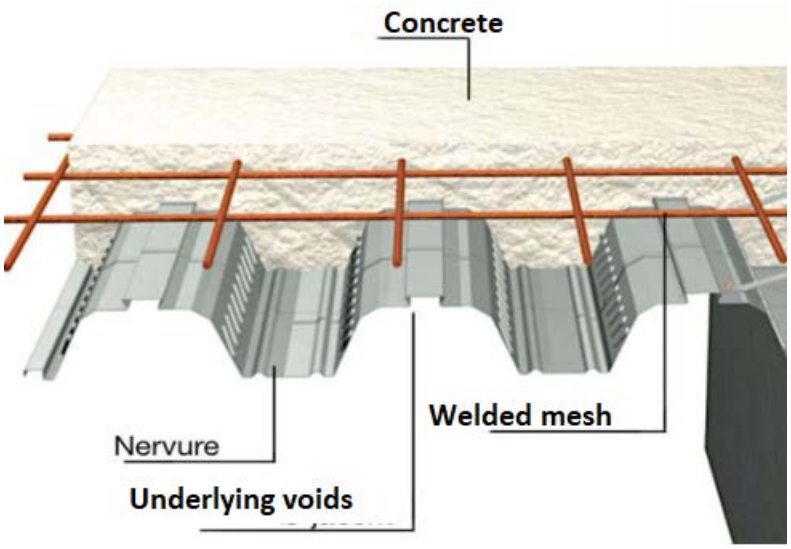


Figure VIII. 14 : The component elements of the mixed slab.

- The benefits of this constructive system are:
  - portable manual product.
  - Speed of installation.
  - Easy to adjust to the dimensions of the floor and through elements thanks to a length manufacturing and simplicity of cutting.
  - After the installation of the cofrastra 40 and before the casting, a temporary floor is obtained that provides traffic safety during the construction phase of the buildings.
  - The metal plates, once fixed, provide a seal in the pouring phase with regard to the laitance of the concrete.
  - Low concrete consumption, for performance equivalent to a standard floor.
  - Advantages of reinforced concrete: Robustness, circulation comfort, reception of any floor or waterproofing coating, duct passages, ease of hanging ceilings, acoustic screen, fire resistance, ...

- The slab is calculated in two phases for the study of the collaborating slab:
  - **Construction phase**: this is for the verification of the profiled sheet during concreting.
  - **Final phase**: this is the verification of the mixed slab after hardening the concrete.

VIII.3. Verification of profiled steel sheet used as formwork:

VIII.3.1 Construction phase:

VIII.3.1.1. Characteristic of the sheet:

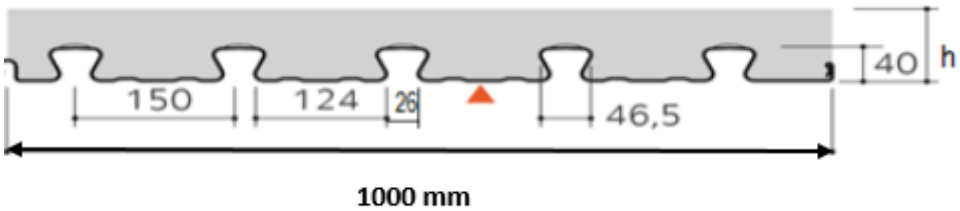


Figure VIII. 15 : Dimension of the ribbed sheet cofrastra 40 (0,85 mm).

VIII.3.1.2. Verification of the deflection of the sheet:

We have to verify that :

$$f_{max} = k.\frac{5}{384}.p.l^4 \frac{1}{E\,I_y} \leq \bar{f} = \frac{l}{180}$$

$$p_{p.acier} = A_{eff}\,.\gamma_{acier} = 1359.10^{-6}.7860 = 10{,}68\,kg/m^2$$

$$p_{p.béton} = \gamma_{béton} \times (0{,}06 + \frac{0{,}04}{2}) = 2500 \times (0{,}06 + \frac{0{,}04}{2}) = 200\,kg/m^2$$

$$p = (p_{p.acier} + p_{p.béton}) \times 1ml = (10{,}68 + 200) \times 1 = 210{,}68\,kg/m = 2{,}11\,KN/m$$

$$f_{max} = k.\frac{5}{384}.p.l^4 \frac{1}{E\,I_y} = 1 \times \frac{5}{384} \times 2{,}11 \times 1{,}5^4 \times \frac{1}{2{,}1 \times 10^8 \times 20{,}77 \times 10^{-8}} = 3{,}18 \times 10^{-3} = 3{,}18\,mm$$

$$\bar{f} = \frac{1500}{180} = 8.33mm$$

$$f_{max} = 3.18mm \leq \bar{f} = 8.33mm$$

$$f_{max} = 3{,}18\,mm \leq \frac{h_c}{10} = \frac{100}{10} = 10\,mm \rightarrow \text{No tidal effect}$$

VIII.4.Final phase:

VIII.4.1. Verification of the mixed section standard floor floor (slab + joist):

For the floor of the current floor the thickness of the slab is 10 cm and the joists are made of IPE 300 profile.

VIII.4.1.1. Verification of positive flexural strength (type I failure):

➢ **Width of the participating slab:**

$b_{eff} = b_{e1} + b_{e2}$

$\mathbf{b_{ei}} = \min(\frac{l_0}{8}; b_i)$

Where:

L_0 : The span of the joist (L_0 =7.8 m).

$b1 = b2 = \frac{1,5}{2} = 0,75$ m

Thus:

$be_1 = be_2 = min(\frac{7.8}{8}; 0,75) = 0,75$ m

$b_{eff} = b_{e1} + b_{e2} = 0,75 + 0,75 = 1,5$ m

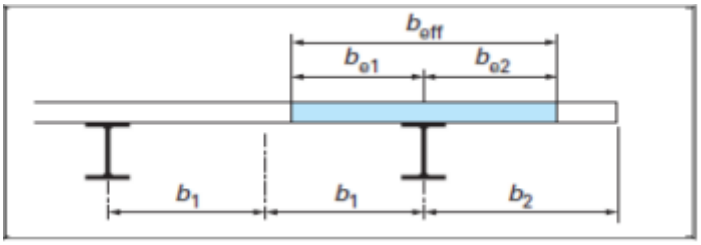


Figure VIII. 16 : Effective length.

➢ **Position of the plastic neutral axis:**

$$F_a = A_a \times \frac{f_y}{\gamma_M}$$

$$F_c = b_{eff} \times h_c \times (0,85 \times \frac{f_{ck}}{\gamma_c})$$

Where :

F_a et F_c the plastic strengths of the tensile profiles and the compression slab.

- h_c = 100 mm
- f_{ck} =25 MPa
- f_y = 235 MPa
- γ_c = 1,5
- γ_M =1,1

- A_a = 5380 mm ^{2} Area IPE 300.

Therefore:

$$F_a = A_a \times \frac{f_y}{\gamma_M} = 5380 \times \frac{235}{1.1} = 1149.36KN$$

$$F_c = b_{eff} \times h_c \times (0{,}85 \times \frac{f_{ck}}{\gamma_c}) = 1500 \times 100 \times (0.85 \times \frac{25}{1{,}5}) = 2125 \text{ KN}$$

$F_a < F_c \rightarrow$ **So the neutral axis is in the concrete section.**

➢ **The position of the neutral axis:**

$$Z_{pl} = \frac{\frac{A_{eff} \cdot f_{yp}}{\gamma_{ap}}}{b_{eff} \times 0{,}85 \times \frac{f_{ck}}{\gamma_c}} \leq hc$$

where : $\gamma_{ap} = 1$

$f_{yp} = 330 \text{ N/mm}^2$ $f_{ck} = 320 \text{ MPa}$

hp : height of the nervures. hc : thickness of the concrete slab.

$$Z_{pl} = \frac{\frac{1359.330}{1}}{1500.0;85.\frac{25}{1{,}5}} = 21{,}10 \text{ mm} < 100 \text{ mm}$$

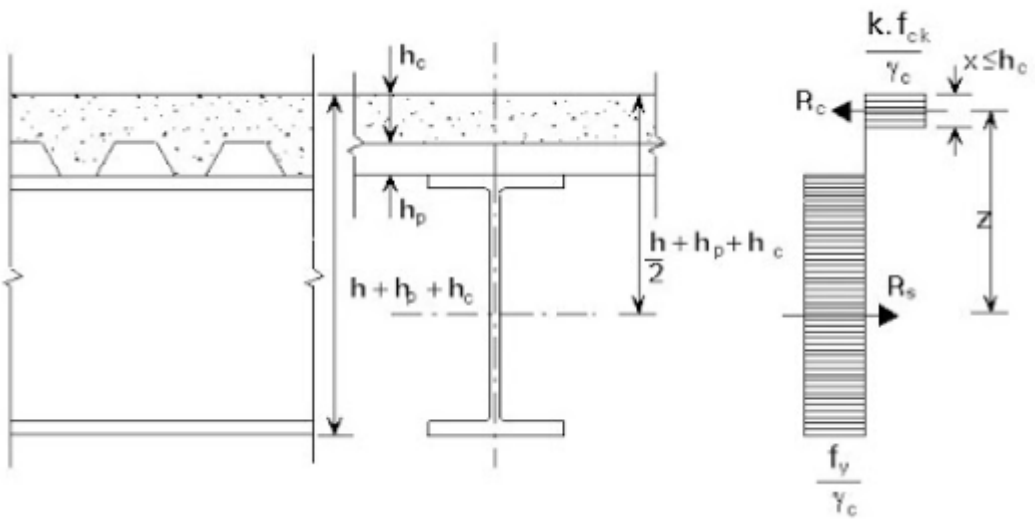


Figure VIII. 17 : Plastic distribution of stresses with neutral axis in the slab.

VIII.4.1.2. Verification of positive flexural strength (type I failure):

It must be verified that: $M_{sd} \leq M_{pl\,Rd}$

$$M_{pl\,Rd} = \left(0{,}85 \times \frac{f_{ck}}{\gamma_c} \times b_{eff} \times z_{pl}\right)\frac{z_{pl}}{2} + \left(A_{eff} \cdot \frac{f_{yp}}{\gamma_{ap}} \times \left(h_c + h_p - e - Z_{pl}\right)\right)$$

$$M_{pl.Rd} = \left(0{,}85 \times \frac{25}{1{,}5} \times 1500 \times 21{,}10\right)\frac{21{,}10}{2} + 1359 \times \frac{330}{1} \times \left(100 + 40 - 21{,}10 - 14{,}2\right) = 51{,}68 \text{ KN.m}$$

- **Load:**

$q_{ELU} = 1{,}35(G_{floor} + p_{p\,joist}) + 1{,}5\,Q_{floor}$

$q_{ELU} = [1{,}35 \times G_{étage} + 1{,}5 \times Qétage] \times 1{,}5 + 1{,}35 \times p_{p\,solive}$

$q_{ELU} = [1{,}35 \times 3{,}87 + 1{,}5 \times 1{,}5] \times 1{,}5 + (1{,}35 \times 0{,}422) = 11{,}78$ KN/ml

- **Soliciting moment**:

$$M_{sd} = 0{,}622 \times \frac{q_{ELU} \times l^2}{8}$$

$$M_0 = \frac{11{,}78 \times 1{,}5^2}{8} = 3{,}31 \text{ KN.m}$$

$M_{sd} = 2{,}06$ KN.m $\leq M_{pl.\,Rd} = 51{,}68$ KN.m……condition verified.

VIII.4.1.3. Verification of negative flexural strength (Type I ruin):

- **Reinforcement:**

$$\rho_l = \left(\frac{A_s}{b_w \cdot d_p}\right) \geq 0{,}04\,\%$$

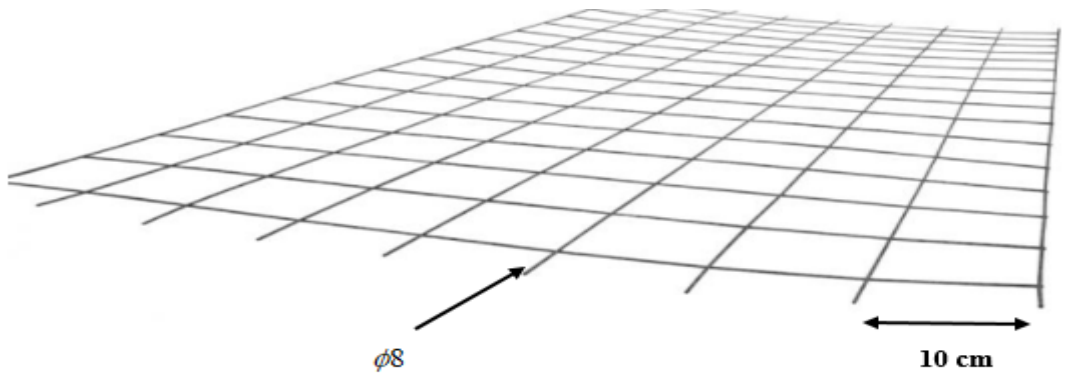
$$A_s \geq 0{,}004 \times 1000 \times 100$$

$$A_s \geq 400\,mm^2$$

Thus we choose : $\varphi 8$ et mesh 10 cm.

$A_s = 502{,}4\ mm^2$

And the coating: C=3 cm.



➢ **Position of l'ANP :**

$$b_c\left(0{,}85\frac{f_{ck}}{\gamma_c}\right)Z_{PL} = A_s\frac{f_s}{\gamma_s}$$

$$b_c = \frac{b_{eff}}{b}b_0 = \frac{1500}{150}\times 103{,}5 = 1035\,mm$$

$$Z_{PL} = A_s\frac{f_s}{\gamma_s}\times\frac{\gamma_c}{0{,}85\,f_{ck}b_c} = 502{,}4\times\frac{400}{1{,}15}\times\frac{1{,}5}{0{,}85\times 25\times 1035} = 11{,}92mm$$

$$M_{pl\,Rd} = \left(0{,}85\times\frac{f_{ck}}{\gamma_c}\times b_{eff}\times z_{pl}\right)\frac{z_{pl}}{2} + \left(As\times\frac{f_{ys}}{\gamma_s}\left(h_c + h_p - Z_{pl} - c\right)\right)$$

$$M_{plRd} = \left(0{,}85\times\frac{25}{1{,}5}\times 1500\times 11{,}92\right)\frac{11{,}92}{2} + \left(502{,}4\times\frac{400}{1{,}15}\times\left(100+40-11{,}92-30\right)\right) = 18{,}64\ KN.m$$

M_{sd}=2,06 KN.m

M_{sd} = 2,06 KN.m $\leq M_{pl.Rd}$ = 18,64 KN.m……condition verified.

VIII.4.1.4 . Longitudinal shear strength (Type II ruin)

We must verify that:

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

Where :

$$V_{pl.Rd} = b_{eff}\times d_p\times\left(m\times\frac{A_p}{b_{eff}\times L_s} + K\right)\times\frac{1}{\gamma_s}$$

$$d_p = h_c + h_p - e = 100 + 40 - 14{,}2 = 125{,}8\,mm$$

L_s: span between two points of zero moment.

$L_s = \frac{L}{4}$ =0,375 m

$$V_{pl.Rd} = 1500 \times 125{,}8 \times \left(276 \times \frac{1359}{1500 \times 375} + 0{,}128\right) \times \frac{1}{1{,}15} = 130{,}41\,KN$$

$$V_{sd} = \frac{q_{ELS} \times l}{2} = \frac{11.78 \times 1.5}{2} = 8.83KN$$

$V_{sd} = 8.83\text{KN} < V_{plrd} = 130.41KN$.....condition verified.

VIII.4.1.5.Vertical shear strength (Type III ruin):

$$V_{rd,c} = \max(0{,}15\,K(100\,\rho_1 \times f_{ck})^{1/3}; \gamma_{min})\, b_c\, d_p$$

$$b_c = \frac{1500}{b} \times b_o = \frac{1500}{150} \times 103{,}5 = 1035\,mm$$

$$\rho_1 = \min\left(\frac{A_p}{b_w \cdot d_p}; 0{,}02\right) = \min\left(\frac{1359}{1035 \times 125{,}8}; 0{,}02\right) = 0{,}01$$

$$K = \min\left(1 + \sqrt{\frac{200}{d_p}}; 2\right) = \min(2{,}26; 2) = 2$$

$$\gamma_{min} = 0{,}035\,K\,2\sqrt{f_{ck}} = 0{,}035 \times 2^{2/3}\sqrt{25} = 0{,}495$$

$$V_{rd,c} = \max(0{,}15 \times 2(100 \times 0{,}010 \times 25)^{1/3}; 0{,}495)1035 \times 125{,}8 = 114{,}21\,KN$$

$V_{sd} = 8.83\text{KN} < V_{Rd,c} = 130.41KN$.....condition verified.

VIII.5. Study of connectors:

To ensure the steel-concrete bond, the sheet must be able to transmit the shear flow to the interface between the sheet and the concrete, This mechanical connection can be made by welded shear connectors.

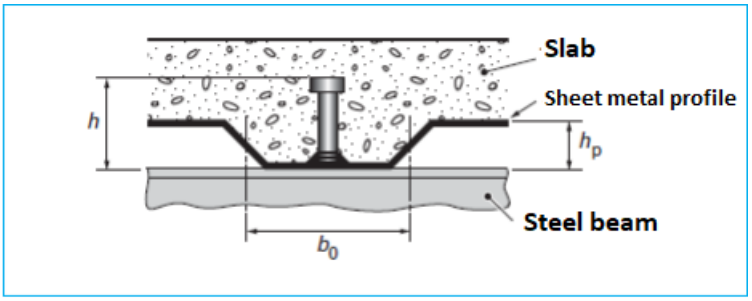


Figure VIII. 18 : Steel-concrete connection.

