UNIVERSITÉ ABDERRAHMANE MIRA DE BÉJAIA

Facculty of sciences and Technology

Civil engineering department

FINAL YEAR PROJECT

For the award of

MASTERS DEGREE IN CIVIL ENGINEERING

Option **: Structures**

Theme :

Study of a reinforced concrete building (Basement + Ground floor + Loft + 8 floors) with multiple uses, braced by a mixed system

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Academic year : 2024/2025

Algerian People's and Democratic Republic

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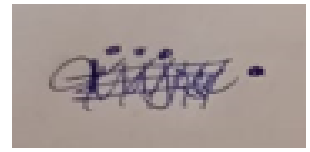

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Dedications

With the expression of love, I dedicate this modest work to whom, whatever the terms used, I will never be able to express my sincere love.

To my dear parents who have never ceased to pray for me, to support me and to help me achieve my goals.

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May they find this document the fruits of their efforts.

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Kimeu Mutia Josephat

symbols and Notations

Symbols	Notations
A'	Section of compressed steel
A_{ser}	Section of steel at the serviceability limit state (SLS)
A_t	Section of a transverse reinforcement course
A	Acceleration coefficient of the seismic zone
a	thickness
α	Neutral fiber coefficient
B	Area of a concrete section
Br	Reduced section
B, b	Width
b_0	Gross thickness of the web of a section, or width of the rib
C_T	Coefficient depending on the bracing system and type of infill
C_u	Soil cohesion
cj	Characteristic compressive strength at “j” days (MPa)
D	Diameter
D	Mean dynamic amplification factor
D	Effective depth
E	Longitudinal modulus off elasticity, earthquake
E_i	Instantaneous modulus of elasticity
E_s	Steel modulus of elasticity
E_d and E_c	Deformation modules
e	Eccentricity, thickness
F	Deflection
F	Force or general action

f_{c28}	Characteristic compressive strength at 28 days (MPa)
f_{t28}	Characteristic tensile strength at 28 days (MPa)
f_{gi}	Deflection corresponding to g
f_{ji}	Deflection corresponding to j
f_{pi}	Deflection corresponding to p
f_{gv}	Deflection corresponding to v
Δf_t	Total deflection
f_{bu}	Concrete compressive stress at ultimate limit s
f_e	Yield strength
Δf_{tadm}	Admissible deflection
F	Safety coefficient
G	Permanent load (dead load)
H	Height, Anchorage depth of a foundation (m)
h_t	Total height of the floor slab and raft foundation
hN	Height measured in meters from the base of the structure to the top level
h_0	Thickness of the compression slab
h_e	Clear story height floor to floor height)
I	Moment of inertia
I_{gi}	Moment of inertia corresponding to g
I_{ji}	Moment of inertia corresponding to j
I_{pi}	Moment of inertia corresponding to p
I_{gv}	Moment of inertia corresponding to v
M	Moment (general)
M_a	Support moment

M_u	Ultimate design moment
M_{ser}	Service design moment
M_s	Span moment
M_0	Isostatic moment
M_i	Moment at support “i”
M_l and M_r	Moments on the left and right sides, with their respective signs
M_g	Moments corresponding to g
M_p	Moments corresponding to p
M_q	Moments corresponding to q
N	Normal (axial) force due to vertical loads
N_u	Ultimate normal (axial) force
N_{ser}	Normal (axial) service force
n	Number of steps in the flight of stairs, or equivalence coefficient
N_{tot}	Total weight transmitted by the superstructure (kN)
P	Applied concentrated load (at SLS or ULS)
P_d and P_g	Uniform loads on the left and right, respectively
Q	Variable load (live load)
Q	Quality factor
q	Load (kN/m)
q_u	Ultimate load
q_s	Service load
R	Global behavior (response) factor
S	Section, surface area
S_{raft}	Surface area of the raft foundation

S_t	Spacing of reinforcement bars
SLS	Serviceability Limit State
tj	Characteristic tensile strength at “j” days (MPa)
ULS	Ultimate Limit State
V_u and V_s	Ultimate shear force and Serviceability shear force respectively
T_2	Characteristic period associated with the site category
W	Self-weight of the structure
W_{Qi}	Live (usage) loads
W_{Gi}	Weight due to permanent loads and any fixed equipment
X, Y and Z	General coordinates
Y	Ordinate of the neutral axis (fiber)
Z	Coordinate, lever arm
Z	Depth below the foundation
σ	Normal stress
σ_b	Compressive stress in concrete
σ_g	Stress corresponding to g
σ_j	Stress corresponding to j
σ_q	Stress corresponding to q
σ_s	Compressive stress in steel
σ_{adm}	Allowable stress at the foundation level (bars)
υ	Poisson’s ratio
γ_b	Safety factor (concrete)
γ_s	Safety factor (steel)
γ_w	Unit weight of water

φ	Internal friction angle of soil (degrees)
τ_{adm}	Ultimate shear strength value as defined by BAEL (Pa)
τ_u	Shear stress (MPa)
η	Damping factor
β	Weighting coefficient depending on the type and duration of the imposed (live) load
μ_l	Reduced limit moment
μ_u	Reduced ultimate moment
λ_i	Instantaneous coefficient
λ_v	Long-term (creep) coefficient

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General introduction

General introduction

Structural analysis is the fundamental engineering process that ensures the safety, functionality, and efficiency of built structures. By rigorously calculating how a building will respond to forces like gravity, wind, earthquakes, and occupancy loads, engineers can predict stresses, deformations, and potential failure points. This scientific understanding is essential for designing structures that not only withstand these demands without catastrophic failure, protecting lives and property, but also perform reliably during their service life (serviceability) and use materials economically. Without accurate structural analysis, designs would be mere guesswork, risking collapse, excessive cost, or impractical performance.

As part of our final Master's thesis in Civil Engineering, commissioned by the design office BART, we are conducting the structural study of a reinforced concrete building located in Bejaia, Algeria. This multi-purpose structure integrates commercial spaces, storage areas, and residential units.

Our primary objectives are to:

- Analyse all structural components of the building.
- Design the formwork layouts and reinforcement detailing.
- Ensure full compliance with Algerian construction and seismic regulations (including CBA93, BAEL91, RPA99 v2003, and relevant DTRs), critically important given Algeria's significant seismic risk.

To achieve these objectives, our work is structured into three main phases:

1. Preliminary Design & Sizing:
    - Presentation of the building and material specifications.
    - Pre-dimensioning of structural elements (slabs, beams, columns, walls).
    - Reinforcement design for secondary elements (slabs, staircases).
2. Structural Analysis & Primary Reinforcement:
    - Development and analysis of a detailed 3D model using ETABS Structural Analysis Professional.
    - Determination of vibrational characteristics (natural periods, mode shapes).
    - Calculation of internal forces (moments, shears, axial loads) in beams, columns, and shear walls.
    - Design of reinforcement for primary structural members based on the analysis results.
3. Foundation Design: Analysis of the foundation system considering building loads and site-specific geotechnical data. The civil engineering demands the creation of safe, efficient, functional, and code-compliant infrastructure.

Chapter I

Generalities

I.1: Introduction

The study of a reinforced concrete building requires basic knowledge on which the engineer draws in order to obtain a structure that is both safe and economical. Therefore, the objective of this first chapter is to identify the geometric characteristics of the structure and the mechanical properties of the materials used in its construction as well as the purpose of our study.

I.2: Project presentation

Our project involves the study and structural design of a building (Ground floor+ 8 floors + loft + basement) for commercial and residential to be constructed in Ihaddaden, in the wilaya of Béjaïa.

Its total height is less than 48 meters, which leads us to classify it, according to the Algerian seismic regulations “RPA99/version 2003 Art 3.2” in group 2 with medium importance and as a moderate seismic zone (IIa).

I.3: Geometric characteristics of the structure

- *Architectural features*
- Basement meant for parking use
- Ground floor and loft for commercial use
- The remaining floors for residential use,
- Accessible terrace
- Roofing

I.3.1: Elevation dimensions

- ✓ Total height ………………………………………………………………………38.34m
- ✓ Basement height…..3.57m
- ✓ Ground floor height……………………………………………………….3.74m
- ✓ Loft height……………………………………………………………………….2.89m
- ✓ Height of typical floors…………………………………………………3.06m
- ✓ Terrace height………………………………………………………………….3.06m

I.3.2: Plan dimensions

- ✓ Plan length………………………………………………………………………….28.20m
- ✓ Plan width…………………………………………………………………………18.10m

I.3.3 Geotechnical data of the site

We used the data provided by the geotechnical report regarding the foundation soil. It indicates that the structure is located on a relatively flat terrain composed of silty deposits, sand and alluvium soil. The soil has a bearing capacity of 1.5bars (allowable stress) with a minimum anchoring depth of 2.00m. The soil is therefore classified as soft soil(S3) according to RPA 2003 classification.

I.4: Structural elements of the building

I.4.1: The structural framework

The building has a mixed reinforced concrete frame structure composed of frames and shear walls.

a) *Frames*: These are reinforced concrete elements made up of beams and columns, primarily responsible for bearing vertical loads and imposed loads.
b) *Shear walls*: These are rigid in-situ cast reinforced concrete elements. They serve two purposes: to carry part of the vertical loads and to ensure the stability of the structure against horizontal loads.
c) *Floor slabs:* They form the horizontal part of the construction and are mainly used to separate two successive floors. The other functions are:

- Structural function: To support their self-weight and the imposed loads
- Insulation function: To provide thermal and acoustic insulation between the different floors.

Two types of floors are used in this structure,

- Hollow-core slab systems: made up of hollow core blocks and a compression slab resting on joists.
- Solid reinforced concrete slab: a slab cast in place, intended for balconies and elevator shaft level.

I.4.2: Wall infill

Two types of walls are presented in our structure:

1. External walls: Build using double brick partitions of hollow bricks, of 15cm and 10cm separated by 5cm air gap.
2. Interior partition walls: Made of 10cm thick hollow bricks.

I.4.3 Finishes

- Cement mortar for exterior walls and facades.
- Gypsum Plaster for interior partitions and ceilings.
- Tiles for floors and staircase.
- Ceramic tiles for kitchens and bathrooms.

I.4.4: Staircases

A staircase is a structure consisting of horizontal steps (steps and landing) providing access to different levels. The stairs are cast in place.

I.4.5: Balconies

They are reinforced areas at the end of each floor level

I.4.6: Parapet wall (Acroterion)

This is a reinforced concrete element, surrounding the building embedded at its base in the inaccessible terrace floor with a height of 0.7m.

I.4.7: Elevator shaft

The elevator is a lifting device allowing vertical movement and access to the different levels of the building.

I.4.8: Infrastructure

This is the part consisting of the foundation, located below ground level and it ensures the following functions,

- Transmission of vertical and horizontal loads to the ground.
- Limitation of differential settlements.

I.5: The regulations used

The design of this structure complies with the following regulations,

- BAEL91 modified in 1999: Technical rules for the design and calculations of reinforced concrete structures at limit states.
- DTR B.C.2.2: Regulatory technical document for the determination of loads and overloads.
- DTR B.C.3.3.1: Regulatory technical document for the design of shallow foundations.
- RPA99 modified in 2003: Algerian seismic code.
- CBA93: Reinforced concrete code.

I.6: Mechanical characteristics of materials

1.6.1: Concrete

Concrete is a mixture of aggregates (sand and gravel), a hydraulic binder(cement) and mixing water. Its strength varies with the quality of the latter and the age of the concrete.

a) ***Experimental behavior***

1. **Compression test**

Concrete has a relatively good resistance to compression compared to tension. This compressive strength ($f_c\ in\ MPa$) is obtained with compressive tests until rupture on standardized specimens.

2. **Concrete creep**

Under constant loading, the deformation of concrete increases continuously over time.

3. **Shrinkage phenomenon**

After casting, a concrete element stored in air tends to shorten. This is due to the evaporation of water not chemically bound with the cement (as a consequence of shrinkage, internal stresses appear, which can lead to the formation of cracks)

4. **Thermal expansion**

Temperature variation can generate tensile stresses that cause expansion. To accommodate this, expansion joints are regularly placed at intervals of 25 to 50m between structural elements).

b) Compressive strength characteristic

The compressive strength characteristic of concrete (f_{cj}) at “j” days of age is determined from tests performed on standard specimens. In practice, since the number of tests carried out does not allow sufficient statistical analysis, the following simplified relationship is adopted:

$$f_{cj} = \frac{\sigma_j}{1.15}$$

σ_j: Average value of the compressive strengths obtained over all the tests performed

The 28-day strength is commonly used for calculations during the construction phase. Values for “j” days are derived from "f_{c28}" using the following formulas:

$For\ j \ \leq\ 28\ days$

- $f_{cj} = \frac{j}{4.76+0.83j} \times f_{c28}$ for $f_{c28} \leq 40MPa$
- $f_{cj} = \frac{j}{1.40+0.95j} \times f_{c28}$for $f_{c28} \geq 40MPa$

$For\ j\ >\ 28\ days, f_{c28} = f_{cj}$

For this project, based on **BAEL 91 modified 99 art-2.1,13**, the compressive strength characteristic adopted for concrete used in residential and commercial buildings is:

$$f_{c28} = 25MPa$$

c) Tensile strength characteristic

it is defined by the following formula,

$f_{tj} = 0,6 + 0.06\, f_{cj}$ from which $f_{t28} =\ 2.1MPa$

- *Elastic modulus (Art-2.1.21 BAEL 99)*

The elasticity modulus is the ratio of normal stress and the corresponding strain. Depending on the duration of the stress application, two types of elastic modulus moduli are distinguished:

TABLE I 1: TYPES OF ELASTIC MODULI

Instantaneous Elastic Modulus (MPa)	Long-term Elastic Modulus (MPa)	Transverse Elastic Modulus	Poisson’s Ratio V
$E_{ij} = 11000 \times \sqrt[3]{f_{cj}}$	$E_{vj} = 3700 \times \sqrt[3]{f_{cj}}$	$G = \frac{E}{2(1+v)}$	$v = \frac{\frac{\Delta d}{d}}{\frac{\Delta 1}{1}}$
$E_{i25} =\ 32164.196$	$E_{v25} = 10818.866$		

d) Limit stresses

a. Compressive limit stresses

$$f_{bc} = \frac{0.85 * f_{c28}}{\theta \gamma b}(MPa)$$

γ_b: Safety coefficient.

$\gamma_b : = 1.50$ in normal situations $f_{bc} = 14.20MPa$

$\gamma_b = 1.15$ in accidental situations $f_{bc} = 18.48MPa$

$\theta = 0.85\ to\ 1$, depending on the duration of the action combination considered.

b. Shear limit stress (art-5.121 BAEL 99)

- $\tau_u = \min\{0.13 * f_{c28}\ ;\ 5MPa\}$ for slightly harmful cracking
- $\tau_u = \min\{0.10 * f_{c28}\ ;\ 4MPa\}$ for harmful cracking

c. Compressive service stress (art A-4.5.2 BAEL 99)

$\sigma_{bc} = 0.60 * f_{c28}MPa)$ $\sigma_{bc} = 15mpa$

d. stress-strain diagram

This diagram, figure I.1, can be used in all cases. It consists of a second-degree parabolic arc, extended at its apex by a horizontal plateau.

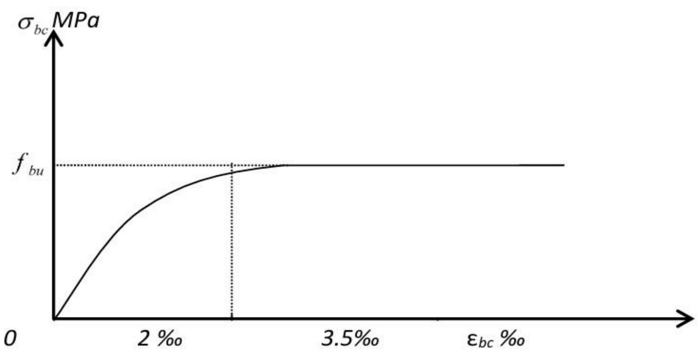


FIGURE I- 1: STRESS-STRAIN DIAGRAM

I.6.2: Steel

Steel is an alloy of iron and carbon in small percentages. Their role is to resist tensile, shear and torsional forces.

