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A THESIS FOR OBTAINING
THE MASTER DEGREE
OF STRUCTURAL ENGINEERING

Design and analysis of a multi-storey building (Basement+ Ground level +07 Floors) for residential use.

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"BE GRATEFUL TO ALLAH" AND WHOEVER (IS) GRATEFUL THEN ONLY HE IS GRATEFUL FOR HIMSELF. AND WHOEVER (IS) UNGRATEFUL, THEN INDEED, ALLAH (IS) SELF-SUFFICIENT AND GLORIOUS. LUKMAN-12

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Dedication

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Kawthar

ملخص

هذا المشروع يهدف إلى دراسة بناية من الخرسانة المسلحة **(**طابق تحت الأرض+طابق ارضي+سبعة طوابق) ذات استعمال سكني والمقرر بناؤها في ولاية الجزائر المصنفة ضمن المناطق ذات زلزال قوي **(**منطقة **III).** مقاومة البناية لكل الحمولات الأفقية محققة بواسطة نظام الجدران المسلحة.

قياس الأبعاد والتسليح بالفولاذ لكل العناصر المقاومة تم طبقا للمعايير و القوانين المعمولة بها في الجزائر: قانون الخرسانة المسلحة **(CBA93)** والقواعد الجزائرية المضادة للزلزال **(RPA99 version 2003)**.

تمت الدراسة الزلزالية وفق الطريقة الرقمية باستعمال برنامج تحليل الهياكل **(SAP2000 V21.0.2)**

وفي الأخير درسنا البنية التحتية للبناية بحساب الأساسات.

ABSTRACT

This project deals with the study of a building with (basement + ground level +07 stories) for a residential use. It is located in the state of Algiers**,** which is classified as a severe intensity seismic zone **(zone III)** according to the Algerian seismic standard code **RPA99 version2003.**

The resistance of structure to horizontal loads is ensured by a frame system with shear walls.

The measurements and the design of reinforcement of all elements were according with the Algerian standards (**CBA93 and RPA99 version 2003**)**.**

The seismic analysis has been realized with the numerical method based on finite elementsmethod, using the program **(SAP2000 V 20).**

Finally, we have studied the infrastructure by calculating the foundations.

RESUMÉ

Ce projet consiste à étudier un bâtiment en: sous-sol + rez-de-chaussée + 07 étages, à usage d'habitation. Il est implanté à Alger. Cette région est classée en zone de forte séismicité **(zone III)** selon le **RPA99 version 2003**.

La résistance de la structure aux charges horizontales est assurée par un système de contreventement mixte.

Le dimensionnement de tous les éléments résistants sont établis conformément aux règlements Algériens en vigueur (**CBA93 et RPA99 version2003**).

L'analyse sismique de la structure a été menée par la méthode numérique basée sur la méthode des éléments finis moyennant le logiciel (**SAP2000V21.0.2)**

Finalement nous avons étudié l'infrastructure en calculant les fondations.

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NOTATION

ROMANS CAPITAL LETTERS

A : tensile steel section.
A' : Compressed steel section.
$Amax, Amin$: Maximum and minimum steel section.
At : Section of a transverse reinforcement course.
Av : Vertical reinforcement section.
Ah : Horizontal reinforcement section.
Ac : Reinforcement section in the current part (skin reinforcements).
AD : Diagonal reinforcement section.
B : Concrete area.
Br : Reduced section of column concrete.
Bt : Section of the tensile part.
Lt : Width of the tensile section.
S : Service Limit State.
ULS : Ultimate Limit State.
Ev : Modulus of elasticity (deferred).
Ei : Modulus of elasticity (Instantaneous).
Es : Modulus of elasticity longitudinal.
I : Moment of inertia.
Ie : Equivalent moment of inertia.
M : Bending moment.
Mt : Bending moment in span of a beam.
Ma : Bending moment in support of a beam.
Mu : Bending moment at the ULS.
$Mser$: Bending moment at the SLS.
N : Normal effort.
$Nser$: Normal effort at SLS.
Nu : Normal effort at ULS.
Tu : shear force at the ULS.
G : dead load.
Q : live load.
E : Seismic load.
W : weight.

ROMANS SMALL LETTERS

h : Concrete section height.
b : Concrete section width.
b0 : Width of the concrete rib (T-section)
$c,'$: Upper and lower coating.
d : Useful height of a section.

f : Arrow.
fcj : Characteristic compressive strength of concrete at "d" days.
$fc28$: Characteristic compressive strength of concrete at "28" days.
fe : Yield strength of steel.
ftj : Conventional tensile strength of concrete at "d" days.
$ft28$: Conventional concrete tensile strength at "28" days.
i : Radius of gyration.
lf : Buckling length,
St : Transversal reinforcement spacing.
Sh : Horizontal reinforcement spacing.
Sv : Vertical reinforcement spacing.

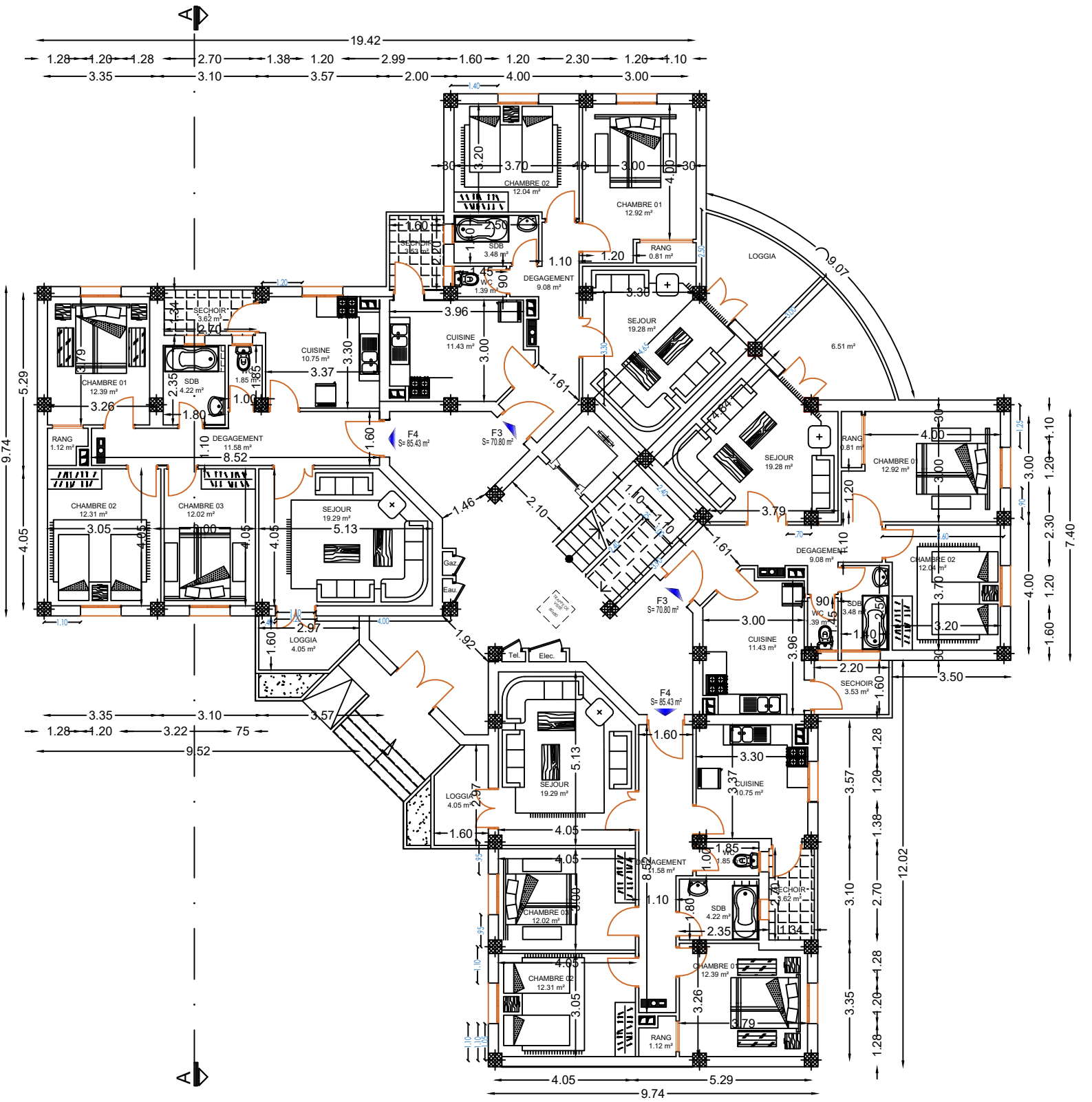
GREEK CAPITAL LETTERS

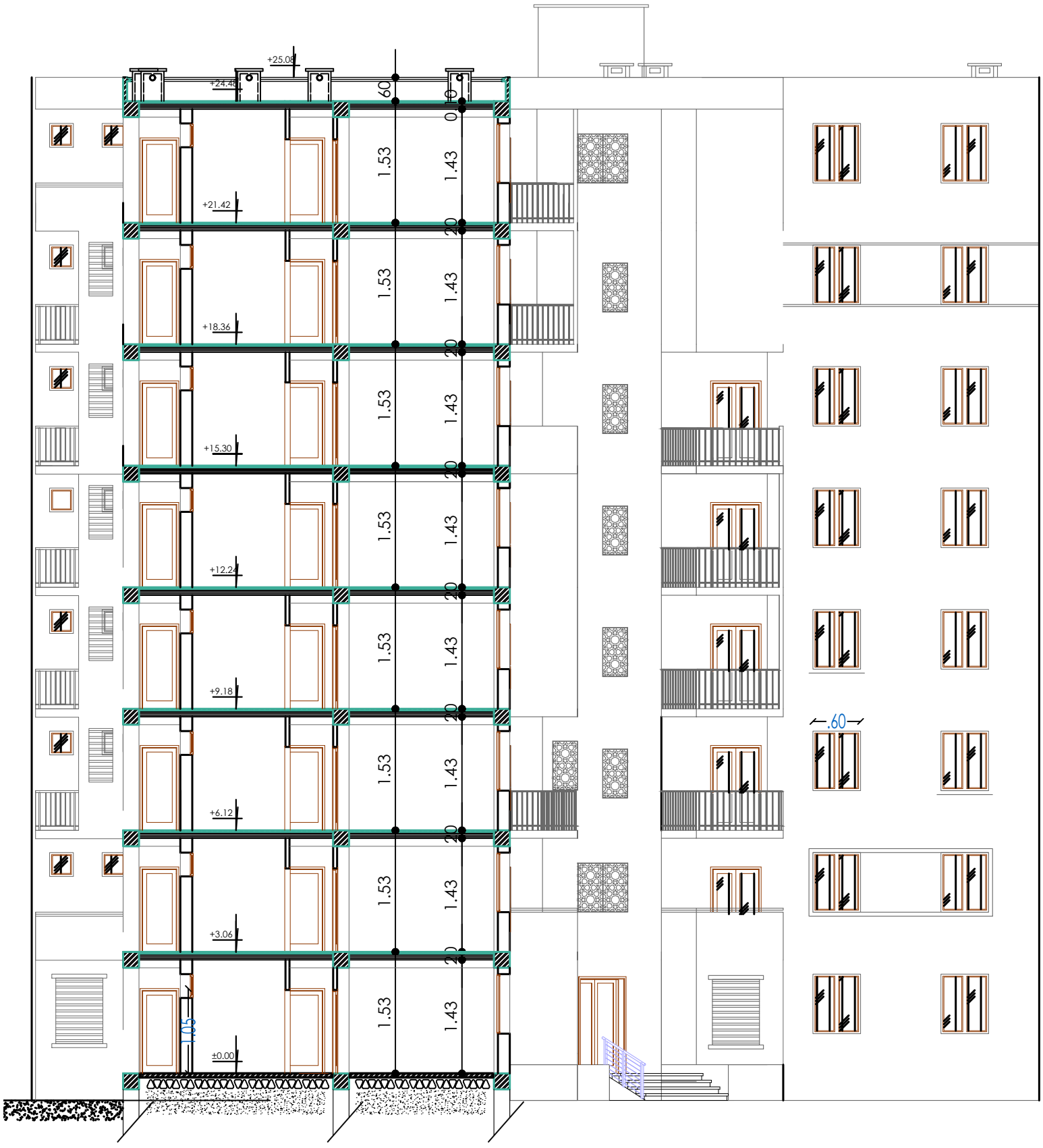
Δ : Variation.
Σ : sum.
Φ : Nominal diameter of a steel bar.

GREEK SMALL LETTERS

α : Angle.
γ : Safety coefficient.
ε : Relative deformation.
η : Cracking coefficient.
λ : Mechanical slenderness.
μ : Coefficient of friction.
ν: Poisson coefficient.
ρ : Two-dimensional ratio
ζ : Normal stress.
η : Tangential stress.
δ : displacement of the level.




A
19.42
1.28 1.20 1.28 2.70 1.38 1.20 2.99 1.60 1.20 2.30 1.20 1.10
3.35 3.10 3.57 2.00 4.00 3.00
CHAMBRE 02
12.04 m²
CHAMBRE 01
12.92 m²
SDB
3.48 m²
SECHOIR
3.53 m²
DEGAGEMENT
9.08 m²
RANG
0.81 m²
LOGGIA
6.51 m²
SEJOUR
19.28 m²
CUISINE
11.43 m²
WC
1.39 m²
F3
S= 70.80 m²
F4
S= 85.43 m²
CHAMBRE 01
12.39 m²
SECHOIR
3.62 m²
CUISINE
10.75 m²
SDB
4.22 m²
WC
1.85 m²
RANG
1.12 m²
DEGAGEMENT
11.58 m²
CHAMBRE 02
12.31 m²
CHAMBRE 03
12.02 m²
SEJOUR
19.29 m²
LOGGIA
4.05 m²
Gaz
Eau
Tel.
Elec.
9.74
5.29
4.05
8.52
5.13
2.97
3.96
3.00
3.30
3.37
3.26
3.79
2.35
1.80
1.00
1.61
1.46
2.10
1.92
1.60
1.10
1.20
1.45
0.90
2.50
3.70
3.20
4.00
3.00
1.40
4.64
9.07
3.50
2.20
2.30
1.28
1.38
2.70
3.10
3.57
12.02
3.35
4.20
7.40
9.52
3.22
75
A


+25.08
+24.48
+21.42
+18.36
+15.30
+12.24
+9.18
+6.12
+3.06
±0.00
1.53
1.43
.60
1.05



INTRODUCTION

General introduction

The end of studies project represents the last phase of our cycle at Medea University, throw it we apply all the knowledge acquired all along our journey in the field of Civil Engineering. It is about a subject allowing to associate dimensioning and calculation of execution with the conditions of Algerian reglementation in order to have a better price-quality ratio.

The resistance of the structure to the horizontal actions is principally ensured by the bracing system of our structure (shear walls and columns).

This document is a summary of the work carried out. It is detail the calculation procedures usedwith the references on which we have relied on.

We will follow successively the following steps:

1) Presentation of the structure and the characteristics of the materials;
2) Pre-sizing of the structural elements;
3) Calculation of secondary elements (parapet, floor, staircase, solid slab, machine slab);
4) Dynamic and seismic Analysis, we used the finite element calculation software (SAP2000 version 22 and Etabs 19);
5) Studying the structure under the action of different combinations of vertical and horizontal forces andthe adoption of the reinforcement of the structure;
6) Estimation of the considered stresses at the level of the infrastructure, choice of the most suitable type of Foundation, calculation and reinforcement of all the elements to ensure the stability of the superstructure;
7) Summary of the work carried out summarized in the general conclusion.

CHAPTER I

Project presentation

I.1 Project description

Our project is a multi-story concrete building (Basement +ground floor + 07 floors) for a residential use. It will be located in Algiers which is a severe intensity seismic zone (Zone III) according to the classification of RPA 99 version 2003.

Our research will be conducted in accordance with the design and calculations rules of reinforced concrete structures CBA93 and the requirements of RPA99 2003 earthquake regulations and DTR B.C2.2 permanent loads and service loads (regulations /codes).



Figure I. 1. 3D image for our structure.

I.2 Geometric characteristics of the project

I.2.1 Dimensions in plan of the building

- Total length of the building : Lx=28.40m.
- Total width of the building : Ly=28.40m.

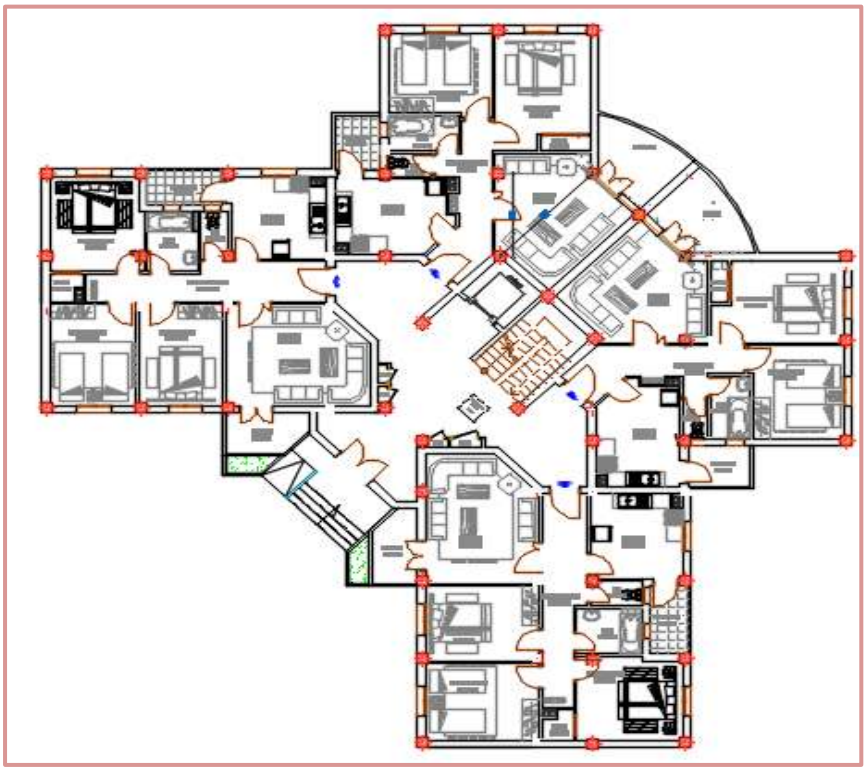


Figure I. 2. Plan of the interior design.

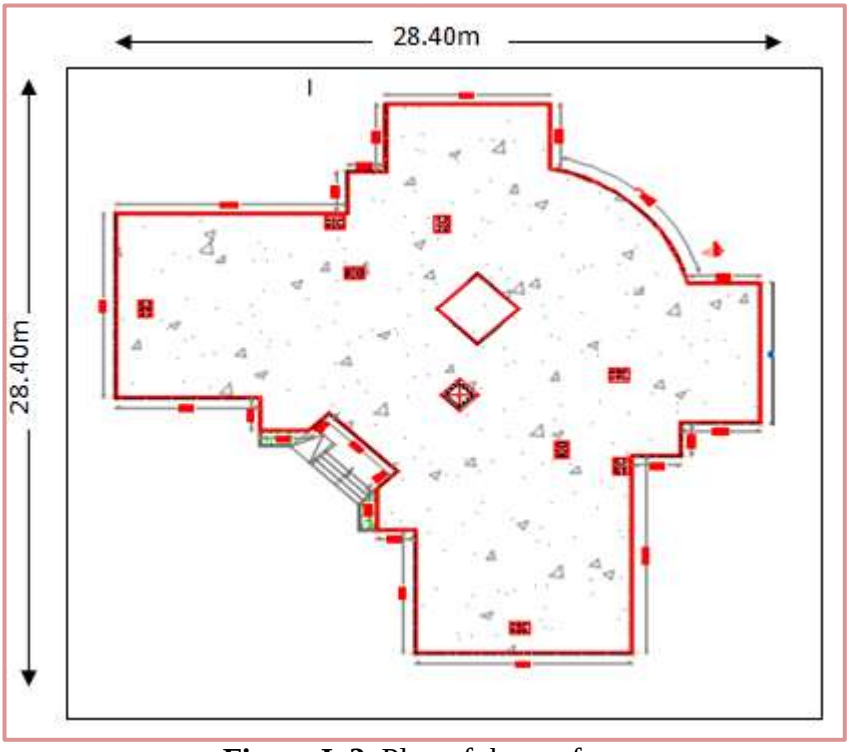


Figure I. 3. Plan of the roof.

I.2.2 Dimensions in elevation of the building

- Height of the basement, the ground and the typical floors : 3.06m .
- Total height of the building : 24.48m (without the basement)



Figure I. 4. Section view and the main façade plan.

I.3 Structural characteristics

I.3.1 Bracing system

Our building has eight floors with height of 24.48m, the RPA99 Version 2003 limits self-stable portal structures in (zone III) to 08m, this condition leads us to insert supporting sails so we will consequently obtain a bracing system provided by shear walls to control the impact of earthquake force and wind load.

I.3.2 Floor

We used two types of floors, their function is to support the vertical loads and transmit it to the beams which transmit it as well to the vertical elements of the structure.

- Hollow block/ coffer or pot floor slab.
- Flat slab.

I.3.3 Masonry

The masonry of the building will be executed by hollow brick, and we have two types of walls:

- External walls (Hollow wall/double partition): two hollow brick partitions with 10 cm of thickness separated by 10 cm air space to provide thermal insulation between the exterior and interior of the building.
- Internal walls (single partition): these are walls for interior dividing, they are 10 cm thick.

I.3.4 Acroterion

At the level of the roof-terrace, the structure is surrounded by acroterion designed with reinforced concrete, 60 cm high and 10 cm thick.

I.3.5 Staircase

In our project, vertical circulation between floors is provided by straight two-flight stairs.

I.3.6 Elevator

An Elevator is a mobile or semi-mobile device that ensures the movement of people and objects in height on defined levels of a construction.

I.3.7 Infrastructure

It is the structure-soil interaction part, which forms a rigid assembly ensuring the embedding of the structure in the ground as well as the transmission of the loads brought by the structure to the good soil.

I.4 Mechanical characteristics of materials

The mechanical characteristics of the materials used in construction will comply with the technicalrules for the design and calculation of reinforced concrete structures (CBA 93).

I.4.1 Concrete

We use a concrete that has a cement dosage CPJ-CEMII 42,5 of 350Kg/m³ which gives us a compressive strength at 28 days equal to a minimum $f_{c28} = 25\ MPa$.

➢ **Concrete tensile strength**

The tensile strength is noted by « f_{tu} », it is defined by the relation:

$f_{tu} = 0.6 + 0.06 f_{cu}$

$f_{c28} = 25MPa$ so: $f_{t28} = 2.1MPa$

Table I. 1. Other characteristics of concrete.

Modulus of elasticity (Instantaneous) E_{ij}		32164.20 MPa
Modulus of elasticity (deferred) E_{vj}		10818.86 MPa
Poisson's ratio (υ)	**ULS (Ultimate limit state)**	0
	SLS (serviceability limit state)	0.2
Shear stress (r_u)	**prejudicial cracking or very prejudicial cracking**	2.50 MPa
	Little harmful cracking	3.33 MPa
compressive stress (σ_{bc})	**Durable situation**	14.20 MPa
	Accidental situation	18.50 MPa

I.4.2 Steel

- **Types of steel**

According to the external surface which will be in contact with the concrete, we have two types of steel: smooth surface steel and a rough surface steel, each one has its own utility.

Table I. 2. The types of steels used and their grades.

Type of steel	Grades	Elastic limit (MPa)	Use
Plain round bar	F_eE220	215	Frames, stirrupsfor columns and beams, prefabricated parts
	F_eE240	235	
deformed or ribbed (steel) bar	F_eE400	400	All reinforcedconcrete work
	F_eE500	500	
Welded mesh	$TLE520$	520	Application of : Raft /Mat Foundation , shearwall , Floors , flat slab
	$TLE500$	500	

Table I. 3. Other characteristics of Steel.

Modulus of elasticity longitudinal (E_s)				200000 MPa
Ultimate stress (σ_s)	**ULS**	**Durable situation σ_s**		348 MPa
		Accidental situation σ_s		400 MPa
	SLS	**Prejudicial cracking**	**Deformed bar $\overline{\sigma_s}$**	201.63 MPa
			Welded mesh $\overline{\sigma_s}$	159.40 MPa
		very prejudicial cracking	**Deformed bar $\overline{\sigma_s}$**	165 MPa
			Welded mesh $\overline{\sigma_s}$	130.42 MPa
		Little harmful cracking		No verification to do

CHAPTER II

Pre-design of structural elements

II.1 Introduction

In this chapter we will do the pre-sizing of the elements of our project; it is to calculate predicted dimensions of the frame and area elements such as floors, flat slabs, beams, columns and shear walls according to some conditions and rules extracted from DTR documents. These dimensions and sections can be changed in the following chapters or can be adopted.

II.2 Pre-design of the floors

Floors are horizontal elements that separate the stories of a building, their role is to:

- support the vertical loads in order to transmit it to the supporting elements;
- ensure the transmission of horizontal loads to the bracing elements (Rigid diaphragm);
- ensure the acoustic insulation and fire safety.

II.2.1 Hollow pot floor

Hollow pot floors consist of prefabricated slab beams associated with hollow blocks, the thickness is calculated using the following formula:

$$\frac{L}{25} \leq h_t \leq \frac{L}{20}$$ **[1] ART B.6.8.4.2.4**

h: Floor Thickness

L: the longest span of the slab beams

In our project we have L=4.00m so:

$$\frac{400}{25} \leq h_t \leq \frac{400}{20} \quad \rightarrow \quad 16 \leq h_t \leq 20$$

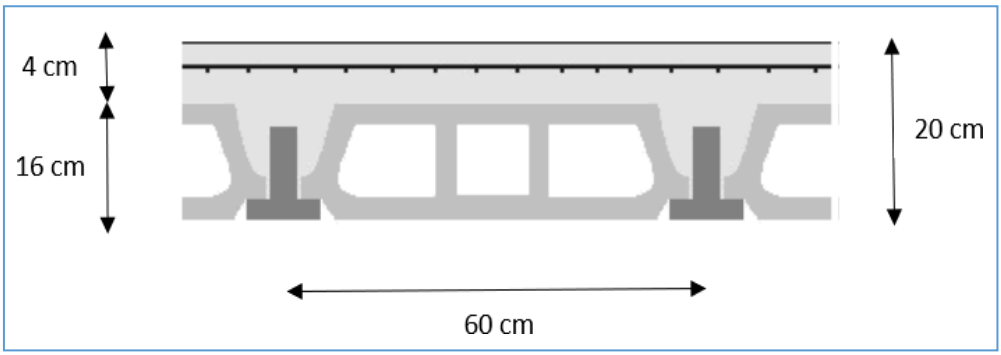
We adopt: h_t = 20 cm

h_t = d +e = (16+4) cm

With:

d: the thickness of the hollow block;

e: the thickness of the compression slab.



II.2.2 Solid (flat) slab floors

The thickness of the flat slab must check the resistance condition according to the nature of the supports on which the slab rests, we have a slab that is fixed on three supports.

- **Strength criterion**

$$\frac{L}{45} \leq e \leq \frac{L}{40}$$

L: the longest span.
e: slab thickness.
We have L=2.50m

$$\frac{250}{45} \leq e \leq \frac{250}{40} \quad \rightarrow \quad 5.55\mathbf{cm} \leq \mathbf{e} \leq \mathbf{6.25cm}$$

- **Fire safety and insulation**

e = 7 cm: provide a one-hour fire break.
e = 11cm: provide a two-hour fire break.
e = 15cm: Acoustic insulation condition.
The CBA requires 15 cm as a minimum thickness [**1] ART A.5.2.3.**
So we adopt: e = 15 cm.

II.3 Pre-design of Beams

The beams are linear elements its section is rectangular or in T section, their spans are much greater than the dimensions of the section. The horizontal structural elements (the beams) are responsible for the transmission of vertical and horizontal loads to the vertical elements (columns and shear walls). They are dimensioned according to the conditions of resistance and arrow.

$$\frac{L}{15} \leq h \leq \frac{L}{10}$$

$$0.27*h \leq b \leq 0.36*h$$

L: the length of the beam having the greatest span measured on the supporting node;
h: the height of the beam;
b: the width of the beam.

Table II. 1. Pre-sizing of the beams

	Main beams	Secondary beams
Strength Conditions	L= 605 cm 40.33 cm ≤ h ≤ 60.5cm We adopt : h = 45 cm ⇒ 12.15 cm ≤ b ≤16.20 cm We adopt : b = 30 cm	L= 400 cm 26.66 cm ≤ h ≤40 cm We adopt : h = 35 cm ⇒9.45 cm ≤ b ≤ 12.6 cm We adopt : b = 30 cm
Verification according to RPA 99 (2003 version) Art.7.5	b = 30 cm ≥ 20 cm h = 45 cm ≥ 30cm h/b = 1.5≤ 4.0 **Conditions checked**	b = 30 cm ≥ 20 cm h = 35 cm ≥ 30cm h/b = 1.16 ≤ 4.0 **Conditions checked**

Conclusion:

- For main beams, we adopted a section of: (b x h)=(30 x 45)cm²
- For secondary beams, we adopted a section of: (b x h) = (30 x 35) cm² (we choose h=35 cm instead of h=30cm because our secondary beam supports a ring beam which will be as a point load).
- We have also a ring beam in the same direction of the main beams, we adopted for it a section of: (b x h) =(30x35) cm².