- We used the **TRW NELSON KB3/4 -80** shear connectors.

VIII.5.1. The characteristics of the connectors are as follows:

$$d_{fut} = 19\,mm$$

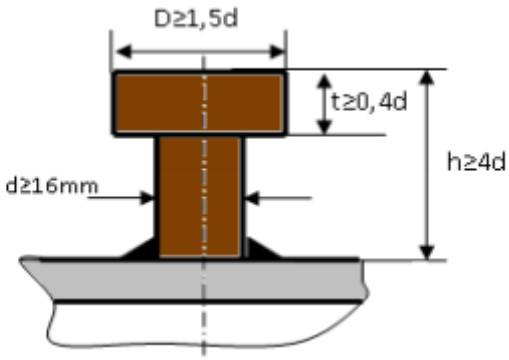


Figure VIII. 19 : Welded shear connector.

$$d_{tete} = 31{,}7\,mm$$

$$h_{t,tete} = 10\,mm$$

$$h_{sc} = 80\,mm$$

$$f_y = 350\,N/mm^2$$

$$f_u = 450\,N/mm^2$$

VIII.5.2. Verification of resistance of connectors:

$$p_{Rd} = \min(p_{rd,1}; p_{Rd,2}; p_{Rd,3})$$

Where :

- $p_{Rd,1} = 0{,}8.K.f_u.\left(\frac{\pi.d^2}{4}\right).\frac{1}{\gamma_v}$ → **Steel shear.**
- $p_{Rd,2} = 0{,}29.K.\alpha.d^2.\sqrt{f_{ck}.E_{cm}}.\frac{1}{\gamma_v}$ → **Concrete ruin.**
- $p_{Rd,3} = K_\varphi.\left(\frac{d_{d0}.t.f_{yp}}{\gamma_{ap}}\right)$ → **Diametric pressure.**

Where :

γ_v : Partial safety coefficient for connectors taken equal to 1.25.

d : Diameter d shank of the shear-connector.

$f_{u:}$: Ultimate tensile strength of shear-connector steel.

E_{cm} : 28000 Mpa for a concrete of class C25/30.

f_{ck} : Characteristic strength of concrete equal to 25Mpa .

α **:** Corrective factor.

K : Rib parallel to the shear force: $K = K_L = 0{,}60.\frac{b_0}{h_p}.\left(\frac{h_{sc}}{h_p} - 1\right) \leq 1$

$$p_{Rd,1} = 0{,}8.K.450.\left(\frac{\pi.19^2}{4}\right).\frac{1}{1{,}25}$$

Where : $K = K_L = 0{,}60.\frac{103{,}5}{40}.\left(\frac{80}{40} - 1\right) = 1{,}55 \succ 1$ **……**we take**:** $K = 1$

$$p_{Rd,1} = 0{,}8.1.450.\left(\frac{\pi.19^2}{4}\right).\frac{1}{1{,}25} = 81{,}66\,KN$$

$$p_{Rd,2} = 0{,}29.K.\alpha.d^2.\sqrt{f_{ck}.E_{cm}}.\frac{1}{\gamma_v}$$

Where : $\frac{h_{sc}}{d_{fut}} = \frac{80}{19} = 4{,}21 \geq 4 \Rightarrow \alpha = 1$

$$p_{Rd,2} = 0{,}29 \times 1{,}1 \times 19^2 \times \sqrt{25 \times 28000} \times \frac{1}{1{,}25} = 70{,}07\,KN$$

$$p_{Rd,3} = K_\varphi \times \left(\frac{d_{d0}.t.f_{yp}}{\gamma_{ap}}\right)$$

Where : $d_{d0} = 1{,}1 \times 19$

a : Distance between the shear-connector and the extremity of the sheet $\Rightarrow a \geq 1{,}5.d_{d0}$

$$K_\varphi = 1 + \frac{a}{d_{d0}} = 1 + \frac{31{,}25}{20{,}9} = 2{,}5$$

$$p_{Rd,3} = K_{\varphi} . \frac{d_{d0} . t . f_{yp}}{\gamma_{ap}} = 2,5. \frac{1,1 \times 19 \times 0,85 \times 330}{1} = 14,65\,KN$$

$$\Rightarrow p_{Rd} = \min(p_{rd,1}; p_{Rd,2}; p_{Rd,3}) = \min(81.667; 0.07; 14.65) = \mathbf{14.65}\boldsymbol{KN}$$

$p_{.Rd} = 14,65\,KN \succ V_{sd} = 8,90\,KN$ .....condition verified

So it is not necessary to increase the number of shear-connectors by 1 ml.

VIII.5.3. Number of the connectors :

$$n = \frac{V1}{p_{rd}}$$

$$V1 = \min(F_a, F_c) = \min(1553;2125) = 1553,13\,KN$$

$$n = \frac{1553,13}{70,07} = 22,16$$

We then take 25 connectors for each main beam

VIII.5.4. Spacing between connectors:

Let S be the spacing between the connectors given by the following formula:

$$S = \frac{l}{n-1} = \frac{1200}{25-1} = 50\,cm$$

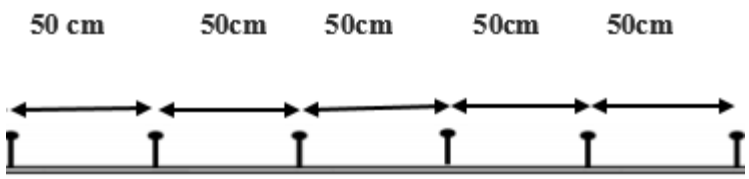


Figure VIII. 20 : Spacing between the shear connectors.

VIII.6. Verification of the mixed section plank current floor (slab + beam):

VIII.6.1. Final Phase:

For the standard floors the thickness of the slab is 10 cm and the main beams are made of HEB450 section.

VIII.6.1.1. Verification of the resistance in positive bending (type I ruin):

- **Width of the participating slab:**

$b_{eff} = b_{e1} + b_{e2}$

$b_{ei} = \min(\frac{l_0}{8}; b_i)$

Where:

L_0 : The span of the main beam (L_0=9.8m)

$b1 = \frac{7.8}{2} = 3.9m$

$b1 = \frac{6.4}{2} = 3.2m$

Thus:

$$be_1 = min\left(\frac{9.8}{2}; 3.9\right) = \min(4.9; 3.9) = 3.9m$$

$$be_2 = min\left(\frac{9.8}{2}; 3.2\right) = \min(4.9; 3.2) = 3.2m$$

$$b_{eff} = be_1 + be_2 = 7.1m$$

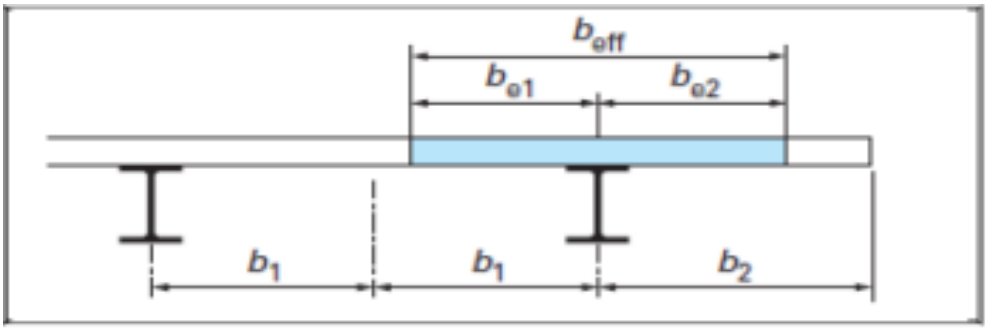


Figure VIII. 21 : Effective length.

- **Position of the plastic neutral axis:**

$$F_a = A_a \times \frac{f_y}{\gamma_M}$$

$$F_c = b_{eff} \times h_c \times (0{,}85 \times \frac{f_{ck}}{\gamma_c})$$

Where:

F_a et F_c the plastic strengths of the tensile sections and the compression slab.

- h_c = 100 mm.
- f_{ck}=25 Mpa.
- f_y = 235 Mpa.
- γ_c = 1,5.
- γ_M =1,1.
- A_a = 218 ×10^2 mm^2 Area of HEB 450 section.

$$F_a = A_a \times \frac{f_y}{\gamma_M} = 218 \times 10^2 \times \frac{235}{1.1} = 4657.272KN$$

$$F_c = b_{eff} \times h_c \times (0.85 \times \frac{f_{ck}}{\gamma_c}) = 7100 \times 100 \times (0.85 \times \frac{235}{1.1}) = 128929545.KN$$

$F_a < F_c$ Thus : The neutral axis is in the steel section.

Thus the neutral axis is found in the concrete section of the beam

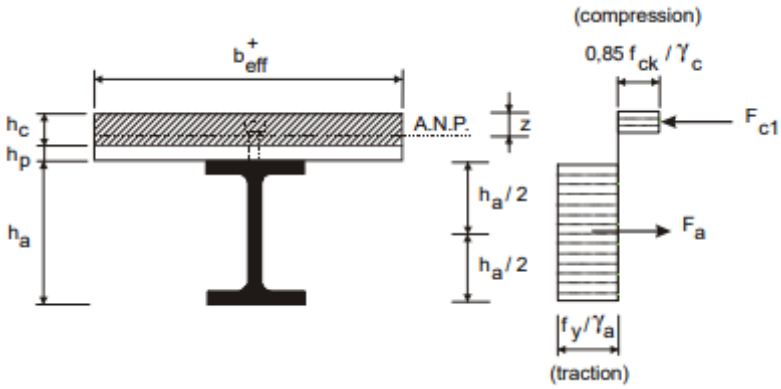


Figure VIII. 22 : Plastic distribution of normal stresses case of the plastic neutral axis in the slab (positive bending).

So the plastic resistant moment developed by the mixed section is:

$$M_{pl.Rd} = F_{acier}(0,5h_a + h_c + h_p - 0,5z)$$

$$z = \frac{F_a}{b_{eff} \times 0,85 \times f_{ck}/\gamma_c}$$

VIII.6.1.2. Verification of Positive Moment of Plastic Resistance (Type I Failure):

It must be verified that : $M_{sd} \leq M_{pl.Rd}$

Applied bending moment :

$M_{sd} = 483.04KNm$

Resistant plastic moment developed by the mixed section:

$$z = \frac{F_a}{b_{eff} \times 0.85 \times \frac{f_{ck}}{\gamma_c}} = \frac{4657.272}{7100 \times 0.85 \times \frac{25}{1.5}} = 46.30mm$$

$M_{pl.Rd} = 4657.272$(0.5x450 +100+40-0.5x 46.30)=1592.088KN.m

$M_{sd\ span} = 483.04KNm < M_{plrd} = 1592.088KNm$ ……condition verified

VIII.6.1.3. Verification of negative moment of plastic resistance (Type I ruin):

- **Position of the neutral plastic axis of the negative moment:**

$F_s = A_a \times \frac{f_{sK}}{\gamma_s}$ where : f_{5k} = 400 MPa et γ_s=1.15

$$F_s = 218 \times 10^2 \times \frac{400}{1.15} = 7582.6KN$$

$$F_a = 4657.272KN < F_s = 7582.6KN$$

$F_a - F_s \leq 2b.t_f.f_{yd}$

$F_a - F_s = -2925.328KN$

$2b.t_f.f_{yd} = 2 \times 300 \times 26 \times 235 = 3666.0KN$

$F_a - F_c = 2925.328KN \leq 2b.t_f.f_{yd} = 3666.0KN$ …….. Condition verified

Thus : The plastic neutral axis is in the steel superior flange

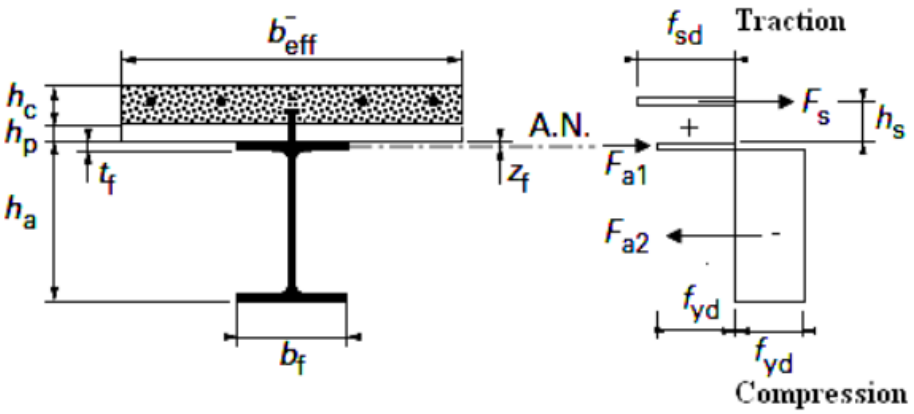


Figure VIII. 23 : Stress distribution with plastic neutral axis in the superior flange

The plastic resistant moment developed by the mixed section is:

$$M_{plrd}^{-} = Fa\left(\frac{h_a}{2} + h_s\right) - (F_a - F_c)(\frac{z_f}{2 + h_f})$$

The thickness z_f of the flange in tension is deduced from the following equilibrium equation:

$$F_a = F_s + 2b_f.z_f.fy_d$$

Therefore $z = \frac{F_a - F_s}{2b_f - fy_d}$

$$z = \frac{F_a - F_s}{2b_f - fy_d} = -\frac{2925.328 \times 10^3}{2 \times 300 - 235} = -8.014mm$$

$$M_{plrd}^{-} = 4657.272 \times 10^3\left(\frac{450}{2} + 109.6 \times 10^{-3}\right) - ((4657.27 - 3456) \times 10^3)(\frac{-8.0}{2 + 0.055})$$

M_{plrd}^{-} =1053.08KNm

$M_{sd\ span} = 483.04KNm < M_{plrd}^{-} = 1053.08$KNm……condition verified

VIII.6.1.4. Shear verification:

It must be verified that: $V_{Sd} \leq V_{plRd} = \frac{f_y A_v}{\sqrt{3}\ \gamma_{M_0}}$

Where:

$V_{pl.Rd}$: plastification of the transverse shear force of the section

A_v : shear area.

$A_v = A - 2.b.t_f + (t_w + 2.r).t_f$

A_v = 21800 – 2x300x26+(124+ 2x27) x26

A_v = 7968mm^2.

$V_{pl.Rd} = \frac{235 \times 7968}{\sqrt{3} \times 1} = 1081.07KN$

$V_{sd} = 451.61KN < V_{pl.Rd} = 1081.07KN$ ……Condition verified.

VIII.6.1.5. Verification of the interaction of the transverse shear force:

$V_{sd} = 451.61KN < 0.5V_{pl.Rd} = 540.535KN$

There is no interaction between bending force and the transverse shear force. Both verifications are done independently.

VIII.6.1.6. Verification of the deflection condition:

It must be verified that: $f^{max} \leq \bar{f}$

$$z = \frac{A_a . h_a + \frac{b^+_{eff} . h_c^2}{2n}}{A_a + \frac{b^+_{eff} . h_c}{n}}$$

$$I_C = I_a + \frac{b_{eff}^+ . h_c^3}{12n} + A_a (h_a - z)^2 + \frac{b_{eff}^+ . h_c}{n} (z - \frac{h_c}{2})^2$$

Avec:

I_a : Moment of inertia of the Joist ($I_a = I_y$)

n : Equivalence coefficient with n= Ea /Ec'

Ea : Elasticity modulus of structural steel (Ea= 210000 MPa)

Ec' : Concrete equivalence modulus where $Ec' = Ecm/2$

E cm = 31000 MPa For a class concrete C25/30

$$n = \frac{210000}{31000/2} = 13.54$$

$$z = \frac{218{,}6\times10^2\times360+\frac{2440.100^2}{2\times13{,}54}}{218.6\times10^2+\frac{2440\times.100}{13{,}54}} = 268.981\text{mm}$$

$$I_C = 79890 \times 10^4 + \frac{2440 \times 100^3}{12 \times 13.54} + 218 \times 10^2 (100 - 268.981)^2 + \frac{2440 \times 100}{13.54} (268.981 - \frac{100}{2})^2$$

$$I_C = 230054.68 \times 10^4 mm^4$$

The value of the distributed load deflection is:

$$f_q = \frac{q_s \times L^4}{384 \times EI_c} = \frac{47.71 \times 10^3 \times 9800^4}{384 \times 2.1 \times 10^8 \times 230054.68 \times 10^4} = 2.37mm$$

The value of the deflection for the distributed load is:

$$f_p = \frac{n \times p \times L^3}{76.8 \times EI_c} = \frac{\frac{9.8}{1.5} \times 0.422 \times 10^3 \times 7800^3}{76.8 \times 2.1 \times 10^8 \times 230054.68 \times 10^4} = 0.352 \times 10^{-4} mm$$

The value of the max deflection is:

$$f^{max} = f_q + f_p = 2.37 + 0.352 \times 10^{-4} = 2.77mm$$

The value of the allowable deflection is:

$$\bar{f} = \frac{L}{250} = \frac{9800}{250} = 39.2mm$$

$f^{max} = 2.77mm \leq \bar{f} = 39.2mm$..... The deflection condition of the arrow is verified.

VIII.7. Calculation of the parapet

VIII.7.1 Introduction:

The parapet is a secondary element in reinforced concrete whose realization is necessary to ensure the waterproofing of inaccessible terraces.

The parapet is a fixed console on the terrace floor. The calculation is done at the compound bending, for a strip of 1 m in length. The normal effort caused by the own weight and the moment of flexion caused by the handrail.