- ***Characteristics of the steel used***

The main characteristics are grouped in Table I.2:

TABLE I 2: STEEL CHARACTERISTICS.

Types of steel	symbols	Yield strength MPa	Ultimate strength	Relative elongation at break (%)	Cracking coefficients	Sealing coefficient
Smooth Round Fe 235	SM	235	410-490	22%	1	1
High Bond F_e 400	HA	400	480	14%	1.6	1.5

Welded wire mesh	TS	520	550	8%	1.3	1

- **Longitudinal yield strength**

It is denoted as E_S. Its value is constant regardless of the steel grade.

$E_S = 200000MPa$

- **Design stress-strain diagram**

In ultimate limit state (ULS) design, a simplified stress-strain diagram is used (figure I.2)

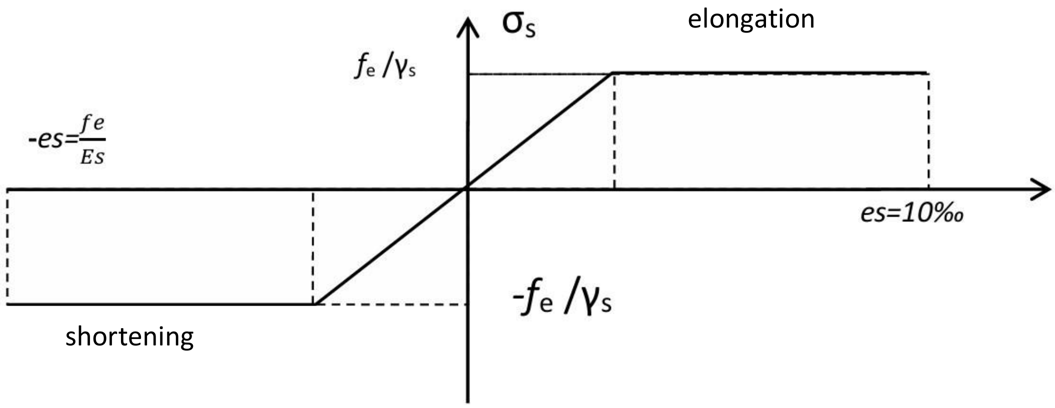


FIGURE I- 2: ULTIMATE ELASTIC LIMIT STRESS (ART A.2.2.2 BAEL 99)

$\sigma_S = \frac{f_e}{\gamma_s}$

γ : safety factor (γ =1.15 in a normal situation and γ =1.00 in an accidental situation)

- **Allowable stress at SLS (art A.4.5 BAEL 91)**

Slightly harmful cracking; $\sigma_{st} = f_e$

Harmful cracking; $\sigma_{st} = min\ (\frac{2}{3}f_e\ :\ 110\sqrt{\eta f_{t28}})$

Very harmful cracking; ; $\sigma_{st} = min(0.5f_e\ :\ 90\sqrt{n f_{t28}})$

Where n: cracking coefficient with $\begin{cases} n = 1\ for\ smooth\ round\ bars(RL) \\ n = 1.6\ for\ high - bond\ bars(HA) \end{cases}$

- **Protection of reinforcement**

In order to ensure proper concreting and protect the reinforcement from the effects of weathering, care must be taken to ensure that the cover C of the reinforcement complies with the following requirements,

$C \geq 5cm$: For the elements exposed to the sea, sea spray or salt fog, and for elements exposed to very aggressive atmospheres.

$C \geq 3cm$: For elements located with a liquid (tanks, pipes, pipelines)

$C \geq 1cm$: For walls located in rooms not exposed to condensation.

I.7: Conclusion

In this chapter, we presented the structure to be studied and defined the different elements that compose it, as well as the choice of materials used. This is with the aim of deepening this study to carry out precise pre-dimensioning in the following chapter, in order to ensure good resistance of the materials that make up our construction.

Chapter II

Pre-dimensioning of elements

Introduction

The purpose of pre-dimensioning is to give the initial dimensions to the different elements of our building in order to absorb the different forces due to stresses according to the requirements of the regulations in force, CBA 93, BAEL 99 and RPA 2003.

II.1: Pre-dimensioning of secondary elements

II.1.1: Hollow core slab

It is composed of (Figure 1):

- Reinforced joists
- Hollow core
- Reinforced concrete compression slab

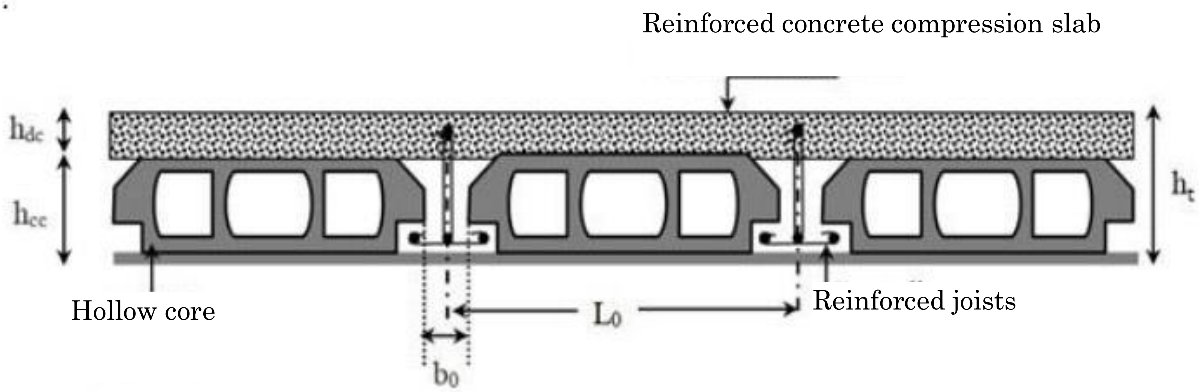


FIGURE II- 1: HOLLOW CORE SLAB

Its preliminary design is done by satisfying the CBA condition (art B.6.8.4.2.4) given by:

$$h_t \geq \frac{L_{max}}{22{\cdot}5}$$

- h_t is the total height of the floor (h_{cc} + h_{dc}).
- L_{max} is the maximum length between supports according to the adopted arrangement of the joists (Figure II.2).

❖ For the inaccessible terrace:

$Lmax = 550 - 30 = 520cm$

$h_t \geq \frac{520}{22{\cdot}5} = 23.11\text{cm}$

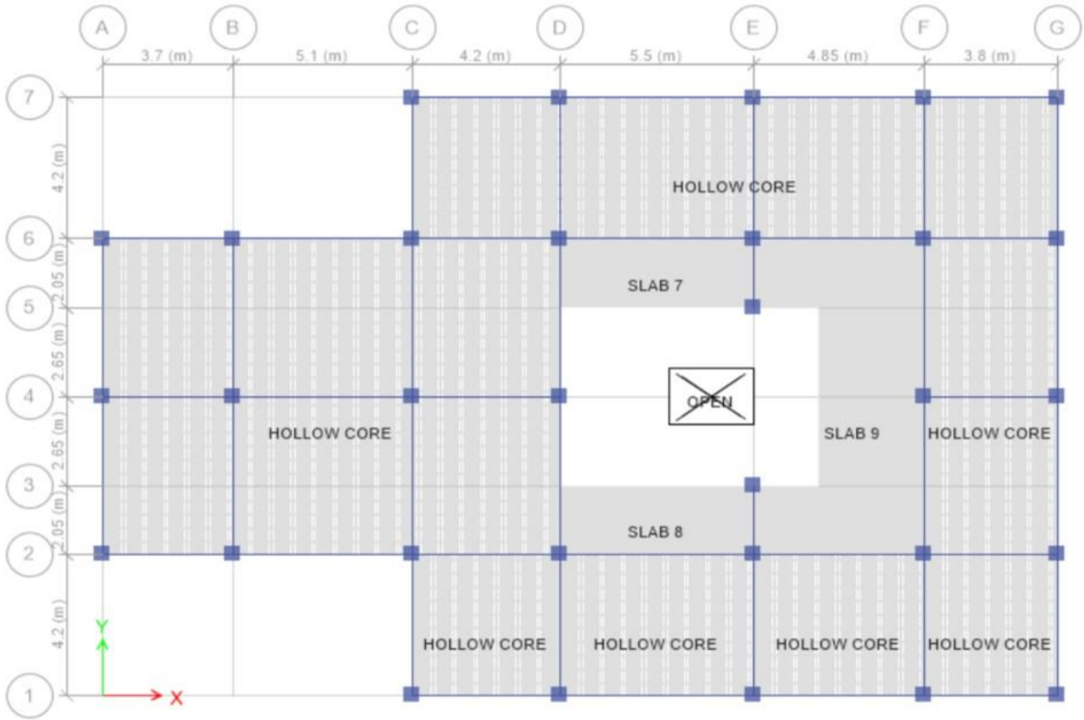
We take **h_t = 24cm (20+4) cm**

❖ For all the other floors,

$Lmax = 470 - 30 = 440cm$

$ht \geq \frac{440}{22{\cdot}5} = 19.55cm$

We take h_t = **20cm (16+4) cm**



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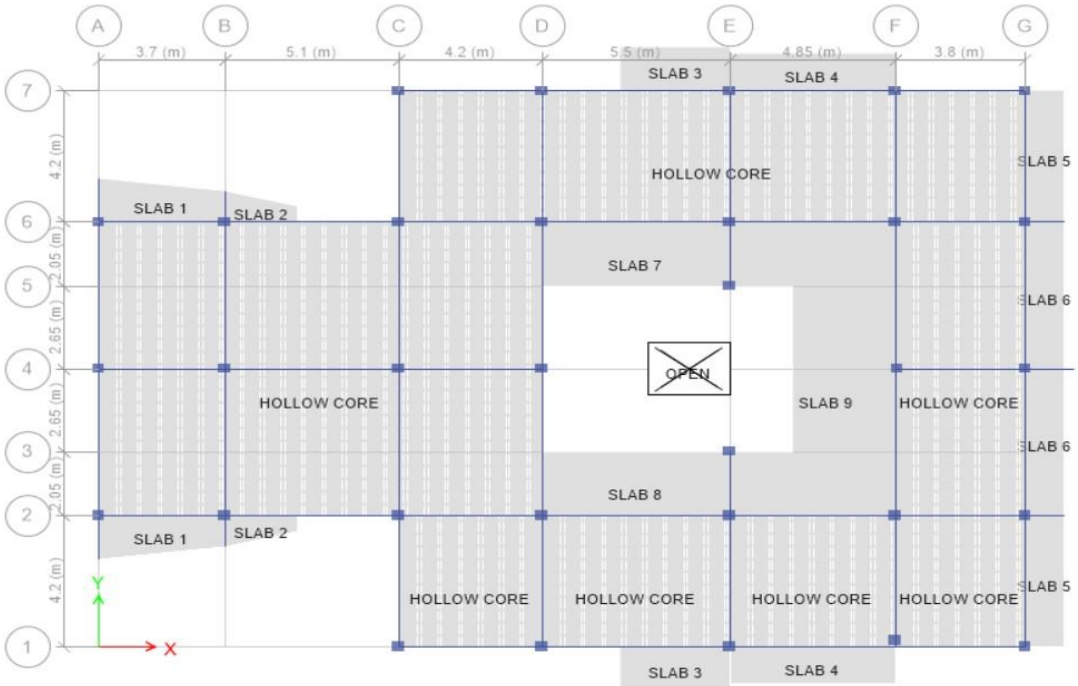


-b-

Figure II- 2 : The adopted joists arrangement

-a- Basement and ground floor

-b- 1st floor to inaccessible terrace

- ❖ **Joists preliminary design**

The joists are prefabricated or cast-in-place elements, designed for simple bending as T-sections.

Figure II.3 shows the dimensions of the T-section required for its calculation.

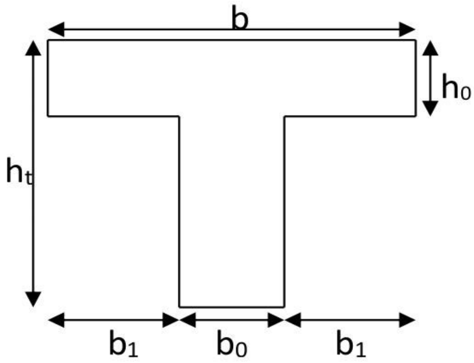


FIGURE II- 3 : THE T-SECTION DIMENSIONS

- h_t is the total height of the slab.
- h_o is the compression slab thickness.
- b_o is the rib width.
- b_o= (0.4 to 0.6) ht b_o= (9.6 to 14.4) cm we choose bo=10cm
- b-rib spacing. **b is the effective width of the compression slab, calculated using the following formula**

$\frac{b-bo}{2} \leq \min\left(\frac{lx}{2}, \frac{ly}{10}\right)$ (CBA art 4.1.3)

lx is the distance between two successive joists.

ly is the minimum spam length in the direction of the joist arrangement between bearing faces

$lx = 60 - 10 = 50cm$

$ly = 370 - 30 = 340cm$ Therefore $\frac{b-10}{2} \leq min(\frac{50}{2}; \frac{340}{10})$ which gives b = 60cm

II.2.2: Solid concrete slab

The preliminary design of a solid slab floor involves determining its thickness (t) which depends on the number of supports on which it rests and its fire resistance and sound proofing in accordance with CBA 93.

- ❖ **Bending resistance**
  - Slab resting on 1 support (cantilever slab) $t \geq \frac{lx}{20}$
  - Slab resting on 2,3 or 4 supports with ƒ≥0.4; $\frac{lx}{45} \leq t \leq \frac{lx}{40}$
  - Slab resting on 2 or 4 slabs with ƒ<0.4 $\frac{lx}{35} \leq t \leq \frac{lx}{30}$
  - Slab resting on 2 parallel supports $\frac{lx}{30} \leq t \leq \frac{lx}{20}$

$\rho = \frac{lx}{ly}$

lx is the smallest spam of the slab,

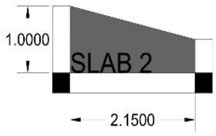
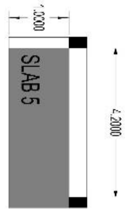
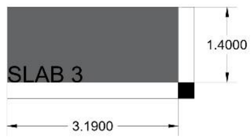
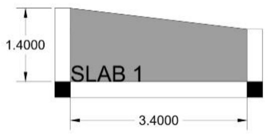
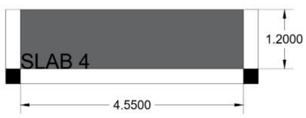
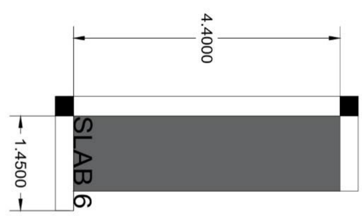
ly is the largest spam of the slab.

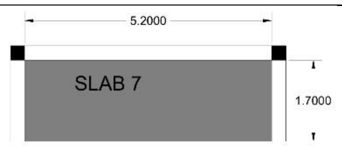
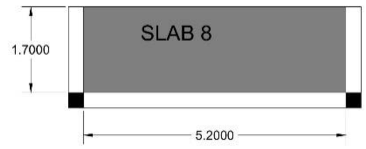
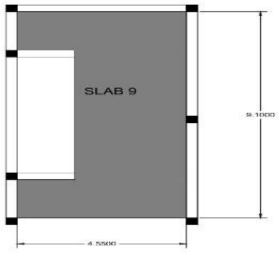
- ❖ **Fire resistance**
- $t \geq 7cm$; for 1 hour of firing
- $t \geq 11cm$; for 2 hours of firing
- $t \geq 14cm$; for 4 hours of firing
- ❖ **Sound proofing**

According to CBA 93, the thickness of the floor must be superior or equal to 13cm (for internal slabs) to obtain good soundproofing.