II.4 Pre-design of columns

The columns are vertical load-carrying elements in reinforced concrete, their role is to transmit vertical loads to the foundations. For the pre-sizing of columns, a calculation based on the bearing capacity is used and the calculation of vertical loads dead and live loads at the ultimate limit state (ULS). According to Article **B.8.11** of the **CBA93**, this load can be increased by 10% for intermediate columns close to the edge(corner) columns in the case of buildings with at least three spans. So, this charge becomes:

$$N_u^* = 1.1N_u$$

II.4.1 Pre-design steps [1] ART B.8.4.1

- Calculation of the bearing capacity $\bar{N}$ of the column;
- Choosing the most stressed/loaded column;
- Calculation of the area/surface taken by the column;
- Loads assessments/estimate;

- Determination of the dead and live loads returning to this column;
- Verification of increased ultimate axial load;
- Verification of the dimensions of the cross-section of the columns according to the conditions of the RPA 99 version 2003: **[Art 7.4.1].**

For rectangular or square columns, we should respect the following formula: **[1] ART B.8.4.1**

$$N_u \leq \bar{N} = \alpha\left(\frac{B_r\, f_{c28}}{0.9\gamma_b} + A\frac{f_e}{\gamma_s}\right)$$

A=Br/1000: area of compressed reinforcement taken into account in the calculation.

Br: the reduced section of column obtained by reducing its real section in one cm of thickness over its entire periphery.

$$\begin{cases} \alpha = \dfrac{0.85}{1 + 0.2\left(\frac{\lambda}{35}\right)^2} & Si \quad \lambda \leq 50 \\ \alpha = 0.3\left(\dfrac{50}{\lambda}\right)^2 & Si \quad 50 \leq \lambda \leq 70 \end{cases}$$

With: $\lambda = \frac{l_f}{i}$ and $l_f = 0.7\, l_0$

λ: mechanical slenderness;

i: radius of gyration (turning radius);

l_0: column length;

l_f: buckling length;

We have: $I = \frac{b\, h^3}{12}$ and b=h, so: $I = \frac{b^4}{12}$

$$i = \sqrt{\frac{I}{b^2}} = \sqrt{\left(\frac{b^4}{12}\right)/b^2} = \frac{b}{\sqrt{12}}$$

I: moment of inertia of column;

b: width of column.

f_{c28} = 25 MPa;
f_e = 400 MPa;
durable situation : (γ_b=1.5 ; γ_s=1.15)

II.4.2 Calculation of bearing capacity $\bar{N}$

Table II. 2. Bearing capacity $\bar{N}$ for each column section.

N°	B(mm²)	$B_r(\mathrm{mm}^2)$	$l_f(m)$	i (mm)	λ	α	$A_s(cm^2)$	$\bar{N}(KN)$
7	350x350	108900	2.142	101.03	21.19	0.79	1225	1626.83
6	350x350	108900	2.142	101.03	21.19	0.79	1225	1626.83
5	350x350	108900	2.142	101.03	21.19	0.79	1225	1626.83
4	400x400	144400	2.142	115.47	18.55	0.80	1600	2183.78
3	400x400	144400	2.142	115.47	18.55	0.80	1600	2183.78
2	400x400	144400	2.142	115.47	18.55	0.80	1600	2183.78
1	450x450	184900	2.142	129.9	16.49	0.82	2025	2830.55
Gr.	450x450	184900	2.142	129.9	16.49	0.82	2025	2830.55
Bas.	450x450	184900	2.142	129.9	16.49	0.82	2025	2830.55

- **Choosing the most stressed column**

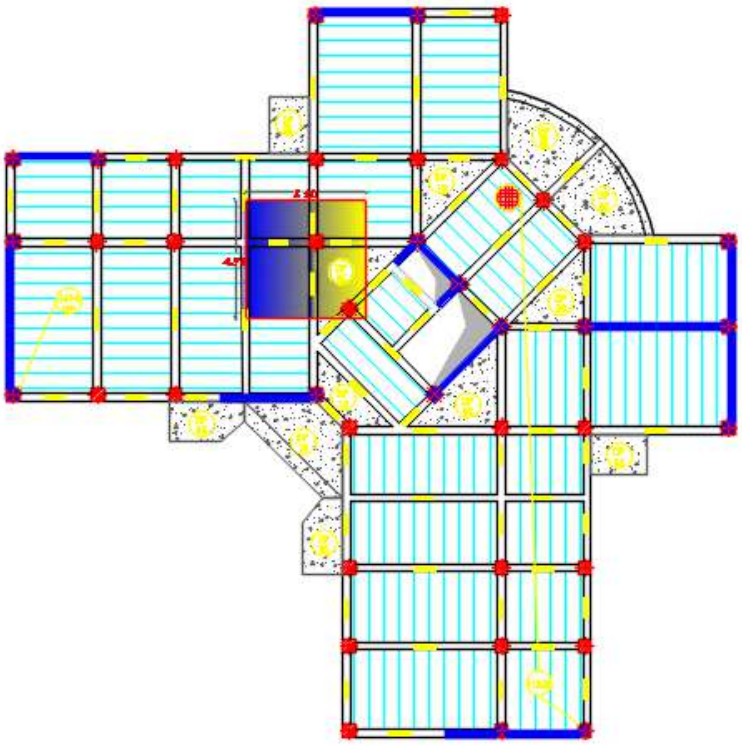


Figure II. 2.The most stressed column (colored square).

➢ **Calculation of the area taken by the column**

$S_{total} = (3.00 + 0.3 + 1.65) \times (2.90 + 0.3 + 2.00) = 25.74\ m^2$

$S_{floor} = (3.00 + 1.65) \times (2.90 + 2) = 22.785\ m^2$

II.4.3 Loads estimation [3] [DTR B.C 2.2]

A. Dead loads

A.1.Masonry

Internal walls (single partition)

Table II. 3. Dead load of the internal wall

N°	Components	Thickness (m)	Density (kN/m³)	Surface density(kN/m²)	Reference article
1	**Cement rendering**	0.02	18	0.36	[3] DTR B.C. 2.2 C.2-Enduits
2	**Hollow brick**	0.10	9	0.90	[3] DTR B.C. 2.2 C.1 maçonnerie
3	**Cement rendering**	0.02	18	0.36	[3] DTR B.C. 2.2 C.2-Enduits
TOTAL				**1.62**	

External walls (double partition)

Table II. 4. Dead load of the external wall

N°	Components	Thickness (m)	Density (kN/m³)	Surface density (kN/m²)	Reference article
1	**Cement rendering**	0.02	18	0.36	[3] DTR B.C. 2.2 C.2-Enduits
2	**Hollow brick**	0.10	90	0.90	[3] DTR B.C. 2.2 C.1 maçonnerie
3	**Air-gap**	0.05	/	/	/
4	**Hollow brick**	0.10	90	0.90	[3] DTR B.C. 2.2 C.1 maçonnerie
5	**Cement rendering**	0.02	18	0.36	[3] DTR B.C. 2.2 C.2-Enduits
TOTAL				**2.52**	

A.2.Floors

- **Terrace floor (inaccessible)**

Table II. 5. terrace floor (hollow pot floor slab) dead load

N°	Components	Thickness (m)	Density (kN/m³)	Surface density (kN/m²)	Reference article
1	**Rolled Gravel 40mm**	0.04	20	0.80	[3] DTR B.C. 2.2 C.5-TOITURES
2	**Multi-layered water proofing**	0 .02	6	0.12	[3] DTR B.C. 2.2 C.5-TOITURES
3	**Kraft paper**	/	/	0.05	[3] DTR B.C. 2.2 C.5-TOITURES
4	**Slope shape**	0.10	22	2.20	[3] DTR B.C. 2.2 C.5-TOITURES
5	**Polyethylene film**	/	/	0.01	[3] DTR B.C. 2.2 C.5-TOITURES
6	**Cork slab**	0.04	4	0.16	[3] DTR B.C 2.2 B-TOITURES
7	**Vapour barrier**	/	/	0.06	[3] DTR B.C. 2.2 C.5-TOITURES
8	**low core slab (16+4)**	0.16+0.04	/	2.80	[3] DTR B.C. 2.2 C.3-PLANCHERS
9	**Plaster rendering**	0.02	10	0.20	[3] DTR B.C. 2.2 C.2-ENDUITS
TOTAL				**6.40**	

- **Typical floor**

Table II. 6. Typical floor (hollow pot floor slab) Dead load

N°	Components	Thickness (m)	Density (kN/m³)	Surface density (kN/m²)	Reference article
1	**Floor tile**	0.02	20	0.40	[3] DTR B.C. 2.2 C.4-Revetement du plancher
2	**Bedding mortar**	0.02	20	0.40	[3] DTR B.C. 2.2 C.4-Revetement du plancher
3	**Sand Blinding**	0.03	18	0.54	[3] DTR B.C. 2.2 ANNEXE VI-A
4	**Hollow core slab (16+4)**	0.20	/	2.80	[3] DTR B.C. 2.2 C.3-PLANCHERS
5	**Plaster rendering**	0.02	10	0.20	[3] DTR B.C. 2.2 C.2-ENDUITS
6	**Single partition**	0.10	/	1.62	[3] DTR B.C. 2.2 C.1-MACONNERIE
TOTAL				**5.96**	

Table II. 7. Typical floor (flat slab) Dead load

N°	Components	Thickness (m)	Density (kN/m³)	Surface density (kN/m²)	Reference article
1	**Floor tile**	0.02	20	0.40	[3] DTR B.C. 2.2 C.4-Revetement du plancher
2	**Bedding mortar**	0.02	20	0.40	[3] DTR B.C. 2.2 C.4-Revetement du plancher
3	**Sand Blinding**	0.03	18	0.54	[3] DTR B.C. 2.2 ANNEXE VI-A
4	**Solid/flat slab**	0.15	25	3.75	[3] DTR B.C. 2.2 B-POIDS VOLUMIQUE
5	**Plaster rendering**	0.02	10	0.20	[3] DTR B.C. 2.2 C.2-ENDUITS
6	**Single partition**	0.10	/	1.62	[3] DTR B.C. 2.2 C.1-MACONNERIE
TOTAL				**6.91**	

A. Total Dead Loads

$$G_{level} = G_{floor} + G_{mb} + G_{sb} + G_{col}$$

With:

G_{floor}= floor dead load× S_{floor}

$G_{(mb)or(sb)} = b \times h \times L \times \gamma_b$

$G_{col} = b \times h_{str} \times L \times \gamma_b$

γ_b: concrete density, $\gamma_b = 25kN/m^3$.

Table II. 8. Cumulative dead loads of the surfaces covered (taking over)

level	$G_{floor}(kN)$	$G_{mb}(kN)$	$G_{sb}(kN)$	$G_{col}(kN)$	$G_{level}(kN)$	$G_{cum}(kN)$
7	145.82	15.69	12.86	9.37	183.74	183.74
6	135.80	15.69	12.86	9.37	173.72	357.46
5	135.80	15.69	12.86	9.37	173.72	531.18
4	135.80	15.69	12.86	12.24	176.59	707.77
3	135.80	15.69	12.86	12.24	176.59	884.36
2	135.80	15.69	12.86	12.24	176.59	1060.95
1	135.80	15.69	12.86	15.49	179.84	1240.79
Ground	135.80	15.69	12.86	15.49	179.84	1420.63
Basement	135.80	15.69	12.86	15.49	179.84	1600.74

B. Live Loads

- Accessible terrace : Q= 1.5 kN/m² **[3] ART 7.5**
- Inaccessible terrace : Q= 1 kN/m² **[3] ART 7.5**
- Residential: Q=1.5 kN/m² **[3] ART 7.1**

Total Live Loads

To determine live loads, we used the following degressive law: **[3]**

$$\sum Q = Q_0 + \frac{3+n}{2n}\sum_{i=1}^{n} Q_i$$

The coefficient (3+n)/2n is valid for $n \geq 5$

$Q_0 = 1kN/m^2$ (inaccessible terrace);

$Q = 1.5\ kN/m^2$ (for residential use);

7th storey: Q_0
6th storey: $Q_0 + Q_1$

5th storey: $Q_0 + 0.95(Q_1 + Q_2)$

4th storey: $Q_0 + 0.90(Q_1 + Q_2 + Q_3)$

3rd storey: $Q_0 + 0.85(Q_1 + Q_2 + Q_3 + Q_4)$

2nd storey: $Q_0 + 0.80(Q_1 + Q_2 + Q_3 + Q_4 + Q_5)$

1st storey: $Q_0 + 0.75(Q_1 + Q_2 + Q_3 + Q_4 + Q_5 + Q_6)$

Ground: $Q_0 + 0.71(Q_1 + Q_2 + Q_3 + Q_4 + Q_5 + Q_6 + Q_7)$

Basement: $Q_0 + 0.69(Q_1 + Q_2 + Q_3 + Q_4 + Q_5 + Q_6 + Q_7 + Q_{Gr})$

$$Q = S_{total} * Q_i$$

Table II. 9. Cumulative live loads of the surfaces covered

Level	*Q (kN)*	*3+n/2n*	$Q_{cum}(kN)$
7	25.74	/	25.74
6	25.74	1	51.48
5	25.74	0.95	75.93
4	25.74	0.90	99.09
3	25.74	0.85	120.97
2	25.74	0.80	141.56
1	25.74	0.75	160.86
Ground	25.74	0.71	179.14
Basement	25.74	0.69	196.9

C. Ultimate axial load N_u

- $N_u = 1.35G + 1.5Q$
- $Nu^* = N_u + 10\%N_u = 1.1N_u$

Table II. 10. Verification of the axial load of the columns

Level	B (mm²)	G (kN)	Q (kN)	N_u (kN)	Nu* (kN)	$\bar{N}$(kN)	Nu* ≤ $\bar{N}$
7	350x350	183.74	25.74	286.66	315.32	1626.83	Verified
6	350x350	357.46	51.48	559.79	615.77	1626.83	Verified
5	350x350	531.18	75.93	830.99	914.08	1626.83	Verified
4	400x400	707.77	99.09	1104.12	1214.53	2183.78	Verified
3	400x400	884.36	120.97	1375.34	1511.87	2183.78	Verified
2	400x400	1060.95	141.56	1644.62	1809.08	2183.78	Verified
1	450x450	1240.79	160.86	1916.35	2107.98	2830.55	Verified
Gr.	450x450	1420.63	179.14	2186.77	2405.45	2830.55	Verified
Bas.	450x450	1600.74	196.9	2456.35	2701.98	2830.55	Verified

Verifications according to (RPA99 V2003)

- **Section verification : (RPA 99 (version 2003) Art.7.4.1)**

Table II. 11. Section verification according to (RPA99 V2003)

Condition to check	Condition application	verification
Zone III **Min (b, h) ≥ (30x30) cm** **Min (b, h) ≥ h_e /20** **0.25 ≤ (b/h) ≤ 4**	Min (b, h) = (35x35) cm h_e /20 = (306-h_{mb}) /20 = 13.05 cm ≤ 35cm 0.25 ≤ (b/h) = 1 ≤ 4	Verified

Conclusion

The sections adopted are: (45x45)cm² ; (40x40)cm² and (35x35)cm²

The pre-sized sections of the columns of all levels are sufficient comparing to the ultimate normalforces, and satisfied the conditions of the RPA99 version 2003.

II.5 Pre-design of sail walls

They are made of reinforced concrete; their role is to support a fraction of the vertical loads as well as horizontal loads. We call sail walls the elements satisfying the condition L ≥ 4a. Otherwise, these elements are considered as columns.

The (**RPA99 v2003 Art7.7.1**) requires a minimum thickness of 15 cm, in addition, the thickness depends on the clear height (headroom) h_e and stiffness conditions at the ends.

$$a \geq max\left[\frac{h_e}{22} = \frac{306 - 35}{22} = 12.31cm \; and \; 15cm\right]$$

We adopted $\boldsymbol{a}$ **=15 cm.**
For the peripheric shell, we adopted $\boldsymbol{a}$ **=20 cm.**

II.6 Conclusion

The adopted sections and dimensions of the previous elements are:

- For the hollow pot floor we adopted a thickness of e =(16+4)cm =20cm;
- The solid slab is with a thickness of e =15cm;
- The main beams took the section of (30x45)cm²;
- The secondary beams took the section of (30x35)cm²;
- The ring beams also have a section of (30x35)cm²;
- For the columns we adopted three sections:
  - (45x45)cm² for the three first levels;
  - (40x40)cm² for the three next levels;
  - (35x35)cm² for the last three levels.
- The shell walls took two thicknesses:
  - a=15cm for bracing sails;
  - a=20cm for periphery shells.

CHAPTER III

Calculation of secondary elements

III.1 Introduction

The third chapter is devoted to the calculation of the secondary elements such as the acroterion, the slab beams, the stairs and the different types of slabs. We will calculate the moments and the shear forces for each element to obtain a quantity of reinforcement that can maintain it stable and resistant against shearing and deflection.

III.2 Acroterion

The acroterion is made of reinforced concrete placed on the periphery of the terrace floor. It prevent the water runoff from getting behind the waterproofing statements, it protects also the rendering and facade cladding. The reinforcement calculation is done for a strip one meter wide, it is an isostatic system similar to a 01m wide console, embedded in the floor terrace and subjected to a normal force due to its own weight and a moment during installation due to an operating load of 01kN/ml applied to its end. The cracking is considered detrimental because of the bad weather.

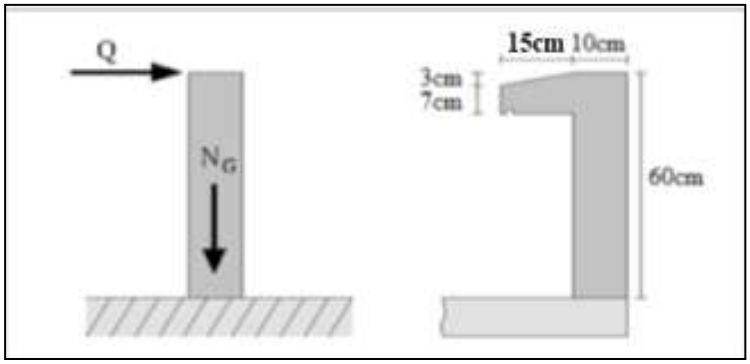


Figure III. 1. Acroterion (dimensions and loads exercised)

III.2.1 Evaluation of the loads and the overloads

Table III. 1. Loads assessments/estimate of the acroterion.

Dead loads	G = 1.875 kN/ml
Live loads	Q=1 kN/ml

III.2.2 Determination of the efforts

Table III. 2. The efforts exerted on the acroterion.

	ULS	**SLS**
Normal load/axial load	$N_u = 1.35G = 2.53kN$	$N_{ser} = G = 1.875$ kN
(Fixed-end /restraint) moment	$M_u = 1.5\ Q.h = 0.9$ kN.m	$M_{ser} = Q.h = 0.6$ kN.m
Transverse Shear load	$T_u = 1.5\ Q = 1.5$ kN	/

III.2.3 Reinforcement calculation

The reinforcement calculation carried out with a rectangular section (100x10) cm² subjected to compound bending and in a durable situation at ULS.

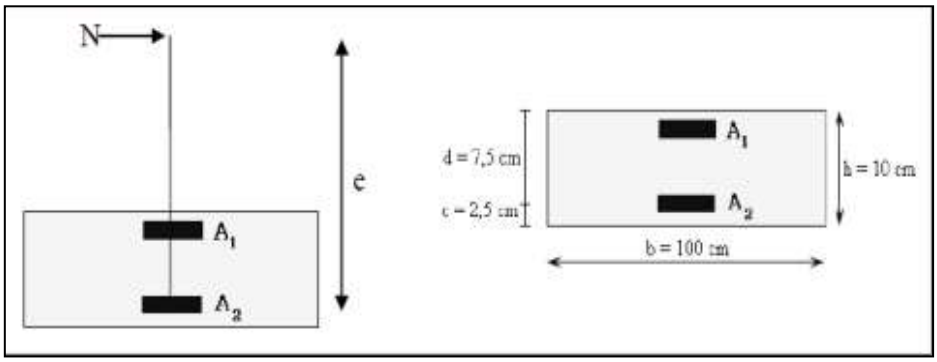


Figure III. 2. Reinforced concrete section in the acroterion.

Table III. 3. Reinforcement adopted

Section (cm²)	A_1 (cm²)	A_2 (cm²)	A_1 Chosen (cm²)/ml	A_2 Chosen (cm²)/ml	A_{dis} (cm²)/0.6m	St (cm)	St_{dis} (cm)
100x10	4.40	4.40	7HA10 = 5.50	7HA10 = 5.50	4HA8 = 2.01	15	15

III.2.3.1 Verification at the service limit state SLS

$$\overline{\sigma_s} = \text{Min}\left[\frac{2}{3}f_e; 110\sqrt{\eta . f_{tj}}\right] = 201.63\ \text{Mpa} \quad ; \quad \overline{\sigma_b} = 0.6 f_{c28} = 15 Mpa$$

$$\begin{cases} \sigma_b = 0.34\text{Mpa} \leq \overline{\sigma_b} = 15\text{Mpa} & \Longrightarrow \text{Verified} \\ \sigma_s = 2.22\text{Mpa} \leq \overline{\sigma_s} = 201.63\text{Mpa} & \Longrightarrow \text{Verified} \\ \sigma_s' = 2.70\text{Mpa} \leq \overline{\sigma_s} = 201.63\text{Mpa} & \Longrightarrow \text{Verified} \end{cases}$$

III.2.3.2 Verification of shear force at (ULS)

We must check this condition: $r \leq r_{adm}$

$$r_{adm} = \frac{0.07 f_{c28}}{\gamma_b} = 1.16\text{Mpa} > r = \frac{T_u}{b.d} = 0.02 Mpa \implies \text{verified}$$

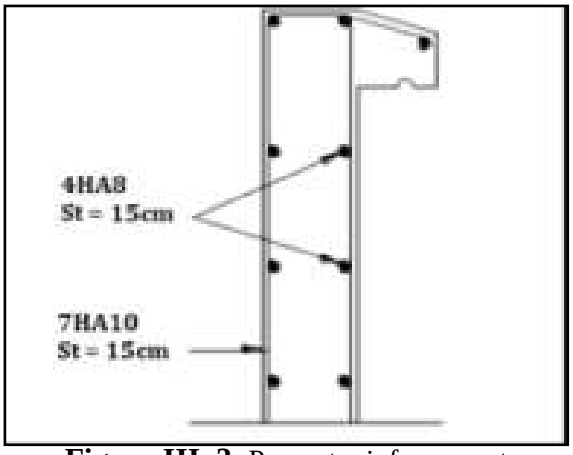


Figure III. 3. Parapet reinforcement.

III.3 Hollow pot floor

This type of floor is commonly used in residential buildings, it is consisting of:

- Prefabricated reinforced concrete slab beams, arranged in parallel, 60 cm apart.
- Hollow body (Entrevous) in concrete suitable for beams.
- A 04 cm thick superior concrete compression slab.

The calculation of floors is done exclusively under the action of vertical loads, and for floors with hollow bodies, the calculation will be done for two elements:

- The compression slab
- The beams

III.3.1 Compression slab

Used for the correct distribution of vertical loads on the beams, and to limit the risk of cracking as well as to resist the forces applied on the slab; grid reinforcement is provided.

The reinforcement section to be provided must be met under the following conditions:

- Spacing for reinforcements perpendicular to the ribs do not exceed 20 cm;
- Spacing for reinforcements parallel to ribs do not exceed 33 cm;
- The cross-section of reinforcements in the perpendicular direction to the ribs is given by:

$$A_t \geq \frac{200}{f_e} \qquad if: \quad L \leq 50cm$$

$$A_t \geq 0.02L\frac{200}{f_e} = 4\frac{L}{f_e} \qquad if: \quad 50cm \leq L \leq 80cm$$

With:

L: Spacing between axes of the ribs (60 cm in our work).

Fe: Grade of Steel FeE500 (F_e = 500MPA).

The welded mesh which will be used is a square grid of 20cm x 25cm

We have: L=60cm $\Longrightarrow A_t \geq 0.02 * 60\frac{200}{f_e} = 4\frac{60}{f_e} = 0.48cm^2/ml$

$$A_{t2} = \frac{A_{t1}}{2} = \frac{0.48}{2} = 0.24cm^2/ml$$

A_{t1}: Reinforcement section perpendicular to the ribs.

A_{t2} The reinforcement section in the parallel direction to the ribs.

We will choose:

$A_{t1} = 5$ HA $5 = 0.98cm^2$ with a spacing of $20\ cm$.

$A_{t2} = 4$ HA $5 = 0.78cm^2$ with a spacing of $25\ cm$.

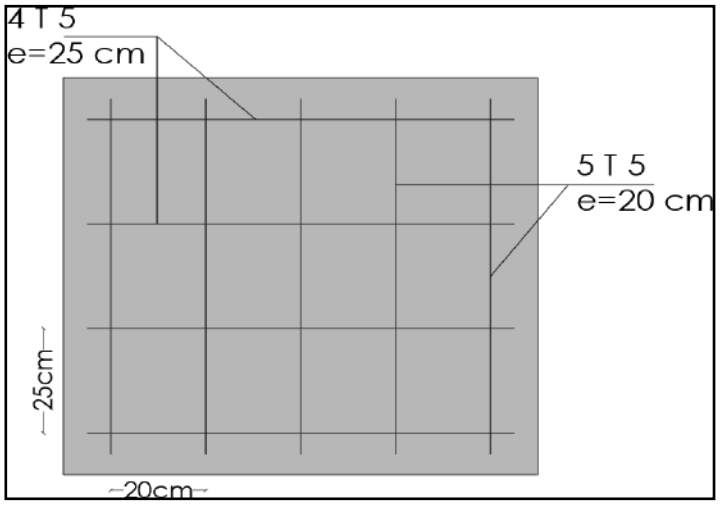


Figure III. 4. Disposition of the reinforcement of the compression slab

III.3.2 Slab beams

The slab beams are prefabricated, they are calculated by the loads (G, Q) with simple bending,the surface returning to each slab beam is S = 0.60*L, the slab beams are arranged in the direction of the short span, the calculation is done in two steps:

- Before casting (pouring).
- After casting (pouring).

III.3.2.1 Before casting

The slab beam before casting is an isostatic beam subjected to the following loads:

- Its self-weight,
- The weight of hollow core (ceiling infill between slab beams).
- A load concentrated in the middle with the greatest intensity of 1 KN or (0.5*L) KN, which represents the weight of a worker.

When $\mu \geq \mu_l$, the beam at this stage will show a large deformation (arrow). Therefore, it is necessary to provide support to eliminate risks and perform shoring calculations so that they support the slab beams after the concrete is poured and not only before the concrete is poured.

Table III. 4. Calculation the number of the shoorings

N°	$L\ (m)$	$G\ (kN/ml)$	$Q\ (kN)$	μ	μ_l	$\mu \leq \mu_l$
0	4	0.72	2	3.23	0.392	Not Verified
1	2	0.72	1	0.81	0.392	Not Verified
2	1.33	0.72	1	0.47	0.392	Not Verified
3	1	0.72	1	0.320	0.392	Verified

We did not use any shoring in the first case and we got $\mu \geq \mu_l$, it means that the deflection is big; so we used one shoring but it was not enough, with two shorings it was not verified as well. The deflection was accepted in the fourth case with three shorings where we got $\mu \leq \mu_l$.

III.3.2.2 After casting

After the concrete is poured, all the beams are considered as a continuous beam. For the floors studied, there will be three types of beams to be treated separately under the loads and overloads of the current floor and the terrace:

- Two span beams.
- Four span beams.
- Five span beams.