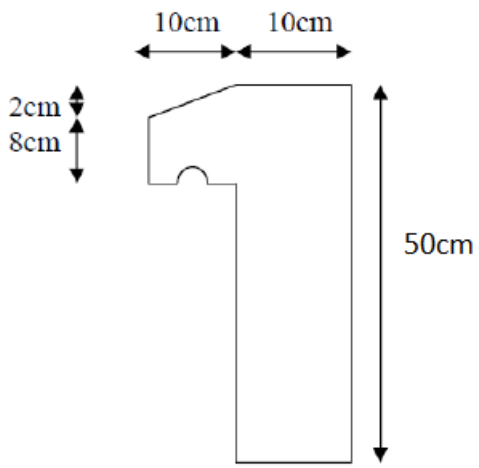


Figure VIII. 24 : Static diagram of the parapet

VIII.7.2. Earthquake calculation

According to the [RPA 99 version 2003, art 6-2-3] the parapet is considered to be a non-structural element on which acts a horizontal force "FP" due to the earthquake calculated according to the formula:

$$F_p = 4 \times A \times C_p \times W_p$$

$$F_p = 4 \times 0.10 \times 0.8 \times 1.725 = 0.552KN/ml < Q = 1KN/ml$$

Where:

$\boldsymbol{A = 0.08}$: acceleration coefficient (zone I, groupe 2 selon le tableau (4.1 [2])

$\boldsymbol{C_p = 0,8}$: acceleration coefficient (according to Table 6.1 of the RPA).

Wp=Pp=1.475KN/ml : Weight of the parapet

The Live load Q of the parapet (due to a handrail) is therefore more preponderant than the action of the force due to the earthquake, so we take Q = 1KN / ml

VIII.7.3. Sollicitations on the parapet G

The sollicitaions that act on the secondary element **Q** non-structural parapet are:

Permanent load (self-weight) $G_{parapet}$=1.475KN

Live load (handrail) Q = 1KN/ml

- **ELU :**

$N_u = 1.35 \times 1.475 = 1.9912 KN/ml$

$N_u = 1.5 \times Q \times h = 1.5 \times 1 \times 0.5 = 0.75 KNm$

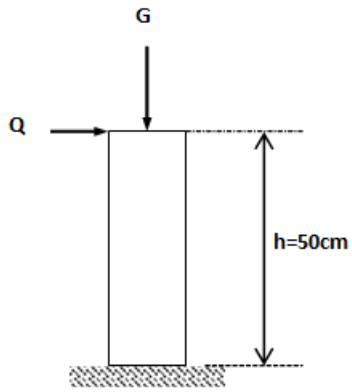


Figure VIII. 25 : Loads on the Parapet.

Eccentricity of the 1st order:

$$e_1 = \frac{Mu}{Nu} + e_0 = \frac{0.75}{1.991} + 0.02 = 0.39m$$

Where :

$$e_0 = max(2\,cm; \frac{h}{250}) = \left(2cm; \frac{50}{250}\right) = (2; 0.2) = 2cm$$

Eccentricity of the 2nd order (fixed-free)

$$e_2 = \frac{3l_f^2}{10^4}(2 + \phi\alpha) = \frac{3 \times 1.0^2}{10^4 \times 0.1} \times (2 + 0.2 \times 0.59 \times 0.2) = 0.0063$$

Where :

Lf=2l=2x0.5=1m

φ =0.2

$$\alpha = \frac{G}{G + Q} = \frac{1.475}{1.475 + 1} = 0.59$$

- **Corrected stresses for compound bending calculation:**

$N_u = 1.35 \times 1.475 = 1.9912 KN/ml$

$M_u = 1.991(0.39 + 0.00631) = 0.789 KN/ml$

- **Corrected stresses for compound bending calculation:**

$$e_A = (e_1 + e_2) + \left(d - \frac{b}{2}\right) = (0.41 + 0.0092) + \left(0.09 - \frac{0.1}{2}\right) = 0.459m$$

$M_{uA} = N_u e_A = 1.991 \times 0.436 = 0.868 KNm$

ELS :

$N_s = 1.475 KN/ml$

$M_s = Q \times h = 1 \times 0.5 = 0.5 KNm$

$$e_{0s} = \frac{M_s}{N_s} = \frac{0.5}{1.475} + 0.02 = 0.358m$$

- **Sollicitations reduced to the centre of gravity of tensioned steels:**

$$e_A = e_{0s} + \left(d - \frac{b}{2}\right) = 0.358 + \left(0.09 - \frac{0.1}{2}\right) = 0.398m$$

$M_{sA} = N_s \times e_A = 1.475 \times 0.398 = 0.5870 KNm$

VIII.7.4. Reinforcement of the Parapet

BAEL 91 (cf. article 5-3-1) recommends that for elements exposed to the weather, reinforcement is carried out for strips 1m wide.

- **Reduced reference moment:**

$$\mu = \frac{0{,}8h}{d}(1 - 0{,}4\frac{h}{d}) = \frac{0{,}8 \times 0{,}1}{0{,}09}(1 - 0{,}4\frac{0{,}1}{0{,}09}) = 0{,}493$$

- **Reduced acting moment:**

$$\mu_{\mu} = \frac{M_{uA}}{b.d^2.fbc} = \frac{0.00086}{1 \times 0.09^2 \times 14.16} = 0.00749$$

Where :

$$f_{bc} = \frac{0{,}85 \times f_{c28}}{\gamma_b} = 14{,}16\,\text{MPa}$$

$$\text{d} = 0{,}9\,\text{h} = 0{,}09\,\text{m}$$

In this case $\mu_u < \mu_R = 0{,}493 \longrightarrow$ Section with simple reinforcements

$$A_{st} = \frac{M_{uA}}{z \times \sigma_{st}} = \frac{0.00086}{0.09 \times 348} = 0.274 cm^2/ml$$

Where:

$z = d \times (1 - 0.4\alpha) = 0{,}09 \times (1 - 0.4 \times 0{,}0093) = 0{,}089m$

$\alpha = 1{,}25 \times \left(1 - \sqrt{1 - 2\mu}\right) = 1.25 \times (1 - \sqrt{1 - (2 \times 0.00749)})$=0.0093

$$\sigma_{st} = \frac{f_e}{\gamma_s} = \frac{400}{1{,}15} = 348\,\text{MPa}$$

The minimum reinforcement section:

$$A_{min} = \max\left(\frac{4}{perimeter}; \frac{0.28}{100}\right) = \max\left(\frac{4}{12.96}; \frac{0.28}{100}\right) = \max(0.308; 0.0028)$$
$$= 0.308 cm^2/ml$$

So we opt for longitudinal reinforcements 3T5 with $A_{st} = 0.59 cm^2/ml$

for the distribution reinforcements we have:

$A_{st} = \frac{A_T}{4} = \frac{0.59}{4} = 0.14 cm^2$ So we opt for distribution reinforcements 3T5 with

$A_{st} = 0.59 cm^2/ml$

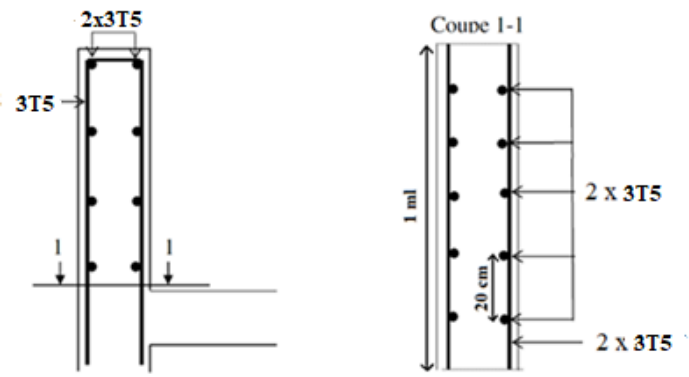


Figure VIII. 26 : Arrangement of reinforcements in the parapet.

VIII.8. Conclusion :

The calculation of the various elements of the composite floor uses the rules of EURECODE 4. It takes into account the mechanical advantages of concrete (good compressive strength) and steel (good tensile strength).

Chapter 9:
Connections

IX.1. Introduction :

The essential characteristic of metal frames is that they are composed of elements elaborated in different places and times that are then assembled on the construction site. The links thus have a dual role: to allow the construction of a spatial structure and to ensure the reliability and stability of this structure.

IX.2. Principe :

The connections serve to bring together two or more parts by ensuring the proper transmission of efforts. Without going into the calculations of verification of the strength of the assemblies, compliance with a few simple rules and the choice of good constructive arrangements will give satisfactory results.

To make a metal structure, individual parts are available, which must be assembled:

Either end to end (connection with fishplates, splice joint).

Either concurrent (beam / post fasteners, lattices and cross-linked systems).

To conduct calculations according to classical schemes of material strength, a distinction must be made between the connections:

Articulated connections, which transmit only transverse shear forces.

Rigid connections, which also transmit the various moments.

IX.2. Modes of executions of the connections:

The modes of execution of the connections are:

- Welded connections
- Bolted connections
- Riveted connections
- Glued assemblies.

In our work, bolted and welded mode of connections are used.

IX.2.1. Bolting:

Bolting is the means of connection frequently used in metal framing because of its easy use, while allowing adequate rules.

In our study we chose high strength HR bolting comprising a threaded rod with hexagonal head.

The table shows the characteristics of the bolts in Appendix C.

IX.2.2. Welding: Welding is a permanent connection process. Its purpose is to ensure the continuity of the material to be assembled.

IX.3. Calculation of the connections:

IX.3.1. Column-beam connection :(HEB650, HEB450)

The column-beam connection is of the rigid type, consists of welding a plate at the end of the beam, symmetrically pierced on either side of the beam to allow its assembly with the post.

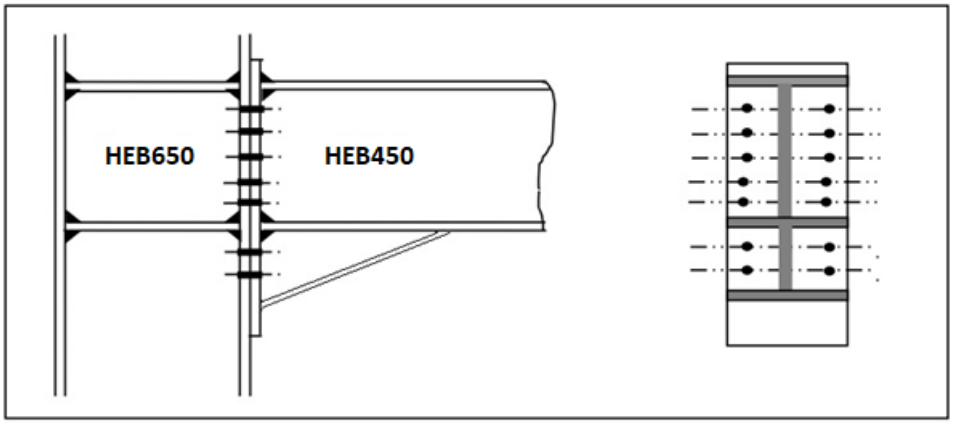


Figure IX. 7 : Column-beam assembly.

- **Beam HEB 450:**

h=450 mm ; b=300mm ; t_f=26mm ; t_w=14mm

A=218 cm^2

- **Solliciting forces** :

We use the efforts taken by the Robot software:

V_{sd} =451.61KN

M_{sd}=813.71 KN.m

IX.3.1.1. Verification of the beam- metal plate:

It will be assumed that the moment is taken only by the welded seam connecting the flange to the metal plate, It will also be assumed that the transverse shear force is taken up only by the seams connecting the web to the metal plate. This assembly is subject to the system of the following forces:

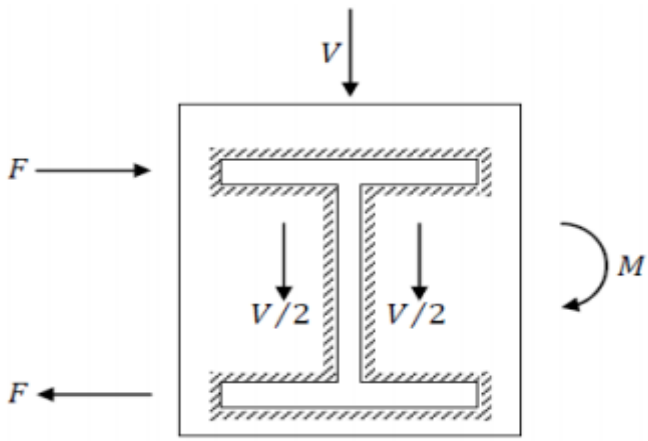


Figure IX. 8 : Assembly by welding beam-metal plate.

- **Welded seam**:

To choose the right gorge for HEB 450:

We choose a metal plate (700x300x28) mm.

Plate thickness: e_p= 28 mm→ a_{min}= 7 mm ≤ a ≤ a_{max}= 18 mm

Thickness of the flange of HEB 450:t_f=26mm → a $_{min}$= 6 mm ≤ a ≤ a_{max}=15,5 mm

Thickness of the web of HEB 450 :t_w=14mm → a $_{min}$= 3,25 mm ≤ a ≤ a_{max}=8 mm

We take : a_f= 16 mm

a_w=10mm

- **Verification the weld of the flange to tension:**

We verify : $N_{sd} \leq F_{w.rd}$

$$N_{sd} = \frac{M_{sd}}{h - t_f}$$

$$F_{w.rd} = \frac{a \times \sum l \times f_u}{\sqrt{2} \times \beta_w \times \gamma_{mw}}$$

The steel grade used is Fe360 so: $\beta_w = 0{,}8$, $\gamma_{mw} = 1{,}25$

The length of the flange weld seam:

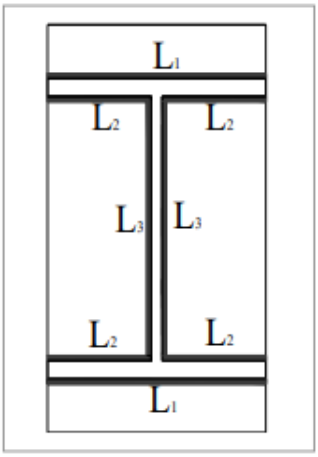


Figure IX. 9 : Beam–platinum weld lengths.

Where:

$l_1 = 300mm$

$l_2 = \frac{b}{12} - \frac{tw}{2} = \frac{300}{12} - \frac{14}{2} = 18mm$

$l_3 = 398mm$

The total length of the weld seam of the flange**:** $\sum l$ =l1+2l2+2tf=388mm

$F_{w.rd} = \frac{a \times \sum l \times f_u}{\sqrt{2} \times ß_w \times ɣ_{mw}} = \frac{16 \times 388 \times 2 \times 360}{\sqrt{2} \times 0.8 \times 1.25} = 3160.597KN$

$N_{sd} = \frac{M_{sd}}{h} = \frac{813.71}{0.45} = 1080.24KN$

$N_{sd} = 1080.24KN \leq F_{wrd} = 3160.597KN$…….. Condition verified

We verify that:

$V_{sd} \leq F_{v.rd}$

Where :

$$F_{v.rd} = \frac{a \times \sum l \times f_u}{\sqrt{3} \times \beta_w \times \gamma_{mw}}$$

The steel grade used is Fe360 so:

$\beta_w = 0{,}8$

$\gamma_{mw} = 1{,}25$

The total length of the welded seam of the flange : $\sum l$ = 2xl3=630mm

$$F_{v.rd} = \frac{a \times \sum l \times f_u}{\sqrt{2} \times ß_w \times ɣ_{mw}} = \frac{10 \times 796 \times 2 \times 360}{\sqrt{2} \times 0.8 \times 1.25} = 1654.45KN$$

$$V_{sd} = 451.61KN \leq F_{vrd} = 1654.45KN$$

IX.3.1.2. Constructive provision:

For practical reasons, bolts of different diameters are always avoided in the same assembly.

- **Choice of bolt diameter:**

The choice of diameter will be made, determining their resistances all being proportional to the thickness of the assembled parts as follows:

t≤ 10mm d=(12 ;14)mm

10≤t≤25mm d= (16 ;20 ;24) mm

t> 25mm d= (24;27;30)=mm

- **The thinnest thickness:**

t = min (tf ; $t_{platine}$) = min (12,5; 26) =12,5 mm

We take two lines of 5 boulons Ø22 classe HR 8.8 : fyb =640MPa

fub = 800MPa

d0 = Ø + 2 = 24 mm

- **Spacing and longitudinal clamp (p1 ;e1) :**

2,2 d0=52,8mm≤ p1 ≤ [14t ;200]= 175mm Thus we take :p1 =60mm.

1,2 d0=mm ≤ e1 ≤ [12t ;150]=150mm Thus we take: e1 =60 mm.

- **Spacing and Transverse clamp (p2 ;e2):**

3 d0 =72mm≤ p2 ≤ [14t ;200]=175 mm Thus we take: p2 =160 mm.

1,5 d0 =mm ≤ e2 ≤ min[12t ;150]=150 mm Thus we take: e2 =70 mm.

We have a plate of dimension (700x300x28) mm.