In our building we have several types of slabs whose location is shown on figure II.2. We have summarized them in table II.1:

TABLE II 1: DIFFERENT TYPES OF SOLID SLABS AND THEIR DIMENSIONS

Number of Supports	Slabs	f	t (cm)	Resistance
2 supports		f_2=0.46 f_3=0.43 f_5=0.24	$2.22 \leq t \leq 2.5$ $3.11 \leq t \leq 3.5$ $2.88 \leq t \leq 3.33$	$t \geq 11$ $t \geq 11$ $t \geq 11$
3 supports		f_1=0.41 f_4=0.26 f_6=0.30 f_7=0.32 f_8=0.32	$3.11 \leq t \leq 3.5$ $t \geq 6$ $t \geq 6.5$ $t \geq 8.50$ $t \geq 8.50$	$t \geq 11$ $t \geq 11$ $t \geq 11$ $t \geq 13$ $t \geq 13$

				
4 supports		ρ_9=0.50	10.11 ≤ t ≤ 11.36	t ≥ 13

We opt for the solid slabs D1, D2, D3, D4, D5 and D6 a thickness t=15cm, and a thickness of 20cm (adjusted thickness after reinforcement calculation) for solid slabs D7, D8, and D9 because they are supporting the stairwell and the elevator.

II.1.3: Preliminary design of staircases

To guarantee the staircase performs its role in the most comfortable conditions, we must verify the following conditions:

Blondel condition: $59 \leq 2h + g \leq 64$

14cm<h<18cm ; with $h = \frac{H}{n}$

25cm<g<32cm ; with $g = \frac{Lo}{n-1}$

With

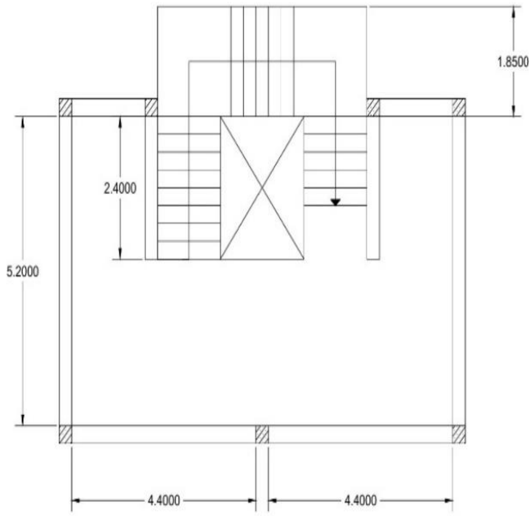
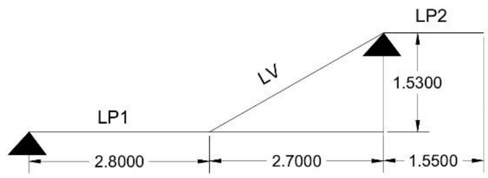
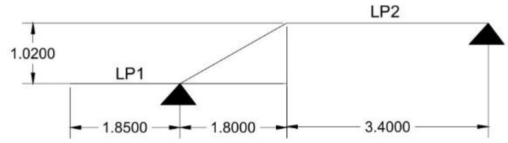
- Lo: Horizontal projection of the stair slab.
- Tread(g): Width of the step.
- H: Height of the step.
- n: Number of risers.

We have three types of staircases in our building:

- **Stairs with three flights**
  - **Basement**

g = 30cm; h = 17cm

TABLE II 2: DIAGRAMS OF THE BASEMENT STAIRS

	1st Flight
	
	2nd Flight
	
	3rd Flight
	

- **Flight 1**

L= $Lp_1 + L_v$

$L_V = \sqrt{1.53^2 + 2.70^2} = 3.10\text{m}$

$L = 2.80 + 3.10 = 5.90\text{m}$

$\frac{L}{30} \leq \text{ t } \leq \frac{L}{20}$

$\frac{590}{30} \leq \text{ t } \leq \frac{590}{20}$

$19.67cm \leq t \leq 29.50cm$

$t \geq \frac{L_{p2}}{20}$

$t \geq \frac{155}{20} = 7.75cm$

- **<u>2^nd Flight</u>**

t$\geq \frac{L}{20} = \frac{155}{20} = 7.75cm$

<u>3^rd Flight</u>

$t \geq \frac{Lp1}{20} = \frac{185}{20} = 9.25cm$

L= $Lp_1 + L_v$

$L_V = \sqrt{1.8^2 + 1.02^2} = 2.07cm$

$L = 2.07 + 3.40 = 5.47m$

$\frac{L}{30} \leq \frac{L}{20} \quad = \quad \frac{547}{30} \leq \frac{547}{20}$

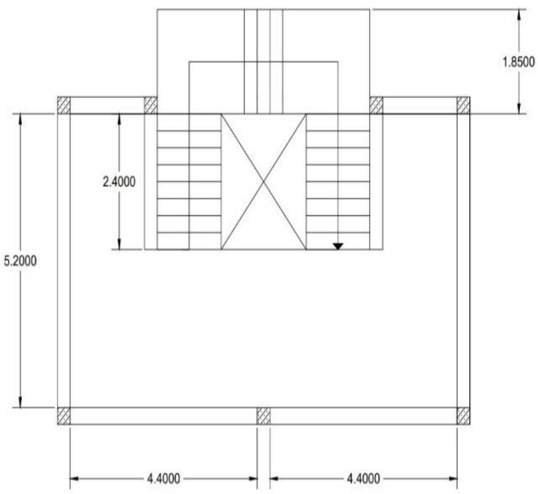
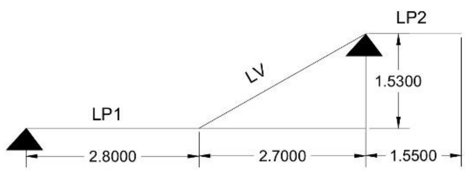
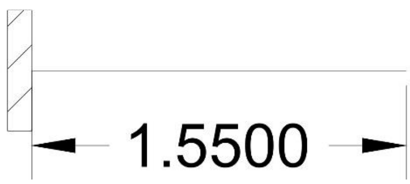
$18.23cm \leq t \leq 27.35cm$

For the staircases on basement, we have taken **t=20cm**

- **Ground floor (Figure**

The calculations were conducted in the same manner as the basement staircase.

TABLE II 3: DIAGRAMS OF THE GROUND FLOOR STAIRS.

<table>
<tr><td></td><td>1ST AND 3RD FLIGHT</td></tr>
<tr><td rowspan="3">
</td><td>
</td></tr>
<tr><td>2ND FLIGHT</td></tr>
<tr><td>
</td></tr>
</table>

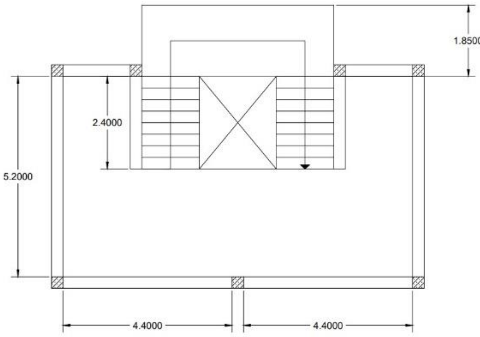
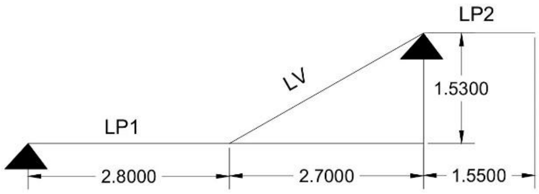
For all the ground floor staircases, we take t=20cm

NB: All the other staircases on this floor are made up of wood.

- ❖ **Stairs with two flights (all the main floors)**

Table II.4: Diagrams of the main floor's stairs.

TABLE II 4: DIAGRAMS OF THE MAIN FLOOR'S STAIRS.

<table>
<tr><th></th><th>1st and 2nd flights</th></tr>
<tr><td>
</td><td>
</td></tr>
</table>

L= $Lp_1 + L_v = 5.90m$

$19.67cm \leq t \leq 29.5cm$

t $\geq \frac{Lp2}{20} = \frac{155}{20} = 7.75cm$

We take t=20cm

II.1.4 Acroterion

Acroterion is considered as a vertical cantilever embedded in the terrace floor. Its role is to prevent rainwater infiltration between the slope and the terrace floor. It is also used to hang building maintenance materials. Its dimensions are shown in Figure II.7.

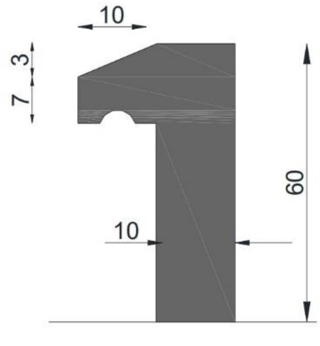


FIGURE II- 4: ACROTERION DIAGRAM

Surface area = (0.6*0.1) + (0.07*0.1) +($\frac{0.03*0.1}{2}$) =0.0685m^2

II.2 Pre-dimensioning of principal elements

II.2.1 Preliminary design of beams

The beam height h is given by the following formula:

$\frac{lmax}{15} \leq h \leq \frac{lmax}{10}$ l_{max} -The largest span (between bearing faces) in the considered direction.

- **Main beams**
  - ***Inaccessible terrace***

l_{max} =570cm-30cm = 540cm

$\frac{540}{15} \leq h \leq \frac{540}{10}$ $36 \leq h \leq 54$We take h=40cm and b=30cm.

- ***The other floors***

l_{max} =550cm-30cm = 520cm

$\frac{520}{15} \leq h \leq \frac{520}{10} 34.66 \leq h \leq 52$We take h=35cm and b=30cm.

- **The secondary beams**
  - ***Inaccessible terrace***

l_{max} = 420cm-30cm = 390cm

$26 \leq h \leq 39$We take h=30cm and b=30cm.

- ***The other floors***

l_{max} = 470cm-30cm = 440cm

$29.33 \leq h \leq 44$We take h=30cm and b=30cm.

- **Verification of the RPA 2003 conditions**

Table II.5 summarizes the verification results of the RPA 2003 requirements for the beams.

TABLE II 5: VERIFICATIONS OF RPA CONDITIONS FOR THE BEAMS

	Main beams		Secondary beams	Observation
	Roofing	Other floors		
Height (cm)	40	35	30	Verified
Width (cm)	30	30	30	Verified
Height⁄Width	1.33	1.16	1	Verified

II.2.2 Preliminary design of columns

The columns preliminary design is done according to the following criteria:

- Resistance criteria
- Form-stability criteria Buckling
- Verifying the RPA99/VERSION 2003 conditions

The predesigning of columns is done by simple compression according to the rules of BAEL 91 ART 8.8.4.1.

Once the comprehensive strength is verified, this column must meet the criteria and recommendations of RPA99 VERSION 2003.The dimensions assumed are fixed after the distribution of loads.

The column that we will be studying is the most stressed column, that is to say the column that will receive the maximum compression, which is D4 (Figure II.2)

We will fix the dimensions of the columns as follows,

TABLE II 6: COLUMN DIMENSIONS

STORY	Section (cm^2)	Height (cm)	Weight (kN)
Basement	55×55	3.57	26.998
Ground floor	50×55	3.74	25.713
Loft	50×50	2.89	18.063
1st floor	50×50	3.06	19.125
2nd and 3rd floor	45×45	3.06	15.491
4th and 5th floor	40×40	3.06	12.24
6th and 7th floor	35×35	3.06	9.371
8th floor and terrace	30×30	3.06	6.89

II.2.2.1 Evaluation of loads and overloads

The evaluation of loads and overloads of each element will make it possible to make the distribution of loads on the columns in order to design the building.

The results are summarized in the tables below,

TABLE II 7: EVALUATION OF LOADS ON INACCESSIBLE TERRACE FLOOR WITH HOLLOW CORE SLAB

Designation of elements	t(m)	γ(kN/m^3)	Weight(kN/m^2)
Protective gravel	0.05	20	1.00
Waterproof layer	0.02	6	0.12
Thermal insulation	0.04	4	0.16
Slope shape	0.10	22	2.20
Hollow core slab	0.24	/	3.30

Cement coating	0.02	18	0.36
Dead load G			7.14
Live load Q			1.00

TABLE II 8: EVALUATION OF LOADS ON ACCESSIBLE TERRACE FLOOR WITH HOLLOW CORE SLAB

Designation of elements	t(m)	γ(kN/m^3)	Weight(kN/m^2)
Tile covering	0.02	20	0.40
Setting mortar	0.02	20	0.40
Sand bed	0.02	18	0.36
Hollow core slab	0.20	/	2.85
Slope shape	0.10	22	2.20
Gypsum coating	0.02	10	0.20
Dead load G			6.41
Live load Q			1.50

TABLE II 9: EVALUATION OF LOADS ON ACCESSIBLE TERRACE FLOOR WITH SOLID SLAB

Designation of elements	t(m)	γ(kN/m^3)	Weight(kN/m^2)
Tile covering	0.02	20	0.40
Setting mortar	0.02	20	0.40
Sand bed	0.02	18	0.36
Solid slab	0.15	25	3.75
	0.20	25	5.00
Slope shape	0.10	22	2.20
Gypsum coating	0.02	10	0.20
Dead load G(t=0.15m)			7.31
Dead load G(t=0.20m)			8.56

Live load Q	1.50

TABLE II 10: EVALUATION OF LOADS ON MAIN FLOORS WITH HOLLOW CORE SLAB

Designation of elements	t(m)	γ(kN/m^3)	Weight(kN/m^2)
Tile covering	0.02	20	0.40
Setting mortar	0.02	20	0.40
Sand bed	0.02	18	0.36
Hollow core slab	0.20	/	2.85
Gypsum coating	0.02	10	0.20
Partition wall	0.10	/	1.00
Dead load G			5.21
Live load Q(Residential)			1.50
Live load Q(commercial)			5.00

TABLE II 11: EVALUATION OF LOADS ON MAIN FLOORS WITH SOLID SLAB

Designation of elements	t(m)	γ(kN/m^3)	Weight(kN/m^2)
Tile covering	0.02	20	0.40
Setting mortar	0.02	20	0.40
Sand bed	0.02	18	0.36
Solid slab	0.15	25	3.75
	0.20	25	5.00
Partition wall	0.10	/	1.00
Gypsum coating	0.02	10	0.20
Dead load G	t=0.15m		6.11
	t=0.20m		7.36

Live load Q	Residential	1.5
	Commercial	5.00
	Balcony	3.50

TABLE II 12: EVALUATION OF LOADS ON THE FLIGHTS

Designation of elements	t(m)	γ(kN/m^3)	Weight(kN/m^2)
Stair slab	$\frac{0.20}{cos\ \alpha} = 0.23$	25	5.75
Horizontal tiles	0.02	20	0.40
Horizontal setting mortar	0.02	20	0.40
Vertical tiles	$0.02 \times {}^{h}/_{g} = 0.0113$	20	0.23
Vertical setting mortar	$0.02 \times {}^{h}/_{g} = 0.0113$	20	0.23
Weight of steps	${}^{h}/_{2} = 0.085$	22	1.87
Cement coating	0.02	18	0.36
Dead load G			9.24
Live load Q			2.50

TABLE II 13: EVALUATION OF LOADS ON STAIRCASE SLABS

Designation of elements	t(m)	γ(kN/m^3)	Weight(kN/m^2)
Stair slab	0.20	25	5.00
Sand bed	0.02	18	0.36
Setting mortar	0.02	20	0.40
Tiles covering	0.02	20	0.40
Cement layer	0.02	18	0.36
Dead load G			6.52
live load Q			2.50

TABLE II 14: EVALUATION OF LOADS OF EXTERIOR WALLS

Designation of elements	t(m)	γ(kN/m³)	Weight(kN/m²)
Cement coating	0.02	18	0.36
Hollow bricks	0.15	/	1.30
Hollow bricks	0.10	/	0.90
Plaster coating	0.02	10	0.20
Dead load G			2.76

II.2.2.2 Distribution of loads

The distribution of loads is the path followed by the different actions loads and overloads from the highest to the lowest level of the structure before its transmission to the ground.