Reinforcement calculations will be carried out through the sum/flat rate method (Forfitaire) or through the Caquot method. The conditions of the flate rate method are not checked so, we will calculate with the Caquot method.**(Annex 01)**

A. Terrace roof

➢ **Five span beams**

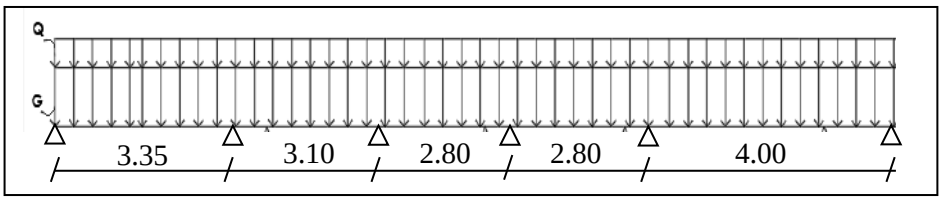


Figure III. 5. Equivalent static figure of 05 span beam with dimensions

Table III. 5. Moments and shear forces of 05 span beam for the terrace roof in ULS

Span	P (KN)	L	L'	M_0 (kN.m)	M_ω (kN.m)	M_e (kN.m)	M_t (kN.m)	T_ω (kN)	T_e (kN)
1--2	6.084	3.35	3.35	8.53	-1.71	-6.49	4.44	8.76	-11.62
2--3	6.084	3.1	2.48	7.31	-6.49	-4.02	2.06	10.23	-8.63
3--4	6.084	2.8	2.24	5.96	-4.02	-4.71	1.60	8.27	-8.77
4--5	6.084	2.8	2.8	5.96	-4.71	-9.05	-0.92	6.97	-10.07
5--6	6.084	4	4	12.17	-9.05	-2.43	6.43	13.82	-10.51

Table III. 6. Moments of 05 span beam for the terrace roof in SLS

Span	P (KN)	L	L'	M_0 (kN.m)	M_ω (kN.m)	M_e (kN.m)	M_t (kN.m)
1--2	4.44	3.35	3.35	6.23	-1.25	-4.74	3.24
2--3	4.44	3.1	2.48	5.33	-4.74	-2.93	1.50
3--4	4.44	2.8	2.24	4.35	-2.93	-3.44	1.17
4--5	4.44	2.8	2.8	4.35	-3.44	-6.60	-0.67
5--6	4.44	4	4	8.88	-6.60	-1.78	4.69

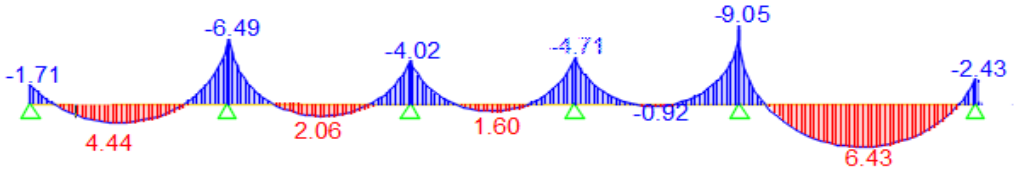


Figure III. 6. Diagram of the moment of the 5-span beam at ULS for terrace roof

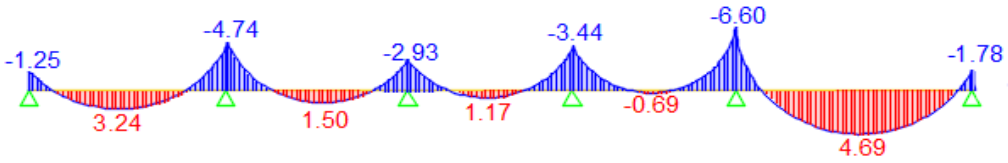


Figure III. 7. Diagram of the moment of the 5-span beam at SLS For terrace roof

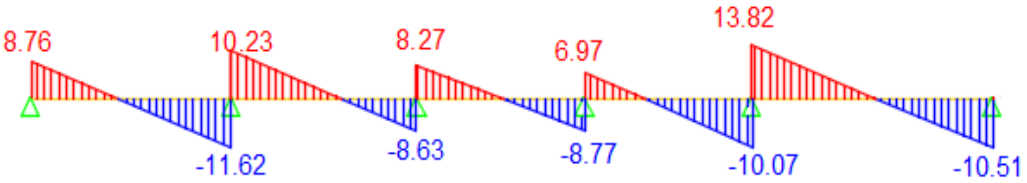


Figure III. 8. Diagram of shear force of the 5-span beam for terrace floor

- **Four span beams**

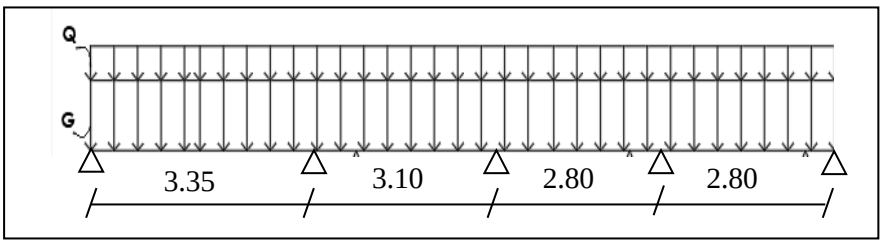


Figure III. 9. Equivalent static figure of 04 span beam with dimensions

Table III. 7. Moments and shear forces of 04 span beam for the terrace roof in ULS

Span	P	L	L'	M_0 (kN.m)	M_ω (kN.m)	M_e (kN.m)	M_t (kN.m)	T_ω (kN)	T_e (kN)
1--2	6.084	3.35	3.35	8.53	-1.71	-6.49	4.44	8.76	-11.62
2--3	6.084	3.1	2.48	7.31	-6.49	-4.02	2.06	10.23	-8.63
3--4	6.084	2.8	2.24	5.96	-4.02	-4.71	1.60	8.27	-8.77
4--5	6.084	2.8	2.8	5.96	-4.71	-1.19	3.01	9.78	-7.26

Table III. 8. Moments of 04 span beam for the terrace roof in SLS

Span	P	L	L'	M_0 (kN.m)	M_ω (kN.m)	M_e (kN.m)	M_t (kN.m)
1--2	4.44	3.35	3.35	6.23	-1.25	-4.74	3.24
2--3	4.44	3.1	2.48	5.33	-4.74	-2.93	1.50
3--4	4.44	2.8	2.24	4.35	-2.93	-3.44	1.17
4--5	4.44	2.8	2.8	4.35	-3.44	-0.87	2.20

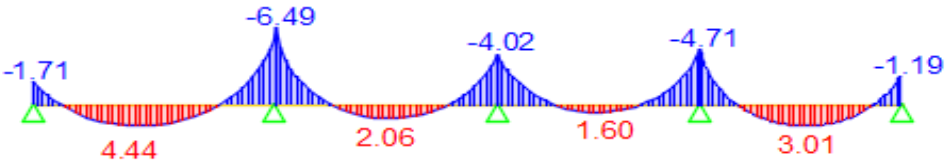


Figure III. 10. Diagram of the moment of the 4-span beam at ULS for terrace roof

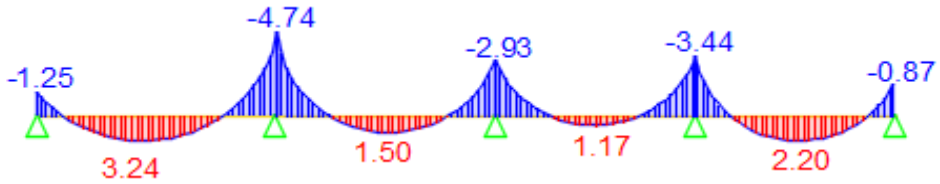


Figure III. 11. Diagram of the moment of the 4-span beam at SLS for terrace roof

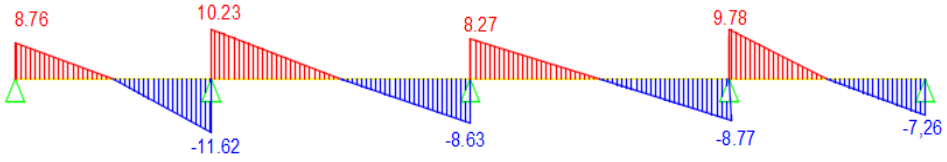


Figure III. 12. Diagram of the shear force of the 4-span beam for terrace roof

- **Two span beams**

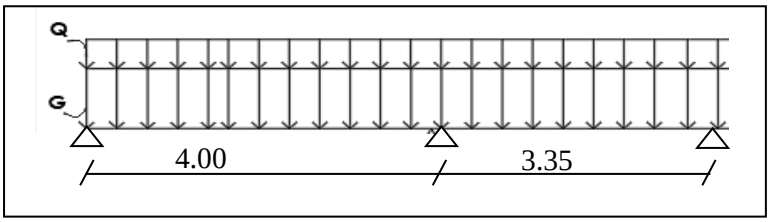


Figure III. 13. Equivalent static figure of 02 span beam with dimensions

Table III. 9. Moments and shear forces of 02 span beam for the terrace roof in ULS

Span	P	L	L'	M_0 (kN.m)	M_ω (kN.m)	M_e (kN.m)	M_t (kN.m)	T_ω (kN)	T_e (kN)
1--2	6.084	4	4	12.17	-2.43	-9.89	6.00	10.30	-14.03
2--3	6.084	3.35	3.35	8.53	-9.89	-1.71	2.73	12.63	-7.75

Table III. 10. Moments of 02 span beam for the terrace roof in SLS

Span	P	L	L'	M_0 (kN.m)	M_ω (kN.m)	M_e (kN.m)	M_t (kN.m)
1--2	4.44	4	4	8.88	-1.78	-7.22	4.38
2--3	4.44	3.35	3.35	6.23	-7.22	-1.25	2.00

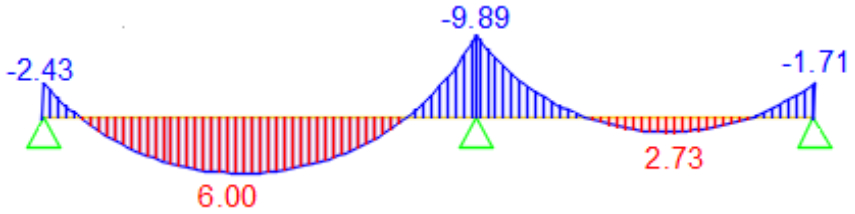


Figure III. 14. Diagram of the moment of the 2-span beam at ULS for terrace roof

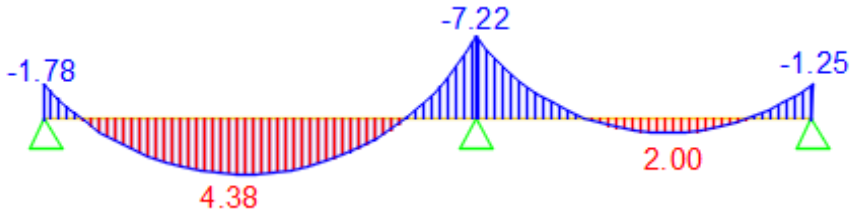


Figure III. 15. Diagram of the moment of the 2-span beam at SLS for terrace roof

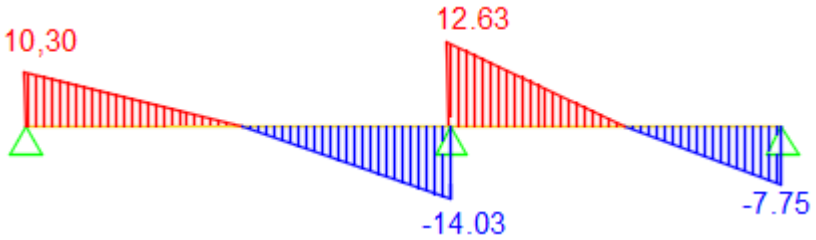


Figure III. 16. Diagram of the shear force of the 2-span beam for terrace roof

B. Typical roof

- **Five span beams**

Table III. 11. Moments and shear forces of 05 span beam for the typical roof in ULS

Span	P	L	L'	M_0 (kN.m)	M_ω (kN.m)	M_e (kN.m)	M_t (kN.m)	T_ω (kN)	T_e (kN)
1--2	6.177	3.35	3.35	8.67	-1.73	-6.59	4.51	8.90	-11.80
2--3	6.177	3.1	2.48	7.42	-6.59	-4.08	2.09	10.38	-8.77
3--4	6.177	2.8	2.24	6.05	-4.08	-4.79	1.62	8.40	-8.90
4--5	6.177	2.8	2.8	6.05	-4.79	-9.19	-0.93	7.08	-10.22
5--6	6.177	4	4	12.36	-9.19	-2.47	6.53	14.03	-10.68

Table III. 12. Moments of 05 span beam for the typical roof in SLS

Span	P	L	L'	M_0 (kN.m)	M_ω (kN.m)	M_e (kN.m)	M_t (kN.m)
1--2	4.476	3.35	3.35	6.28	-1.26	-4.77	3.26
2--3	4.476	3.1	2.48	5.38	-4.77	-2.96	1.51
3--4	4.476	2.8	2.24	4.39	-2.96	-3.47	1.17
4--5	4.476	2.8	2.8	4.39	-3.47	-6.66	-0.68
5--6	4.476	4	4	8.95	-6.66	-1.79	4.73

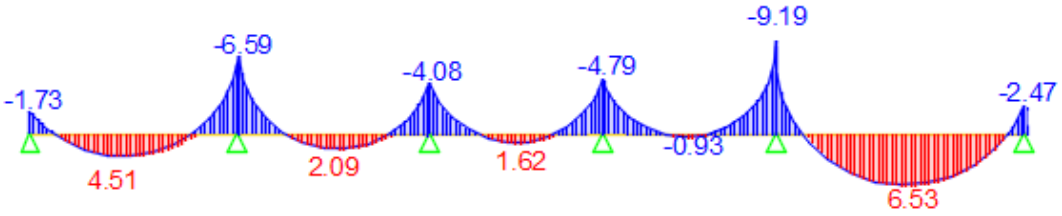


Figure III. 17. Diagram of the moment of the 5-span beam at ULS for typical roof

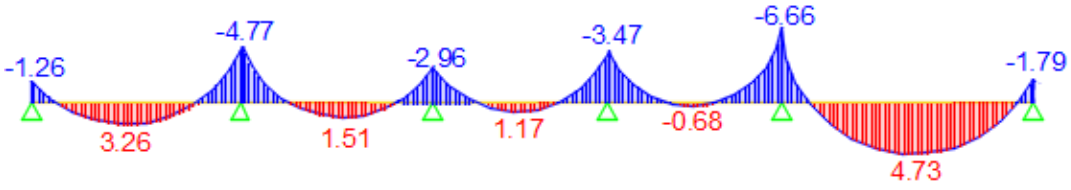


Figure III. 18. Diagram of the moment of the 5-span beam at SLS for typical roof

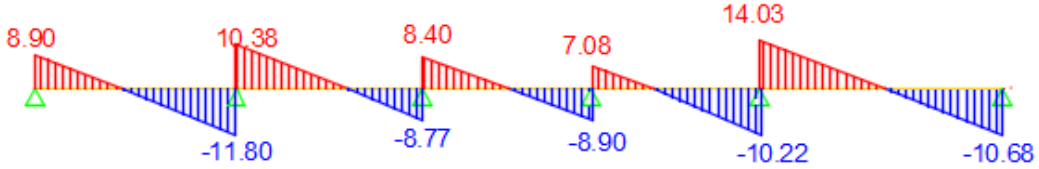


Figure III. 19. Diagram of the shear force of the 5-span beam for typical roof

- **Four span beams**

Table III. 13. Moments and shear forces of 04 span beam for the typical roof in ULS

Span	P	L	L'	M_0 (kN.m)	M_ω (kN.m)	M_e (kN.m)	M_t (kN.m)	T_ω (kN)	T_e (kN)
1--2	6.177	3.35	3.35	8.67	-1.73	-6.59	4.51	8.90	-11.80
2--3	6.177	3.1	2.48	7.42	-6.59	-4.08	2.09	10.38	-8.77
3--4	6.177	2.8	2.24	6.05	-4.08	-4.79	1.62	8.40	-8.90
4--5	6.177	2.8	2.8	6.05	-4.79	-1.21	3.06	9.93	-7.37

Table III. 14. Moments of 04 span beam for the typical roof in SLS

Span	P	L	L'	M_0 (kN.m)	M_ω (kN.m)	M_e (kN.m)	M_t (kN.m)
1--2	4.476	3.35	3.35	6.28	-1.26	-4.77	3.26
2--3	4.476	3.1	2.48	5.38	-4.77	-2.96	1.51
3--4	4.476	2.8	2.24	4.39	-2.96	-3.47	1.17
4--5	4.476	2.8	2.8	4.39	-3.47	-0.88	2.20

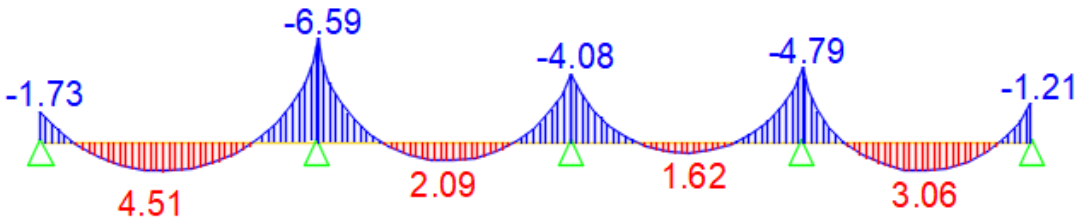


Figure III. 20. Diagram of the moment of the 4-span beam at ULS for typical roof

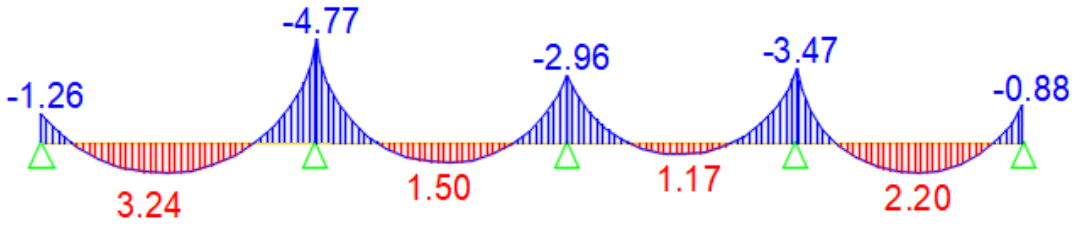


Figure III. 21. Diagram of the moment of the 4-span beam at SLS for typical roof

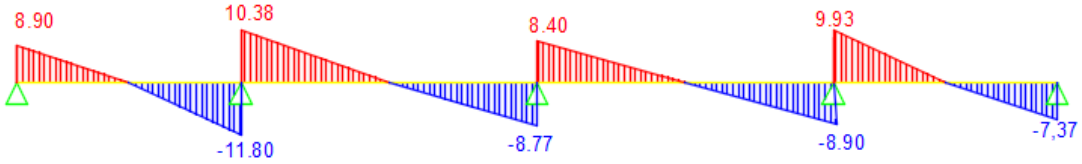


Figure III. 22. Diagram of shear force of the 4-span beam for typical roof

- **Two span beams**

Table III. 15. Moments and shear forces of 02 span beam for the typical roof in ULS

Span	P	L	L'	M_0 (kN.m)	M_ω (kN.m)	M_e (kN.m)	M_t (kN.m)	T_ω (kN)	T_e (kN)
1--2	6.177	4	4	12.36	-2.47	-10.05	6.10	10.46	-14.25
2--3	6.177	3.35	3.35	8.67	-10.05	-1.73	2.78	12.83	-7.87

Table III. 16. Moments of 02 span beam for the typical roof in SLS

Span	P	L	L'	M_0 (kN.m)	M_ω (kN.m)	M_e (kN.m)	M_t (kN.m)
1--2	4.476	4	4	8.95	-1.79	-7.28	4.42
2--3	4.476	3.35	3.35	6.28	-7.28	-1.26	2.01

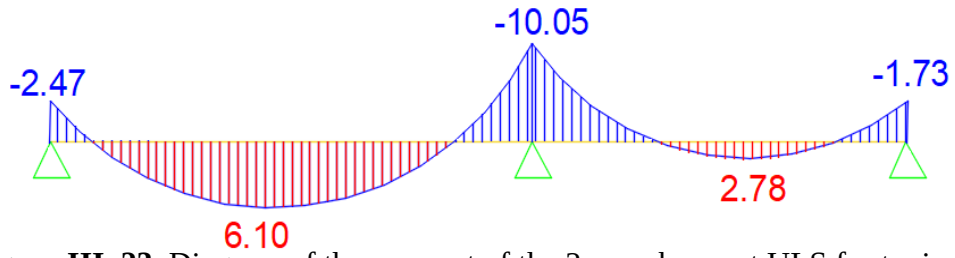


Figure III. 23. Diagram of the moment of the 2-span beam at ULS for typical roof

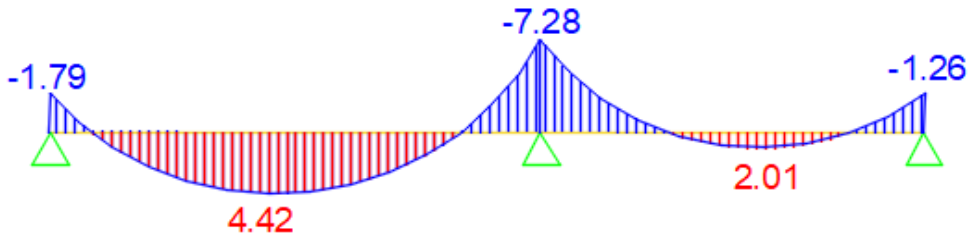


Figure III. 24. Diagram of the moment of the 2-span beam at SLS for typical roof

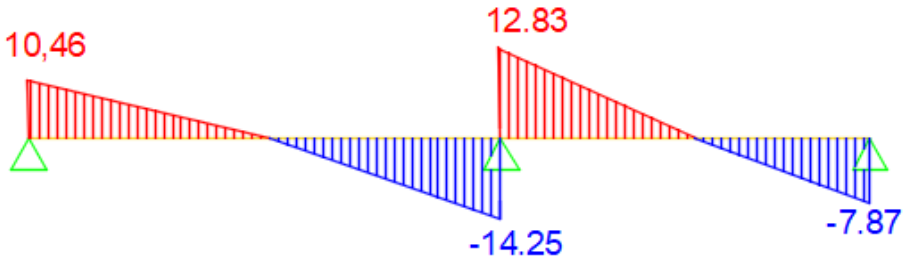


Figure III. 25. Diagram of the shear force of the 2-span beam for typical roof

Table III. 17. Maximum moments and shear forces

Level	Maximum moment at supports (kN.m)		Maximum moment at spans (kN.m)		Maximum Shear force (kN)
	ULS	SLS	ULS	SLS	
Terrace floor	9.89	7.22	6.43	4.69	14.03
Typical floor	10.05	7.28	6.53	4.73	14.25
Beam	02 span beam		05 Span beam		02 span beam

Our worst-case beams are the beams of the typical floor because the maximum moments are in this floor; so with this moments we will calculate the reinforcement.

III.3.2.3 Calculation of the longitudinal reinforcement

The reinforcement is calculated in simple bending with a little harmful cracking, using the Robot Expert calculation software. We calculate the reinforcement with the moments of the worst-case beams and generate it for all the other beams.

Table III. 18. Calculation of the reinforcement for beams at spans and at supports

Position	Level	M_{max} (kN. m)	A_{calcul} (cm^2)	$A_{choosen}$ (cm^2)
At supports	Typical floor	10.05	2.1	1HA10+1ch HA14 = 2.32
At spans	Typical floor	6.53	1,1	2HA10 =1,57

- **Verification at the SLS**

Cracking is almost harmless, so we just check: $\sigma_b \leq \overline{\sigma_b}$ and $\sigma_s \leq \overline{\sigma_s}$.

Table III. 19. Verification of concrete stress at the SLS

Position	Moment at the SLS (kN. m)	Stress (MPa)		Allowable stress (MPa)		Verification
		σ_b	σ_s	$\overline{\sigma_b}$	$\overline{\sigma_s}$	
Supports	7.28	3.6	188.4	15	400	Verified
Spans	4.73	2.8	178.1			Verified

III.3.2.4 Calculation of the distribution bar ϕ_t

We have: $\phi_t \leq$ Min [h /35; b0/10; ϕ_l] =Min [5.71; 12; 10]mm

So: ϕ_t =04mm

$$a_t = \frac{\pi \phi_t^2}{4} = 12.56mm^2; \quad n_t = 2.$$

$A_t = a_t \times n_t = 25.12mm^2$=0.25cm². → 2HA4=0.25cm²

- **Spacing**

$$S_t \leq Min[0.9d; 40cm] = 16.2cm$$

So, we adopt: $S_t = 15cm$.

- **Verification of shear stresses at the ULS**

Table III. 20. Verification of shear stress at ULS.

Floor	$T_u(kN)$	$r_u(MPa)$	$\bar{r}$ (MPa)	Verification
Typical	14.25	0.98	3.33	verified

III.3.2.5 Verification of the deflection

For the verification of the deflection, we used the software **ROBOT EXPERT 2010.**

$$F_{adm} = {}^{l_0}/_{500} = {}^{400}/_{500} = 0.8cm = 08mm$$

Table III.21. verification of the deflection

Support moment (left) Kn.m	Span moment Kn.m	Support moment (right)Kn.m	[F] (mm)	$[\bar{F}]$ (mm)	[F] <$[\bar{F}]$
-6.66	4.73	-1.73	3.6	8.00	Verified

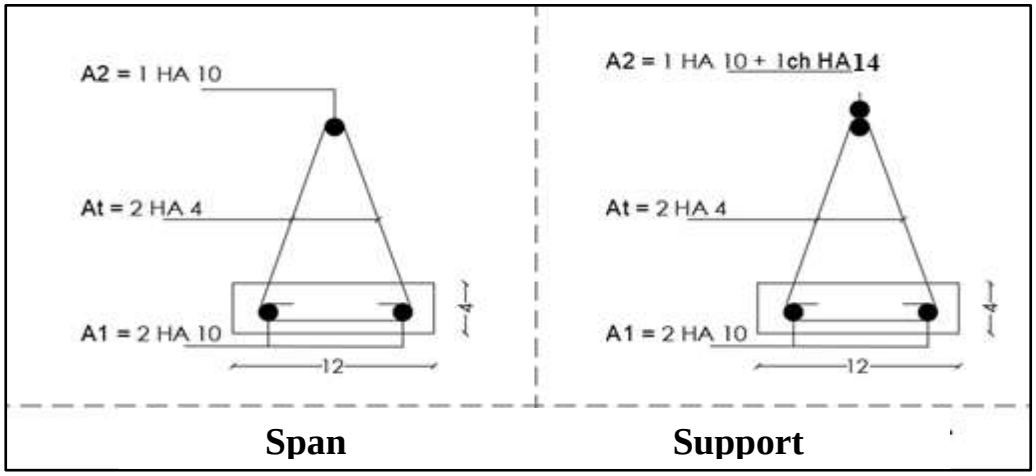


Figure III. 26. Reinforcement of the slab beams

III.4 Stairs

A staircase is a series of horizontal planes arranged in steps, allow to move on foot from one level to another.

III.4.1 Sizing

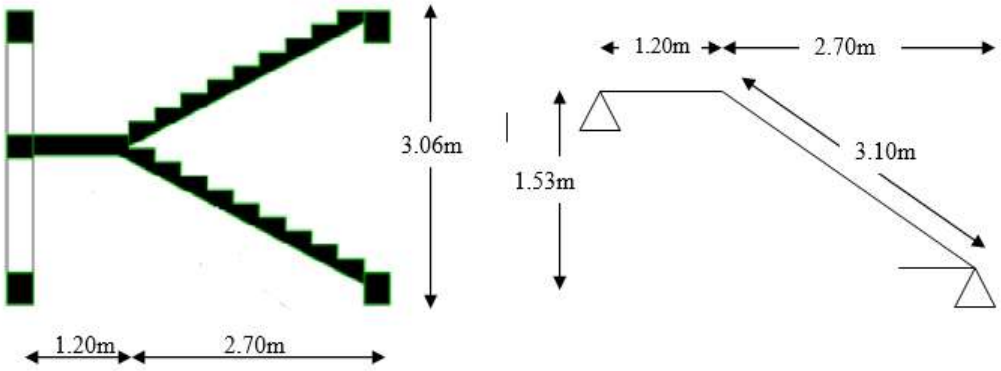


Figure III. 27. Static scheme for flight of stairs on typical floors

- Calculation of the thickness: $\frac{L}{35} \leq e \leq \frac{L}{30}$

With: L=310+120 = 430 cm

$$\frac{430}{35} \leq e \leq \frac{430}{30} \Rightarrow 12.28 \leq e \leq 14.33$$

We adopt: e= 17cm

Note:

We started with e=15cm but the conditions were not verified and the same thing with e=16cm; so we went for e=17cm.

III.4.2 Evaluation of the loads and the overloads

III.4.2.1 Dead loads

Table III. 22. Dead load of Waist slab

N°	Components	Thickness (m)	Density (kN/m³)	Surface density (kN/m²)	Reference article
1	Floor tile	0.02	20	0.40	DTR B.C. 2.2 C.4- Revêtement du plancher
2	Bedding mortar	0.02	20	0.40	DTR B.C. 2.2 C.2-ENDUITS
3	Sand Blinding	0.03	18	0.54	DTR B.C. 2.2 ANEXES VI -A
4	Self-weight of the step/tread	0.17/2	25	2.13	DTR B.C. 2.2 B-POIDS VOLUMIQUE
5	self-weight of Waist slab	0.17/cosα	25	4.88	DTR B.C. 2.2 B-POIDS VOLUMIQUE
6	Cement rendering	0.02	18	0.36	DTR B.C. 2.2 C.2-ENDUITS
TOTAL				8.71	

Table III. 23. Dead load of Intermediate landing

N°	Components	Thickness (m)	Density (kN/m³)	Surface Density (kN/m²)	Reference article
1	**Floor tile**	0.02	20	0.40	DTR B.C. 2.2 C.4- Revêtement du plancher
2	**Bedding mortar**	0.02	20	0.40	DTR B.C. 2.2 C.2- ENDUITS
3	**Sand Blinding**	0.03	18	0.54	DTR B.C. 2.2 ANEXES VI -A
4	**Self-weight of Intermediate landing**	0.17	25	4.25	DTR B.C. 2.2 B-POIDS VOLUMIQUE
5	**Cement rendering**	0.02	18	0.36	DTR B.C. 2.2 C.2- ENDUITS
TOTAL				5.95	

III.4.2.2 Live loads

- For the Intermediate landing and the flight:

  $Q=2.5\ kN/m^2$ **DTR B.C 2.2-Art.7.2.1**

III.4.3 Determination of externally applied effect

ULS: $P_U = 1.35G + 1.5Q$

SLS: $P_{ser} = G + Q$

The calculation of this type of staircase is done in simple bending for one meter wide, simply leaning on the beams and subjected to two types of loads G and Q, under the combination of the ultimate limit state and the verification is done under the combination of the service limit state.

