UNIVERSITY SAAD DAHLEB BLIDA 1

Faculty of Technology

Civil Engineering Department

MASTER'S THESIS

Option: Steel and Composite Constructions

DESIGN OF A STEEL FRAME RESIDENTIAL BUILDING (G+8)

By

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ملخص

يهدف مشروع نهاية التخرج هذا الى دراسة بناية متكونة من هياكل معدنية للاستخدام السكني، تقع في ولاية سطيف التي تصنف من ضمن المنطقة الزلزالية ***IIa*** وفقا للنسخة **RPA99/2003**، تتكون البناية من طابق أرضي و8 طوابق وشرفة لا يمكن الوصول إليها، تم حساب ابعاد العناصر المكونة للبناية وفقا للقواعد والمعايير المعمول بها والمتمثلة في: **RPA 99/2003, CCM97, EC3 et BAEL 99.**

هذا المشروع يبدأ بمقدمة عامة وينتهي بخلاصة عامة.

الجزء الأول من هذا المشروع هو الوصف الشامل للبناية ثم الدراسة المناخية وأخيرا دراسة العناصر الأساسية والثانوية.

الجزء الثاني يتضمن الدراسة الديناميكية للبناية وبعدها التحقق من عناصر الإطار المعدني. التحليل الزلزالي للهيكل اجري بواسطة برنامج ***Autodesk Robot 2020***.

الجزء الأخير مخصص لدراسة الوصلات المعدنية ثم دراسة الأساسات والتحقق منها.

Abstract

This graduation project consists of studying a steel building of metal frame for residential use in the city of SETIF, which is classified as seismic zone ***IIa*** according to the RPA99 version 2003. The building consists of a ground floor and 08 floors and an inaccessible roof, braced by triangular V-shaped bracing system, the calculation and verification of all resistant elements were carried out in accordance with the following regulations: **RPA 99/2003, CCM97, EC3 et BAEL 99.**

This project starts with a general introduction and finishes with general conclusion.

The first part of this project is the presentation of the structure after that dimensioning of the climatic loads then sizing of structural and secondary elements.

The second part is the dynamic analysis of the building then the verification of structural elements. The seismic analysis of the structure was carried out using the software Autodesk Robot structural analysis 2020.

The last part is the design of steel connections and the design of foundations.

Résumé

Ce projet de fin d'étude consiste à étudier un bâtiment en charpente métallique à usage d'habitation dans la wilaya De SETIF qui est classée en zone sismique ***IIa*** selon le RPA99 version 2003. Le bâtiment est constitué d'un RDC et de 08 étages et une terrasse inaccessible, contreventé par des palées triangulaires en V. Le calcul et la vérification de l'ensemble des éléments résistants ont été effectués conformément à la règlementation suivantes: **RPA 99/2003, CCM97, EC3 et BAEL 99.**

Ce projet commence par une introduction générale et se termine par une conclusion générale.

La première partie de ce projet c'est la présentation de l'ouvrage, ensuite, l'étude climatique et enfin le prédimensionnement des éléments structuraux et des éléments secondaires.

La deuxième partie c'est l'étude dynamique du bâtiment, ensuite, la vérification des éléments structuraux. L'analyse sismique de la structure a été réalisée par le logiciel de calcul Autodesk Robot structural analysis 2020.

La dernière partie c'est l'étude des assemblages et le dimensionnement et vérification des fondations.

ACKNOWLEDGMENTS

First and foremost, I thank my god **ALLAH** who gave me the health, strength and patience to finish this work.

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All my friends especially Hanan and all my family.

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Doua Sirine Elbatoul

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- **σ_{bu} :** The allowable ultimate compressive stress at ultimate limit state (ULS).
- **σ_{bc}:** The allowable ultimate compressive stress at serviceability limit state (SLS).
- **f_{c28}:** Compressive strength at 28 days.
- **f_{t28}:** Tensile strength at 28 days is estimated.
- **υ:** Poisson's ratio.
- **E:** Young's modulus.
- **I:** Inertia
- **σ_s:** Stress of steel.
- **γ_s:** Steel safety factor.
- **G:** Shear modulus.
- **S_k**: Snow loads on the ground.
- **S:** Snow loads on the roof.
- **q_p:** Peak dynamic pressure.
- ***Ce :*** Wind exposure coefficient
- **Ze**: Reference height.
- ***Cr:*** Roughness factor.
- **I_v:** Turbulence intensity.
- ***Cd:*** Dynamic coefficient.
- ***W:*** Aerodynamic pressure.
- **C_{pe}:** Coefficient of external pressure.
- **C_{pi}:** Coefficient of internal pressure.
- **F_{we}:** External forces.
- **F_{wi}:** Internal forces.
- **M_{plrd}:** Plastic moment resistance.
- **M_{sd}:** Soliciting moment.
- **V_{plrd}:** Plasticizing shear force.
- **V_{sd}:** Shear force.
- **f_{allow}:** Allowable deflection.
- **f^{max}:** Maximum deflection.
- **W_{ply}:** Plastic resistant modulus.
- **W_{ely}:** Elastic resistant modulus.

- R_s**:** Force section of steel.
- R_c**:** Force section of concrete.
- b_{eff}**:** Effective slab width.
- N_{sd}**:** Normal force.
- N_{plrd}**:** Plastic normal force.
- A_v**:** Shearing area.
- **V:** Seismic force
- **A:** Zone acceleration coefficient.
- **D:** Medium amplification factor.
- **Q:** Quality factor.
- **W:** Total weight of structure.
- **R:** Ratio behavior of the structure.
- **T:** Period.
- Δ_k**:** Relative displacement.
- δ_k**:** Horizontal displacement at each level (k).
- δ_{ek}**:** Displacement of seismic forces F_i.
- P_k**:** Total weight of the structure at level k.
- k_y, k_z**:** The interaction factors.
- χ_z, χ_y**:** The reduction factors due to flexural buckling.
- λ**:** Slenderness.
- $\overline{\lambda}$**:** Reduced slenderness.
- **Ø:** Curve factor.
- χ_{LT}**:** The reduction factor due to lateral torsional buckling.
- t: The thickness.
- F_{pc}**:** Preloading force.
- S_f**:** The area of all footings.
- S_b**:** The area of the building.
- L_e**:** Elastic length.
- **K:** Elastic coefficient of soil.
- A_s**:** Section of bars.

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GENERAL INTRODUCTION

In this thesis we try to apply all the knowledge acquired during our 5 years of study on a real project, the main objective is to comprehend and complete the information that we already have acquired at university. The second objective is to present an accurate and correct work in order to obtain the Master's degree.

The purpose of civil engineering studies is to design and build structures that must be able to resist to multiple phenomenon, including earthquakes, wind and fires. The engineer is therefore to design structures with good strength in order to protect human life and to restrict any potential damage.

Metal frames are generally made of steel. The design of the steel structures is based on the analysis of forces. This type of structures is recommended for all kinds of constructions: whether they are storage, industrial, office or housing and for bridges, towers, reservoir...

For any construction project, there are different design and calculation processes to be used according to standards and codes, which must be respected. Each building project has multiple objectives: **Safety** (The most important which ensure the stability of the structure), **Economy** (to reduce project costs)**, Comfort, Artistic.**

This final thesis consists of study of **a steel building (G + 8 floors)** made of metal frames for residential use, the work will be organized according to the following chapters:

- **The First chapter:** is a general presentation of the project, it covers information about the building, the geotechnical characteristic, location, used materials and also the technical regulations that are used in this study.

- **The Second chapter:** is a climatic load analysis which includes the calculation of snow loads and wind loads using the regulations RNV2013.

- **The Third chapter:** is the pre-design of structural elements, which are beams, joists and columns that will be used in chapter 5 for initial model of building, in this chapter the adopted regulations are EC3 EC4 and DTR2.2.

- **The Fourth chapter:** is the design of secondary elements, which are stairs, parapet and balconies.

- **The Fifth chapter:** is the dynamic analysis, which covers the modeling of the building using the Robot software 2020 which must be done according to the conditions of modal analysis and seismic analysis to verified the final model, the regulations RPA99/2003 are used in this chapter.

- **The Sixth chapter:** is the verification of structural elements, the results of chapter 5 are used in this chapter to verified the stability and strength of columns, beams, joists, and bracing system.

- **The Seventh chapter:** is the design and calculation of steel connection between different elements such as: column-beam, column-column, beam-joist, bracing system and column base.

- **The Eighth chapter:** is the study and design of the infrastructure of the building.

Finally, the work is completed with a general conclusion.

CHAPTER 1

GENERAL INRORMATION

1.1 INTRODUCTION:

Structural steel is one of the most used construction materials, mostly is a carbon steel that is composed of a mix of ductile ferrite and strong pearlite microstructures. To increase the strength of the material and to have a good weldability, usually 0.2% maximum of carbon is added to the material, also manganese, chromium or copper can be inserted.

Figure 1.1: Structural steel fabrication.

The building of this project has 18 housing, inaccessible roof, two housing for each floor, each housing has: three rooms, kitchen, living room, bathroom, wc, drying room, hallway and two balconies. The openings are balconies, doors and windows.

The building has two facades that are main facade and rear facade; it has a complicated shape, not symmetrical at two axes, symmetrical in opposite direction.

The building constructed with a steel frame, collaborative floors, structural elements of steel, and walls of hollow brick.

1.2 PRESENTATION OF THE PROJECT:

This project consists of design and studying of a steel building composed of 8 floors, for residential use. This project is located in the center of Setif which is a region located at an altitude of 1100 m in eastern Algeria, classified according to the Algerian seismic design code “RPA99/03” as an area of high seismicity (***IIa***). The structure consists of steel beams and columns and a composite floor (Concrete – Steel).

1.3 GEOMETRICAL CHARACTERISTICS OF THE BUILDING:

1.3.1 Plan view:

- Total length………………...19.56 m

- Total width…………………19.56 m

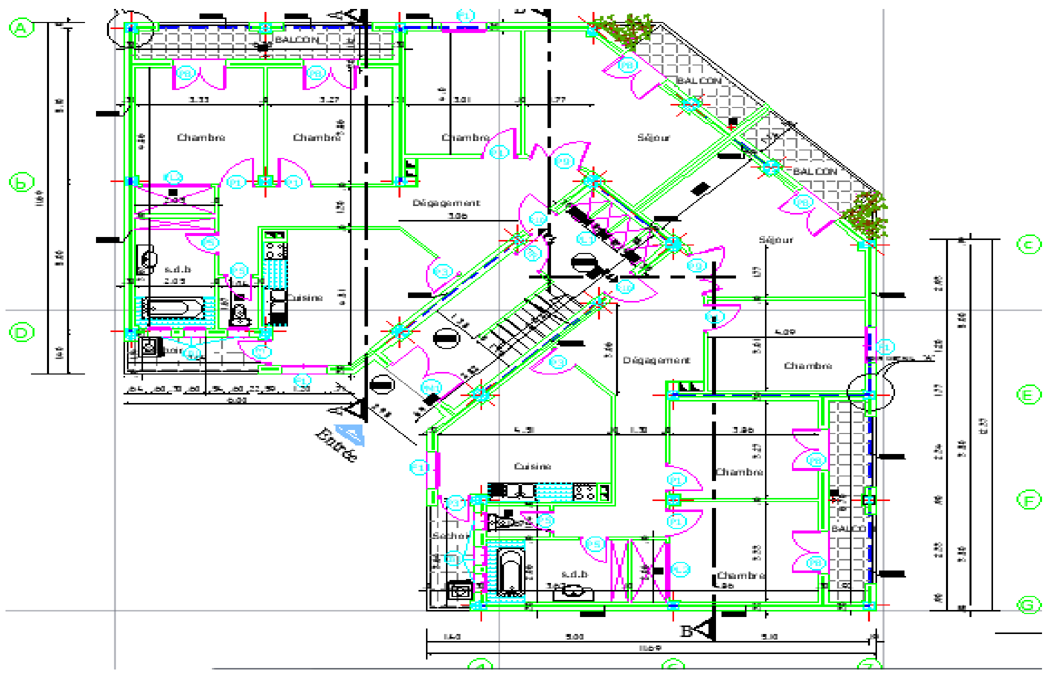


Figure 1.2: Plan view of the building.

1.3.2 Elevation view:

- Floor height………………...3.4 m.

- Parapet height………………0.6 m.

- Total height………………...31.2 m.

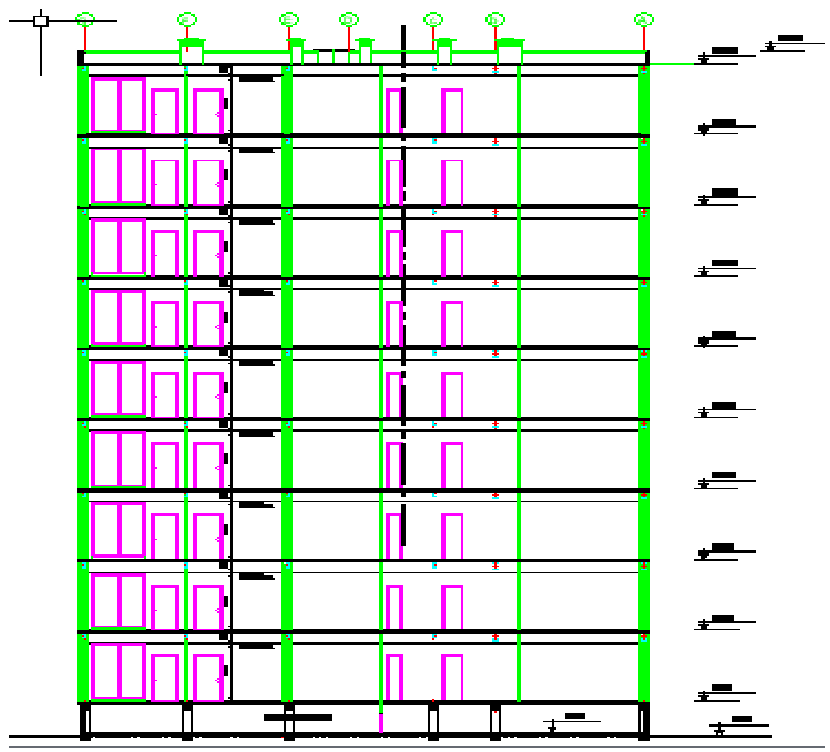


Figure 1.3: Elevation view of the building.

1.4 LOCATION AND DATA CONCERNING THE SITE:

The building's location is in the center of Setif whish has the following information:

- Allowable soil stress……………..2.00 bars.

Figure 1.4: Location of the project (Setif center).

1.5 STRUCTURAL BUILDING SYSTEM:

1.5.1 Type of the structure:

Structure is the sum of assembled elements, designed to ensure the rigidity of the construction. The structure of this study is composed of beams and columns which is a frame structure and by bracing system. The vertical loads of building (dead load and live loads) are taken by the frame structure (beams, columns), and the horizontal loads (seismic force) are taken by the bracing system.

Steel elements have a poor fire resistance due to this; load-bearing structural members are usually protected by thermally insulating materials if fire loads are relevant.

The coating on floors is a series of protective layers composed of: Multilayer waterproofing, slop form, Thermal insulation, Gravel protection.

The external walls and internal walls are constituent of hollow clay brick.

1.5.2 Floors:

The structure includes a composite floor (concrete-steel) which is composed of:

-Slab of concrete-steel.

- Profiled steel decking TN40.

- Beams and joists.

- Connectors (studs).

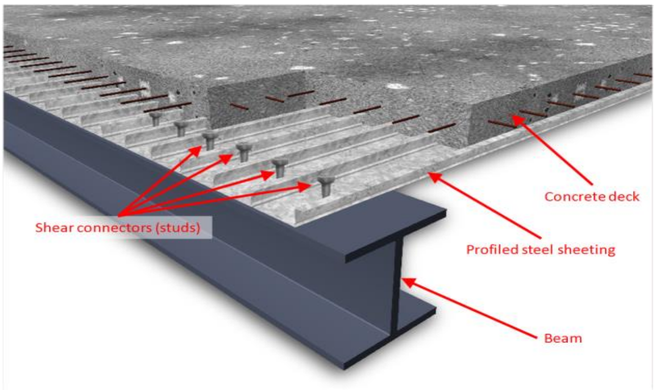


Figure 1.5: Composition of the composite slab.

1.5.3 Connection:

The connection between the elements of the structure is ensured by :

- Bolted assembly: hight-strength bolts and ordinary bolts.

- Welded connection.

1.5.4 Bracing system:

Bracings system are vertical framework of stability, they are designed to ensure the stability of structure, which are reprend the horizantal effect (wind loads, sismic force). There is a different types of bracing system, in this project the type adopted is: Braced frames by triangulated blades V.

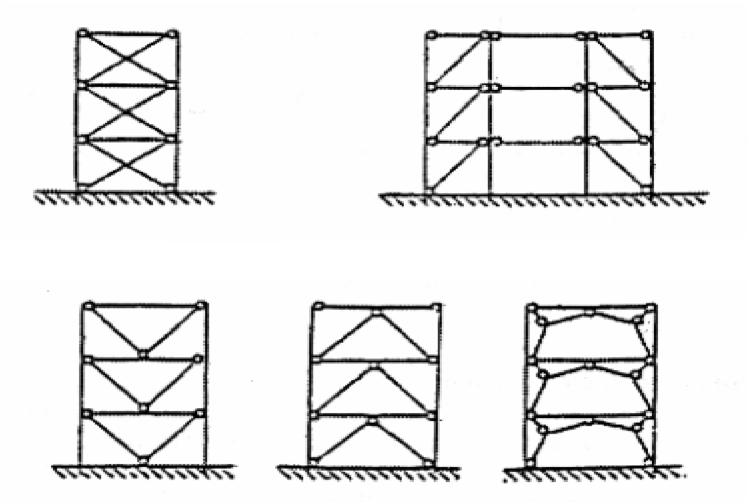


Figure 1.6: Braced frames by triangulated blades V and X.

1.5.5 Stairs:

A stair is an architectural construction consisting of a regular series of steps march between the floors, from one level to another going up and down. Several diffrents types of stairs can be adopted which include :

Figure 1.7 : Different types of stairs.

The staircase used in this buliding are the half turn stair. It consists of two straight flights with two 90° turns.

Figure 1.8: Half turn stair.

1.5.6 Foundation:

The foundation is the link between the superstructure and the ground, the choice of foundation is adopted according to a geothecnic study and the structral analysis of the building. There are three types of foundation which includes: shallow foundation, semi-deep foundation, deep foundation. The type used in this project is shallow foundation. The shallow foundation diveded on three types: pad footing, strip footing and raft foundation. The raft foundation is adopted for this building.

1.6 CHARACTERISTIC OF MATERIAL:

1.6.1 Adopted and used Materials:

- Steel.
- Concrete.
- Hollow clay brick for masonry.
- Plaster rendering interior.
- Cement rendering exterior.
- Gravel protection.
- Tile.
- Sand.

1.6.2 Concrete:

Concrete is a construction material that is composed of a mixture of mortar and gravel, it has an excellent strength. The compressive strength of concrete increases with time, in order for concrete to be allowed for use in structures according to EC4, the characteristic compressive yield stress must be between 20 and 60 MPa. The tensile strength of concrete is very low and the shear strength is also low.

The following types of concrete are used in this project:

- Reinforced concrete………….350kg/m^3.
- Blinding concrete…………….150 kg/m^3.

➢ ***Concrete strength:***

According to BAEL 91:

- Compressive strength at 28 days: $f_{c28} = 25$ MPa
- Tensile strength at 28 days is estimated by: $f_{t28} = 0,6 + 0.06\ fc28$, $ft_{28} = 2.1$ MPa
- Shrinkage coefficient: $\varepsilon = 2 \times 10^{-4}$
- Density: $\rho = 25\ kN/m^3$.

- ***Ultimate stress:***

According to BAEL 91:

The allowable ultimate compressive stress at ultimate limit state (ULS) is estimated by:

$$\sigma_{bu} = \frac{0.85fc28}{\gamma_b}$$

γ_b = 1.5 in case of persistent and transient actions.

γ_b = 1.15 in case of accidental actions.

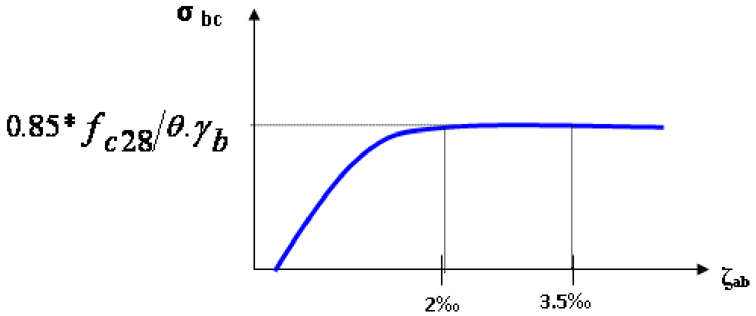


Figure 1.9: Stress-strain Diagram (ULS).

The allowable ultimate compressive stress at serviceability limit state (SLS) is estimated by:

σ_{bc}= 0,6.f_{c28} = 15 MPa (BAEL 91).

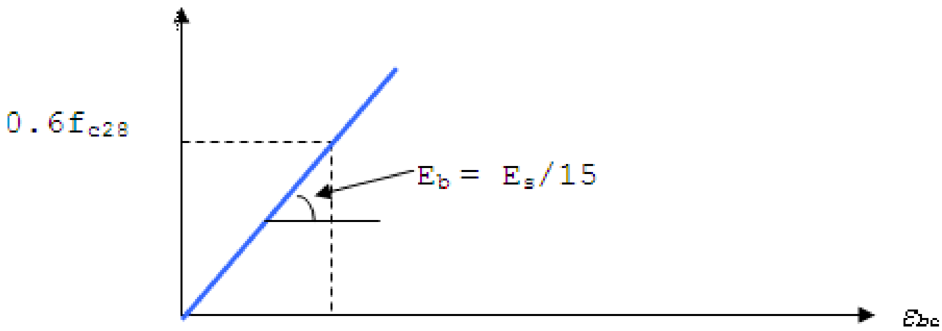


Figure 1.10: Stress-strain Diagram (SLS).

- ***Shear stress:***

The ultimate shear stress takes the following values according to BAEL 91:

Little damaging cracking: $\bar{\tau}$= min (0.13. f_{c28}, 4 MPa) = 3.25 MPa.

Damaging or very damaging cracking: $\bar{\tau}$= min (0.10. f_{c28}, 3MPa) = 2.5 MPa.

- ***Poisson's ratio:***

From BAEL 91, the values of Poisson's ratio are as follows: $\upsilon=0$ at ULS; $\upsilon= 0.2$ at SLS.

- ***Young's modulus:***

The Young's modulus is defined under the action of normal stress of long or short duration.

  - ***Instantaneous Young's modulus:***

For the load of short duration (less than 24h):

$E_{ij} = 11000\sqrt[3]{fcj}$, $E_{i28} = 32164.195MPa$ (BAEL 91).

  - ***Long-term Young's modulus:***

For the load of long duration: $E_{ij} = 3700\sqrt[3]{fcj}$, $E_{i28} = 10818.865\ MPa$ (BAEL 91).

1.6.3 Steel:

A steel is a metal alloy composed of iron and carbon. It is one of the basic materials that are used nowadays in civil engineering. The tensile strength of steel is very high and the compressive strength is low.

In this project, the following types of steel have been used:

- ***Reinforcing steel:***

- Smooth rebar FeE235
- Ribbed bars: FeE400
- Welded fabric: TLE52, $\varnothing$ = 6 mm for the slabs

Ultimate limit state ULS:

According to BAEL 91:

- σ_s : stress of steel $\sigma_s = f_e/\gamma_s$
- γ_s : steel safety factor:

  $\gamma_s = 1.15$ in case of persistent and transient actions

  $\gamma_s = 1.00$ in case of accidental actions.
- ε_s : relative elongation of steel $\varepsilon_s = \Delta L/L$

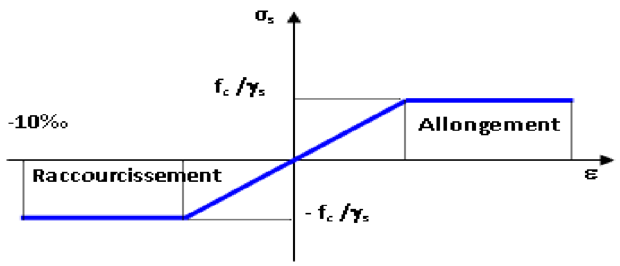


Figure 1.11: Stress-strain Diagram (ULS).

Serviceability limit state SLS:

Little harmful cracking, no verification.

Little damaging cracking: σ_s = min [$\frac{2}{3}$ f_e ; 150η]

Damaging or very damaging cracking: σ_s = min [$\frac{1}{2}$ f_e ; 110η]

With:

η : cracking factor:

η = 1 for a smooth rebar

η = 1.6 for a Ribbed bar

Young's modulus:

Young's modulus of steel is: E=2.1 x 105 MPa (Eurocode 3)

- ***Construction steel:***

The mechanical characteristics of the grades of steel that is used according to Eurocode 3 are as follows:

FeE235(S235) is used:

Yield strength:	F_y= 235 N/mm²
Tensile strength:	F_u= 360N/mm²
Young's modulus	E= 2.1 E5 Mpa
Poisson's ratio	υ = 0.3
Shear modulus	G= 8.1 E4 Mpa
Density of steel	ρ=78.5 kN/m3

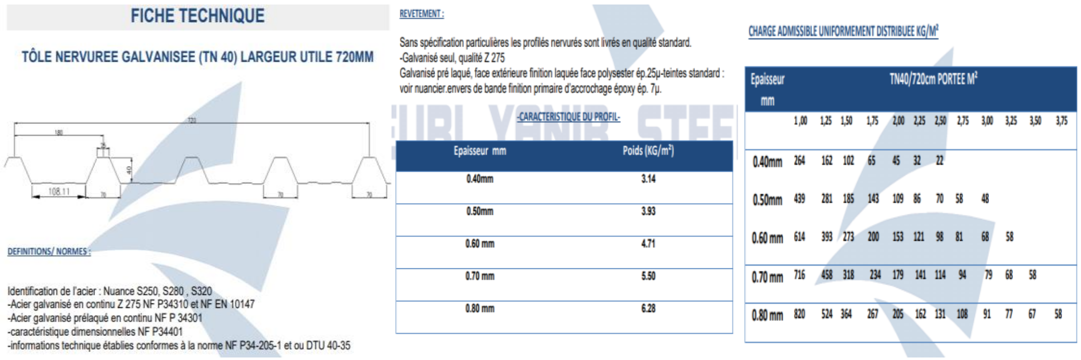
- ***Profiled steel decking:***

The profiled steel decking used with concrete and steel work together to create a composite floor system or as structural element to support a built-up. It ensures:

- Efficient and waterproof formwork by eliminating framework-stripping action.
- Building a work platform for the concrete implementation.
- Most of the time it will avoid using props during construction, and save time.

Metal profile sheets can have different profiles, with different heights and different thickness; there are many commercially available profiles for the steel sheets.

The profiled steel decking used is TN40, which has the following characteristics:



FICHE TECHNIQUE

TÔLE NERVUREE GALVANISEE (TN 40) LARGEUR UTILE 720MM

DEFINITIONS/ NORMES :

Identification de l'acier : Nuance S250, S280 , S320
-Acier galvanisé en continu Z 275 NF P34310 et NF EN 10147
-Acier galvanisé prélaqué en continu NF P 34301
-caractéristique dimensionnelles NF P34401
-informations technique établies conformes à la norme NF P34-205-1 et ou DTU 40-35

REVETEMENT :

Sans spécification particulières les profilés nervurés sont livrés en qualité standard.
-Galvanisé seul, qualité Z 275
Galvanisé pré laqué, face extérieure finition laquée face polysester ép.25µ-teintes standard : voir nuancier.envers de bande finition primaire d'accrochage époxy ép. 7µ.

-CARACTERISTIQUE DU PROFIL-

Epaisseur mm	Poids (KG/m²)
0.40mm	3.14
0.50mm	3.93
0.60 mm	4.71
0.70 mm	5.50
0.80 mm	6.28

CHARGE ADMISSIBLE UNIFORMEMENT DISTRIBUEE KG/M²

Epaisseur mm	TN40/720cm PORTEE M²											
	1,00	1,25	1,50	1,75	2,00	2,25	2,50	2,75	3,00	3,25	3,50	3,75
0.40mm	264	162	102	65	45	32	22					
0.50mm	439	281	185	143	109	86	70	58	48			
0.60 mm	614	393	273	200	153	121	98	81	68	58		
0.70 mm	716	458	318	234	179	141	114	94	79	68	58	
0.80 mm	820	524	364	267	205	162	131	108	91	77	67	58

Figure 1.12: Technical data sheet of TN40.

1.6.4 Connectors:

Connectors ensure the link between steel and concrete where the connection is designed to resist shear forces. In this project studs connectors are adopted. Height stud h=95 mm, diameter d=19 m, they are assembled by welding.

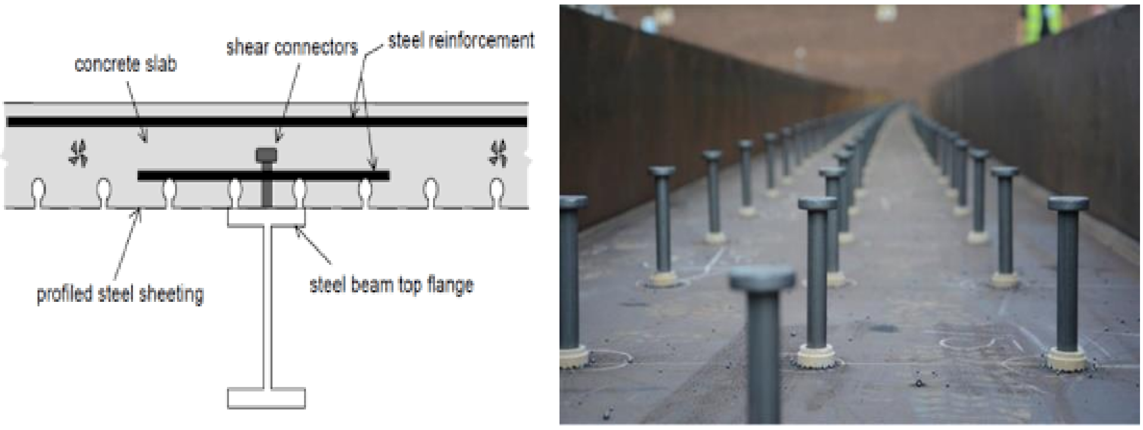


Figure 1.13: Shear connector- headed studs.

1.7 TECHNICAL REGULATION USED:

- CCM 97: Calculation rule for steel constructions.
- RPA2003: Algerian Seismic Regulation version 99/2003.
- RNV2013: Rules defining the effects of snow and wind.
- DTR C2.2
- Eurocode 2
- Eurocode 3

- Eurocode 4
- BAEL91/99

1.8 CONCLUSION:

- ✓ Concrete will continue to be an important construction material.
- ✓ Steel offers many advantages, primarily high strength and ductility.
- ✓ Steel reinforcement bars are used to resist tension force in the tension region.
- ✓ Steel structures have: less durable, more resistance to disasters, high load carrying capacity.
- ✓ A V-bracing system is used to ensure the stability of the building.

CHAPTER 2

CLIMATIC LOADS

2.1 INTRODUCTION:

The objective of this chapter is to evaluate the effects of wind and snow action; the calculation will be conducted in accordance with the Snow and Wind Regulations 2013 **(RNV2013)**.

2.2 SNOW LOADS:

The snow load depends on the variation of the altitude and the geographical area determined on the snow map.

2.2.1 *Site data*:

Altitude: H = 1100m

Snow zone: zone A

2.2.2 Characteristic value of snow on the ground S_k:

The value of S_k in kN/m^2 is determined by the following law:

$S_k = \frac{0.07H+15}{100}$ (zone A) with: H=1100m

$S_k = \frac{0.07(1100)+15}{100} = 0.92$ kN/m^2

$$\mathbf{S_k=0.92\ kN/m^2}$$

2.2.3 Snow load on the roof S:

The minimum snow load S per unit of horizontal roof area is obtained by the following formula: $S=\mu.S_k$ [kN/m^2].

μ: snow load shape coefficient, for this construction it has a terrace (α=0°) so μ= 0.8.

and S_k that is calculated.

$$S=0.8\times0.92=0.736\ kN/m^2$$

$$\mathbf{S=0.736\ kN/m^2}$$

2.3 WIND LOADS:

The wind study consists of determining the effect of wind pressure at each level of the building. The calculation shall be carried out separately for each of directions perpendicular to the different walls of the building.

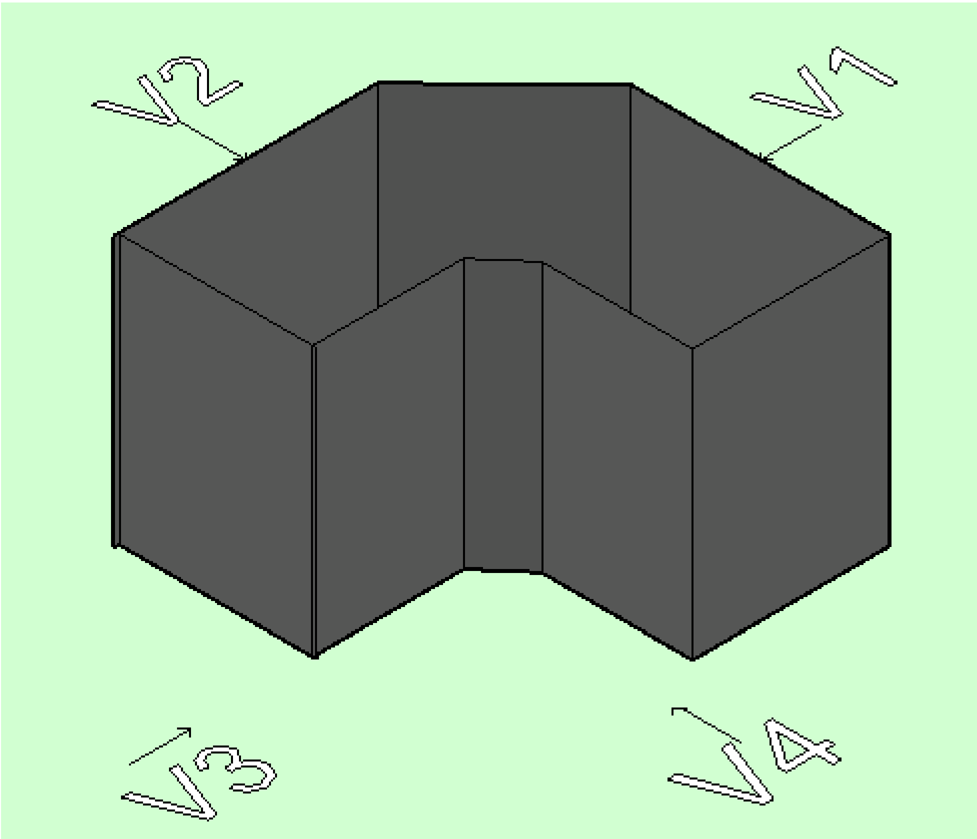


Figure 2.1 : Wind direction of the building.

2.3.1 Site data:

- ✓ Zone of wind: zone Ⅱ so: $q_{ref} = 435\ N/m^2$
- ✓ Category of terrain: Ⅲ
- ✓ The terrain factor: K_T=0.215
- ✓ The roughness length: z_0=0.3m
- ✓ The minimum height: z_{min}=5
- ✓ Coefficient: ε=0.61
- ✓ Topographic coefficient: Flat site: C_t=1

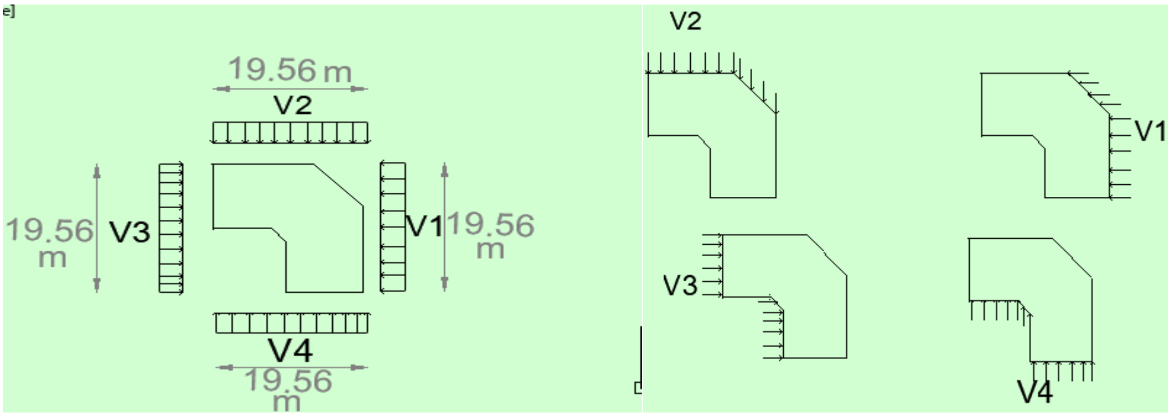


Figure 2.2: Wind in the 4 directions of the building.

According to the figure 2.2 there are $V_1=V_2$ and $V_3=V_4$ because it has the same distances at the different directions and the same openings.

2.3.2 Peak dynamic pressure q_p(ze):

Peak dynamic pressure of the reference height ze is given from the RNV2013 chapter 2 (2.3.1) by the following formula: $q_p=q_{ref}\times Ce(ze)$ [kN/m^2].

2.3.3 Wind exposure coefficient Ce(z) :

which is given by the following formula: $Ce(z) = Ct(z)^2\times Cr(z)^2\times[1+7.Iv(z)]$.