IX.3.1.3. Determination of bolt forces :

$$N_i = \frac{M.d_i}{\sum d_i^2}$$

d_6=70mm

d_5=190mm

d_4=250mm

d_3=310mm

d_2=370mm

d_1=430mm

$$\sum d_i^{\ 2} = (0{,}070)^2 + (0{,}190)^2 + (0{,}250)^2 + (0{,}310)^2 + (0{,}370)^2 + (0{,}430)^2 = 0{,}522\,m^2$$

$$N_6 = \frac{813.71.\times 0{,}07}{0{,}522\times 2} = 54.55KN$$

$$N_5 = 148.08KN$$

$$N_4 = 194.85KN$$

$$N_3 = 241.618KN$$

$$N_2 = 288.38KN$$

$$N_1 = 335.14KN$$

- The calculation tensile force per bolt at the ELU equals :

$F_{t.sd} = N_1 = 335.14$ KN

- The calculation shear force per bolt at the ELU equals :

$$F_{v.sd} = \frac{N_1}{n} = \frac{451.61}{14} = 32.25KN$$

IX.3.1.4. Calculation of bolt shear strength:

We verify that:

$$F_{v.sd} < F_{v.rd}$$

Where :

$$F_{v.rd} = \frac{\alpha_v \times f_{ub} \times A_s}{\gamma_{M2}}$$

$$\alpha_v = 0{,}6$$

$$F_{v.rd} = \frac{0.6 \times 800 \times 303}{1.25} = 116.35KN$$

$$F_{v.sd} = \frac{N_1}{n} = \frac{451.61}{14} = 32.25KN$$

$F_{v.sd} = 32.25 < F_{v.rd} = 116.35KN$…….. Condition verified

IX.3.1.5. Bolt tensile strength check:

We have to verify that : $F_{t.sd} \leq F_{t.rd}$

$$F_{t.rd} = \frac{0,9 \times f_{ub} \times A_s}{\gamma_{mb}}$$

$$F_{t.rd} = \frac{0,9 \times 800 \times 303}{1,25} = 174,52\,KN$$

$$F_{t.sd} = \frac{335.14}{14} = 23.93$$

$$F_{t.sd} = 23.93KN < F_{t.sd} = 174.52KN$$

IX.3.1.6. Diametric pressure check:

We verify that :

$$F_{b.rd} = \frac{K_1 \times \alpha \times f_u \times d \times t_{ac}}{\gamma_{M2}}$$

Where: $F_{v.rd} \leq F_{b.rd}$

$$\alpha = \min\left(\frac{e_1}{3d_0}; \frac{p_1}{3d_0} - \frac{1}{4}; \frac{f_{ub}}{f_u}; 1\right) = \min(0,83;0,58;2,22;1) = 0,58$$

$$F_{b.rd} = \frac{2.5 \times 0.58 \times 360 \times 22 \times 12.5}{1.25} = 114.840KN$$

$$F_{v.sd} = \frac{335.89}{14} = 23.99$$

$F_{v.sd} = 23.99KN < F_{brd} = 114.840KN$..... condition verified

IX.3.1.7. Slip check (combined traction and shear):

We verify that : $F_{v.sd} < F_{s,Rd,ser}$

$$F_{s,Rd,ser} = \frac{k_s . n . \mu . (F_{p,c} - 0,8 F_{t,Ed,ser})}{\gamma_{M3,ser}}$$

k_s : bolts used in normal holes.

$\mu = 0,5 \Rightarrow$ class of surface A

$$F_{p.c} = 0,7 . f_{ub} . A_s = 0,7 \times 800 \times 303 = 169,86\,KN$$

$$F_{s,Rd,ser} = \frac{1.1 \times 0.5 \times (169.86 - 0.8 \times 23.99)}{1.1} = 75.33KN$$

$$F_{v.sd} = \frac{V_{sd}}{n} = \frac{451.61}{14} = 32.25KN$$

$F_{v.sd} < F_{s,Rd,ser}$ ……condition verified

IX.3.1.8. Column web tensile strength:

$F_t \leq F_{t.rd}$

$$F_{t.rd} = f_y \times t_{wc} \times \frac{b_{eff}}{\gamma_{M0}}$$

t_w : Column web thickness

b_{eff} : row spacing of bolts

$$F_{t.rd} = 235 \times 14 \times \frac{160}{1,25} = 421,12\,KN$$

$$F_t = \frac{M}{h - t_f} = \frac{813.71}{650 - 0.031} = 1314.55KN$$

$F_t = 1314.55 < F_{t.rd} = 421.12KN$…… condition not verified

The condition is not verified, so we add a stiffener of thickness equal to the thickness of the web of the column ep = 14mm.

IX.3.2. Column to column connection (HEB650-HEB650):

The assembly is using cover plate with two vertical wires of HR10.9 bolt, Two cover plates that provide the connection between the webs and two others that connect the ends of the flange of the assembled columns.

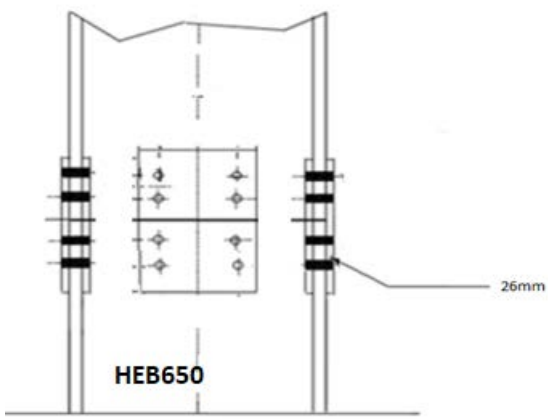


Figure IX. 10 : Column-Column Assembly.

- **Column HEB 650:**

h=650mm; b= 300mm ; t_f=31 mm ;t_w=16 mm

A=286.3 cm^2

- **Solliciting forces**:

We use the forces given by the Robot software under the G+Q-E_y combination.

V_{sd} = 5908.19 KN.

M_{sd}= 0KN.m.

N_{sd}= 87.90 KN.

IX.3.2.1 Verification of the metal plate of the web:

- **Constructive arrangement of the metal plate of the column web:**

- **Choice of bolt diameter:**

The choice of diameter will be made, determining their resistances all being proportional to the thickness of the assembled parts as follows:

t≤ 10mm d=(12 ;14)mm

10≤t≤25mm d= (16 ; 20 ;24)mm

t> 25mm d= (24;27;30)mm

We take two lines of 4 bolts Ø22 class HR10.9: fyb = 900MPa.

fub =1000 MPa.

d_0 = Ø + 2 = 24 mm.

- **Spacing and longitudinal clamp (p1 ;e1) :**

2,2 d0=52,8mm≤ p1 ≤ min [14t ;200] =196mm therefore we take:p1 =120mm.

1,2 d0=28,8mm ≤ e1 ≤ min [12t ; 150] =150mm therefore we take: e1 =50mm.

- **Between axis and transverse clamp (p2 ;e2) :**

1,5 d0 =36mm ≤ e2 ≤ min [12t ;150] =150 mm therefore we take e2 = 90 mm.

3 d0 =72mm≤ p2 ≤min [14t ;200] =196mm therefore we take : p2 =160 mm.

We have a plate of dimension (460x300x14) mm.

➢ Calculation of bolt shear strength:

We have to verify :

$$F_{v.sd} < F_{v.rd}$$

Where :

$$F_{t.rd} = \frac{\alpha_v \times f_{ub} \times A_s}{\gamma_{M2}}$$

$$\alpha_v = 0{,}6$$

$$F_{v.rd} = \frac{0.6\times1000\times303}{1.25} = 145.44KN$$

$$F_{v.sd} = \sqrt{{F_{v.sd1}}^2 + {F_{v.sd2}}^2}$$

$$F_{v.sd1} = \frac{N_{sd}}{n} = \frac{87.90}{2\times8} = 5.49KN$$

$$F_{v.sd2} = \frac{V_{sd}}{n} = \frac{5908.19}{2\times8} = 369.26KN$$

$$F_{v.sd} = \sqrt{5.49^2 + 369.26^2} = 369.30KN$$

$F_{v.sd}$ = 369.30KN<145.44KN…… Condition non verified

Solution : we use M24 (As=353) and increase the number of bolts to 18

$$F_{v.rd} = \frac{0.6\times1000\times353}{1.25} = 169.440KN$$

$$F_{v.sd} = \sqrt{{F_{v.sd1}}^2 + {F_{v.sd2}}^2}$$

$$F_{v.sd1} = \frac{N_{sd}}{n} = \frac{87.90}{2\times18} = 2.44KN$$

$$F_{v.sd2} = \frac{V_{sd}}{n} = \frac{5908.19}{2\times18} = 164.11KN$$

$$F_{v.sd} = \sqrt{2.44^2 + 164.11^2} = 164.12KN$$

$F_{v.sd}$ = 164.12KN<169.440KN…… Condition verified

➢ Resistance to diametric pressure:

We have to verify that :

$$F_{v.rd} \leq F_{b.rd}$$

Where :

$$\alpha = \min\left(\frac{e_1}{3d_0};\frac{p_1}{3d_0}-\frac{1}{4};\frac{f_{ub}}{f_u};1\right) = \min(0,69;1,41;2,77;1) = 0,69$$

$$F_{v.sd} = \frac{2.5 \times 0.69 \times 360 \times 22 \times 26}{1.25} = 310KN$$

$$F_{v.sd} = \frac{87.90}{2 \times 8} = 5.49KN$$

$F_{v.sd} = 5.49KN < F_{brd} = 310KN$ …….Condition verified

IX.3.2.2 Verification of the flange plate:

➢ **Constructive arrangement of the metal plate of the column flange:**

- **Choice of bolt diameter:**

The choice of diameter will be made, determining their resistances all being proportional to the thickness of the assembled parts as follows:

t≤ 10mm d=(12 ;14)mm

10≤t≤25mm d= (16 ; 20 ;24)mm

t> 25mm d= (24;27;30)mm

t=26mm then we take two lines of 4 bolts Ø24 class HR10.9 : fyb =900MPa

fub =1000 MPa

d_0 = Ø + 2 = 22mm

- **Vertical spacing (p_1,p_2) :**

2,2 d0=57,2mm≤ p1 ≤ min[14t ;200]=200mm We take :p1 =120mm.

3 d0 =78mm≤ p2 ≤min[14t ;200]=200mm We take : p2 =160 mm.

- **Longitudinal clamp (e1) :**

1,2 d0=31,2mm ≤ e1 ≤ [12t ; 150]=150mm We take e1 =50mm.

- **Transverse clamp (e2) :**

1,5 d0 =39 mm ≤ e2 ≤ min[12t ;150]=150 mm We take e2 =70 mm.

We have a plate of dimension (650x300x16) mm.

➢ **Calculation of the Bolt Shear Strength:**

We verify that :

$$F_{v.sd} < F_{v.rd}$$

Where :

$$F_{v.rd} = \frac{\alpha_v \times f_{ub} \times A_s}{\gamma_{M2}}$$

$$\alpha_v = 0,6$$

$$F_{v.rd} = \frac{0,6 \times 1000 \times 353}{1,25} = 169,44\,\text{KN}$$

$$F_{v.sd} = \sqrt{{F_{v.sd1}}^2 + {F_{v.sd2}}^2}$$

$$F_{v.sd1} = \frac{N_{sd}}{n} = \frac{87.90}{2\times8} = 5.49KN$$

$$F_{v.sd2} = \frac{V_{sd}}{n} = \frac{5908.19}{2\times8} = 369.26KN$$

$$F_{v.sd} = \sqrt{5.49^2 + 369.26^2} = 369.30KN$$

$F_{v.sd}$ = 369.30KN<169.44KN KN ……Condition non verified.

Solution : we increase the number of bolts to 18

$$F_{v.sd} = \sqrt{{F_{v.sd1}}^2 + {F_{v.sd2}}^2}$$

$$F_{v.sd1} = \frac{N_{sd}}{n} = \frac{87.90}{2\times18} = 2.44KN$$

$$F_{v.sd2} = \frac{V_{sd}}{n} = \frac{5908.19}{2\times18} = 164.11KN$$

$$F_{v.sd} = \sqrt{2.44^2 + 164.11^2} = 164.12KN$$

$F_{v.sd}$ = 164.12KN<169.440KN…… Condition verified

- **Resistance to diametric pressure:**

We verify that :

$$F_{v.Sd} \leq F_{b.rd}$$

Where :

$$F_{b.rd} = \frac{K_1 \times \alpha \times f_u \times d \times t}{\gamma_{Mb}}$$

$$F_{b.rd} = \frac{2,5 \times 0,69 \times 360 \times 24 \times 26}{1,25} = 300\,KN$$

$$F_{v.sd} = \frac{87.90}{2 \times 8} = 5.49KN$$

$F_{v.sd} = 5.49KN < F_{brd} = 300KN$ …….Condition verified

IX.3.3.Beam-Joist connection:

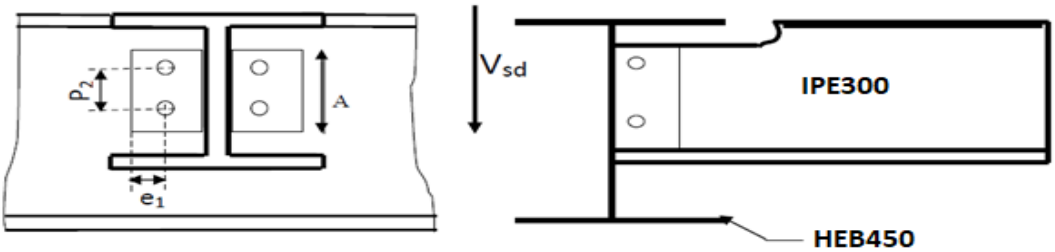


Figure IX. 11 : Beam–Joist assembly diagram (IPE300-HEB450).

- **Beam HEB450 :**

h=450mm; b=300mm; tf=26mm ;t_w=14mm

A=218 cm^2

- **Joist IPE300 :**

h=300 mm ; b=150 mm ;t_f=10,7mm ;t_w=7.1mm

A=53.8 cm^2

- **Tranverse shear force:**

V_{sd} = 53,93KN

- **Choice of bolt diameter:**

The choice of diameter will be made, determining their resistances all being proportional to the thickness of the assembled parts as follows:

t≤ 10mm d=(12 ;14)mm

10≤t≤25mm d= (16 ; 20 ;24)mm

t> 25mm d= (24;27;30) mm

t=8mm

We take two lines of 2 bolts Ø16 class 8.8: fyb =640MPa

fub =800 MPa

d_0 =16 + 2 = 18mm

➢ Constructive arrangements:

- **spacing and longitudinal clamp (p_1 ;e1):**

$2{,}2d_0$=39,6mm≤ p1 ≤ [14t ;200] =112mm We take :p_1 =100mm.

1,2 d_0=21,6mm ≤ e1 ≤ [12t ; 150] =96mm We take e_1 =70mm.

- **Spacing and transverse clamp (p2 ;e2) :**

1,5 d_0 =27mm ≤ e2 ≤ min [12t ;150] =96 mm We take e_2 = 30 mm.

3 d0 =54mm≤ p2 ≤ [14t ;200]=112mm We take : p_2 = 60mm.

The Joist IPE300 and the beam HEB450 , it will be necessary to make a grunting that does not exceed 6% of the height of the web of the joist

Where A :height after grugeage

A≤0,06h → A≤216mm on we take : A=180mm

We take 120×120×10 Angle iron.

➢ Calculation of Bolt Shear Strength:

We have to verify that :

$$V_{Tsd} \leq F_{v.rd}$$

Where:

$$F_{v.rd} = \frac{\alpha_v \times f_{ub} \times A_s}{\gamma_{M2}}$$

$$\alpha_v = 0{,}6$$

$$F_{v.rd} = \frac{0{,}6 \times 800 \times 157}{1{,}25} = 60{,}28\,KN$$

➢ Total shear ($V_{T,sd}$):

$$V_{T.sd} = \sqrt{V'_{sd}{}^2 + V_{m,sd}{}^2} = \sqrt{13.48^2 + 8.08^2} = 15.71KN$$

$F_{v.sd} = 13.48KN < F_{brd} = 60.28KN$.....Condition verified.

IX.3.4. X-bracing connection:

The assembly of 2UPN140 is done by bolts, a single row of 3 bolts 8.8 Ø20 are fixed on a gusset welded on the flange of the column also to the web of the beam with a gusset of 330×330×10.

- **Solliciting forces:**

For this type of bracing according to ROBOT-AUTODESK software the most unfavorable loads are under the combination G+Q+1.2Ex

N_{sd}=196.988KN

IX.3.4.1. Column gusset weld check

- **Haunch-column weld seam :**

We take a haunch of 330 ×330×14.

$$a \geq \beta_w \cdot \gamma_{M.w} \cdot \frac{N.\sqrt{3}}{f_u \sum l}$$

$$a \geq 0{,}7 \times 1{,}25 \times \frac{196.988 \times 10^3 \times \sqrt{3}}{360 \times 330} = 2.51mm$$

So we take a welded seam of a=6mm

- **Constructive disposition**:

- **Spacing and longitudinal clamp (p_1,e_1) :**

2,2 d0=48,4≤ p1 ≤ [14t ;200]=196mm So we take :p1 =60mm.

1,2 d0=26,4mm ≤ e1 ≤ [12t ; 150]=168mm So we take: e1 =50 mm.

- **Spacing and transverse clamp (e2) :**

3d0 =66mm≤ p2 ≤ [14t ;200]=196mm So we take: p2 =80mm.

1,5 d0 =33mm ≤ e2 ≤ min[12t ;150]=168mm So we take e2 =75 mm.

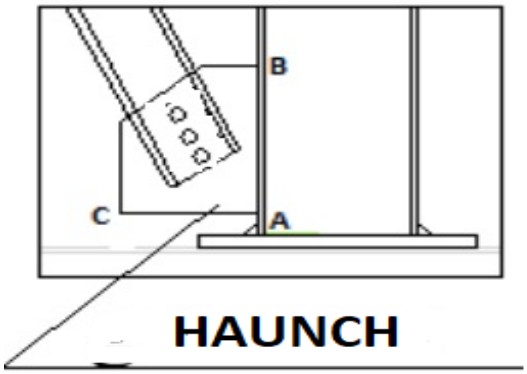


Figure IX. 12 : Haunch assembly-UPN.