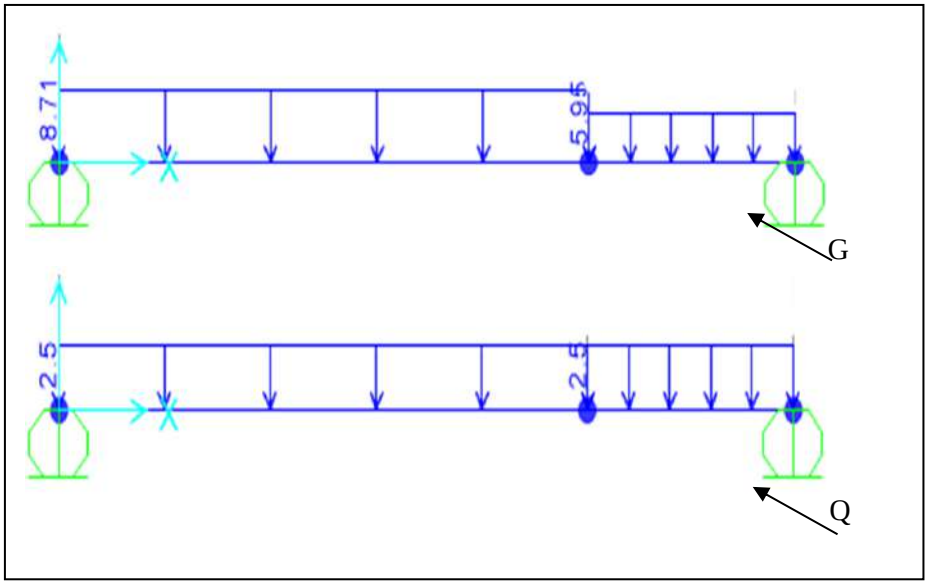


Figure III. 28. Loading on the staircase.

Table III. 24. Load combinations on the staircase.

	G(kN/ml)	Q(kN/ml)	$P_u(kN/ml)$	$P_{ser}(kN/ml)$
Flight	8.71	2.5	15.51	11.21
Landing	5.95	2.5	11.78	8.45

III.4.4 Determination of the moments and shear stresses

Table III. 25. Determination of efforts

Externally applied loads	$M_t(kN.m)$	$M_a(kN.m)$	$T(kN)$
ULS	29.27	-17.22	-32.72
SLS	21.13	-12.43	/

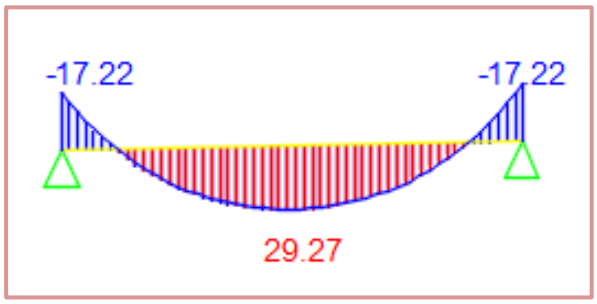


Figure III. 29. Diagram of moment of the staircase at ULS.

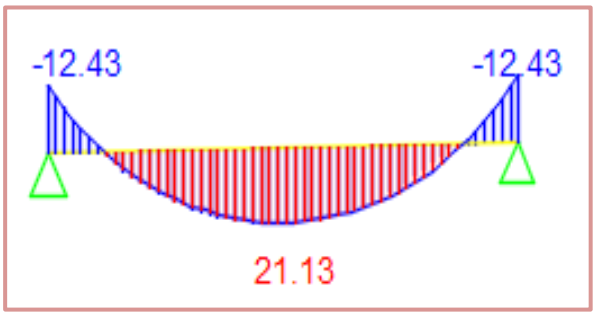


Figure III. 30. Diagram of moment of the staircase at SLS.

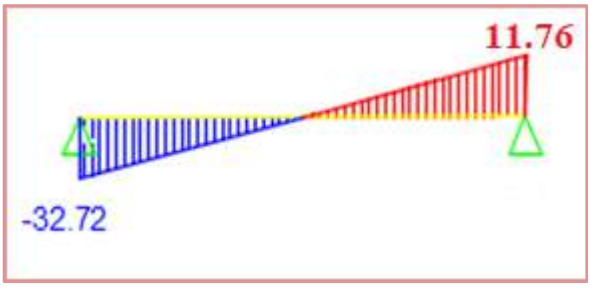


Figure III. 31. Diagram of shear force of the staircase at ULS.

III.4.5 Reinforcement

The calculation of the reinforcement is done for a one meter wide.

III.4.5.1 Longitudinal bars

Table III. 26. Longitudinal reinforcement section for the staircase

	M_{max} $(kN.m)$	A_{calcul} (cm^2)	$A_{choosen}$ (cm^2)	$S_t(cm)$	$[S_t]$adm (cm)	$S_t \leq \bar{S}$
Span	29.27	6.12	7HA12=7.92	15	33	Yes
Supports	-17.22	3.52	7HA12=7.92	15	33	Yes

- Condition of non-fragility ($A \geq A_{min}$=B/1000)

Table III. 27. Verification of the longitudinal reinforcement

	$A(cm^2)$	$A_{min}(cm^2)$	$A_{min} \leq A$
Span	7.92	1.70	Verified
Support	7.92	1.70	Verified

III.4.5.2 Distribution bars

Table III. 28. Distribution reinforcement section for the staircase.

	$A_{dist} = A/4$ (cm^2)	$A_{choosen}$ (cm^2)	$S_t(cm)$	$\bar{S}(cm)$	$S_t \leq \bar{S}$
In span	1.98	7HA8=3.52	15	33	Verified
In support	1.98	7HA8=3.52	15	33	Verified

III.4.5.3 Verification of the normal stress at SLS [1] ART A.4.1.5.3

Table III. 29. Verification of normal stress at the SLS on staircase.

Position	**Moment at the SLS (kN. m)**	**Stress (MPa)**		**Allowable stress (MPa)**		**Verification**
		σ_b	σ_s	$\overline{\sigma_b}$	$\overline{\sigma_s}$	
Supports	-12.43	4.01	121.66	15	400	Verified
Spans	21.13	6.82	206.82			Verified

III.4.5.4 Verification of the shear force at ULS [1] ART A.5.1.2.1.1

Table III. 30. Verification of shear force at the ULS for staircase

Section (cm)	T(kN)	$r(MPa)$	$\bar{r}(MPa)$	$r \leq \bar{r}$
100x17	32.72	0.22	1.16	Verified

III.4.5.5 Verification of the deflection

We used the software **ROBOT EXPERT 2010** to calculate the deflection, and the results are in the following table.

Table III. 31. Verification of the deflection

Support moment (left) Kn.m	Span moment Kn.m	Support moment (right)Kn.m	[F] (mm)	$[\bar{F}]$ (mm)	[F] <$[\bar{F}]$
-12.43	21.13	-12.43	6.1	8.6	Verified

III.5 Landing beam

The landing beam is a beam located at the level of the intermediate landing, it has the role of supporting the weight of the stairs, and it will be calculated in simple bending.

III.5.1 Pre-sizing

$$\frac{L}{15} \leq h \leq \frac{L}{10}$$

0.27h≤b≤0.36h

Table III. 32. Landing beam pre-sizing

Resistance conditions	Verification RPA2003 [Art 7.5.1]
L = 2.40 m $16cm \leq h \leq 24cm$ $We\ adpot\ h = 35cm$ $9.45cm \leq b \leq 12.6cm$ $We\ adopt\ b = 30cm$	$b = 30\ cm \geq 20\ cm$ $h = 35\ cm \geq 30\ cm$ h/b=1.16 ≤ 4.0

III.5.2 Loading on the landing beam



Figure III. 32. Loading on the landing beam.

Using the software "**SAP 2000 23**", we have:

$$\begin{cases} R_G = 15.88\ KN/ml \\ R_Q = 5.38\ \ KN/ml \end{cases}$$

So:

$$G = \frac{[self\ load + 2R_G]}{L} = 14.33\ KN/ml$$

$$Q = \frac{2R_Q}{L} = 4.48\ KN/ml$$

III.5.3 Calculation of internal loads

Table III. 33. Internal loads on landing beam

Efforts	$M_t(kN.m)$	$M_a(kN.m)$	$T(kN)$
ULS	15.95	-9.39	31.28
SLS	11.51	-6.77	/

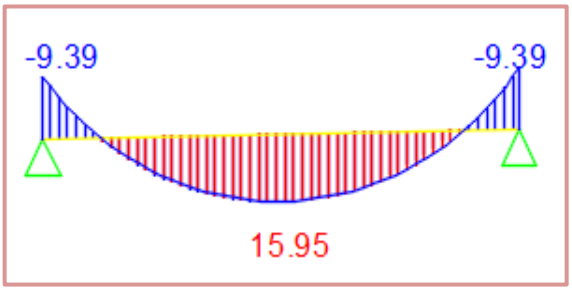


Figure III. 33. Diagram of moment of the landing beam at ULS.

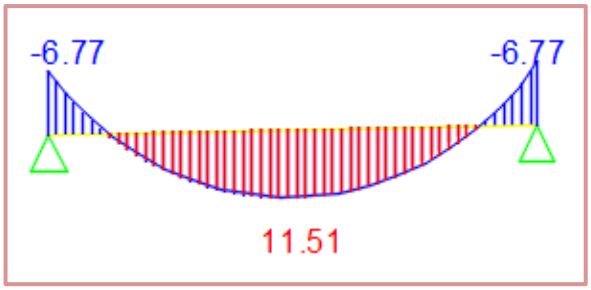


Figure III. 34. Diagram of moment of the landing beam at SLS.

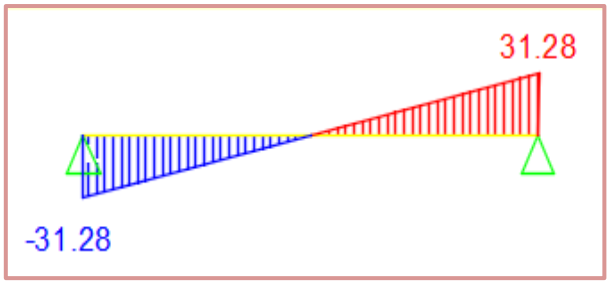


Figure III. 35. Diagram of shear force of the landing beam at ULS.

III.5.4 Reinforcement calculation

III.5.4.1 Longitudinal reinforcement

Table III. 34. Longitudinal reinforcement of landing beam.

	M_{max} $(kN.m)$	A_{calcul} (cm^2)	A_{ch} (cm^2)	S_t (cm)	A_{min} (cm^2)	$A_{ch} > A_{min}$
Span	15.95	1.44	3HA12=3.39	15	1.05	Yes
Supports	-9.39	1.10	3HA12=3.39	15	1.05	Yes

III.5.4.2 Distribution bars

Table III. 35. Distribution reinforcement of landing beam.

Spacing (cm)		**Reinforcement**	
Nodal area	**Current area**	$\emptyset(mm)$	$A_t(cm^2)$
10	15	8	3HA8 = 1.51

III.5.4.3 Verification of normal stress at SLS

Table III. 36. Normal stress of landing beam.

Position	Moment at the SLS (kN. m)	Stress (MPa)		Allowable stress (MPa)		Verification
		σ_b	σ_s	$\overline{\sigma_b}$	$\overline{\sigma_s}$	
Supports	-6.77	0.91	11.71	15	400	Verified
Spans	11.51	1.55	19.91			Verified

III.5.4.4 Verification of shear force at ULS

Table III. 37. Shear force of landing beam.

Section (cm)	T(kN)	$r(MPa)$	$\bar{r}(MPa)$	$r \leq \bar{r}$
30x35	31.28	0.32	2.5	Verified

III.5.4.5 Verification of the deflection

We used the software **ROBOT EXPERT 2010** to calculate the deflection of the landing beam, and the results are in the following table.

Table III. 38. Verification of the deflection of the landing beam.

Support moment (left) Kn.m	Span moment Kn.m	Support moment (right)Kn.m	[F] (mm)	$[\bar{F}]$ (mm)	$[F] < [\bar{F}]$
-6.77	11.51	-6.77	0.1	4.8	Verified

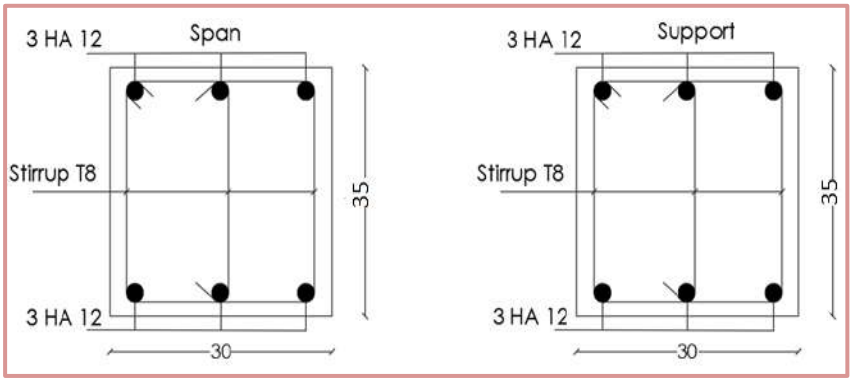


Figure III. 36. Reinforcement of landing beam.

III.6 Solid slab

III.6.1 Introduction

The solid slab is a reinforced concrete floor cast on a flat formwork. The diameter of the reinforcements incorporated and their number vary according to the size of the slab and the importance of the supported loads. The calculation is done in simple bending. This type of floor is widely used in structures, especially for irregular shapes.

In the pre-sizing chapter (chapter two), we adopted e=15cm.

III.6.2 Determination of loads

Table III. 39. Loading on the solid slab.

	G(kN/m)	Q(kN/m)	
		Balcony	Residential
ULS	7.35	3.5	1.5
SLS			

III.6.3 Reinforcement calculation

In our project, we have different shapes of solid slabs, some of them are embedded by three supports and others by two supports.

III.6.3.1 Calculation principle

The yield line method is a method for determining the bending moment of a solid flat panel, that is valid for all load conditions, all types of supports and all possible geometries. The calculation principle is based on the virtual work theory, which includes ensuring static balance from a working perspective. **Annex 2**

We used the software **ETABS 19** and **SAP 22** to extract the moments and the shear forces on the slabs; and **ROBOT EXPERT 2010** to calculate the reinforcement and to verify the deflection.

The solid slabs are colored with orange (following figure), we studied the different shapes and most of them gave us almost the same quantity of reinforcement, so we generated the reinforcement between them, except the shape number one which gave us a bigger quantity of reinforcement.

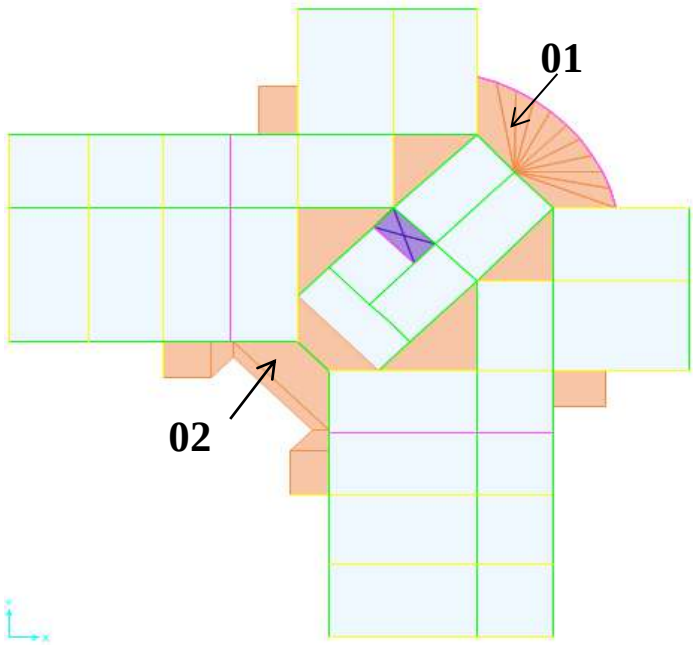


Figure III. 37. Shapes of the solid slabs (orange).

III.6.3.2 Solid slab shape 01 (Balcony)

This shape is embedded by three supports, we will study it in the two direction (X) and (Y).

III.6.3.2.1 Longitudinal reinforcement

- **Direction (x)**

Table III. 40. Longitudinal reinforcement (X) of solid slab 01.

	M_{max} $(kN.m)$	A_{calcul} (cm^2)	A_{ch} (cm^2)/ml	S_t (cm)	A_{min} (cm^2)	$A_{ch} > A_{min}$
Span	00	00	7HA8=3.52	15	1.5	verified
Supports	-35.73	9.02	7HA14=10.78	15	1.5	Verified

- **Direction (y)**

Table III. 41. Longitudinal reinforcement (y) of solid slab 01.

	M_{max} $(kN.m)$	A_{calcul} (cm^2)	A_{ch} (cm^2)	S_t (cm)	A_{min} (cm^2)	$A_{ch} > A_{min}$
Span	7.21	1.75	7HA8=3.52	15	1.5	Verified
Supports	-2.60	1.75	3HA8=3.52	15	1.5	Verified

III.6.3.2.2 Verification of normal stress at SLS

- **Direction (x)**

Table III. 42. Normal stress (x) of solid slab 01.

Position	Moment at the SLS (kN. m)	Stress (MPa)		Allowable stress (MPa)		Verification
		σ_b	σ_s	$\overline{\sigma_b}$	$\overline{\sigma_s}$	
Supports	-26.08	11.32	201.63	15	266.67	Verified
Spans	2.16	0.53	5.27			Verified

- **Direction (y)**

Table III. 43. Normal stress (y) of solid slab 01.

Position	Moment at the SLS (kN. m)	Stress (MPa)		Allowable stress (MPa)		Verification
		σ_b	σ_s	$\overline{\sigma_b}$	$\overline{\sigma_s}$	
Supports	-1.83	0.45	4.46	15	266.67	Verified
Spans	5.27	1.28	12.85			Verified

III.6.3.2.3 Verification of shear force at ULS

Table III. 44. Shear force of solid slab 01.

Section (cm)	T(kN)	$r(MPa)$	$\bar{r}(MPa)$	$r \leq \bar{r}$
15x100	96.45	0.77	1.16	Verified

III.6.3.2.4 Verification of the deflection

Table III. 45. Deflection of solid slab 01.

Span moment Kn.m	Support moment Kn.m	[F] (mm)	$[\bar{F}]$ (mm)	[F] <$[\bar{F}]$
2.16	-26.08	11.6	12	Verified

III.6.3.3 Solid slab (Other shapes)

We will generate the reinforcement of the shape number two to all the other shapes because it was the one that gave us moments bigger than the others.

III.6.3.3.1 Longitudinal reinforcement

- **Direction x**

Table III. 46. Longitudinal reinforcement (X) of solid slab 02.

	M_{max} $(kN.m)$	A_{calcul} (cm^2)	A_{ch} (cm^2)	S_t (cm)	A_{min} (cm^2)	$A_{ch} > A_{min}$
Span	00	00	7HA10=5.50	15	1.5	Verified
Supports	-18.79	4.52	7HA10=5.50	15	1.5	Verified

- **Direction y**

Table III. 47. Longitudinal reinforcement (Y) of solid slab 02.

	M_{max} $(kN.m)$	A_{calcul} (cm^2)	A_{ch} (cm^2)	S_t (cm)	A_{min} (cm^2)	$A_{ch} > A_{min}$
Span	11.60	2.74	7HA8=3.52	15	1.5	Verified
Supports	-5.05	1.75	7HA8=3.52	15	1.5	Verified

III.6.3.3.2Verification of normal stress at SLS

- **Direction x**

Table III. 48. Normal stress (x) of solid slab 02 .

Position	**Moment at the SLS (kN. m)**	**Stress (MPa)**		**Allowable stress (MPa)**		**Verification**
		σ_b	σ_s	$\overline{\sigma_b}$	$\overline{\sigma_s}$	
Supports	-13.73	7.40	201.63	15	266.67	Verified
Spans	1.69	0.41	4.12			Verified

➢ **Direction y**

Table III. 49. Normal stress (y) of solid slab 02 .

Position	Moment at the SLS (kN. m)	Stress (MPa)		Allowable stress (MPa)		Verification
		σ_b	σ_s	$\overline{\sigma_b}$	$\overline{\sigma_s}$	
Supports	-3.68	0.90	8.97	15	266.67	Verified
Spans	8.49	4.58	201.63			Verified

III.6.3.3.3Verification of shear force at ULS

Table III. 50. Shear force of solid slab 02 .

Section (cm)	T(kN)	$r(MPa)$	$\bar{r}(MPa)$	$r \leq \bar{r}$
15x100	20.11	0.16	1.16	Verified

III.6.3.3.4Verification of the deflection

Table III. 51. Deflection of solid slab 02 .

Span moment Kn.m	Support moment Kn.m	[F] (mm)	$[\bar{F}]$ (mm)	$[F] < [\bar{F}]$
1.69	-13.73	4.6	9.6	Verified

III.7 Machine slab

III.7.1 Introduction

The machine slab is a solid slab that is supposed to support more loads than the previous solid slabs like its weight and loads due to the elevator's movement.

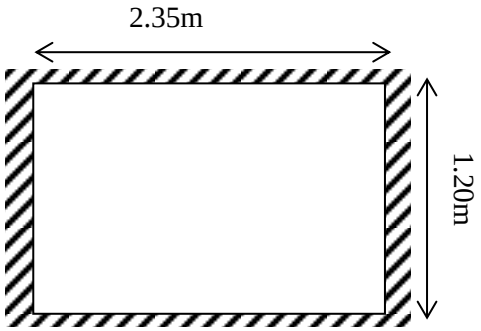


Figure III. 38. Dimensions of the machine slab

III.7.2 Sizing

Our slab is embedded on three sides, so:

$$\frac{L_{max}}{35} \leq e \leq \frac{L_{max}}{30} \quad ; \quad 3.43cm \leq e \leq 04cm$$

The National Elevator Company recommends that the thickness of the machine slab be: $e \geq 25$ cm.

So we adopt: e = 25cm.

III.7.3 Dead and live loads

According to the dimension of our machine slab, the number of persons that is allowed is eight; so we will have:

Q= 0.75x08 = 6 KN

The weight of the elevator is composed of the weight of all the materials used in it (travelling cable, motor generator, guide rail, counterweight …). When we take all these components in consideration; we find that:

G= 30 KN

III.7.4 Reinforcement calculation

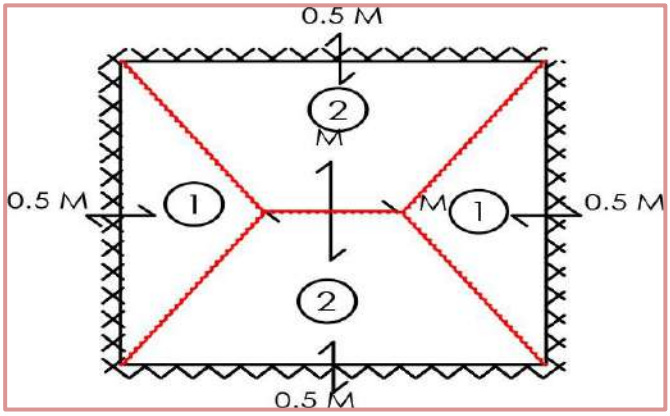


Figure III. 39. internal work of the machine slab.

III.7.4.1 Longitudinal reinforcement

- **Direction (x)**

Table III. 52. Longitudinal reinforcement (x) of the machine slab.

	M_{max} $(kN.m)$	A_{calcul} (cm^2)	A_{ch} $(cm^2)/ml$	S_t (cm)	A_{min} (cm^2)	$A_{ch} > A_{min}$
Span	5.05	2.70	7HA8=3.52	15	2.5	verified
Supports	-16.11	2.70	7HA8=3.52	15	2.5	Verified

➢ **Direction (y)**

Table III. 53. Longitudinal reinforcement (y) of the machine slab.

	M_{max} $(kN.m)$	A_{calcul} (cm^2)	A_{ch} (cm^2)/ml	S_t (cm)	A_{min} (cm^2)	$A_{ch} > A_{min}$
Span	6.55	2.70	7HA8=3.52	15	2.5	verified
Supports	-3.57	2.70	7HA8=3.52	15	2.5	Verified

III.7.4.2 Verification of normal stress at SLS

➢ **Direction (x)**

Table III. 54. Normal stress (x) of the machine slab.

Position	Moment at the SLS (kN. m)	Stress (MPa)		Allowable stress (MPa)		Verification
		σ_b	σ_s	$\overline{\sigma_b}$	$\overline{\sigma_s}$	
Supports	-11.73	1.04	12.50	15	266.67	Verified
Spans	3.66	0.33	3.9			Verified

➢ **Direction (y)**

Table III. 55. Normal stress (y) of the machine slab.

Position	Moment at the SLS (kN. m)	Stress (MPa)		Allowable stress (MPa)		Verification
		σ_b	σ_s	$\overline{\sigma_b}$	$\overline{\sigma_s}$	
Supports	-2.60	0.42	5.07	15	266.67	Verified
Spans	4.76	0.23	2.77			Verified

III.7.4.3 Verification of shear force at ULS

Table III. 56. Shear force of the machine slab.

Section (cm)	T(kN)	$r(MPa)$	$\bar{r}(MPa)$	$r \leq \bar{r}$
25x100	-86.04	0.38	1.16	Verified

III.7.4.4 Verification of the deflection

Table III. 57. Deflection of the machine slab.

Span moment Kn.m	Support moment Kn.m	[F] (mm)	$[\bar{F}]$ (mm)	$[F] < [\bar{F}]$
1.69	-13.73	4.6	9.6	Verified

III.7.4.5 Verification of the punching shear Art A.5.2.4.2

If the following condition is verified, no additional provision is necessary.

$$Q_u \leq F_{limit} = 0.045 * U_c * h \frac{F_{c28}}{\gamma_b}$$

$$U_c = (a + b + 2h) \times 2$$

With:

h: The total height of the slab.

a, b: Size of the application surface of the concentrated load.

Table III. 58. Verification of the punching shear for the machine slab.

Q_u (kN)	h (cm)	f_{c28} (Mpa)	γ_b	a (cm)	b (cm)	U_c (cm)	F_{limite} (kN)	$Q_u \leq F_{limit}$
49.5	25	25	1.5	25	25	200	375	Verified

III.8 Conclusion

To resume this chapter, we will mention the important results extracted for each element in the following lines:

- The acroterion: it was with a standard dimensions (10x60)cm²;
- The compression slab took a thickness of 04cm;
- The slab beams verified all the conditions and their reinforcement adopted is:
  - Span: 2HA10;
  - Supports: 1HA10+ 1ch HA14.
- The stairs are with a thickness of e=17cm; and for the landing beam we adopted a section of (30x35)cm²;
- The solid slab verified all the conditions with a thickness of e=15cm;
- The elevator needs a machine slab with a depth of e =25cm.

CHAPTER IV

Dynamic analysis

IV.1 Introduction

The dynamic analysis is mainly concerned with finding out the behavior of a structure when subjected to a seismic action. This analysis requires initially the creation of a calculation model that represents our structure in the best way; the model will be executed on the software **SAP22** to determine the dynamic characteristics of the structure.

IV.2 Geometrical characteristics of the structure

The geometrical features involved in the evaluation of horizontal forces of seismic origin are:

- The center of Masse.
- The center of Torsion.

IV.2.1 Center of masse

The masses are considered concentrated at the floor level, so it is necessary to know the mass centers that represent the application points of the lateral seismic forces. The center of mass will be determined for each level by considering all the elements which have an influence on the stability of the building (columns, beams, walls, stairs, shell walls, etc.)

$$X_G = \frac{\sum(W_i.X_i)}{\sum W_i} \quad ; \quad Y_G = \frac{\sum(W_i.Y_i)}{\sum W_i}$$

With:

W_i: The total weight of the concentrated mass of element (i) of the level concerned.

Xi, Yi: The coordinates of the center of gravity of the element (i) relative to the global coordinates.

X_G, Y_G: The Coordinates of the mass center of the level concerned relative to the global coordinates.

IV.2.2 Center of torsion

This is the point through which if a horizontal force is applied only a translation movement occurs without rotation of the floor. For each floor, the center of torsion of the floor is determined, taking into account the bracing elements: columns and shear walls.

Our building is irregularly shaped, so the center of torsion does not coincide with the center of gravity which result a risk of torsion around the vertical axis. To reduce this risk, an arrangement of reinforced concrete sails with interaction with the frames is considered as a bracing system.

The coordinates of the torsion center are given by the following formula:

$$X_G = \frac{\sum (K_{ix}.Y_i)}{\sum K_{ix}} \; ; \; Y_G = \frac{\sum (K_{iy}.X_i)}{\sum K_{iy}}$$

K_X, K_Y: stiffness along (x) and (y) axes respectively of each Bracing element.

Xi, Yi: coordinates of the gravity center for element (i) relative to the global coordinates.

X_G, Y_G: Coordinates of the mass center of the level concerned relative to the global coordinates.

IV.2.3 Eccentricity

The eccentricity is the distance between the center of mass and the center of torsion along the two axes (X) and (Y), it iscalculated as follows:

$$E_{X\,theo} = |X_G - X_R| \quad , \quad E_{Y\,theo} = |Y_G - Y_R| \qquad \textbf{[2] Art 4.2.7}$$

According to the article 4.3.7 of RPA2003, we must add to the value of eccentricity in bothdirections an additional eccentricity which is equal to:

$$E_{X\,add} = 5\% \times L_X \quad , \quad E_{Y\,add} = 5\% \times L_Y \qquad \textbf{Art 4.3.7}$$

L: The dimension of the floor perpendicular to the direction of the seismic action.

Then the values of eccentricity become:

$$E_X = E_{X\,theo} + E_{X\,add} \quad , \quad E_Y = E_{Y\,theo} + E_{Y\,add}$$

At each level and for both directions, the eccentricity does not exceed 15% of the dimension of the building measured perpendicular to the direction of the seismic action considered. **Art 3.5.1-a**

➢ Auto stable structure

Before using the bracing system (frames+ shear walls), we calculated the eccentricity and studied the behavior of the structure as auto-stable; the results are mentioned in the following table.

Table IV. 1. Eccentricity of the auto stable structure on both directions.