2.3.4 Reference height ze :

The reference height depends on dimensions h and b of the construction which are given in the figure below (RNV2013, figure 2.1):

h=31.2 m and b=19.56 m so: $b < h < 2.b$

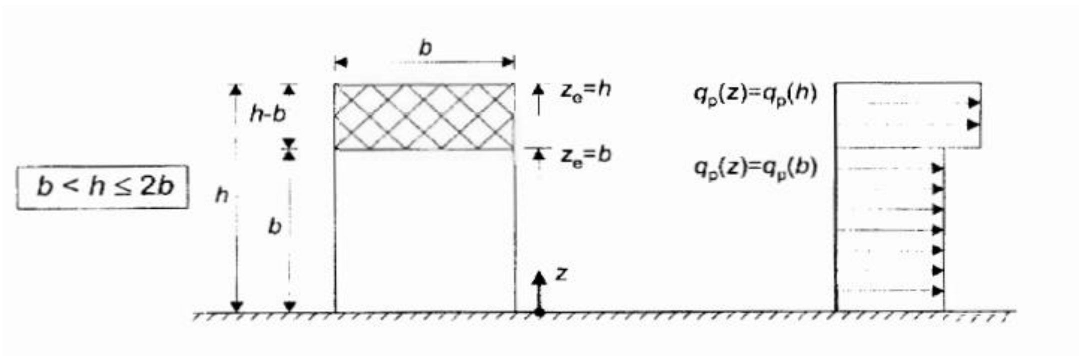


Figure 2.3: Reference height ze corresponding to the dynamic pressure.

Lower band: ze=b=19.56 m, Upper band: ze=h= 31.2 m

2.3.5 Roughness factor Cr:

The roughness factor is given from 2.4.4 RNV by this expression:

$Cr(z)=Kt\times ln\left(\frac{z}{z0}\right)$ $\quad z_{min}=5$ m < z=h=31.2 m < 200m

To determine the peak dynamic pressure $q_p(ze)$ it must be determined the Cr in function ze

There are two different values of ze , Kt=0.215 , z_0=0.3 so:

-$Cr(ze=19.56m)=0.215\times ln\left(\frac{19.56}{0.3}\right)=0.898$

-$Cr(ze=31.2\ m)=0.215\times ln\left(\frac{31.2}{0.3}\right)=0.998$

2.3.6 Turbulence intensity I_v:

The turbulence intensity is given by: $I_v(z)=\frac{1}{Ct(z)\times\ln(\frac{z}{z0})}$ $\quad z=31.2$ m > z_{min}=5 m

Iv in function ze:

$Iv(ze=19.56m)=\frac{1}{1\times\ln(\frac{19.56}{0.3})}=0.239$

$Iv(ze=31.2m)=\frac{1}{1\times\ln(\frac{31.2}{0.3})}=0.215$

The values of Cr and Iv is replaced in the formulas of Ce and q_p which is obtain these values in the following table:

Table 2.1: Data and calculation results of peak dynamic pressure $q_p(ze)$

Ze(m)	Ct(ze)	Cr(ze)	Iv(ze)	Ce(ze)	$q_{ref}(N/m^2)$	$q_p(N/m^2)$
19.56	1	0.898	0.239	2.155	435	937.425
31.2	1	0.998	0.215	2.497	435	1086.195

2.3.7 Dynamic coefficient Cd:

The height of this building is 31.2 m greater than 15 m so the dynamic coefficient is determined according to this expression that it's given from 3.1 RNV2013:

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

Such as:

z_{eq}: the height equivalent of the construction (C*f* fig 3.1 RNV):

$z_{eq}=0.6\times h=0.6\times 31.2=18.72$ m > $z_{min}=5$m (vertical construction such as building).

$Iv(z_{eq}=18.72m) = \frac{1}{Ct(zeq)\times \ln\left(\frac{zeq}{z0}\right)} = \frac{1}{1\times \ln(\frac{18.72}{0.3})} = 0.242$

- ***Quasi-static part Q^2:***

Given by this formula: $Q^2=\frac{1}{1+0.9\times\left(\frac{(b+h)}{Lt(zeq)}\right)^{0.63}}$

such as: b=19.56m, h=31.2m (for all directions)

$L_t(z_{eq})$: the turbulence length scale: $L_t(z_{eq})=300\times(\frac{zeq}{200})^{\varepsilon}$ $\quad z_{min}=5$ m< $z_{eq}=18.72$m <200m

$L_t(z_{eq}=18.72m)=300\times(\frac{18.72}{200})^{0.61} = 70.73$ so: $Q^2=\frac{1}{1+0.9\times\left(\frac{(19.56+31.2)}{70.73}\right)^{0.63}}=$***0.58***

- ***Resonant part R^2:***

Resonant part is defined as follows: $R^2=\frac{\pi}{2\times\delta}\times R_N\times R_h\times R_b$

Such as: R_N: the non-dimensional function of the spectral density of the power given by following expression: $R_N=\frac{6.8\times Nx}{(1+10.2\times Nx)^{5/3}}$

And N_x: the non- dimensional frequency: $N_x=\frac{nIx\times Lt(zeq)}{Vm(zeq)}$

n_{lx}: fundamental frequency, according to 3.3.4 RNV2013 is given by:

$n_{lx}=\frac{0.5}{\sqrt{f}}$ (H=31.5m <50m) $\quad f=\frac{H}{100}=0.312$ so $n_{lx}=0.895$ Hz

V_m (z_{eq}): wind mean speed according to Annex 2: $V_m(z_{eq})=Cr(z_{eq})\times Ct(z)\times V_{ref}$

$V_{ref}=27$ m/s (zone II of wind)

$Cr(z_{eq}=18.72m) = 0.889$, Ct=1 so $V_m=0.889\times1\times27=$ **24 m/s**

$N_x=\frac{0.895\times 70.73}{24} = 2.64$

$R_N=\frac{6.8\times 2.64}{(1+10.2\times 2.64)^{5/3}}=0.0698$ $\quad$ **R_N=0.07**

R_h and R_b are the aerodynamic function given by:

$R_h=\frac{1}{\eta h}-\frac{1}{2\times\eta h^2}\times(1-e^{-2\times\eta h^2})$ for $\eta_h>0$

$R_b=\frac{1}{\eta b}-\frac{1}{2\times\eta b^2}\times(1-e^{-2\times\eta b^2})$ for $\eta_b>0$

$\eta_h=\frac{4.6\times Nx\times h}{Li(zeq)}=\frac{4.6\times 2.64\times 31.2}{70.73}=5.36$ so: $R_h=0.17$

$\eta_b=\frac{4.6\times Nx\times h}{Li(zeq)}=\frac{4.6\times 2.64\times 19.56}{70.73}=3.36$ so: $R_b=0.253$

δ: logarithmic decrement of vibration damping which is given by : $\delta=\delta_s+\delta_a$

δ_s: logarithmic decrement of structural damping =0.05 because we have steel building

δ_a: logarithmic decrement of aerodynamic damping =0

so $\delta=0.05$ and $R^2=\frac{\pi}{2\times 0.05}\times 0.07\times 0.17\times 0.253=0.297$

$$\mathbf{R^2=0.297}$$

- ***Peak factor g:***

Peak factor is given by this expression: $g=\sqrt{2\times\ln(600\times v)}+\frac{0.6}{\sqrt{2\times\ln(600\times v)}}>3$

V: Mean frequency given by: $V=n_{lx}\times\sqrt{\frac{R^2}{Q^2+R^2}}\geq 0.08$

$V=0.895\times\sqrt{\frac{0.297}{0.58+0.297}}=0.521>0.08$

$g=\sqrt{2\times\ln(600\times 0.521)}+\frac{0.6}{\sqrt{2\times\ln(600\times 0.521)}}=3.57>3$ $g=3.57$

so: $C_d=\frac{1+2\times 3.57\times 0.242\times\sqrt{0.58+0.297}}{1+7\times 0.242}=0.972$

So, for all direction of V_1 V_2 V_3 V_4 the dynamic coefficient takes this value: **Cd=0.972**

2.3.8 Aerodynamic pressure W(zj):

Aerodynamic pressure determined by 2.5.2 RNV2013: $W(zj)=q_p(z_e)\times[C_{pe}-C_{pi}]$ [N/m^2].

- ***Coefficient of external pressure C_{pe} :***

The coefficient of external pressure given by 5.1.1.2 RNV2013:

$-C_{pe}=C_{pe.1}$ if: $S \leq 1\ m^2$

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

$-C_{pe}= C_{pe.10}$ if: $S \geq 10\ m^2$

- **Vertical Walls:**

From 5.1.2 RNV2013, the division of the walls is obtained According to the figure 2.4 below: d=19.56m, b=19.56m, e=min(b;2h), h=30.6m e=min (19.56m;61.2m) e=19.56m, e=d which means:

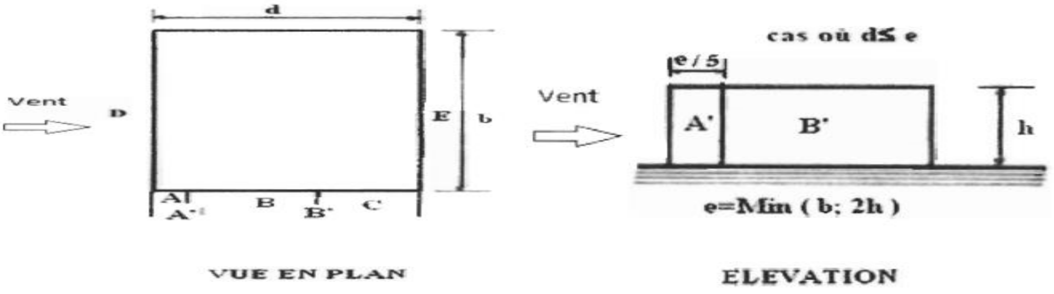


Figure 2.4: Wind in the walls.

- ***Direction 1 of wind (V_1):***

Is shown in the figure 2.5 below:

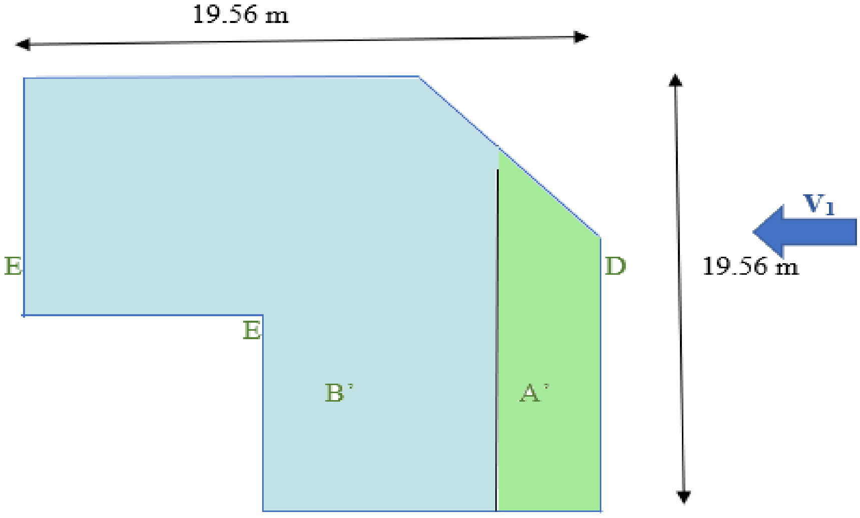


Figure 2.5: Direction of wind V1 in the vertical walls.

From the figures 2.5 and 2.4, the values of coefficient C_{pe} according to the areas of vertical walls zones is in the following table:

Table 2.2: The values of coefficient C_{pe} according to the areas of each zone of vertical walls

zone	h(m)	L(m)	$S(m^2)>10m^2$	$C_{pe}=C_{pe10}$
A'	30.6	3.912	119.71	-1
B'	30.6	15.648	478.83	-0.8
D	30.6	19.56	598.536	+0.8
E	30.6	19.56	598.536	-0.3

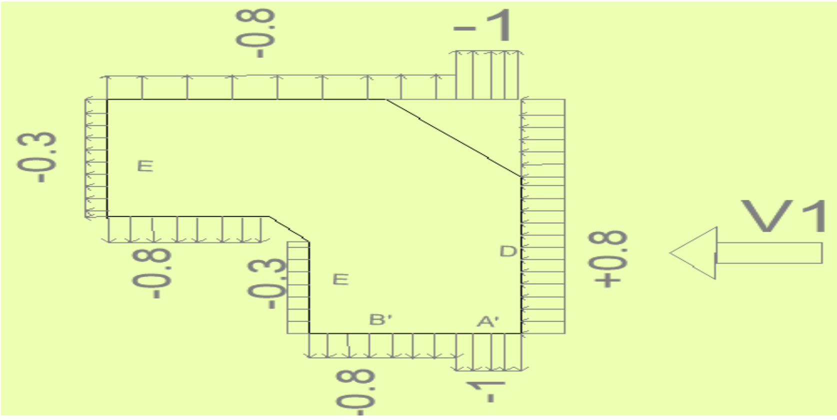


Figure 2.6: Distribution of C_{pe} for the vertical wall V1.

- ***Direction 2 of wind (V_2):***

Is shown in the figure 2.7 below:

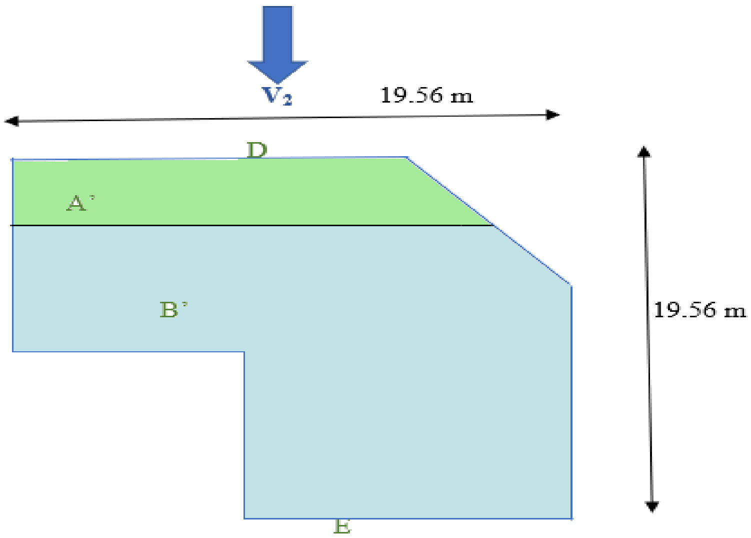


Figure 2.7: Direction of wind V2 in the vertical walls.

Notice 1: There is the same distance in b, d and e in the direction 1 and 2 that's mean the direction 2 has the same values of the Table 2.2 which is given the distribution of C_{pe} in the following figure:

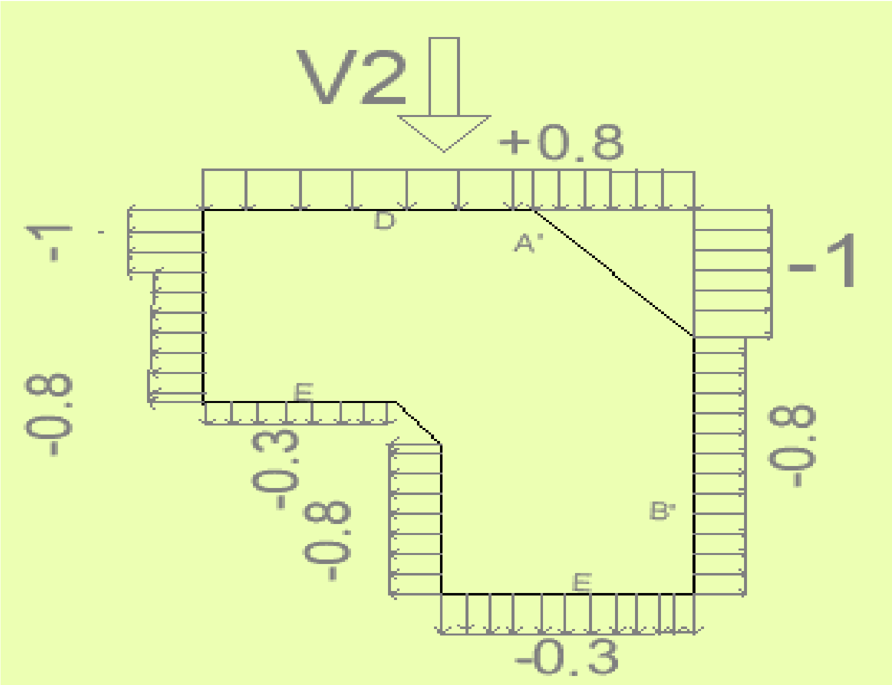


Figure 2.8: Distribution of C_{pe} for the vertical wall V2.

- ***Flat roof:***

The roof should be divided as shown in the following figure:

d=19.56m , b=19.56m , e=min(b;2h) , h=30.6m e=min(19.56m;61.2m) e=19.56m

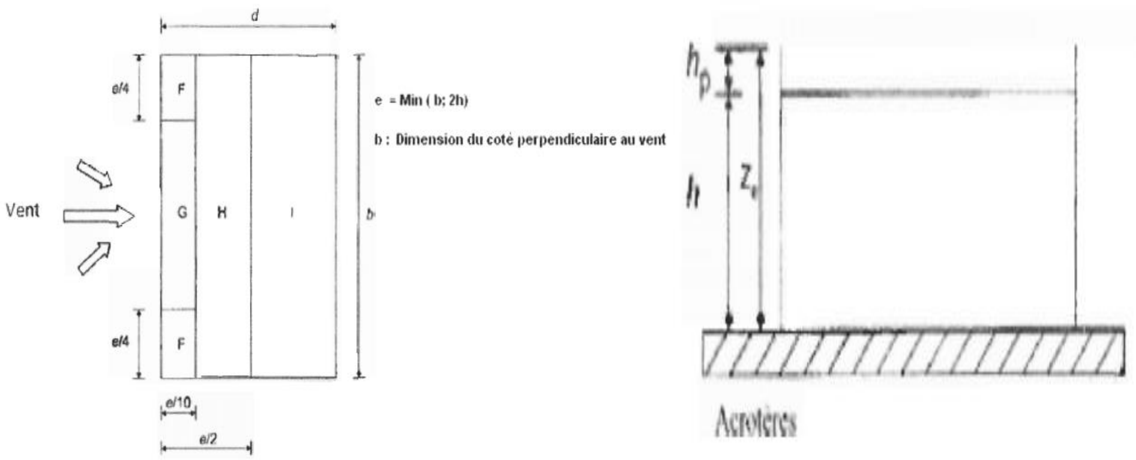


Figure 2.9: Wind in the flat roof(parapet).

- ***Direction 1 of wind (V_1):***

Is shown in the figure 2.10 below:

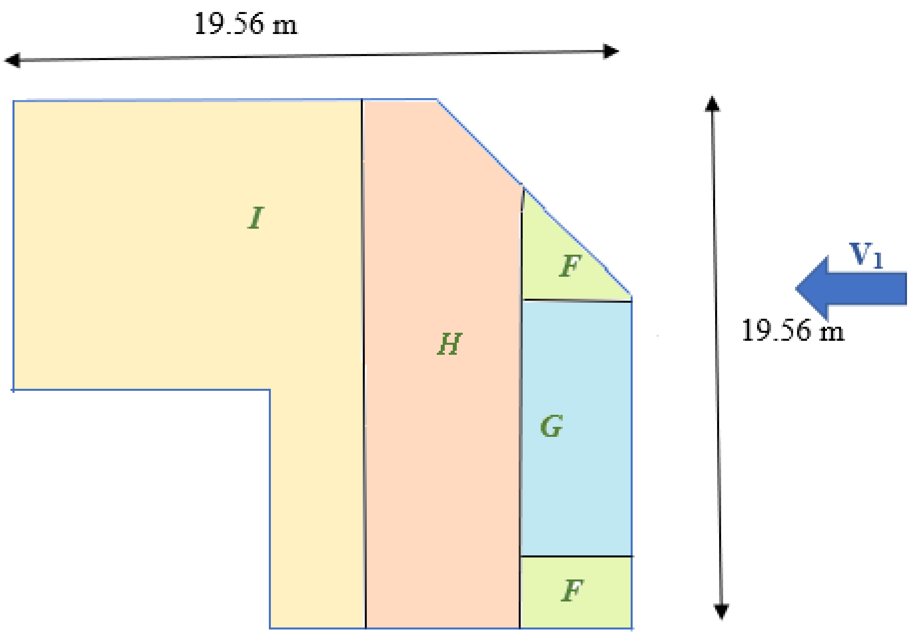


Figure 2.10: Direction of wind V1 in the flat roof.

h_p/h=0.6/30.6=0.02 approx to 0.025. **(5.1.3 table 5.3 RNV2013).**

The result of C_{pe} and areas are in the following table:

Table 2.3: The values of coefficient C_{pe} according to the areas of each zone of the roof

Zone	Area S(m^2) >10m^2	Coefficient C_{pe}= C_{pe10}
F	19.129	-1.6
G	19.129	-1.1
H	153.03	-0.7
I	128.336	±0.2

- ***Direction 2 of wind (V_2):***

Is shown in the figure 2.11 below:

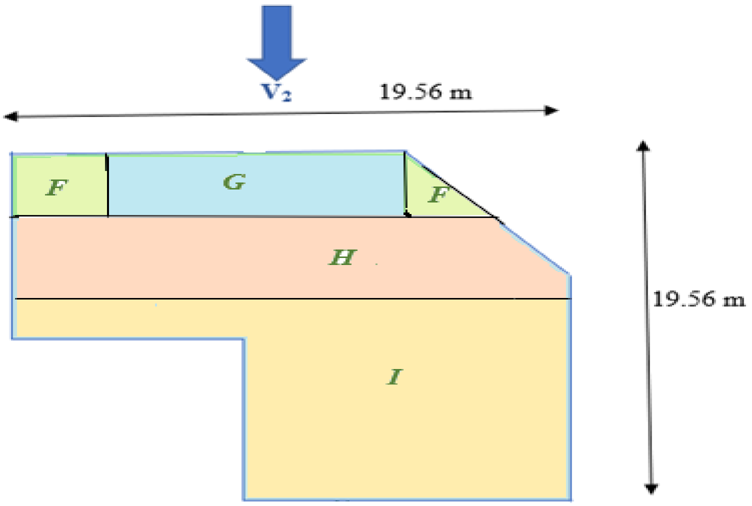


Figure 2.11: Direction of wind V2 in the flat roof.

Notice 2: There are the same distance of b, d and e in the direction 1 and 2 so direction 2 has the same values of the Table 2.3.

- ***Coefficient of Internal pressure C_{pi}:***

To determine the coefficient C_{pi} the following conditions must be checked:

- ✓ The total area of the openings in each face must be less than 30 % of the area of the face. $A_{openings}$= 119.448 m^2 and A_{face}=598.536 m^2

  $\frac{119.448}{598.536}$=19.95 % < 30 % ***condition verified.***

- ✓ The area of the openings in the face is more than three times of the area of the openings in the other faces:

$A_{openings1}$=119.448m^2 ; $A_{openings2}$=17.388m^2 ***condition verified.***

The coefficient of internal pressure C_{pi} is determined according to the following expression: $\mathbf{C_{pi}=0.9C_{pe}}$

According to RNV2013:

In the vertical walls: C_{pe}=0.8; $\mathbf{C_{pi}=0.9\times0.8=0.72}$

In the roof: C_{pe}=0.7; $\mathbf{C_{pi}=0.9\times0.7=0.63}$

The results of aerodynamic pressure W(zj) in the direction 1 and 2 of wind V1, V2 are in the following tables:

Table 2.4: Values of aerodynamic pressure in the vertical walls (V1 and V2)

Zone	Ze(m)	$q_{p(}(zj)[N/m^2]$	C_{pe}	C_{pi}	$W(zj)[N/m^2]$
A'	19.56	937.425	-1	0.72	-1612.371
	31.2	1086.195	-1	0.72	-1868.255
B'	19.56	937.425	-0.8	0.72	-843.6825
	31.2	1086.195	-0.8	0.72	-1424.886
D	19.56	937.425	+0.8	0.72	74.994
	31.2	1086.195	+0.8	0.72	86.896
E	19.56	937.425	-0.3	0.72	-956.174
	31.2	1086.195	-0.3	0.72	-1107.919

Table 2.5: Values of aerodynamic pressure in the roof (V1 and V2)

Zone	Ze(m)	$q_{p(}(zj)[N/m^2]$	C_{pe}	C_{pi}	$W(zj)[N/m^2]$
F	19.56	937.425	-1.6	-0.63	-909.3
	31.2	1086.195	-1.6	-0.63	-1053.609
G	19.56	937.425	-1.1	-0.63	-440.59
	31.2	1086.195	-1.1	-0.63	-510.51
H	19.56	937.425	-0.7	-0.63	-65.62
	31.2	1086.195	-0.7	-0.63	-76.034
I	19.56	937.425	+0.2	-0.63	778.063
			-0.2	-0.63	403.09
	31.2	1086.195	+0.2	-0.63	901.542
			-0.2	-0.63	467.064

2.3.9 Friction forces:

The friction effect of wind in the area can be neglected if: 2d.h≤ 4×2b.h **(2.6.3 RNV2013)** 2×19.56×31.2=1220.544m<4×2×19.56×31.2=4882.176m

The condition is verified for V1 and V2 so the friction forces are neglected.

2.3.10 Calculation of forces using pressure surfaces:

Can be determined F_w by summation F_{ew} and F_{wi} by the following expressions according to 2.6.2 RNV2013:

-external forces: $F_{ew}=C_d \times\sum W_e \times A_{ref}$

Such as: W_e: external pressure given by: $W_e=q_p(z_e)\times C_{pe}$

-internal forces: $F_{wi}=\sum W_i\times A_{ref}$

Such as W_i: internal pressure given by: $W_i=q_p(z_i)\times C_{pi}$

The results are in the following tables for the vertical walls and roof according to directions V1 and V2:

Table 2.6: Results of external forces and internal forces in vertical walls

Ze(m)	Zone	$q_p(z)$	C_{pe}	C_{pi}	$W_e[N/m^2]$	W_i	$S(m^2)$	C_d	$F_{ew}(kN)$	$F_{wi}(kN)$
19.56	A’	937.425	-1	0.72	-937.425	674.946	119.71	0.972	-185.430	1178
	B’	937.425	-0.8	0.72	-749.94	674.946	478.83			
	D	937.425	+0.8	0.72	749.94	674.946	598.536			
	E	937.425	-0.3	0.72	-281.228	674.946	598.536			
31.2	A’	1086.195	-1	0.72	-1086.195	782.06	119.71	0.972	-221.047	1364.96
	B’	1086.195	-0.8	0.72	-868.956	782.06	478.83			
	D	1086.195	+0.8	0.72	868.956	782.06	598.536			
	E	1086.195	-0.3	0.72	-325.859	782.06	598.536			

Table 2.7: Results of external forces and internal forces in the roof

Ze(m)	zone	$q_p(z)$	C_{pe}	C_{pi}	$W_e[N/m^2]$	W_i	$S(m^2)$	C_d	$F_{ew}(kN)$	$F_{wi}(kN)$
31.2	F	1086.195	-1.6	0.63	-1737.912	684.3	19.129	0.972	-167.626	310.67
	G	1086.195	-1.1	0.63	-1194.814	684.3	19.129			
	H	1086.195	-0.7	0.63	-760.337	684.3	153.03			
	I	1086.195	+0.2 -0.2	0.63	217.239 -217.239	684.3	128.336			

2.4 CONCLUSION:

- ✓ The results of snow loads are used in chapter 3 to calculate the element's profiles of the roof.
- ✓ The wind loads can be neglected in the dynamic study because the building is heavy and therefore it is not necessary to consider winds loads.

CHAPTER 3

PRE-DIMENSIONING OF THE STRUCTURAL ELEMENTS

3.1 INTRODUCTION:

The purpose of the pre-dimensioning is to define the dimensions of the structure's different elements (beams, columns...). These dimensions are chosen according to the regulations EC3, EC4 and DTR2.2. The application of these rules leads to the optimal compromise between cost and safety.

Each element is pre-dimensioned according to two conditions:

-The deflection condition to determine the appropriate profile type;

-The verification according to the strength condition.

3.2 LOAD ASSESSMENTS AND OVERLOADS:

3.2.1 *Dead loads:*

- ***Inaccessible roof:***
  - ✓ Gravel protection (thickness=4cm): 17×0.04 = 0.68 kN/m^2
  - ✓ Thermal insulation (4cm): 0.16 kN/m^2
  - ✓ Multilayer waterproofing: 0.12 kN/m^2
  - ✓ Plaster ceiling (3cm): 10×0.03 = 0.30 kN/m^2
  - ✓ Slop form (1.5%) = 2.2 kN/m^2
  - ✓ Profiled steel decking TN40: 0.12 kN/m^2
  - ✓ Reinforced concrete slab (12cm): 25×0.12 = 3.00 kN/m^2

Therefore, the total dead loads of inaccessible roof are: **G = 6.58kN/m^2**

- ***Current floor:***
  - ✓ Laying bricks: 0.40 kN/m^2
  - ✓ Tile: 0.40 kN/m^2
  - ✓ Sand bed: 0.54 kN/m^2
  - ✓ Inner wall: 1.00 kN/m^2
  - ✓ Plaster ceiling (3cm): 0.30 kN/m^2
  - ✓ Profiled steel decking TN40: 0.12 kN/m^2
  - ✓ Reinforced concrete slab (12cm): 3.00 kN/m^2

Therefore, the total dead loads of current floor are: **G = 5.76 kN/m^2**

3.2.2 Live loads:

- ***Inaccessible roof:*** **Q = 1 kN/m²**
- ***Current floor:*** residential use: **Q = 1.5 kN/m²**

3.2.3 Loads combination:

- ***Inaccessible roof :***
  - ✓ ***Ultimate limit state ULS :*** q_u=1.35G+1.5Q=1.35×6.58+1.5×1=10.38kN/m^2
  - ✓ ***Serviceability limit state SLS:*** q_s=G+Q=6.58+1=7.58kN/m^2
- ***Current floor:***
  - ✓ ***Ultimate limit state ULS :*** q_u=1.35G+1.5Q=1.35×5.76+1.5×1.5=10.026kN/m^2
  - ✓ ***Serviceability limit state SLS:*** q_s=G+Q=5.76+1.5=7.26kN/m^2

3.3 PRE-DOMENSIONING OF STRUCTURAL ELEMENTS:

3.3.1 Joists:

The joists are beams made of IPE or IPN that work under simple bending. Their spacing is practically determined by the following expression: 0.7m ≤ e ≤ 1.5m,

Length of main beams: L=5.1m so the spacing between joists are: e=1.275m.

The most stressed joists have the length L = 5m.

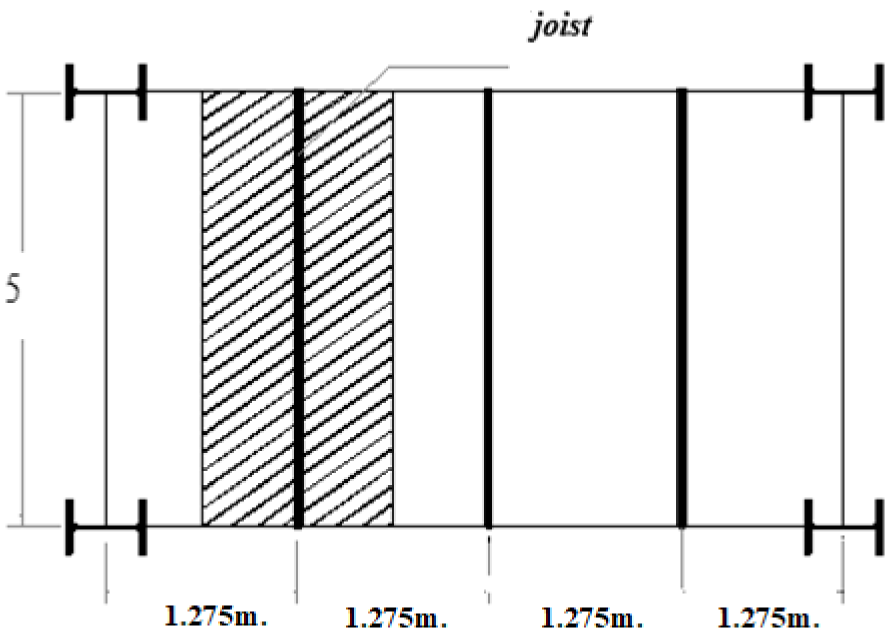


Figure 3.1: joists position.

- ***The profile of joists:***

The profile of joists adopted by the following expression: $\frac{L}{25} \leq h \leq \frac{L}{15}$ such as:

h: is the height of the profile of joists.

L=5m: is the span of the joists.

$200 \leq h \leq 333.33$ h=200 the profile of joists adopted is IPE200.

Table 3.1: characteristics of profile IPE200

profile	Weight G(Kg/m)	Area A(cm^2)	h(mm)	b(mm)	t_f(mm)	t_w(mm)	r(mm)	W_{ply}(cm^3)	W_{plz}(cm^3)	I_y(cm^4)	I_z(cm^4)
IPE200	22.4	28.5	200	100	8.5	5.6	12	220.6	44.61	1943	142.4

- ***Section classification:***

➢ Part subject to bending (web):

$c/t \leq 72\varepsilon$ such as: $\varepsilon=\sqrt{\frac{235}{fy}}$ and fy=235MPa so ε=1

c/t= (200-2×8.5)/5.6=32.68<72 so web of class 1

➢ Part subject to compression (flange):

$c/t \leq 9\varepsilon$ = (100-5.6)/2-12 = 35.2/8.5 = 4.14< 10 so flange of class 1

So, profile IPE200 is classified in class 1.

- ***Construction phase:***

The construction phase corresponds to the concreting phase of the slab, when the concrete has not yet hardened, strength is then ensured by the steel profile. The loads during the construction phase are the same in inaccessible roof and current floor which are in the following:

-Self-weight of the steel profile (IPE200): G_p=0.224 kN/m

-Dead weight of fresh concrete: G_c= 3.00 kN/m^2

-Self-weight of profiled steel decking: G_{sd} =0.12 kN/m^2

-Construction overload of worker: Q_w=0.75 kN/m^2

- ***Loads combinations:***
  - **ULS:** q_u=1.35G+1.5Q=1.35(G_p+e× (G_{c+} G_{sd})) +1.5×e×Q_w

q_u=1.35(0.224+1.275× (3+ 0.12)) +1.5×1.275×0.75

q_u =7.11 kN/m

  - **SLS:** q_s=G+Q= G_p+e× (G_{c+} G_{sd}) + e×$Q_{w=}$0.224+1.275× (3+ 0.12) +1.275×0.75

q_s =5.16 kN/m

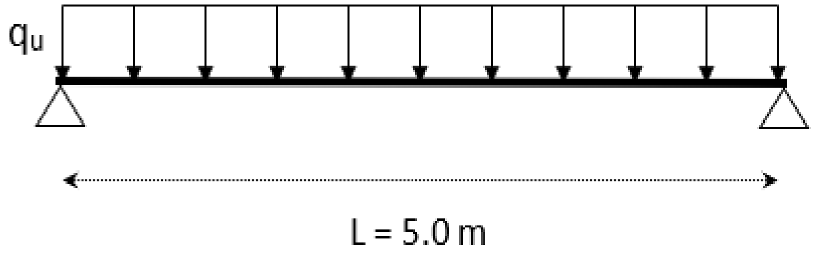


Figure 3.2 : Most stressed static joist diagram

- ***Verification of profile:***
  - ***Bending strength:***

The condition following should be verified: $M_{sd} \leq M_{plrd}=\frac{Wply \times fy}{\gamma Mo}$ such as:

$M_{sd:}$ the moment applied to the joist which is the maximum moment: $M_{max}=\frac{qu \times L^2}{8}$

M_{plrd}: plastic moment resistance. Such as: γ_{Mo}=1

$M_{sd}=M_{max}=\frac{7.11 \times 5^2}{8}$=22.22 kN.m and: $M_{plrd}=\frac{220.6 \times 235}{1} \times 10^{-3}$=51.84 kN.m

M_{sd} =22.22 kN.m< M_{plrd}=51.84 kN.m ***Condition verified***

The bending strength is verified

  - ***Shear strength:***

The condition following should be verified: $V_{sd} \leq V_{plrd}=\frac{Av \times fy}{\sqrt{3} \times \gamma Mo}$ such as:

V_{sd}: shear force that is calculated in the joist. $V_{sd}=\frac{qu.L}{2}$

V_{plrd}: plasticizing shear force of the section.

A_v: area of shear which is given by: $A_v=A-2.b.t_f+(t_w+2.r).t_f$

$A_v=2850-2\times100\times8.5+(5.6+2\times12)\times8.5=1401.6\ mm^2$

$V_{sd}=\frac{7.11\times5}{2}=17.78$ kN and: $V_{plrd}=\frac{1401.6\times235}{\sqrt{3}\times1}\times10^{-3}=190.165$ kN

$V_{sd}=17.78$ kN$<V_{plrd}=190.165$ kN ***Condition verified***

Shear strength is verified

- ***Verification of interaction between moment and shear force:***

If: $V_{sd}\leq 0.5V_{plrd}$ that's mean there is no interaction between moment and shear force.