IX.3.4.2Calculation of bolt shear strength:

We verify that :

$$F_{v.Sd} \leq F_{v.rd}$$

Avec :

$$F_{v.rd} = \frac{\alpha_v \times f_{ub} \times A_s}{\gamma_{M2}}$$

$$\alpha_v = 0{,}6$$

$$F_{v.rd} = \frac{0{,}6 \times 800 \times 245}{1{,}25} = 940{,}80\,KN$$

$$F_{v.sd} = \frac{V_{sd}}{n} = \frac{196.988}{2 \times 3} = 32.83KN$$

$F_{v.sd} = 32.83KN < F_{brd} = 940.80KN$ ……. Condition verified

Resistance to diametric pressure:

We have to verify that:

$$F_{v.Sd} \leq F_{b.rd}$$

Where :

$$F_{b.rd} = \frac{K_1 \times \alpha \times f_u \times d \times t}{\gamma_{Mb}}$$

$$\alpha = \min\left(\frac{e_1}{3d_0}; \frac{p_1}{3d_0} - \frac{1}{4}; \frac{f_{ub}}{f_u}; 1\right) = \min(0{,}75; 0{,}66; 2{,}22; 1) = 0{,}66$$

$$F_{b.rd} = \frac{2,5 \times 0,66 \times 360 \times 22 \times 14}{1,25} = 143,36\,KN$$

$$F_{v.sd} = \frac{N_{sd}}{n} = \frac{196.988}{2 \times 3} = 32.83KN$$

$F_{v.sd} = 32.83KN < F_{brd} = 143.36KN$……condition verified.

Verification of the UPN140 tensile stress:

The tensile strength of the section determined by two conditions, one in gross section, the other in net section.

We verify that :

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

Where :

Gross section :

$$N_{pl\,rd} = \frac{A.f_y}{\gamma_{M0}} = \frac{(12\times10^2)\times2\times235}{1,1} = 525.545\ KN$$

$N_{sd} = 32.83KN < N_{pl\,rd} = 525.545KN$…….condition verified

Net section :

$$N_{pl\,rd} = \frac{\beta_3 . f_u . A_{net}}{\gamma_{M2}}$$

$$A_{net} = A - t \times d = (1230 \times 2) - (8 \times 10) = 2380mm^2$$

$$N_{p\,rd} = \frac{0.9 \times 360 \times 2380}{1.25} = 616.896KN$$

$N_{sd} = 32.83\ KN < N_{pl\,rd} = 616.896\ KN$…….condition verified.

IX.4.Conclusion :

The results found respond in accordance with regulation CCM97 [3] as well as Eurocode3 [4], so we can say that the elements represent a good connection.

Chapter 10 :
Infrastructure Study

X.1.Base of column:

The base of column are the transition zone between the metal frame and the reinforced concrete foundations. They are allowed either articulated or fixed.

The base of the columns consist of a base plate (the metal plate) welded to the column and the types of anchors embedded in concrete.

For our struct ure, we opted for the fixed type.

The fixing of the metal poles with the foundations is done with the help of the tie-bar.

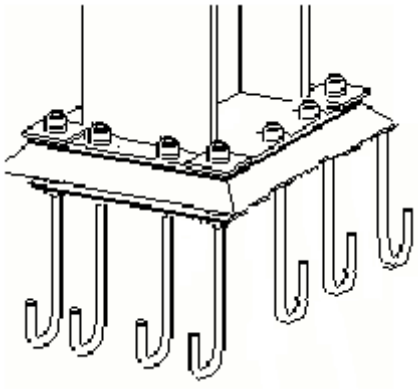


Figure X. 11 : Fixed base of column assembly.

Efforts to be taken into consideration according to ROBOT software:

Table X.1 : Solicitations on the base of column from the ROBOT software.

Maximum sollicitation	N correspondant (daN)	V correspondant (daN)	M correspondant (daN.m)	Load Combinaition
N compression	593674.02	6481.46	6510.21	1.35G+1.5Q
M max	443669.72	15128.22	3364.96	G+Q+1.2Ex
V max	437434.84	22388.53	7758.69	G+Q+1.2Ey

Column HEB650 b=300mm → L=300+100+100=500mm

h=650mm→l=650+100+600=850mm
Lxl →500x850
C=200mm

X.1. 1. Verification of concrete compression stress:

$$\sigma_b = \frac{N}{L \times l} + \frac{6 \times M}{l \times L^2} < f_{bu}$$

$$\sigma_b = \frac{5936.74}{500 \times 850} + \frac{6 \times 65.10}{850 \times 500^2} = 13.97\,MPa < f_{bu}\frac{25}{1,5} = 16.67Mpa$$

X.1.2. Base plate sizing:

It is a rectangular steel plate welded to the base of the post by a weld seam applied to the contour of the section of the profile constituting the column.

The base plate must be sized to withstand the compressive force

X.1.2.1. The thickness of the seat plate:

C=200 mm

$$M_c = \frac{\sigma_b \times c^2}{2} = \frac{13.97 \times 200^2}{2} = 279400N$$

$$t \geq \sqrt{\frac{6 \times M_c}{235}} = \sqrt{\frac{6 \times 279400}{235}} = 26.7mm$$

We take: t=30 mm.

X.1.2.2 Constructive disposition:

The assembly is carried out with four lines of tie bars Ø30 class 8.8.

$$t = t_p = 30\,mm$$

$$d_0 = \varphi + 3 = 33\,mm$$

- **Spacing (p_1, p_2):**

$2,2d_0$=72,6mm≤ p1 ≤ min [14t ; 200] =200mm So we take: p1 =170mm.

3 d_0 =99mm≤ p2 ≤min [14t ; 200]=200mm So we take: p2 =110mm.

- **Spacing (e_1, e_2):**

1,2 d_0=39,6mm ≤ e1 ≤ min [12t ; 150] =150mm So we take e_1 =95mm.

1,5 d_0 =49,5mm ≤ e2 ≤ min [12t ; 150] =150 mm So we take e_2 = 85 mm.

The dimension of the metal plate is (700×500) mm²

Sizing of fut:

$B_{metal\ plate}$ = 500 mm $H_{metal\ plate}$ =700 mm

$$t_{fut} \geq 2 \times \frac{B_{platine} \times H_{platine}}{2(B_{platine} + H_{platine})} = 2 \times \frac{500 \times 700}{2(500 + 700)} = 291,67\,mm$$

$t_{fut} = 300\,mm$

X.1.2.Verification of the base plate-column weld:

X.1.2.1.Weld bead:

The Base plate-column weld bead is chosen using the gorge pre-sizing abacus (a) (see Appendix C).

- **Section HEB 650:**

$t_f = 31mm$

$a_{min} = 6{,}5\,mm \leq t_f \leq a_{max} = 18\,mm$

$t_W = 16mm$

$a_{min} = 2{,}5\,mm \leq t_w \leq a_{max} = 10\,mm$

We choose a weld bead of 18 mm.

X.1.2.2. Verification of the weld of the flange to traction:

We verify that : $N_{sd} \leq F_{w.rd}$

$$F_{w.rd} = \frac{a \times \sum l \times f_u}{\sqrt{2} \times \beta_w \times \gamma_{mw}}$$

- The steel grade used is Fe360 so: $\beta_w = 0{,}8$, $\gamma_{mw} = 1{,}25$

- The length of the weld seam of the flange:

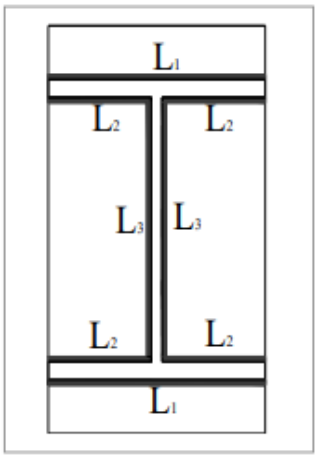


Figure X. 12 : Weld length beam – metal plate.

Where :

$l_1 = 300mm$

$l_2 = 142mm$ (l1-tw)/2

$l_3 = 588mm$ (h-(tf x 2)

- The total length of the weld seams of the flange: $\sum l$ =l1+2l2+2tf=646mm

$$F_{w.rd} = \frac{a \times \sum l \times f_u}{\sqrt{2} \times ß_w \times ɣ_{mw}} = \frac{18 \times 646 \times 2 \times 360}{\sqrt{2} \times 0.8 \times 1.25} = 5920.01KN$$

$$N_{t.sd} = 0\ KN$$

$N_{t.sd} = 0\ KN < F_{w.rd} = 5920.01KN$…….. Condition verified

X.1.2.3. Verification of the weld of the web to traction:

We have to verify that : $N_{sd} \leq F_{w.rd}$

$$F_{w.rd} = \frac{a \times \sum l \times f_u}{\sqrt{2} \times \beta_w \times \gamma_{mw}}$$

- The steel grade used is Fe360 so: $\beta_w = 0{,}8$ $\gamma_{mw} = 1{,}25$

- The total length of the core weld seam : $\sum l$ = 1764mm

- $F_{w.rd} = \frac{a \times \sum l \times f_u}{\sqrt{2} \times ß_w \times ɣ_{mw}} = \frac{18 \times 1764 \times 2 \times 360}{\sqrt{2} \times 0.8 \times 1.25} = 16165.47KN$

$N_{t.sd} = 0\ KN$

$N_{t.sd} = 0\ KN < F_{w.rd} = 16165.47KN$…….. Condition verified

X.1.2.Verification of the base of column resistance:

X.1.2.1.Verification of bending force in the presence of axial stress:

Flexural strength depends on eccentricity:

$$e_N = \frac{M_{Ed}}{N_{sd}} = \frac{65.1021}{5936.7402} = 10.96m$$

$$Z_c = \frac{h_c}{2} - \frac{t_f}{2} = \frac{650}{2} - \frac{31}{2} = 309.5mm = 0.3095m$$

$e_N \succ Z_c$: So we are in the case: Compressed left side

Right side tensed

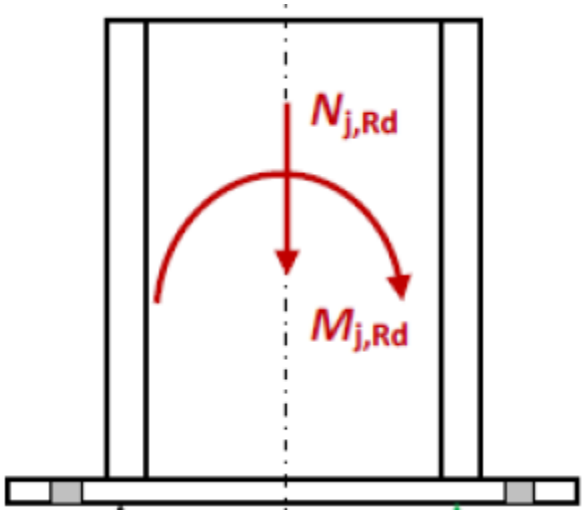


Figure X. 13 : The stresses acting in the base of the column.

$$z = z_c \times 2 = 309.5 \times 2 = 619mm$$

$$F_{c,rd} = 14200.20kg = 142KN$$

$$F_{t.Rd} = 981.74 \text{ x}12kg = 117.72KN$$

$$M_{sd} = 1119.83kgm = 11.19KNm$$

$$z = z_c \times 2 = 212 \times 2 = 424\,mm$$

$$F_{c,rd} = 1859{,}57\,KN$$

$$F_{t,Rd} = 60{,}318 \times 12 = 723{,}81\,KN$$

$$M_{rd} = min\left[\frac{-F_{C,Rd} \times z}{z_c/e + 1}; \frac{F_{t,Rd} \times z}{\frac{z_c}{e} - 1}\right] = \left[\frac{-142 \times 10^3 \times 619}{619/\,10960 + 1}; \frac{117.72 \times 10^3 \times 619}{619/\,10960 - 1}\right]$$

$$M_{rd} = min[-83.19; -77.23]$$

$M_{sd} = 11.19KNm < M_{rd} = 77.23KN$……Condition verified

X.1.2.2. The compressive strength of an X T-section of the flange:

The resistance of a base of a column on its foundation is ensured by equivalent compressed T-sections, one for each flange and one for the web of the pole.

For each equivalent T-section, the crushing strength is determined by multiplying its supporting surface by the strength of the sealing material.

We verify that **:**

$$N_{c.sd} \leq N_{c,rd}$$

$$F_{c,rd} = f_j \times b_{eff} \times L_{eff}$$

Where :

b_{eff} :Efficient width of the T-section such that: $b_{eff} \leq t + 2c$

L_{eff} :Efficient length of the T-section.

Where :

$$L_{eff} = \min(b_p ; b_{fc} + 2c)$$

$$L_{eff} = \min(500;300 + 2 \times 99) = 498\,mm$$

$$b_{eff} = \min(c;\frac{h_c}{2} - t_{fc}) + t_{fc} + \min\left(c;\frac{h_p - h_c}{2}\right) = \min\left(99;\frac{450}{2} - 26\right) + 26 + \min\left(99;\frac{700-450}{2}\right) = 224\,mm$$

$$F_{c,rd} = 16,67 \times 224 \times 498 = 1859,57\,KN$$

$$N_{c.sd} = 2959,14\,KN < N_{c.Rd} = 4 \times F_{c,rd} = 4 \times 1859,57 = 7438,28\,KN$$ ….condition verified.

X.1.2.3. Determination of the resistance to the axial tensile force:

➢ Traction strength of an anchor rod :

The design strength of anchor bolts $F_{t,rd,anchor}$ is equal to the smallest of the values between the design tensile strength of the anchor bolt $F_{t,rd}$ and the calculation strength of the adhesion between the concrete and the anchor bolt $F_{t,bond,rd}$.

$$F_{t,rd,anchor} = \min(F_{t,bond,rd} ; F_{t,rd})$$

Where:

$F_{t,rd,anchor}$: is the strength of the adhesion of the rod with concrete.

$$F_{t,rd,anchor} = \frac{\pi . d . I_b . f_{bd}}{\alpha}$$

Si : d<32mm $\quad f_{t.bonde\,Rd} = \frac{1}{2,25}(\pi.\phi.l_b.f_{bd})$

$$f_{bd} = \frac{0{,}36 \times \sqrt{f_{ck}}}{\gamma_c} = \frac{0{,}36 \times \sqrt{25}}{1{,}5} = 1{,}2$$

Where:

d : rod diameter d=30mm

I_b : anchoring in concrete I_b=600mm

f_{ck} : concrete strength f_{ck}=25Mpa

$$f_{bd} = \frac{0{,}36 \times \sqrt{25}}{1{,}5} = 1{,}2\,\text{MPa}$$

$$f_{t.bonde\,Rd} = \frac{1}{2{,}25}(\pi.30.600.1{,}2) = 30{,}159\,\text{KN}$$

$F_{t,rd}$: Design strength to tensile strength of the anchor bolt section

$$F_{t,rd} = 0{,}9\frac{A_s.f_{ub}}{\gamma_{mb}} = 0{,}9\frac{561 \times 800}{1{,}5} = 269{,}28\,\text{KN}$$

$$F_{t,rd,anchor} = \min(F_{t,bond,rd}; F_{t,rd}) = \min(30{,}159; 269{,}28) = 30{,}159\,\text{KN}$$

- **Participating lengths of the equivalent T-section:**

The possible ruin modes of an equivalent stretched T-section are illustrated in the form of diagrams in Figure X.4 The participating length of the T-section is as follows:

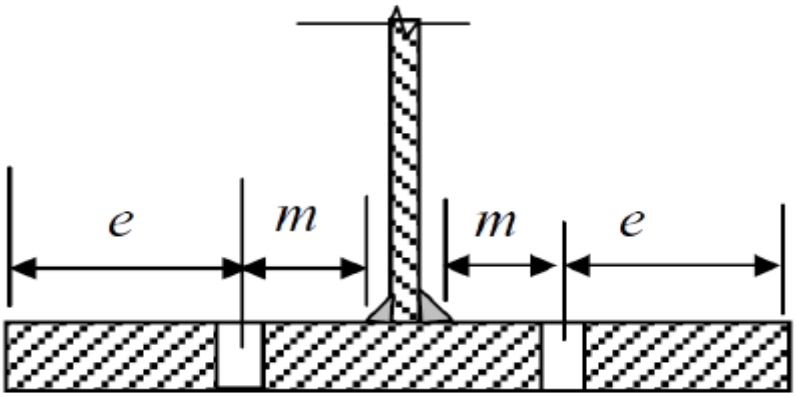


Figure X. 14 : Geometric parameters for the equivalent T-section in tension.