Eccentricity		
Level(m)	**Ex = |XR-XG| (m)**	**Ey = |YR-YG| (m)**
3.06	3.050	3.607
6.12	2.204	2.898
9.18	2.137	2.808
12.24	2.111	2.781
15.30	2.096	2.765
18.36	2.083	2.754
21.42	2.075	2.752
24.48	2.073	2.732
27.54	2.070	2.757

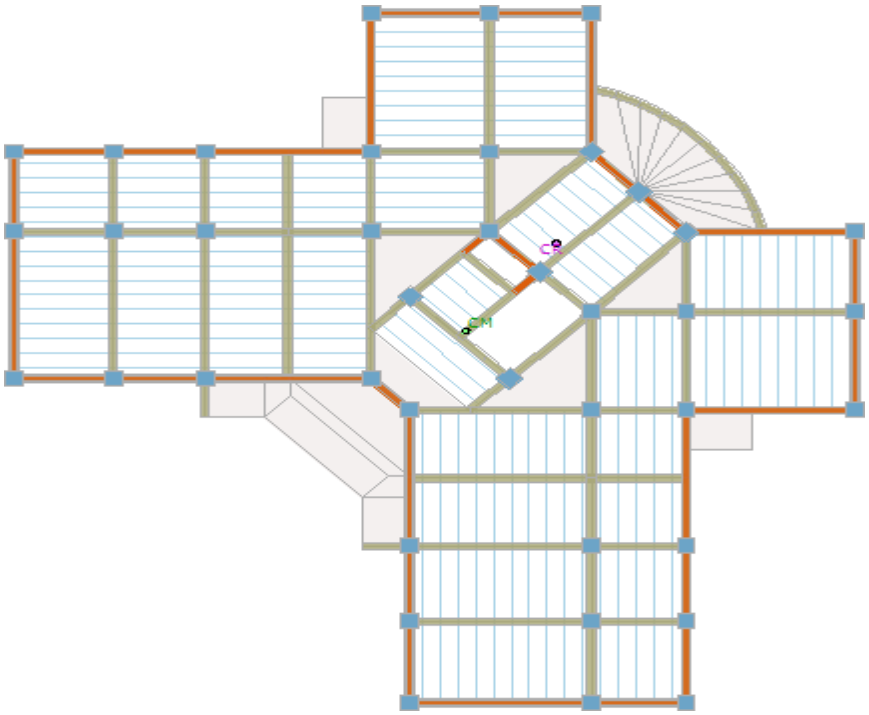
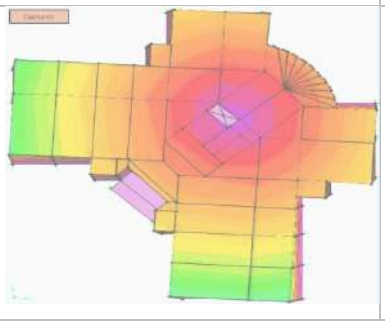
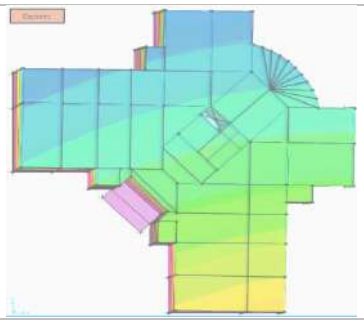
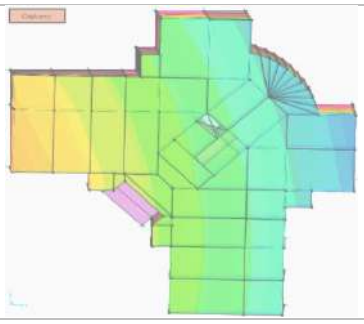


Figure IV. 1. The eccentricity of the auto-stable structure.

As we can see, the eccentricity is big between the mass center and the torsion center, which gives us torsion around the vertical axis and affects the behavior of the structure as shown in the next table.

Table IV. 2. Modal behavior of the auto-stable structure.

Modes	**Mode 1 / T1 = 1.018 s**	**Mode 2 / T2 = 0.895 s**	**Mode 3 / T3 = 0.792 s**
Direction	**Mass participation %**	**Mass participation %**	**Mass participation %**
Along X	4.45	59.03	2.99
Along Y	2.47	4.97	59.08
Around Z	60.12	2.59	4.00
Graphique			
Behavior	Rotation around axis Z	Translation along axis X	Translation along axis Y

The fundamental mode is T=1.018s and the behavior is a rotation because of the lever arm which created a torsion moment.

We tried so many dispositions for the bracing system but some conditions were not verified and the last one gave us the best results (small eccentricity, short period of dissipation…); so we adopted it.

➢ Disposition N°01

Table IV. 3. Eccentricity of disposition 01 on both directions.

Eccentricity		
Level(m)	**Ex = |XR-XG| (m)**	**Ey = |YR-YG| (m)**
3.06	0.541	0.823
6.12	0.140	0.448
9.18	0.034	0.096
12.24	0.142	0.064
15.30	0.216	0.158
18.36	0.268	0.220
21.42	0.305	0.264
24.48	0.339	0.277
27.54	0.382	0.245

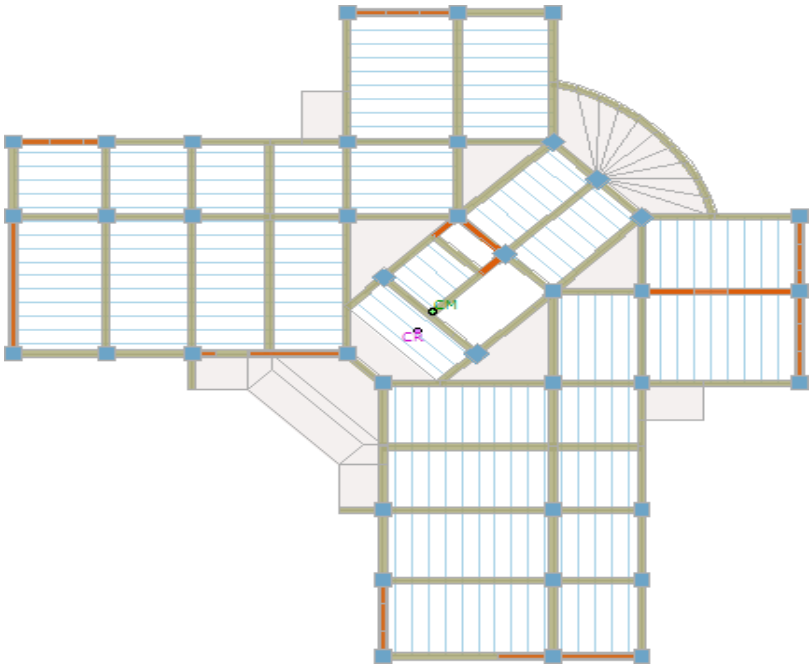
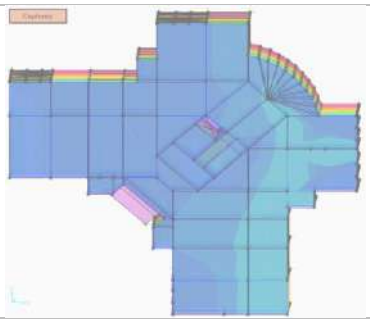
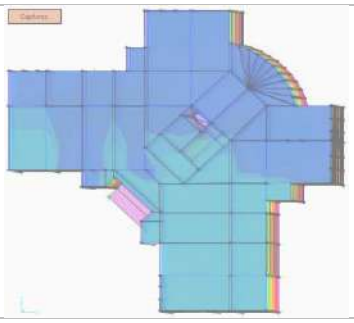
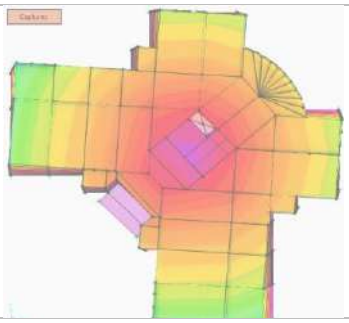


Figure IV. 2. The eccentricity of disposition 01.

Table IV. 4. Modal behavior of disposition 01.

Modes	**Mode 1 / T1 = 0.591 s**	**Mode 2 / T2 = 0.532 s**	**Mode 3 / T3 = 0.412 s**
Direction	**Mass participation %**	**Mass participation %**	**Mass participation %**
Along X	1.45	60.57	0.00
Along Y	60.45	1.46	0.00
Around Z	0.00	0.01	60.64
Graphique			
Behavior	Translation along axis Y	Translation along axis X	Rotation around axis Z

➢ Disposition N°02

Table IV. 5. Eccentricity of disposition 02 on both directions.

Eccentricity		
Level(m)	**Ex = |XR-XG| (m)**	**Ey = |YR-YG| (m)**
3.06	0.981	3.417
6.12	0.356	2.373
9.18	0.151	2.084
12.24	0.036	1.925
15.30	0.041	1.816
18.36	0.094	1.737
21.42	0.134	1.678
24.48	0.171	1.653
27.54	0.207	1.638

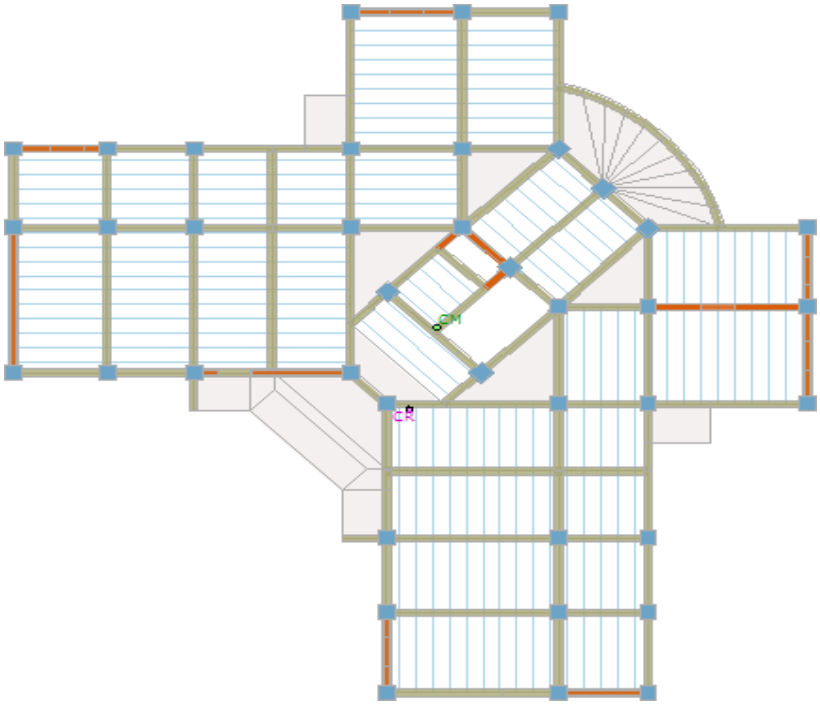
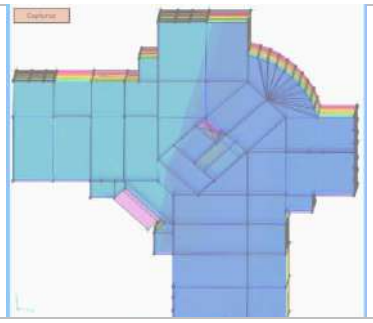
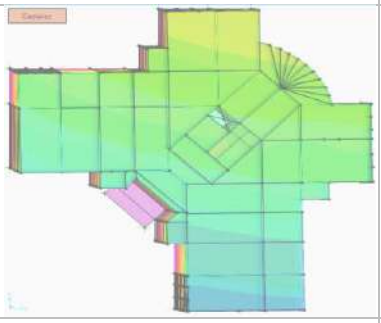
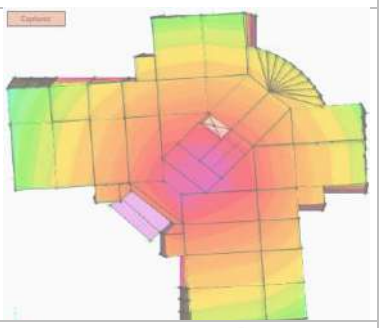


Figure IV. 3. The eccentricity of disposition 02.

Table IV. 6. Modal behavior of disposition 02.

Modes	**Mode 1 / T1 = 0.591 s**	**Mode 2 / T2 = 0.559 s**	**Mode 3 / T3 = 0.430 s**
Direction	**Mass participation %**	**Mass participation %**	**Mass participation %**
Along X	3.78	56.44	1.92
Along Y	58.07	3.77	0.03
Around Z	0.04	1.73	58.85
Graphique			
Behavior	Translation along axis Y	Translation along axis X	Rotation around axis Z

➢ Disposition N°03

Table IV. 7. Eccentricity of disposition 03 on both directions.

Eccentricity		
Level(m)	**Ex = |XR-XG| (m)**	**Ey = |YR-YG| (m)**
3.06	0.970	2.005
6.12	0.541	0.990
9.18	0.359	0.746
12.24	0.252	0.638
15.30	0.182	0.573
18.36	0.134	0.534
21.42	0.098	0.508
24.48	0.071	0.514
27.54	0.117	0.530

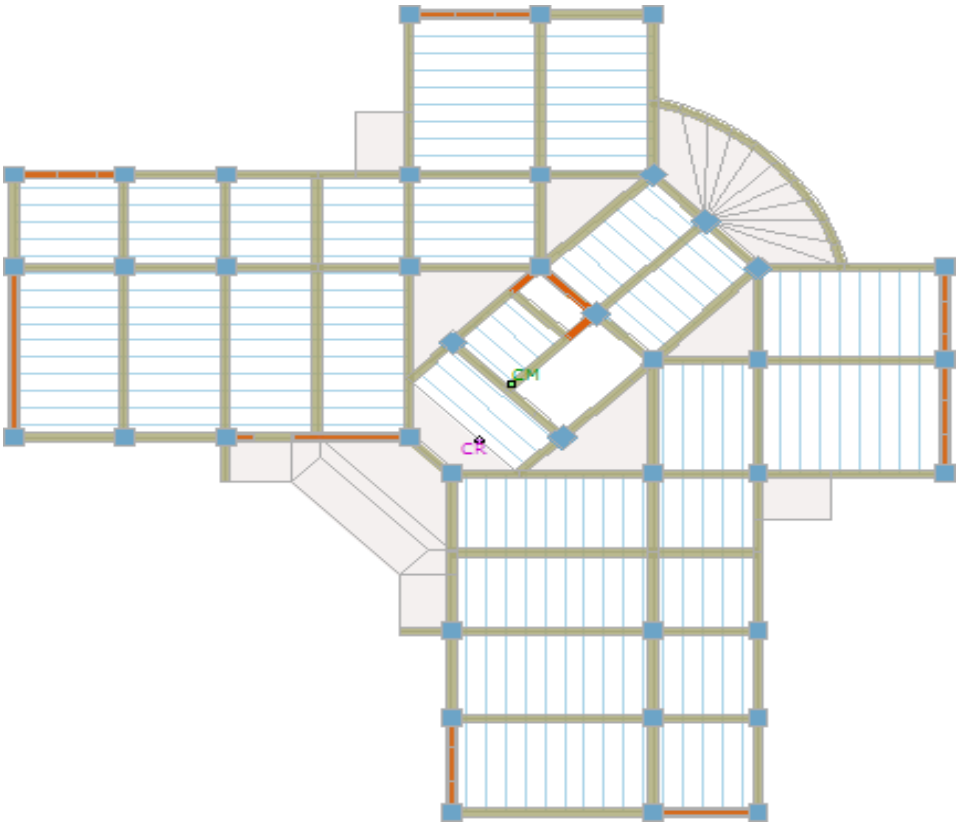
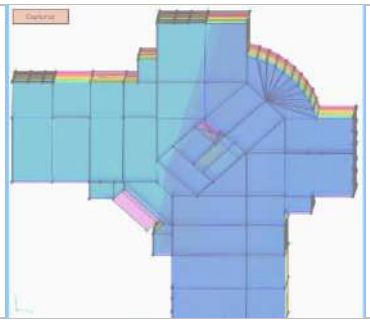
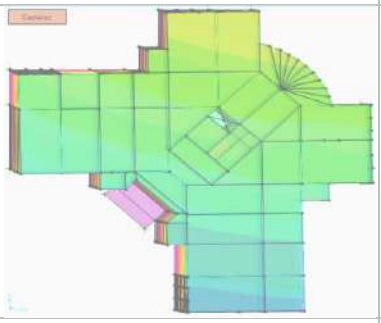
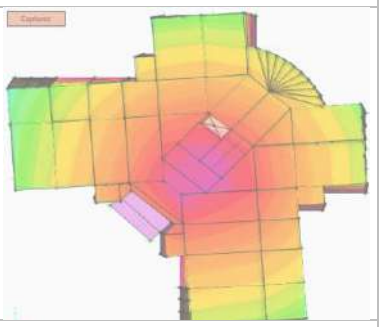


Figure IV. 4. The eccentricity of disposition 03.

Table IV. 8. Modal behavior of disposition 03.

Modes	Mode 1 / T1 = 0.591 s	Mode 2 / T2 = 0.559 s	Mode 3 / T3 = 0.430 s
Direction	**Mass participation %**	**Mass participation %**	**Mass participation %**
Along X	3.78	56.44	1.92
Along Y	58.07	3.77	0.03
Around Z	0.04	1.73	58.85
Graphique			
Behavior	Translation along axis Y	Translation along axis X	Rotation around axis Z

➢ Adopted disposition

Table IV. 9. Center of Mass and Center of torsion of the adopted disposition.

Level (m)	Inertia and center of torsion				Mass and center of mass		
	$\sum I_x$ (m^4)	$\sum I_y$ (m^4)	X_R (m)	Y_R (m)	W (kN)	X_G (m)	Y_G (m)
3.06	1.0E+5	7.32E+4	14.17	14.87	7572.96	15.15	15.20
6.12	1.0E+5	7.32E+4	14.41	14.98	6364.24	14.95	15.01
9.18	1.0E+5	7.32E+4	14.63	15.29	6301.80	14.95	15.01
12.24	1.0E+5	7.32E+4	14.75	15.44	6243.53	14.95	15.01
15.30	1.0E+5	7.32E+4	14.83	15.52	6243.53	14.95	15.01
18.36	1.0E+5	7.32E+4	14.89	15.57	6187.04	14.95	15.02
21.42	1.0E+5	7.32E+4	14.93	15.61	6134.32	14.95	15.02
24.48	1.0E+5	7.32E+4	14.97	15.64	5876.21	14.94	15.04
27.54	1.0E+5	7.32E+4	14.99	15.66	4494.39	14.90	15.11

Table IV. 10. The eccentricity on both directions of the adopted disposition.

Level (m)	Eccentricity according to X			Eccentricity according to Y		
	$E_X\ (m)$	15 %. L_x	Verification	$E_Y\ (m)$	15 %. L_y	Verification
3.06	0.984	4.26	Verified	0.332	4.26	Verified
6.12	0.541	4.26	Verified	0.029	4.26	Verified
9.18	0.321	4.26	Verified	0.282	4.26	Verified
12.24	0.197	4.26	Verified	0.422	4.26	Verified
15.30	0.116	4.26	Verified	0.503	4.26	Verified
18.36	0.058	4.26	Verified	0.555	4.26	Verified
21.42	0.016	4.26	Verified	0.591	4.26	Verified
24.48	0.024	4.26	Verified	0.596	4.26	Verified
27.54	0.094	4.26	Verified	0.553	4.26	Verified

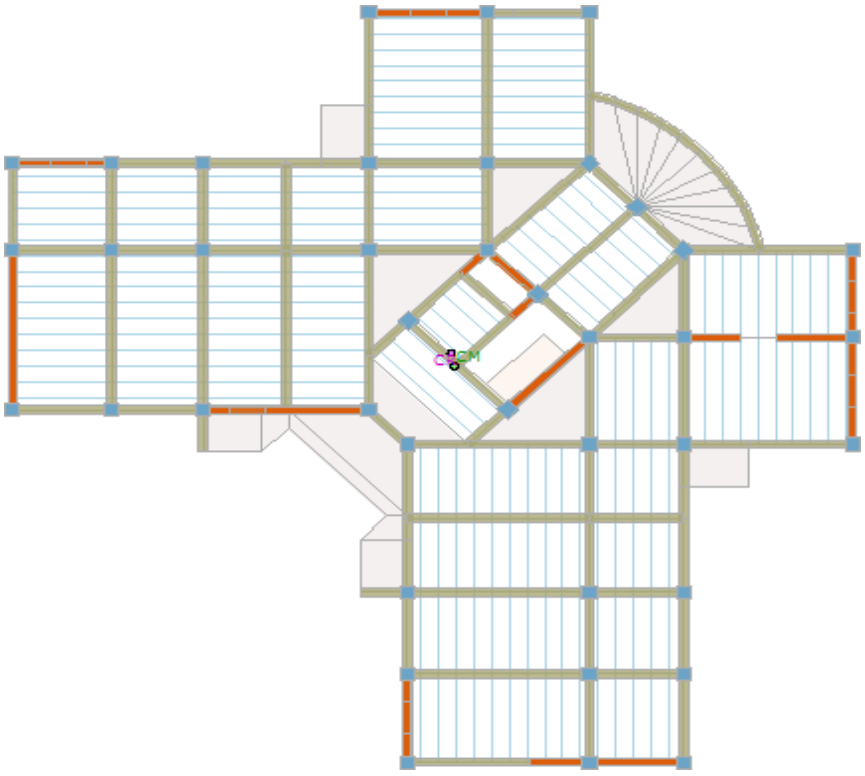


Figure IV. 5. The eccentricity of the adopted disposition.

This final disposition gave us the best eccentricity (small distance between mass center and torsion center) that is why the rotation is in the third mode. We will verify all the other conditions in the following pages.

IV.3 Choice of the calculation method

IV.3.1 Introduction

We always admit that horizontal forces are applied to the floors. To analyze these efforts, wegenerally use three methods:

- Equivalent static method;
- Method of spectral modal analysis;
- Method of dynamic analysis by accelerograms.

For our project, the conditions for applying the equivalent static method are not all verified (Irregular shape, zone III, group 2, and the height of our building exceeds 8.5m), So we have to use the dynamic calculation with the spectral modal analysis method.

We used the software **SAP2000 v22.2.0** in the calculation.

IV.3.2 The spectral modal analysis method

IV.3.2.1 Principle

This method seeks, for each mode of vibration, the maximum of the effects generated in the structure by the seismic forces represented by a computational response spectrum. These effects are then combined to obtain the response of the structure.

IV.3.2.2 Modelling

The model chosen in the modeling on the **"SAP2000 v22.2.0"** is the rigid diaphragm. Following the concentration of the mass, its movement is given by the movement of the floor.

IV.3.2.3 The response spectrum of calculation

The seismic action is represented by the following calculation spectrum:

$$\frac{S_a}{g} = \begin{cases} 1.25A\left[1+\frac{T}{T_1}\left(2.5\eta\frac{Q}{R}-1\right)\right] & pour: 0 \le T \le T_1 \\ 2.5\eta(1.25A)\frac{Q}{R} & pour: T_1 \le T \le T_2 \\ 2.5\eta(1.25A)\frac{Q}{R}\left(\frac{T_2}{T}\right)^{2/3} & pour: T_2 \le T \le 3sec \\ 2.5\eta(1.25A)\frac{Q}{R}\left(\frac{T_2}{T}\right)^{2/3}\left(\frac{3}{T}\right)^{2/3} & pour: T > 3sec \end{cases}$$

a) Acceleration coefficient A [2] Art 4.2.3 Tab 4.1

A: is in function of the zone and the group of use, in our project (Zone III, use group 2) ⇒ A = 0.25.

b) Damping correction factor η [2] Art 4.2.3 Tab 4.2

$\eta = \sqrt{\frac{7}{2+\xi}}$; Avec $\xi = 07 \Rightarrow \eta = 0.882$.

c) Behavior coefficient R [2] Art 4.2.3 Tab 4.3

The shear walls of our building take up more than 20% of the stresses due to vertical loads,which obliges us to consider that the horizontal stresses are taken up only by the shear walls.

So: **R = 3.5**

Table IV. 11. Distribution of externally applied effect according to X.

Level (m)	Externally applied effect due to vertical		Externally applied effect due to horizontal	
	$N_{z\,(columns)}$ (%)	$N_{z\,(walls)}$ (%)	$T_{x\,(columns)}$ (%)	$T_{x\,(walls)}$ (%)
27.54	66.446	33.554	00	100
24.48	67.716	32.284	00	100
21.42	68.326	31.674	00	100
18.36	70.277	29.723	00	100
15.30	70.554	29.446	00	100
12.24	71.060	28.940	00	100
9.18	73.118	26.882	00	100
6.12	74.754	25.246	00	100
3.06	51.509	48.491	00	100

Table IV. 12. Distribution of externally applied effect according to Y.

Level (m)	Externally applied effect due to vertical		Externally applied effect due to horizontal	
	$N_{z\,(columns)}$ (%)	$N_{z\,(walls)}$ (%)	$T_{y\,(columns)}$ (%)	$T_{y\,(walls)}$ (%)
27.54	73.929	26.071	00	100
24.48	74.665	25.335	00	100
21.42	75.076	24.924	00	100
18.36	76.758	23.242	00	100
15.30	76.877	23.123	00	100
12.24	77.175	22.825	00	100
9.18	78.721	21.279	00	100
6.12	78.717	21.283	00	100
3.06	54.480	45.520	00	100

d) Quality factor Q **[2] Art 4.2.3 Tab 4.4**

The value of Q is determined by the following formula:

$$Q = 1 + \sum_{1}^{6} P_q$$

P_q:The penalty is based on whether the quality criterion "q" is met or not.

Table IV. 13. Quality factor

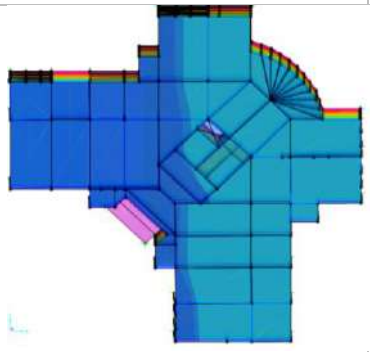
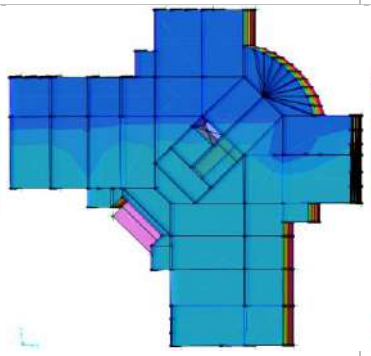
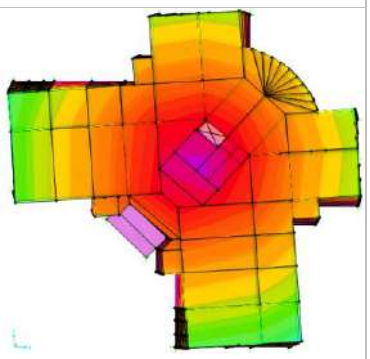
Criterion (q)	According to X		According to Y	
	Observed	Value	Observed	Value
Minimum conditions on the bracing lines	Yes	0	Yes	0
Plan redundancy	Yes	0	Yes	0
Regularity in plan	No	0.05	No	0.05
Regularity in elevation	Yes	0	Yes	0
Quality control of materials	Yes	0	Yes	0
Execution quality control	Yes	0	Yes	0
Total	0.05		0.05	
Q	1.05		1.05	

IV.3.3 Results of dynamic analysis

IV.3.3.1 Modal analysis

IV.3.3.1.1 Modal behavior [Art 4.2.7]

Table IV. 14. Modal behavior of our structure.

Modes	**Mode 1 / T1 = 0.556 s**	**Mode 2 / T2 = 0.508 s**	**Mode 3 / T3 = 0.411 s**
direction	**Mass participation %**	**Mass participation %**	**Mass participation %**
Direction X	0.02	62.35	0.08
Direction Y	61.98	0.01	0.09
Around Z	0.06	0.11	60.49
Figure			
Behavior	Translation along Y axis	Translation along X axis	Rotation around Z axis

T: Fundamental period in both directions. ⇒ $T=\frac{0.09.h_n}{\sqrt{D}}$ **[2] Art 4.2.4.4**

T1=0.556s < T=1.669s → the period of the first mode(fundamental) is verified.

Comparing the two systems, the period of this system is less than the period of the auto-stable system due to the global stiffness (columns and shear walls); and the rotation is in the third mode because the lever arm is short.

IV.3.3.1.2 Mass participation (dissipation of energy)

For structures represented by plane models in two orthogonal directions, the number of vibration modes to be retained in each of the two excitation directions shall be such that: The sum of the effective modal masses for the modes selected is equal to at least 90% of the total mass of the structure according to: **(RPA99 v2003 Art.4.3.4.a).**

Table IV. 15. Mass participation of our structure.

Mode N°	Period (s)	Cumulative of effective modal masses (%)		
		Ux	Uy	Uz
1	0.556	0.016	61.979	0.065
2	0.508	62.364	61.991	0.176
3	0.411	62.446	62.084	60.67
…	…	…	…	…
184	0.03	89.934	90.814	88.185
…	…	…	…	…
192	0.028	90.001	90.889	91.775

Our structure dissipated more than 90% in 192 modes with a time of 13.94 seconds.

IV.3.3.2 Static and dynamic analysis

IV.3.3.2.1 Structure stability

A. Verification of the resultant of the seismic design forces ([2] Art.4.3.6)

The resultant of the seismic forces at base V_t obtained by combining the modal values shall not be less than 80% of the resultant of the seismic forces determined by the equivalent static method V for a fundamental period value given by the empirical formula appropriate.