$V_{sd}=17.78$ kN$<V_{plrd}=0.5\times190.165=95.083$ kN **No interaction**

- ***The deflection:***

The condition following should be verified: $f^{max}=\frac{5\ qs.L^4}{384.E.Iy}\leq f_{allow}$

Such as: $f_{allow}=\frac{L}{250}=\frac{5000}{250}=20mm$, $f^{max}=\frac{5\times5.16\times5000^4}{384\times2.1\times10^5\times1943\times10^4}=10.29mm$

$f^{max}=10.29mm < f_{allow}=20mm$

- ***Lateral buckling:***

The condition following should be verified: $M_{sd}\leq M_{brd}=X_{lt}\times\beta_w\times W_{ply}\times\frac{fy}{\gamma M1}$ with:

$X_{lt}=\frac{1}{\varnothing lt+\sqrt{\varnothing lt^2-\lambda lt^2}}$ such as: $\varnothing_{lt}=0.5[1+\alpha_{lt}(\bar{\lambda}_{lt}-0.2)+\bar{\lambda}_{lt}^2]$ and $\bar{\lambda}_{LT}=\sqrt{\frac{\beta w.Wply.fy}{Mcr}}=\frac{\lambda LT}{\lambda 1}\sqrt{\beta w}$

The following expression can be used: $\lambda_{LT}=\frac{\frac{Kl}{iz}}{\sqrt{C1}\times\left[\left(\frac{k}{kw}\right)^2+\frac{1}{20}\left(\frac{\frac{kl}{iz}}{\frac{h}{tf}}\right)^2\right]^{1/4}}$

With: $\gamma M1=1.1$

k=1 no lateral support, $k_w=1$, $C_1=1.132$, $\beta_w=1$ section of class 1, $\alpha_{lt}=0.21$ rolled profile

$\lambda_{LT}=\frac{\frac{5000}{22.4}}{\sqrt{1.132}\times\left[\left(\frac{1}{1}\right)^2+\frac{1}{20}\left(\frac{\frac{5000}{22.4}}{\frac{200}{8.5}}\right)^2\right]^{1/4}}=136.997$, $\lambda_1=93,9\,\varepsilon=93,9\sqrt{\frac{235}{f_y}}=93,9$

$\bar{\lambda}_{LT}=\frac{136.997}{93.9}\sqrt{1}=1.46>0.4$ There is a risk of lateral buckling.

$Ø_{lt}$=0.5[1+0.21(1.46-0.2) + 1.46^2]=1.698

$X_{lt}=\frac{1}{1.698+\sqrt{1.698^2-1.46^2}}$=0.4

$M_{brd}=0.4\times1\times220.6\times10^3\times\frac{235}{1.1}\times10^{-6}$=18.85 kN.m

M_{sd} =17.97 kN.m < M_{brd}=18.85 kN.m ***Condition verified***

- ***Final phase:***

The concrete gets hardened, steel decking and slab (mixed section) are working together.

➢ ***Calculated of the position plastic neutral axis:***

- ✓ Effective slab width: b_{eff}=inf $\{\frac{2L}{8}$, b$\}$ = $\{\frac{2*5}{8} = 1.25$, 1.275m$\}$ **b_{eff} =1250mm**
- ✓ force section of steel: $R_s=0.95\times A_s\times f_y=0.95\times2850\times235\times10^{-3}$ **R_s=636.263kN**
- ✓ force section of concrete: $R_c=0,57\times f_{ck}\times b_{eff}\times h_c$

R_c =0.57×25×1250×65×10^{-3} **R_c=1157.81kN**

R_c=1157.81kN > R_s=636.263kN ***the situation of neutral axis is in the concrete slab***

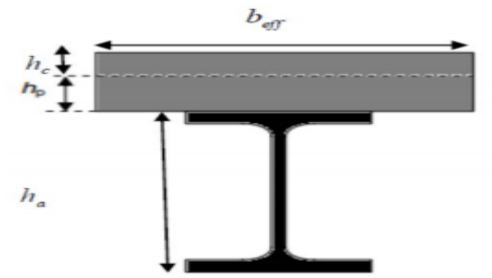


Figure 3.3: Position of neutral axis.

- ***Inaccessible roof:***

-Self-weight of the steel profile (IPE200): G_p=0.224 kN/m

-Dead loads: G=6.58 kN/m^2

-Live loads: Q=1 kN/m^2

➢ ***Loads combinations:***

- ✓ **ULS :** q_u=1.35G+1.5Q=1.35(G_p+e×G) +1.5×e×Q

q_u=1.35(0.224+1.275×6.58) +1.5×1.275×1 **q_u =13.54 kN/m**

- **SLS:** q_s=G+Q= G_p+e×G+e×Q=0.224+1.275×6.58+1.275×1

q_s =9.89 kN/m

- ***Verification of profile:***
  - ***Bending strength:***

The condition following should be verified: $M_{sd} \leq M_{plrd}$ such as:

$M_{sd}=\frac{qu.l^2}{8}=\frac{13.54\times5^2}{8}$=42.31 kN.m, $M_{plrd}=R_s\times[\frac{ha}{2}+hc+hp-(\frac{Rs}{Rc}\times\frac{hc}{2})]$

$M_{plrd}=636.263\times[\frac{200}{2}+65+55-(\frac{636.263}{1157.81}\times\frac{65}{2})]\times10^{-3}$=128. 61kN.m

M_{sd}=42.31 kN.m < M_{plrd}=128.61 kN.m ***Condition verified*** ***Bending strength is verified***

- ***Shear strength:***

The condition following should be verified: $V_{sd} \leq V_{plrd}=\frac{Av\times fy}{\sqrt{3}\times\gamma Mo}$ such as:

$V_{sd}=\frac{qu.L}{2}=\frac{13.54\times5}{2}$=33.85kN, and: V_{plrd}=190.165 kN

V_{sd}=33.85 kN<V_{plrd}=190.165 kN ***Condition verified***

Shear strength is verified

- ***Verification of interaction between moment and shear force:***

V_{sd}=33.85kN< V_{plrd}=0.5×190.165 =95.08 kN **No interaction**

- ***The deflection:***

The condition following should be verified: $f^{total}=f_1+f_2=\frac{5\ qs.L^4}{384.E.Ic} \leq f_{allow}$

Such as: $f_2=\frac{5\ qs.L^4}{384.E.Ic}$ and $I_c=\frac{A_a.(h_c+2.h_p+h_a)^2}{4.(1+mv)}+\frac{b_{eff}.h_c^3}{12.m}+I_a$ and - $V=\frac{A_a}{A_b}=\frac{2850}{1250\times55}$=0.04

$$m=\frac{E_a}{E_b}=15\ ,\ Ic=\frac{2850\times(65+2\times55+200)^2}{4(1+15\times0.04)}+\frac{1250\times65\text{^}3}{12\times15}+1943\times10^4=8.396\times10^7 mm^4$$

$f^{total}=\frac{5\times9.89\times5000^4}{384\times2.1\times10^5\times8.396\times10^7}$ =4.564mm ***f^{total}=10.29+4.56=14.85mm < f^{allow}=20mm C.V***

- ***Lateral buckling:***

The lateral buckling should not be checked because the upper flange of joist is held laterally by the concrete slab.

All the conditions of resistance are verified in the inaccessible roof, so the profile IPE200 is adopted.

The same steps are followed for the current floor using the same profile IPE200, all the conditions are verified, so the profile IPE200 is adopted for the inaccessible roof and for the current floor. The results are in the following table:

Table 3.2: Results of strength verification of joists in the current floor in the final phase profile IPE200

Current floor (Final phase)			
q_u=13.09 kN/m	M_{sd}=40.91 kN.m	M_{plrd}=128.61 kN.m	Condition verified
	V_{sd}=32.73 kN	V_{plrd}=190.165kN	Condition verified
q_s=9.48kN/m	f^{total}=14.67 mm	f^{allow}=20mm	Condition verified

- ***Connectors:***

The connectors are elements that work as a link between the concrete slab and the structural steel element. Headed stud shear connectors are used in this structure, which have the following characteristics: height h=95mm and diameter d=19mm.

The design shear resistance of a headed stud automatically welded in accordance with EN14555 should be determined from: $P_{rd}= K_T \times min[\frac{0.8 fu \times \pi \times \frac{d^2}{4}}{\gamma v} \ ; \ \frac{0.29.\alpha.d^2 \times \sqrt{fck\ Ecm}}{\gamma v}]$

f_u=360MPa; f_{ck}=25MPa; E_{cm}=30500MPa; γ_v=1.25; α=1 for $\frac{h}{d}$=5>4

$P_{rd}= K_T \times \min [73.13; 65.33] = K_T \times 65.33$

Profiled steel decking influence:

The reducing coefficient K_T must be inferior of 1 and determined by the following formula:

$K_T = \frac{0.7}{\sqrt{Nr}} \times \frac{bo}{hp} \times [\frac{h}{hp} - 1]$ N_r=1: number of connectors per rib, b_o=88.5mm; h_p=55mm

$K_T = \frac{0.7}{\sqrt{1}} \times \frac{88.5}{55} \times [\frac{65}{55} - 1]$=0.25<1 ; P_{rd}= 65.33kN

Shear force taken up by the connectors:

R_L=inf ($R_{concrete}$; R_{steel}) RL= (1157.81kN;636.263kN) =636.263kN

Connectors Number in half-span:

$N_{br}=\frac{RL}{Prd}=\frac{636.263}{65.33}$=9.74 N_{br}=10 in half-span which means 20 in all the beam

Connector spacing:

E_{min}>5×d=5×19=95mm

E_{max} <6×95=750mm

$E_{sp}=\frac{L}{Nbr-1}=\frac{5000}{20-1}$=263.16mm E_{min}< 263.16mm <E_{max} so: E_{sp}=260mm

3.3.2 Beams calculation:

- ***Main beams:***

The main beams are structural elements, they support the floor loads and transmit them to columns. They are stressed mainly under bending moment.

The most stressed beam has a length of 5.1m; it supports four concentrated loads (R_{joist}) which represent the reaction of the joists, a uniformly distributed load (Own weight of beam) and the weight of the concrete slab over the width of the flange.

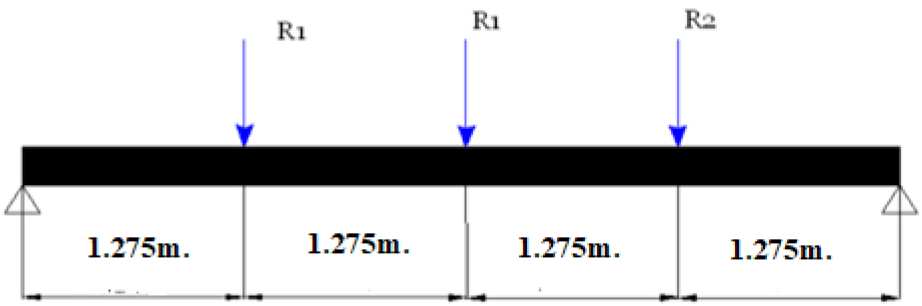


Figure 3.4: static diagram of the main beam.

- ***The profile of main beams:***

The profile of main beam adopted by the following expression: $\frac{L}{25}\leq h \leq\frac{L}{15}$ such as:

h: is the height of the profile of main beam.

L=5.1m: is the span of the main beam.

$204 \leq h \leq 340$ h=270 the profile of main beams is IPE270.

Table 3.3: characteristics of profile IPE270

profile	Weight G(Kg/m)	Area A(cm^2)	h(mm)	b(mm)	t_f(mm)	t_w(mm)	r(mm)	W_{ply}(cm^3)	W_{plz}(cm^3)	I_y(cm^4)	I_z(cm^4)
IPE270	36.1	45.94	270	135	10.2	6.6	15	484	96.95	5790	419.9

- ***Section classification:***

➢ Part subject to bending (web):

$c/t \leq 72\varepsilon$ such as: $\varepsilon=\sqrt{\frac{235}{fy}}$ and fy=235MPa   so $\varepsilon=1$

$c/t= (270-2\times10.2)/6.6=37.82<72$ so web of class 1

➢ Part subject to compression (flange):

$c/t \leq 9\varepsilon$ = (135-6.6)/2-15 = 49.2/10.2 = 4.82< 10 so flange of class 1

So, profile IPE270 is classified in class 1.

- ***Construction phase:***

The loads on the construction phase are the same in inaccessible roof and current floor which are in the following:

-Self-weight of the steel profile (IPE270): G_p=0.361 kN/m

-Dead weight of fresh concrete: G_c= 3.00 kN/m^2

-Self-weight of profiled steel decking: G_{sd} =0.12 kN/m^2

-Construction overload of worker: Q_w=0.75 kN/m^2

➢ ***Loads combinations:***

✓ **ULS:** $q_u=1.35G+1.5Q=1.35(G_p+b_b\times(G_{c+}\ G_{sd}))+1.5\times b_b\times Q_w$

$$q_u=1.35(0.361+0.135\times(3_+\ 0.12))+1.5\times0.135\times0.75$$

$$\mathbf{q_u =1.21\ kN/m}$$

✓ **SLS:** $q_s=G+Q= G_p+b_b\times (G_{c+}\ G_{sd}) + b_b\times Q_{w=}0.361+0.135\times (3_+\ 0.12) +0.135\times0.75$

$$\mathbf{q_s =0.883\ kN/m}$$

- ***Calculation of joists reactions:***

For each phase (construction phase and final phase) the joists reactions are calculated using the following formula: $R_{joist} = \frac{qu.L}{2}$

- **ULS:**

-Joists of span 5m: $R_{u1}= \frac{7.11\times5}{2}$=17.775kN

- Joists of span 3.5m: $R_{u2}= \frac{7.11\times3.5}{2}$=12.44 kN

$Ru1+R_{u2}$=17.775+12.44 **R_u=30.22 kN**

- **SLS:**

-Joists of span 5m: $R_{s1}= \frac{5.16\times5}{2}$=12.9 kN

-Joists of span 3.5m: $R_{s2}= \frac{5.16\times3.5}{2}$=9.03 kN

$R_{s1}+R_{s2}$=12.9+9.03 **R_s=21.93 kN**

- ***Verification of profile:***
  - ***Bending strength:***

The condition following should be verified: $M_{sd} \leq M_{plrd}=\frac{Wply\times fy}{\gamma Mo}$

$M_{sd}=\frac{qu\times L^2}{8}+\frac{Ru.L}{2}=\frac{1.21\times5.1^2}{8}+\frac{30.22\times5.1}{2}$=81 kN.m and: $M_{plrd}=\frac{484\times235}{1}\times10^{-3}$=113.74 kN.m

M_{sd} =81 kN.m< M_{plrd}=113.74 kN.m ***the bending strength is verified.***

- ***Shear strength:***

The condition following should be verified: $V_{sd} \leq V_{plrd}=\frac{Av\times fy}{\sqrt{3}\times\gamma Mo}$ such as:

$V_{sd}=\frac{qu.L}{2}+\frac{3Ru}{2}=\frac{1.21\times5.1}{2}+\frac{3\times30.22}{2}$=48.42 kN

A_v=4594-2×135×10.2+(6.6+2×15) ×10.2=2213.32 mm^2

$V_{plrd}=\frac{2213.32\times235}{\sqrt{3}\times1}\times10^{-3}$=300.3 kN

V_{sd}=48.42 kN<V_{plrd}=300.3 kN ***Condition verified***

Shear strength is verified

✓ ***Verification of interaction between moment and shear force:***

V_{sd}=48.42kN< V_{plrd}=0.5×300.3=150.15 kN **No interaction**

✓ ***The deflection:***

The condition following should be verified: $f^{\max}=\frac{5\ qs.L^4}{384.E.Iy}+\frac{19.Rs.L^3}{384.E.Iy}\leq f_{allow}$

Such as: $f_{allow}=\frac{L}{250}=\frac{5100}{250}$= 20.4mm,

$$f^{\max}=\frac{5\times0.883\times5100^4}{384\times2.1\times10^5\times5790\times10^4}+\frac{19\times21.93\times10^3\times5100^3}{384\times2.1\times10^5\times5790\times10^4}=11.84$$

$$f^{max}=11.84mm < f_{allow}=20.4mm$$

- ***Final phase:***

The concrete gets hardened, the mixed section (steel decking and slab) working together.

➢ ***calculated the position of plastic neutral axis:***

✓ Effective slab width: $b_{eff}=\inf\{\frac{2L}{8}, b\}=\{\frac{2*5.1}{8}=1.275, 5m\}$ **b_{eff} =1275mm**

✓ force section of steel: $R_s=0.95\times A_s\times f_y=0.95\times4594\times235\times10^{-3}$ **R_s=1025.61kN**

✓ force section of concrete: $R_c=0{,}57\times f_{ck}\times b_{eff}\times h_c=0.57\times25\times1275\times65\times10^{-3}$

R_c=1180.97kN

R_c=1180.97kN > R_s=1025.61kN ***the situation of neutral axis is in the concrete slab***

- ***Inaccessible roof:***

-Self-weight of the steel profile (IPE270): G_p=0.361 kN/m

-Dead loads: G=6.58 kN/m^2

-Live loads: Q=1 kN/m^2

- Snow loads: S=0.736 kN/m^2

➢ ***Loads combinations:***

✓ **ULS:** $q_u=1.35G+1.5Q+0.8S=1.35(G_p+b_b\times G)+1.5\times b_b\times(Q+S)$

q_u=1.35(0.361+0.135×6.58) +1.5×0.135×1+1.5×0.135×(1+0.736) **q_u =2.24 kN/m**

✓ **SLS:** $q_s=G+Q+S=G_p+b_b\times G+b_b\times(Q+S)$

q_s=0.361+0.135×6.58+0.135×(1+0.736) **q_s =1.48 kN/m**

➢ ***Calculation of joists reactions:***

For each phase (construction phase and final phase) the joists reactions are calculated using the following formula: $R_{joist} = \frac{qu.L}{2}$

✓ **ULS:**

-Joists of span 5m: $R_{u1}= \frac{13.54\times5}{2}$=33.85 kN

-Joists of span 3.5m: $R_{u2}= \frac{13.54\times3.5}{2}$=23.695 kN

$R_{u1}+R_{u2}$=33.85+23.695 **R_u=57.545 kN**

✓ **SLS:**

-Joists of span 5m: $R_{s1}= \frac{9.89\times5}{2}$=24.725 kN

-Joists of span 3.5m: $R_{s2}= \frac{9.89\times3.5}{2}$=17.31 kN

$R_{s1}+R_{s2}$=24.725+17.31 **R_s=42.03kN**

➢ ***Verification of profile:***

✓ ***Bending strength:***

The condition following should be verified: $M_{sd} \leq M_{plrd}$ such as:

$$M_{sd}=\frac{qu\times L^2}{8}+\frac{Ru.L}{2}=\frac{2.24\times5.1^2}{8}+\frac{57.545\times5.1}{2}=154.02 \text{ kN.m},$$

$$M_{plrd}=R_s\times\left[\frac{ha}{2}+hc+hp-\left(\frac{Rs}{Rc}\times\frac{hc}{2}\right)\right]$$

$$M_{plrd}=1025.61\times\left[\frac{270}{2}+65+55-\left(\frac{1025.61}{1180.97}\times\frac{65}{2}\right)\right]\times10^{-3}=232.58 \text{ kN.m}$$

M_{sd}=154.02 kN.m < M_{plrd}=232.58 kN.m ***Condition verified***

Bending strength is verified

✓ ***Shear strength:***

The condition following should be verified: $V_{sd} \leq V_{plrd}=\frac{Av\times fy}{\sqrt{3}\times\gamma_{Mo}}$ such as:

$$V_{sd}=\frac{qu.L}{2}+\frac{3Ru}{2}=\frac{2.24\times5.1}{2}+\frac{3\times57.545}{2}=92.03\text{kN} \quad , V_{plrd}=300.3 \text{ kN}$$

V_{sd}=92.03 kN<V_{plrd}=300.3 kN ***Condition verified***

✓ ***Verification of interaction between moment and shear force:***

V_{sd}=92.03kN< V_{plrd}=0.5×300.3 =150.15 kN **No interaction**

✓ ***The deflection:***

The condition following should be verified: $f^{\text{ total}}=f_1+f_2=\frac{5\,qs.L^4}{384.E.Ic}+\frac{19.Rs.L^3}{384.E.Ic}\leq f_{\text{allow}}$

With: $I_c=\frac{A_a.(h_c+2.h_p+h_a)^2}{4.(1+mv)}+\frac{b_{eff}.h_c^3}{12.m}+I_a$ and - $V=\frac{A_a}{A_b}=\frac{4594}{1275\times55}$=0.066

$$m=\frac{E_a}{E_b}=15\ ,Ic=\frac{4594\times(65+2\times55+270)^2}{4(1+15\times0.066)}+\frac{1275\times65\text{^}3}{12\times15}+5790\times10^4=17.41\times10^7mm^4$$

$f_1=\frac{5\times1.48\times5100^4}{384\times2.1\times10^5\times17.41\times10^7}$ =0.36mm**,** $f_2=\frac{19\times42.03\times10^3\times5100^3}{384\times2.1\times10^5\times17.41\times10^7}$ =7.55mm

f^{total}=0.36+7.55=7.91mm < f_{allow}=20.4mm C.V

✓ ***Lateral buckling:***

The lateral buckling should not be checked because the upper flange of beams is held laterally by the concrete slab.

All the conditions of resistance are verified in inaccessible roof, therefore the profile IPE270 is adopted.

The same steps are followed for the current floor using the same profile IPE270, all the conditions are verified, so the profile IPE270 is adopted for the inaccessible roof and for the current floor. The results are in the following table:

Table 3.4: Results of strength verification of main beams in the current floor in the final phase profile IPE270

Current floor (Final phase)				
R_u=44.75 kN	q_u=1.84kN/m	M_{sd}=120.1 kN.m	M_{plrd}=232.58 kN.m	Condition verified
		V_{sd}=94.192 kN	V_{plrd}=300.3kN	Condition verified
R_s=32.43 kN	q_s=1.34kN/m	f^{total}=6.39 mm	f^{allow}=20.4mm	Condition verified

- ***Secondary beams:***

The secondary beams are stressed in the same way as the joists, and have the same span 5m, which means that the same section (IPE200) are adopted.

- ***Connectors of main beams:***

P_{rd}= K_T×65.33

K_T =0.6×$\frac{bo}{hp}$× [$\frac{h}{hp} - 1$] N_r=1: number of connectors per rib, b_o=88.5mm; h_p=55mm

K_T =0.6×$\frac{88.5}{55}$× [$\frac{65}{55} - 1$]=0.17<1

P_{rd}= 65.33kN

Shear force taken up by the connectors:

R_L=inf ($R_{concrete}$; R_{steel}) RL= (1180.97kN;1025.61kN) =1025.61kN

Connectors Number in half-span:

N_{br}=$\frac{RL}{Prd}$=$\frac{1025.61}{65.33}$=15.7 Nbr=16 in half-span which means 32 in all the beam

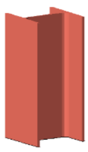
Connector spacing:

E_{min}>95mm , E_{max} <750mm

E_{sp}=$\frac{L}{Nbr-1}$=$\frac{5000}{32-1}$=161.29mm E_{min}< 161.29mm <E_{max} so: E_{sp}=160mm

3.3.3 Calculation of columns:

Column are vertical elements that resist axial compression loads. Generally, steel columns of HEA or HEB are used. They support different loads on the floor (self-weight, dead loads, snow loads and live loads) and transmit them to the foundation. The building of this project includes only steel columns.



- ***Center columns:***

The most stressed columns take for the calculation and pre-dimensioning are **b-3** and **E-6** according to the following area: S=4.25×5.05=21.463 m^2

➢ ***Inaccessible roof:***

-Dead loads of the inaccessible roof: 6.58×21.463=141.22 kN

-Weight of main beams (IPE270): 0.361×5.1=1.841kN

-Weight of secondary beams (IPE200): 0.224×5=1.12 kN

-Weight of joists (IPE200): 0.224×5×4=4.48 kN

Total dead loads**: G_t=148.661 kN**

Live loads: Q_t=1×21.463 **Q_t=21.463 kN**

Section of class 1: $N_{sd} \leq N_{crd}=N_{plrd}=\frac{A \times fy}{\gamma mo}$ so : $A \geq \frac{Nsd \times \gamma mo}{fy}$ and $\gamma_{mo}=1$

$N_{sd}=1.35G_t+1.5Q_t$=1.35×148.661+1.5×21.463=232.89 kN

$A \geq \frac{232.89 \times 10^3 \times 1}{235}$ =991.02mm^2 **A≥9.91cm^2** the profiled used is **HEA240**

- ***Current floor:***

The same loads of inaccessible roof the different in dead loads of the current floor: 5.76×21.463=123.63 kN

Total dead loads**: G_c=131.071 kN**

Table 3.5: live loads in each floor level by law of degression

Level	Overloads (kN/m^2)	∑overloads
Roof	Q_o=1	1
8th	Q_1=1.5	Q_o+ Q_1=2.5
7th	Q_2=1.5	Q_o+0.9(2×1.5) =3.7
6th	Q_3=1.5	Q_o+0.8(3×1.5) =4.6
5th	Q_4=1.5	Q_o+0.7(4×1.5) =5.2
4th	Q_5=1.5	Q_o+0.6(5×1.5) =5.5
3rd	Q_6=1.5	$Q_o+\frac{3+6}{2\times6}\times(6\times1.5)$=7.75
2nd	Q_7=1.5	$Q_o+\frac{3+7}{2\times7}\times(7\times1.5)$ =8.5
1st	Q_8=1.5	$Q_o+\frac{3+8}{2\times8}\times(8\times1.5)$ =9.25

The results are in the following table:

Table 3.6: results of vertical loads and choice profiled of center columns

Level	G(kN)	Q(kN)	N_{sd}(kN)	A(cm^2)	profile
Roof	148.661	21.463	232.89	9.91	HEA240
8th	279.732	53.66	458.13	19.49	HEA240
7th	410.803	79.41	673.7	28.67	HEA240
6th	541.874	98.73	879.62	37.43	HEA260
5th	672.945	111.61	1075.89	45.78	HEA260
4th	804.016	118.047	1262.49	53.72	HEA260
3rd	935.087	166.34	1511.88	64.34	HEA300
2nd	1066.158	182.44	1712.97	72.89	HEA300
1st	1197.23	198.53	1914.06	81.45	HEA300

- ***Edge columns:***

The most stressed columns take for the calculation and pre-dimensioning are **A-3** and **E-7** according to the following area:

$S=4.25\times2.55=10.837\ m^2$; $Gt=6.58\times10.837$, $G_c=5.76\times10.837$

Total dead loads and live loads of roof**: G_t=78.751 kN , Q_t=10.837 kN**

Total dead loads of current: **G_c=62.42 kN**

Table 3.7: Results of vertical loads and choice profiled of Edge columns

Level	G(kN)	Q(kN)	N_{sd}(kN)	A(cm^2)	profile
Roof	78.751	10.837	122.57	5.22	HEA180
8^{th}	141.171	27.09	231.22	9.83	HEA180
7^{th}	203.591	40.1	335	14.26	HEA180
6^{th}	266.011	49.85	433.89	18.46	HEA200
5^{th}	328.431	56.35	527.91	22.46	HEA200
4^{th}	390.851	59.6	617.05	26.26	HEA200
3^{rd}	453.271	83.99	737.9	31.4	HEA220
2^{nd}	515.691	92.11	834.35	35.5	HEA220
1^{st}	578.111	100.24	930.81	39.61	HEA220

- ***Corner columns:***

The most stressed columns take for the calculation and pre-dimensioning are **A-5** and **C-7** according to the following area: $S=2.5\times2.55=6.375\ m^{2;}$ $G_t=6.58\times6.375$, $G_c=5.76\times6.375$

Total dead loads and live loads of roof**: G_t=41.95 kN, Q_t=6.375 kN**

Total dead loads of current: **G_c=36.72 kN**

Table 3.8: Results of vertical loads and choice profiled of corner columns

Level	G(kN)	Q(kN)	N_{sd}(kN)	A(cm^2)	profile
Roof	41.95	6.375	66.195	2.82	HEA120
8^{th}	78.67	15.94	130.11	5.54	HEA120
7^{th}	115.39	23.59	191.16	8.13	HEA120
6^{th}	152.11	29.325	249.34	10.61	HEA120
5^{th}	188.83	33.15	304.65	12.96	HEA120
4^{th}	225.55	35.06	357.08	15.195	HEA120
3^{rd}	262.27	49.41	428.18	18.22	HEA160
2^{nd}	298.99	54.19	484.92	20.63	HEA160
1^{st}	335.71	58.97	541.66	23.05	HEA160

- ***Verification of buckling resistance:***

The condition following should be verified: $N_{sd}\leq N_{brd}$ with: $N_{brd}=\frac{\chi\ \beta_A\ A\ f_y}{\gamma_{M_o}}$

Table 3.9: Characteristics of profile HEA300 of center columns of ground level

profile	Weight G(Kg/m)	Area A(cm^2)	h(mm)	b(mm)	t_f(mm)	t_w(mm)	r(mm)	W_{ply}(cm^3)	W_{plz}(cm^3)	iy(cm)	iz(cm)
HEA300	88.3	112.5	290	300	14	8.5	27	1383	641.2	12.74	7.49

- ***Section classification:***

➢ Part subject to bending (web):

$c/t \leq 72\varepsilon$ such as: $\varepsilon=\sqrt{\frac{235}{fy}}$ and fy=235MPa so $\varepsilon=1$

c/t= (290-2×14)/8.5=30.82<72 so web of class 1

➢ Part subject to compression (flange):

$c/t \leq 9\varepsilon$ = (300-8.5)/2-27 = 118.75/14 = 8.48< 9 so flange of class 1

So, profile HEA300 is classified in class 1.

$\beta_{A=}1$ (class 1) , $\gamma_{Mo}=1.1$, $\lambda_1 = 93,9\,\varepsilon = 93,9\sqrt{\frac{235}{f_y}} = 93,9$

$\lambda=\frac{Lf}{i}$ such as : $L_f=\frac{1+0.145\times(\eta1+\eta2)-0.265\times(\eta1\times\eta2)}{2-0.364\times(\eta1+\eta2)-0.247\times(\eta1\times\eta2)}\times H$

H: height of ground level =3.4m

$\eta_1=\frac{Kc+Kc1}{Kc+Kc1+Kb11+Kb12}$; $\eta_2=\frac{Kc+Kc2}{Kc+Kc2+Kb21+Kb22}$

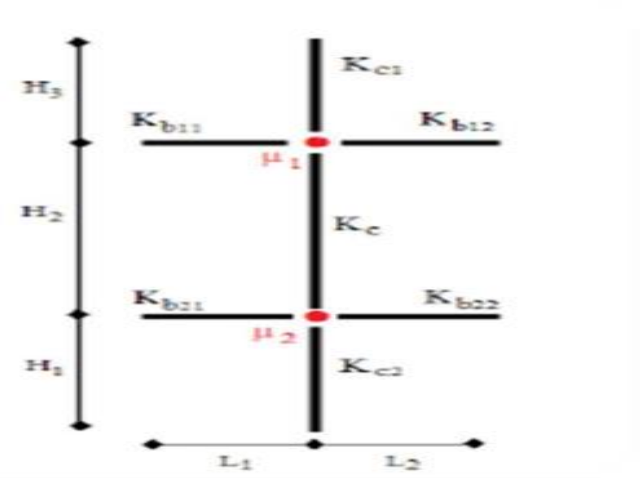


Figure 3.5: Coefficient of the determination of L_f

According to axis y-y:

K_c(HEA300) = $\frac{I}{H}=\frac{18260\times10^4}{3400}$ =53705.88mm^3; K_B(IPE270)= $\frac{I}{L}=\frac{5790\times10^4}{5100}$=11352.94mm^3

$K_{c2} = 0$; $K_{b21} = K_{b22} = 0$

$\eta_1 = \frac{53705.88+53705.88}{53705.88+53705.88+11352.94+11352.9}=0.825$

$\eta_2=0$ (Fixed support)

$L_f=\frac{1+0.145\times(0.825+0)}{2-0.364\times(0.825+0)}\times3.4=0.66\times3.4=2.244m=2244mm$

According to axis z-z:

$K_c(HEA300) = \frac{I}{H} =\frac{6310\times10^4}{3400} =18558.82mm^3$; $K_B= \frac{I}{L}=\frac{5790\times10^4}{5100}=11352.94mm^3$

$K_{c2} = 0$; $K_{b21} = K_{b22} = 0$

$\eta_1 = \frac{18558.82+18558.82}{18558.82+18558.82+11352.94+11352.94}=0.62$

$\eta_2=0$ (Fixed support)

$L_f=\frac{1+0.145\times(0.62+0)}{2-0.364\times(0.62+0)}\times3.4=0.61\times3.4=2.074m=2074mm$

- Maximum slenderness: $\lambda_y=\frac{Lfy}{iy}=\frac{2074}{127.4}=16.28$ and $\lambda_z=\frac{Lfz}{iz}=\frac{2074}{74.9}=27.69$

$\lambda_y=16.28< \lambda_z=27.69$ so buckling around y-y axis.

- ✓ Reduced slenderness: $\bar{\lambda} = \frac{\lambda}{\lambda_1}\sqrt{\beta_A} =\frac{27.69}{93.9}=0.29>0.2$ there is a risk of buckling
- ✓ Choice of curve: $\frac{h}{b}=\frac{290}{300}=0.96<1.2$ $t_f=14<100$ and axis y-y so curve b $\alpha=0.34$
- ✓ Reduction factor: $\emptyset=0.5[1+\alpha(\bar{\lambda}-0.2)+\bar{\lambda}^2]=0.55$ $\chi=\frac{1}{\emptyset+\sqrt{\emptyset^2-\bar{\lambda}^2}}=0.945<1$

$N_{brd}=\frac{0.945\times1\times11250\times235}{1.1}\times10^{-3}$, $\mathbf{N_{sd}=1914.06kN<N_{brd}=2278.431\ kN}$ **C.V**

The profile HEA300 resist in buckling.

All levels have the same height 3.4m the result of calculation of buckling resistance of all profiles are in the following table:

Table 3.10: Result and verification of all profiles of columns

Column	level	profile	χ	N_{brd}(kN)	N_{sd}(kN)	Verification
Corner	4 to roof	HEA120	0.69	378.6612	357.08	C.V
	1,2,3	HEA160	0.82	676.264	541.66	C.V
Edge	7,8, roof	HEA180	0.86	828.11	335	C.V
	4,5,6	HEA200	0.88	1011.44	617.05	C.V
	1,2,3	HEA220	0.90	1239.171	930.81	C.V
Center	7,8, roof	HEA240	0.91	1493.06	673.7	C.V
	4,5,6	HEA260	0.93	1724.96	1262.49	C.V
	1,2,3	HEA300	0.945	2278.431	1914.06	C.V

3.4 CONCLUSION:

- ✓ In this chapter, different sections of the resistant elements of the structure are determined according to the resistance and stability conditions.
- ✓ According to the results, the building does not carry important loads, therefore the profiles are not important, the most stressed columns (ground level) require a profile of HEA300, and the main beams is a IPE270.
- ✓ The results are used for the modeling of the building's structure using Robot Structural Analysis software.
- ✓ The profile adopted of the main beams is a IPE270, for the secondary beams is a IPE200, and for the joists is IPE200, and the profiles of all types of columns are in the following table:

Table 3.11: The profiles adopted of Center, Corner and Edge columns

level	Center columns	Corner columns	Edge columns
Roof	**HEA240**	**HEA180**	**HEA120**
8	**HEA240**	**HEA180**	**HEA120**
7	**HEA240**	**HEA180**	**HEA120**
6	**HEA260**	**HEA200**	**HEA120**
5	**HEA260**	**HEA200**	**HEA120**
4	**HEA260**	**HEA200**	**HEA120**
3	**HEA300**	**HEA220**	**HEA160**
2	**HEA300**	**HEA220**	**HEA160**
1	**HEA300**	**HEA220**	**HEA160**

CHAPTER 4

STUDY OF THE SECONDARY ELEMENTS

4.1 *INTRODUCTION:*

The main objective of this chapter is the calculation of the secondary elements of the building that are the balconies, the stairs and the parapet. The calculation of these elements is generally depends on the dead loads and live loads.

4.2 *STAIRS:*

Stairs are structures that allow access to different building levels, they are constructed of steel and they are composed of:

Landing: is a flat space that marks a floor after a series of walks, whose function is to allow rest during the climb.

Stolen: a straight (or curved) part of a stair between two landing successive.

Tread: they may be embedded between two strings or resting on one or two strings.

String: inclined element supporting tread and riser

Handrail: it's used to ensure safety.

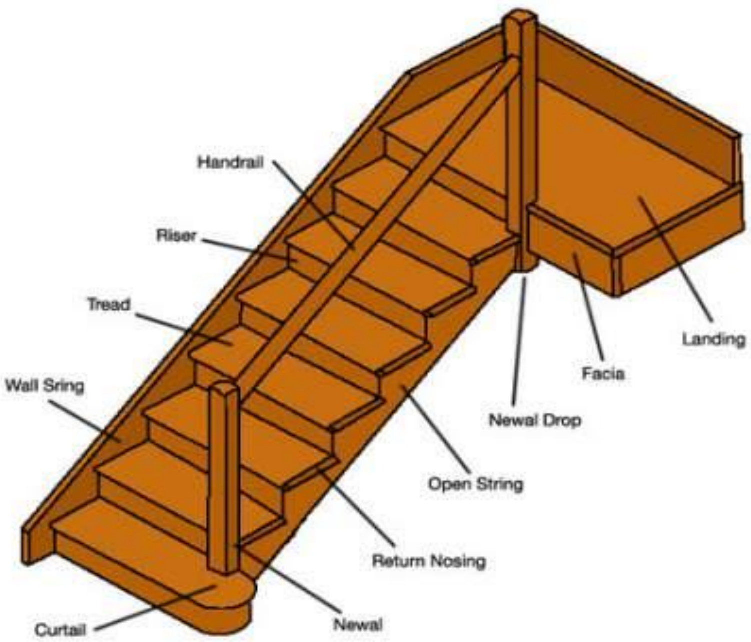
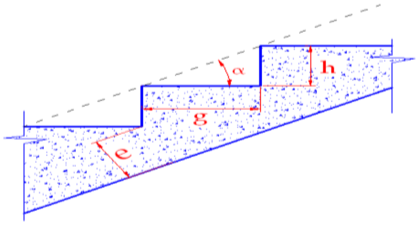


Figure 4.1: Stair composition.