The distribution of the loads is carried out for the most stressed column and which often has the larger related surface area.

- **The law of load degression of live or operating load (DTR B.C 2.2 ART. 6.3)**

The live loads that will be adopted from the highest point of the building to the lowest point as follows,

Level 12 ;Q_0

Level 11 ;Q_0+Q_1

Level 10 ;$Q_0+0.95(Q_1+Q_2)$

Level 9 ;$Q_0+0.90(Q_1+Q_2+Q_3)$

Level 8 ;$Q_0+0.85(Q_1+Q_2+Q_3+Q_4)$

Level 7 ;$Q_0+0.80(Q_1+Q_2+Q_3+Q_4+Q_5)$

Level 6 ;$Q_0+0.75(Q_1+Q_2+Q_3+Q_4+Q_5+Q_6)$

Level 5 ;$Q_0+0.714(Q_1+Q_2+Q_3+Q_4+Q_5+Q_6+Q_7)$

Level 4 ;$Q_0+0.687(Q_1+Q_2+Q_3+Q_4+Q_5+Q_6+Q7+Q8)$

Level 3 ;$Q_0+0.667(Q_1+Q_2+Q_3+Q_4+Q_5+Q_6+Q7+Q8+Q_9)$

Level 2 ;$Q_0+0.650(Q_1+Q_2+Q_3+Q_4+Q_5+Q_6+Q7+Q8+Q_9) +Q_{10}$

Level 1 ;$Q_0+0.636(Q_1+Q_2+Q_3+Q_4+Q_5+Q_6+Q7+Q8+Q_9) +Q_{10}+Q_{11}$

NB: From n>5, the law of degression becomes;

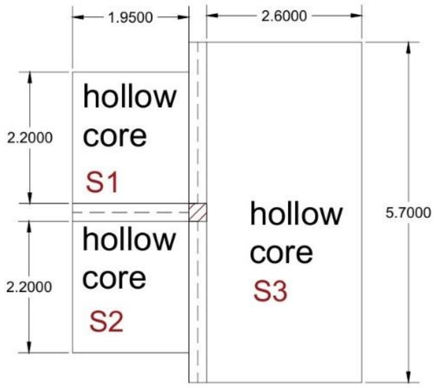
$$Q_0+\frac{3+n}{2n}(Q_1+Q_2+\cdots+Q_n)$$

n; floor number from the top of the building.

According to article 6.2 of DTR B.C 2.2 the commercial premises will be taken into account without reduction.

II.2.2.3 Predesigning of column D4

The different surface area of column D4 are shown in figure II.8.



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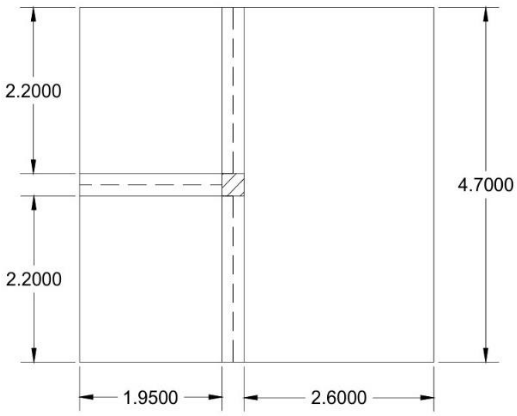

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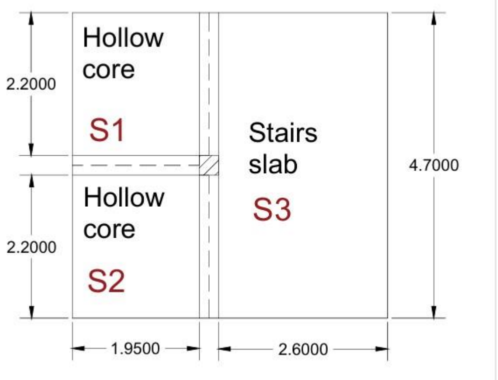


-C-

FIGURE II- 5: SURFACE AREA OF COLUMN D4

-a- Roof

-b- Accessible terrace

-c- Ground floor to 8th floor

- **LEVEL 13 (ROOF)**

G_{floor} = 7.14×23.4 = 167.08kN

Main beams weight = 25× (5.7-0.3) ×0.3×0.4=16.2kN

Secondary beams weight= 25×1.95×0.3×0.3=4.39kN

Q_{floor} = 1×23.4 = 23.4Kn

- **Level 12** (TERRACE)

Column (30×30) weight = 6.89kN

Main beams weight = 25× 1.95 ×0.3×0.35=5.119kN

Secondary beams weight = 25× 0.3×0.3×4.40=9.90kN

S_{wall}= (2.2+2.2+1.95) (3.06-0.3) =17.53m^2

G_{wall}=2.76×17.53=43.37kN

- **Level 11** (FLOOR 8)

Column weight (30×30) =6.89KN

Main and secondary beams weight=15.019kN

G_{floor}=(S1+S2) ×$G_{hollow\ core\ slab}$ + S3× $G_{Stair\ slab}$

G_{floor}= (4.29+4.29) ×5.21+(12.22×6.52) =124.276kN

S_{wall}= (1.95+1.4+1.4) × 2.76 =13.11m^2

G_{wall}=13.11×2.76=36.184kN

Q= (12.22×2.5) +(4.29+4.29) ×1.5=43.42kN

- **Level 10 and 9** (FLOOR 7 and FLOOR 6)

Column weight (35×35) =9.371KN

G_{floor}=124.276kN

S_{wall}= (1.95× (3.06-0.3)) +((1.4+1.4) × (3.06-0.2) =13.39cm^2

G_{wall}=13.39×2.76=36.956Kn

- **Level 8 and 7** (FLOOR 5 AND FLOOR 4)

Column weight (40×40) =12.24KN

G_{floor}=124.276kN

G_{wall}=36.956kN

- **Level 6 and 5** (FLOOR 3 AND FLOOR 2)

Column weight (45×45) =15.491KN

G_{floor}=124.276kN

G_{wall}=36.956kN

- **Level 4** (FLOOR 1)

Column weight (50×50) =19.125KN

G_{floor}=124.276kN

G_{wall}=36.956kN

- **Level 3** (LOFT)

Column weight (50×50) =18.063KN

G_{floor}=124.276kn

S_{wall}= (2.2+2.2) × (2.89-0.3) =11.396m^2

G_{wall}=11.396×2.76=31.45kN

- **Level2** (GROUND FLOOR)

Column weight (50×55) =25.713KN

G_{floor}=124.276Kn

S_{wall}= (2.2+2.2) × (3.74-0.3) =15.136m^2

G_{wall}=15.136×2.76=41.78kN

- **Level 1** (BASEMENT)

Column weight (55×55) =26.998KN

S_{wall}= (2.2+2.2) × (3.57-0.3) =14.388m^2

G_{wall}=14.388×2.76=39.71Kn

TABLE II 15: RESULTS OF LOAD CALCULATION ON COLUMN D4

LEVEL	FLOOR	ELEMENT	G(kN)	Q(kN)	N_U (kN)
L13	Roof	Floor	167.68		
		Beam	20.59		
		SUM	188.27		
L12	Terrace	Origins of L13	188.27		
		Column	6.885		
		Wall	43.37		
		SUM	238.525	Q0=23.4	357.109
L11	Floor 8	Origins of L12	238.525	Q1=0	

		Beam	15.019	23.4	
		column	6.685		
		wall	36.184		
		SUM	296.613		435.528
L10	Floor 7	Origins of L11	296.613	Q2=43.42	
		Beam	15.019		
		Column	9.371		
		Floor	124.376		
		Wall	36.956		
		SUM	482.335	64.649	748.126
L09	Floor 6	Origins of L10	482.335	Q3=43.42	
		Beam	15.019		
		Column	9.371		
		Floor	124.376		
		Wall	36.956		
		SUM	668.057	101.556	1054.211
L08	Floor 5	Origins of L09	668.057	Q4=43.42	
		Beam	15.019		
		Column	12.24		
		Floor	124.376		
		Wall	36.956		
L07	Floor 4	SUM	856.648	134.121	1357.656
		Origins of LO8	856.648	Q5=43.42	
		Beam	15.019		
		Column	12.24		
		Floor	124.376		

		Wall	36.956		
		SUM	1045.239	162.344	1654.589
L06	Floor 3	Origins of L07	1045.239	Q6=43.42	
		Beam	15.019		
		Column	15.491		
		Floor	124.376		
		Wall	36.956		
		SUM	1237.081	186.225	1949.397
L05	Floor 2	Origins of L06	1237.081	Q7=43.42	
		Beam	15.019		
		Column	15.91		
		Floor	124.376		
		Wall	36.956		
		SUM	1428.923	209.411	2243.163
L04	Floor 1	Origins of L05	1428.923	Q8=43.42	
		Beam	15.019		
		Column	19.125		
		Floor	124.376		
		Wall	36.956		
		SUM	1624.399	232.207	2541.249
L03	Loft	Origins of L04	1624.399	Q9=43.42	
		Beam	15.019		
		Column	18.063		
		Floor	124.376		
		Wall	31.45		
		SUM	1813.307	255.089	2830.60

L02	Ground floor	Origins of L03	1813.307	Q10=73.45	
		Beam	15,019		
		column	25.713		
		floor	124.376		
		wall	41.78		
		SUM	2020.195	322.634	3211.21
L01	Basement	Origins of L02	2020.195	Q11=73.45	
		Beam	15.019		
		column	26.998		
		Floor	124.376		
		Wall	39.71		
		SUM	2226.298	391.22	3591.93
$\boldsymbol{Nu^{D4} = 3591.93kN}$					
NB/ We did not take into consideration the beam overload					

We followed the same approach to calculate the maximum effort at the base of column E3, from which we got $Nu^{E3} = 3200.84\text{kN}$

In order to take into account, the continuity of the gantries, the CBA (ART B.8.1.1) requires us to increase the force Nu as follows,

- **10%** ; Internal columns adjacent to the edge in the case of a building with at least 3 spans.
- **15%** ; Central columns in the case of a building with 2 spans.

In our case, our most stressed column is intermediate, which means the force N_U with not be increased.

$$Nu = Nu^{D4} = 3591.93kN$$

II.2.2.4 Verifications

- **Compression Verification**

The preliminary design is done by ULS, we verify the following condition;

$$\frac{N_U}{B} \leq f_{bu} \rightarrow B \geq \frac{N_U}{f_{bu}}$$

The verification results are summarized in table II.16:

TABLE II 16: COMPRESSION VERIFICATIONS

LEVEL	N_U	B (m^2)	$B_{cal} = \frac{Nu}{f_{bu}}$ (m^2)	OBS
12	357.09	0.090	0.025	Verified
11	435.528	0.090	0.031	//
10	748.126	0.123	0.053	//
9	1054.211	0.123	0.074	//
8	1357.656	0.160	0.096	//
7	1654.589	0.160	0.117	//
6	1949.397	0.203	0.137	//
5	2243.163	0.203	0.158	//
4	2541.249	0.250	0.179	//
3	2830.60	0.250	0.199	//
2	3211.21	0.275	0.226	//
1	3591.93	0.303	0.253	//

- **Shape stability verification (BUCKLING)**

According to CBA 93, we must verify,

$\mathrm{NU} * (\frac{Br \times fc28}{0.9 \times \gamma b}) + \mathrm{As} \times \frac{fe}{\gamma s}$

α=Reduction factor which depends on the slenderness (λ)

$\alpha = \frac{0.85}{\left(1+0.2\frac{\lambda}{35}\right)^2}$ if $0 \leq \lambda \leq 50$

$\alpha = 0.6\left(\frac{50}{\lambda}\right)^2$ if $50 \leq \lambda \leq 70$

With $\lambda = \frac{Lf}{i}$ and Lf = 0.7 × Lo

$i = \sqrt{\frac{I}{B}}$ and $I = \frac{b \times h^3}{12}$

According to BAEL,$\frac{As}{Br}$ ∈ (0.8%;1,2%) and we take $\frac{As}{Br} = 1\%$

We must verify that Br ≥ Br^{cal}.

The verification results are summarized in table II.17.

TABLE II 17 : SHAPE STABILITY VERIFICATIONS

LEV	N_U	l_o(m)	L_f(m)	i(m)	λ	α	Br(m^2)	Br^{cal}(m^2)	OBS
12	357.09	3.06	2.142	0.087	24.621	0.653	0.078	0.024	Verified
11	435.528	3.06	2.142	0.087	24.621	0.653	0.078	0.030	//
10	748.126	3.06	2.142	0.101	21.208	0.676	0.102	0.050	//
09	1054.21	3.06	2.142	0.101	21.208	0.676	0.102	0.071	//
08	1357.66	3.06	2.142	0.115	18.626	0.694	0.144	0.089	//
07	1654.59	3.06	2.142	0.115	18.626	0.694	0.144	0.108	//
06	1949.37	3.06	2.142	0.130	16.477	0.710	0.185	0.125	//
05	2243.13	3.06	2.142	0.130	16.477	0.710	0.185	0.144	//
04	2541.29	3.06	2.142	0.144	14.875	0.722	0.230	0.160	//
03	2830.60	2.89	2.023	0.144	14.049	0.728	0.230	0.176	//
02	3211.21	3.79	2.653	0.159	16.686	0.708	0.254	0.206	//
01	3591.93	3.54	2.478	0.159	15.585	0.717	0.281	0.228	//

All the columns are verified, hence there is no risk of buckling.

➢ **Verification of RPA conditions**

Our structure is implanted in the IIA zone, so the section of the columns must meet the following requirements;

- $\text{Min (b, h)} = 30\text{cm} > 25\text{cm}$… … … …verified
- $\text{Min (b, h)} = 30\text{cm} > 25\text{cm}\,\frac{he}{20} = 18.7$… … … …verified
- $\frac{1}{4} \leq \frac{h}{b} \leq 4$… … … …verified

Conclusion

The pre-dimensioning of the elements was developed based on the regulations in force namely BAEL and RPA. The dimensions adopted are summarized below,

❖ Hollow core slab
- Inaccessible terrace 20+4
- All the other floors 16+4

❖ Solid slab
- t=20cm for slab 7,8and 9
- t=15cm for all the other slabs

- ❖ Main beams
  - Roofing is 30*40 cm^2
  - All the other floors are 30*35 cm^2
- ❖ Secondary beams; 30*30 cm^2
- ❖ stairs=20cm
- ❖ Columns ; Their dimensions are summarized in the following table,

TABLE II 18: COLUMN DIMENSIONS

Story	Basement	Ground floor	Loft and 1^{st} floor	2^{nd} and 3^{rd} floor	4^{th} and 5^{th} floor
Dimensions	55*55	50*55	50*50	45*45	40*40

Story	6^{th} and 7^{th} floor	8^{th} floor and terrace
Dimensions	35*35	30*30

Chapter III

Study of secondary elements

III.1: Introduction

Having predesigned all the load-bearing elements in the previous chapter, this chapter will focus on calculating the following elements:

- Hollow-core slab and Solid slab floors,
- Stairs and the landing beam,
- Ring beam/ Tie beam/Perimeter beam,
- The acroterion.

III.2: Study of different types of floors

In our building, we have two types of floors namely, hollow-core slab and Solid slab floors.

III.2.1: Hollow-core slab

III.2.1.1: Methods of calculating stresses in joists

The joists are analyzed as continuous beams under the floor loads. The methods used for reinforced concrete beams are:

a. Comprehensive method (BAEL 99 Art B.6.2.210)

Conditions for application of the method

The method only applies to flexed members which satisfy the following conditions:

- Moderate service loading floor: Q $\leq$ Min (2G; 5 kN/m^2;
- The successive spans are in the ratio between 0.8 and 1.25 $0.8 \leq \frac{Li}{Li-1} \leq 1.25$
- The moments of inertia of the transverse sections are the same in the various continuous spans;
- Low harm cracking (LHC).

b. Calculating bending moments

✓ **Support moments**

✓ ***Edge support***: The moments on the edge support are zero but the BAEL99 **(**Reinforced Concrete at Limit States) requires placing crack-control reinforcement to balance 0.15 Mo.