So, it is necessary to verify that:

$$\frac{E_t}{V} \geq 80\%$$

➢ **Calculation of the total seismic force V ([2] Art.4.2.3)**

The total seismic force V must be calculated in the two directions X and Y according to the following formula:

$$V_x = \frac{A.D_x.Q_x}{R} W \quad ; \quad V_y = \frac{A.D_y.Q_y}{R} W$$

With:

A : acceleration coefficient (Zone III, Groupe 2) ⇒ A=0.25

R : behavior coefficient ⇒ R=3.5

Q : quality factor ⇒ $Q_x=Q_y=1.05$

D: Average dynamic amplification factor.

T_2: Characteristic period. (Site 3) ⇒ $T_2 = 0.5\ sec$

$$\eta = 0.882$$

T: Fundamental period in both directions. ⇒ $T=\frac{0.09.h_n}{\sqrt{D}}$ **[2] Art 4.2.4.4**

$$\begin{cases} T_x=\frac{0.09*27.54}{\sqrt{2.205}}=1.669s \\ T_y=\frac{0.09*27.54}{\sqrt{2.205}}=1.669s \end{cases} \quad ; D_x=D_y=2.205$$

W : total weight of the structure.

$W=\sum_{i=1}^{n} W_i=\sum_{i=1}^{n}(W_{G\,i}+\beta.W_{Q\,i})$; $\beta=0.2$ **[2] Table 4.5**

Table IV. 16. The values of seismic force.

A	D_x	D_y	Q_x	Q_y	R	$W(kN)$	$V_x(kN)$	$V_y(kN)$
0.25	2.205	2.205	1.05	1.05	3.5	64878.78	10704.99	10704.99

The seismic force given by the spectral function have been calculated by the software "SAP2000 V.22.2.0":

$$E_{tx} = 7731.2\ kN \quad , \quad E_{ty} = 7352.54\ kN$$

Table IV. 17. Verification of the result of the seismic design forces.

Direction	$V(kN)$	$E_t(kN)$	E_t/V	$E_t/V \geq 0.8$
X	10704.99	7731.2	0.73	Not verified
Y	10704.99	7352.54	0.69	Not verified

In order to verify this condition $\frac{V_t}{V} \geq 80\%$; we use a scale factor SF in the seismic force given by the spectral function.

$$SF = 0.8 * \frac{V}{V_t}\ ; \rightarrow SF_x = \frac{0.8}{0.73} = 1.10 \text{ and } SF_y = \frac{0.8}{0.69} = 1.16$$

B. Verification of the overturning **[2] Art 4.4.1**

- The overturning moment is equal to:

$$M_R=\sum_{i=1}^{n} M_{ri}=\sum_{i=1}^{n}(h_i.F_i)$$

With:

F_i: The forces distributed at each level i;

h_i: The height of the floor relative to the base.

- The stability moment equals:

$$M_s = \sum_{i=1}^{n} M_i = \sum_{i=1}^{n} (X_{Gi}.W_i)$$

With:

X_G: The coordinates of the center of gravity of the structure;
W_i: The mass of the floor of each level.

Table IV. 18. Values of stability moments and overturning moments.

h_i (m)	W_i (kN)	F_{ix} (kN)	F_{iy} (kN)	M_{Rxi} $(kN.m)$	M_{Ryi} $(kN.m)$	M_{Sxi} $(kN.m)$	M_{Syi} $(kN.m)$
27.54	4494.39	12621.58	1320.25	1323.46	36359.60	66966.41	67910.23
24.48	5876.21	14668.58	1534.37	1538.11	37561.33	87790.58	88378.20
21.42	6134.32	13398.78	1401.54	1404.96	30021.07	91708.08	92137.49
18.36	6187.04	11583.37	1211.65	1214.60	22245.85	92496.25	92929.34
15.30	6243.53	9740.95	1018.93	1021.41	15589.57	93340.77	93715.39
12.24	6243.53	7792.76	815.14	817.13	9977.33	93340.77	93715.39
9.18	6301.80	5899.11	617.06	618.56	5664.62	94211.91	94590.02
6.12	6364.24	3971.71	415.45	416.46	2542.56	95145.39	95527.24
3.06	7572.96	2363.01	247.18	247.78	756.36	114730.34	115108.99

In order to have a construction that resists to the overturning, the next condition must be verified:

$$\frac{M_S}{M_R} \geq 1.5$$

Table IV. 19. Verification of the overturning.

	M_S	M_R	M_S/M_R	**Verification**
X	829730.51	160718.29	5.16	Verified
Y	834012.28	161109.99	5.17	Verified

We can say that our structure is stable against the overturning.

C. Verification of inter-story displacement

The horizontal displacement at each level k of the structure is calculated as follows: **Art.4.43**

$$\delta_k = R * \delta_{ek}$$

With:

δ_r: the reel displacement on the head of the structure due to a seismic force;

δ_{th}: the theoretic displacement;

R: behavior coefficient.

The relative displacement to the level k_i compared to the level $(_{i-1})$ (inter-storey) equals to:

$$\Delta_k = \left(\delta_k - \delta_{(k-1)}\right) * R$$

The relative displacement of one storey compared to other adjacent stories must not exceed the admissible displacement of:

$$\Delta_{adm} = 0.01 h_e \quad \textbf{[2] Art.5.10}.$$

Table IV. 20. Verification of inter-storey displacements along X.

Level (m)	h_e (m)	δ_{ek} (cm)	δ_k (cm)	Δ_k (cm)	Δ_{adm} (cm)	$\Delta_k \leq \Delta_{adm}$	δ	δ_{adm}	$\delta \leq \delta_{adm}$
27.54	3.06	2.29	8.02	1.11	3.06	Verified	2.54	3.50	Verified
24.48	3.06	1.98	6.91	1.15	3.06	Verified	2.64	3.50	Verified
21.42	3.06	1.65	5.76	1.18	3.06	Verified	2.69	3.50	Verified
18.36	3.06	1.31	4.58	1.16	3.06	Verified	2.65	3.50	Verified
15.30	3.06	0.98	3.42	1.10	3.06	Verified	2.51	3.50	Verified
12.24	3.06	0.67	2.33	0.97	3.06	Verified	2.22	3.50	Verified
9.18	3.06	0.39	1.36	0.78	3.06	Verified	1.79	3.50	Verified
6.12	3.06	0.16	0.57	0.51	3.06	Verified	1.17	3.50	Verified
3.06	3.06	0.02	0.06	0.06	3.06	Verified	0.14	3.50	Verified

Table IV. 21. Verification of inter-storey displacements along Y.

Level (m)	h_e (m)	δ_{ek} (cm)	δ_k (cm)	Δ_k (cm)	Δ_{adm} (cm)	$\Delta_k \leq \Delta_{adm}$	δ	$\delta\, adm$	$\delta \leq \delta\, adm$
27.54	3.06	2.78	9.75	1.43	3.06	Verified	3.27	3.50	Verified
24.48	3.06	2.38	8.32	1.45	3.06	Verified	3.33	3.50	Verified
21.42	3.06	1.96	6.87	1.46	3.06	Verified	3.33	3.50	Verified
18.36	3.06	1.55	5.41	1.41	3.06	Verified	3.24	3.50	Verified
15.30	3.06	1.14	4.00	1.32	3.06	Verified	3.01	3.50	Verified
12.24	3.06	0.77	2.68	1.14	3.06	Verified	2.62	3.50	Verified
9.18	3.06	0.44	1.54	0.90	3.06	Verified	2.06	3.50	Verified
6.12	3.06	0.18	0.63	0.57	3.06	Verified	1.31	3.50	Verified
3.06	3.06	0.02	0.06	0.06	3.06	Verified	0.14	3.50	Verified

D. Verification of the P-Δ effect [2] Art.5.9

The effects of the second order (or the P-Δ effect) can be ignored if the following condition issatisfied at all levels:

$$\theta = \frac{P_k . \Delta_k}{V_k . h_k} \leq 0.10$$

P_k : Total weight of the structure and associated live load above the level "k";

V_k : Shearing load from the floor at level "k";

Δ_k: Relative displacement of level "k" compared to level "k-1";

h_k : Height of floor "k".

Table IV. 22. Verification of the P-Δ effect.

Level (m)	P_k *(kN)*	Δ_{kx} *(cm)*	Δ_{ky} *(cm)*	V_{kx} *(kN)*	V_{ky} *(kN)*	h_k *(cm)*	θ_{kx}	θ_{ky}	$\theta \leq 0.1$
27.54	4494.39	1.11	1.43	1320.25	1323.47	306	0.012	0.016	Verified
24.48	5876.21	1.15	1.45	1534.37	1538.11	306	0.014	0.018	Verified
21.42	6134.32	1.18	1.46	1401.54	1404.96	306	0.017	0.021	Verified
18.36	6187.04	1.16	1.41	1211.65	1214.6	306	0.019	0.023	Verified
15.30	6243.53	1.09	1.32	1018.93	1021.41	306	0.022	0.026	Verified
12.24	6243.53	0.97	1.14	815.14	817.13	306	0.024	0.028	Verified
9.18	6301.8	0.79	0.91	617.06	618.56	306	0.026	0.03	Verified
6.12	6364.24	0.51	0.56	415.45	416.46	306	0.026	0.028	Verified
3.06	7572.96	0.06	0.07	247.18	247.78	306	0.006	0.007	Verified

The condition is verified which means that the P-Δ effect is ignored.

IV.3.3.2.2 Resistance of the elements

A. Verification of the shear forces [Art 7.4.3.2]

- **Beams**

For every type of beams, we took one beam that receives the biggest shear force; and if this beam verifies the next condition it means that the others beams will verify it as well.

$$\tau = \frac{T_{max}}{b.d} \leq \tau_{adm}$$

Table IV. 23. Beam shear verification.

Elements	Section (cm^2)	V_u (kN)	Combination	r (MPa)	Type of cracking	$\bar{r}(MPa)$	$r \leq \bar{r}$
Main beams	30x45	162.14	G+Q+Edx	1.27	prejudicial	3.26	Yes
Secondary beams	30x35	108.96	G+Q+Edx	1.12	prejudicial	3.26	Yes
Landing beams	30x35	64.45	G+Q-Edx	0.66	prejudicial	3.26	Yes
Foundation beam	30x30	110.59	1.35G+1.5Q	1.34	prejudicial	2.50	Yes
Ring beams	30x30	110.59	1.35G+1.5Q	1.34	Little harmful	3.33	Yes

- **Columns**

The shearing force in the columns is zero (T=00%) because we considered that the horizontal loads are taken up only by the shear walls (R = 3.5).

- **Shear walls**

The shear resistance is checked with the value of the shear force found at the base of the sail plus 40%.

$$\tau = \frac{1.4\,|V_u|}{a.d} \leq \bar{\tau} = 0.2\,F_{c28}$$

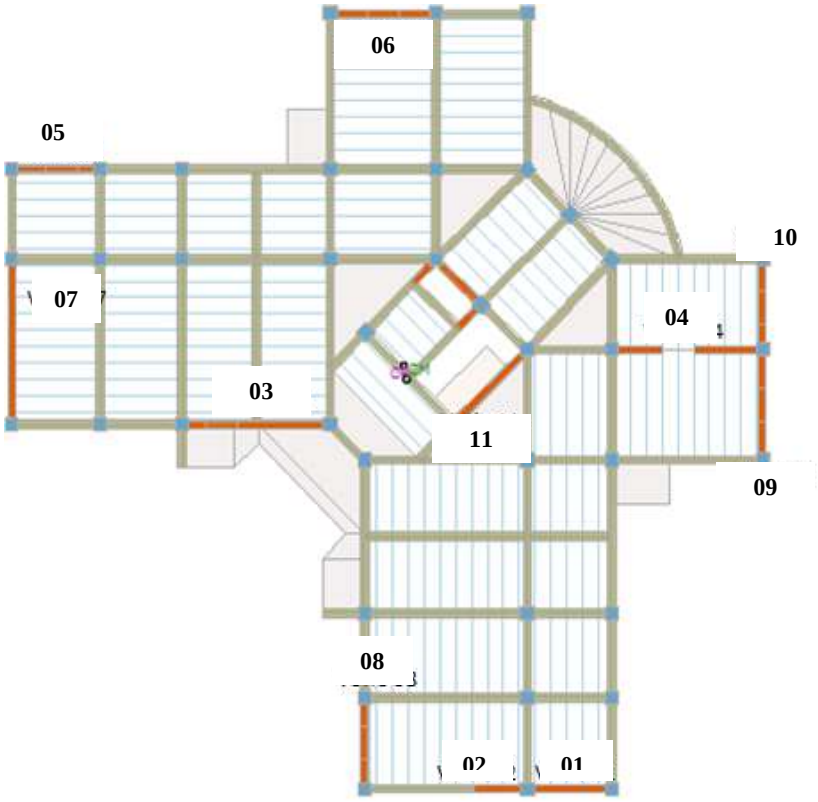


Figure IV. 6. Numeration of the shear walls.

Table IV. 24. Verification of shear force for shear walls.

Direction	Wall N°:	b(cm)	L(m)	$\|V_u\|$	Combination	r (MPa)	$\bar{r}$ (MPa)	$r \leq \bar{r}$
X	**01**	25	3.20	981.80	G+Q-Edx	1.9	5	Verified
	02	25	2.00	1155.27	G+Q+Edx	3.5	5	Verified
	03	25	5.60	1938.29	G+Q+Edx	2.15	5	Verified
	04	25	5.70	1995.63	G+Q-Edx	2.18	5	Verified
	05	25	3.35	834.73	G+Q+Edx	1.55	5	Verified
	06	25	4.00	1323.24	G+Q+Edx	2.06	5	Verified
Y	**07**	25	6.05	3225.31	G+Q-Edy	3.32	5	Verified
	08	25	3.35	975.75	G+Q+Edy	1.8	5	Verified
	09	25	3.30	1648.20	G+Q-Edy	3.1	5	Verified
	10	25	4.05	0.00	G+Q+Edy	00	5	Verified
	11	25	3.89	555.08	G+Q-Edy	0.88	5	Verified

B. Verification of the reduced normal load [Art 7.4.3.1]

To verify the column crashing, the next condition must be satisfied:

$$\nu = \frac{N_d}{B_c f_{c28}} \leq 0.3$$

With:

N_d: Reduced normal load;

B_c: Cross section of concrete.

The column sections adopted in the pre-sizing chapter (second chapter) did not satisfy the verifications, therefore we adopted new ones (mentioned in the following table) in order to have a good resistance.

➢ **Direction X**

Table IV. 25. Verification of reduced normal load direction X.

level	b(cm)	h(cm)	$N_d(kN)$	$B_c(cm^2)$	Combination	ν	$\nu \leq 0.3$
Basement	60	60	2169.04	3364	$G + Q - Ed_x$	0.26	Yes
ground	55	55	2078.29	2809	$G + Q - Ed_x$	0.29	Yes
1st	55	55	1797.89	2809	$G + Q - Ed_x$	0.25	Yes
2nd	50	50	1520.11	2304	$G + Q - Ed_x$	0.26	Yes
3rd	50	50	1248.75	2304	$G + Q - Ed_x$	0.22	Yes
4th	50	50	979.75	2304	$G + Q - Ed_x$	0.17	Yes
5th	45	45	712.51	1849	$G + Q - Ed_x$	0.15	Yes
6th	45	45	450.56	1849	$G + Q - Ed_x$	0.09	Yes
7th	45	45	239.4	1849	$G + Q - Ed_x$	0.05	Yes

The new adopted sections verified the conditions and we have no risk of crashing on the columns in the direction X.

➢ **Direction Y**

Table IV. 26. Verification of reduced normal load direction Y.

level	b(cm)	h(cm)	$N_d(kN)$	$B_c(cm^2)$	Combination	ν	$\nu \leq 0.3$
Basement	60	60	2169.04	3364	$G + Q - Ed_x$	0.26	Yes
ground	55	55	2078.29	2809	$G + Q - Ed_x$	0.29	Yes
1st	55	55	1797.89	2809	$G + Q - Ed_x$	0.25	Yes
2nd	50	50	1520.11	2304	$G + Q - Ed_x$	0.26	Yes
3rd	50	50	1248.75	2304	$G + Q - Ed_x$	0.22	Yes
4th	50	50	979.75	2304	$G + Q - Ed_x$	0.17	Yes
5th	45	45	712.51	1849	$G + Q - Ed_x$	0.15	Yes
6th	45	45	450.56	1849	$G + Q - Ed_x$	0.09	Yes
7th	45	45	239.4	1849	$G + Q - Ed_x$	0.05	Yes

The columns sections verified the crashing conditions in the direction Y as well.

IV.4 Conclusion

After all the necessary verifications, we adopted the following final dimensions for the elements of our structure:

Table IV. 27. Final dimensions of the elements.

Elements:			**Section (cm²)**
Main beams			30x45
Secondary beams			30x35
Landing beams			30x35
Ring beams			30x30
Foundation beams			30x30
Shear walls			25xL
Columns	**Level**	**5th, 6th, 7th**	45x45
		2nd, 3rd, 4th	50x50
		Gr., 1st	55x55
		Basement	60x60

With the bracing system chosen (R=3.5), we could provide:

- An equilibrated structure and a good modal behavior (appropriate disposition);
- a big energy dissipation;
- a stability against the overturning;
- an adequate and admitted deformation in the two directions;
- a sufficient resistance for all the elements it means a safe structure;
- a good ductility;
- An economic execution .

CHAPTER V

Reinforcement of structural elements

V.1. Introduction

We studied our structure in the previous chapters as an element made of concrete only, and it verified the stability and the resistance conditions. In this current chapter we will reinforce the elements to maintain the structure more stable and to make sure that every element resists, according to CBA93 and RPA99 using their combinations.

V.2. Beams reinforcement

We have different types of beams: main beams, secondary beams, ring beams landing beams and the foundation beams; those elements should be reinforced by longitudinal and transverse bars and the calculation is done in simple bending.

V.2.1. Longitudinal reinforcement for beams

The steel chosen for the longitudinal reinforcement is of type F_eE400 ribbed bars.

Table V. 1. The calculated sections and the adopted sections of the beam reinforcement.

Element	$M(kN.m)$	$AS1$ (cm^2)	$AS1\ chosen$ (cm^2)	$AS2$ (cm^2)	$A_{min}CB$ (cm^2)	$AS2\ chosen$ (cm^2)
Main beams 30x45	Mt=99.09 ELU	0	3HA14 = 4.62	7.02	6.75	3HA14+3ch HA16=10.65
	Ma=-178.55 G+Q+Edx	0	3HA14 = 4.62	11.66		3HA14+4ch HA16=12.66
Secondary beams 30x35	Mt=32.98 ELU	0	3HA14 = 4.62	3.03	5.25	3HA14 = 4.62
	Ma=-73.49 ELU	0	3HA14+2ch HA14 =7.69	7.14		3HA14+2ch HA14 = 7.69
Foundation beams 30x30	296.32	/	/	8.52	5.4	8HA14=12.32
Ring beams 30x30	Mt=70.21 ELU	0	3HA16 = 6.03	8.39	4.50	3HA16+ 2ch HA16=10.05
	Ma=-136.07 G+Q+Edx	17.17	10HA16= 20.11	19.19		10HA16=20.11

V.2.2. Verification of the stresses at SLS

We must verify for every beam that:

$$\begin{cases} \sigma_b \leq \overline{\sigma_b} \\ \sigma_s \leq \overline{\sigma_s} \\ \sigma'_s \leq \overline{\sigma_s} \end{cases}$$

Table V. 2. Verification of beam stress at SLS.

Element	Loads at SLS ($kN.m$)		Supports Stresses (MPa)			Span Stresses (MPa)			Allowable Stresses (MPa)		$\begin{cases}\sigma_b \le \overline{\sigma_b} \\ \sigma_s \le \overline{\sigma_s} \\ \sigma'_s \le \overline{\sigma_s}\end{cases}$
	M_t	M_a	σ_b	σ_s	σ_s'	σ_b	σ_s	σ_s'	$\overline{\sigma b}$	$\overline{\sigma_s}$	
Main beams	72.31	-71.73	6.66	179.09	83.47	6.28	150.72	79.37	15	201.63	Yes
Secondary beams	24.08	-53.16	44.42	47.75	175.74	7.47	235.34	84.54	15	400	Yes
Ring beams	51.31	-98.83	10.07	244.86	115.02	10.32	202.11	122.4	15	400	Yes
Foundation beams	216.04	/	/	191.09	201	/	/	/	15	201.63	Yes

All the stresses of the different types of beams are less than the allowable stresses (with different cracking situations).

V.2.3. Verification of shear stress ULS/ALS

We must verify the next condition:

$$r = \frac{V_u}{b.d} \le \bar{r}$$

Table V. 3. Verification of shear stress for beams.

Element	Section	$V_{max}(kN)$	r (MPa)	$\overline{r}(MPa)$	$r \le \overline{r}$
Main beams	30x45	132.56	1.09	3.26	Yes
Secondary beams	30x35	85.49	0.9	3.33	Yes
Ring beams	30x30	110.59	1.37	3.33	Yes
Foundation beams	30x30	8.26	0.1	3.33	Yes

We have no risk of shear stress since the condition is verified for all the beams.

V.2.4. Transverse reinforcement for beams

Table V. 4. Transverse bars end spacing for the beams.

Element	$r_u(MPa)$	$S_{ch}(cm)$		$At\ chosen$ (cm²)
		$S(ZN)$ (cm)	$S'(ZC)$ (cm)	
Main beams	1.09	10	15	$3HA8 = 1.51$
Secondary beams	0.9	10	15	$3HA8 = 1.51$
Ring beams	1.37	10	15	$3HA8 = 1.51$
Foundation beams	0.1	10	15	$3HA8 = 1.51$

V.2.5. Verification of the arrow

Table V. 5. Deflection of the beams.

Element	L(m)	M_{tser} $(KN.m)$	f(mm)	$[f]adm$ (mm)	$f<[f]adm$
Main beams	6.05	72.31	5.3	6.5	Yes
Secondary beams	4.69	24.08	2	9.4	Yes
Ring beams	6.05	51.31	7.1	12.6	Yes

All the beams verified the condition of deflection, and all the arrows are less than the allowed arrow which means that we have no risk of deflection.

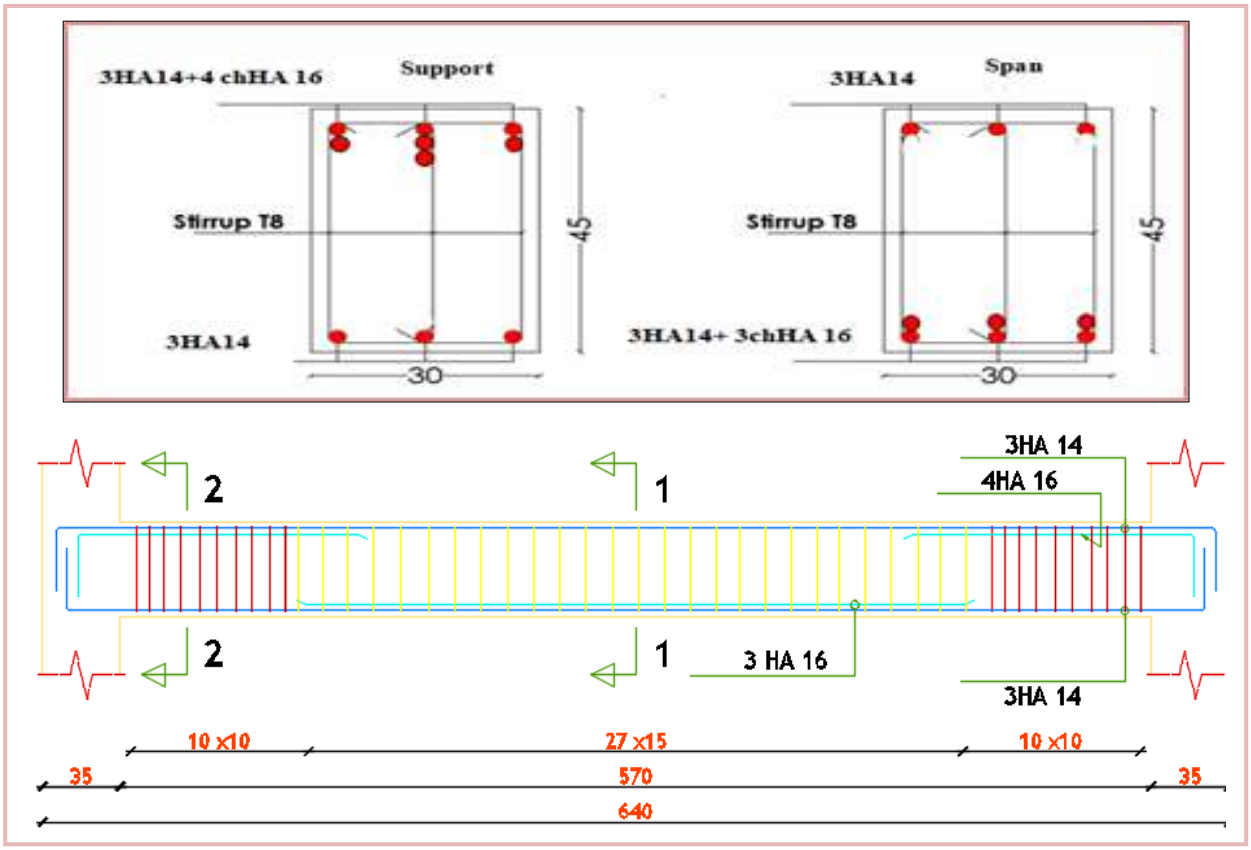


Figure V. 1. Reinforcement details of the main beams.

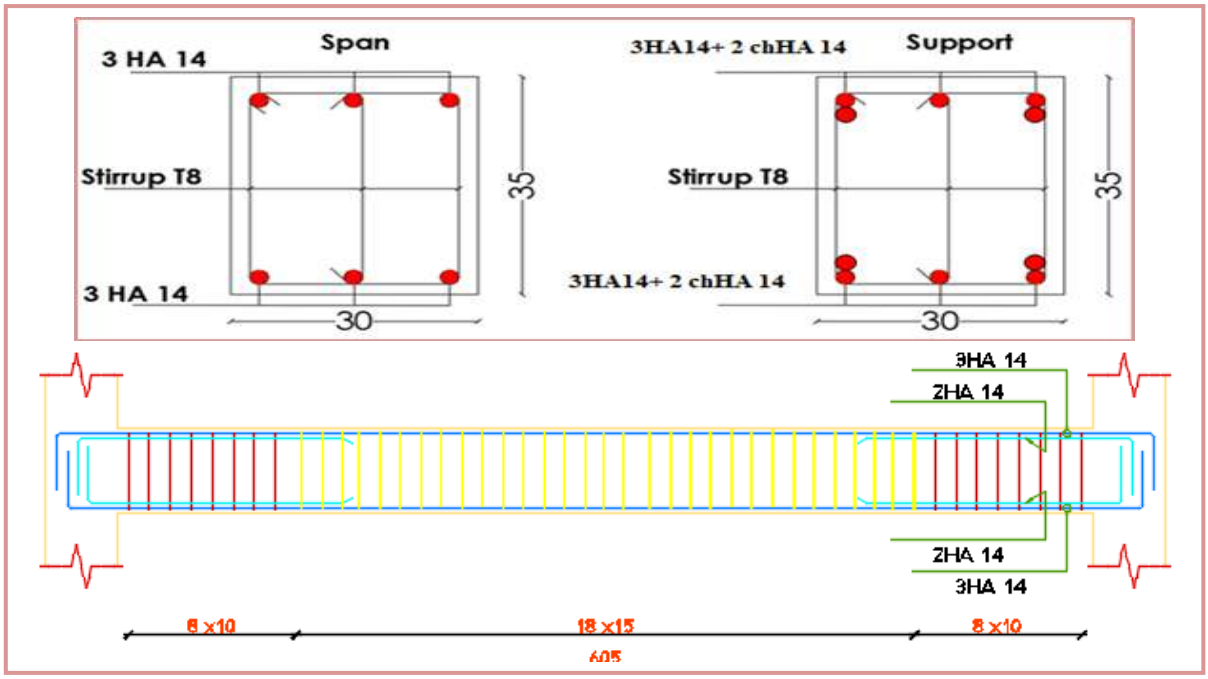


Figure V. 2. Reinforcement details of the secondary beams.

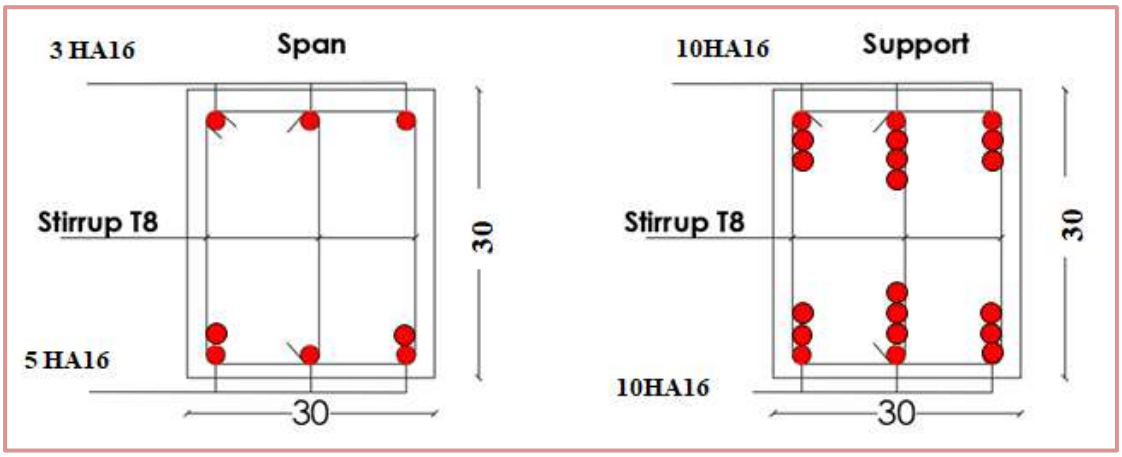


Figure V. 3. Reinforcement details of the ring beams.