The proportions of the successful stairs respect Blondel formula: $59cm \leq g+2h \leq 66cm$

4.2.1 Dimensional characteristics of the elements constituent of stair:

H: height of level……3.4m



h: height of tread (varying between 14cm and 20cm) h=17cm

g: width of tread (varying between 22cm and 30cm) g=30cm

The condition 59<g+2h=30+2×17=64cm<66 condition verified

Number of risers: n=$\frac{H/2}{h}$=$\frac{3.4/2}{0.17}$=10

Number of treads: m=n-1=10-1=9 treads per stolen

The length of the stride line: L= g(n-1) =30(10-1) =30×9=270cm=2.7m

The inclination of the bench: tangα=$\frac{n\times h}{L}$=$\frac{10\times17}{270}$=0.629 α=32.2°

The length of the bench: l=$\frac{n\times h}{sin\alpha}$=$\frac{10\times17}{sin32.2}$=319cm=3.19m

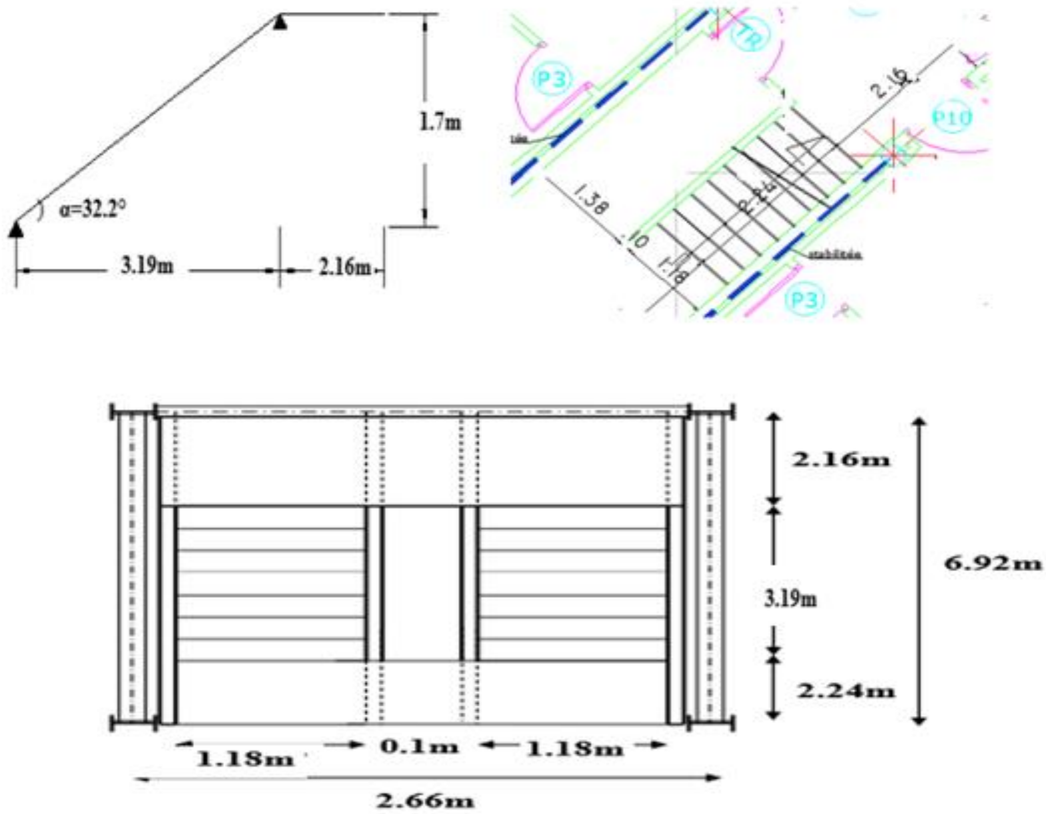


Figure 4.2: Dimension elements stair.

4.2.2 Sizing of load-bearing elements:

- ***Treads support:***

The length of treads 1.18m

The width of treads 0.3m

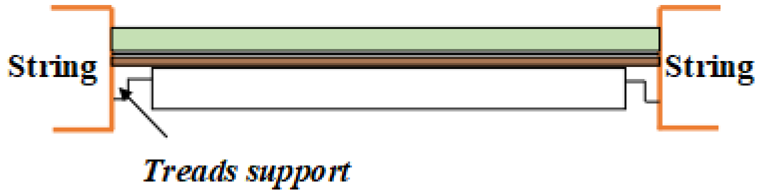


Figure 4.3 : Composants of stairs.

✓ ***Loads and overloads:***

Dead loads:

-Treads support: 0.08 kN/m

-Tile: 0.40 kN/m^2

-Laying bricks: 0.40 kN/m^2

-Sheet-metal: e_p =1 cm 0.785 kN/m^2

Total dead loads: G=0.08+(0.4+0.4+0.785) ×0.3=0.56kN/m

Live loads: Q=2.5×0.3=0.75 kN/m

Loads combination:

ULS : q'_u=1.35G+1.5Q=1.35×0.56+1.5×0.75=1.881 kN/m

SLS : q'_s=G+Q=0.56+0.75=1.31 kN/m

✓ ***Deflection condition :***

$f^{\max}=\frac{5\ qs.L^4}{384.E.Iy} \leq f_{\text{allow}} =\frac{L}{300}$, $I_y \geq \frac{5\ qs.L^3 \times 300}{384.E.Iy} = \frac{5 \times 1.31 \times 1180^3 \times 300}{384 \times 2.1 \times 10^5}$=4cm^4

choice of Leg angle it's L40×40×4 class 1

Table 4.1: Characteristics of leg angle L40×40×4

Leg angle	Weight G(Kg/m)	Area A(cm^2)	h=b(mm)	t(mm)	r_1(mm)	r_2(mm)	$W_{ply}=W_{plz}$(cm^3)	I_y(cm^4)
L40×40×4	2.42	3.68	40	4	6	3	1.55	4.47

ULS : $q_u=q'_u+1.35G_L=1.881+1.35\times0.0242=1.91$kN/m

SLS : $q_s= q'_s+G_L=1.31+0.0242=1.334$ kN/m

✓ ***Strength condition:***

Bending strength:

The following condition should be verified: $M_{sd}\leq M_{plyrd}$

$M_{sd}=\frac{qu.l^2}{8}=\frac{1.91\times1.18^2}{8}=0.333$ kN.m; $M_{plyrd}=\frac{Wply\times fy}{\gamma mo}=\frac{1.55\times10^3\times235}{1}\times10^{-6}=0.364$ kN.m

M_{sd}=0.333 kN.m < M_{plyrd}=0.364 kN.m ***condition verified***

Shear strength:

The following condition should be verified: $V_{sd}\leq V_{plyrd}$

$V_{sd}=\frac{q.l}{2}=\frac{1.91\times1.18}{2}=1.13$ kN; $V_{plyrd}=\frac{A\times fy}{\sqrt{3}\times\gamma Mo}=\frac{8.73\times10^2\times235}{\sqrt{3}\times1}\times10^{-3}=49.93$kN

V_{sd}=1.13 kN ≤ V_{plyrd}=49.93kN ***condition verified***

The deflection:

The following condition should be verified: $f^{max}\leq f_{allow}$

$f^{max}=\frac{5\,qs.L^4}{384.E.Iy}=\frac{5\times1.334\times(1180)^4}{384\times2.1\times10^5\times4.47\times10^4}=3.59$mm; $f_{allow}=\frac{L}{300}=\frac{1180}{300}=3.93$mm

f^{max} =3.59mm ≤ f_{allow}=3.93mm ***condition verified***

- ***String:***

✓ ***Loads and overloads:***

Stolen:

Dead loads:

-Leg angle: 0.0242 kN/m

- Handrail: 1 kN/m

-Laying bricks: 0.40 kN/m^2

-Tile: 0.40 kN/m^2

-Sheet-metal: e_p =1 cm 0.785 kN/m^2

Total dead loads: G=0.0242+1+(0.4+0.4+0.785) ×1.18=2.8945kN/m

Dead loads for 1 string: $G_{1string}$=G/2=2.8945/2=1.45 kN/m

Live loads: Q=2.5×1.18=2.95 kN/m

Live loads for 1 string: $Q_{1string}$=Q/2=2.95/2=1.48kN/m

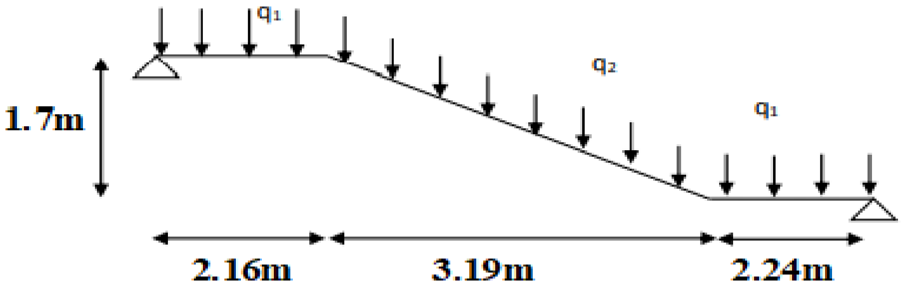


Figure 4.4: Loads on one string.

Loads combination:

ULS : q'_u=1.35G+1.5Q=1.35×1.45+1.5×1.48=4.18 kN/m

SLS : q'_s=G+Q=1.45+1.48=2.93 kN/m

✓ ***Deflection condition :***

$I_y \geq \frac{5\ qs.L^3 \times 300}{384.E.Iy} = \frac{5 \times 2.93 \times 3190^3 \times 300}{384 \times 2.1 \times 10^5}$=176.92cm^4

Limon of profile UPN100 class 1

Table 4.2: Characteristics of profile UPN

profile	Weight G(Kg/m)	Area A(cm^2)	h(mm)	b(mm)	t_f(mm)	t_w(mm)	W_{ply}(cm^3)	I_y(cm^4)
UPN100	10.6	13.5	100	50	8.5	6	49	206

ULS : q_u=4.32kN/m, **SLS :** q_s= 3.036 kN/m

✓ ***Strength condition:***

Bending strength:

M_{sd}=5.5 kN.m; M_{plyrd}=11.515 kN.m

M_{sd}=5.5 kN.m < M_{plyrd}=11. 515kN.m ***condition verified***

Shear strength:

V_{sd}=6.67 kN; V_{plyrd}=183.16kN

V_{sd}=6.67 kN ≤ V_{plyrd}=183.16kN ***condition verified***

The deflection:

The following condition should be verified: $f^{max} \leq f_{allow}$

f^{max}=9.46mm; f_{allow}=10.63mm

f^{max} =9.46mm ≤ f_{allow}=10.63mm ***condition verified***

The UPN100 is adopted for the string

- ***Landing:***

Dead loads:

-Reinforced concrete slab (12cm): 25×0.12 = 3.00 kN/m^2

-Profiled steel decking TN40: 0.12 kN/m^2

-Laying bricks: 0.40 kN/m^2

-Tile: 0.40 kN/m^2

Dead loads for one string: G= ((3+0.12+0.4+0.4) ×1.18)/2=2.31kN/m

Live loads for one string: Q=2.5×1.18/2=1.48kN/m

Equivalent loads:

Dead loads: $G_{eq}=\frac{2.31(2.16+2.24)+3.19\times4.32}{2.66}$=9 kN/m

Live loads: $Q_{eq}=\frac{1.48(2.16+2.24)+3.036\times3.19}{2.66}$=6.09 kN/m

4.3 *PARAPET:*

The parapet is not a structural element, it's calculated as an embedded cantilever at the roof level, it's stressed by compound bending which support the following loads:

Own weight as a normal vertical force.

A horizontal force of handrail Q=1kN/ml.

The calculation of parapet it's for a band of 1m width whose dimensions as the following:

Height: H=60cm

Width: b=100cm

Thickness: e=10cm

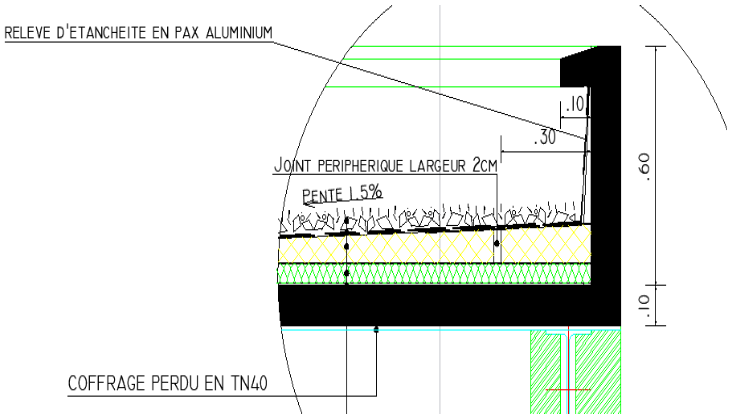


Figure 4.5: Dimensions of parapet.

4.3.1 Loads and overloads:

Dead loads: $G=S\times\gamma_{concrete}$

$S=[\frac{0.1+0.03}{2}\times 0.1\times 0.07+(0.6\times0.1)]=0.0685\ m^2$

G=0.0685×25=1.712 kN/m

Live loads: Q=1 kN/m

Solicitation:

Normal force N :

ULS : $N_u = 1{,}35G = 1{,}35\times(1.712) = 2.311$ kN

SLS: N_s= G = 1,712 kN

Embedding moment M:

ULS : M_u = 1.5.Q.H = 1.5×1×0.6= 0.9 kN.m

SLS: M_s=Q.H = 1×0,6= 0,6 kN.m

Transverse shear force :

ULS : V_u = 1.5× Q=1.5 ×1=1.5 kN

SLS: V_s= Q =1 kN

4.3.2 Calculation of the reinforcement section:

Data: f_{c28} = 25 MPa; f_{t28} = 2.1 MPa; h=10cm; H=60cm; b =100 cm; C=2 cm

d=8 cm h=10cm, σs= fe/γs=400/1.15=348MPa; $\sigma_{bc}=0.6f_{c28}=0.6\times25=15$MPa

Calculation of eccentricity:

$e_o=\frac{Mu}{Nu}=\frac{0.9}{2.311}$=0.389m e_o=389cm

e_o=389 cm>$\frac{h}{2}-c=\frac{10}{2}-2$=3 cm the pressure center is outside the section compressed

The calculation will be in simple bending then in compound bending

$M_1=M_u+N_u(d-\frac{h}{2})$=0.9692kN.m

$\mu=\frac{M1}{bd^2\sigma bc}$=0.01 < μ_R=0.392 A’s=0 Compressed reinforcements not necessary.

$\alpha=1.25(1-\sqrt{1-2\mu})$=0.0126

Z=d(1-0.4α) =79.6 cm

$A_{s1}=\frac{M1}{Z.\sigma s}$=0.35 cm$^{2;}$ A’$_s$=0; $A_{s2}= A_{s1}-\frac{Nu}{b\times\sigma s}$=0.283 cm^2

4.3.3 Condition of non-fragility:

$A^{min}_s \geq$Max (A_{s2} ;0.23b.d$\frac{ft28}{fe}$)=Max (0.283 cm^2;0.966 cm^2) =0.966cm^2

$A^{min}_s \geq 0.966$cm^2 so are: 4T6 A=1.13cm^2

Spacing:

From B.A.E.L 99 $e = \min(3h\ ;\ 33cm) = 30$ cm ; $e = 25$ cm.

Distribution bars:

$A_r \geq \frac{A}{4}$= 1,13/4= 0.282 cm^2 so are: 4T6 A_r =1,13cm²

Spacing:

$e \leq \min(4.h\ ;\ 45\ cm) = 40$ cm ; $e = 20$ cm

4.3.4 Verification of shear force:

From BAEL99 the condition following should be verified: $\tau_u \leq \overline{\tau}_u$

Such as: $\tau_u = \frac{Vu}{b.d} = \frac{1.5\times10^3}{1000\times80}$=0.018 MPa

$\overline{\tau}_u = \min\{\frac{fc28}{10}; 3MPa\}$=min {2.5MPa; 3MPa} =2.5MPa

τ_u =0.018 MPa < $\overline{\tau}_u$=2.5 MPa *condition verified*

Transversal bars:

For the thin elements if the condition following is verified the transversal bars are not necessary:

$\tau_u < 0.05 f_{c28}$ 0.018 MPa < 1.25MPa ***condition verified***

4.3.5 Verification at SLS:

Eccentricity :

$e_a = \frac{Mser}{Nser} + (d - \frac{h}{2})$= 38.04cm > 8cm

The pressure center is located outside the calculated section

$C = d - e_a$=8-38,04= -30,04cm

$y_{ser} + y_c = C$

y_c: distance between neutral axis and center of pressure

C: distance between center of pressure and the most compressed fiber

The following expression must be determined:

$y_c^3 + Py_c + q = 0$

$p = -3C^2 - (c-d)^2 \times \frac{6\times\eta\times A's}{b} + (c-d) \times \frac{6\times\eta\times A's}{b}$ $\eta=15$

$p = -2668.52\ cm^2$

$q = -2C^3 - (c-d)^2 \times \frac{6\times A's}{b} - (c-d)^2 \times \frac{6\times\eta\times A's}{b}$

$q = 52744.64$

$y_c^3 - 2668.52\, y_c + 52744.64 = 0$

Calculation of Δ:

$\Delta = q^2 + 4(\frac{p^3}{27}) = -33192318.06 < 0$

$\Phi = \text{Arc cos}\ (\frac{3q}{2p}\sqrt{\frac{-3}{p}}) = 173.76°$

$A = 2\sqrt{\frac{-p}{3}} = 59.65cm$

There are three solutions:

$y_1 = a\times\cos(\frac{\varphi}{3}) = 31.68cm$; $y_2 = a\times\cos(\frac{\varphi}{3} + \frac{2\times\varphi}{3}) = -59.61cm$; $y_3 = a\times\cos(\frac{\varphi}{3} + \frac{4\times\varphi}{3}) = 27.93cm$

$y_{ser1} = y_1 + C = 1.64cm$

$y_{ser3} = y_3 + C = -2.11cm$

y_{ser} must be >0 so: $y_{ser} = y_{ser1} = 1.64cm$ and $y_1 = 31.68cm$

4.3.6 Determination of stress:

Condition following should be verified: $\sigma_s \leq \overline{\sigma}_s$

Moment of inertia:

From BAEL99:

$I = \frac{b\times Yser^3}{3} + 15\times[A_s(d-y_{ser})^2 + A_s'(y_{ser}-d)^2]A_s' = 0$

$I = \frac{b\times Yser^3}{3} + 15\times A_s(d-y_{ser})^2 = \frac{100\times 1.64^3}{3} + 15\times 1.13(8-1.64)^2 = 832.65\ cm^4$

Coefficient K of stress:

K=$\frac{Nser}{I}$ ×y_c=$\frac{1.712\times10^3}{832.65}$ ×31.68×10^{-3} =0.065 N/mm^3

Limit state of concrete compression:

Damaging cracking:

σ_s= 15K(d-y_{ser}) =15×0.065(80-16.4) =62.01 MPa

$\overline{\sigma}_s$=min {$\frac{2}{3}$$f_e$; max (0.5 $f_{e;}$ 110$\sqrt{\eta \times ft28}$)} such as: η=1.6

$\overline{\sigma}_s$=min {$\frac{2}{3}$400; max (0.5×400; 110$\sqrt{1.6 \times 2.1}$}= min {266.66 MPa; max (200 MPa; 201.63 MPa)}

$\overline{\sigma}_s$=201.63 MPa

σ_s =62.01 MPa < $\overline{\sigma}_s$=201.63 MPa ***Condition verified***

4.3.7 Verification at earthquake:

Action of horizontal forces:

The condition should be verified: F_p≤1.5Q such as:

From RPA2003***:*** F_p=4.A×C_p ×W_p

A=0.15 group 2 , zone IIa from the table4.1

C_p=0.8 from table 6.1

W_p=1.712 kN/m weight of element for a band of 1m

F_p=4×0.15×0.8 ×1.712=0.82 kN/m

F_p=0.82 kN/m<1.5×1=1.5kN/m ***condition verified***

The seismic calculation is not necessary, parapet resist at this force.

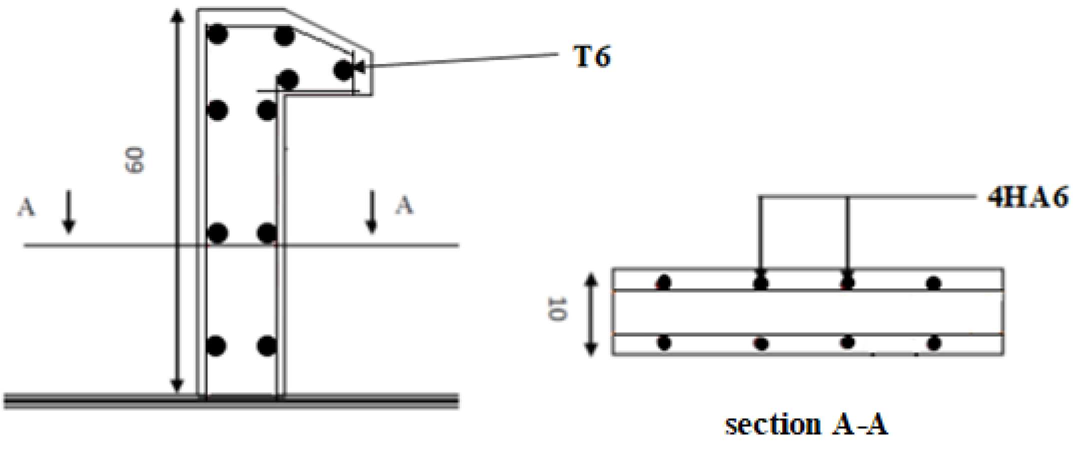


Figure 4.6: Reinforcement of parapet.

4.4 *BALCONIES:*

The balcony is a platform outside the building. It is stressed by simple bending, supports the dead loads and live loads.

Handrail had: height: 1m and thickness: 0.1m

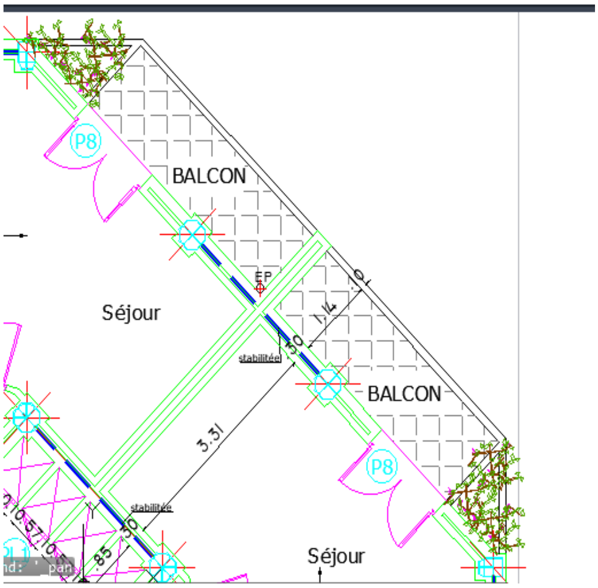


Figure 4.7: Dimensions of balcony.

4.4.1 Loads and overloads:

Dead loads:

-concrete slab: 3.00 kN/m^2

-Tile: 0.40 kN/m^2

-Laying bricks: 0.40 kN/m^2

-Inner wall: 1.00 kN/m^2

-Plaster ceiling (3cm): 0.30 kN/m^2

-Profiled steel decking TN40: 0.12 kN/m^2

Total dead loads: G= (3+0.4+0.4+1+0.3+0.12) ×6.7/2=17.49kN/m

Live loads: Q=3.5×6.7/2=11.73 kN/m

-Handrail: 1.3×0.1×1=0.13 kN

Loads combination:

ULS *:* q_u=1.35G+1.5Q=1.35×17.49+1.5×11.73=41.21 kN/m ;

P_u=1.35×$G_{handrail}$=1.35×0.13=0.18 kN

SLS : q_s=G+Q=17.49+11.73=29.22 kN/m ; P_s=$G_{handrail}$=0.13 kN

4.4.2 Deflection condition :

$f^{max}=\frac{5\ qs.L^4}{384.E.Iy}+\frac{19.Rs.L^3}{384.E.Iy}=\frac{L}{250}$, $I_y \geq \frac{(5\ qs.L^3+19\ Ps.L^2)\times 250}{384.E.Iy}$

$I_y \geq \frac{(5\times 29.22\times 1220^3+19\times 0.13\times 10^3\times 1220^2)\times 250}{384\times 2.1\times 10^5}$=83.39 cm^4

IPE180 class 1

Table 4.3: Characteristics of profile IPE180

profile	Weight G(Kg/m)	Area A(cm^2)	h(mm)	b(mm)	t_f(mm)	t_w(mm)	W_{ply}(cm^3)	W_{plz}(cm^3)	I_y(cm^4)	I_z(cm^4)
IPE180	18.8	23.9	180	91	8	5.3	166.4	34.6	1317	100.9

4.4.3 Verification of Bending strength:

The condition following should be verified: $M_{sd} \leq M_{plrd}=\frac{Wply\times fy}{\gamma Mo}$

$M_{sd}=\frac{qu\times L^2}{2}+Pu.L=\frac{41.21\times 1.22^2}{2}+0.18\times 1.22$=30.89 kN.m;

$M_{plrd}=\frac{166.4\times235}{1}\times10^{-3}=39.104$ kN.m

M_{sd} =30.89 kN.m< M_{plrd}=39.104 kN.m ***the bending strength is verified***

4.4.4 Shear strength:

The condition following should be verified: $V_{sd}\leq V_{plrd}=\frac{Av\times fy}{\sqrt{3}\times\gamma Mo}$ such as:

$V_{sd}=q_u\times L+P_u=41.21\times1.22+0.18=50.46$ kN

$A_v=2390-2\times91\times8+(5.3+2\times9)\times8=747.6$ mm^2

$V_{plrd}=\frac{747.6\times235}{\sqrt{3}\times1}\times10^{-3}=101.43$kN

V_{sd}=50.46 kN<V_{plrd}=101.43 kN ***Condition verified***

Shear strength is verified

4.4.5 Verification of interaction between moment and shear force:

V_{sd}=50.46kN< V_{plrd}=0.5×101.43=50.72 kN **No interaction**

4.4.6 The deflection:

The condition following should be verified: $f^{\max}=\frac{5\ qs.L^4}{384.E.Iy}+\frac{19.Ps.L^3}{384.E.Iy}\leq f_{allow}$

Such as: $f_{allow}=\frac{L}{250}=\frac{1220}{250}=4.88$mm,

$f^{\max}=\frac{5\times29.22\times1220^4}{384\times2.1\times10^5\times1317\times10^4}+\frac{19\times0.13\times10^3\times1220^3}{384\times2.1\times10^5\times1317\times10^4}=3.24$mm

$f^{\max}$=3.24mm < f_{allow}=4.88mm

4.5 *CONCLUSION:*

- ✓ Stair components are made of steel.
- ✓ The balcony is also made of steel.
- ✓ All strength conditions of balconies and stairs are checked.
- ✓ The parapet is an element that constitutes of reinforced concrete.
- ✓ Parapets are used as guard rails, to reduce wind loads on the roof, and to prevent the spread of fires.

CHAPTER 5

DYNAMIC ANALYSIS

5.1 INTRODUCTION:

Earthquakes have always presented one of the most serious disasters for humanity. Earthquakes are a brutal release of the potential energy accumulated in rocks due to relative movements of different parts of the earth's crust. Earthquakes are caused mostly by ruptures of geological faults. Algeria is a country where seismic activities are very important especially in the northern part.

The purpose of this chapter is to define a structure model that respect the conditions and safety criteria imposed by Algerian Seismic Regulation version 99/2003.The structural modelling was carried out using the software Robot Structural Analysis which is an automatic structural calculation software.

According to the Algerian Seismic Regulation version 99/2003, the design and analysis of a structure can be done using one of the following three methods of analysis:

- ✓ Equivalent static method.
- ✓ Dynamic spectral analysis method.
- ✓ Accelerograms dynamic analysis method.

5.2 CLASSIFICATION CRITERIA BY RPA99/2003:

5.2.1 Classification of seismic areas:

According to the map and the table from RPA99/2003, **Setif** is located in an area of medium seismicity Zone ***IIa***.

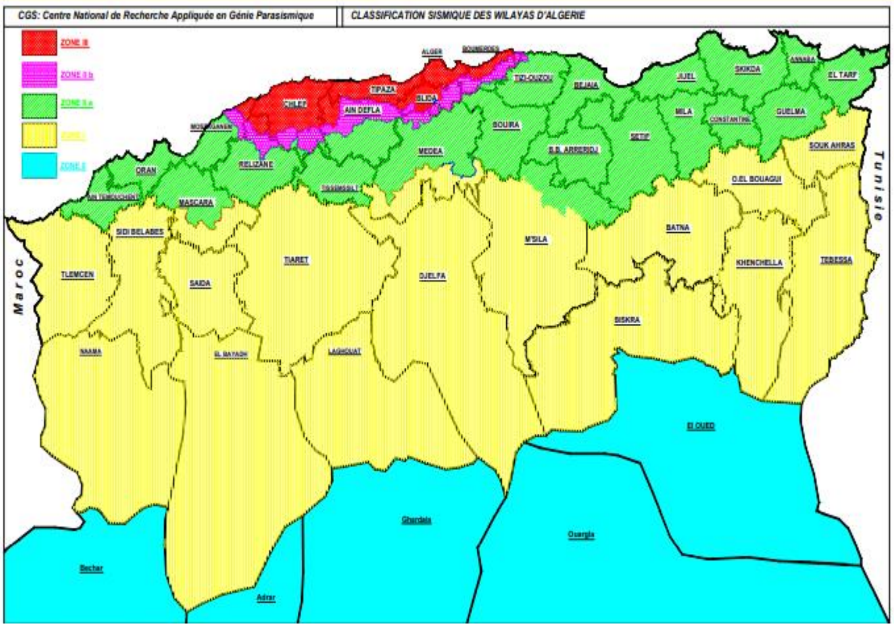


Figure 5.1*:* National map of seismic areas.

5.2.2 Classification of the building's use:

The classification of the building's use is based on the criteria of the importance of the building in terms of safety, economic and social. The building's use is residential; it has 31.2 m of height, which is less than 48m so it's considered to be a common or medium importance "***Group 2***".

5.2.3 Classification of the site:

According to the geotechnical report for this project, the soil of category is ***S3*** (loose soil).

5.3 CHOICE OF METHOD OF CALCULATION:

To be able to use the equivalent static method it's necessary that:

The conditions of regularity in plan and elevation must be checked with the building's height is less than 65m in the seismic zone ***IIa*** , in this case the equivalent static method is not applicable (according to RPA99/03; zone IIa , Group 2 , **height over than 23m and more than 7 levels** condition not checked) so the seismic analysis will be done using the spectral dynamic method.

5.4 SEISMIC FORCE V:

The total seismic force applied to the base of the structure, shall be calculated successively in two orthogonal directions according to the formula: $V=\frac{A.D.Q}{R}\times W$

A: Zone acceleration coefficient (table 4.1 of RPA2003).

D: Medium amplification factor determined from 4.2 of RPA2003.

Q: Quality factor (table 4.4 of RPA2003).

W: Total weight of structure determined by the formula of 4.5 from RPA2003.

R: Ratio behavior of the structure (table 4.3 of RPA2003).

5.5 THE SPECTRAL MODAL DYNAMIC METHOD:

5.5.1 *Principle:*

By this method, it's sought for each mode of vibration the maximum of effects generated in the structure by the seismic forces represented by a computational response spectrum. These effects are combined to obtain the structure response.

5.5.2 Calculation response spectrum:

Seismic forces are represented using the following computing spectrum:

$$\frac{S_a}{g} = \begin{cases} 1.25A\left(1+\frac{T}{T_1}\left(2.5\eta\frac{Q}{R}-1\right)\right) & 0 \leq T \leq T_1 \\ 2.5\eta(1.25A)\left(\frac{Q}{R}\right) & T_1 \leq T \leq T_2 \\ 2.5\eta(1.25A)\left(\frac{Q}{R}\right)\left(\frac{T_2}{T}\right)^{2/3} & T_2 \leq T \leq 3.0s \\ 2.5\eta(1.25A)\left(\frac{T_2}{3}\right)^{2/3}\left(\frac{3}{T}\right)^{5/3}\left(\frac{Q}{R}\right) & T > 3.0s \end{cases}$$

Where:

η: Damping correction factor (when the damping is different than 5%).

ξ: Critical damping percentage (table 4.2 of RPA2003).

T_1, T_2: Characteristic period associated with the site category (table 4.7 of RPA2003).

- ***Zone acceleration coefficient A:***

According to the table 4.1 of RPA2003 the seismic area is zone IIa and group 2 so ***A = 0.15***

- ***Critical damping percentage ξ:***

According to the table 4.2: steel frames, heavy filling, $\boldsymbol{\xi = 5\%}$

- ***Damping correction factor η:***

$\eta=\sqrt{\frac{7}{2+\xi}} \geq 0.7$ such as: ξ=5% so $\boldsymbol{\eta = 1}$ >0.7

- ***Characteristic period associated with the site category T_1, T_2:***

According to the table 4.7 of RPA2003 the site of the building is ***S3*** (loose soil) so:

T_1 = 0.15 sec and **T_2 = 0.50 sec**

- ***Ratio behavior R:***

According to the table 4.7 of RPA2003: Braced frames by triangulated blades V, so ***R = 3***

- ***Quality factor Q:***

According to the RPA2003, the factor Q is determined by the following formula:

$$Q = 1 + \sum_{1}^{5} P_q$$

P_q: depend of the conditions of the table 4.4 of RPA

- ✓ ***Minimum conditions in bracing system lines:***

Frames system: each line of frame must have minimum of 3 spans this condition not verified because there is a line of frames that have 2 spans.

According to the table 4.4 of RPA: q_{p1} = 0.05 for both axis x and y.

- ✓ ***Redundancy in plan:***

Each level must have minimum of 4 lines frames, each level of this building has 6 lines frames so the condition is verified for the axis x and y.

The rapport of the maximum spacing between the files and the minimum spacing must be less than 1.5, this condition is not verified because the minimum spacing is 2.07m and the maximum is 5.1 so 5.1/2.07=2.46 >1.5 for the both of axis x and y.

According to the table 4.4 of RPA2003: q_{p2}=0.05

- ✓ ***Regularity in plan:***

The building is not symmetrical in the axis x and axis y condition not verified

3.5/12.37=0.28 > 0.25 condition not verified

According to the table 4.4 of RPA2003: q_{p3}=0.05.

- ***Regularity in elevation:***

The levels of the building have the same dimensioning so the condition is verified in the both axes.

According to the table 4.4 of RPA2003: q_{p4}=0

- ***Quality control of materials:***

Is verified according to the table 4.4 of RPA2003: q_{p5}=0

- ***Execution quality control:***

Is not verified according to the table 4.4 of RPA2003: q_{p6}=0.10

So $\mathbf{Q_x = Q_y}$ =1+0.05+0.05+0.05+0.1 = ***1.25***

- ***Period:***

$T= C_T.h_N^{3/4}$; $T=0.09\times h_N/\sqrt{D}$

C_T=0.05 "*auto-stable"* steel frame with masonry filling.

h_N = 30.6m: height of building.

$D_X = D_Y$ =**19.56 m** the length of building along the axis x and y.

$T = 0.05\times 30.6^{3/4} = 0.65$ sec ; $T = 0.09\times 30.6/\sqrt{19.56}$=0.62 sec, So T_{emp} = **0.62 sec**

$1.3T_{emp} = 1.3\times 0.62$ = **0.81 sec**

- ***Medium amplification factor D:***

T_2=0.50s < T=0.65<3.0s So : $D=2.5\eta\ (T_2/T)^{2/3} = 2.5\times 1(0.50/0.65)^{2/3}$ D **= 2.1**

5.5.3 Number of modes considered:

For structures represented by plane models in two orthogonal directions, the number of vibration modes in each of the two directions the excitation must be such as:

The sum of effective modal masses for the modes selected is equal to at least 90% of the total mass of the structure.

Where all modes with an effective modal mass greater than 5% of the total structural mass are used for determining the total structural response.

The minimum of mode that be obtained is three in each direction considered.

5.6 THE INITIAL ROBOT MODEL:

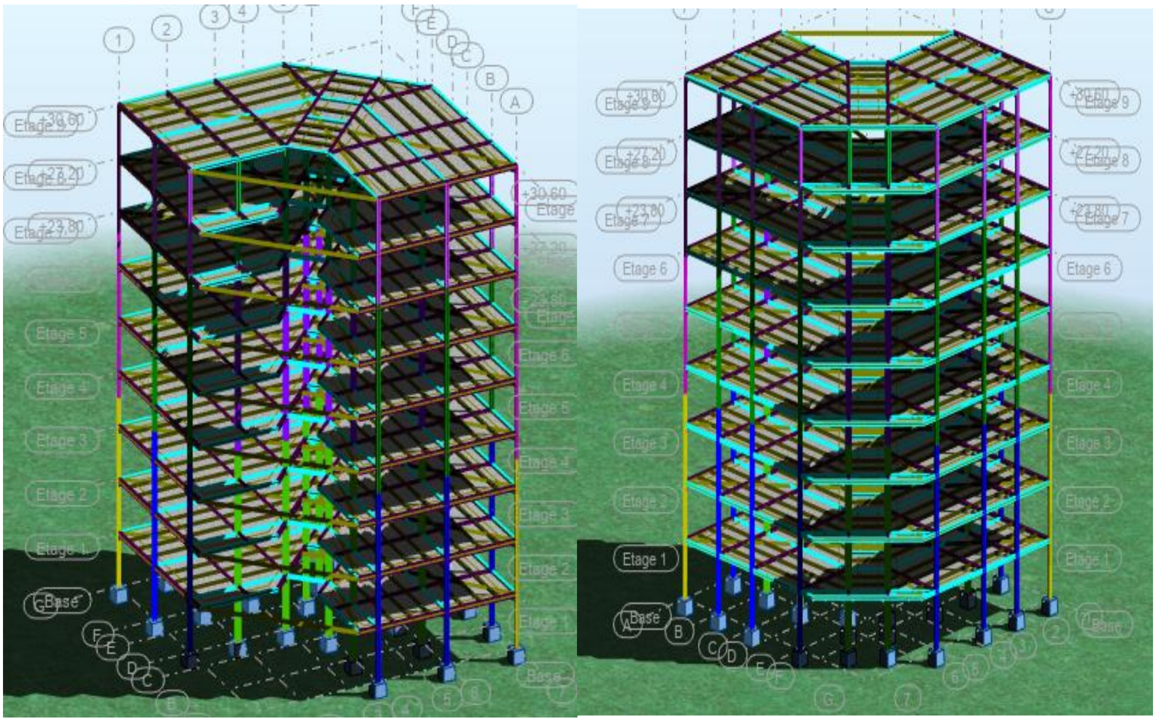


Figure 5.2: Initial model of the building by robot software principal façade.