✓ ***Intermediate support***: The bending moments are on the order of;

--**0.6 Mo** on the central support of two span beams.

- 0.5 Mo for the supports adjacent to the end supports in beams with more than two spans

- 0.4 Mo on all the other intermediate supports.

With: Mo: being the maximum of the two isostatic moments surrounding the support considered.

✓ **Maximum span moment M_t**

Let:

- ***Mₒ*** be the maximum value of the bending moment in the « reference span », i.e. In the independent span having the same free span as the span in question subject to the same loads.
- ***M_w*** and ***M_e*** respectively are the absolute values of the moments on the left and right hand supports and ***M_t*** is the maximum span moment which are taken into account in the calculations for the span in question.
- ***α*** be the ratio of the service loads to the sum of the dead and the service loads

$$\alpha = \frac{Q}{Q+G}$$

The values of ***M_t***, ***M_w*** and ***M_e*** are to be checked under the following conditions:

1. $M_t + \frac{M_w+M_e}{2} \geq (1.05\,;1+0.3\alpha)\,M_o$

2. $M_t \geq \left(\frac{1.2+0.3\alpha}{2}\right) M_o$ *in the case of an edge span*

$M_t \geq \left(\frac{1+0.3\alpha}{2}\right) M_o$ *in the case of an intermediate span*

- **Support Shear forces**

For shear design, the continuous beam is analyzed assuming discontinuous elements at supports, with the exception for the first intermediate support where the corresponding shear force must be increased by:

- ✓ 15% in the case of joists with two spans;
- ✓ 10% in the case of joists with more than two spans.

c. Caquot

The method applies essentially to floors having a relatively high service load (service load more than twice the dead load or 5 kN/m²). It can also be applied to floors with moderate service load when one of the additional conditions of the comprehensive method is not fulfilled. In this case we use ***Reduced Caquot method*** where the support's bending moments are determined by the Caquot's *method* using G'= $\frac{2}{3}$ G instead of the floor dead load G.

- **Calculating bending moments**

✓ **<u>Support moments</u>**

$$M_i = -\frac{q_w\,l_w'^3 + q_e\,l_e'^3}{8.5\,(l'_w + l'_e)}$$

Where:

- l'_w *and* l'_e are the length of imaginary spans on the left and on the right of the support

$l' = l$ If it is an end span

$l' = 0.8\,l$ If it is an intermediate span

- **<u>Span moment</u>**

The span moment is given by the following expression (method of sections):

$$M(x) = M_o(x) + M_w\left(1 - \frac{x}{l}\right) + M_e\frac{x}{l}$$

The maximum bending moment in the span is obtained for $x = x_0$

- $\boldsymbol{x_o = \frac{l}{2} - \frac{M_w - M_e}{q.l}}$

- **Support Shear forces**

The support shear forces, determined using the method of sections, can be determined by the following formula:

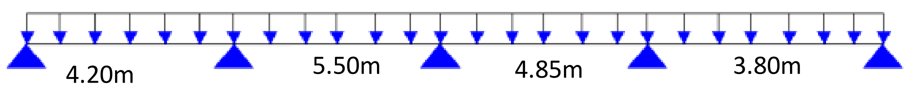
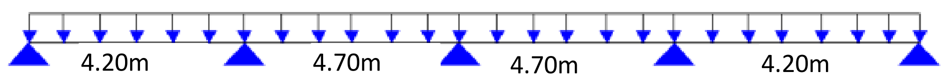
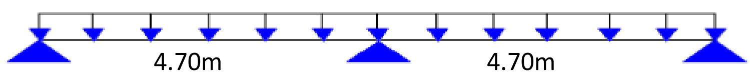
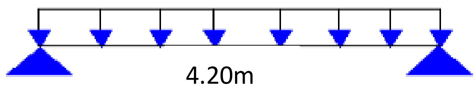
$$V = \{\frac{q.l}{2} - \frac{M_w - M_e}{L} \quad On\ the\ left\ support\ of\ a\ span$$

$$V = -\frac{q.l}{2} - \frac{M_w - M_e}{L} \quad On\ the\ right\ support\ of\ a\ span$$

III.2.1.2: Types of joists

There are several types of joists in our structure, which we will illustrate in the table III.1.

TABLE III 1: DIFFERENT TYPES OF JOISTS IN THE STUDIED BUILDING

Type	Static diagrams of joists
1	
2	
3	
4	

Note: The inaccessible terrace has type 1 only, and the other types are found in all the other levels. Type 1 has a hollow-core slab of (20+4) and the other types have a hollow core slab of (16+4).

III.2.1.3: Operating loads and overloads on the joists

The load on the joist q is calculated based on the floor slab load p and the spacing of the joists l_o :
$//q = p * l_o$

The calculations for the Ultimate Limit State (ULS) and Serviceability Limit State (SLS) are summarized in Table III.2.

Table III.2: Floors and joists Loads.

TABLE III 2: FLOORS AND JOISTS LOADS.

Designation	G (kN/m²)	Q (kN/m²)	ULS (kN/m)		SLS (kN/m)	
			P_U	q_U	P_S	q_s
Inaccessible Terrace	7.14	1.00	11.14	6.68	8.14	4.88
Accessible Terrace	6.41	1.50	10.90	6.54	7.91	4.75
Main Floor	5.21	1.50	9.28	5.57	6.71	4.03
Commercial	5.21	5.00	10.90	8.72	10.21	6.13

With Lo= 60cm

- **Illustrative examples**

Some calculation examples are presented below. The calculation results for the remaining joists will be summarized in tables.

<u>Type 1:</u> Inaccessible terrace.

For this type of joist, only the span ratio condition $\frac{Li}{Li-1}$ is not satisfied; hence, the reduced Caquot method is applied.

$$\frac{Li}{Li-1} = \frac{3.80}{4.85} = 0.78 < 0.8$$

$$G' = \frac{2}{3}\,G$$

$G' = \frac{2}{3} * 7.14 = 4.76\ kN/m^2$

$q'_u = (1.35\ G' + 1.5\ Q) * 0.6 = \ 4.76kN/m$

$q'_s = \ (G' + Q) * 0.6 = \ 3.46kN/m$

a) Support Moments

- ***Edge Support*** M_A=M_E= $-0.15\ M_0^1$

M_A=M_E = $\{ULS = -1.57kN.m \qquad SLS = -1.14kN.m$

- ***Intermediate Support***

By applying the moment formula provided by Caquot, the bending moment at support B is calculated as follows:

$$lw = l = 4.20\ m$$

$$le = 0.8 *\ l = 0.8 * 5.5 = 4.40\ m$$

ULS

$$M_B = -\frac{4.76(4.2^3 + 4.4^3)}{8.5 * (4.2 + 4.4)} = -10.37kN.m$$

SLS

$$M_B = -\frac{3.46(4.2^3 + 4.4^3)}{8.5 \times (4.2 + 4.4)} = -7.54\ kN.m$$

The calculation results of the bending moments at the various supports are summarized in Table III.3:

TABLE III 3: SUPPORT MOMENTS ON TYPE 1 AT THE INACCESSIBLE TERRACE.

<table>
<tr><th rowspan="2">Support</th><th rowspan="2">l'_w</th><th rowspan="2">l'_e</th><th colspan="3">ULS</th><th colspan="3">SLS</th></tr>
<tr><th>q_w</th><th>q_e</th><th>M(kN.m)</th><th>q_w</th><th>q_e</th><th>M(kN.m)</th></tr>
<tr><td>B</td><td>4.20</td><td>4.40</td><td colspan="2" rowspan="3">4.76</td><td>-10.37</td><td colspan="2" rowspan="3">3.46</td><td>-7.54</td></tr>
<tr><td>C</td><td>4.40</td><td>3.88</td><td>-9.71</td><td>-7.06</td></tr>
<tr><td>D</td><td>3.88</td><td>3.80</td><td>-8.26</td><td>-6</td></tr>
</table>

b) Span moments

$$\boldsymbol{qu = 6.68kN/m \qquad and \qquad qu = 4.88kN/m}$$

-SPAN AB

$$x_o = \frac{4.2}{2} - \frac{0 - (-10.37)}{6.68 * 4.2} = 1.73\ m$$

The span bending moments:

ULS

$$M_t^{max} = \frac{6.68 * 1.73}{2}\ (4.2 - 1.73) - 10.37 * \frac{1.73}{4.2} = 10\ kN.m$$

SLS

$$M_t^{max} = \frac{4.88 * 1.73}{2}\ (4.2 - 1.73) - 7.54 * \frac{1.73}{4.2} = 7.32\ kN.m$$

The calculation results of the bending moments at the various spans are summarized in Table III.4.

TABLE III 4: SPAN MOMENTS ON TYPE 1 AT THE INACCESSIBLE TERRACE.

Span	$M_0(x)$(kN.m)		$xo(m)$	M_{tu}(kN.m)	M_{ts}(kN.m)
	ULS	SLS			
AB	14.27	10.43	1.73	10	7.32
BC	25.26	18.45	2.77	15.22	11.15
CD	19.68	14.34	2.47	10.38	7.82
DE	11.69	8.54	2.23	8.28	6.06

c) Shear forces

$$V_A = \frac{6.68 * 4.2}{2} - \frac{0 - (-10.37)}{4.2} = 11.56\ kN$$

$$V_B = -\frac{6.68 * 4.2}{2} - \frac{0 - (-10.37)}{4.2} = -16.5\ kN$$

$V_i = V_o - \frac{M_l - M_r}{l}$;with $V_o = \pm \frac{ql}{2}$

The calculation of the various spans is performed in a similar manner. The results obtained are presented in Table III.5

TABLE III 5: SHEAR FORCE RESULTS OF TYPE 1 ON THE INACCESSIBLE TERRACE.

Span	L (m)	$q_U\ (kN/m)$	$M_w\ (kN.m)$	$M_w\ (kN.m)$	Shear forces (kN)
A-B	4.2	6.68	0	10.37	$V_A = 11.56$ $V_B = -16.50$
B-C	5.5		10.37	9.71	$V_B = 18.49$ $V_C = -18.25$
C-D	4.85		9.71	8.26	$V_C = 16.5$ $V_D = -15.9$
D-E	3.8		8.26	0	$V_D = 14.87$ $V_E = -10.51$

Type 2: Main floors

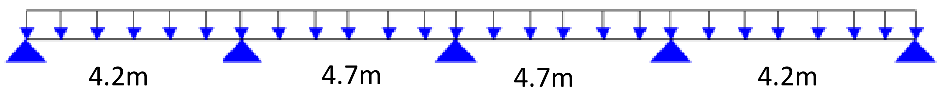


FIGURE III- 1: TYPE 2 JOIST ON MAIN FLOORS

For this type, all the conditions are verified hence we will apply the comprehensive method.

- **Calculation of reference Moments Mo**

$$M_o = \frac{q * l^2}{8}$$

TABLE III 6: REFERENCE MOMENTS CALCULATION OF TYPE 2 IN MAIN FLOORS

	ULS	SLS
AB or DE	12.28	8.88
BC or CD	15.38	11.12

a) Support Moments

- **Edge Support**

At ULS; $M_A = M_E = -0.15 \frac{q_u * l^2}{8} = -0.15 \frac{5.57*4.2^2}{8} = -1..84kN.m$

At SLS; $M_A = M_E = -0.15 \frac{q_u * l^2}{8} = -0.15 \frac{4.03*4.2^2}{8} = -1.33kN.m$

- **Intermediate Support**

$$M_B = M_D = -0.5\ M_o^{BC}$$

ULS $\boldsymbol{M_B} = \boldsymbol{M_D} = -0.5 * 15.38 = -7.69\ kN.m$

SLS $\boldsymbol{M_B} = \boldsymbol{M_D} = -0.5\ * 11.12 = -5.56\ kN.m$

$$M_C = -0.4\ M_o$$

ULS $M_C = -0.4 * 15.38 = -6.15\ kN.m$

SLS $\boldsymbol{M_C} = -0.4 * 11.12 = -4.45\ kN.m$

b) Span Moments

$$\alpha = \frac{Q}{G+Q} = \frac{1.5}{5.21+1.5} = 0.224$$

SPAN AB

1. $M_{AB}+\frac{|M_A+M_B|}{2}\geq(1+0.3\propto;1.05)M_o^{AB}$

$$M_{AB}\geq 1.06\,M_o^{AB}-\frac{0.5M_o^{BC}}{2}$$

$$M_{AB}\geq 1.06\,M_o^{AB}-0.25\,M_o^{BC}$$

$$M_{AB}=\{9.17\,kN.m(\,ULS)\qquad and \qquad 6.64\,kN.m(SLS)$$

2. $M_{AB}\geq\left(\frac{1.2+0.3\propto}{2}\right)M_o^{AB}=O.634\,M_o^{AB}$

$$M_{AB}=\{7.78\,kN.m\,(\,ULS)\qquad 5.63\,kN.m(SLS)$$

Therefore: $M_{AB}\;=max(M_{AB}^1;M_{AB}^2\,)=\{9.17\,kN.m(\,ULS)\qquad 6.64\,kN.m\,(SLS)$

SPAN BC

1. $M_{BC}+\frac{|M_B+M_C|}{2}\geq(1+0.3\propto;1.05)M_o^{BC}$

$$M_{AB}\geq 1.06\,M_o^{BC}-0.46\,M_o^{BC}$$

$$M_{AB}\geq 0.61M_o^{BC}$$

$$M_{BC}=\{9.38kN.m(\,ULS)\qquad and \qquad 6.79\,kN.m(SLS)$$

2. $M_{BC}\geq\left(\frac{1+0.3\propto}{2}\right)M_o^{BC}=O.534\,M_o^{BC}$

$$M_{AB}=\{8.21\,kN.m\,(\,ULS)\qquad 5.94\,kN.m(SLS)$$

Therefore: $M_{BC}\;=ma\,x(M_{BC}^1;M_{BC}^2\,)=\{9.38kN.m(\,ULS)\qquad 6.79\,kN.m\,(SLS)$

c) Shear forces

$$Span\ AB\ and\ DE\ \{V_A=\frac{5.57*4.2}{2}=11.70kN\qquad V_B=-1.1*\frac{5.57*4.2}{2}=-12.87kN$$

$$Span\ BC\ and\ CD\ \{V_B=1.1*\frac{5.57*4.7}{2}=14.22kN\ V_C=-\frac{5.57*4.7}{2}=-13.09kN$$

The stresses of Type 2 of commercial and accessible terrace floors are shown in the table III.7.

TABLE III 7: BENDING MOMENT AND SHEAR FORCES IN THE TYPE 2 JOISTS OF COMMERCIAL AND ACCESSIBLE TERRACE FLOORS

Stresses	Segment	Commercial floors		Accessible terrace	
		ULS (kN.m)	SLS (kN.m)	ULS (kN.m)	SLS (kN.m)
Support moment	M_A	-2.88	-2.03	-2.16	-1.57
	$M_B\ and\ M_D$	-12.04	-8.46	-9.03	-6.56
	M_C	-9.63	-6.77	-7.22	-5.25

Span moment	$M_{AB} = M_{DE}$	14.36	10.10	10.77	7.87
	$M_{BC} = M_{CD}$	14.69	10.33	11.02	8.00
Shear forces	*Span AB and DE*	$V_A = 18.31$ $V_B = -20.14$		$V_A = 18.31$ $V_B = -20.14$	
	Span BC and CD	$V_B = 22.54$ $V_C = -20.49$		$V_B = 16.91$ $V_C = -15.91$	

The calculation results for the other types of joists are summarized in Table III.8

TABLE III 8: SOLICITATIONS OF THE OTHER JOISTS TYPES.