- **Outer row of anchor bolts:**

Circular mechanism: $l_{eff.cp} = \min[(2.\pi.m); (\pi.m_x + w); (\pi.m_x + 2e)]$

$$m = \frac{p_2}{2} - \frac{t_w}{2} - 0{,}8\sqrt{2a_w}$$

a_w : Weld beam taken equal to 18mm.

$$m = \frac{110}{2} - \frac{14}{2} - 0{,}8\sqrt{2 \times 18} = 43{,}2\,mm$$

$$w = 110\,mm \quad e = 85\,mm \quad e_x = 95\,mm \quad m_x = 43{,}2\,mm$$

$$l_{eff.cp} = \min[(2.\pi.m);(\pi.m_x + w);(\pi.m_x + 2e)]$$

$$l_{eff.cp} = \min[(2.\pi.43{,}2);(\pi \times 43{,}2 + 110);(\pi \times 43{,}2 + 2 \times 85)] = 245{,}72\,mm$$

- **Non-circular mechanism:**

$$l_{eff.nc} = \min[0{,}5b_p;(4m + 1{,}25e_x);(e + 2m_x + 0{,}625e_x);(0{,}5w + 2m_x + 0625e_x)]$$

$$l_{eff.nc} = \min[0{,}5 \times 500;(4 \times 43{,}2 + 1{,}25 \times 95);(85 + 2 \times 43{,}2 + 0{,}625 \times 95);(0{,}5 \times 110 + 2 \times 43{,}2 + 0{,}625 \times 95)]$$

$$l_{eff.nc} = \min[250;291{,}55;230{,}76;200{,}76] = 200{,}76\,mm$$

- **Participating length of section T for the first mode**: $l_{eff.1} = \min(l_{eff,cp};l_{eff.nc})$

$$l_{eff.1} = \min(245{,}72;200{,}76) = 200{,}76\,mm$$

- **Participant length of section T for 2nd mode**: $l_{eff.2} = l_{eff.nc}$

$$l_{eff.2} = 200{,}76\,mm$$

➢ **Verification of the presence of leverage:**

the special ruin mode that replaces the first and second modes is only possible when the following condition relating to the length of the anchor bolt is met:

$$L_b \geq L_b^* = \frac{8{,}8A_s}{l_{eff,1}}\left[\frac{m}{t_p}\right]^3$$

Where the distance **m** is that shown in Figure X-5 and the participating length of the T-section is determined above.

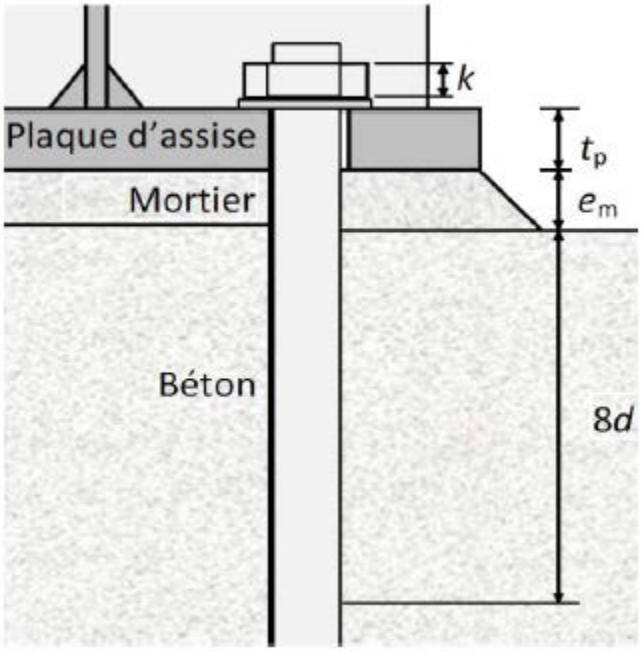


Figure X. 15 : Length of elongation of the anchor bolt.

t_{wa} : Washer thickness $t_{wa} = 5mm$

L_b : Length of elongation of the anchor bolt: $L_b = 8d + e_m + t_p + t_{wa} + 0{,}5k$

k : Nut thickness: $k = 0{,}8d = 24$

e_m : Tile mortar thickness $e_m = 30\text{mm}$

$L_b = 8 \times 30 + 30 + 30 + 5 + 0{,}5 \times 24 = 317mm$

L_b^* : Ultimate length of anchor bolt elongation.

$$L_b^* = \frac{8{,}8 \times 561}{200{,}76}\left[\frac{43{,}2}{30}\right]^3 = 73{,}42\,\text{mm}$$

$L_b = 317\,\text{mm} \geq L_b^* = 200{,}76\,\text{mm}$

➢ **Calculation of tensile strength of the assembly:**

The strength of the equivalent T-section in tension for the row of anchor bolts shall be determined. The design strength of a row of anchor bolts will correspond to the smallest of the values of the ruin modes shown in Figure X. 6 When considering the modes of ruin of a stretched T-section, the design tensile strength of a tensioned anchor bolt shall be taken. $F_{t.anchor.Rd}$

$$F_{t.Rd} = \min(F_{t1-2.Rd}; F_{t.2.Rd}; F_{t.3.Rd})$$

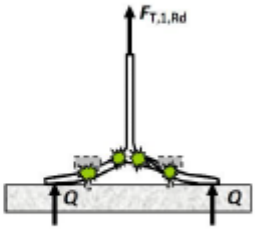
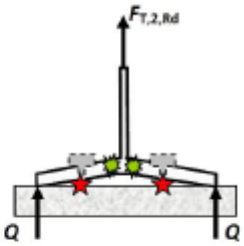
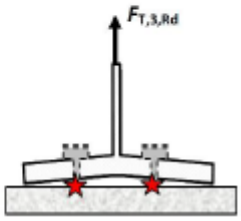
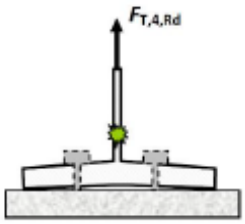
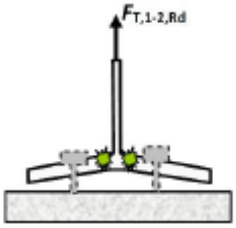
Mode	Description	schéma	Résistance
Mode 1	Mécanisme plastique complet		$F_{t,1,Rd} = \frac{4\,M_{pl,1,Rd}}{m}$
Mode 2	Mécanisme plastique partiel avec ruine du boulon d'ancrage		$F_{t,2,Rd} = \frac{4\,M_{pl,2,Rd}}{m+n} + \frac{2\,n\,F_{t,Rd,anchor}}{m+n}$
Mode 3	Ruine du boulon d'ancrage		$F_{t,3,Rd} = 2\,F_{t,Rd,anchor}$
Mode 4	Plastification de l'âme tendue		$F_{t,4,Rd} = \frac{b_{eff,t}\,t_w\,f_y}{\gamma_{m0}}$ $b_{eff,t} = l_{eff,1}$
Mode 1-2	Ruine par plastification en flexion de la semelle conjointement avec la séparation de la plaque d'assise de la fondation du fait de l'allongement du boulon d'ancrage (effort de levier annulé). Ce mode de ruine remplace les premier et deuxième modes de ruine.		$F_{t,1-2,Rd} = \frac{2\,M_{pl,1,Rd}}{m}$
$M_{pl,1,Rd} = m_{pl,Rd}\,l_{eff,1}$; $m_{pl,Rd} = \frac{t_p^2 f_{yp}}{4\,\gamma_{m0}}$; $l_{eff,1} = \min(l_{eff,cp}\,;\,l_{eff,nc})$ $M_{pl,2,Rd} = m_{pl,Rd}\,l_{eff,2}$; $l_{eff,2} = l_{eff,nc}$; $n = \min(e\,;1.25\,m)$			

Figure X. 16 : Ruin modes for a T-section in tension.

- The bending strength of the plastic ball joint is obtained by:

$$M_{pl.Rd} = l_{eff}.m_{pl.Rd} = l_{eff}.\frac{t_p^{\,2} f_y}{4.\gamma_{M0}}$$

$$M_{pl.Rd} = 200,76 \times \frac{30^2 \times 235}{4 \times 1,1} = 9,614\,KN.m$$

This value may differ between the first

- Mode 1 : anchor bolt mechanism: $F_{t.1.Rd} = \frac{4.M_{pl.Rd}}{m}$
- Mode 2 : ruin of the anchor bolt: $F_{t.2.Rd} = \frac{2.M_{pl.Rd.2} + 2.n.F_{t,anchor.Rd}}{m+n}$
- Mode 1-2 : If the condition requiring the adoption of a special mode is met, the resistors for modes 1 and 2 are replaced by:

$$F_{t.1-2.Rd} = \frac{2.M_{pl.Rd.1}}{m} = \frac{2 \times 9,614}{43,2 \times 10^{-3}} = 445,09\,KN$$

- Mode 3 : ruin of the anchor bolt

$$F_{t.3.Rd} = 2.F_{t,anchor.Rd} = 2 \times 30,159 = 60,318\,KN$$

- Mode 4 : Plastification of the web in tension for the inner rows of anchor bolts:

$$F_{t.wc.Rd} = b_{eff}.t_{wc}.\frac{f_{y.wc}}{\gamma_{M0}} = 224 \times 14 \times \frac{235}{1,1} = 669,96\,KN$$

- The strength of all rows of anchor bolts present is calculated. The calculation resistance for the side in tension is taken as being equal to:

$$F_{t.Rd} = \min(445,09;60,318;669,96) = 60,318\,KN$$

$$N_{T.Rd} = \sum F_{t,Rd} = 4 \times 60,318 = 241,27\,KN$$

- The axial tensile strength of the symmetrical assembly is given as follows:

$$N_{T.Rd} = 4.F_{T,Rd} = 4 \times 60,318 = 241,27\,KN$$

$$N_{T.sd} = 147,39\,KN$$

- We verify :

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

$N_{T.sd} = 147,39\,KN < N_{T.Rd} = 241,27\,KN$ ……condition verified.

X.1.2.4. Resistance to diametrical pressure check:

We verify that : $F_{v.rd} \leq F_{b.rd}$

Where : $F_{b.rd} = \dfrac{K_1 \times \alpha \times f_{u.b} \times d \times t}{\gamma_{Mb}}$

$$\alpha = \min\left(\frac{e_1}{3d_0}; \frac{p_1}{3d_0} - \frac{1}{4}; \frac{f_{ub}}{f_u}; 1\right) = \min(0{,}95; 1{,}46; 1{,}73; 1) = 0{,}95$$

$$F_{b.rd} = \frac{2{,}5 \times 0{,}95 \times 360 \times 30 \times 30}{1{,}25} = 615{,}6\,\text{KN}$$

$$F_{v.sd} = \frac{V_{sd}}{n} = \frac{223.88}{12} = 18.65KN$$

$F_{v.sd} = 18.65KN < F_{b.rd} = 615.6KN$ …….. condition verified

X.1.2.5. Calculation of shear strength of anchor rods :

We verify that:

$$F_{v.sd} < F_{v.rd}$$

Where:

$$F_{v.rd} = \frac{\alpha_v \times f_{ub} \times A_s}{\gamma_{M2}}$$

$$\alpha_v = 0{,}6$$

$$F_{v.rd} = \frac{0{,}6 \times 800 \times 561}{1{,}25} = 215{,}42\,\text{KN}$$

$$F_{v.sd} = \frac{V_{sd}}{n} = \frac{223.88}{12} = 18.65KN$$

$F_{v.sd} = 18.65KN < F_{v.rd} = 215.42KN$ ……. Condition verified.

X.2.Fondations :

X.2.1 Definition of foundations:

The foundations of a construction are made up of the parts of the structure that are in contact with the ground to which they transmit the loads of the superstructure. The foundation elements transmit the loads to the ground, either directly (in the case of flange resting on the ground) or via other organs (example: flange on piles).

X.2.2 Stability of the foundations.

The foundation beds must be in equilibrium under the action:

The stresses due to the superstructure which are:

- Ascending or descending vertical forces.
- Oblique forces (such as the thrust of the land).
- Horizontal forces (earthquake).
- Moments of bending or torsional stress.

Des stresses due to the ground which are:

- Ascending or descending vertical forces.
- Oblique forces (adhesion, embankments, etc.).

The foundation beds must be stable to the uniform settlements that are permissible within certain limits, and differential settlements are rarely compatible with the performance of the structure. It is therefore necessary to adapt the type and structure of the foundations to the nature of the soil that will support the structure: the geological and geotechnical study aims to specify the type, number, and size of foundations necessary to base a given work on a given soil.

X.2. 3. The choice of the type of foundation:

The choice of the type of foundation depends essentially on the strength of the soil. In practice, the value of the soil stress to be introduced into the calculations (designated by σsol) is given by experiment or from the results of the sounding carried out by a soil mechanics lab in the foundations with a minimum coating equal to 3 cm.

X.2.4. Different types of foundations:

The foundations can be classified in relation to the land according to 03 types:

1. Superficial foundations.
2. Semi-deep foundations (wells)
3. Deep foundations (piles).

X.2.5. Pre-dimensioning:

For our structure we have provided insulated flanges under poles, with a soil stress equal to 2 bars from the geotechnical report (see Appendix D).

The sizing of the foundations is done at the ELS, and the reinforcement will be done at the ELU.

The loads to be taken into consideration according to ROBOT**:**

ELU :

$$\begin{cases} M_u = 65.1021KN \\ N_u = 5936.7402KN \end{cases}$$

ELS :

$$\begin{cases} M_s = 333.6496KN \\ N_u = 4436.69KN \end{cases}$$

$$\overline{\sigma_{sol}} = 2\,\text{bar} = 0{,}2\,\text{MPa}$$

X.2.5.1 Calculation of A and B .

$\overline{\sigma_{sol}} \geq \frac{N_S}{S}$ avec : $S = A \times B = A^2$

$$A > \sqrt{\frac{4436.69 \times 10^2}{20000}} = 4.7m$$

On prend : $A = B = 5m$

X.2.5.2.Calculation of the height (h) of the flange :

$d > \frac{B-b}{4}$ $d > \frac{500-300}{4} = 50cm$

On prend :$d = 50cm$

$h \geq d + 5\text{cm}$ $h \geq 50 + 5cm$

On prend :$h = 55cm$

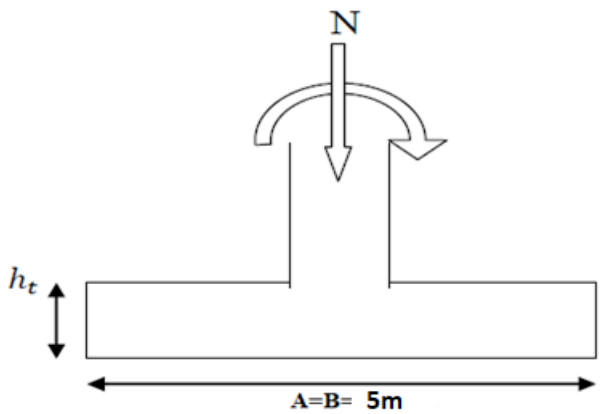


Figure X. 17 : Dimension of the isolated footing.

X.2.6. Verification of the stability :

The eccentricity of the resulting vertical gravitational forces and the seismic forces remaining within the central half of the base of the foundation elements resistant to overturning shall be checked.

$$e_0 = \frac{Ms}{N_s} = \frac{33.6496}{4436.69} = 0.075m$$

We verify that : $A \times B \geq \left(1 + \frac{3e_0}{B}\right)\frac{N_s}{\bar{\sigma}_{sol}}$

$30.25\,m^2 \geq \left(1 + \frac{3\times0.075}{5}\right)\frac{4436.69}{200} = 32.16m^2$ ……Condition non verified.

Solution: we increase the dimension of the foundation to 6x6m^2

Thus : 6x6=36m^2

36m^2>32.16m^2 …….. condition verified

$e_0 = 0{,}075\,m < \frac{B}{6} = \frac{6}{6} = 1$ Thus : a trapezoïdale diagram.

$$\sigma_M = \left(1 + \frac{6e_0}{B}\right) \times \frac{N_u}{AB} = \left(1 + \frac{6 \times 0.075}{0.6}\right) \times \frac{5936.7402}{0.6 \times 0.6} = 177.27KN$$

$$\sigma_m = \left(1 - \frac{6e_0}{B}\right) \times \frac{N_s}{AB} = \left(1 - \frac{6 \times 0.075}{0.6}\right) \times \frac{333.6496}{0.6 \times 0.6} = 231.70KN$$

$$\sigma_{\frac{3}{4}} = \frac{3\sigma_M + \sigma_m}{4} = 190.87KN/m^2$$

$\bar{\sigma}_{sol} = 200\,KN/m^2$

$\sigma_{3/4} \leq \bar{\sigma}_{sol}$ ……. condition verified

X.2.7. Calculation of reinforcement in ELU :

$$e_0 = \frac{M_u}{N_u} = \frac{65.1021}{5936.7402} = 0.0109m \qquad \frac{B}{24} = 0.20$$

$$e_0 \leq \frac{B}{24}$$

For this calculation we will apply the BIELLES Method which is the most commonly used, the principle of this method is to calculate with a constant stress and N' increased to find the reinforcement of our soles. $\sigma_{3/4}$

Where: $N_u' = N_u(1 + \frac{3e_0}{B})$

$$N_u' = 5936.7402\left(1 + \frac{3\times0.075}{5}\right) = 6203.89KN$$

$$A_s = \frac{N_u'(A-a)}{8d\sigma_{st}} = \frac{6203.89 \times (5-3)}{8 \times 0.9 \times 0.65 \times \frac{400}{1.15} \times 10^3} = 72.22cm^2$$

We choose **10T32=80.42 cm².**

X.2.8. Calculation of stirrup spacing:

According to RPA99 the spacing of the stirrups must not exceed**: (20cm ; 15Φ).**

$$S \leq \min(20\,cm; 15\varphi) = \min(20; 15\times2) = \min(20; 37,5)$$

Thus : S= 20cm

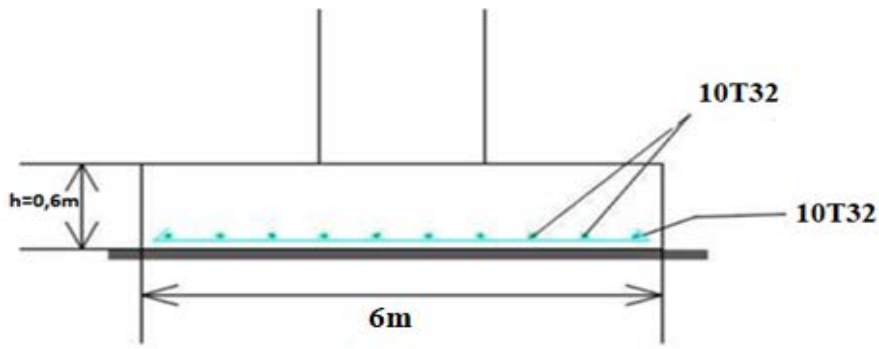


Figure X. 18 : Size of the footing and constructive arrangement of the reinforcements.