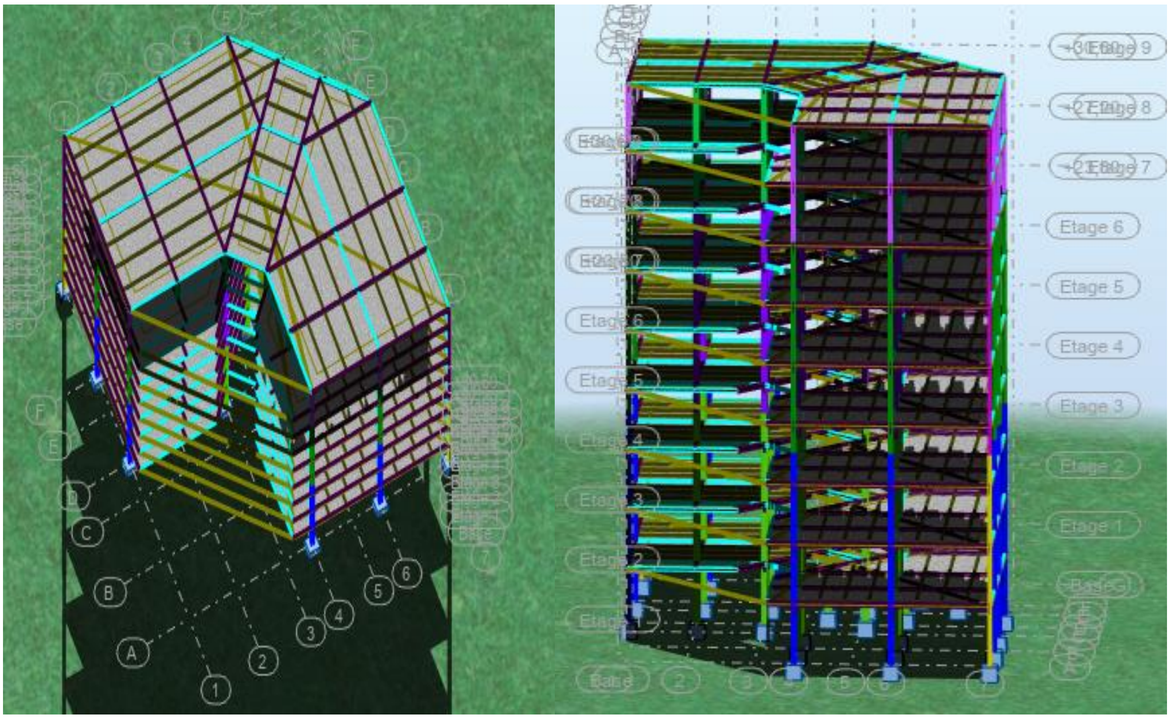


Figure 5.3: Initial model in side and high view.

5.6.1 Modal analysis:

The results of modal analysis are in the following table:

Table 5.1: The dynamic results of the initial model

Cas/Mode	Période [sec]	Masses Cumulées UX [%]	Masses Cumulées UY [%]	Masse Modale UX [%]	Masse Modale UY [%]	Tot.mas.UX [kg]	Tot.mas.UY [kg]
4/ 1	2,24	0,01	78,68	0,01	78,68	1699938,33	1699938,33
4/ 2	1,77	0,03	78,69	0,02	0,01	1699938,33	1699938,33
4/ 3	1,36	77,75	78,71	77,72	0,02	1699938,33	1699938,33
4/ 4	0,82	77,75	91,61	0,01	12,90	1699938,33	1699938,33
4/ 5	0,65	77,76	91,61	0,00	0,00	1699938,33	1699938,33
4/ 6	0,51	77,76	95,72	0,00	4,11	1699938,33	1699938,33
4/ 7	0,49	91,41	95,72	13,64	0,00	1699938,33	1699938,33
4/ 8	0,41	91,41	95,72	0,00	0,00	1699938,33	1699938,33
4/ 9	0,37	91,41	97,38	0,00	1,66	1699938,33	1699938,33
5/ 1	2,24	0,01	78,68	0,01	78,68	1699938,33	1699938,33
5/ 2	1,77	0,03	78,69	0,02	0,01	1699938,33	1699938,33
5/ 3	1,36	77,75	78,71	77,72	0,02	1699938,33	1699938,33
5/ 4	0,82	77,75	91,61	0,01	12,90	1699938,33	1699938,33
5/ 5	0,65	77,76	91,61	0,00	0,00	1699938,33	1699938,33
5/ 6	0,51	77,76	95,72	0,00	4,11	1699938,33	1699938,33
5/ 7	0,49	91,41	95,72	13,64	0,00	1699938,33	1699938,33
5/ 8	0,41	91,41	95,72	0,00	0,00	1699938,33	1699938,33
5/ 9	0,37	91,41	97,38	0,00	1,66	1699938,33	1699938,33
6/ 1	2,24	0,01	78,68	0,01	78,68	1699938,33	1699938,33
6/ 2	1,77	0,03	78,69	0,02	0,01	1699938,33	1699938,33
6/ 3	1,36	77,75	78,71	77,72	0,02	1699938,33	1699938,33
6/ 4	0,82	77,75	91,61	0,01	12,90	1699938,33	1699938,33
6/ 5	0,65	77,76	91,61	0,00	0,00	1699938,33	1699938,33
6/ 6	0,51	77,76	95,72	0,00	4,11	1699938,33	1699938,33
6/ 7	0,49	91,41	95,72	13,64	0,00	1699938,33	1699938,33
6/ 8	0,41	91,41	95,72	0,00	0,00	1699938,33	1699938,33
6/ 9	0,37	91,41	97,38	0,00	1,66	1699938,33	1699938,33

The following figures 5.4, 5.5 and 5.6 represent the three first modes of the initial model:

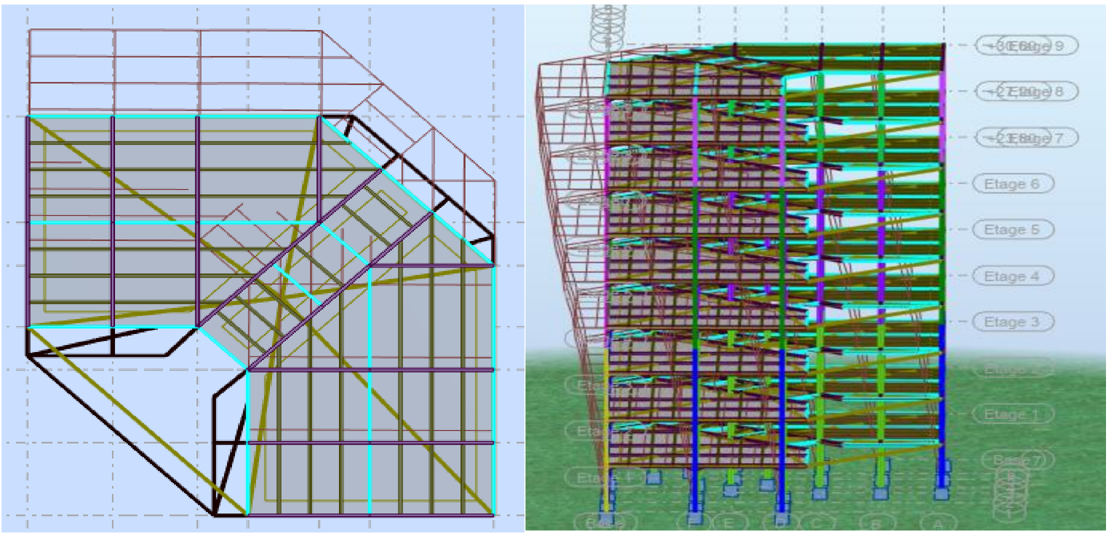


Figure 5.4: Mode 1: Translation along the y-axis.

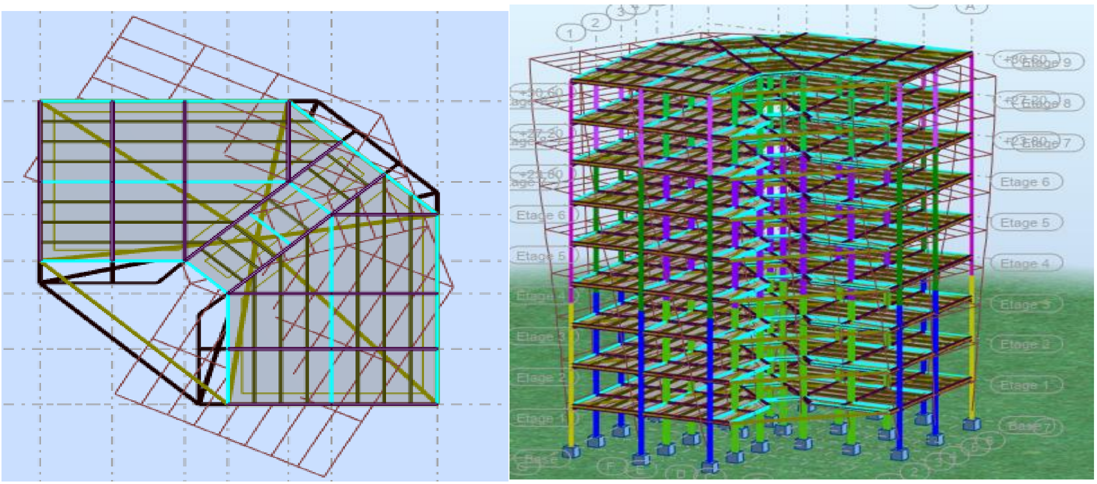


Figure 5.5: Mode 2: Rotation around the z-axis

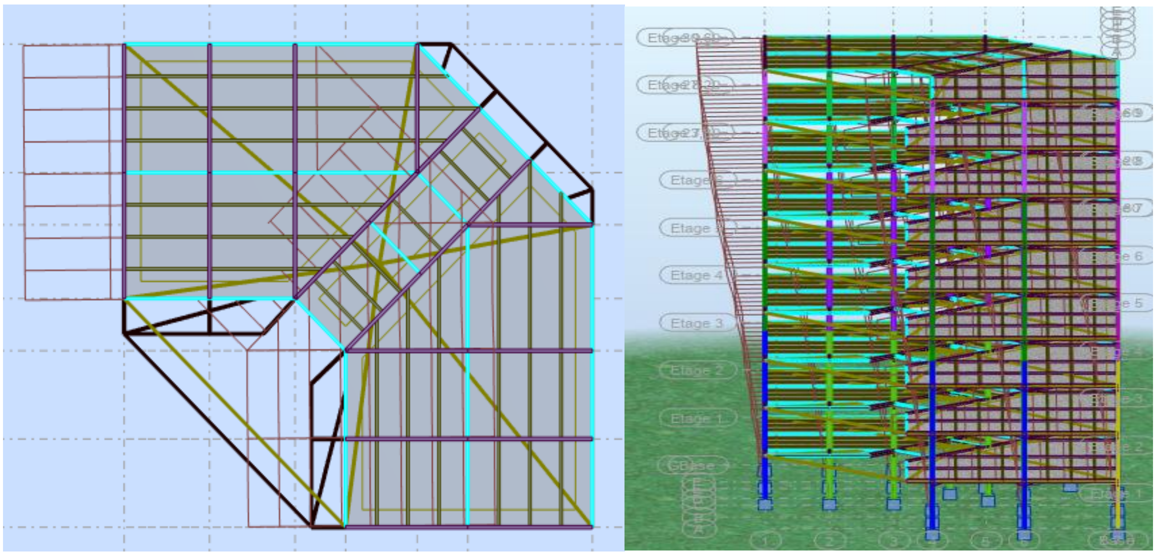


Figure 5.6: Mode 3: Translation along the x-axis

According to the table 5.1 and the figures 5.4, 5.5 and 5.6:

- Period T=2.24 sec > $1.3T_{emp}$=0.81 sec → ***condition not verified***

  T must be between T_{emp} and $1.3T_{emp}$
- The first mode is a translation along the y-axis.
- The second mode is a rotation around the z-axis.
- The third mode is a translation along the x-axis
- Mass participation exceeds 90 % starting from the 7th mode along the x-axis and from the 4th mode along the y-axis, so the condition is checked.
- The initial model is not verified, and it must be reinforced using an appropriate bracing system and check all the RPA99/03 conditions.

5.7 THE FINAL ROBOT MODEL:

The final model is reinforced by the bracing system of **V**, the profiles of columns are changing as following:

- Bracing system 2UPN200 in the both directions.

The following table represent the different adopted profiles of columns:

Table 5.2: The profile of columns of finale model.

level	Profile of columns
Roof	HEA360
8	HEA360
7	HEA400
6	HEA400
5	HEA400
4	HEA450
3	HEA450
2	HEA450
1	HEA450

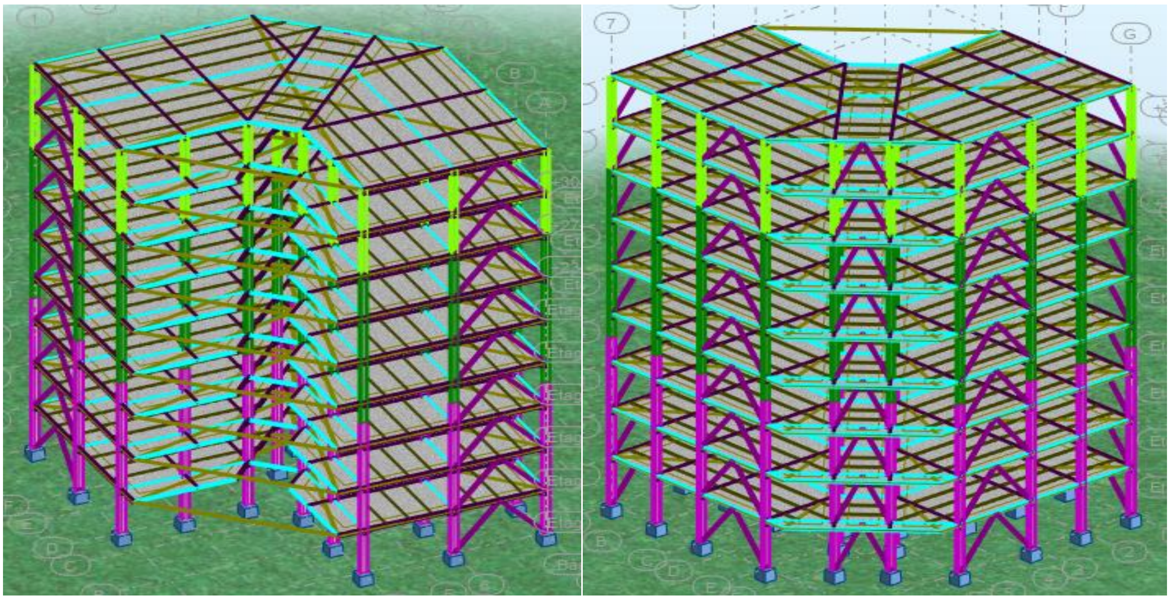


Figure 5.7: Final model of the structure, the building is reinforced by the bracing system.

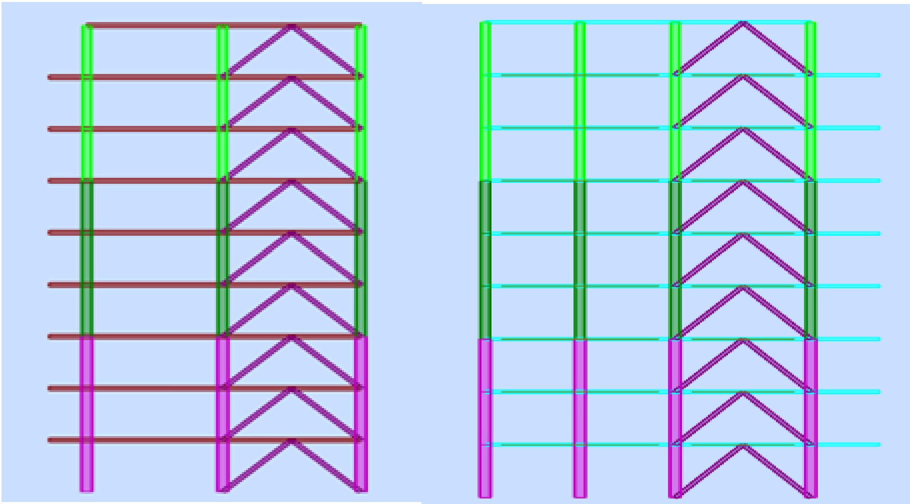


Figure 5.8: Disposition of the bracing system 2UPN200 V in x direction.

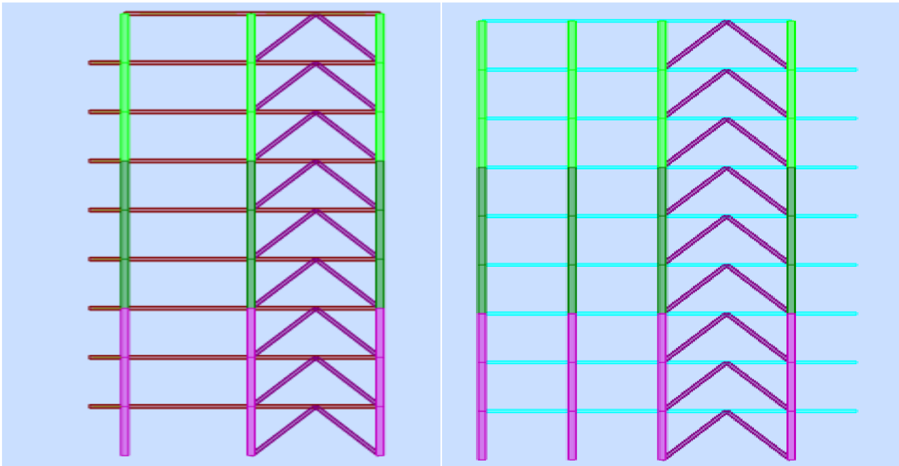
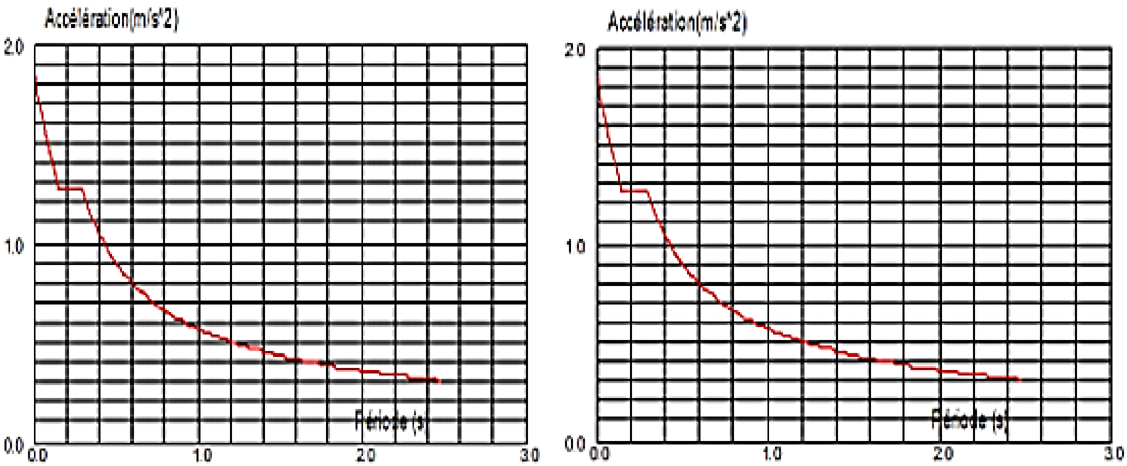


Figure 5.9: Disposition of the bracing system 2UPN200 ***V*** in y direction.



According to x

According to y

Figure 5.10: Response spectrum along both axis x and y.

5.7.1 Modal analysis:

The results of modal analysis are in the following table:

Table 5.3: The dynamic analysis results of the final model.

Cas/Mode	Période [sec]	Masses Cumulées UX [%]	Masses Cumulées UY [%]	Masse Modale UX [%]	Masse Modale UY [%]	Tot.mas.UX [kg]	Tot.mas.UY [kg]	Masse Modale RZ [%]	Masses Cumulées RZ [%]
4/ 1	0,69	2,03	80,16	2,03	80,16	1773709,56	1773709,56	0,00	0,00
4/ 2	0,50	78,48	82,67	76,45	2,51	1773709,56	1773709,56	0,00	0,00
4/ 3	0,36	78,50	82,69	0,02	0,02	1773709,56	1773709,56	0,01	0,01
4/ 4	0,23	79,15	93,42	0,65	10,73	1773709,56	1773709,56	0,00	0,01
4/ 5	0,17	92,45	93,67	13,30	0,25	1773709,56	1773709,56	0,00	0,01
4/ 6	0,14	92,59	96,69	0,14	3,02	1773709,56	1773709,56	0,00	0,01
4/ 7	0,12	92,62	96,70	0,03	0,01	1773709,56	1773709,56	0,00	0,01
4/ 8	0,12	92,64	96,70	0,03	0,00	1773709,56	1773709,56	0,00	0,01
4/ 9	0,12	92,64	96,70	0,00	0,00	1773709,56	1773709,56	0,00	0,01

- Period of 1st mode: $T = 0.69$ sec $\leq 1.3T_{emp} = 0.81$ sec $\rightarrow$ ***condition verified***
- Mass participation exceeds 90 % starting from the 5th mode along x-axis x and from the 4th mode along y-axis $\rightarrow$ ***condition checked.***

The following figures 5.11, 5.12 and 5.13 represent the three first modes of final model:

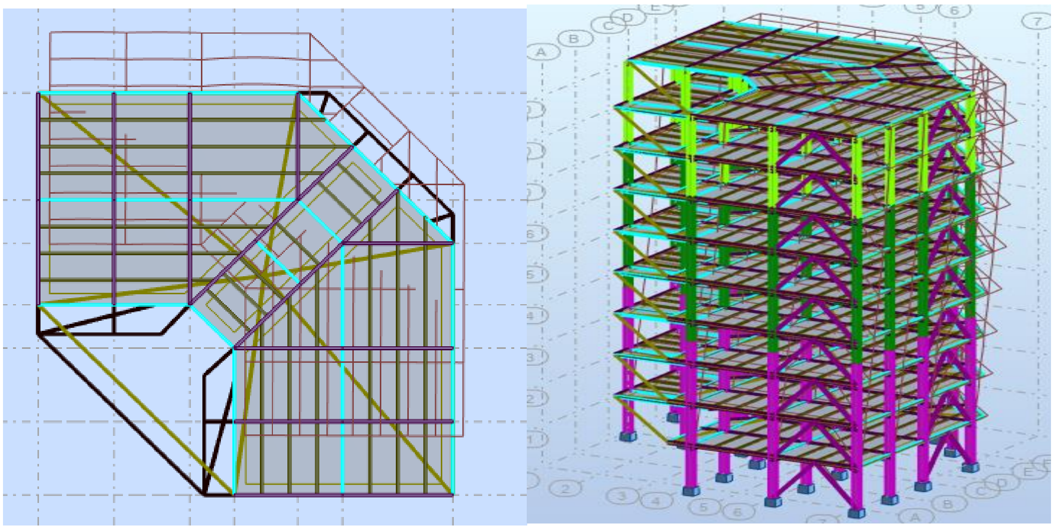


Figure 5.11: 1st mode of the final model (Translation along y-axis)

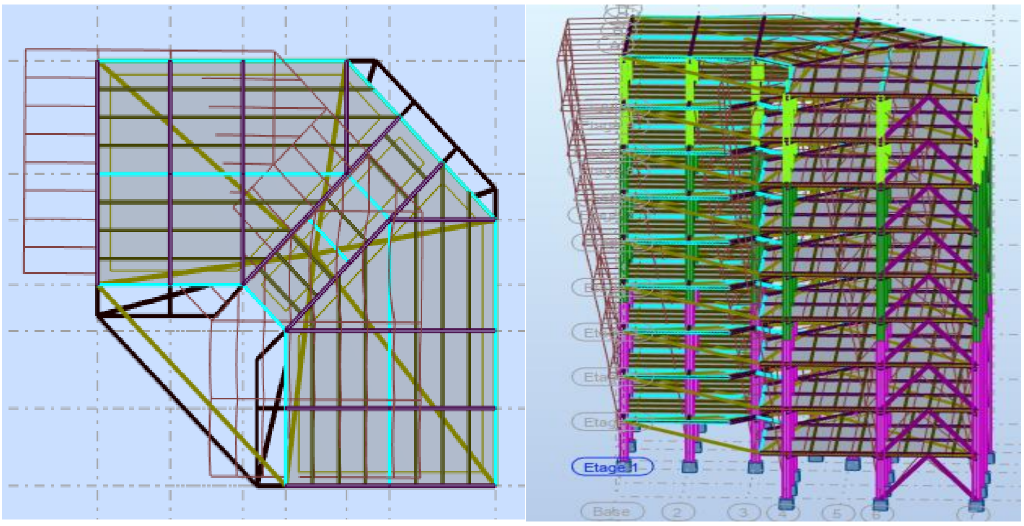


Figure 5.12: 2nd mode of the final model (Translation along x-axis)

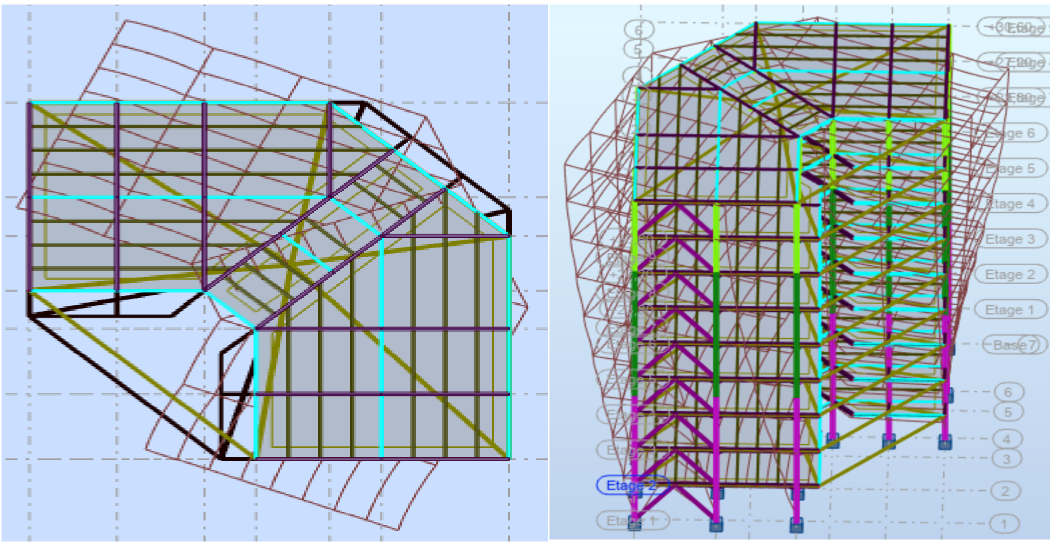


Figure 5.13: 3rd mode of the final model (Rotation around z-axis)

According to the table 5.3 the period is verified and in the three first modes there is two translations (along y and x-axes) and rotation around z-axe (in the 3rd mode) so the disposition of the bracing system is accepted.

5.7.2 seismic analysis:

✓ ***Verification of seismic force V:***

According to RPA2003 the resultant of the seismic forces at the base obtained by combining the modal values must not be less than 80 % of the resultant of the seismic forces that is determined by the equivalent static method V: $V_{dynamic} > 0.8 \times V$

A = 0.15, D=2.1; Q = 1.25; W = 17737.1 kN (from Robot software) kN;

R = 3 (bracing system V)

$V = \frac{A.D.Q}{R} \times W$; $V_X = V_Y = V = \frac{0.15\times2.1\times1.25}{3} \times 17737.1 = 2328$ kN

$0.8\times V = 1862.4$ kN

The results of $V_{dynamic}$ are adopted from the Robot software after the application of the spectrum calculation in both directions x and y:

Seismic force in the x direction: V_{xdyn}= 2644,63kN

Seismic force in the y direction: V_{ydyn}=2234,61kN

V_{xdyn} = 2644,63kN > 0.8V = 1862.4kN → condition verified

V_{ydyn} = 2234,61kN > 0.8V = 1862.4kN → condition verified

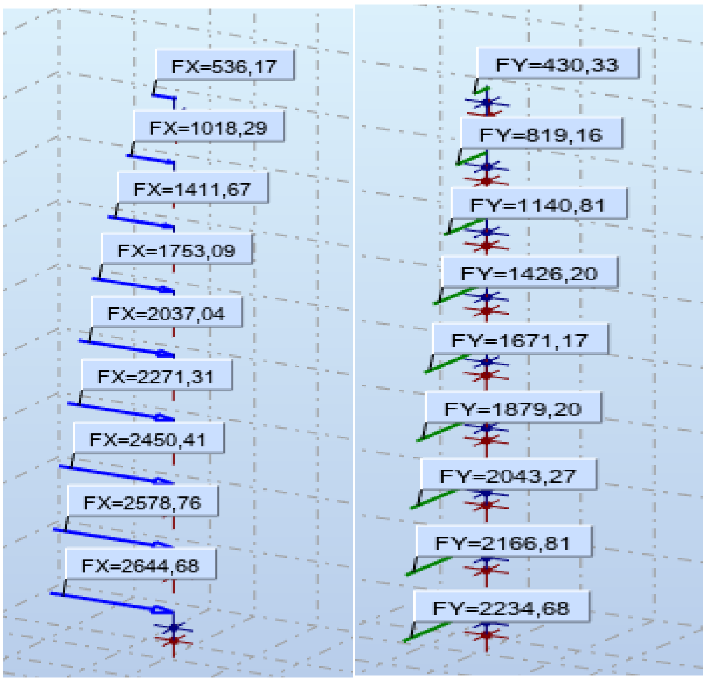


Figure 5.14: Distribution of seismic force in the both directions x and y.

✓ ***Verification of inter floor displacement:***

The horizontal displacement at each level (k) is calculated under the seismic actions (E_x and E_y) as follow: $\delta_k = R.\delta_{ek}$ such as:

R = ratio behavior: R = 3 bracing system of V.

δ_{ek} = displacement of seismic forces F_i; Δ_k: relative displacement $\Delta k = \delta_k - \delta_{k-1}$.;

The relative lateral displacements of a floor that is in relation to the next adjacent floor shall not exceed 1% of the floor's height, which means that the relative displacement must be less than 3.4 cm. The inter-floor displacements do not exceed allowable displacements; therefore, the condition is verified. The results of calculation are in the following table:

Table 5.4: Inter-floor displacements in both directions Δ_k

		Direction x			Direction y		Condition
Level	**h_{level}(m)**	**δ_k(cm)**	**Δ_k (cm)**	**Δ_k< 1%$h_{étage}$**	**δ_k(cm)**	**Δ_k (cm)**	**Δk < 1% $h_{étage}$**
Ground-floor	3.4	0,5	0,5	**Verified**	1,0	1,0	**Verified**
1	3.4	1,1	0,5	**Verified**	2,0	1,0	**Verified**
2	3.4	1,6	0,6	**Verified**	3,0	1,0	**Verified**
3	3.4	2,2	0,6	**Verified**	4,0	1,0	**Verified**
4	3.4	2,9	0,6	**Verified**	4,9	0,9	**Verified**
5	3.4	3,4	0,6	**Verified**	5,7	0,8	**Verified**
6	3.4	3,9	0,5	**Verified**	6,3	0,7	**Verified**
7	3.4	4,4	0,5	**Verified**	6,8	0,5	**Verified**
8	3.4	4,7	0,3	**Verified**	7,2	0,4	**Verified**

- ✓ ***The P- δ effect:***

The eccentricity of vertical loads resulting from relative floor displacements causes additional axial forces on the structure components. This phenomenon is known as the P-Δ effect, this effect can be neglected in the building if the following condition is checked at all levels:

$$\theta = P_k \times \Delta_k / V_k \times h_k < 0{,}10$$

Where:

P_k: Total weight of the structure at level k;

V_k: Shear force at level k, given by $V_k = \sum Fi$;

h_k: Height of level k.

The following tables 5.5 and 5.6 represent the verification of P-Δ in both directions x and y:

Table 5.5: Verification of the P-Δ effect in x direction

			Direction x			condition
Level	h_{level}(cm)	P_k(kN)	V_x(kN)	Δ_k cm	θ	θ < 0,10
Ground-floor	340	17394,10	2644,68	0,5	0,010	Verified
1	340	15390,06	2578,76	0,5	0,009	Verified
2	340	13456,86	2450,41	0,6	0,010	Verified
3	340	11552,19	2271,31	0,6	0,009	Verified
4	340	9603,07	2037,04	0,6	0,008	Verified
5	340	7684,99	1753,09	0,6	0,008	Verified
6	340	5764,77	1411,67	0,5	0.006	Verified
7	340	3895,47	1018,29	0,5	0.0056	Verified
8	340	1940,22	536,17	0,3	0.0032	Verified

Table 5.6: Verification of the P-Δ effect in y direction

			Direction y			condition
Level	h_{level}(cm)	P_k(kN)	Vy(kN)	Δ_k cm	θ	θ < 0,10
Ground-floor	340	17394,10	2234,68	1,0	0,023	Verified
1	340	15390,06	2166,81	1,0	0,021	Verified
2	340	13456,86	2043,27	1,0	0,019	Verified
3	340	11552,19	1879,20	1,0	0,018	Verified
4	340	9603,07	1671,17	0,9	0,015	Verified
5	340	7684,99	1426,20	0,8	0,013	Verified
6	340	5764,77	1140,81	0,7	0,010	Verified
7	340	3895,47	819,16	0,5	0,007	Verified
8	340	1940,22	430,33	0,4	0,005	Verified

According to the obtained results in both directions, the P-Δ effect can be neglected.

5.8 CONCLUSION:

- ✓ The initial model did not verify the RPA99/03 requirements and therefore a bracing system was added to the final model.
- ✓ All conditions of modal analysis (modes, period, mass participation) are checked.
- ✓ All required verifications by the RPA99/03 are checked.
- ✓ The inter floor displacements are verified.
- ✓ The effect of the second order (or the effect P-Δ) can be neglected.
- ✓ This implies that the final reinforced model is acceptable.

CHAPTER 6

VERIFICATION OF STRUCTURAL ELEMENTS

6.1 INTRODUCTION:

The purpose of any structural calculations is to check all the resistant elements and to ensure the stability of the structure's frames.

Static stability is ensured at the level of the structure and at the level of each element. Therefore, two types of instability phenomena must be checked which are:

✓ ***Buckling:*** A bending of a long element under a compressive effort (simple buckling) or a compressive effort and a bending moment (bending buckling).

✓ ***Lateral buckling:*** A type of buckling that creates a horizontal displacement, a vertical displacement and a rotation of the compressed sole.

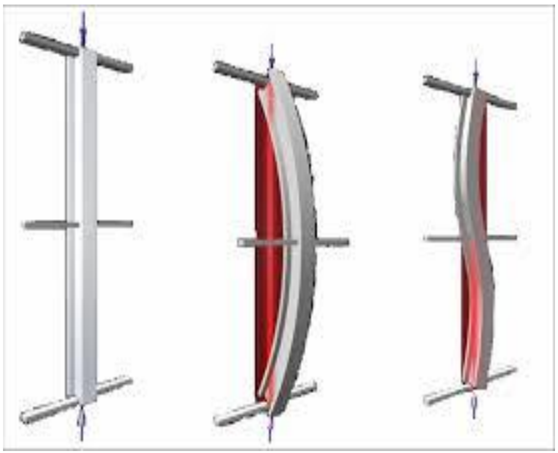


Figure 6.1: Buckling phenomena

Figure 6.2: Lateral buckling phenomena

6.2 STABILITY CHECK:

6.2.1 Verification of columns:

The efforts are extracted considering the following combinations:

- ULS=1.35G+1.5Q
- G+Q+E
- 0.8G±E
- G+Q+1.25E
- 0.8G±1.25E

- ***Verification of buckling:***

Members which are subjected to combined bending and axial compression should satisfy:

$$\frac{Nsd}{Xmin \times A \times fy/\gamma m1} + \frac{ky \times My.sd}{Wply \times fy/\gamma m1} + \frac{kz \times Mz.sd}{Wplz \times fy/\gamma m1} \leq 1 \quad \text{from CCM97 (5.5.1)}$$

Where N_{sd}, $M_{y,sd}$ and M_{zsd} are the design values of the compression force and the maximum moments about the y-y and z-z axis along the member, respectively.

k_y, k_z: are the interaction factors where:

$$\mu_y = \lambda_y \times (2\beta_{M.y} - 4) + \frac{(Wply - Wely)}{Wely} \leq 0.90$$

$$k_y = 1 - \frac{\mu y \times Nsd}{Xy \times A \times fy} \leq 1.50$$

$$\mu_z = \lambda_z \times (2\beta_{M.z} - 4) + \frac{(Wplz - Welz)}{Welz} \leq 0.90$$

$$k_z = 1 - \frac{\mu z \times Nsd}{Xz \times A \times fy} \leq 1.50$$

□$_z$, □$_y$: are the reduction factors due to flexural buckling from 5.5.1

$\beta_{M.y}$, $\beta_{M.z}$: are the equivalent uniform moment factors for flexural buckling.