Type	Level	Edge moments		Intermediate moments		Span moments		Shear forces
		ULS	SLS	ULS	SLS	ULS	SLS	
3	Main floor	M_A= M_C=-2.3	M_A= M_C=-1.67	M_B=-9.23	M_B=-6.68	$M_{AB} = M_{BC}$ 11.69	$M_{AB} = M_{BC}$ 8.46	V_{max} = -15.05
	Commercial floors	M_A= M_C=-3.61	M_A= M_C=-2.54	M_B=-14.45	M_B=10.16	$M_{AB} = M_{BC}$ 18.3	$M_{AB} = M_{BC}$ 12.86	V_{max} = -23.57
	Accessible terrace	M_A= M_C=-2.71	M_A= M_C=-1.97	M_B=-10.84	M_B=-7.87	$M_{AB} = M_{BC}$ 13.72	$M_{AB} = M_{BC}$ 9.97	V_{max} = -17.76
4	Main floors	M_A= M_B=-1.84	M_A= M_B=-1.33	/	/	M_{AB} = 12.28	M_{AB} = 8.89	V_{max} = -11.7
	Commercial floors	M_A= M_B=-2.88	M_A= M_B=-2.03	/	/	M_{AB} = 19.23	$M_{AB} = M_{BC}$ 13.52	V_{max} = -18.31
	Accessible terrace	M_A= M_B=-2.16	M_A= M_B=-1.57	/	/	M_{AB} = 14.42	M_{AB} = 10.47	V_{max} = -13.73

III.2.1.3: Reinforcement of concrete joists

The joists are reinforced for simple bending as T-sections. An example of reinforcement detailing is provided for the **commercial floor** joists. The reinforcement design outcomes for the other floors will be summarized in Table III.9.

a) Longitudinal Span_reinforcement

$b = 60cm;\ b_0 = 10cm; h = 20cm;\ h_0 = 4cm; d = 18cm;$

$fe = 400MPa;\ f_{c28} = 25MPa; f_{bu} = 14.2MPa$

The Moment balanced by the compression flange:

$M_{tu} = f_{bu} * b * h_0 * \left(d - \frac{ho}{2}\right) = 14.2 * 0.6 * 0.04 * \left(0.18 - \frac{0.04}{2}\right) = 0.054528MN.m$

$M_{tu} => \ M_t = 0.01923$The T-section is designed as a rectangular section of width b and height h

$$\mu_{bu} = \frac{M_t}{b * d^2 * f_{bu}} = \frac{0.01923}{0.6 * 0.18^2 * 14.2} = 0.07$$

$$\mu_{bu} = 0.07 < 0.186 \ \Rightarrow \ pivot\ A \rightarrow \ F_S = \frac{f_e}{\gamma_s} = \frac{400}{1.15} = 348MPa$$

$$\mu_{bu} = 0.07 < \mu_l = 0.3916 \Rightarrow A' = 0$$

$$A = \frac{M_u}{Z \times f_{st}}$$

$$\propto = 1.25\left(1 - \sqrt{1 - 2\mu_{bu}}\right) = 0.09$$

$$Z = d(1 - 0.4 \propto) = 0.1734m$$

$$A = \frac{0.01923 * 10^4}{0.1734 * 348} = 3.19cm^2$$

Verification of the non-brittleness condition:

$A_{smin} = 0.23b * d * \frac{f_{t28}}{f_e};$

With $f_{t28} = 0.6 + 0.06f_{c28} = 2.1MPa$

$A_{smin} = 0.23 * 60 * 18 * \frac{2.1}{400} = 1.30cm^2;$

$$A_{smin} = 1.30cm^2 < A_{cal} = 3.19cm^2$$

$We\ take\ 3HA12 = 3.39cm^2$

❖ ***<u>Supports reinforcement</u>***

The moment at the support is negative, which means the flange is in tension and therefore does not contribute to the resistance. The calculation simplifies to reinforcing a rectangular section $b_o * h$.

- **Intermediate support**

$$\mu_{bu} = \frac{M_a}{b_0 * d^2 * \ f_{bu}} = \frac{0.01445}{0.1 * 0.18^2 * 14.2} = 0.314$$

$$\mu_{bu} = 0.314 > 0.186 \ \Rightarrow \ pivot\ B$$

$$\mu_{bu} = 0.314 < \mu_l = 0.3916 \Rightarrow A' = 0$$

$$\propto = 1.25\left(1 - \sqrt{1 - 2\mu_{bu}}\right) = 0.488$$

$$Z = d(1 - 0.4 \propto) = 0.1449m$$

$$\varepsilon_{st} = \frac{3.5}{1000}\left(\frac{1 - \alpha}{\alpha}\right) = 3.67 * 10^{-3}$$

$$\varepsilon_{st} = 3.67 * 10^{-3} > \varepsilon_l = 1.74 * 10^{-3} \Rightarrow f_{st} = \frac{f_e}{\gamma_s} = \frac{400}{1.15} = 348Mpa$$

$$A_a^{int} = \frac{M_a}{Z * f_{st}} = \frac{0.01445 * 10^4}{0.1449 * 348} = 2.87cm^2$$

- **Edge support**

$$\mu_{bu} = \frac{M_{ta}}{b * d^2 * f_{bu}} = \frac{0.00361}{0.1 * 0.18^2 * 14.2} = 0.078$$

$$\mu_{bu} = 0.078 < 0.186 \Rightarrow pivot\ A$$

$$\mu_{bu} = 0.078 < \mu_l = 0.3916 \Rightarrow A' = 0$$

$$A = \frac{M_a}{Z * f_{st}}$$

$$\propto = 1.25\left(1 - \sqrt{1 - 2\mu_{bu}}\right) = 0.102$$

$$Z = d(1 - 0.4 \propto) = 0.1730m$$

$$A = \frac{0.00361 * 10^4}{0.1730 * 348} = 0.60cm^2$$

$$A_{smin} = 0.23 * 10 * 18 * \frac{2.1}{400} = 0.217cm^2;$$

For Edge support: $A = 0.60cm^2; we\ take\ 1HA14 = 1.54cm^2$

For Intermediate support: $A = 2.87cm^2; we\ take\ 2HA14 = 3.08cm^2$

b) Transverse reinforcement

- **Shear stress verification**

$$\tau_u = \frac{V_u}{b_0 * d} = \frac{0.02357}{0.1 * 0.18} = 1.31MPa$$

$$\overline{\tau_u} = \left(0.2\frac{f_{c28}}{\gamma_s}; 5MPa\right) = 3.33MPa\ ; Case\ of\ Low\ harm\ cracking$$

$\tau_u = 1.31MPa < \overline{\tau_u} \Rightarrow$ No risk of shear failure.

- **Cross-sectional area of transverse reinforcement**

$$\emptyset_t \geq \left(\emptyset_l^{min}; \frac{h}{35}; \frac{b_0}{10}\right)$$

$$\emptyset_t \geq \left(12mm; \frac{200}{35}; \frac{100}{10}\right) = (12;\ 5.71; 10) = 5.71mm$$

$$\emptyset_t = 6mm\ with\ A_t = 2HA8 = 0.57cm^2$$

The spacing of the transverse reinforcement is determined based on the following three conditions:

1. $S_t \leq (0.9d; 40)\ cm = min = (0.9 * 18; 40)cm \Rightarrow S_t \leq 16.cm$
2. $S_t \leq \frac{A_t \times}{0.4 \times b_0} = \frac{0.57 \times 400}{0.4 \times 10} = 57cm$
3. $S_t \leq \frac{A_t \times f_e \times 0.8 \times (sin \propto + cos \propto)}{b_0 \times (\tau_u - 0.3 \times f_{t28})} \Rightarrow S_t \leq 47.53cm$

Therefore, we opt for spacing of $S_t = 15cm$

c) Limit state verifications

❖ **ULS verifications**

- **Verification of longitudinal reinforcement for shear resistance**

-Edge support:

$A_l^{min} \geq \frac{\gamma_s}{F_E} * V_U$; With $A_l^{min} = 3.39 + 1.54 = 4.93cm^2$

$A_l^{min} = 4.93cm^2 > \frac{1,15}{400} * 0.02357 = 0.68cm^2$ ⇒ condition verified

-Intermediate support

$A_l^{min} \geq \frac{\gamma_s}{f_e} * \left(V_U + \frac{M_a^{min}}{0.9*d}\right)$ = $\frac{1.15}{400} * (0.02357 + \frac{(-0.01445)}{0.9*0.18} = -1.88$

-1.88<0; No need of verifying.

- **Strut verification**

$V_U \leq 0.267 b_0 * a * f_{C28}$

$a = 0.9 * d = 0.9 * 18 = 16.2cm$

$V_U = 23.57kN < 0.267 * 0.1 * 0.162 * 25 * 10^3 = 108.14kN.$

The condition is verified hence there no risk of crashing of the strut.

- **Shear verification at the flange-web junction**

$\tau_u = \frac{V_u \times b_1}{0.9 \times b \times h_0 \times d} \leq \underline{\tau_u}$ $\underline{\tau_u} = \left(0.2\frac{f_{c28}}{\gamma_s}; 5MPa\right) = 3.33MPa\,; LHC\ case$.

$With \quad b_1 = \frac{b-b_0}{2} = \frac{60-10}{2} = 25cm$

$\tau_u = \frac{0.02357 \times 0.25}{0.9 \times 0.18 \times 0.60 \times 0.04} = 1.52MPa < \underline{\tau_u} = 3.33MPa$

No risk of shear failure at flange-web junction.

❖ **Serviceability Limit State verifications.**

- **Crack width limit state**

-Span verification

$$H = \frac{b * h_0^2}{2} - 15A(d - h_0) = \frac{60 * 4^2}{2} - 15 * 3.39(18 - 4)$$

$H = -231.9cm < 0 \Rightarrow contraint\ verification\ for\ a\ T - Section$

$\sigma_{bc} = \frac{M_{ser}}{I} * y \leq \sigma_{bc} = 0.6 * f_{c28}$

Calculation of the neutral axis position and the moment of inertia $\frac{b_0}{2}y^2 + [15A(b - b_0)h_0]y - 15(Ad - A'd') - \frac{b-b_0}{2}h_0 = 0$

$5y^2 + 250.85y - 1315.3 = 0$

$y = 4.79cm$

$I = \frac{b}{3}y^3 - (b - b_0)\frac{(y-h_0)^3}{3} + 15A(d-y)^2 + 15A'(d'-y)^2$

$I = \frac{60}{3}4.79^3 - (60-10)\frac{(4.79-4)^3}{3} + 15 \times 3.39(18-4.79)^2 = 11063.36cm^4$

$\sigma_{bc} = \frac{0,01352}{11063.36\times10^{-8}} \times 0.0479 = 5.85MPa < \sigma_{bc} = 0.6 \times 25 = 15MPa$; verified.

-Intermediate support

$\frac{b_o}{2}y^2 + 15(A + A')y - 15(Ad - A'd') = 0 \Rightarrow 5y^2 + 250.85y - 1315.3 = 0$

$$y = 9.079\ cm$$

$I = \frac{b_o}{3}y^3 + 15A'(y-d')^2 + 15A(d-y)^2;$

I= $\frac{10}{3} \times 9.079^3 + 15 \times 3.08(18 - 9.079)^2 = 6171.345cm^4$

$\sigma_{bc} = \frac{0,01016}{6171.345\times10^{-8}} \times 0.09079 = 14.95Mpa < \sigma_{bc} = 0.6 \times 25 = 15Mpa$ Condition verified

- **Deflection verification**

Conditions of deflection verification

Data

$l = 4.20m$: length of the most stressed span

$M_t^s = M_0^s = 13.52kN.m$

Deflection verification is not necessary if the following conditions are observed:

$\frac{h}{l} \geq \frac{1}{16}$ $\quad 0.0476 < 0,0625 \quad$ $Condition\ not\ verified$

$\frac{h}{l} \geq \frac{M_t^s}{10M_0}$ $\quad 0.0476 \leq 0.1 \quad$ $Condition\ not\ verified$

$\frac{A_t}{b_0 d} \geq \frac{l}{fe}$ $\quad 0.018 > 0.0105 \quad$ $Condition\ verified$

The first two conditions are not verified; therefore, the deflection verification is necessary.

$\Delta f \leq f \quad = \frac{L}{500} \quad since\ l = 4.20m < 5m$

Δf: Deflection to be calculated according to BAEL, considering the properties of reinforced concrete (shrinkage, cracking).

$\Delta f = (f_{gv} - f_{ji}) + (f_{pi} - f_{gi}) \qquad BAEL99$

$G = 5.21kN/m^2 \qquad M_s^g = 0.6 * 5.21 * \frac{4.2^2}{8} = 6.89kN.m$

$J = 3.85kN/m^2 \qquad M_s^j = 0.6 * 3.85 * \frac{4.2^2}{8} = 5.09kN.m$

$P = G + Q = 10.21kN/m^2 \qquad M_s^g = 0.6 * 10.21 * \frac{4.2^2}{8} = 13.51kN.m$

Instantaneous young's modulus

$Ev = 3700(\sqrt[3]{f_{c28}}) = 10818.86\text{MPa}$

$Ei = 3 * Ev = 32456.6MPa$

Geometric characteristic of the section

$$Y_G = \frac{\frac{b_0*h^2}{2}+\frac{(b-b_0)*h_0^2}{2}+n(Ad+A'*d')}{(b_0*h)+(b-b_0)*h_0+n(A+A')} = \frac{\frac{10*20^2}{2}+\frac{(60-10)*4^2}{2}+15(3.39*18)}{(10*20)+(60-10)*4+15(3.39)}$$

$Y_G = 7.35cm$

$$I_0 = \frac{b*Y_G^3}{3} + b_0 * \frac{(h-Y_G)^2}{3} - \frac{(b-b_0)*(Y_G-h_0)^3}{3} + 15 * A * (d - Y_G)^2$$

$$I_0 = \frac{60*7.35^3}{3} + 10 * \frac{(20-7.35)^2}{3} - \frac{(60-10)*(7.35-4)^3}{3} + 15 * 3.39 * (18 - 7.35)^2$$

$I_0 = 19829.87cm^2$

Coefficient λ, μ

They are defined by BAEL to take into account concrete cracking.

$$\rho = \frac{A_t}{b_0*d} = \frac{3.39}{10*18} = 0.0188$$

$$\lambda_i = \frac{0.05*b*f_{t28}}{(2*b+3*b_0)*\rho} = \frac{0.05*60*2.1}{(2*0.6+3*0.1)*0.0188} = 2.234$$

$\lambda_v = 0.4 * \lambda_i = 0.4 * 2.234 = 0.894$

Calculating σ_{st}(steel tensile stresses)

$$\sigma_{st}^G = 15 * \frac{M_s^G}{I} * (d - y) = 15 * \frac{0.00689}{11063.36*10^{-8}} * (0.18 - 0.0479) = 123.40MPa$$

$\sigma_{st}^j = 91.16MPa$

$\sigma_{st}^p = 241.97MPa$

Calculating μ

$$\mu_g = 1 - \frac{1.75*f_{t28}}{4*\rho*\sigma_{st}^G+f_{t28}} = 1 - \frac{1.75*2.1}{4*0.0188*123.4+2.1} = 0.677$$

$\mu_j = 0.59$

$\mu_p = 0.819$

Calculating the fictitious moments of inertia I_f

$$I_{fgi} = \frac{1.1*I_0}{1+\lambda_i*\mu_g} = \frac{1.1*19829.87*10^{-8}}{1+2.234*0.677} = 8.682 * 10^{-5}m^4$$

$$I_{fgv} = \frac{1.1*I_0}{1+\lambda_v*\mu_g} = \frac{1.1*19829.87*10^{-8}}{1+0.894*0.677} = 1.3589 * 10^{-4}m^4$$

$$I_{fpi} = \frac{1.1*I_0}{1+\lambda_i*\mu_p} = \frac{1.1*19829.87*10^{-8}}{1+2.234*0.819} = 7.7087 * 10^{-5}m^4$$

$$I_{fji} = \frac{1.1*I_0}{1+\lambda_i*\mu_j} = \frac{1.1*19829.87*10^{-8}}{1+2.234*0.59} = 9.410 * 10^{-5}m^4$$

Deflection calculation

$$f_{gv} = M_s^g * \frac{l^2}{10*Ev*I_{fgv}} = \frac{0.00689*4.2^2*10^3}{10*10818.89*1.3589*10^{-4}} = 8.27mm$$

$$f_{gi} = M_s^g * \frac{l^2}{10*Ei*I_{fgi}} = \frac{0.00689*4.2^2*10^3}{10*32456.58*8.682*10^{-5}} = 4.31mm$$

$$f_{ji} = M_s^j * \frac{l^2}{10*Ei*I_{fji}} = \frac{0.00689*4.2^2*10^3}{10*32456.58*9.410*10^{-5}} = 3.59mm$$

$$f_{pi} = M_s^p * \frac{l^2}{10*Ei*I_{fpi}} = \frac{0.00689*4.2^2*10^3}{10*32456.58*7.7087*10^{-5}} = 7.80mm$$

$$\Delta f = (8.27 - 3.59) + (7.80 - 4.31) = 8.17mm$$

$$\Delta f = 8.17mm < f = 8.4mm \qquad condition\ verified$$

d) Calculation of stresses at the different levels

❖ Calculation of reinforcement at ULS

The reinforcement is carried out in the same manner as the previous calculation on the commercial floor. The results are shown in the following table,

TABLE III 9: CALCULATIONS OF REINFORCEMENT AT ULS AT THE DIFFERENT LEVELS.