X.2.9. Calculation of Foundation beam:

Foundation beams are reinforced concrete elements that connect all the foundations together in the 2 directions to solidarize the foundations. They are subject to traction.

According to the RPA99, for a soil of type S2 the minimum dimensions of the cross-section of the foundation beams are (25×30) cm^2.

X.2.9.1.Calculation of reinforcements:

The foundation beams or equivalent device shall be calculated to withstand traction under the action of a force equal to:

$$F = \frac{N_u}{\alpha} \geq 20\,KN$$

Where : N is the maximum value of the vertical gravity loads provided by the solidarized points of support.

$$F = \frac{5936.7402}{15} = 395.78\,KN > 20KN$$

α= coefficient function of the seismic zone and the category of site considered. (soil **S_2: α=15)**

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

Where: $\sigma_{st} = \frac{f_e}{\gamma_s}$

$$A_{st} = \frac{395.78}{\frac{400 \times 10^3}{1.15}} = 11.37cm^2$$

RPA99 requires a minimum cross-section: $A_{st..min} = 0.6\%$ of the section.

$$A_{st.min} = 0.006 \times 25 \times 30 = 4{,}5\,cm^2$$

We take : $A_{st} = 6{,}79cm^2$ (6T12)

Figure X. 19 : Reinforcement of the foundation beams.

X.2.9.2.Condition of non-fragility:

$$A_{st.min} \leq 0{,}23 b \times d \times \frac{f_{t28}}{f_e}$$

$$A_{st.min} \leq 0{,}23 \times 25 \times 27 \times \frac{2{,}1}{400} = 0{,}81 cm^2$$

We take: $\phi 8$ $A_s = 2{,}01 cm^2$

X.2.9.3. Calculation of stirrups spacing:

According to RPA99, the spacing of the frames must not exceed**: (20cm; 15Φ).**

$$S_t \leq \min(20 cm; 15 \times 0{,}8) = 12 cm$$

So we take the spacing 10cm $S_t = 10cm$

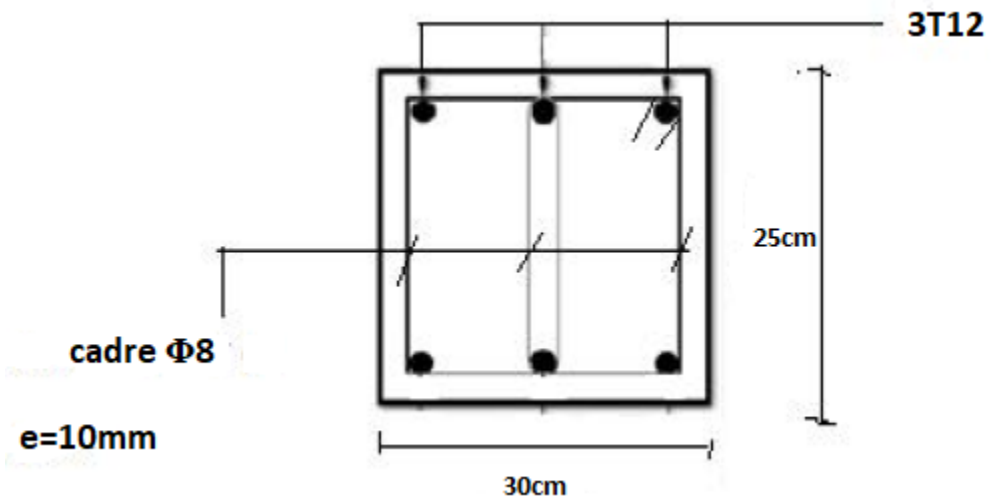


Figure X. 20 : Arrangement of stirrups.

General Conclusion

General conclusion

The study we conducted as part of this project allowed us to enrich our knowledge and knowledge in the field and also to learn the different calculation techniques and concepts of a metallic construction building.

This work allowed us to draw the following conclusions:

The mastery of the calculation software is more than necessary, this allows us to have a behavior close to reality and justified results.

- Actions due to the weight of the structure and live loads are the most unfavourable.
- Steel makes it possible to construct important structures from the point of view of load-bearing capacity, lightness compared to reinforced concrete and speed of execution.
- Possibility of making spatial structures with long-span beams and significant spacing between columns.
- The presence of rigid composite floors makes the structure sensitive to seismic actions, but the bracing system (X-shaped stability blades) gives it good resistance vis-à-vis the latter.
- We had significant efforts at the base of the structure this is due to the spacing between the chosen columns. After having a solution to choose beams in welded reconstituted sections.
- A reduction in the height of the structural floor and, therefore, an increase in the useful height of each floor.
- This study allowed us to immerse ourselves in the different methodologies of sizing foundations on isolated footings.

At the end of this project which constitutes for me a first experience in this vast field, I acquired crucial knowledge. It is important to mention that much remains to be done to enrich my knowledge which I am humbly looking forward to.

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ANNEX

ANNEX A : ARCHITECTURAL PLANS

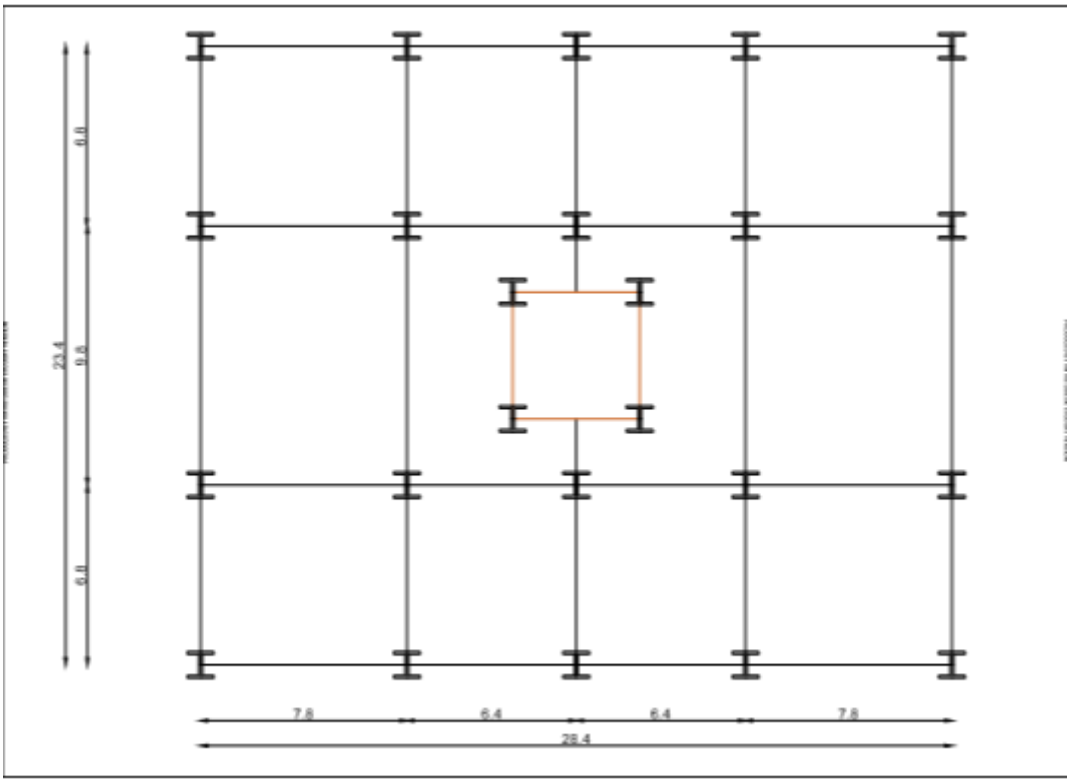
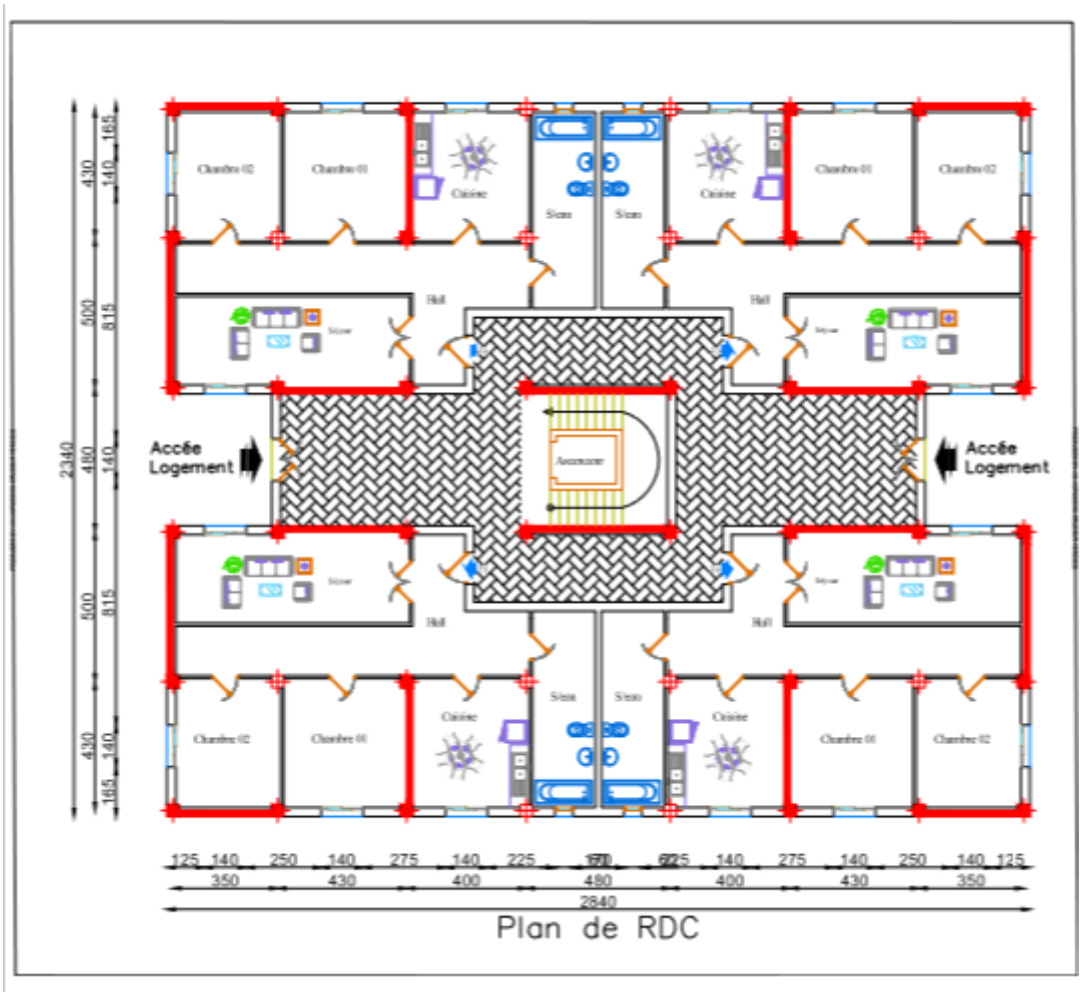