□$_{min}$: minimum of □$_y$ and □$_z$

Determined of buckling length:

Determined of buckling length using the fixed nodes method:

$$L_f = \frac{1 + 0.145 \times (\eta 1 + \eta 2) - 0.265 \times (\eta 1 \times \eta 2)}{2 - 0.364 \times (\eta 1 + \eta 2) - 0.247 \times (\eta 1 \times \eta 2)} \times H$$

$$\eta_1 = \frac{Kc + Kc1}{Kc + Kc1 + Kb11 + Kb12} \; ; \; \eta_2 = \frac{Kc + Kc2}{Kc + Kc2 + Kb21 + Kb22}$$

Table 6.1: Characteristics of profile HEA450 of columns of ground level

Profile	Weight G(Kg/m)	Area A(cm^2)	h(mm)	b(mm)	I_y cm^4	I_z cm^4	W_{ply}(cm^3)	W_{plz}(cm^3)	iy(cm)	iz(cm)
HEA450	140	178	440	300	63720	9465	3216	965.5	18.92	7.29

According to axis y-y:

$K_c(HEA450) = \frac{I}{H} = \frac{63720\times10^4}{12000} = 53100mm^3$; $K_b(IPE270) = \frac{I}{L} = \frac{5790\times10^4}{5100} = 11352.94\ mm^3$

$K_{c2} = 0$; $K_{b}21 = K_{b}22 = 0$

$\eta_1 = \frac{53100+53100}{53100+ 53100+11352.94+11352.94} = 0.824$

$\eta_2 = 0$ (Fixed support)

$L_f = \frac{1+0.145\times(0.824+0)}{2-0.364\times(0.824+0)}\times3.4 = 0.66 \times 3.4 = 2.244\ m = 2244\ mm$

According to axis z-z:

$K_c(HEA450) = \frac{I}{H} = \frac{9465\times10^4}{12000} = 7887.5mm^3$; $K_b(IPE270) = \frac{I}{L} = \frac{5790\times10^4}{5100} = 11352.94\ mm^3$

$K_{c2} = 0$; $K_{b21} = K_{b22} = 0$

$\eta_1 = \frac{7887.5+7887.5}{7887.5+7887.5+11352.94+11352.94} = 0.41$

$\eta_2 = 0$ (Fixed support)

$L_f = \frac{1+0.145\times(0.41+0)}{2-0.364\times(0.41+0)}\times3.4 = 0.57\times3.4 = 1.938\ m = 1938\ mm$

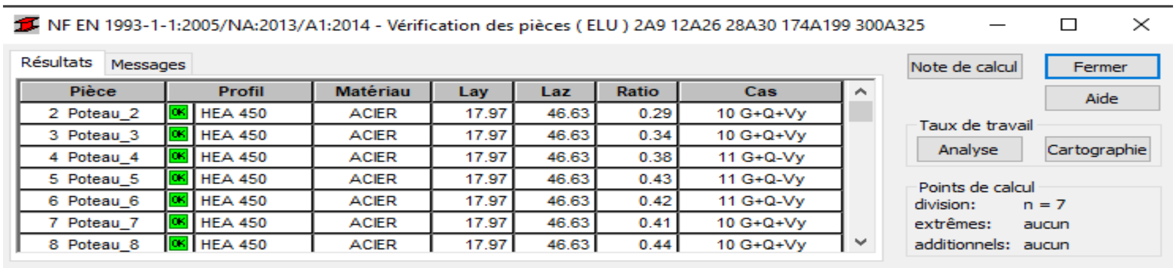
- Maximum slenderness: $\lambda_y = \frac{Lfy}{iy} = \frac{2244}{189.2} = 11.86$ and $\lambda_z = \frac{Lfz}{iz} = \frac{1938}{72.9} = 26.58$

$\lambda_y = 11.86 < \lambda_z = 26.58$ so buckling along y-y axis.

- Reduced slenderness: $\bar{\lambda} = \frac{\lambda}{\lambda_1}\sqrt{\beta_A} = \frac{26.58}{93.9} = 0.283 > 0.2$ there is a risk of buckling.
- Choice of curve: $\frac{h}{b} = \frac{440}{300} = 1.46 > 1.2$ $t_f = 21 < 40$ so:
  buckling along axis y-y so curve a $\alpha = 0.21$
- Reduction factor: $Ø = 0.5[1+\alpha(\bar{\lambda}-0.2) + \bar{\lambda}^2] = 0.5487$ $\chi = \frac{1}{Ø+\sqrt{Ø^2-\bar{\lambda}^2}} = 0.97 < 1$

$$N_{brd} = \frac{\chi\ \beta_A\ A\ f_y}{\gamma_{Mo}},\ N_{brd} = \frac{0.97\times1\times17800\times235}{1.1}\times10^{-3}$$

$N_{sd} = 1899.14kN < N_{brd} = 3705.74\ kN$ … C.V



Pièce	Profil	Matériau	Lay	Laz	Ratio	Cas
2 Poteau_2	OK HEA 450	ACIER	17.97	46.63	0.29	10 G+Q+Vy
3 Poteau_3	OK HEA 450	ACIER	17.97	46.63	0.34	10 G+Q+Vy
4 Poteau_4	OK HEA 450	ACIER	17.97	46.63	0.38	11 G+Q-Vy
5 Poteau_5	OK HEA 450	ACIER	17.97	46.63	0.43	11 G+Q-Vy
6 Poteau_6	OK HEA 450	ACIER	17.97	46.63	0.42	11 G+Q-Vy
7 Poteau_7	OK HEA 450	ACIER	17.97	46.63	0.41	10 G+Q+Vy
8 Poteau_8	OK HEA 450	ACIER	17.97	46.63	0.44	10 G+Q+Vy

Figure 6.3: Results of verification of columns HEA450 from Robot software.

The same steps are followed for checking the other columns, the results are in the following table:

Table 6.2: Verification of columns for flexural buckling

Level	Profile		N_{sd}(kN)	N_{bd}(kN)	$M_{sd,y}$	$M_{sd,z}$	χ_{min}	N_{sd}/N_{brd}	k_y	k_z	r	r≤1
Roof		G+Q+Ey	225.65	2852.25	14.05	8.65	0.85	0.08	0.51	0.97	0.15	CV
8	HEA360	G+Q+Ey	485.6	2852.25	16.29	9.46	0.85	0.17	0.51	0.95	0.20	CV
7		G+Q+Ey	979.3	2852.25	16.59	9.84	0.85	0.34	0.51	0.93	0.25	CV
6		G+Q+Ey	1225.69	3314.54	21.13	11.64	0.89	0.37	0.51	0.91	0.29	CV
5	HEA400	G+Q+Ey	1450.63	3314.54	23.61	12.57	0.89	0.44	0.51	0.89	0.34	CV
4		G+Q+Ey	1618.87	3314.54	25.83	13.89	0.89	0.49	0.51	0.85	0.39	CV
3		G+Q+Ey	1963.12	3705.74	24.56	13.36	0.89	0.53	0.30	0.46	0.40	CV
2	HEA450	G+Q+Ey	2089.5	3705.74	33.6	10.93	0.89	0.56	0.52	0.84	0.46	CV
1		G+Q+Ey	2215.9	3705.74	47.53	10.88	0.89	0.598	0.52	0.86	0.56	CV

Verification of lateral buckling:

If there is a risk of lateral buckling it must be checked using the following expression:

$$\frac{Nsd}{Xz\times A\times fy/\gamma m1}+\frac{ky\times My.sd}{Xlt\times Wply\times fy/\gamma m1}+\frac{kz\times Mz.sd}{Wplz\times fy/\gamma m1}\leq 1$$

Where:

k_{LT}=1- $\frac{\mu lt\times Nsd}{Xz\times A\times fy}\leq 1.50$

$\mu_{LT}= 0.15\times\bar{\lambda}_{LT}\times\beta_{M.LT}-0.15\leq 0.90$

χ_{LT}: is the reduction factor due to lateral torsional buckling.

$\bar{\lambda}_{LT}=\sqrt{\frac{\beta w\times Wply\times fy}{Mcr}}=\frac{\lambda LT}{\lambda 1}\sqrt{\beta w}<0.4$ if this condition checked means that there is no risk of lateral buckling, so not necessary to check the expression above.

λ_{LT} determined according to CCM97 Annex" B" as following:

$$\lambda_{LT}=\frac{k\times L\times(\frac{wpl^2}{Iw\times Iz})\text{^}0.25}{\sqrt{c}\{\frac{k}{kw}\times 2+\frac{kl\times 2\times G\times Lt}{\pi\times 2\times E\times Iw}\}\text{^}0.25}$$

Table 6.3: Verification of the columns for lateral buckling

Profile	iz(cm)	K	Kw	C1	λ_{LT}	βw(class1)	$\overline{\lambda}_{LT}$
HEA360	7.43	0.5	1	3.149	24.21	1	0.258
HEA400	7.34	0.5	1	3.149	28.19	1	0.3
HEA450	7.29	0.5	1	3.149	32.5	1	0.342

Table 6.4: Finale profile sections of all columns

	Profile columns
Roof	HEA360
8	HEA360
7	HEA400
6	HEA400
5	HEA400
4	HEA450
3	HEA450
2	HEA450
1	HEA450

6.2.2 *Verification of the main beams:*

The profiles of main beams are IPE270, which are obtained from chapter 3.

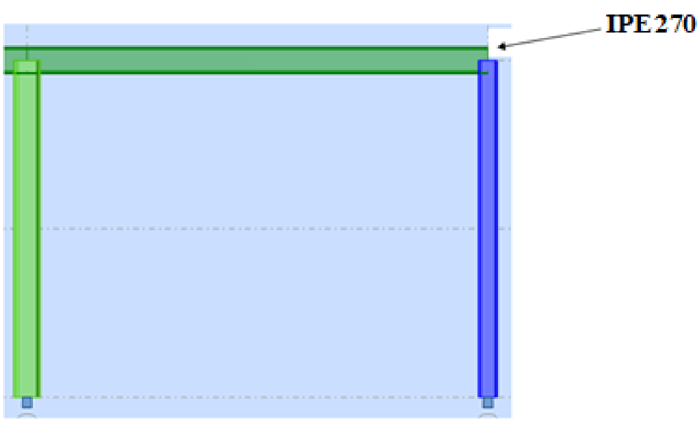
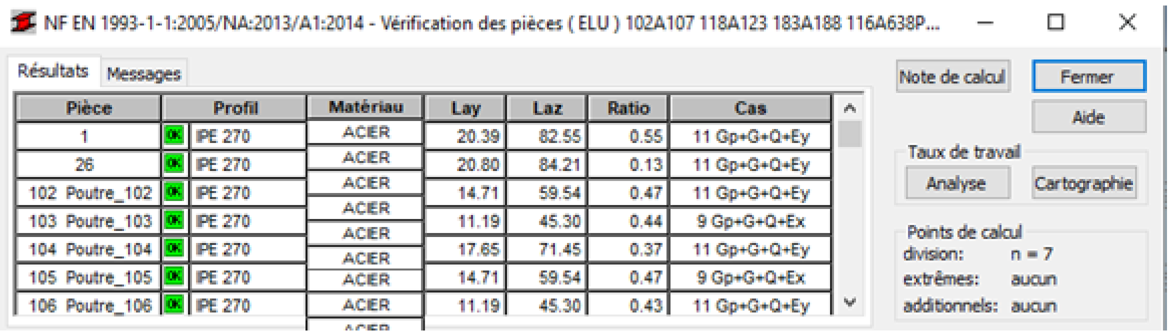


Figure 6.4: Position of main beams.



Pièce	Profil	Matériau	Lay	Laz	Ratio	Cas
1	OK IPE 270	ACIER	20.39	82.55	0.55	11 Gp+G+Q+Ey
26	OK IPE 270	ACIER	20.80	84.21	0.13	11 Gp+G+Q+Ey
102 Poutre_102	OK IPE 270	ACIER	14.71	59.54	0.47	11 Gp+G+Q+Ey
103 Poutre_103	OK IPE 270	ACIER	11.19	45.30	0.44	9 Gp+G+Q+Ex
104 Poutre_104	OK IPE 270	ACIER	17.65	71.45	0.37	11 Gp+G+Q+Ey
105 Poutre_105	OK IPE 270	ACIER	14.71	59.54	0.47	9 Gp+G+Q+Ex
106 Poutre_106	OK IPE 270	ACIER	11.19	45.30	0.43	11 Gp+G+Q+Ey

Figure 6.5: Results of verification of main beams IPE270 from robot software.

- ***Verification of bending strength:***

The value of plastic resistance moment of IPE270 from chapter 3 is $\mathbf{M_{pl,rd} = 232.58}$ **kN.m**

The maximum value of the bending moment of IPE270 from robot software obtained by the combination 1.35G+1.5Q is: $\mathbf{M_{sd} = -156.7}$ **kN.m**

$$\mathbf{M_{sd} = 156.7\ kN.m < M_{pl,rd} = 232.58\ kN.m} \qquad \textit{condition verified}$$

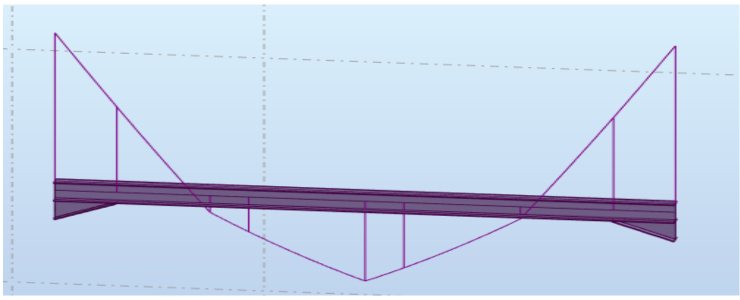


Figure 6.6: Bending moment Diagram of main beam.

- ***Verification of shear strength:***

The value of plastic resistance shear force of IPE270 from chapter 3 is $\mathbf{V_{plrd} = 300.3}$ **kN**

The maximum value of the shear force of IPE270 from robot software obtained by the combination 1.35G+1.5Q is: $\mathbf{V_{sd} = 77.45}$ **kN**

$$\mathbf{V_{sd} = 77.45\ kN < V_{plrd} = 300.3\ kN} \qquad \textit{condition verified}$$

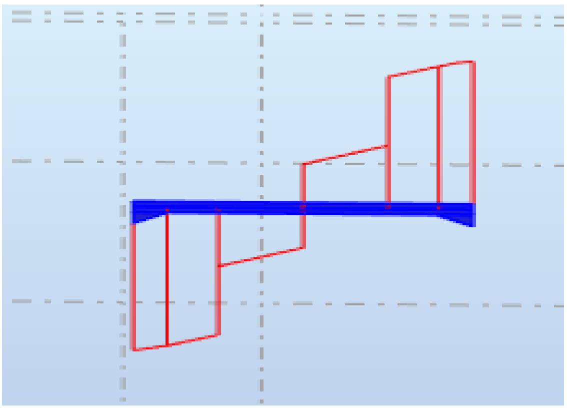


Figure 6.7: Shear force diagram of main beam.

- ***Verification of interaction between moment and shear force:***

$0.5\times V_{plrd}$=150.15 kN > V_{sd}= 77.45 kN → *There is no interaction*

6.2.3 Verification of joists:

The profile of joists are IPE200, which are obtained from chapter 3.

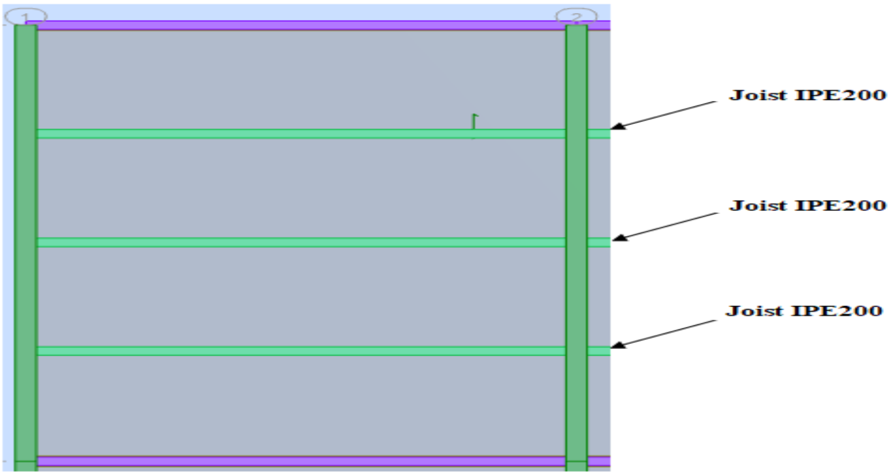
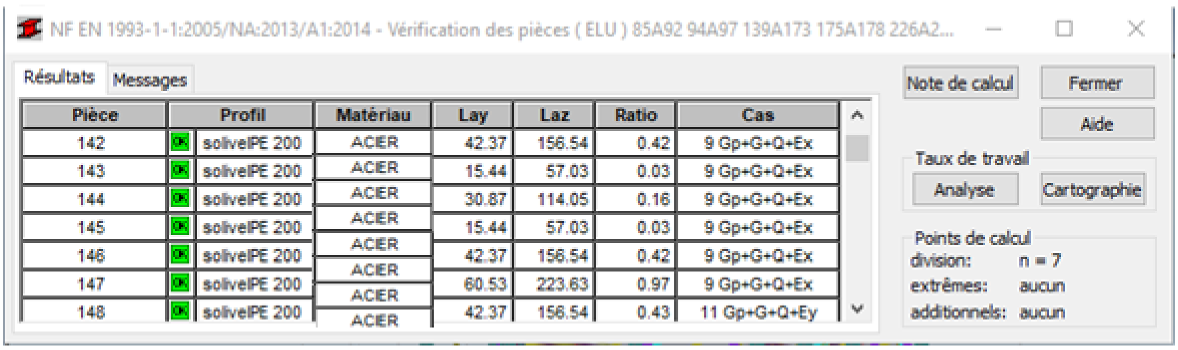


Figure 6.8: Position of joists.



Pièce	Profil	Matériau	Lay	Laz	Ratio	Cas
142	solivelPE 200	ACIER	42.37	156.54	0.42	9 Gp+G+Q+Ex
143	solivelPE 200	ACIER	15.44	57.03	0.03	9 Gp+G+Q+Ex
144	solivelPE 200	ACIER	30.87	114.05	0.16	9 Gp+G+Q+Ex
145	solivelPE 200	ACIER	15.44	57.03	0.03	9 Gp+G+Q+Ex
146	solivelPE 200	ACIER	42.37	156.54	0.42	9 Gp+G+Q+Ex
147	solivelPE 200	ACIER	60.53	223.63	0.97	9 Gp+G+Q+Ex
148	solivelPE 200	ACIER	42.37	156.54	0.43	11 Gp+G+Q+Ey

Figure 6.9: Results of verification of joists IPE200 from Robot software.

- ***Verification of bending strength:***

The value of plastic resistance moment of IPE200 from chapter 3 is $\mathbf{M_{plrd} = 128.61}$ **kN.m**

The maximum value of the bending moment of IPE200 from robot software obtained by the combination 1.35G+1.5Q is: $\mathbf{M_{sd} = 72.15}$ **kN.m**

$\mathbf{M_{sd} = 72.15kN.m < M_{plrd} = 128.61\ kN.m}$ ***Condition verified***

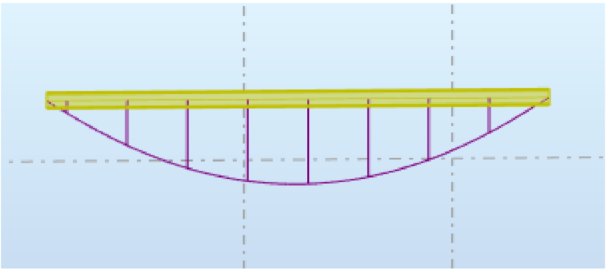


Figure 6.10: Bending moment diagram of joists.

- ***Verification of shear strength:***

The value of plastic resistance shear force of IPE200 from chapter 3 is:

$\mathbf{V_{plrd} = 190.165\ kN}$

The maximum value of the shear force of IPE200 from robot software obtained by the combination 1.35G+1.5Q is $\mathbf{V_{sd} = 48.27kN}$

$\mathbf{V_{sd} = 48.27kN < V_{plrd} = 190.165\ kN}$ **Condition** ***Verified***

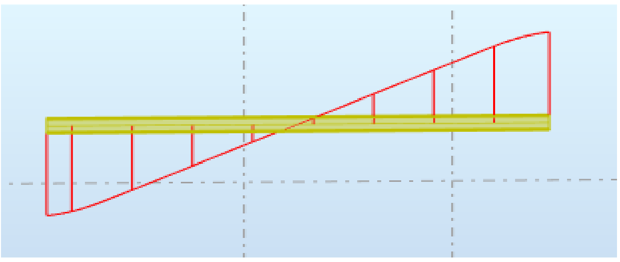


Figure 6.11: Shear force diagram of joists.

- ***Verification of interaction between moment and shear force:***

$0.5 \times V_{plrd} = 95.08\ kN > V_{sd} = 48.27kN \rightarrow$ *There is no interaction*

6.2.4 Verification of bracing system:

The types of bracing system that are used in this study are the bracing system of inverted V, the profile used are 2UPN200.

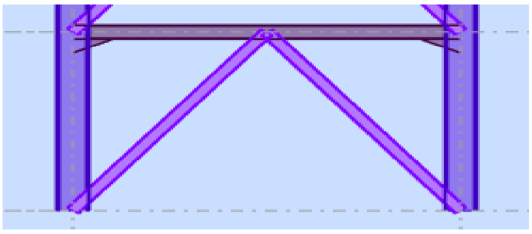
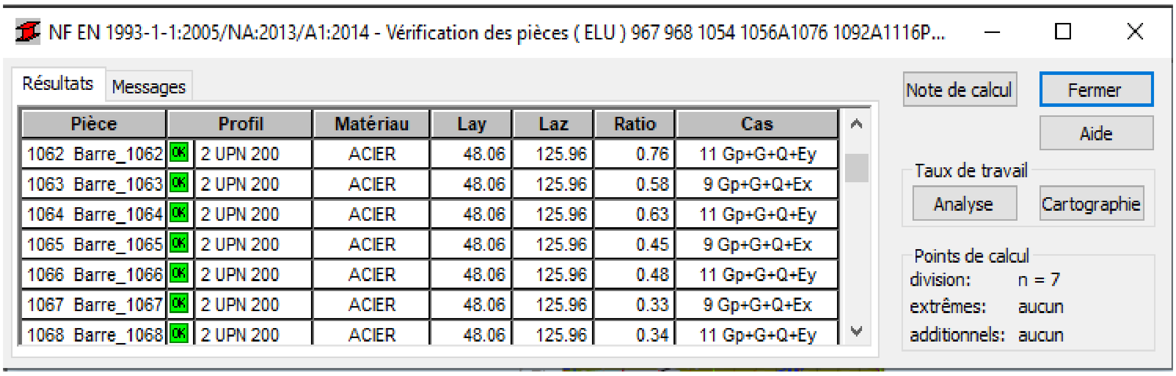


Figure 6.12: Position of bracing system.



Pièce	Profil	Matériau	Lay	Laz	Ratio	Cas
1062 Barre_1062 OK	2 UPN 200	ACIER	48.06	125.96	0.76	11 Gp+G+Q+Ey
1063 Barre_1063 OK	2 UPN 200	ACIER	48.06	125.96	0.58	9 Gp+G+Q+Ex
1064 Barre_1064 OK	2 UPN 200	ACIER	48.06	125.96	0.63	11 Gp+G+Q+Ey
1065 Barre_1065 OK	2 UPN 200	ACIER	48.06	125.96	0.45	9 Gp+G+Q+Ex
1066 Barre_1066 OK	2 UPN 200	ACIER	48.06	125.96	0.48	11 Gp+G+Q+Ey
1067 Barre_1067 OK	2 UPN 200	ACIER	48.06	125.96	0.33	9 Gp+G+Q+Ex
1068 Barre_1068 OK	2 UPN 200	ACIER	48.06	125.96	0.34	11 Gp+G+Q+Ey

Figure 6.13: Results of verification of bracing 2UPN200 from Robot software.

Table 6.5: Characteristics of profile UPN200

profile	Area $A(cm^2)$	h(mm)	b(mm)	$W_{ply}(cm^3)$	$W_{plz}(cm^3)$	$I_y(cm^4)$	$I_z(cm^4)$
2UPN200	32.2	200	75	228	51.8	1910	148

- ***Verification at tension:***

The most stressed bar has the maximum tension force $\mathbf{N_{sd} = 584.35\ kN}$, the following condition must be checked:

$$N_{sd} = 584.35 \text{ kN} \leq 2\times N_{trd}$$

Where:

N_{sd}: Maximum normal force obtained from robot software by the combination G+Q+1.25E

N_{trd}: Plastic normal force given by the following expression:

$N_{trd}=\frac{A\times fy}{\gamma m0}=\frac{3220\times 235}{1.1}\times 10^{-3}=687.91$ kN

N_{sd} = 584.35kN< 2×N_{trd} = 1375.82 kN ***Condition verified***

- ***Verification at compression:***

The value of compression force **N_{sd} = 609.6 kN** at each cross section shall satisfy:

$$N_{sd} \leq 2\times N_{crd}$$

N_{sd}= 609.6kN< 2×N_{crd}= 1375.82 kN ***Condition verified***

- ***Verification of Buckling:***

The condition N_{sd} =612.6kN ≤ N_{brd} must be checked:

N_{brd} = □×β_A×A×$\frac{fy}{\gamma m1}$

$\lambda_y=\frac{lfy}{2\times iy}=\frac{4250}{2\times 77}=27.597$; $\lambda_z=\frac{lfz}{2\times iz}=\frac{4250}{2\times 21.4}=99.3$ $\lambda_y < \lambda_z$ buckling is around y-y

$\overline{\lambda}_y=\frac{99.3}{93.91}=1.06 \leq 0.02$ there is a risk of buckling; curve a: α= 0.21; $Ø_y$=1.15; □$_y$=0.62

$N_{plrd}=2\times 0.62\times 1\times 3220\times\frac{235}{1.1}=853$ kN; N_{sd}=609.6 kN

N_{sd} =609.6 kN ≤ N_{brd}=853 kN ***Condition verified***

6.3 CONCLUSION:

- ✓ All condition of resistance and stability are verified.
- ✓ The obtained profiles for main beams are IPE270, for joists are IPE200, and for bracing system 2UPN200.
- ✓ The obtained profiles for columns are HEA360, HEA400 and HEA450.
- ✓ The results are used in the calculation of steel connections “Chapter 7” and also in the design of the infrastructure in the last chapter.

CHAPTER 7

DESIGN OF

STEEL CONNECTIONS

7.1 INTRODUCTION:

In steel buildings' frames, the structural elements are connected by steel connections; steel connections are used to join different elements together, ensuring the transmission and distribution of stresses.

The types of such links are: column-column, column-beam, beam-joist, base of column, bracing system connections...

The types of connections used in this project are welded connections, bolted connections (by ordinary bolts, and HR bolts).

7.2 MAIN BEAM - COLUMN CONNECTION:

To model the deformational behavior of a joint, shear deformation of the web panel and rotational deformation of the connections should be taken into account.

Joint configurations should be designed to resist the internal bending moments, normal forces and shear forces.

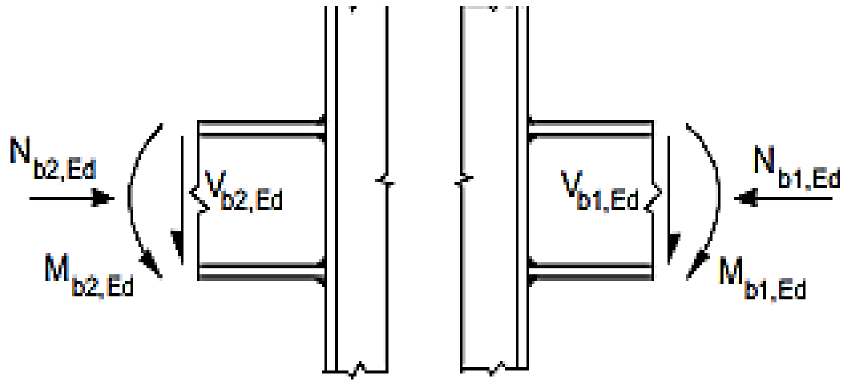


Figure 7.1: Connections, with forces and moments in beams.

The type of connection: **BeamIPE270-ColumnHEA450**

The results of the most stressed joint from Robot software are:

Bendig moment: M_{sd}=156.7 kN.m

Shear force: V_{sd}= 77.45 kN

Axial force: N_{sd}=0

Bolts used 10 high strength bolts **HR** of M20 class 10.9.

Dimension of steel plate: h_p=480mm; b_p=150mm; t_p=20mm

According to CCM97 class 10.9 M20: f_{yb}=900 MPa; f_{ub}=1000 MPa;

d=20mm; d_o=d+2=22mm.

7.2.1 Positioning of holes for bolts:

The positioning of holes for bolts, determined according to CCM97 as follows:

t: is the thickness of the thinner outer connected part. t_p=20mm

$1.2d_o \leq e_1 \leq \max(12t;\ 150mm)$ $\quad 26.4mm \leq e_1 \leq 240mm \quad e_1=60mm$

$2.2d_o \leq p_1 \leq \min(14t;\ 200mm)$ $\quad 48.4mm \leq p_1 \leq 200mm \quad p_1=80mm$

$1.5d_o \leq e_2 \leq \max(12t;\ 150mm)$ $\quad 33mm \leq e_2 \leq 240mm \quad e_2=80mm$

$3d_o \leq p_1 \leq \min(14t;\ 200mm)$ $\quad 66mm \leq p_1 \leq 200mm \quad p_2=90mm$

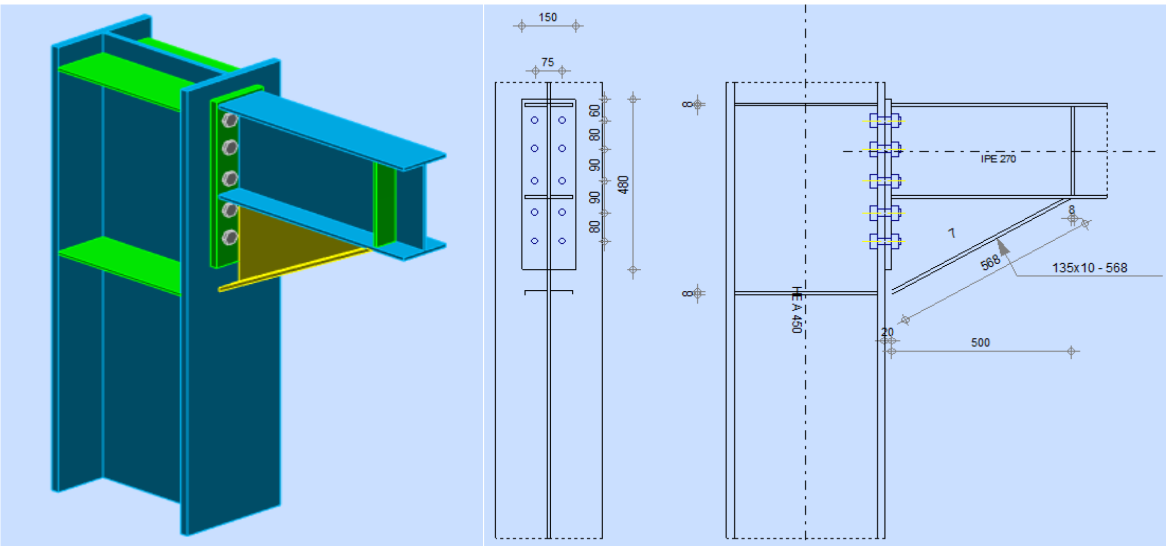


Figure 7.2 : BeamIPE270-ColumnHEA450 connection.

7.2.2 Verification of weld: (plate-beam)

- ***Effect N and V:***

The following condition must be checked*:*

$$\sqrt{2\left(\frac{Nsd}{\sum liai}\right)^2 + 3\left(\frac{Vsd}{2l3a}\right)^2} \leq \frac{fu}{\gamma Mw \times \beta w}$$

According to CCM97: FeE360 so Fu=360Mpa; β_w=0.8; γ_{Mw}=1.25 $\sum$liai=(l_1+2l_3+4l_2) a

N_{sd}=0; V_{sd}=77.45 kN; a=10mm l_1=135mm; l_2=(b_b-t_w)/2=64.2mm;

l_3=h_b-2t_f=270-2×10.2=249.6mm

$\sum$liai=l_1+2l_3+4l_2= (135+2×249.6+4×64.2)×10=8910mm^2

$\sqrt{3\left(\frac{77.45}{2\times0.2496\times0.01}\right)^2}\times10^{-3}$**=26.87MPa<**$\frac{360}{1.25\times0.8}$**=360 MPa** ***condition verified***

- ***Effect N and M:***

The following condition must be checked:

$$\sqrt{2}\left(\frac{\text{Nsd}}{\sum\text{liai}}+\frac{\text{Msd}}{\text{Iys}}\times\text{Vmax}\right)\leq\frac{\boldsymbol{fu}}{\boldsymbol{\gamma Mw\times\beta w}}$$

$N_{sd}=0$; $M_{sd}=156.7$ kN.m; $V_{max}=\frac{h}{2}+a$; $I_{y/s}=(l_1\times a)\times d'^2_1+4\times l_2\times a\times d'^2_2$ where:

$d'_1=\frac{h}{2}+\frac{a}{2}=\frac{270}{2}+\frac{10}{2}=140$mm; $d'_2=\frac{h}{2}-tf-\frac{a}{2}=\frac{270}{2}-10.2-\frac{10}{2}=119.8mm$

$I_{y/s}=(135\times10)\times140^2+4\times64.2\times10\times119.8^2=6.33\times10^7$ mm^4

$\sqrt{2}\times\frac{156.7\times10^6}{6.33\times10^7}\times145$=**357.9MPa< 360 MPa** ***condition verified***

7.2.3 Verification of bolts: (plate-column)

- ***Shear force:***

The following condition must be checked:

$$\mathbf{F_{vsd}\leq F_{s,\,rd}}$$

Where:

$F_{vsd}=\frac{Vsd}{np.nb}=\frac{77.45}{1\times10}=7.75$ kN; $\mathbf{F_{s,\,rd}}=\frac{Ks\times\mu\times n.Fp}{\gamma Ms}$

F_{pc}: preloading force F_{pc}= 0, 7.f_{ub}.As=0.7×1000×245=171.5 kN

n: the number of the friction surfaces=1

K_s=1,0 Bolts in normal holes

μ: Slip factor depend on Class of friction surfaces = 0.3 (class c)

$\mathbf{F_{s,rd}}=\frac{1\times0.3\times1\times171.5}{1.25}$=41.16 kN

F_{vsd} =7.75 kN≤ $F_{s,rd}$ =41.16 kN ***condition verified***

- ***Tension effect:***

Shear force of the most stressed bolt:

$$F_{M1}=\frac{Msd.d1}{nf\sum di^2}$$

n_f:: number of bolts in line= 2

d_1=80+90+90+80+60-10.2/2=394.9mm;

d_2=314.9mm; d_3=224.9 mm; d_4=134.9mm; d_5=54.9mm

$\sum di^2=d_1^2+d_2^2+d_3^2+d_4^{2+}d_5^2$; $\sum di^2=326.9\times10^3$mm^2

$F_{M1}=\frac{156.7\times 394.9\times 10^{-3}}{2\times 326.9\times 10^{-3}}$=94.65 kN

The following condition must be verified: $\mathbf{F_{ts.d} \leq F_{trd}}$;

γ_{Ms}=1.5; $F_{ts.d}=F_{M1}$= 94.65kN; $F_{trd}=0.7\times A_s\times F_{ub}=0.7\times 245\times 1000\times 10^{-3}$=171.5 kN

$F_{ts.d}$ = 94.65 kN ≤ F_{trd} =171.5 kN →*Condition verified*

- ***Verification of tension and shear force:***

The following condition must be checked:

$$\mathbf{F_{vsd} \leq F_{s,rd}} = \frac{Ks\times\mu\times n.(Fp-0.8Ftsd)}{\gamma Ms}$$

$$\mathbf{F_{s,rd}} = \frac{1\times 0.3\times 1(171.5-0.8\times 87.94)}{1.25} = 24.28 \text{ kN}$$

F_{vsd}=7.75 kN < $F_{s,rd}$=24.28 kN → *condition verified*

7.2.4 Verification of resistance pieces:

- ***Tension zone:***

The following expression must be checked:

$$\mathbf{N_t \leq F_t} = \frac{fy.twc.beff}{\gamma Mo}$$

N_t=M/d, M=156.7 kN.m; d=h_b-t_{fb}=270-10.2 = 259.8mm so: N = 603.16 kN;

t_{wc}=11.5mm (thickness web of column);

$beff=t_{fb}+2t_p+5(t_{fc}+r_c)$ =10.2+2×20+5(21+27) =290.2mm

$$\mathbf{F_t} = \frac{235\times 11.5\times 290.2}{1.1}\times 10^{-3}=712.97 \text{ kN}$$

N_t =603.16 kN≤ F_t=712.97 kN → *condition checked*

- ***Compression zone:***

No verification is necessary because the web has stiffener of thickness = 8mm

7.3 BEAM-JOIST CONNECTION:

In this project there is one type of connection between beam and joist which is:

BeamIPE270-JoistIPE200

Shear force: V_{sd}=48.27 kN;

Bolts used from Robot software: 4 ordinary bolts of M16 class 4.8.

and legs angles 100×100×10.