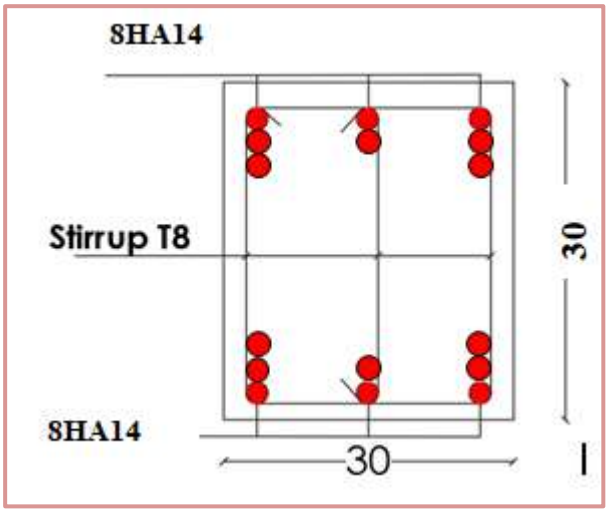


Figure V. 4. Reinforcement details of the foundation beams.

V.3.Columns reinforcement

The columns are subjected to the vertical and horizontal loads; therefore, the calculation ofreinforcement is done in bi-axial compound bending (N, M_x, M_y).

V.3.1.Longitudinal reinforcement

The stresses are calculated with the combination (1.35G + 1.5Q) because the horizontal loads are taken up only by the shear walls so we do not calculate the stresses with the accidental combinations.

All the quantities of the reinforcement calculated with the following stresses were less than the minimum quantity of the CBA, so we will adopt an adequate quantity of reinforcement according to the minimum of CBA.

Table V. 6. Longitudinal reinforcement of the columns.

Section	**Loads**						$A_{min}CBA$ ***(cm²)***	$A_{choosen}$ ***(cm²)***
60x60	N_{min}	386.26	$M_{x\,corr}$	0.95	$M_{y\,corr}$	-0.16	32.40	20 HA 16 = 40.21
	M_{max}	2374.34	$M_{x\,corr}$	-15.45	$M_{y\,corr}$	20.50		
	N_{max}	2613.04	$M_{x\,corr}$	6.15	$M_{y\,corr}$	16.16		
55x55	N_{min}	-956.75	$M_{x\,corr}$	2.74	$M_{y\,corr}$	-23.30	27.22	20HA14= 30.79
	M_{max}	-1390.44	$M_{x\,corr}$	83.30	$M_{y\,corr}$	-37.11		
	N_{max}	-2575.12	$M_{x\,corr}$	0.78	$M_{y\,corr}$	12.43		
50x50	N_{min}	-487.53	$M_{x\,corr}$	8.38	$M_{y\,corr}$	-24.34	22.50	16HA14 =24.63
	M_{max}	-646.45	$M_{x\,corr}$	-91.49	$M_{y\,corr}$	34.65		
	N_{max}	-1872.77	$M_{x\,corr}$	-12.43	$M_{y\,corr}$	34.05		
45x45	N_{min}	-68.87	$M_{x\,corr}$	19.63	$M_{y\,corr}$	-34.38	18.22	12HA14= 18.47
	M_{max}	-104.46	$M_{x\,corr}$	-108.15	$M_{y\,corr}$	39.23		
	N_{max}	-875.48	$M_{x\,corr}$	-7.96	$M_{y\,corr}$	33.90		

V.3.2. Verification of stresses at SLS

Table V. 7. Verification of the stresses of the columns.

Section	Loads			Stresses (MPa)					$\sigma \leq \overline{\sigma}$
	N	M_x	M_y	σ_b	σ_s	σ_s'	$\overline{\sigma b}$	$\overline{\sigma_s}$	
60x60	1911.84	4.48	11.73	4.99	00	73.72	15	201.63	Verified
	1732.04	-11.23	14.91	4.78	00	70.01	15	400	Verified
	283.80	0.69	-0.07	0.7	00	10.49	15	400	Verified
55x55	-746.32	10.32	-0.99	2.52	00	36.75	15	201.63	Verified
	-1882.69	11.21	-2.68	5.91	00	87.27	15	400	Verified
	-1018.08	-17.68	39.59	4.67	00	65.23	15	400	Verified
50x50	-1369.27	-8.97	24.79	6.21	00	81.81	15	201.63	Verified
	-512.59	61.32	-24.36	5.65	27.11	72.85	15	400	Verified
	-397.67	-66.59	25.27	6.12	55.09	76.11	15	400	Verified
45x45	-65.58	-11.15	18.37	3.03	51.50	33.65	15	201.63	Verified
	-91.74	66.24	-24.24	9.28	220.63	96.22	15	400	Verified
	-640.38	-5.71	24.69	4.54	00	62.28	15	400	Verified

V.3.3. Transverse reinforcement

Table V. 8. Transverse reinforcement of the columns.

Storey	*Section*	*Zone*	*t (cm)*	Atx	Aty	$A_{t\ ch}(cm^2)$
Basement	**60x60**	Nodal	10	0.22	0.25	2HA8 = 1.01
		Current	15	0.22	0.25	2HA8 = 1.01
Ground ; 1st	**55x55**	Nodal	10	0.43	0.62	2HA8 = 1.01
		Current	15	0.43	0.62	2HA8 = 1.01
2nd; 3rd; 4th	**50x50**	Nodal	10	0.49	0.72	2HA8 = 1.01
		Current	15	0.49	0.72	2HA8 = 1.01
5th; 6th; 7th.	**45x45**	Nodal	10	0.75	0.9	2HA8 = 1.01
		Current	15	0.75	0.9	2HA8 = 1.01

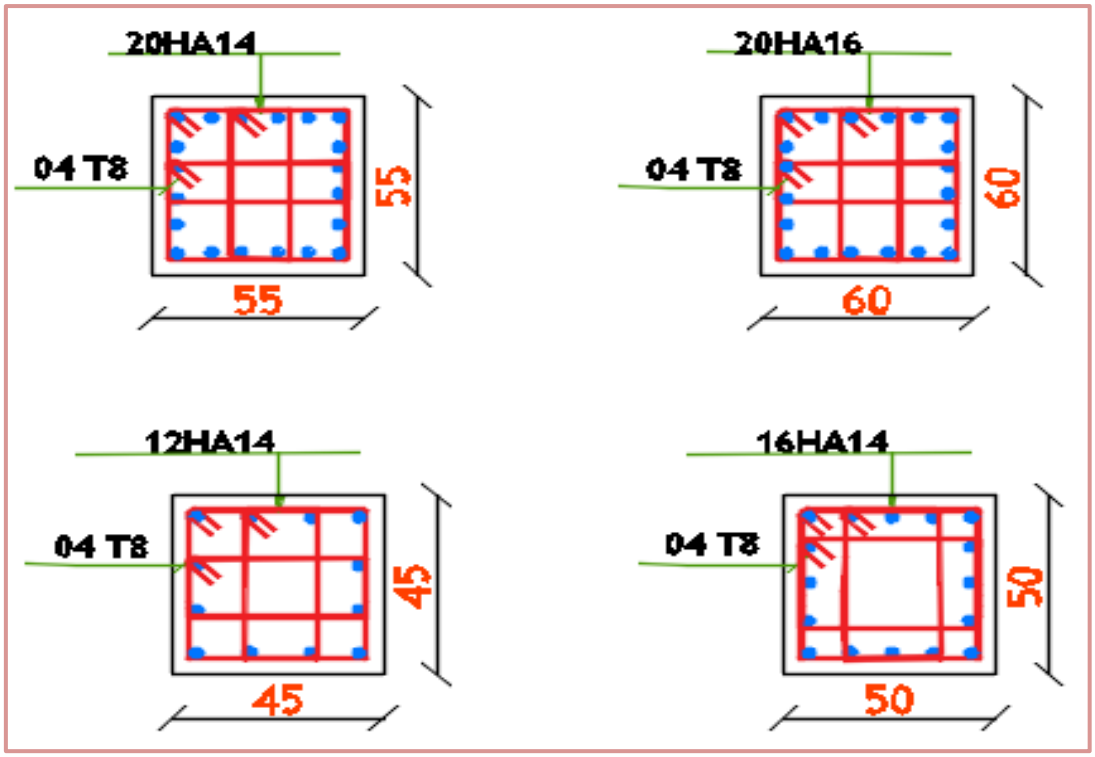


Figure V. 5. Reinforcement details of the different sections of columns.

V.4. Shear wall reinforcement

The reinforcement of the shear walls will be calculated from the stresses due to the vertical loads "G","Q" and horizontal "E", the vertical forces cause the normal forces, while the horizontal forces cause normal forces, cutting forces as well as bending moments.

In our project, we have full shear walls and Coupled shear walls. Shear walls with openings (windows or doors) have two parts (Piers and coupling beam), the reinforcement of the Piers will be calculated in compound bending (like full shear walls), and the reinforcement of coupling beam will be calculated in simple bending.

V.4.1. Reinforcement of the full shear walls and the piers

V.4.1.1. Calculation of the vertical reinforcement

In the following table, we will mention a full shear wall, shear wall with opening (one with window and one with door) and one inclined shear wall.

We have two zones in the shear wall: nodal zone (which must be well reinforced) and a current zone (with quantity of reinforcement less than the quantity in the nodal zone).

Table V. 9. Stresses and calculated Vertical reinforcement of the piers.

Direction	Section [cm²]	N (kN)	M (kN.m)	T	Av(nodal) [cm²]	Av(current) [cm²]
X	430x25	718.106	-2073.04	-563.85	24.09	10.75
	315x25	-234.21	-1527.09	-849.35	17.41	7.88
	149x25	-274.93	-97.59	-51.00	9.19	3.5
Y	712x25	1193.04	-14796.44	-2185.08	42.07	17.88
	165x25	-430.40	-241.43	-154.69	10.83	4.13
Inclined	508x25	242.26	1099.69	-381.16	14.53	12.47

Table V. 10. Vertical reinforcement of the piers.

Direction	Section [cm²]	Nodal zone		Current zone	
		A_{min}	Av_{choo}	A_{min}	Av_{choo}
X	430x25	5.40	2x10HA14=30.79	5.35	2x10HA10=15.71
	315x25	3.95	2x8HA12=18.10	3.92	2x8HA8=8.04
	149x25	2.5	2x5HA12=11.31	1.00	2x2HA12=4.52
Y	712x25	3.6	2x7HA20=43.96	14.27	2x29HA8=29.15
	165x25	2.5	2x5HA12=11.31	1.62	2x4HA10=6.28
Inclined	508x25	2.55	2x5HA14 =15.39	10.15	2x20HA8=20.10

The quantity adopted in both zones is more than the minimum required from the CBA. We adopted for the nodal zone a spacing of: S=10cm and for the current zone a spacing of S=20cm.

V.4.1.2. Stress verification at SLS

Table V. 11. Verification of normal stress at SLS of the piers.

Direction	Section [cm²]	N_{ser} (kN)	M_{ser} (kN.m)	σ_b (MPa)	σ_s (MPa)	σ_s' (MPa)	$\overline{\sigma}$ (MPa)	$\overline{\overline{\sigma_s}}$ (MPa)	$\sigma_i \leq \overline{\sigma}$
X	430x25	1100.8	-6.77	0.95	00	14.21	15	201.63	Verified
	315x25	510.89	-772.65	2.38	4.19	22.39	15	400	Verified
	149x25	282.64	-3.94	0.78	00	11.24	15	400	Verified
Y	712x25	2245.6	5.27	1.18	00	17.64	15	201.63	Verified
	165x25	437.58	-29.43	1.23	00	16.18	15	201.63	Verified
Inclined	508x25	2343.3	50.62	1.87	00	27.85	15	400	Verified

Table V. 12. Verification of the shear stress of the piers.

Direction	Section [cm²]	T (kN)	r (MPa)	$\bar{r}$ (MPa)	$r \leq \bar{r}$
X	430x25	563.85	0.525	5	Verified
	315x25	894.35	1.13	5	Verified
	149x25	51.00	0.146	5	Verified
Y	712x25	2185.08	1.22	5	Verified
	165x25	154.69	0.375	5	Verified
Inclined	508x25	381.16	0.42	5	Verified

V.4.1.3. Horizontal reinforcement and spacing

Table V. 13. Horizontal reinforcement of the piers and spacing.

Direction	Section [cm²]	St CBA (cm)	St_{ch} (cm)	At
X	430x25	228.57	20	2x14HA8=14.07
	315x25	24.92	20	2x14HA8=14.07
	149x25	50.61	20	2x14HA8=14.07
Y	712x25	26.73	20	2x14HA8=14.07
	165x25	41.31	20	2x14HA8=14.07
Inclined	508x25	91.55	20	2x14HA8=14.07

V.4.2. Lintel reinforcement

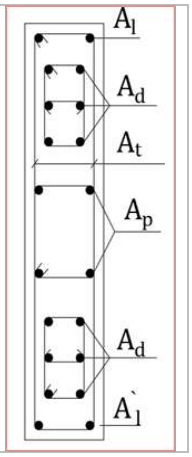
We studied lintels that work on simple bending (SP) and others which work on shear/compression stress (SCS).

Table V. 14. Shear forces verification for lintels.

Lintel	e(cm)	L (cm)	h(cm)	comb	V(KN)	τ_b(MPa)	$0.06F_{c28}$	case
01	25	120	96	0.8G+Edx	-865.21	5.61	1.5	SCS
02	25	120	146	G+Q+Edx	-516.43	2.20	1.5	SCS
03	25	120	56	G+Q+Edx	-48.17	0.54	1.5	SP
04	25	120	146	G+Q+Edy	-1026.63	4.38	1.5	SCS

Table V. 15. Reinforcement sections of the lintels.

	$A_{calculated}$				$A_{choosen}$				
Lintel	A_d	A_l	A_t	A_p	A_d	A_l	A_t	A_p	**S**
01	16.05	3.60	0.94	4.80	8HA16 =16.08	4HA12 =4.52	2HA8 =1.00	4HA14 =6.16	15
02	8.22	5.48	0.94	7.30	8HA12 =9.05	4HA14 =6.16	2HA8 =1.00	12HA10 =9.42	15
03	/	2.07	/	2.80	/	4HA10 =3.14	2HA8 =1.00	2HA14 =3.08	15
04	16.0	5.48	0.94	7.30	8HA16 =16.08	4HA14 =6.16	2HA8 =1.00	2HA10 =9.42	15



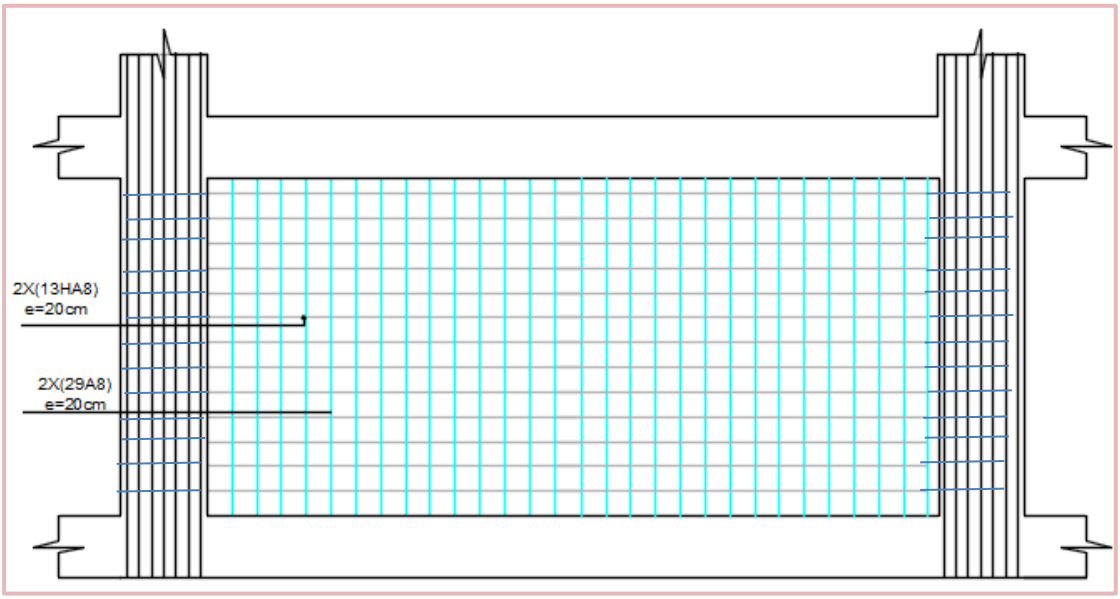


Figure V. 6. Reinforcement details of a full shear wall.

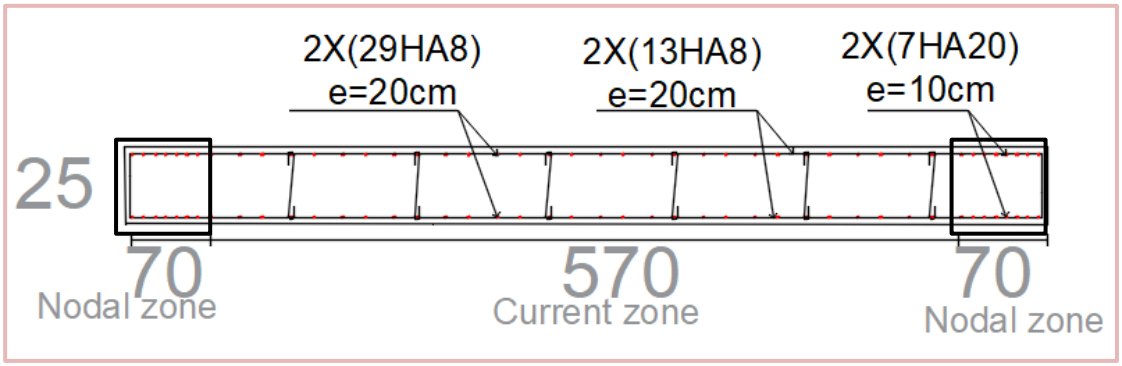


Figure V. 7. Reinforcement details for the current zone of a full shear wall.

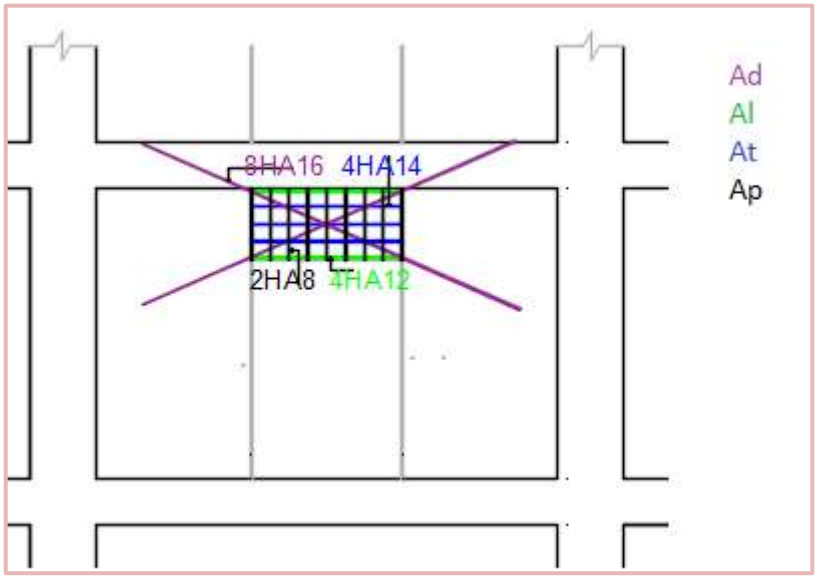


Figure V. 8. Reinforcement details of a lintel.

V.5. Conclusion

All the structural elements of our structure are reinforced with an adequate quantity of reinforcement and verified all the conditions required from the CBA and the RPA documentations.

CHAPTER VI

Calculation of foundations

VI.1 Introduction

The foundation is the lower part of a structure resting on the base, to which are transferred all the loads borne by the structure. It can be calculated only if the superstructure and its loads are known on the one hand and the characteristics of the ground on the other hand.

The function of foundations is the following:

- Take over the loads and overloads supported by the structure.
- Transmit loads and overloads to the ground in good conditions, to ensure the stability of thestructure.

VI.2 Types of foundations

Foundations can be classified into 3 types:

- Shallow foundation (Isolated/pad footing , strip footing, mat foundation);
- Semi-deep foundations (pier).
- Deep foundations (piles).

VI.3 Soil characteristics

To choose the right type of foundation for our project, we need to know the characteristics of the soil that are mentioned in a soil report made by a geotechnical laboratory.

According to the soil report of the location of our project we have:

- Capacity of the soil: $\bar{\sigma}_s = 03$ bars
- Bulk density: $\gamma_{soil} = 18KN/m3$
- Depth D= 02m.
- Non aggressive soil.

The foundations and their reinforcement bars shall not be placed directly on the ground, so it is necessary to place a layer of concrete fill with 10cm of thickness in order to protect the bars and to have the same level on ground.

VI.4 Choice of the type of foundation

The choice of the type of foundation depends on:

- The type of structure to be built;
- The nature and homogeneity of the soil;
- The bearing capacity of the foundation ground;
- Facility of realization;
- The distance between axes of the columns;
- Economic reasons.

VI.4.1 Isolated/pad footing

We have three groups of footings according to the following stresses at the base.

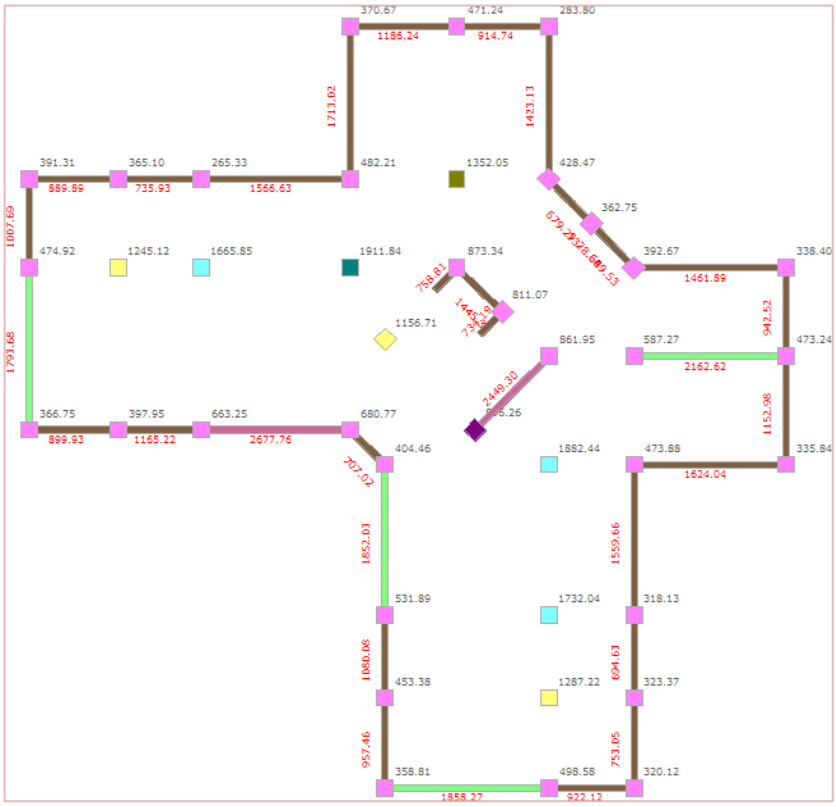


Figure VI. 1. Solicitations at the base**.**

Table VI. 1. Sizing of the isolated footings at SLS.

footing	***Nser*** ***(kN)***	M_x $(kN.m)$	M_y $(kN.m)$	A (m)	B (m)	A_{ch} (m)	B_{ch} (m)	h_{ch} *(cm)*
footing 1	1911.84	4.48	11.73	2.52	2.52	2.7	2.7	70
footing 2	1352.05	-11.28	0.32	2.12	2.12	2.3	2.3	55
footing 3	896.26	5.51	3.77	1.73	1.73	1.8	1.8	45

VI.4.1.1 Checking the overlapping between the pressure bubbles

The following condition must be checked so that we can say that we have no risk of overlapping:

$$L \geq 1.5 \left(\frac{A + A'}{2} \right)$$

With:

A and A': dimensions of two adjacent footings.

Table VI. 2. Verification of the overlapping .

Direction	**A**	**A'**	$1.5\left(\frac{A+A'}{2}\right)$	**L**	$L \geq 1.5\left(\frac{A+A'}{2}\right)$
X	2.7	2.3	3.75	3.1	Not verified
Y	2.7	1.8	3.37	3.30	Not verified

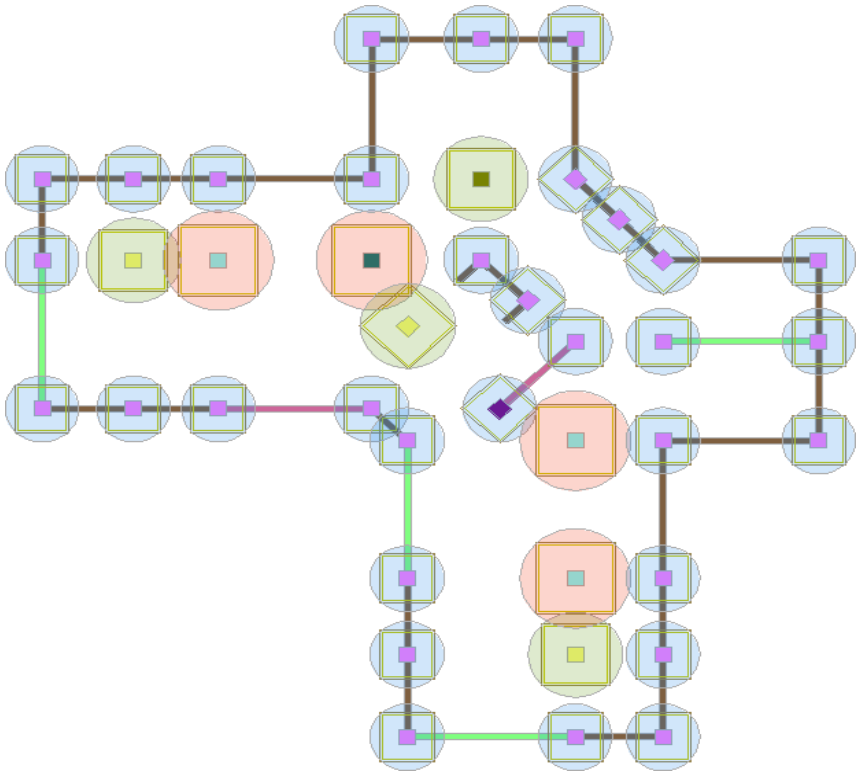


Figure VI. 2. Pressure bubbles overlap under the footing.

Note:

There is a risk of overlapping between the pressure bubbles of the footing in both directions, so we will switch to the strip footings, but there is one isolated footing which does not overlap with others so we will start the calculation with it.

VI.4.2 Isolated/pad footing under a column

Table VI. 3. Dimensions of the pad footing .

footing	*Nser* (kN)	M_x $(kN.m)$	M_y $(kN.m)$	A (m)	B (m)	A_{ch} (m)	B_{ch} (m)	h_{ch} *(cm)*
footing	1352.05	-11.28	0.32	2.12	2.12	2.20	2.20	45

VI.4.2.1 Verification of the solicitations

Table VI. 4. Verification of the solicitations of the pad footing .

footing	e_a (cm)	σ_{a1} (Bars)	σ_{a2} (Bars)	σ_{b1} (Bars)	σ_{b2} (Bars)	σ_{sol} (Bars)	$\sigma \leq \sigma_{sol}$
footing	0.826	2.76	2.88	2.82	2.82	03	Verified

VI.4.2.2 Verification of the overturning of the pad footing

Table VI. 5. Verification of the overturning of the pad footing .

	e_a	A/4	$e_a \leq A/4$	e_b	B/4	$e_b \leq B/4$
Footing	0.826	55	Verified	0.023	55	Verified

VI.4.2.3 Verification of the punching shear

Table VI.6. Verification of the punching shear of the pad footing .

	N_u	F_{limit}	$N_u \leq F_{limit}$
Footing	1852.82	1897.50	Verified

The adopted isolated/pad footing verified all the necessary conditions to support different loads.

The location of the pad footing in our structure and it position is shown in the next figure.

VI.4.3 Strip footing

Table VI. 7. Sizing of the strip footings.

Footing	N_{col}	N_{shear}	L	B_{cal}	h_{calc}	σ_{sol}	σ_{ser}	$B_{adopted}$	$h_{adopted}$
Along X :									
1	704.76	2100.98	9.25	1.01	30	3	1.96	**1.5**	
2	1076.20	3192.45	13.85	1.03	30	3	1.99	**1.5**	
3	4104.35	0	13.85	0.99	30	3	1.93	**1.5**	
3'	365.54	1461.89	7.50	0.81	30	3	1.56	**1.5**	
4	530.26	2162.62	7.50	1.20	30	3	2.32	**1.5**	
5	1584.97	4742.91	13.85	1.42	30	3	2.98	**1.5**	
6	1583.02	1624.04	10.70	1	30	3	1.94	**1.5**	
7	588.76	2780.39	11.15	1.01	30	3	1.95	**1.5**	
Along Y :									
A	616.50	2801.37	11.15	1.02	30	3	1.98	**1.5**	
B	1382.37	1713.02	10.80	0.96	30	3	1.85	**1.5**	**40**
C	1366.91	3889.57	13.85	1.27	30	3	2.47	**1.5**	
D	356.14	1423.13	7.50	0.79	30	3	1.52	**1.5**	
D'	2624.94	0	8.25	1.06	30	3	2.08	**1.5**	
E	1038.50	3700.34	13.85	0.77	30	3	1.88	**1.5**	
F	742.94	2095.50	9.15	1.03	30	3	2	**1.5**	
Inclined									
1	542.62	707.02	3.64	1.14	30	3	2.22	**1.5**	
2	806.19	2657.50	6.40	1.80	40	3	2.63	**2**	
3	1876.72	1445.31	6.57	1.58	40	3	2.33	**2**	
4	1015.03	758.81	5.58	1.06	40	3	1.55	**2**	
5	879.11	2449.30	5.69	1.95	40	3	2.87	**2**	

We have no overlapping between the strip footings so we are going to adopt the strip footings with the dimensions mentioned in the previous table.