Figure A.0: Actual dimensions used



FigureA.1: Ground floor plan

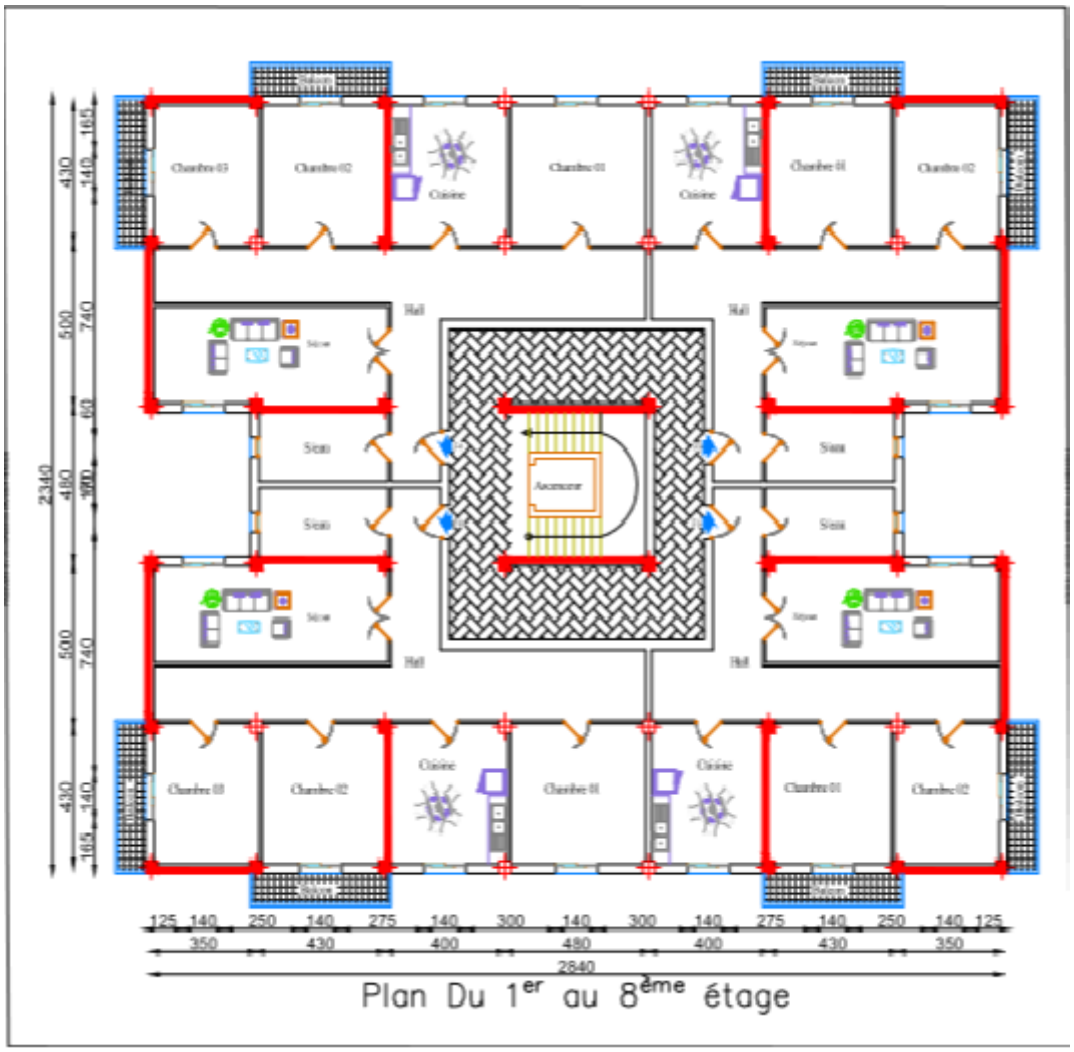


Figure A.2: 1st to 8th floor plan

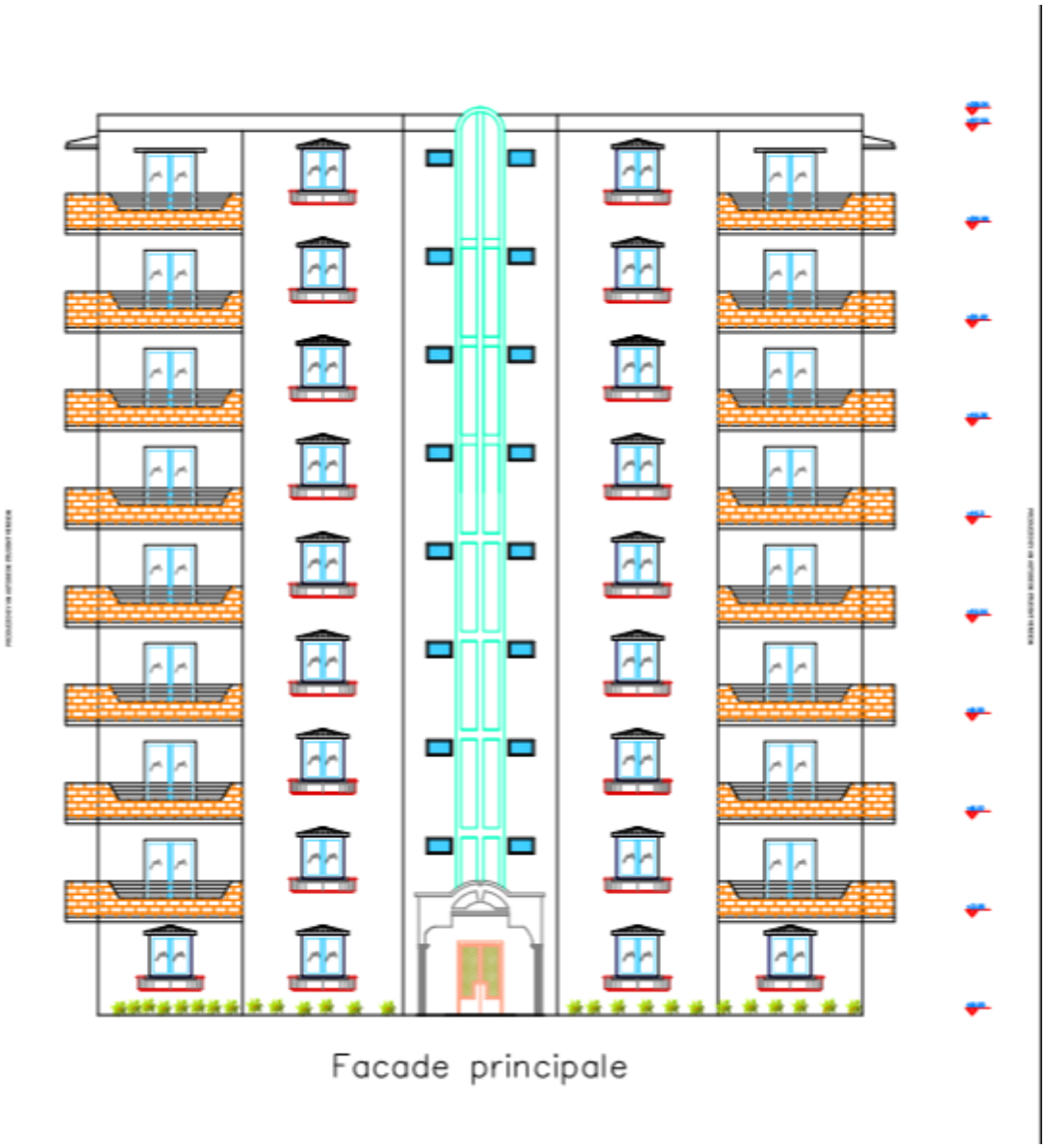


Figure A.3: Principal facade

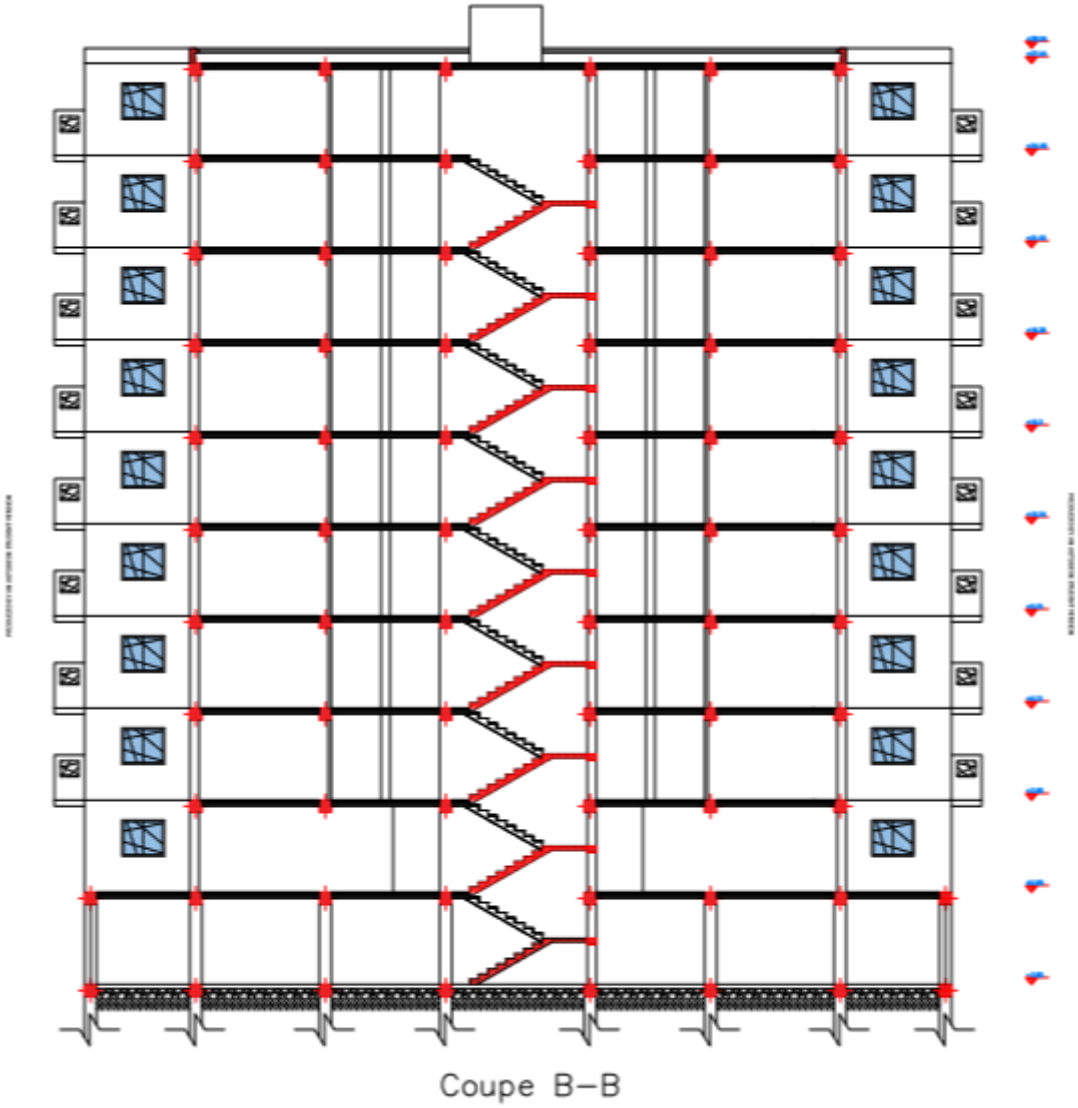
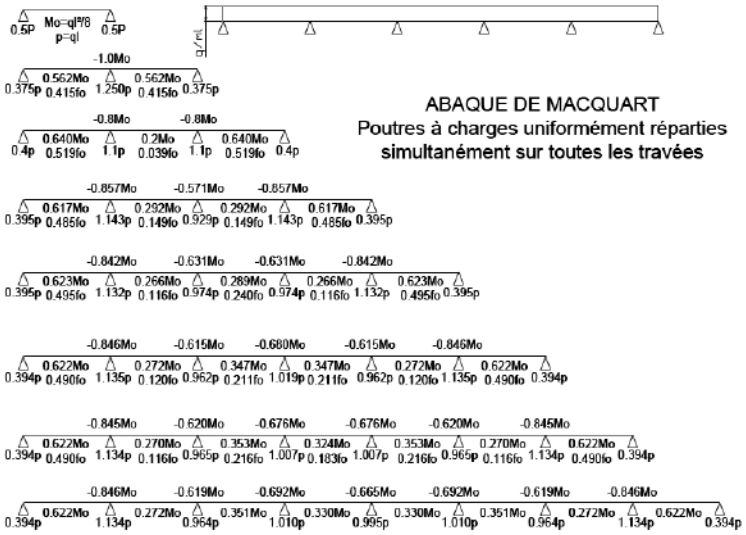


Figure A.3:coupe BB

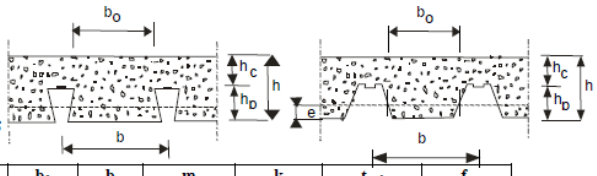
ANNEX B : COMPOSITE FLOOR STUDY

ABAQUE DE MACQUART



dans cette abaque on calcule le moment maximum Mo, les réactions et la flèche maximum de la travée simple considérée comme isostatique, puis on applique les coefficients donnés ci-dessus pour trouver les différents moments, flèches et réactions des poutres hyperstatiques

8.5 Mini catalogue de tôles nervurées pour dalles mixtes



Tôle	t (mm)	h_p (mm)	A_{eff} (mm²/m)	I_{eff} (cm⁴/m)	e (mm) CG	b_0 (mm)	b (mm)	m (N/mm²)	k (N/mm²)	t_{urd} (N/mm²)	f_y (N/mm²)
SUPER-FLOOR (t=0.75)	0.75	77	1216	100.24	42.3	82	192	138.3	0.0095	0	320
SUPER-FLOOR (t=0.88)	0.88	77	1427	118.6	42.3	82	192	138.3	0.0095	0	320
SUPER-FLOOR (t=1.00)	1	77	1621	135.5	42.3	82	192	138.3	0.0095	0	320
HAIRCOL60S(t=0.75)	0.75	60	1026	65.4	30.1	151	300	130.8	0.0469	0	320
HAIRCOL60S (t=0.88)	0.88	60	1213	76.08	29.7	151	300	130.8	0.0459	0	320
HAIRCOL60S(t=1.00)	1	60	1387	85.91	29.4	151	300	130.8	0.0459	0	320
HAIRCOL60S(t=1.25)	1.25	60	1748	106.34	29	151	300	130.8	0.0469	0	320
HAIRCOL59S (t=0.75)	0.75	59	1039	55.15	32.4	89	205	0	0	0	320
HAIRCOL59S (t=0.80)	0.8	59	1112	59.03	32.4	89	205	0	0	0	320
HAIRCOL59S(t=0.88)	0.88	59	1229	65.24	32.4	89	205	0	0	0	320
HAIRCOL69S(t=1.00)	1	59	1405	74.56	32.4	89	205	0	0	0	320
HAIRCOL59S(t=1.20)	1.2	59	1698	90.1	32.4	89	206	0	0	0	320
HAIRCOL59S (t=1.25)	1.25	59	1771	93.98	32.4	89	205	0	0	0	320
COFRASTRA 40 t=0.75)	0.75	40	1202	15.64	14.2	103.5	150	276.0	0.128		330
COFRASTRA 40 (t=0.85)	0.85	40	1359	20.77	14.2	103.5	150	276.0	0.128		330
COFRASTRA 70 (t=0.75)	0.75	73	1195	64.07	31.4	113	183	332.0	0.085	0	330
COFRASTRA 70 (t=1.00)	1	73	1622	91.32	31.4	113	183	332.0	0.085		330
COFRAPLUS60 (t=0.75)	0.75	58	1029	52.38	33.6	125.5	207	323.8	0.01286	0	330
COFRAPLUS60 (t=1.00)	1	58	1391	69.35	33.8	125.5	207	323.8	0.01286	0	330

Tableau 6.2 : Limites supérieures $k_{t,max}$ pour le facteur de réduction k_t

Nombre de goujons par nervure	Epaisseur *t* de la plaque (mm)	Goujons d'un diamètre n'excédant pas 20 mm et soudés à travers la plaque nervurée en acier	Plaques nervurées avec trous et goujons d'un diamètre de 19 mm ou 22 mm
$n_r = 1$	≤ 1,0	0,85	0,75
	> 1,0	1,0	0,75
$n_r = 2$	≤ 1,0	0,70	0,60
	> 1,0	0,8	0,60

Dénomination du goujon à tête	h_{sc} (mm)	d fût (mm)	d1 tête (mm)	ht tête (mm)	fy (N/mm²)	fu (N/mm²)
TRW Nelson KB 5/8"-35	35	16	31.7	8	350	450
TRW Nelson KB 5/8"-50	50	16	31.7	8	350	450
TRW Nelson KB 5/8"-75	75	16	31.7	8	350	450
TRW Nelson KB 5/8"-100	100	16	31.7	8	350	450
TRW Nelson KB 5/8"-150	150	16	31.7	8	350	450
TRW Nelson KB 5/8"-175	175	16	31.7	8	350	450
TRW Nelson KB 3/4"-50	50	19	31.7	10	350	450
TRW Nelson KB 3/4"-60	60	19	31.7	10	350	450
TRW Nelson KB 3/4"-75	75	19	31.7	10	350	450
TRW Nelson KB 3/4"-80	80	19	31.7	10	350	450
TRW Nelson KB 3/4"-100	100	19	31.7	10	350	450
TRW Nelson KB 3/4"-125	125	19	31.7	10	350	450
TRW Nelson KB 3/4"-150	150	19	31.7	10	350	450
TRW Nelson KB 3/4"-175	175	19	31.7	10	350	450
TRW Nelson KB 7/8"-75	75	22	34.9	10	350	450
TRW Nelson KB 7/8"-90	90	22	34.9	10	350	450
TRW Nelson KB 7/8"-100	100	22	34.9	10	350	450
TRW Nelson KB 7/8"-125	125	22	34.9	10	350	450
TRW Nelson KB 7/8"-150	150	22	34.9	10	350	450
TRW Nelson KB 7/8"-175	175	22	34.9	10	350	450
TRW Nelson KB 7/8"-200	200	22	34.9	10	350	450
TRW Nelson KB 7/8"-250	250	22	34.9	10	350	450

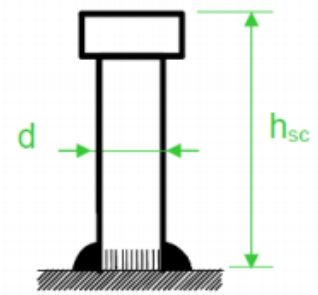


Tableau des armatures

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 cm^2 de N armatures de diamètre Φ (mm)

ANNEX C : ASSEMBLY STUDY

Diamètre nominal d	Pas P	Clef	Diamètre du noyau de la vis d3	Diamètre intérieur de l'écrou d1	Diamètre de la rondelle	Section résistante As	Diamètre moyen dm	Tôle usuelle	Cornière usuelle
8	1.25	13	6.466	6.647	16	36.6	14	2	30
10	1.5	17	8.160	8.376	20	58.0	18.3	3	35
12	1.75	19	9.853	10.106	24	84.3	20.5	4	40
14	2	22	11.546	11.835	27	115	23.7	5	50
16	2	24	13.546	13.835	30	157	24.58	6	60
18	2.5	27	14.933	15.294	34	192	29.1	7	70
20	2.5	30	16.933	17.294	36	245	32.4	8	80
22	2.5	32	18.933	19.294	40	303	34.5	10.14	120
24	3	36	20.319	20.752	44	353	38.8	>14	>120
27	3	41	23.319	23.752	50	459	44.2	-	-
30	3.5	46	25.706	26.211	52	561	49.6	-	-
33	3.5	50	28.706	29.211		694		-	-
36	4		31.093	31.670		817		-	-

Tableau 1 : Principales caractéristiques géométriques

Classe	4.6	4.8	5.6	5.8	6.8	8.8	10.9
f_{yb} (MPa)	240	320	300	400	480	640	900
f_{ub} (MPa)	400	400	500	500	600	800	1000

Tableau 2 : Caractéristiques mécaniques correspondant aux différentes classes

Acier	f_u (MPa)	βw	γ_{Mw}
S 235	360	0,8	1,25
S 275	430	0,85	1,30
S 355	510	0,9	1,35

β_w γ_{mw} variables selon la nuance d'acier

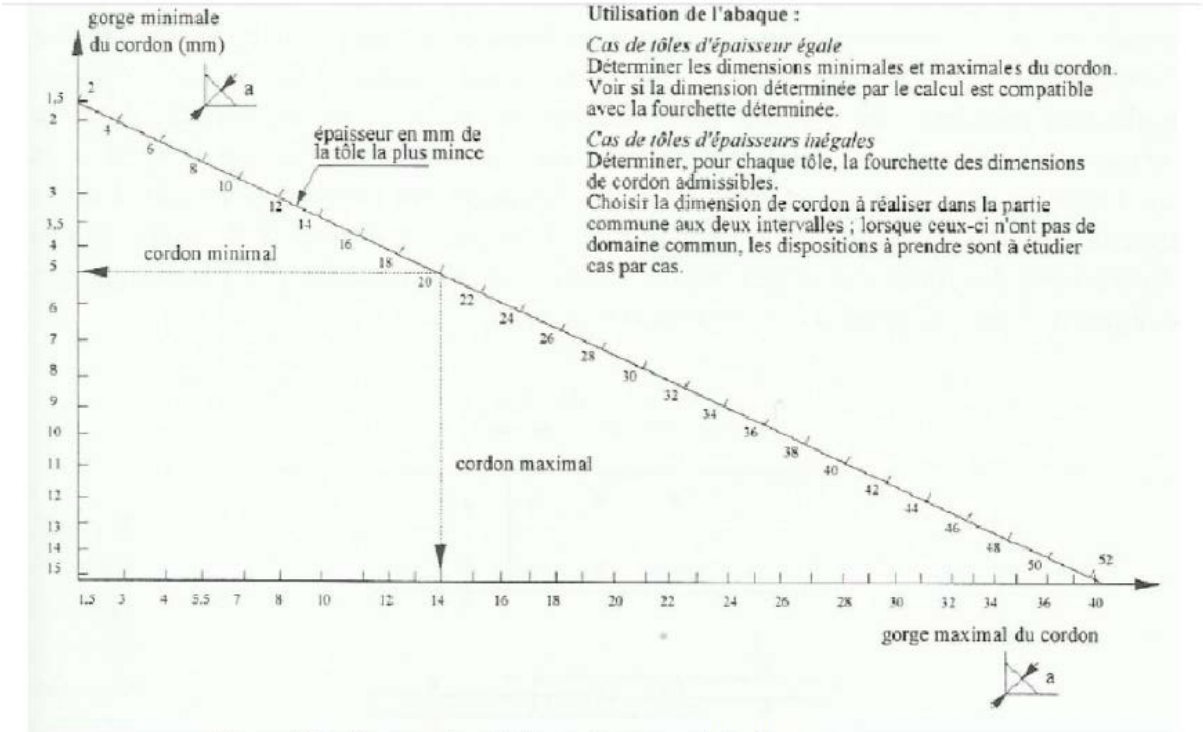


Figure 19 : Abaque de pré-dimensionnement de la gorge a

Coefficients	trou nominal	trou surdimensionné	trou oblong
k_s	1	0.85	0.7
$\gamma_{Ms,ser}$ ELS	1.20	1.20	1.20
$\gamma_{Ms,ult}$ ELU	1.10	1.25	1.25

Classe de surface	μ coefficient de frottement	état de surface
A	0.5	Grenaillé ou sablé
B	0.4	Grenaillé, sablé et peint
C	0.3	Brossé
D	0.2	Non traité