According to CCM97 class 4.8 M16:

f_{yb}=320 MPa; f_{ub}=400 MPa; d=16mm; d_o=d+2=18mm

7.3.1 Positioning of holes for bolts:

The positioning of holes for bolts determined according to CCM97 as follows:

t: is the thickness of the thinner outer connected part. t_{wjoist}=5.6mm

$1.2d_o \leq e_1 \leq \max(12t;\ 150mm)$ $\quad 21.6mm \leq e_1 \leq 150mm \quad e_1=35mm$

$2.2d_o \leq p_1 \leq \min(14t;\ 200mm)$ $\quad 39.6\ mm \leq p_1 \leq 78.4mm \quad p_1=60mm$

$1.5d_o \leq e_2 \leq \max(12t;\ 150mm)$ $\quad 27mm \leq e_2 \leq 150mm \quad e_2=50mm$

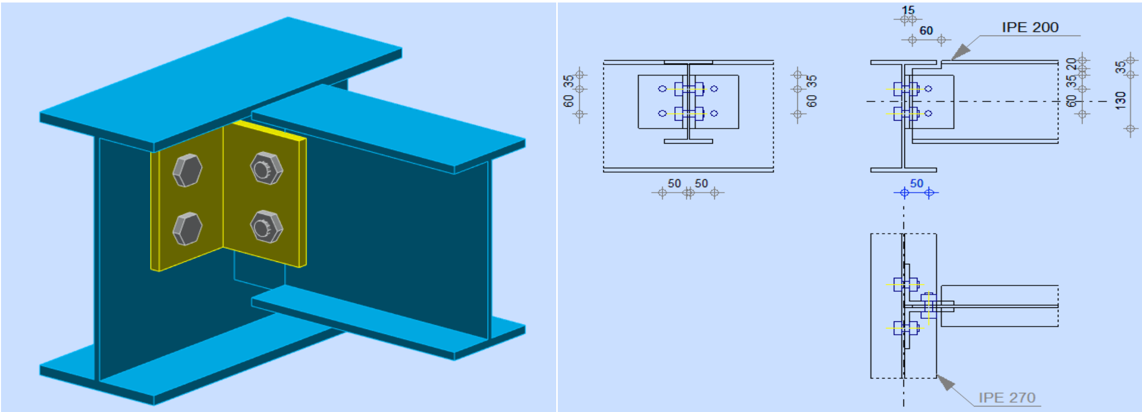


Figure 7.3: BeamIPE270-JoistIPE200 connection.

7.3.2 Verification of the bolts:

✓ ***Verification of shear:***

The shear plane passes through the not threaded part of the bolt, according to CCM97 category of connection is bearing type "A" the following condition must be checked:

$$\mathbf{F_{vsd} \leq F_{vrd}}$$

Where:

$F_{vrd}=\frac{0.6 fub.As}{\gamma Mb}$; γ_{Mb}=1.25(bolts stressed at tension) F_{vsd}=32.43 kN (from Robot)

$F_{vrd}=\frac{0.6\times400\times157}{1.25}$ =30.144 kN

There are 2 bolts so: $F_{vrd}=2\times F_{vrd}=2\times30.144=60.29$ kN

F_{vsd}=32.43 kN ≤ F_{vrd}=60.29 kN → *condition verified*

- ✓ ***Verification of diametric pressure:***

The following condition must be verified:

$$\mathbf{F_{v.sd} \leq F_{b,rd}}$$

$F_{brd}=\frac{2.5\times\alpha\times fu\times d\times t}{\gamma Mb}$; γ_{Mb}=1.25; fu=360 MPa; d=16mm; t=10mm (thickness of leg angle)

$\alpha=\min\{\frac{e1}{3do};\frac{p1}{3do}-\frac{1}{4};\frac{fub}{fu};1\}$

$\alpha=\min\{\frac{35}{54}=0.65;\frac{60}{54}-\frac{1}{4}=0.86;\frac{400}{360}=1.11;1\}$ $\boldsymbol{\alpha}$ **=0.65**

$F_{brd}=\frac{2.5\times0.65\times360\times16\times10}{1.25}$ =74.88 kN

$$\mathbf{F_{v.sd} = 32.43kN \leq F_{b,rd} = 74.88\ kN} \rightarrow \textit{condition verified}$$

7.4 COLUMN-SECONDARY BEAM CONNECTION:

The same stapes are followed to determine the connection between column and secondary beam, the connection is done by Robot software as following:

Type of connection is: **BeamPE200-ColumnHEA450** is done by using the legs angle of 100×10 and 4 ordinary bolts M16 of class 4.8

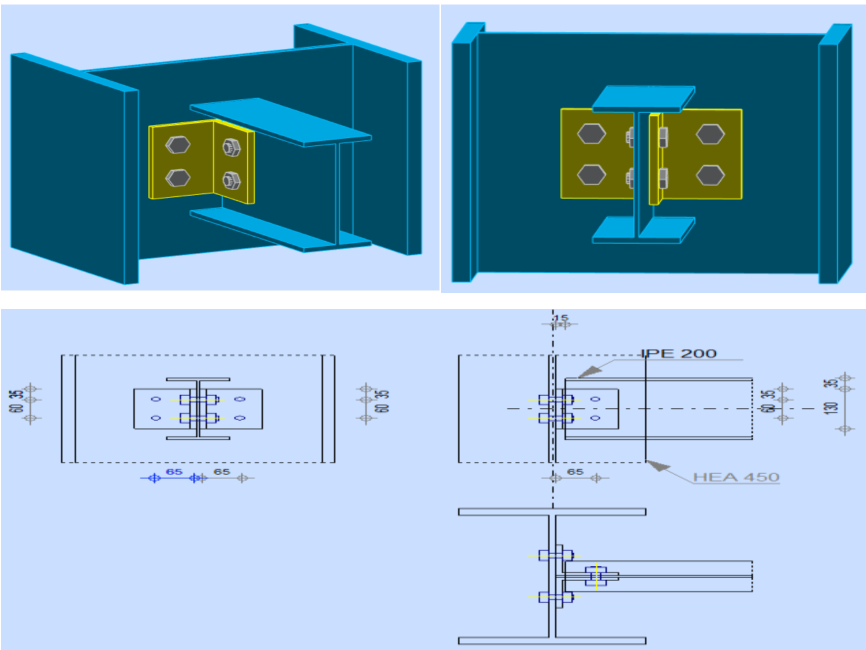


Figure 7.4: BeamIPE200-ColumnHEA450 connection.

7.5 COLUMN-COLUMN CONNECTION:

The most stressed connection between column and column determined by Robot, type of connection is flange-flange and web-web connected with bolted plates.

The values obtained from robot software as following:

Type of connection: **ColumnHEA450-ColumnHEA450**

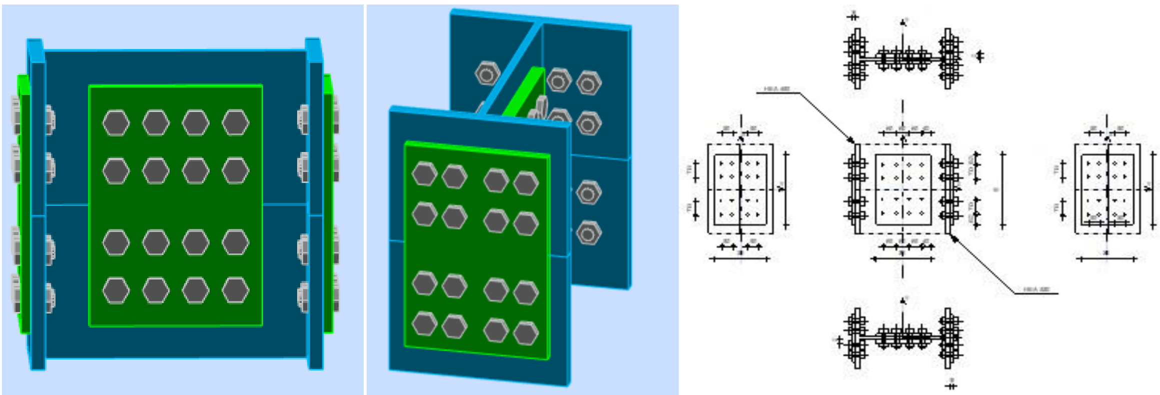


Figure 7.5: ColumnHEA450-ColumnHEA450.

7.5.1 Flange-cover plate connection:

The Bolts used of high strength 8 M20 class 8.8 and Steel plate of (350×240×15)

f_{yb}=600 MPa; f_{ub}=800 MPa; d=20mm; d_o=22mm

The shear plan passes through the filtered part, joint of category C

A_s=2.45 cm^2; A_v=3.14 cm^2 ; Ks=1,0; μ=0.3; n_b=2;

e_1=60mm; p_1=50mm; e_2=70mm

- ***bolts resistance:***
  - ✓ ***diametric pressure:***

$F_{v.sd}$=21.27kN ≤ $F_{b,rd}$= 147,31kN ***CV***

  - ✓ ***shear force of bolts:***

$F_{v,sd}$ =21.27 kN<$F_{s,Rd}$=37,42 kN ***CV***

  - ✓ ***Tension resistance:***

F_{Ed}= -80.65 kN ≤ $N_{net,Rd}$=535.8 kN ***CV***

7.5.2 Web cover plate connection:

The Bolts used of high strength 8 M20 class 8.8 and Steel plate of (350×260×20)

f_{yb}=600 MPa; f_{ub}=800 MPa; d=20mm; d_o=22mm

The shear plan passes through the filtered part, joint of category C

A_s=2.45 cm^2; A_v=3.14 cm^2 ; Ks=1,0; μ=0.3; n_b=2; e_1=40mm; p_1=70mm; e_2=60mm

- ✓ ***diametric pressure:***

$F_{v.sd}$=43.24kN $\leq$ $F_{b,rd}$=113.16kN ***CV***

- ✓ ***shear force of bolts:***

F_{sd}=43.24 kN<$F_{s,rd}$=74.84 kN ***CV***

7.6 BRACING SYSTEM CONNECTION:

The most stressed connection of bracing system determined by robot software, the connection between column and bracing, beam and bracing make by gusset, the values obtained from robot software as following:

4 Ordinary bolt of M20 class 6.8, Gusset of 660×660×20, f_{ub}=600 MPa; f_{yb}=480 MPa

$$26.4mm \leq e_1 \leq 150mm \quad e_1=40mm$$

$$48.4\,mm \leq p_1 \leq 78.4mm \quad p_1=65mm$$

$$33mm \leq e_2 \leq 150mm \quad e_2=140mm$$

The bars of 2UPN200 are connected in gusset by ordinary bolts and the gusset is connected with the beam by welding of thickness of 5 mm.

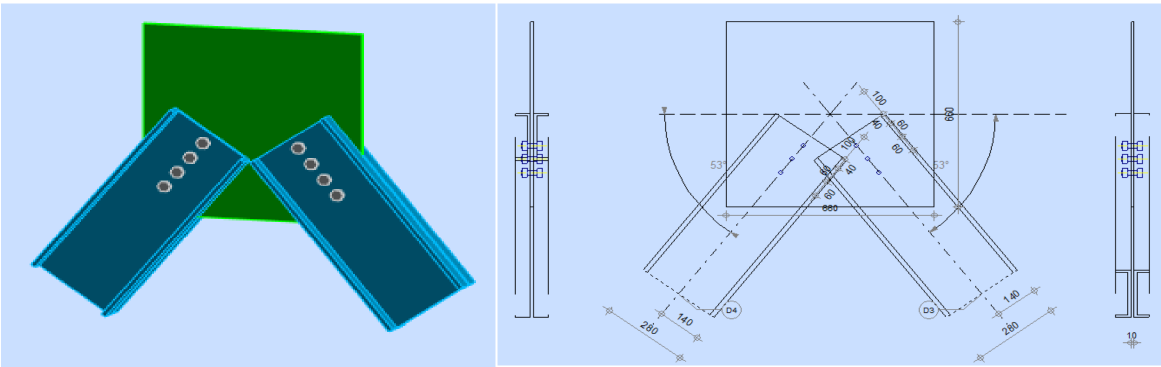


Figure 7.6: Gusset-bars 2UPN200 connection.

7.6.1 Verification of welding:

The following expression must be checked:

$$\mathbf{N_{sd1,2} \leq F_{w,\,rd}}$$

Thickness of welding is 5mm, N_{sd}= 609.6 kN

$N_{sd1}=N_{sd2}= N_{sd}\times\sin45=609.6\times\sin45=431.05$ kN

$$F_{wrd}=\frac{\alpha\times Fu\times 2L}{\sqrt{3-\sin^2\alpha}\times\beta w\times\gamma Mw}=\frac{5\times360\times2\times660}{\sqrt{3-\sin^2 45}\times0.8\times1.25} = 1234.04 \text{ kN}$$

$N_{sd1,2}$=431.05 kN ≤ $F_{w,\,rd}$ = 1234.04 kN→ *Condition verified*

7.6.2 Verification of shear bolts:

The following condition must be checked:

$$\mathbf{F_{vsd} \leq F_{v\,rd}}$$

Where:

$F_{vsd}=\frac{Nsd}{nb}=\frac{609.6}{4}$=152.4 kN; $F_{vrd}=\frac{0.6\times600\times245}{1.25}$=705.6 kN

F_{vsd} = 152.4 kN ≤ $F_{v\,rd}$ = 705.6 kN→ *Condition verified*

7.6.3 Verification of diametric pressure:

The following condition must be checked:

$$\mathbf{F_{vsd} \leq F_{b\,rd}}$$

$\alpha = \min\{\frac{40}{66};\frac{65}{66}-\frac{1}{4};\frac{600}{360};1\}=\min\{0.61;0.73;1.66;1\}=0.61$

$F_{brd}=\frac{2.5\times0.61\times600\times20\times20}{1.25}$ =**292.8 kN >F_{vsd}=152.4 kN→ *condition verified***

7.7 COLUMN BASE CONNECTION:

Column bases should be of sufficient size, stiffness and strength to transmit the axial forces, bending moments and shear forces in columns to their foundations or other supports without exceeding the load carrying capacity of these supports.

The values obtained from robot software as following:

Solicitations: N_{sd} = -2215.9kN; M_{sd} = 47.53kN.m; V_{sd} = 23.4 kN

Concrete: F_{ck} = 25 MPa concrete class 25/30

Spread footing: L = 1700 mm; B = 1700 mm; H = 900 mm

Anchor: 8 Bolt of high strength HR class 8.8 M20***;*** e_h=660mm, e_v=150mm;

L_1=60 mm; L_2=640 mm; L_3= 120mm; L_4=140 mm

Steel plate: L_p=750mm; b_p=550mm; t_p=15mm; F_y=235 MPa; F_u=360MPa

Stiffeners: l_s=750mm; W_s=550mm; h_s=440mm; t_s=25mm;

Dimensions of column base are in the following figure:

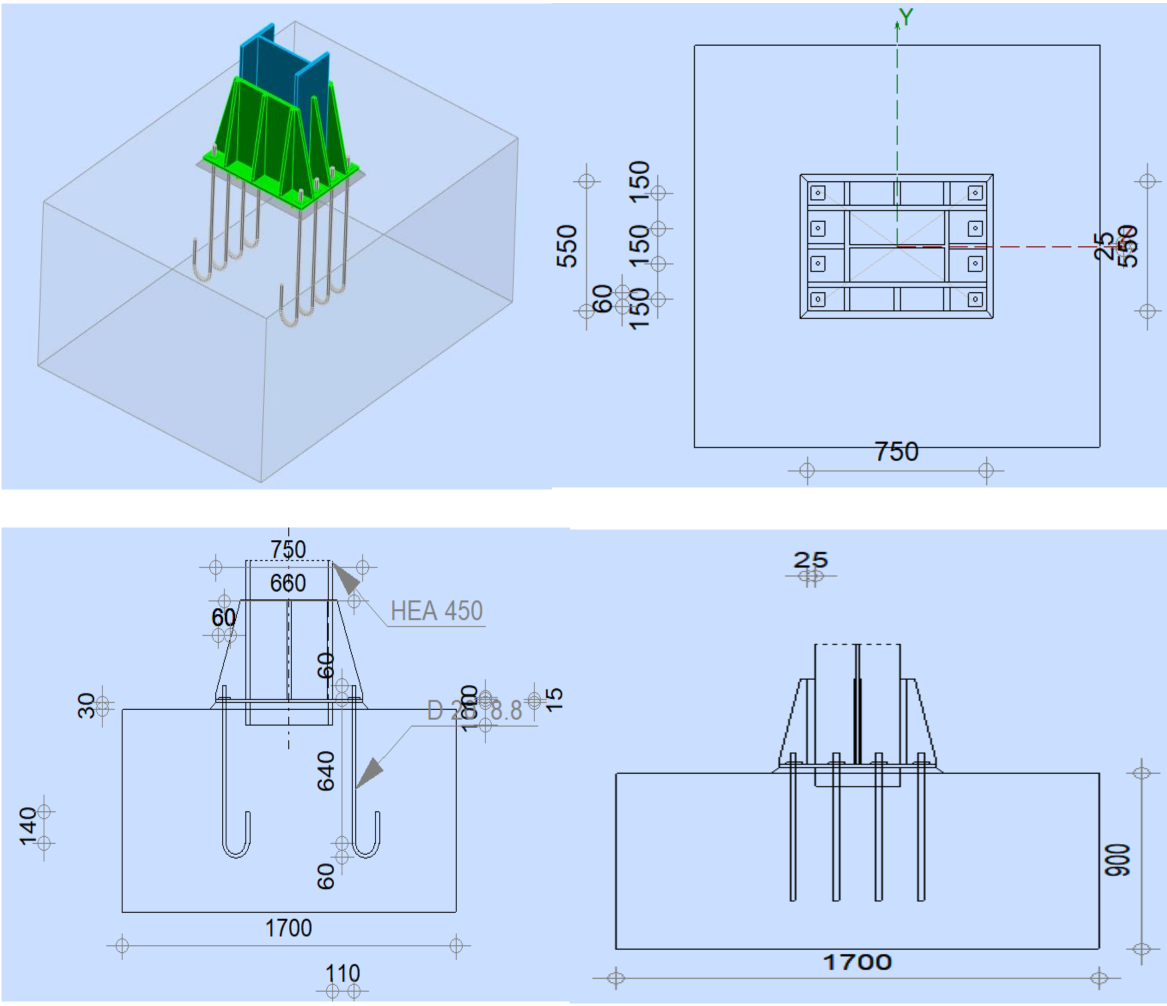


Figure 7.7: Column base connection.

7.7.1 Verification of compressive stress of concrete:

The following condition must be checked: $\boldsymbol{\sigma_b \leq f_j}$

Where:

$\sigma_b=\frac{Nsd}{Aeff}$; $f_j=\beta_j\times K_j\times f_c$; $\beta_j=0.67$; $f_j=33.33$MPa

$A_{eff}=(2\times C+t_{fc})(2\times C+b_{fc})$ and $C=t_p\times\sqrt{\frac{Fyp}{3fj\times\gamma mo}}=26$mm; $A_{eff}=2260.33cm^2$

$\sigma_b=\frac{2215.9\times10^3}{2260.33\times10^2}=$ **9.8MPa<33.33 MPa →** ***condition verified***

7.7.2 Verification of steel plate:

The following condition must be checked: $\boldsymbol{M_{sd}^{max} \leq M_{crd}}$

Where:

$M_{crd}=\frac{wel\times fy}{\gamma mo}$; $w_{el}=\frac{bp\times tp^2}{6}=\frac{550\times15^2}{6}=20625mm^3$

$M_{crd}=4.847$ kN.m; $M_{sd}^{max}=\frac{15^2}{2}\times\sigma_b=\frac{15^2}{2}\times9.8=1.103\times10^{-3}$ kN.m

$M_{sd}^{max}=1.103\times10^{-3}$ kN.m < M_{crd}=4.847 kN.m→ ***Condition verified***

7.7.3 Verification of shear force:

Verification of anchor bolt:

The following condition must be checked:

$$\boldsymbol{V_{sd} \leq F_{vb,Rd}}$$

Where:

$F_{vb,Rd}=a_b\times f_{ub}\times A_{sb}/\gamma_{M2}=43.12$ kN

V_{sd} = 23.4 kN ≤ $F_{vb,Rd}$ = 43.12 kN → ***Condition verified***

7.7.4 Verification of slip on flange:

The following condition must be checked:

$$\boldsymbol{V_{sd} \leq F_{f,Rd}}$$

Where:

$F_{f.Rd}=C_{f.d}\times N_{sd}^{C}$; $C_{f,d}=0.3$

$F_{f.Rd}=0.3\times2215.9=664.77$ kN

$V_{sd}=23.4$ kN $\leq F_{f,Rd}=664.77$ kN

7.7.5 Verification of Stiffener:

The following condition must be checked:

$$\max\left(\sigma_g;\frac{\tau}{0.58};\sigma_z\right)/(f_{yp}/\gamma_{Mo})\leq 1.0$$

M_1 = 20.56 kN.m (bending moment of stiffener); Q_1 = 265.36 kN (Shear force of stiffener)

Zs=194mm (position of neutral axis); τ=24.12 MPa; σ_g=19.14 MPa; σ_z=43.79 MPa

$$\sigma_z/(f_{yp}/\gamma_{Mo})=0.19<1 \rightarrow \textit{Condition verified}$$

7.7.6 Verification of welding:

Welding between the column and seat plate:

The following conditions must be checked:

$\frac{\sigma\perp}{0.9\times fu/\gamma M2}$ **=0.14≤ 1.0 → *Condition verified***

$\frac{\sigma\perp^2+3\,(\tau yII^2+\tau\perp^2))}{fu/(\beta w\times\gamma M2)}$**≤ 1.0 → *Condition verified***

$\frac{\sigma\perp^2+3\,(\tau zII^2+\tau\perp^2))}{fu/(\beta w\times\gamma M2)}$**=0.20 ≤ 1.0 → *Condition verified***

Where**:**

$\sigma\perp$ = 36.48 MPa (normal stress in the welding); $\tau\perp$ = 36.48 MPa (tangential stress)

β_w = 0.85; τ_{yII} = 1.33MPa (tangential stress parallel to V_{sdy});

τ_{zII} = 0.19MPa (tangential stress parallel to V_{sdz});

According to robot software results, all the conditions of resistance are checked and the highest R factor is : R = 0.97 < 1.

7.8 CONCLUSION:

- ✓ In this chapter, all calculations and verifications are performed according the EC3 using the Robot software and checked with manual calculations.
- ✓ The results of column base will be used in chapter 8.

CHAPTER 8

FOUNDATIONS

8.1 INTRODUCTION:

The foundations of a structure are those parts that are in contact with the ground which transmit the loads of the superstructure to the ground. The design and calculation of the foundations are based on different criteria, including to the superstructure's loads and the characteristics of the soil.

8.2 DIFFRENTS TYPE OF FOUNDATIONS:

Foundations can be classified according to soil in 03 types:

- ✓ Shallow foundations.
- ✓ Semi-deep foundations.
- ✓ Deep foundations.

The shallow foundations are used in this project which are foundations with a depth less than 3m, divided on three types:

- The functional foundations: formed by continuous footing or pad footing.
- The linear foundations: formed by strip footing.
- The surface foundations: formed by raft.

8.3 HYPOTHESIS OF CALCULATION:

The calculations are based on the following hypothesis:

- The footings are very rigid, therefore, the stress have linear distribution.
- The dimensioning of the footings is calculated in SLS.
- Batching of concrete:
  Blinding concrete : 150 kg/m^3
  Reinforced concrete : 350kg/m^3
- Allowable soil stress : 2.00 bars

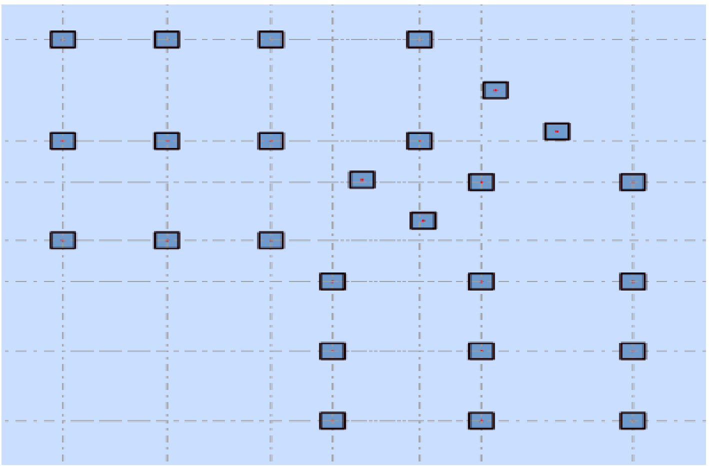


Figure 8.1: view plan at level 0.00.

8.4 LOADS COMBINNATION:

According to BAEL91V99, there are two combinations:

- SLS: G+Q
- ULS: 1.35G+1.5Q

According to RPA99/2003, the surface foundations are designed considering the combinations:

- G+Q+E
- 0.8G±E

8.5 PAD FOOTING:

8.5.1 Dimensioning:

According to Robot software the most stressed column of ground level has an effect of:

N=1885,61 kN   Combination used: SLS=G+Q

$\sigma=\frac{N}{S} \leq \sigma_{soil}$ S: area of footing (square) =B×A

such as B=A; σ_{soil} =2 bar allowable soil stress.

$S \geq \frac{N}{\sigma}=\frac{1885.61}{0.2} \times 10^3=9428050\ mm^2=9.43m^2$

$B=\sqrt{S}=\sqrt{9.43}=3.07m$; $\boldsymbol{B = 3.5m}$

$h \geq \frac{B-b}{4} - 0.05m$ such as: b = 70cm

$h \geq \frac{3.5-0.70}{4} - 0.05 = 0.65\text{m so } h = 65 \text{ cm}$

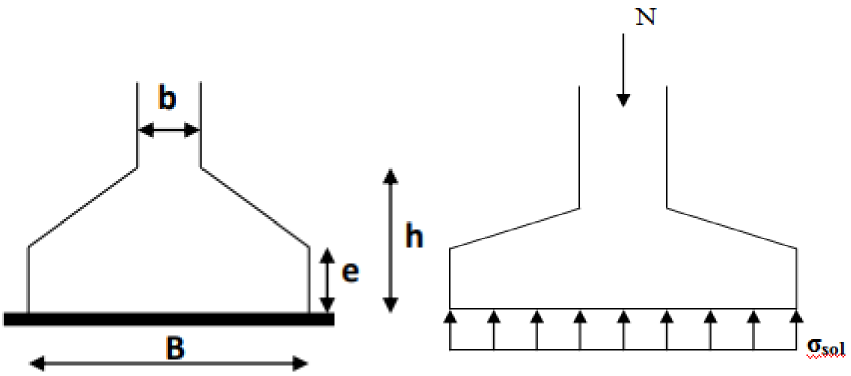


Figure 8.2: Dimensions of pad footing.

8.5.2 Verification of interference between two footing:

The condition following must be verified: $\mathbf{L_{min} \geq 1.5 \times B}$

L_{min} = 2.07m the minimum length between axis of two columns

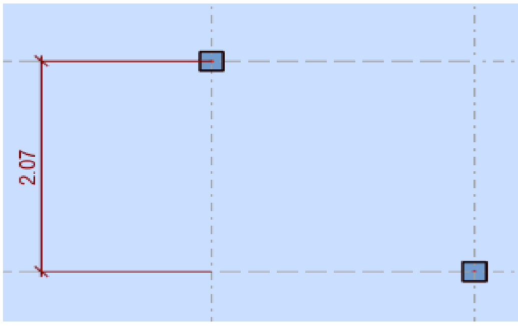


Figure 8.3: The minimum length between two axes of two columns.

$\mathbf{L_{min} = 2.07m < 1.5 \times 3.5 = 5.25\ m}$ ***condition not verified***

There is an overlap between footings, the next type should be considered “strip footings”.

8.6 STRIP FOOTING:

The normal force supported by the strip footing is the sum of the normal forces of all the columns that are located in the same line.

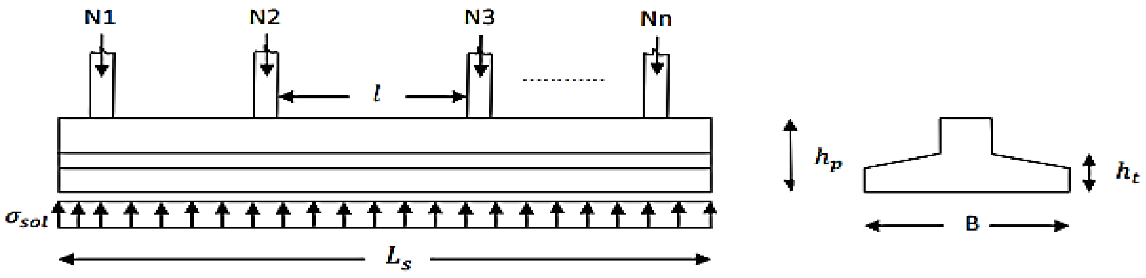


Figure 8.4: Characteristic of strip footing.

The condition following must be checked: $\boldsymbol{\sigma} = \frac{\mathbf{N}}{\mathbf{S}} \leq \boldsymbol{\sigma_{soil}}$ such as:

S = B×L   B: width of footing; L: length of file; N=∑Ni (for each file)

$B \geq \frac{N}{L\times\sigma soil} = \frac{2075.22}{10100\times0.2}\times10^3 = 1027.34$ mm = 1.027 m → ***B = 1.5 m***

The obtained results are presented in the following table:

Table 8.1: Dimensioning and verification of strip footing (SLS)

File	Effect N(kN)	L(m)	B(m)	B(choice)	S(m²)	σ(MPa)	$\sigma \leq \sigma_{soil}$	$L_{min} \geq 1.5\times B$
1	2075.22	10.1	1.027	1.5	15.15	1.37	CV	CV
3	3840.91	10.1	1.9	2	20.2	1.9	CV	CNV
4	2849.28	7	2.03	2.1	14.7	1.93	CV	CNV
5	2806.9	9.11	1.54	1.6	14.576	1.92	CV	CNV
7	3508.47	12	1.46	1.5	18	1.94	CV	CNV
B	3847.41	10.1	1.9	2	20.2	1.9	CV	CNV
F	5794.03	12	2.41	2.5	30	1.93	CV	CNV

L_{min} = **2.07 m** < 1.5 × 2.5 = **3.75 m** → ***Condition not verified.***

There is another condition that should be considered, which is:

The area of all footings must be less than 50% of the area of the building: $\frac{Sf}{Sb} < 50\%$

S_f: Area of all footings $S_f = 132.826\ m^2$

S_b: Area under the building $S_b = 261.725 m^2$

$\frac{132.826}{261.725} = 0.51$ = **51% > 50%** ***Condition not checked.***

The verification of interference between two footing is not checked, which means that there is an overlapping of footings. Also, the area of all footings more than 50 % of the area under the building, in such cases a ***raft foundation*** is more suitable and economical**.**

8.7 RAFT FOUNDATION:

The raft is a solid slab made under the total surface of the construction, this slab can be massive (high thickness), or ribbed (the slab is thin but it is stiffened by ribs), in this case the type adopted is a ribbed raft because it is more economical.

The normal force supported by the raft is the sum of the normal forces of all columns.

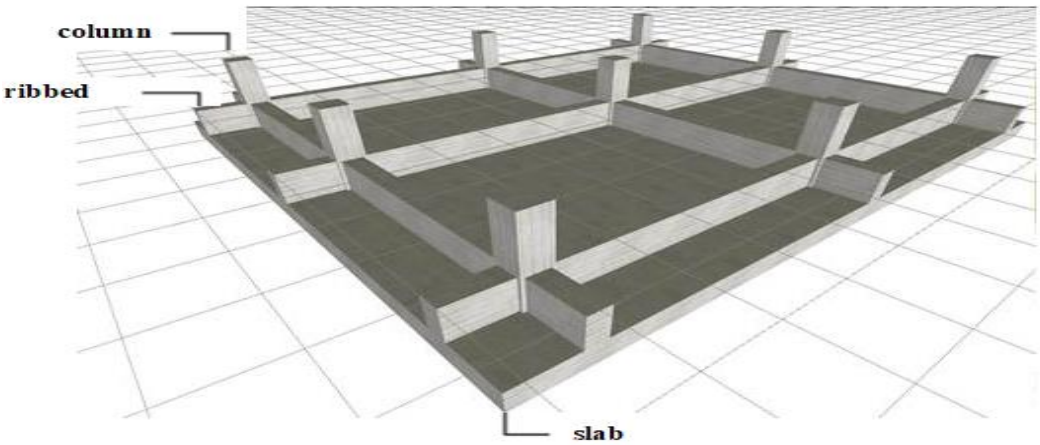


Figure 8.5: Composant of raft foundation.

8.7.1 Dimensioning of raft foundation:

- ***Determine the height of the ribs:***

$$h_r \geq \frac{Lmax}{10} = \frac{5100}{10} = 510 \text{ mm},$$

Height of the ribs: $h_r = 60$ cm

- ***Determine the width of the ribs:***

Width of the ribs must be more or equal the width of anchor that is determined in chapter 7:

$b_{Anchor} = 55$cm;

Width of the ribs obtained is $b_r = 70$ cm

- ***Determine the thickness of slab:***

$$e \geq \frac{Lmax}{20} = \frac{5100}{20} = 255 \text{ mm; So, } e = 40 \text{ cm}$$

- ***Rigidity condition:***

For a rigid raft it must: $L \leq \frac{\pi}{2} \times L_e$

L_e: Elastic length given by: $L_e = \sqrt[4]{\frac{4 \times E \times I}{K \times b}}$

K: Elastic coefficient of soil (from soil report) K=2.1347 kg/cm^3

E: Modulus of elasticity of concrete E = 32164.195MPa

I: Inertia of the raft

b: band of 1 meter b =1m

$h_t \geq \sqrt[3]{\frac{3K(\frac{2\times b}{\pi})^4}{E}}$ $\quad h_t \geq \sqrt[3]{\frac{3\times 21.347(\frac{2\times 1}{\pi})^4}{3.2164195\times 10^4}}$ $\quad h_t \geq 0.06m$

so, height of raft adopted: $\mathbf{h_t=100cm}$

The results of dimension are as following:

- Thickness of slab h_t = 40 cm
- Height of the ribs h_r = 60 cm; width of the ribs b_r = 70 cm
- Total height of raft h_t = 100 cm
- The area of the raft equals the area of the building plus + 50 cm in each side:

$$S_r = (20.17\times 20.17) - (9.07\times 9.07) - (\frac{7.67\times 7.67}{2}) = 295.15 \text{ m}^2$$

The area of the raft is: S_r=295.15m^2

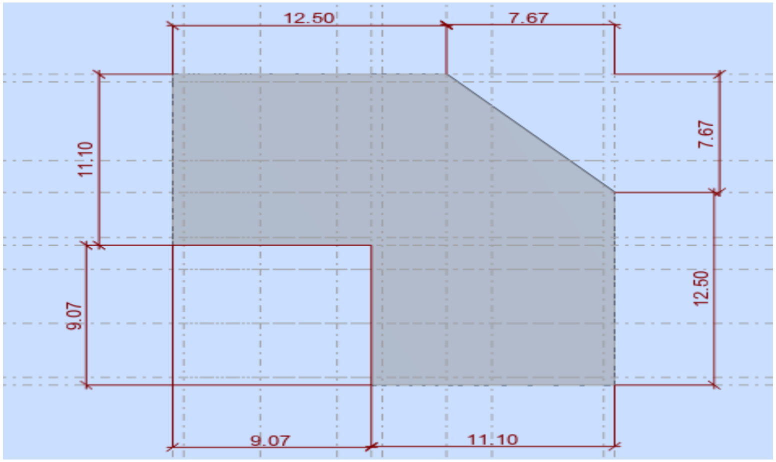


Figure 8.6: Dimensions of the raft.

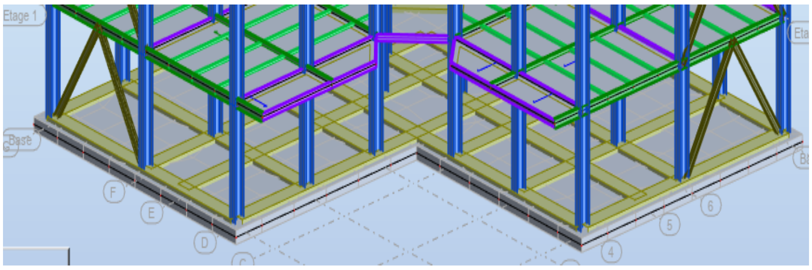


Figure 8.7: Raft foundation by robot software.

8.7.2 Verification of soil stress:

- ***Verification at SLS:***

Manual calculation:

$\sigma_{ser} = \frac{Nmaxser}{Sr} = \frac{35317.65}{295.15}$ = 119.66 MPa = **1.2 bar** < σ_{soil} = 2 bar → C***ondition verified***

The following condition must be checked:

$$\boldsymbol{\sigma_m = \frac{3\sigma1+\sigma2}{4} \leq \sigma_{soil}}$$

From robot: σ_1=119.66kN/m^2 ; σ_2=80.71kN/m^2 → Distribution of the stress is trapezoidal

$\sigma_m = \frac{3\times119.66+80.71}{4}$ =1.1 bar ≤ σ_{soil} =2 bar → ***Condition verified***

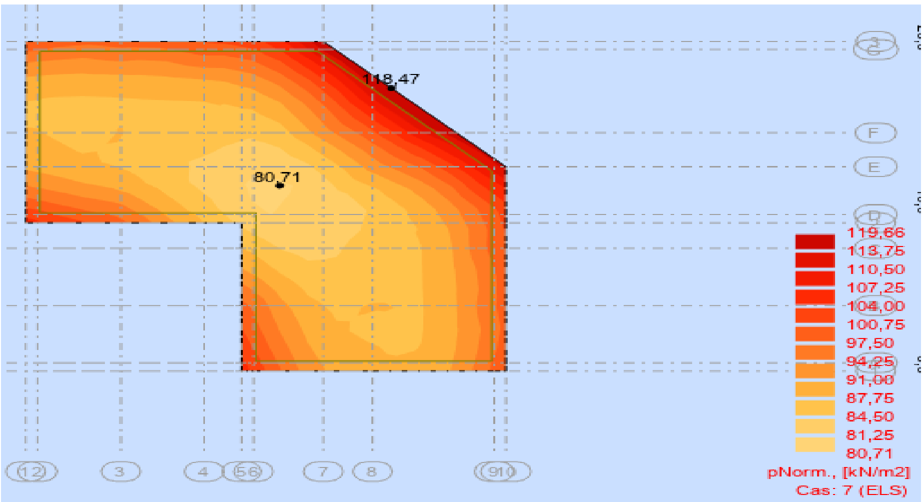


Figure 8.8: Soil stress at SLS from Robot software.