Levels	Position	M(kN.m)	μ_{bu}	$\propto$	Z	$A_{cal}(cm^2)$	$A_{min}(cm^2)$	A_{adopt}
Main floor	Span	12.28	0.044	0.056	0.176	2.00	1.304	3HA10=2.36
	Intermediate support	-9.23	0.20	0.282	0.16	1.66	0.217	2HA12=2.26
	Edge support	-2.30	0.5	0.064	0.175	0.38	0.217	1HA12=1.13
Accessible terrace	Span	14.42	0.052	0.066	0.175	2.37	1.304	3HA12=3.39
	Intermediate support	-10.84	0.236	0.342	0.155	2.01	0.217	2HA12=2.26
	Edge support	-2.71	0.059	0.076	0.175	0.44	0.217	1HA12=1.13
Inaccessible terrace	Span	15.22	0.037	0.047	0.216	2.02	1.59	3HA10=2.36
	Intermediate support	-10.37	0.151	0.206	0,202	1.48	0.27	2HA10=1.57
	Edge support	-1.57	0.023	0.028	0.217	0.21	0.27	1HA10=0.79

❖ Limit state verifications

-ULS Verifications

The joists verifications at limit states illustrated in the table below,

TABLE III 10: VERIFICATIONS REQUIRED FOR ULS AT DIFFERENT LEVELS

Levels	Shear $\tau_u \leq \underline{\tau}$ (MPa)	Longitudinal bars $A_l(cm^2) \geq$		Strut kN $V_u \leq$ $0.267 b_0 a f_{c28}$	Rib slab junction $\tau_u \leq \underline{\tau}$ $en\ MPa$
		$\frac{\gamma_s}{f_e} V_u$	$\frac{\gamma_s}{F_E}(V_u + \frac{M_s^{int}}{0.9d}$		
Main floor	0139<3.33	3.15>0.43	4.62>-1.21	*15.05<108.14*	0.96<3.33
Accessible terrace	0.186<3.33	4.18>0.58	5.65>-1.34	20.14<108.14	1.30<3.33
Inaccessible terrace	0.839<3.33	3.15>0.53	3.93>-0.56	18.49<132.17	0.97<3.33
observation	verified	verified	verified	verified	verified

-Verifications of SLS

The concerned verifications for SLS are;

- ✓ The constraints verification (cracks opening)
- ✓ Deformation verification
  - **Constraints verification**

The results obtained are resumed in the following table:

TABLE III 11: CONSTRAINTS VERIFICATION AT SLS

levels	position	$M_s(kN.m)$	Y(cm)	I (cm^4)	Contraints (MPa) $\sigma_{bc} \leq \underline{\sigma_{bc}}$	OBS
Main floors	Span	8.89	4.06	8217.52	4.39<15	verified
	Support	-.6.68	8.116	5093.513	10.71<15	verified
Accessible terrace	Span	10.47	4.79	11063.36	5.85<15	verified
	Support	-7.87	8.116	5093.513	12.54<15	verified
Inaccessible terrace	Span	11.15	4.57	12660.49	4.18<15	verified
	Support	-7.54	7.149	3990.784	13.51<15	verified

- **Deformation verification**

The calculation results are shown in the table below,

TABLE III 12: DEFLECTION VERIFICATION AT SLS OF THE DIFFERENT LEVELS.

Levels	L(m)	$f_{gv}(mm)$	$f_{ji}(mm)$	$f_{pi}(mm)$	$f_{gi}(mm)$	$\Delta f(mm)$	$\underline{f}(mm)$	observation
Main floor	4.20	10.57	4.02	9.83	5.97	10.41	8.40	Not verified
Accessible terrace	4.20	10.46	1.93	7.11	5.55	10.09	8.40	Not verified
Inaccessible terrace	5.50	15.67	7.16	10.42	8.85	10.08	10.50	verified

Note

Deformation is verified in the inaccessible terrace floor, however for the main floors and the accessible terrace, deformation is not verified.

For this, we have increased the section to 3HA12=3.39cm^2 in the main floors, and to 3HA14=4.62cm^2 in the accessible terrace, from which the deformation is shown in the following table,

TABLE III 13: CORRECTED DEFLECTION VERIFICATION AT SLS OF THE MAIN FLOOR AND ACCESSIBLE TERRACE.

levels	Section (cm^2)	$f_{gv}(mm)$	$f_{ji}(mm)$	$f_{pi}(mm)$	$f_{gi}(mm)$	$\Delta f(mm)$	$\underline{f}(mm)$	observation
MF	3HA12=3.39	8.27	3.59	7.01	4.31	7.38	8.40	verified
T. A	3HA14=4.62	8.51	1.54	5.36	4.21	8.12	8.40	verified

❖ **Compression slab reinforcement**

-Reinforcement bars perpendicular to the direction of joists.

$$50cm \leq l_o \leq 80cm$$

$A_{\perp} = \frac{4l_0}{F_e} \Rightarrow \frac{4\times 60}{235} = 1.02cm^2/m$

$St \leq 20cm$; We take $5HA6 = 1.41cm^2/m\ d$

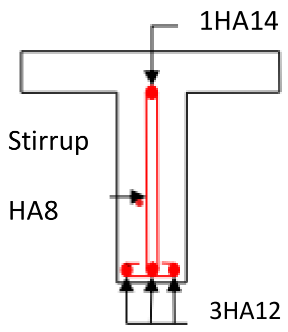
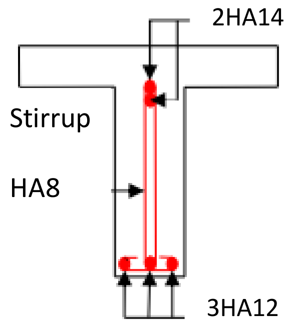
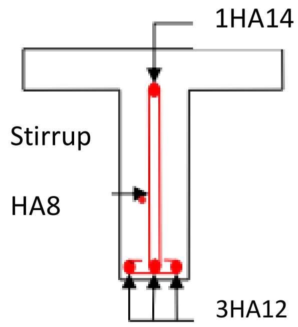
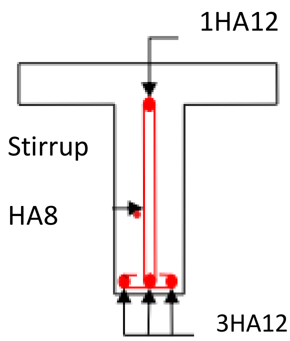
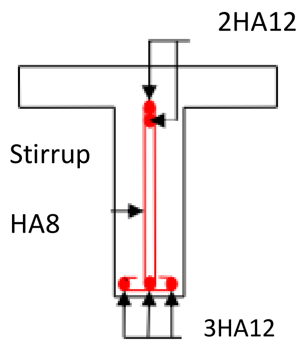
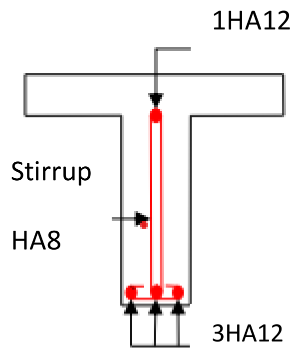
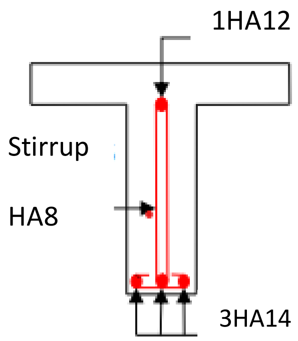
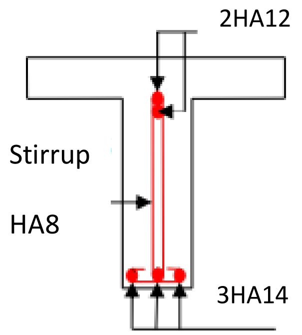
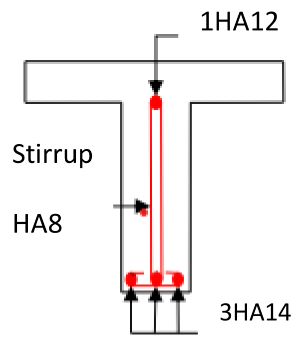
-Reinforcement parallel bars

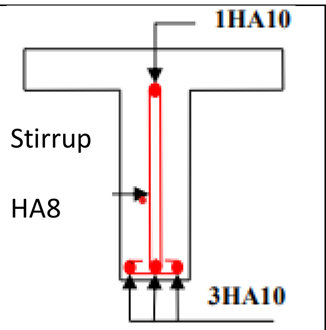
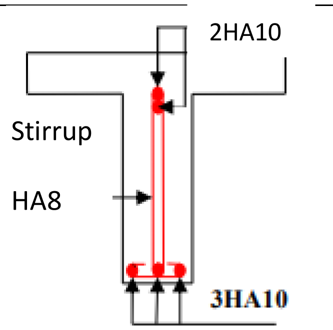
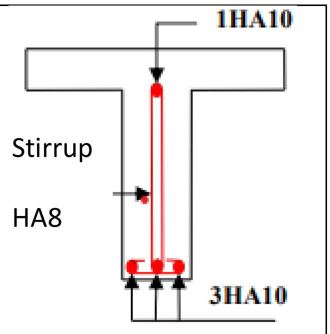
$A_{//} = \frac{A_{\perp}}{2} = \frac{1.41}{2} = 0.705cm^2/m$

$St \leq 33cm$; We take $4HA6 = 1.13cm^2/m$

The different reinforcement diagrams are shown below:

TABLE III 14: REINFORCEMENT DIAGRAMS OF THE JOISTS.

Floors	Span	Edge support	Intermediate support
Commercial			
Main floors			
Terrasse accessible			
Terrasse inaccessible			

III.2.2 Solid slabs

The studied building features several solid slabs. In this work, we present the analysis of **Slab 9**, **Slab 7** and **Slab 5**:

III.2.2.1 Slab with 4 supports (Slab 9)

Calculation example (Commercial floor)

$\rho = \frac{L_x}{L_y} = \frac{455}{910} = 0.5 > 0.4 \Rightarrow the\ slab\ works\ in\ both\ directions.$

Slab 9 is around the staircase; it is designed without accounting for the opening. The cross-sectional area of the cut-off reinforcement at the opening will be replaced by reinforcement bars with an equivalent cross-sectional area to the removed bars.

❖ **Bending moments calculation**

Rectangular slabs which are integrally attached may be analyzed with regard to flexion on the basis of the forces which would be developed if they were freely joined at their perimeter.

The maximum bending moments calculated on the assumption of free joints are reduced by ***15*** to ***25%*** depending upon the nature of the attachment. The attachment moments on the longer sides are to be evaluated at least to ***40%*** and ***50%*** of the maximum bending moments evaluated on the assumption of free joints. It is necessary however to take into account the fact that the attachment moments on the shorter sides reach magnitudes of the same order as those on the longer sides.

The span moments are calculated as:

$M_{sx,y} = 0.85 M_{0x,y} \qquad for\ edge\ spans$

$M_{sx,y} = 0.75 M_{0x,y} \qquad for\ \ intermediate\ spans$

The supports moments:

$M_{ex} = M_{ey} = -0.3 M_{0x} \qquad for\ edge\ supports$

$M_{ex} = M_{ey} = -0.5 M_{0x} \qquad for\ intermediate\ \ supports$

$M_{0x}\ and\ M_{0y}$: The maximum bending moments calculated on the assumption of free joints

$M_{0x} = \mu_x * q_u * L_x^2 \ \ and\ \ M_{0y} = \mu_y * M_{0x}$

$\mu_x\ and\ \mu_y$ Coefficients given as a function of $\rho = \frac{L_x}{L_y}$ and Poisson's ratio

q_u : The uniformly distributed load per unit area of slab.

Shear forces calculation

$\tau_u = \frac{V}{bd} \leq 0.07 \frac{f_{c28}}{\gamma_b}$

Where:

$$V_x = \frac{q_u L_x}{2} * \frac{L_y^4}{L_x^4 + L_y^4}$$

$$V_y = \frac{q_u L_y}{2} * \frac{L_x^4}{L_x^4 + L_y^4}$$

For $G = 7.36\ kN/m^2 and\ Q = 5kN/m^2$

$q_u = 1.35G + 1.5Q = 17.436kN/m$

$q_s = G + Q = 12.36\ kN/m^2$

Using table of Annex 1:

$\boldsymbol{ULS}$: $\mu_x = 0.0966$; $\mu_y = 0.2500$

$M_{0x} = 0.0966 * 17.436 * 4.55^2 = 34.87kN.m$

$M_{0y} = 0.25 * 34.87 = 8.72kN.m$

SLS

$M_{0x} = 0.0966 * 12.36 * 4.55^2 = 24.56\ kN.m$

$M_{0y} = 0.25 * 34.87 = 8.72kN.m$

The span and supports moments as well as the shear forces are summarized in table III.15.

TABLE III 15: SPAN AND SUPPORTS MOMENTS IN THE SLAB 9 (COMMERCIAL FLOOR)

	Span moments (kN.m) $M_{tx} = 0.85M_0$		Supports moments (kN.m) $M_{ax} = -0.3M_{0x}$
	X direction	Y direction	
ULS	29.64	7.41	10.46
SLS	20.88		7.37
Shear forces	Vx=37.33kN	Vy = 4.67kN	

❖ **slab reinforcement**

The reinforcement calculation is performed using simple flexure for a 1m×0.2m slab section. The calculation results are presented in Table III.16,

TABLE III 16: SLAB REINFORCEMENT IN ULS.

Location		$M\ (kN.m)$	μ_{bu}	$\propto$	Z(m)	A_{cal} (cm^2/m)	A_{min} (cm^2/m)	$A_{opt}(cm^2/m)$	St(cm)
Span	X-X	29.64	0.064	0.083	0.174	4.89	2.00	5HA12=5.65	20
	Y-Y	7.41	0.016	0.020	0.178	1.19	1.60	4HA8=2.01	25
Support		-10.46	0.0227	0.029	0.178	1.69	2.00	4HA8=2.01	25

- **Limits states verifications**

a. **ULS Verifications**

- **Longitudinal bar spacing**

$X\ direction\text{: } S_t = 20cm\ \leq min(3e\ ;33cm)\ \ \Rightarrow condition\ verified$

$Y\ direction\text{: } S_t = 25cm\ \leq min(3e\ ;33cm)\ \ \Rightarrow condition\ verified$

- **Shear verification**

The results of the shear force verifications will be summarized in table III.17.