Different dimensions were adopted (B=1.5m and B=2m) and we generated h=40cm.

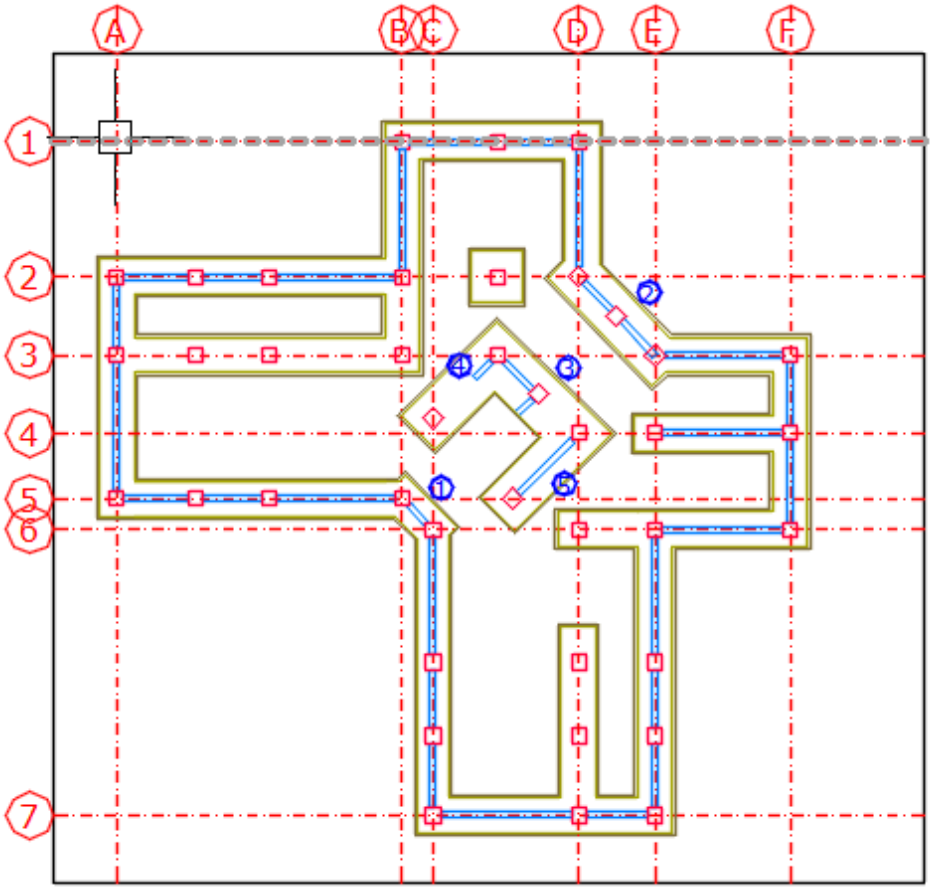


Figure VI. 3. Disposition of the pad footing and the strip footings.

The strip footing are long so there is no risk of overturning, and even if we have the risk of punching shear it will be verified because we will use an upstand beam.

VI.4.3.1 Reinforcement of strip footing

The reinforcement of the strip footing is calculated the same way as the reinforcement of the isolated footing, with the maximum normal stress and his moment. We will calculate the quantity of reinforcement for the most solicited footing and generate it for all the other footings that have the same width.

VI.4.3.1.1 Reinforcement of strip footing under several columns

Table VI. 8. Solicitations of the strip footings under columns.

footing	B (m)	A (m)	N_u (kN/ml)	M_u (kN.m)
SF01	1.5	3.55	2613.04	16.16

Table VI. 9. Reinforcement of the strip footings under columns.

footing	$A_{a.cal}$ (cm^2)	$A_{a.ch}$ (cm^2)	S_{tb} (cm)	$A_{a.dis.cal}$ (cm^2)	$A_{a.dis.ch}$ (cm^2)	$S_{t\,a}$ (cm)
SF01	23.47	24HA14=36.95	15	4.04	2x3HA12=6.79	15

VI.4.3.1.2 Reinforcement of strip footing under shear walls

Table VI. 10. Solicitations of the strip footings under shear walls.

footing	B (m)	A (m)	N_u (kN/ml)	M_u (kN.m)
SF01	1.5	5.6	652.93	2.9
SF02	2	3.88	860.99	2.35

Table VI.11. Reinforcement of the strip footings under shear walls.

footing	$A_{a.cal}$ (cm^2)	$A_{a.ch}$ (cm^2)	S_{tb} (cm)	$A_{a.dis.cal}$ (cm^2)	$A_{a.dis.ch}$ (cm^2)	$S_{t\,a}$ (cm)
SF01	3.27	7HA12=7.92	15	3.96	10HA12=11.31	15
SF02	15.09	10HA14=15.39	15	7.69	13HA12=14.7	15

VI.4.4 Upstand beam

The upstand or the strip beam is a beam placed along the strip footing between the columns; it has several objectives and we use it to:

- Avoid the risk of punching shear between the column and the footing.
- Stiffen the footing more and more.
- Ensure uniform stress distribution.
- Ensure uniform settlement under the footing.

VI.4.4.1 Sizing of the upstand beam

$$h_p \geq \sqrt[3]{\frac{0.5 \times k \times l_{max}^{\ 4}}{E}} \; ; \begin{cases} b_p = b_{column} \\ b_p \geq b_{column} \\ b_p \leq b_{column} \end{cases}$$

With:

h_p: The height of the beam strip;

l_{max}: The largest span;

E: Concrete elastic modulus (E =32164.19MPa) ;

K: stiffness of the soil;

I: Moment of inertia of the beam strip.

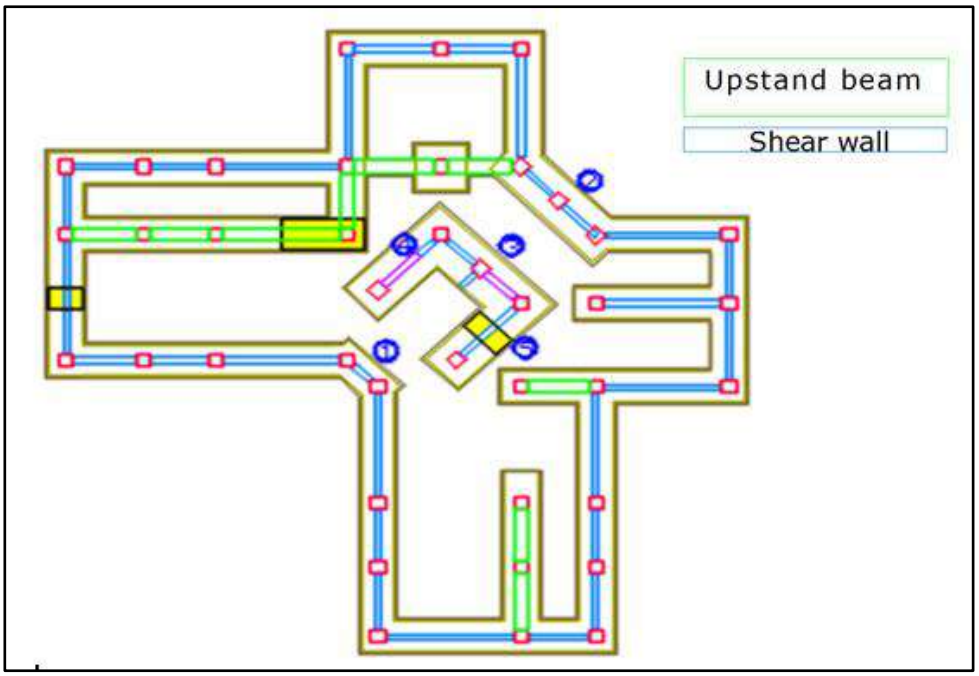


Figure VI. 4. Position of the upstand beam.

We gave many dimensions for the beam but the shear force was not verified, so and in order to have a strip beam that verifies the conditions we adopted the following dimensions:

h_p=120cm and b_p=60cm

VI.4.4.2 Verification of the punching shear

Table VI.12. Verification of the punching shear of the upstand beam.

	N_u	F_{limit}	$N_u \leq F_{limit}$
Footing	1784.52	6480	Verified

VI.4.4.3 Reinforcement of the upstand beam

Table VI.13. Reinforcement of the upstand beam.

	M_{max} $(kN.m)$	A_{calcul} (cm^2)	$A_{choosen}$ (cm^2)	$S_t(cm)$
Span	1110.71	28.74	10HA20=31.42	15
Supports	-1305.98	34.13	12HA20=37.07	15

VI.4.4.4 Verification of the normal stresses al SLS

Table VI.14. Verification of the normal stresses of the upstand

	M_{max} $(kN.m)$	σ_b	σ_s	σ'_s	$\overline{\sigma_b}$	$\overline{\sigma_s}$	$\sigma \leq \overline{\sigma}$
Span	-853.42	5.84	201.63	79	15	201.63	Verified
Supports	1003.45	6.27	201.63	85.32	15	201.63	Verified

VI.4.4.5 Verification of the shear stress

Table VI.15. Verification of the shear force of the upstand beam.

Section (cm)	T(kN)	$r(MPa)$	$\bar{r}(MPa)$	$r \leq \bar{r}$
60x120	1593.04	2.27	2.5	Verified

VI.4.4.6 Verification of the deflection

Table VI.16. Verification of deflection of the upstand beam.

Support moment (left) Kn.m	Span moment Kn.m	Support moment (right)Kn.m	[F] (mm)	$[\bar{F}]$ (mm)	$[F] < [\bar{F}]$
-502.01	853.04	-502.01	1.6	22.4	Verified

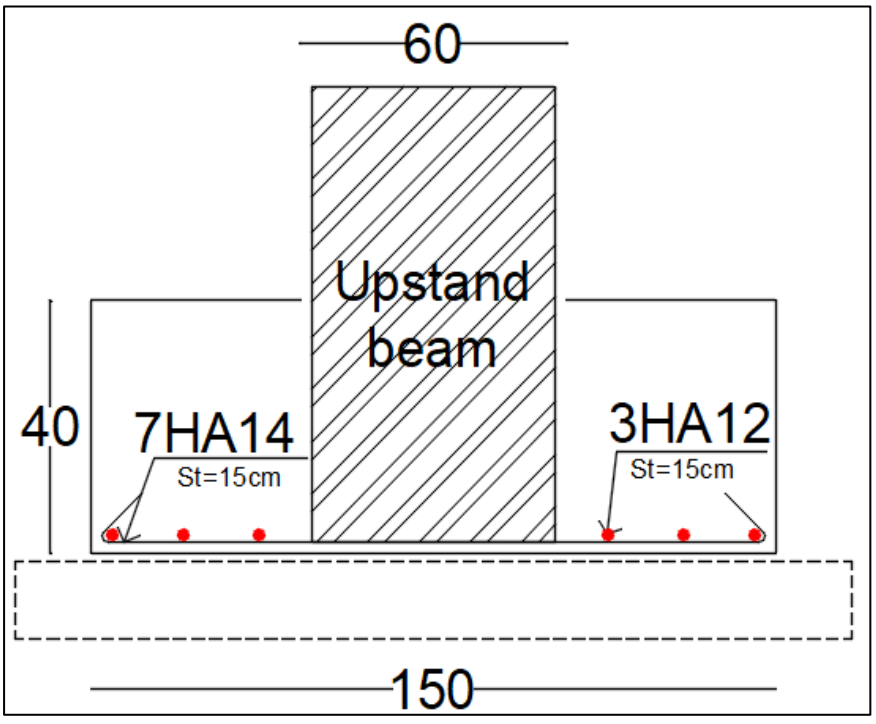


Figure VI. 5. Reinforcement details of the strip footing under column.

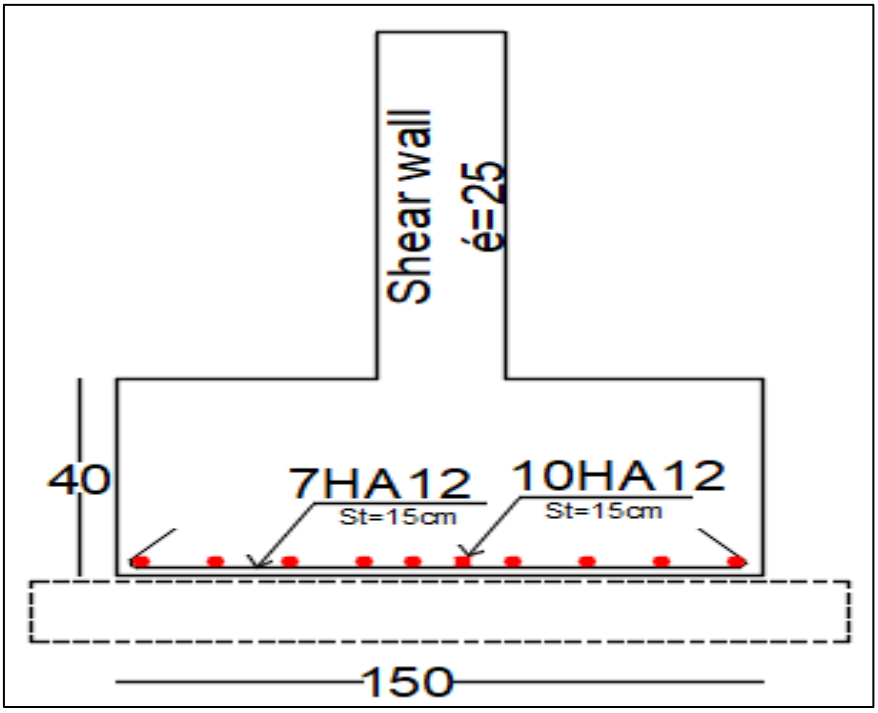


Figure VI. 6. Reinforcement details of the strip footing(B=1.5m) under a shear wall.

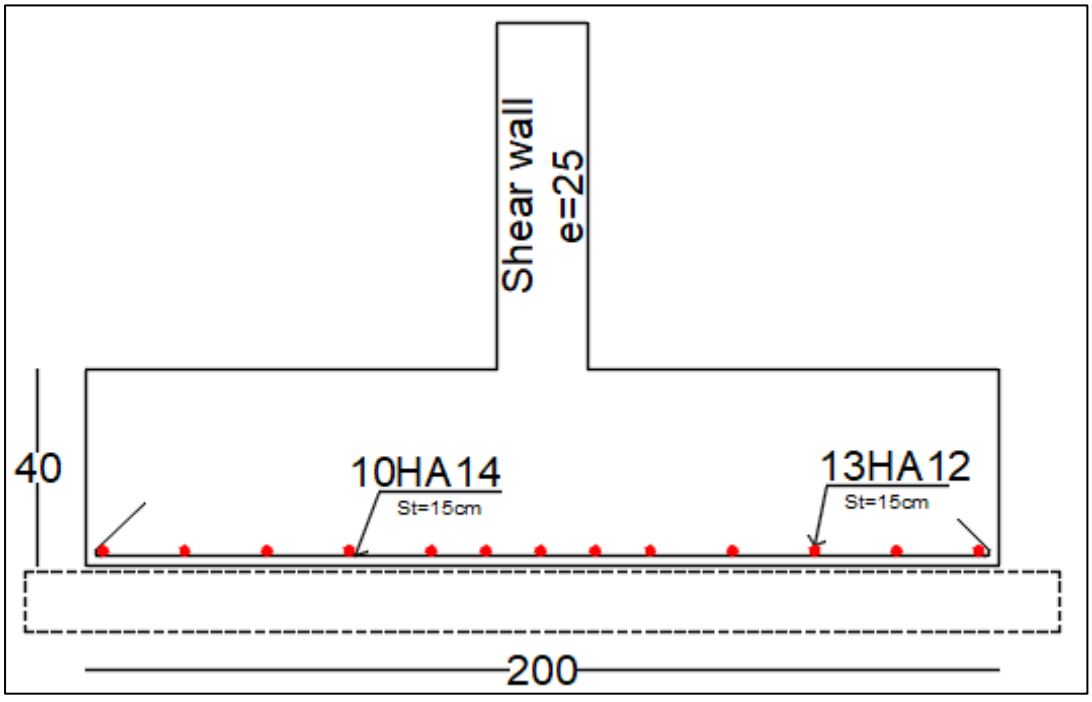


Figure VI. 7. Reinforcement details of the strip footing (B=02m) under a shear wall.

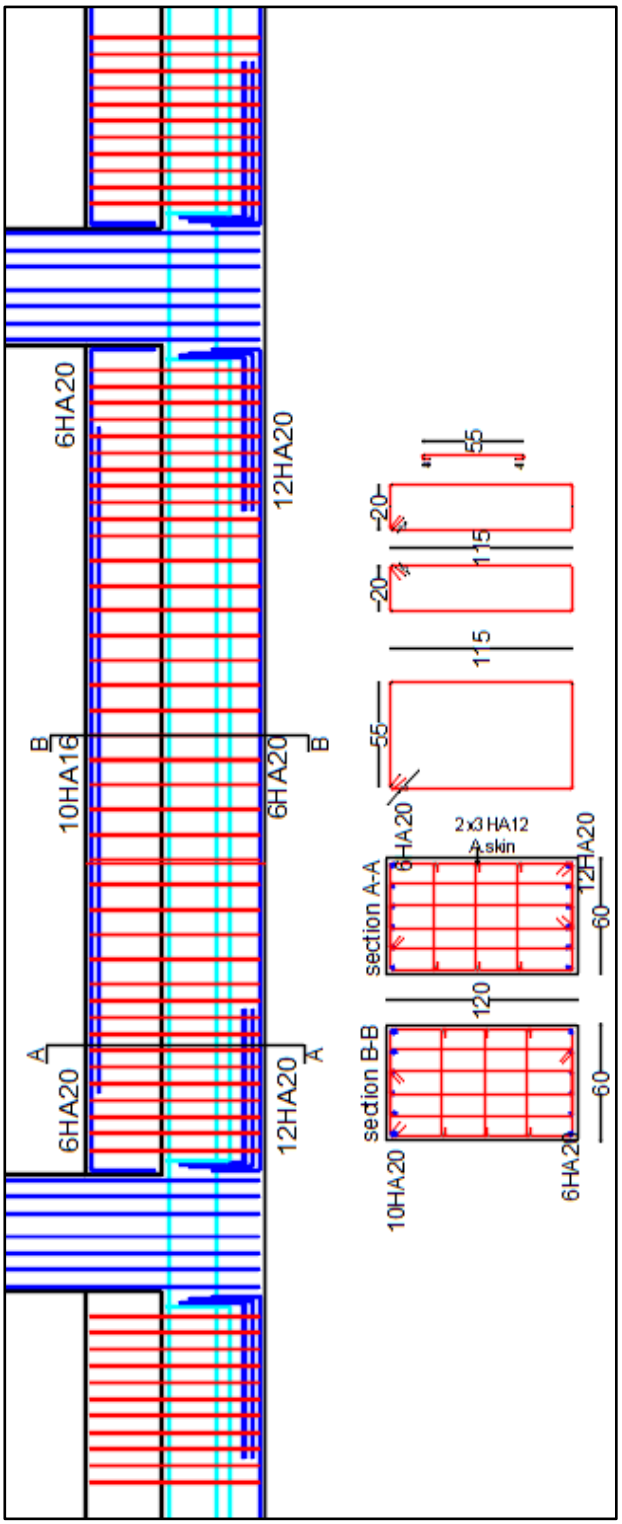


Figure VI. 8. Reinforcement details of the upstand beam.

VI.5 Conclusion

In this project we used two types of footing: a strip footing under columns and under shear wall; and one isolated footing because we had no overlapping between it and the other footings and since it is more economic, we didn't want to generate the strip footing for them all.

For the strip footing, we had two footing with different dimensions; one with B=1.5m and another one with B=2m. For the isolated footing we adopted B=2.2m.

With the strip footing we used an upstand beam for many reasons and we adopted for it a section of (bxh) = (60x120)cm²

All the elements of the infrastructure verified the necessary conditions in order to support all the loads coming from the superstructure.

CONCLUSION

General conclusion

This end of study project, allowed us to put into practice all our knowledge acquired during our engineering training cycle, to deepen our knowledge based on technical and regulatory documents, to apply recent calculation methods, and put in our minds the basic principles that must be taken into the design of building structures.

According to the study carried out as part of this end-of-study project, it should be sure that for a good seismic design, it is very important that the civil engineer and the architect work in close collaboration from the start of the project in order to take care of all the stresses induced by the structure adopted compared to the proposed architecture and to achieve maximum security of the work without significant additional cost.

The main objective of the engineer is to reduce the seismic risk to a minimum level and to facilitate the execution of the work by adopting an optimal design which satisfies the architectural and safety requirements.

ANNEXES

ANNEX 1: Caquot method: (for beams)

- **Presentation of the method**

This method is applicable for high loads and when the flat-rate method is not applicable. It consists in calculating the moments at the supports under the effect of the loads (uniformand concentrated) applied to the spans near to the support, then determining the moments in the span.

- **Fictitious span**

For all spans, the fictitious length will be determined:

$$L' = \begin{cases} L & \text{for an edge span with simple edge support.} \\ 0.8L & \text{for an intermediate span.} \end{cases}$$

- **Rigidity coefficient**

We denote by γ the stiffness coefficient which will be the ratio of the stiffness of the 2spans to the left and to the right of a support considered.

$$\gamma = \frac{K_W}{K_e} \times K_e = \frac{l_e}{l'_e} \times K_w = \frac{l_w}{l'_w}$$

- **Particular case**

For a continuous beam of the same height and width, therefore at constant inertia and in theabsence of concentrated load; we have:

$$\gamma = \frac{l_w \times l'_e}{l_e \times l'_w}$$

- **The moment of support**

$M_{app} = 0.2 \times M_0$ ……edge supports

$M_{app} = \frac{q_w {l'}^3_w + q_e {l'}^3_e}{8.5(l'_w + l'_e)}$ ……Intermediate supports

- **The moment in span**

The Caquot method allows us to take the moment at half span (x = L / 2) as the maximummoment in span:

$$M_t = M_0 - \frac{M_e + M_w}{2}$$

ANNEX 2: Beam transverse reinforcement

The steel chosen for the transverse reinforcement is of type bars with high adhesion . F_eE400. The spacing between successive core reinforcements must be the lesser of the following values:

$$\text{RPA 2003:}\begin{cases} S_t \leq \dfrac{A_t}{0.003\times b} & \\ S_t \leq \text{Min}\left[\dfrac{h}{4}, 12\emptyset\right] & \text{Nodal zone} \\ S' \leq \dfrac{h}{2} & \text{Current zone} \end{cases}$$

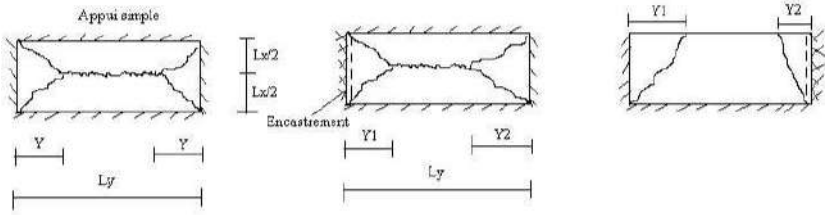
$$\text{CBA 93:}\begin{cases} \dfrac{A_t}{b_0\times S_t} \geq \dfrac{\gamma_s(\tau_u - 0.3f_{t28}\times K)}{0.9f_e(\cos\alpha+\sin\alpha)} \\ S_t \leq [0.9d, 40cm] \\ \dfrac{A_t\times f_e}{b\times S_t} \geq 0.4\ \text{Mpa} \end{cases}$$

ANNEX 3: Yield line method

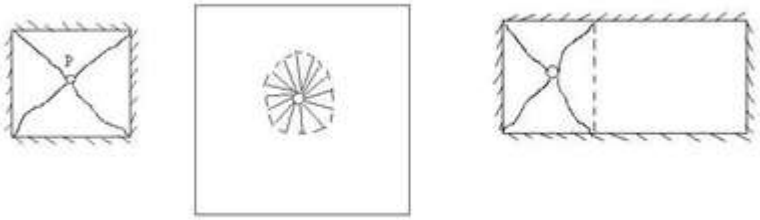
The Yield line method is a more powerful method and considers materials at their elastoplastic limit. This method, which only concerns the ultimate limit state, is based on the Crack Mechanism,it provides for damage to the slab. The cracks that appear on the slab are called breaking lines.

Examples of slab cracking

a) **Case of distributed charges**



b) **Case of concentrated distributed loads**



- **Calculation of moments (principle of virtual work)**

$$\sum \text{External Works} - \sum \text{Internal Works} = 0$$

a) External Works

External work is equal to the product of the force concentrated by its displacement

$$T_{ext} = P.f$$

For the particular case of distributed load:

T_{ext} = distributed load x Surface x displacement of the center of gravity of the surface

b) Internal Works

The internal work is equal to the product of the moment by the projection of the breaking lineon the axis of rotation by the angle of rotation of the rigid region on its axis.

$$T_{int} = M.l.\theta$$

The principle of virtual work allows us to evaluate the bending moment as a function of theexternal load applied to the slab.

ANNEX 4: Minimum and maximum section

• According to RPA2003 Art 7.7.4.1

Av = 0.2% Bt: The tensile horizontal section of shear wall.

Av = 0.15% B B: The global section of shear wall.

Av = 0.1% Bc: The current horizontal section.

ANNEX 5: Lintel reinforcement

Reinforcement of lintels in simple bending $(r_b \leq 0.06 f_{c28})$ **1st case**

We should have

- Longitudinal Reinforcement A_l.
- Transversal Reinforcement A_t.
- Partially common Reinforcement (skin Reinforcement) A_c.

A. Longitudinal Reinforcement

$$A_l \geq \frac{M}{Zf_e} \quad , \quad Z = h - 2c \quad , \quad M = 1.4M_u$$

B. Transversal Reinforcement

We must know if our lintel is short or long with the following verification:

$$\lambda = \frac{l}{h} \quad \Rightarrow \begin{cases} \lambda > 1 & \text{linteau long} \\ \lambda < 1 & \text{linteau court} \end{cases}$$

- **Long lintel**

$$S \leq \frac{A_t * f_e * Z}{\bar{V}}$$

S : Spacing of transverse reinforcement

A_t : Section of a transverse reinforcement.

$Z = h - 2d'$

$\bar{V} = 1.4V_u$

- **Short lintel**

$$S \leq \frac{A_t * f_e * l}{\bar{V} + (A_t * f_e)}$$

$$V = \min(V_1 ; V_2)$$

$$V_2 = 2 * V_u \quad , \quad V_1 = \frac{M_{ci} + M_{cj}}{l_{ij}}$$

- With $M_{ci} and\ M_{cj}$ ultimate resulting moments of the standing sections to the left and right of thespan of the lintel l_{ij} and calculated by:

$$M_c = V_l * f_e * Z$$

Reinforcement of lintels in simple bending ($r_b \geq 0.06 f_{c28}$) 2ed

- **Diagonal Reinforcement:**

The forces (M, V) are taken up according to diagonal connecting rods along the medium axis of thediagonal reinforcement A_Dà disposer obligatoirement.

$$A_D = \frac{V}{2f_e sin\alpha}$$

$$\alpha = Arctg\left(\frac{h - 2d'}{l}\right)$$

Minimum reinforcement (RPA v2003 Art.7.7.3.3)

- **Longitudinal reinforcement**

$$(A_l, A_l') \geq 0.15\% \ (b * h)$$

- **Transverse reinforcement**

$Pour : r_b \leq 0.025 f_{c28} \Rightarrow A_t \geq 0.0015 \ b * s$

$Pour : r_b > 0.025 f_{c28} \Rightarrow A_t \geq 0.0025 \ b * s$

s : Spacing between frames

- **Skin reinforcement**

$$A_c \geq 0.002 * b * h$$

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86 LOGEMENTS PROMOTIONNELS

RUE D'ALGER

MÉDÉA

Propriétaire: **PROMOTEUR KHORCHOF**

References

[1]: Règle de conception et de calcul des structures en béton armé C.B.A 93.

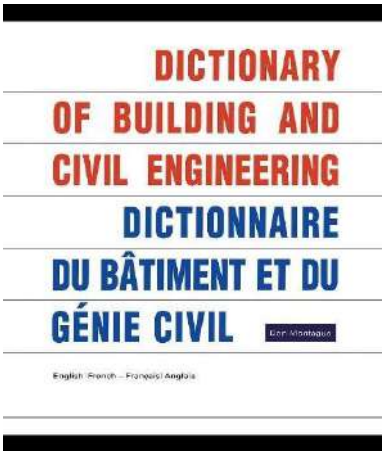
[2]: Règles parasismique Algérienne RPA99 version 2003 ; (Document technique réglementaireD.T.R-B.C.2.48).

[3]: Charges permanentes et charges d'exploitation D.T.R B.C.2.2.

[4]: Calcul des ouvrages en Béton Armé. M. BELAZOUGHI.

[5] : Soil rapport.

[6] Dictionary of Building and Civil Engineering French–English andEnglish–French . (Don Montague)



Programs

- Sap 2000 22.2.0
- Autocad 2017
- Expert 2010
- Etabs 2019