- ***Verification at ULS:***

According to RPA99/2003 the following condition must be checked:

$$\boldsymbol{\sigma_m = \frac{3\sigma1+\sigma2}{4} \leq 1.5\times\sigma_{soil}}$$

Where:

$\sigma_1 = \frac{N}{S} + \frac{Mx}{Ix} \times V_y \quad \sigma_2 = \frac{N}{S} - \frac{Mx}{Ix} \times V_y$

σ_1: Maximum soil stress; σ_2: Minimum soil stress

From robot: $\sigma_1 = 164.11$ kN/m^2;

$\sigma_2 = 109.37$kN/m^2 → Distribution of the stress is trapezoidal

$\sigma_m = \frac{3\times164.11+109.37}{4} = 1.5$ bar $\leq \sigma_{soil}=1.5\times2 = 3$bar → ***Condition verified***

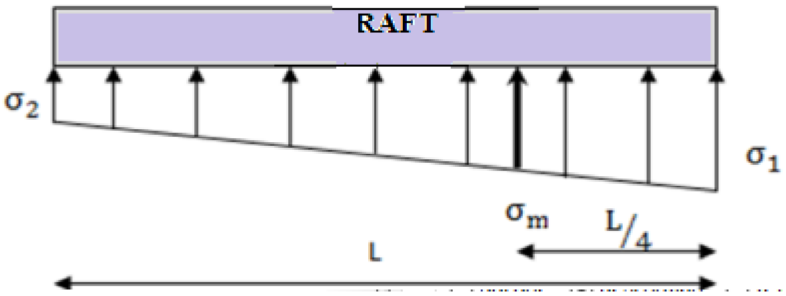


Figure 8.9: Distribution of soil stress in raft.

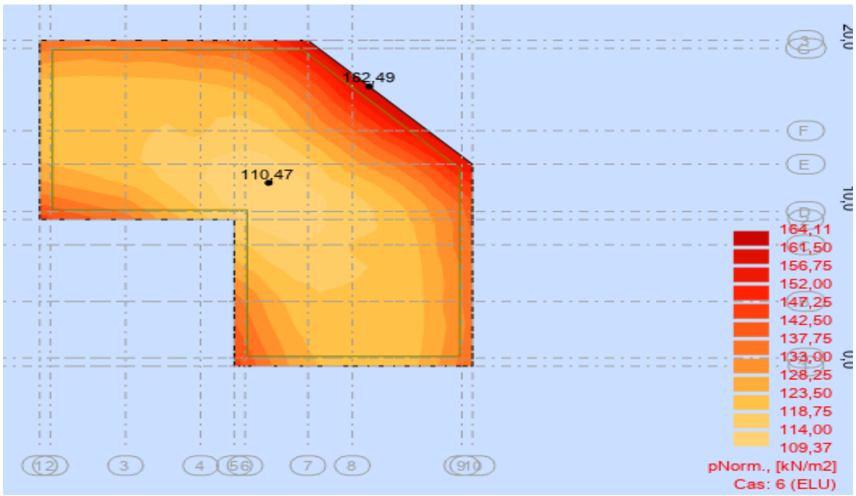


Figure 8.10: Soil stress at ULS from robot software.

- ***Verification at the seismic combination G+Q+E:***

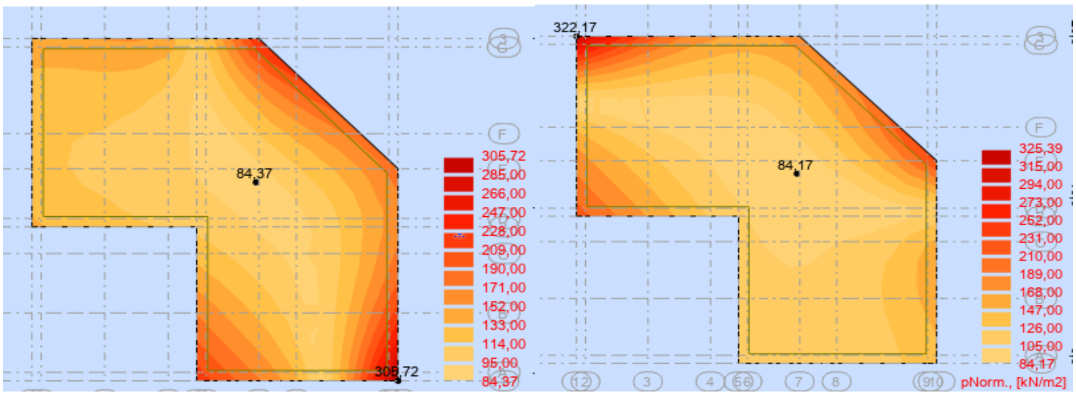
The following condition must be checked:

$$\boldsymbol{\sigma_m = \frac{3\sigma1+\sigma2}{4} \leq 1.5\times\sigma_{soil}}$$

$\sigma_{1x} = 305.72$ kN/m^2 ; $\sigma_{2x} = 84.37$kN/m^2 ; $\sigma_{1y} = 325.39$ kN/m^2 ; $\sigma_{2y} = 84.17$kN/m^2

$\sigma_{my} = \frac{3\times325.39+84.17}{4} = 2.65$ bar $\leq \sigma_{soil} = 3$ bar → ***Condition verified***

σ_{mx} = 2.5 bar ≤ σ_{soil} = 3 bar → ***Condition verified***



G+Q+Ex **G+Q+Ey**

Figure 8.11: Soil stress at G+Q+$E_{x,y}$ combination from robot software.

- ***Verification at the seismic combination 0.8G±E:***

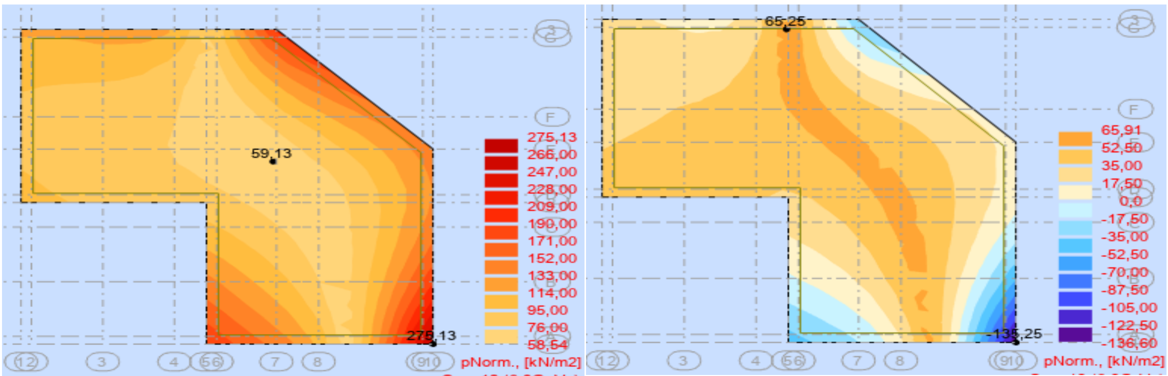
➢ ***Direction x: 0.8G±Ex***

0.8G+Ex: σ_{1+} = 275.13 kN/m^2 ; σ_{2+} = 59.13kN/m^2 → Distribution of the stress trapezoidal

σ_m = 2.21 bar ≤ σ_{soil}= 3 bar → Condition ***verified***

0.8G-Ex: σ_{1-} = 66.91 kN/m^2 ; σ_{2-} = -136.60 kN/m^2 → Distribution of the stress triangular

$\sigma_m = \frac{3\times66.91}{4}$ = 0.5 bar ≤ σ_{soil} = 3 bar → ***Condition verified***



0.8G+Ex **0.8G-Ex**

Figure 8.12: Soil stress at 0.8G±E_x combination from robot software.

➢ ***Direction y: 0.8G±Ey***

0.8G+Ey: σ_{1+} = 291.63 kN/m^2 ; σ_{2+} = 58.94kN/m^2 → Distribution of the stress trapezoidal.

σ_m = 2.33 bar ≤ σ_{soil} = 3 bar → ***Condition verified***

0.8G-E$_y$: $\sigma_{1-} = 66.05$ kN/m^2 ; $\sigma_{2-} = -151.96$ kN/m^2 ⟶ Distribution of the stress triangular

$\sigma_m = \frac{3\times 66.05}{4} = 0.495$ bar $\leq \sigma_{soil} = 3$ bar ⟶ ***Condition verified***

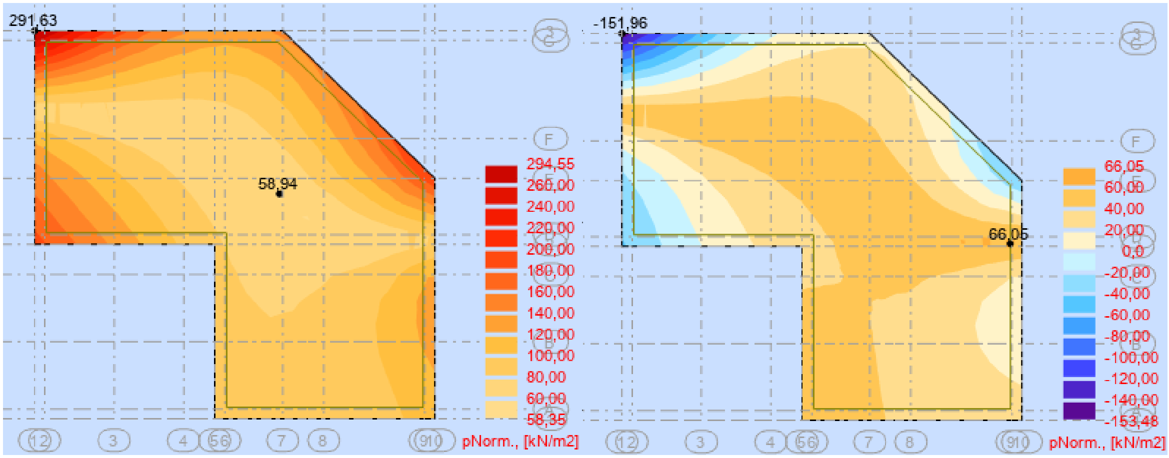


Figure 8.13: Soil stress at 0.8G±E$_y$ combination from robot software.

All conditions of stress are verified in both axes, so there isn't a heaving

8.7.3 Verification of shear stress:

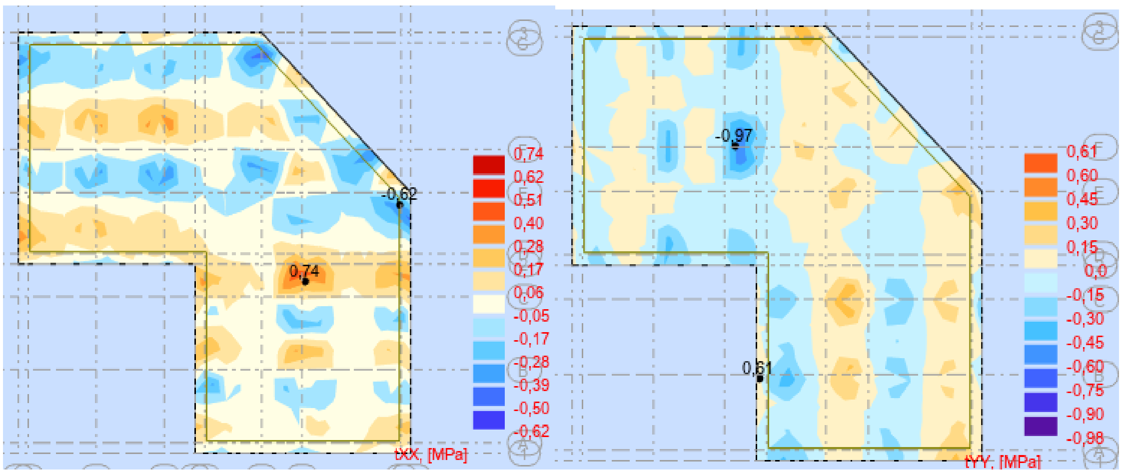
The following condition must be checked:

$$\tau_u < 0.05 f_{c28} = 0.05\times 25 = 1.25\text{MPa}$$

The shear stress of raft determined from robot software at ULS:

Direction x: $\tau_u^{max} = 0.74$MPa $\boldsymbol{\tau_u^{max} = 0.74}$**MPa < 1.25MPa** ⟶ ***Condition verified***

Direction y: $\tau_u^{max} = 0.97$MPa $\boldsymbol{\tau_u^{max} = 0.97}$**MPa < 1.25MPa** ⟶ ***Condition verified***



Direction x **Direction y**

Figure 8.14: Shear stress of raft at ULS in both directions *x* and *y*.

8.7.4 Verification of stability:

According to RPA99/2003, whatever the type of foundation, it is necessary to verify that the eccentricity of the resultant of gravitational vertical forces and seismic forces remains inside the middle half of the base of the foundation elements. $e = \frac{M}{N} \leq \frac{B}{4}$

e: Highest value of eccentricity due to seismic loads.

M: Reversing moment of seismic force.

N: Normal effect of structure by the combination G+0.2Q.

B: Width of raft.

The results are in the following table:

Table 8.2: Verification of stability of reversing in direction x and y

	N(kN)	M(kN.m)	B(m)	e(m)	B/4	Condition
Direction x	21626,04	25420,97	9.07	1.175	2.27	V
Direction y	21626,04	24414,70	9.07	1.129	2.27	V

The conditions are verified; therefore, the building is stable.

8.7.5 Reinforcement of raft:

➢ ***The slab:***

The calculation will be done for 1 linear meter in the most stressed combination of: ULS, SLS, ACC, under damaging cracking.

- ***Determination of moments:***

The moment at span and at the support of the raft should be determined as following:

$L_x/L_y=1$ the slab work in both directions.

$M'_x=\mu_x \times q \times L_x^2$;

$M'_y=\mu_y \times M'_x$;

At ULS: $q_u=\frac{Nu}{Sr} \times b$;

At SLS: $q_s=\frac{Ns}{Sr} \times b$;

At ACC: $q_a=\frac{Na}{Sr}\times b$

b: band of one meter; S_r: area of the raft;

N_u: normal effect from robot at ULS;

N_s: normal effect from robot at SLS;

N_a: normal effect from robot at the seismic combination ACC.

Continuous panel:

At span: $M_x=0.75M'_{x;}$ $M_y=0.75M'_y$

At support: $M_x= -0.5M'_{x;}$ $M_y= -0.5M'_y$

Edge panel:

At span: $M_x=0.85M'_{x;}$ $M_y=0.85\ M'_y$

The extreme edge: $M_x=-0.3M'_{x;}$ $M_y=-0.3M'_y$

The continuous edge: $M_x=-0.5M'_{x;}$ $M_y=-0.5\ M'_y$

The values of moment are obtained from robot software as the following table:

Table 8.3: Maximum values of moment of slab in ULS SLS ACC combinations

combination	ULS		SLS		ACC	
	At span	At the support	At span	At the support	At span	At the support
M_x(kN.m)	132.85	-198.48	96.59	-144	131.06	-82.93
M_y(kN.m)	126.81	-193.58	92.14	-140.44	120.04	-74.91

- ***Calculation of A_S and choice of bars:***

The calculation of reinforcement is done considering the maximum moment at span and at the support. According to table 8.3, in all combinations ULS, SLS and ACC.

The design flowchart in figure 8.15 can be followed to determine the section of bars:

b = 1m; h = 0.4m; f_{c28} = 25MPa; f_{t28} = 2.1 MPa; fe = 400MPa;

f_{bc} = 14.17MPa (durable situation); f_{bc}=18.48MPa (acc.situation);

d = 0.36m; d' = 0.04 m ; γ_b = 1.5; γ_s = 1.15; μ_R = 0.391; σ_s(ULS) = 348MPa;

σ_{st}(SLS)= min$\{\frac{2}{3}Fe$; max $(0.5Fe; 110\sqrt{\eta . Ftj}\}$=201.63MPa.

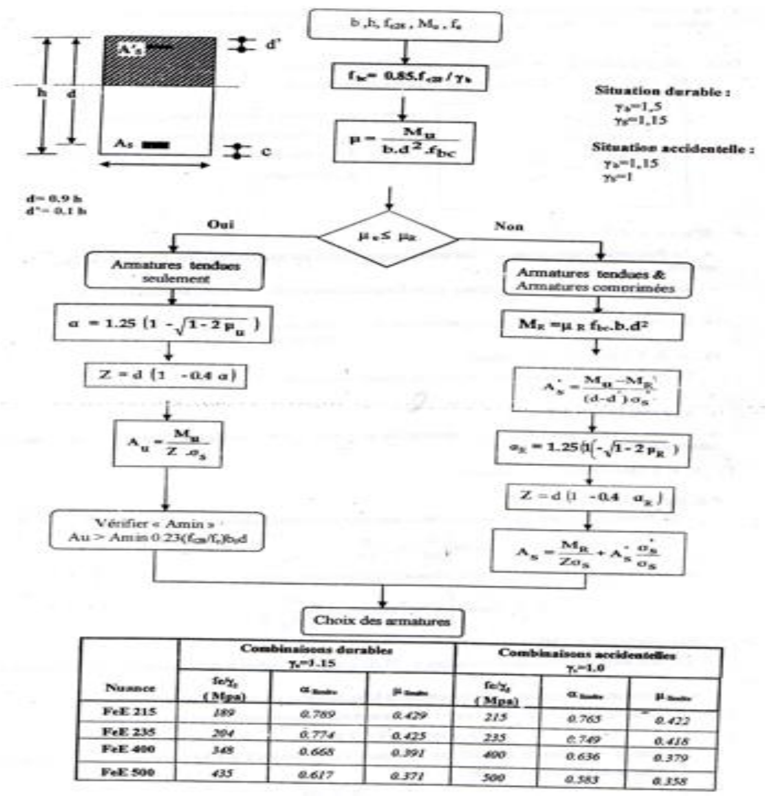


Figure 8.15: Design flowchart of rectangular section under simple bending at ULS.

Condition of non-fragility: A_s^{min} =0. 23b.d f_{t28}/f_e

A_s^{min} =4.347 cm^2 is verified according to table 8.4

Spacing: S_t<h; S_t = min (20cm; 15φ_l) = 20 cm; → ***St = 20 cm***.

Table 8.4: Results of calculation of reinforcement slab in ULS, SLS and ACC

		M(kN.m)	μ_u	$\mu_u \leq \mu_R$	α	Z(mm)	A(cm^2)	A>A_{min}	Bars choice	As(cm^2)	S_t(cm)
ULS	At span	132.85	0.072	CV(A’s=0)	0.094	346.464	11.02	**CV**	9HA16	18.1	20
	At support	-198.48	0.108	CV(A’s=0)	0.143	339.41	16.8	**CV**	6HA20	18.85	20
SLS	At span	96.59	0.052	CV(A’s=0)	0.066	350.496	13.67	**CV**	6HA20	18.85	20
	At support	-144	0.078	CV(A’s=0)	0.102	345.312	20.68	**CV**	5HA25	24.54	20
ACC	At span	131.06	0.055	CV(A’s=0)	0.071	349.776	10.77	**CV**	6HA16	12.06	20
	At support	-82.93	0.034	CV(A’s=0)	0.043	353.808	6.74	**CV**	4HA16	8.04	20

The most unfavorable combination is the SLS combination, therefore the reinforcement bars of slab to be adopted are according to SLS.

- ***Distribution bars:***

At span: $4.7125cm^2 \leq A'_s \leq 9.425cm^2 \rightarrow$ **6HA14 of A'_s=9.24cm²**

At support: $6.135\ cm^2 \leq A'_s \leq 12.27\ cm^2 \rightarrow$ **5HA16 of A'_s=10.05 cm²**

Table 8.5: Results of reinforcement slab in SLS

Combination		M(kN.m)	Bars	$A_s(cm^2)$	Dist.bars	$A'_s(cm^2)$	S(cm)
SLS	At span	96.59	6HA20	18.85	6HA14	9.24	20
	At the support	-144	5HA25	24.54	5HA16	10.05	20

- ***Verification of stress at SLS:***

According to table 8.3 the maximum moment at SLS in direction x:

At span: M_s = 96.59 kN.m

At the support: M_s = -144 kN.m

The design flowchart in figure 8.16 can be followed to verified the section bars at SLS combination: σ_{bc}=15 MPa; damaging cracking; η = 1.6; $\overline{\sigma}_{st}$=201.63MPa; n=15.

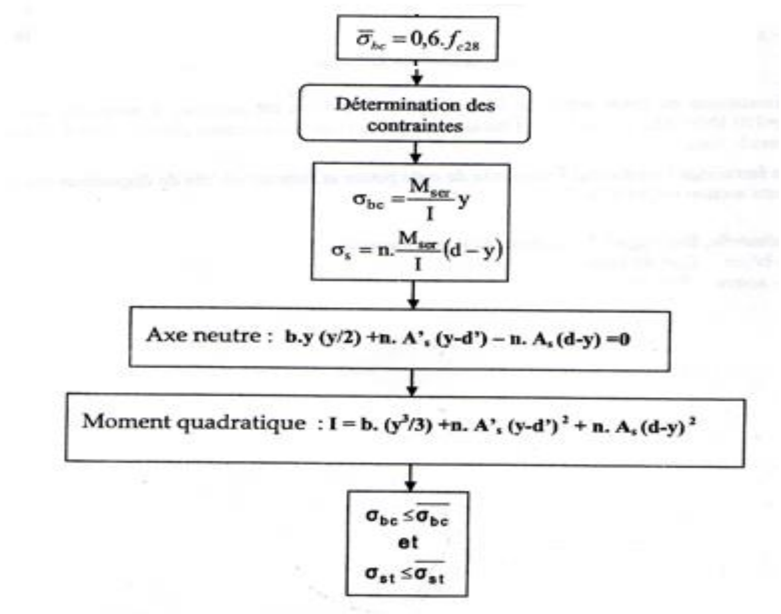


Figure 8.16: Design flowchart of rectangular section in simple flexion at SLS.

Table 8.6: Results of verification of stress at SLS

	M_s(kN.m)	Y(cm)	I(cm^4)	σ_{bc}(MPa)	σ_{st}(MPa)	$\sigma_{bc} \leq \bar{\sigma}_{bc}$	$\sigma_{st} \lesssim \bar{\sigma}_{st}$
At span	96.59	11.03	22.79×10^4	4.67	158.74	CV	CV
At support	-144	12.24	53.52×10^4	3.29	95.89	CV	CV

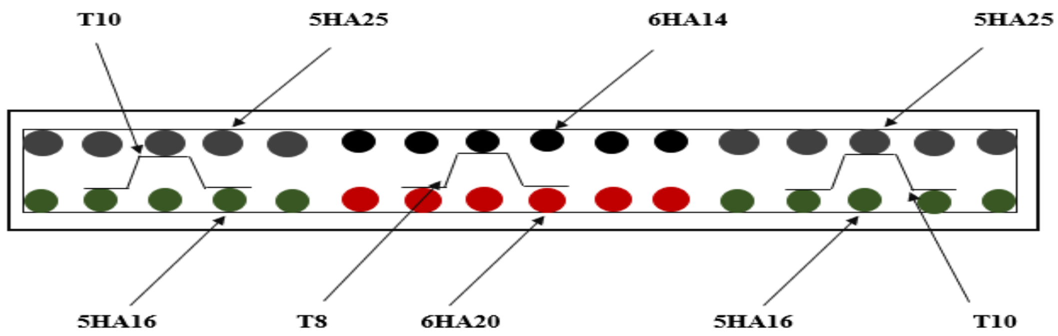


Figure 8.17: Reinforcement of the slab at ***span*** and ***support*** level in direction x and y.

- ***The ribs:***

To determine the moment in the span and in the support of the ribs, it must calculates the percentage of the distribution of loads to find the load in each rib.

At span: $M_{span} = 0.85M_{0;}$

At support: $M_{sup} = 0.5M_0$ such as: $M_0 = \frac{q.L^2}{8}$ and $q = \frac{N}{L}$

From robot the most stressed ribs are:

Direction x: ribbed in file F which has a length of 14.07 meters.

Direction y: ribbed in file 6 which has a length of 14.07 meters.

- ***Moment of ribbed at ULS:***

The following figures represent the maximum moment of ribbed in both direction x and y

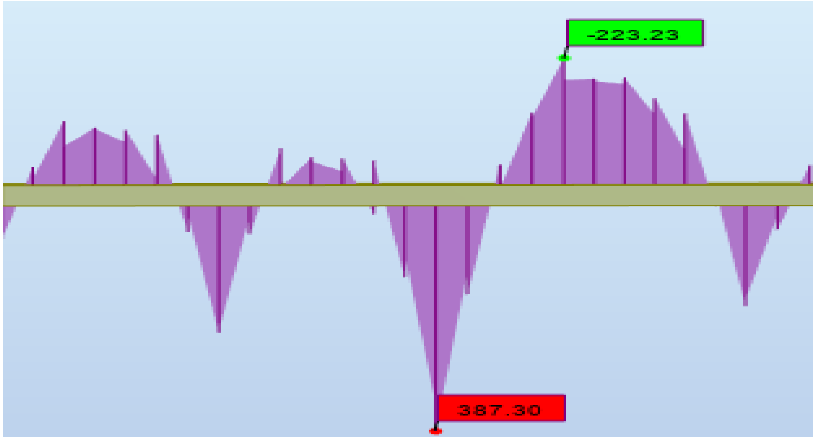


Figure 8.18: Moment diagram of most stressed ribbed at ULS in direction x.

Direction x:

At span: M_{span} = -223.23 kN.m;

At support: M_{sup}=387.3 kN.m

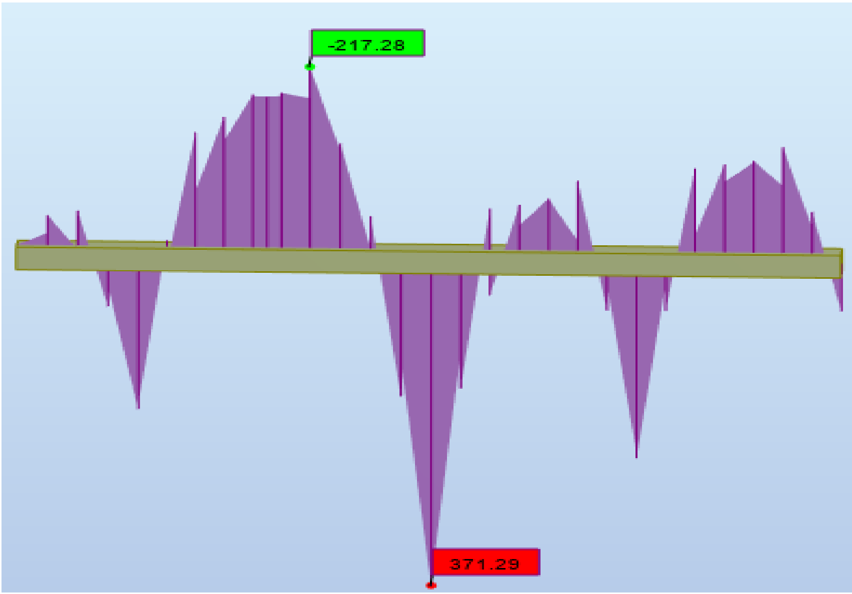


Figure 8.19: Moment diagram of most stressed ribbed at ULS in direction y.

Direction y:

At span: M_{span}= -217.28 kN.m;

At support: M_{sup}=371.29 kN.m

- ***Moment of ribbed at SLS:***

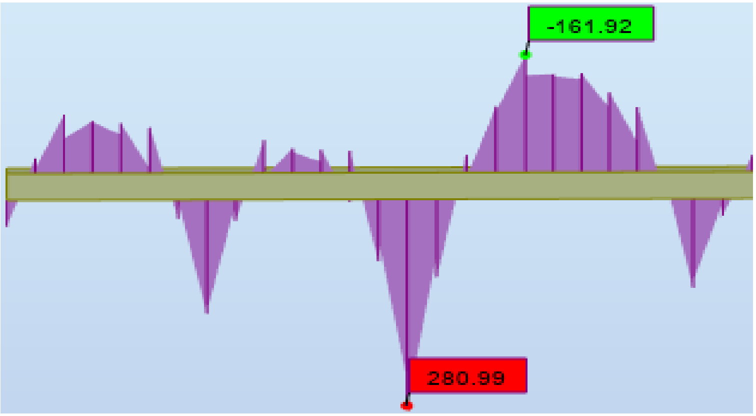


Figure 8.20: Moment diagram of most stressed ribbed at SLS in direction x.

Direction x:

At span: M_{span}= -161.92 kN.m;

At support: M_{sup}=280.99 kN.m

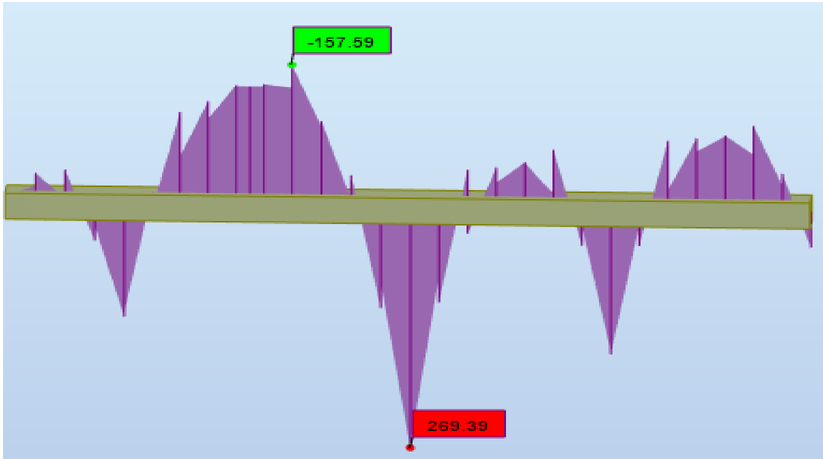


Figure 8.21: Diagram moment of most stressed ribbed at SLS in direction y.

Direction y:

At span: M_{span}= -157.59 kN.m;

At support: M_{sup}=269.39 kN.m

- ***Determination of bars (ULS):***

The same steps of reinforcement of slab will be follows to determine the reinforcement of the ribs, the results are in the following table:

Such as: b = 70 cm; h = 60 cm; d = 54 cm; d'= 6 cm;

C.N.F: $A_s^{min} = 0.23b.d\ f_{t28}/f_e = 0.23 \times 700 \times 540 \times 2.1/400 = 4.56cm^2$

Table 8.7: Determination of section bars at ULS and SLS in direction x and y

			M (kN.m)	µ	$A_s^{cal}(cm^2)$	Choice bars	$A_s(cm^2)$	Distb Bars	$A'_s(cm^2)$	C.N.F
ULS	Direc.x	At span	223.23	0.07	12.33	5HA20	15.71	5HA14	7.70	CV
		At supp	387.3	0.133	22.20	5HA25	24.54	5HA16	10.05	CV
	Direc.y	At span	217.28	0.075	12.03	5HA20	15.71	5HA14	7.70	CV
		At supp	371.29	0.128	21.22	5HA25	24.54	5HA16	10.05	CV
SLS	Direc.x	At span	161.92	0.056	15.31	6HA20	18.85	3HA20	9.42	CV
		At supp	280.99	0.097	27.20	6HA25	29.45	3HA20	9.42	CV
	Direc.y	At span	157.59	0.054	14.88	5HA20	15.71	5HA14	7.70	CV
		At supp	269.39	0.093	26.01	6HA25	29.45	3HA20	9.42	CV

The most unfavorable combination is the combination SLS, therefore the reinforcement bars to be adopted are according to SLS, the same reinforcement bars are adopted in both direction xx and yy.

Table 8.8: Results of reinforcement ribs in SLS

Combination		M(kN.m)	Bars	$A_s(cm^2)$	Dist.bars	$A'_s(cm^2)$
SLS	At span	161.92	6HA20	18.85	3HA20	9.42
	At support	280.99	6HA25	29.45	3HA20	9.42

Verification of spacing:

$e_{th} = \max\{1.5C_g; \varphi_l\} = 3.75cm$ $e_h = (70-2\times2-3\times2.5-3\times2)/4 = 13.125cm > 3.75\ cm \rightarrow$ **CV**

$e_{th} = \max\{1.5C_g; \varphi_l\} = 3.75cm$ $e_h = (70-2\times2-3\times2-3\times2)/4 = 13.5cm > 3.75cm \rightarrow$ **CV**

Transversal bars: $\varphi_t \geq \frac{1}{3}\varphi_l$ and $\varphi_t < 12mm$

- At span: $\varphi_t \geq \frac{1}{3}(20) = 6.67mm \rightarrow$ so: $\varphi_t = 8mm$
- At support: $\varphi_t \geq \frac{1}{3}(25) = 8.33mm \rightarrow$ so $\varphi_t = 10mm$

- ***Verification of section bars (SLS):***

The results of verification are in the following table:

$\overline{\sigma}_{bc}$=15 MPa; damaging cracking; η = 1.6; $\overline{\sigma}_{st}$=201.63 MPa; n=15

Table 8.9: Verification of section bars of the most stressed ribbed at SLS

	M_s (kN.m)	Y(cm)	I(cm^4)	σ_{bc}(MPa)	σ_{st}(MPa)	$\sigma_{bc} \leq \overline{\sigma}_{bc}$	$\sigma_{st} \leq \overline{\sigma}_{st}$
At span	161.92	16.24	79.69×10^4	3.29	115.085	CV	CV
at support	280.99	19.51	72.46×10^4	7.57	200.62	CV	CV

Skin reinforcements:

To determine the skin reinforcements, the following expression must be checked:

$$\mathbf{h > 2(80 - \frac{Fe}{10})}$$

$$h = 60cm < 2(80- \frac{400}{10}) = 80 \text{ cm} \rightarrow \textit{Condition not verified}$$

No need to use the skin reinforcements.

The figures below presents the reinforcement of the ribs at span and support:

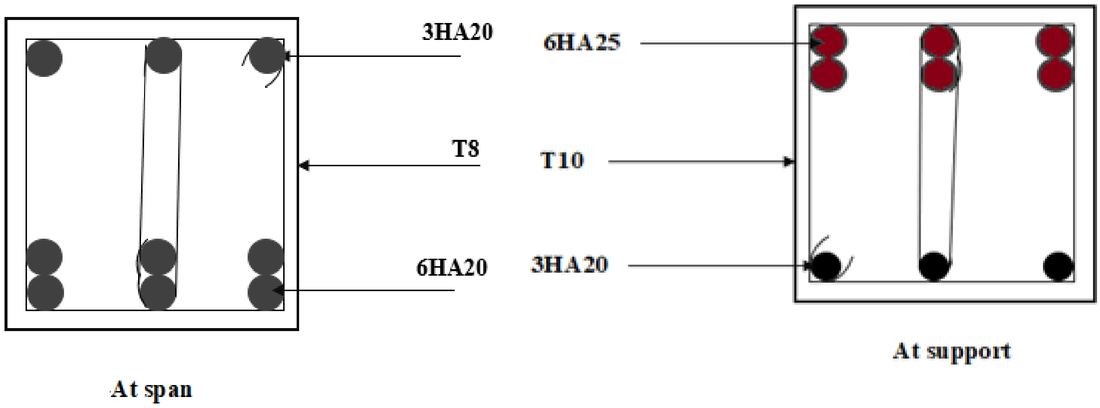


Figure 8.22: Reinforcement of the ribs at span and support level in both direction x and y.

8.8 CONCLUSION:

- ✓ The adopted foundation type is raft foundation, slab stiffened by ribs which is more economical.
- ✓ The reinforcement at SLS is more than the reinforcement at ULS.
- ✓ The modeling of the raft foundation is done using the Robot software.

GENERAL CONCLUSION

The study of this project is our first real test before entering the professional life. This thesis allowed us to realize and practice the theoretical work of engineers and specially to learn different techniques of calculation and to be trained to use different software and to follow the adopted design regulations and standards of buildings.

The purpose of this work is to design a steel residential building that must be both economical and stable, the use of Robot software 2020 allowed us to choose the right concept, the conclusions obtained in this work are summarized in the following points:

- The structure was designed and verified according to EUROCODE 3.
- The dynamic study in the seismic zone was carried out using the spectral method analysis according to the seismic regulation RPA99/2003.
- The calculation of the steel connections is done manually and by using the software Robot.
- The choice of foundation type depends on the type of the soil and the loads from the superstructure, the adopted type in this project is raft foundation.

One of the encountered problems in this work was during the dynamic analysis of the structure. Many of the required conditions of modal analysis were not verified due to the irregularity of the building where the positioning of the bracing system was not simple. Several layouts have been tested until the appropriate one were selected to ensure the stability of the building.

Finally, we can say that this work has enabled us to apply our knowledge of civil engineering, and to develop it, which will help us later in professional life.

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- **Reference 07:** Eurocode 4, Design of composite steel and concrete structures.
- **Reference 08:** D.T.R 2.2, Dead loads and live loads.
- **Reference 09:** Steel frame course (4th year) of Mr. MENNADI.