TABLE III 17: SHEAR VERIFICATION

Type	Direction	$V_u(kN)$	$\tau_u =\leq 0.07\frac{f_{c28}}{\gamma_b}$		OBS
			$\tau_u(MPa)$	$0.07\frac{f_{c28}}{\gamma_b}(MPa)$	
Slab 9	X-X	37.33	0.207	1.17	Verified
	Y-Y	4.67	0.026	1.17	//

No risk of shear failure. It is not necessary to use transverse steel.

b. **SLS Verifications**

- **Stress verification**

Slab 9 is located inside (Low Harm Cracking); therefore, we only verify the compressive strength in concrete.

$$\sigma_{bc} = \frac{M_{ser}}{I} * y\ \leq \bar{\sigma}_{bc}$$

The verifications are summarized in the following table:

TABLE III 18: STRESS VERIFICATIONS AT SLS

Location	Direction	M_{ser} (kN.m)	Y (cm)	I (cm^4)	$\sigma_{bc} \leq \bar{\sigma}_{bc}(MPa)$		Observation
					$\sigma_{bc}(MPa)$	$\bar{\sigma}_{bc}(MPa)$	
SPAN	X-X	20.88	4.71	18452.27	5.364	15	Verified
	Y-Y	5.22	3.01	7683.74	2.045	15	//
SUPPORT	X-X	-7.37	3.67	11319.63	2.389	15	//

No risk of crack openings.

- **Deflection verification**

According to BAEL, if the following conditions are verified, deflection verification is not required.

$$\frac{h}{L} \geq \frac{3}{80}$$

$$\frac{h}{L} \geq \frac{M_t^s}{20 \times M_0}$$

$$\frac{A_t}{b_0 \times d} \geq \frac{2}{f_e}$$

X-direction

$\frac{20}{455} = 0.044 > \frac{3}{80} = 0.0375 \Rightarrow \ verified$

$$0.044 > \frac{0.02175}{20 * 0.02559} = 0.021 \Rightarrow \ verified$$

$$\frac{5.65}{100 * 18} = 3.14 * 10^{-3} < \frac{2}{400} = 5 * 10^{-3} \rightarrow not\ verified$$

Y-direction

$\frac{20}{910} = 0.022 < \frac{3}{80} = 0.0375 \Rightarrow \ not\ verified$

The slab deflection verification is necessary for both directions. The calculations, carried out in the same manner as in the joists, have led to the results presented in the following table:

TABLE III 19: DEFLECTION VERIFICATION.

Direction	L(m)	$M_t^g(kN.m)$	$M_t^j(kN.m)$	$M_t^p(kN.m)$	t(cm)	f_{gv} (mm	f_{gi}(mm	f_{ji}(mm	f_{pi}(mm	Δf(mm	$\bar{f}$(mm	OBS
X-X	4.55	12.95	10.56	21.75	20	3.53	1.37	0.85	4.95	6.26	9.1	Verified
Y-Y	9.1	4.75	3.88	7.98		4.82	1.61	1.31	2.70	4.60	14.1	//

- **Calculation of the cut-off reinforcement cross-sectional area at the opening**

The slab with the opening is shown in figure III.2:

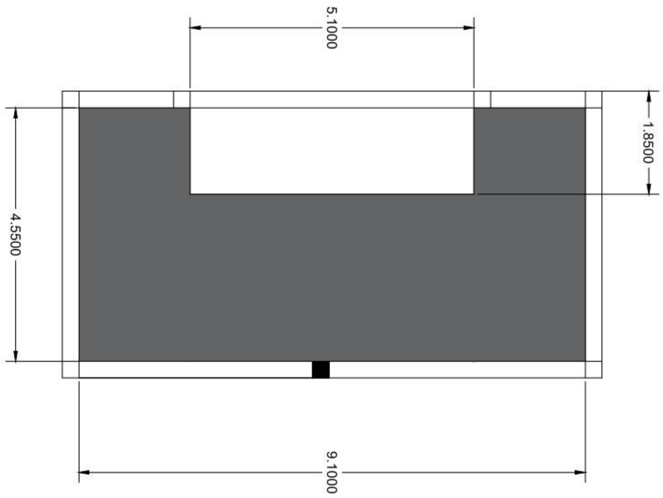


FIGURE III- 2: SOLID SLAB WITH THE OPENING

The opening dimensions: a = 1.85m; b=5.10m

The cross-sectional area of reinforcement calculated in each direction:

$A_{||x} = 5.65\text{cm}^2/\text{ml}$ and $A_{||y} = 2.01\text{cm}^2/\text{ml}$

5.65cm^2in 1m of slab

In 5.1 m, $y = \frac{5.65*5.10}{1} = 28.815\text{cm}^2$

2.01cm^2in 1m of slab

In 1.85 m, $x = \frac{2.01*1.85}{1} = 3.1785\text{cm}^2$ Bar diameter required for reinforcement bars in each direction $\emptyset'$ (*Référence: Maitrise du BAEL par Pierre Charon*)

$$\emptyset' = \begin{cases} \emptyset\sqrt{\frac{n}{2}} & Case\ of\ two\ reiforcement\ bars \\ \emptyset\sqrt{n} & case\ of\ one\ reiforcement\ bar \end{cases}$$

with n - number of barres removed while

Ø - diameter of bars removed

$$n = \frac{\text{length of the opening}}{\text{spacing in the considered direction}}$$

Y- direction : $n = \frac{185}{25} = 7.4\text{bars}$

X- direction : $n\frac{5.10}{0.20} = 25.5\text{bars}$

There for; $\emptyset'_x = 12\sqrt{\frac{25.5}{2}} = 42.85\text{mm}$

$\emptyset'_y = 8\sqrt{7.4} = 21.76\text{mm}$

The diameter of bars in the slab $\leq \frac{1}{10}$ of it's thickness ; hence $\emptyset' \leq \frac{1}{10} \times 200 = 20\text{mm}$

$\emptyset'_x > 20\ mm$ and $\emptyset'_y > 20\ mm$, reinforcement bars cannot be used; reinforcement beams are required.

- **Calculation of reinforced beams**

❖ ***Beam 1***

$\frac{L}{15} \leq h \leq \frac{L}{10} \Rightarrow \frac{510}{15} \leq h \leq \frac{510}{10} \Rightarrow h = 35\text{cm and } b = 30\text{cm}$

- **Loads on the beam**:

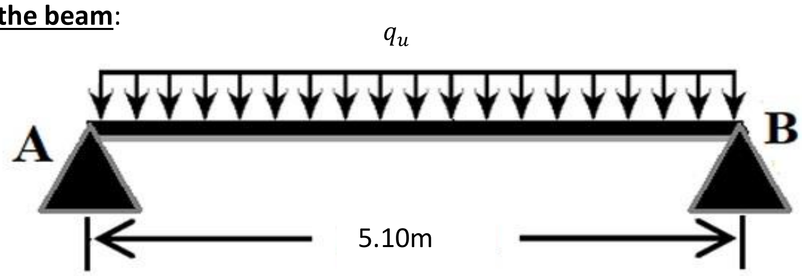


FIGURE III- 3: STATIC DIAGRAM OF REINFORCEMENT BEAM 1

$Self\ weight; G_0 = 25 * 0.30 * 0.35 = 2.625kN/m$

Coating weight; $G_{coating} = (0.4 + 0.4 + 0.36) * 0.3 = 0.348kN/m$

Walls weight: $G_{wall} = 2.76 * (3.74 * 0.3) = 9.494kN/m$

Load transmitted by the slab:

$P_u^{slab} = 1.35 * G_{slab} + 1.5 * Q_{slab} = 1.35 * 7.36 + 1.5 * 5 = 17.436kN/m$

SLAB A: $\rho = \frac{L_x}{L_y} = \frac{240}{510} = 0.47 > 0.4 \Rightarrow the\ slab\ works\ in\ both\ directions.$

$P_m = \frac{q}{2} * \left(1 - \frac{\rho_l^2}{3}\right) L_{xl} = \frac{17.436}{2} * \left(1 - \frac{0.47^2}{3}\right) * 2.4 = 19.38kN/m$

$P_v = \frac{q}{2}\left(1 - \frac{\rho_l}{2}\right) * L_{xl} = \frac{17.436}{2} * \left(1 - \frac{0.47}{2}\right) * 2.4 = 17.65kN/m$

Total load on the beam: $q_u^m = 1.35(2.625 + 0.348 + 9.4944) + 19.38 = 36.21kN/m$

$q_u^v = 1.35(2.625 + 0.348 + 9.4944) + 17.65 = 34.48kN/m$

- **Moments and shear forces on the beam**:

$M_u = \frac{q_u^m * l^2}{8} = \frac{36.21 * 5.1^2}{8} = 117.73kN.m$

$V_u = \frac{q_u^v * l}{2} = \frac{34.48 * 5.1}{2} = 87.92kN.m$

- **Reinforcement of beam 1**:

$$\mu_{bu} = \frac{M_t}{b_0 * d^2 * f_{bu}} = \frac{0.11773}{0.3 * 0.33^2 * 14.2} = 0.2538$$

$\mu_{bu} = 0.2538 > 0.186 \Rightarrow pivot\ B$

$\mu_{bu} = 0.2538 < \mu_l = 0.3916 \Rightarrow A' = 0$

$$A = \frac{M_u}{Z * f_{st}}$$

$$\propto = 1.25\left(1 - \sqrt{1 - 2\mu_{bu}}\right) = 0.373$$

$$Z = d(1 - 0.4 \propto) = 0.2808m$$

$$A = \frac{0.11773}{0.2808 * 348} = 12.05cm^2$$

- ***Verification of the non-brittleness condition:***

$$A_{smin} = 0.23 * b * d * \frac{f_{t28}}{f_e};$$

With $f_{t28} = 0.6 + 0.06 f_{c28} = 2.1MPa$

$$A_{smin} = 0.23 * 30 * 33 * \frac{2.1}{400} = 1.2cm^2;$$

$$A_{smin} = 1.2cm^2 < A_{cal} = 12.05cm^2$$

The simple flexure calculation for Beam 1 results in a required reinforcement cross-sectional area of $12.05cm^2$

$We\ choose\ 6HA16\ = 12.06cm^2$

- **Transverse reinforcement**

$$\tau_u = \frac{V_u}{b_0 * d} \leq \overline{\tau_u} = \min\left(0.2\frac{f_{c28}}{\gamma_s}; 5MPa\right)$$

$$\tau_u = \frac{V_u}{b_0 * d} = \frac{0.08792}{0.3 \times 0.33} = 0.888Mpa$$

$$\overline{\tau_u} = \min\left(0.2\frac{f_{c28}}{\gamma_s}; 5MPa\right) = 3.33MPa;$$

$\tau_u = 0.888MPa < \overline{\tau_u} = 3.33Mpa$ ⇒No risk of shear failure.

As calculated for the joists, we choose Ties and stirrups **φ 6** spaced at ***25cm***.

Therefore, we opt for spacing of $S_t = 25cm < 29.7cm$

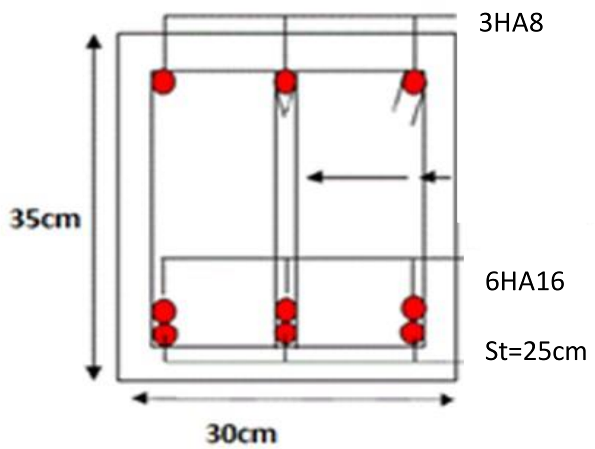


FIGURE III- 4: BEAM 1 REINFORCEMENT DIAGRAM

❖ **Beams 2 and 3**:

$\frac{L}{15} \leq h \leq \frac{L}{10} \quad \Rightarrow 30.33\text{cm} \leq h \leq 45.5\text{cm} \quad \Rightarrow h = 35\text{cm and } b = 30\text{cm}$

- **Loads on the beams**;

Own weight: $G_0 = 25 \times 0.30 \times 0.35 = 2.625kN/m$

Coating weight: $G_{coating} = 0.348kN/m$

Walls weight: $G_{wall} = 2.76 * (3.74 * 0.3) = 9.494kN/m$

- ***Load transmitted by the slab:***

$P_u^{slab} = 1.35G_{slab} + 1.5Q_{slab} = 1.35 * 7.36 + 1.5 * 5 = 17.436kN/m$

SLAB B and C: $\rho = \frac{L_x}{L_y} = \frac{240}{510} = 0.374 < 0.4 \Rightarrow the\ slab\ works\ in\ one\ directions\ only.$

$P_m^r = P_v^r = \frac{ql_x}{2} = \frac{17.436 \times 1.70}{2} = 14.82kN/m$

$P_m^t = \frac{q*l_x}{3} = \frac{17.436*2.40}{3} = 13.95kN/m$

$P_v^t = \frac{q*l_x}{4} = \frac{17.436*2.40}{4} = 10.46kN/m$

$P_m^T = P_m^t + P_m^r = 13.95 + 14.82 = 28.77kN/m$

$P_v^T = P_v^t + P_v^r = 10.46 + 14.82 = 25.28kN/m$

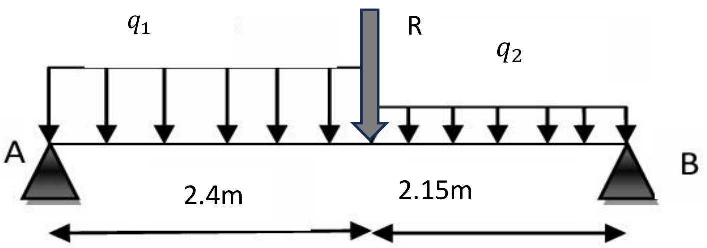


FIGURE III- 5: STATIC DIAGRAM OF REINFORCEMENT BEAM 2 AND 3.

$q_1 = P_m^T + 1.35 * (G_0 + G_{coating}) = 32.78kN/m$

$q_2 = P_m^r + 1.35 * (G_0 + G_{coating} + G_{wall}) = 31.65kN/m$

$R = 92.34kN\ ; Reaction\ of\ beam\ 1\ on\ beam\ 2\ and\ 3.$

- ***Moments and shear forces on the beams***:

The method of sections leads to the following expressions for the bending moment and the shear force.

$\begin{cases} R_A = 117.64KN \\ R_B = 121.43KN \end{cases}$

$M_{max}(x = 2.4m) = R_A * x - \frac{q_1 * x^2}{2} = 187.92kN/m.$

$V_{max} = 121.43kN.$

- **Reinforcement of beam 2 and 3**:

(b × h) = (30×35) cm^2; L=4.55m; $M_S = 187.92kN.m$ d=33cm; d'=2cm

$\mu_{bu} = \frac{M_{tu}}{b*d^2*F_{bu}} = \frac{0.18792}{0.3*0.33^2*14.2} = 0.405$

$\mu_{bu} = 0.405 > 0.186 \Rightarrow pivot\ B \rightarrow\ f_S = \frac{fe}{\gamma_s} = \frac{400}{1.15} = 348MPa$

$\mu_{bu} = 0.405 > \mu_l = 0.3916 \Rightarrow A' = 0$

$\mathrm{M_l = \mu_l * b * d^2 \times f_{bu} = 0.3916 * 0.3 * 0.33^2 * 14.2 = 181.67kN}$

$\varepsilon_{sc} = \left(\frac{3.5}{1000} + \frac{400}{1.15*2*10^5}\right) * (\frac{0.33-0.02}{0.33}) - \frac{400}{1.15*2*10^5} = 3.182 * 10^{-3} > \varepsilon_l = 1.74 * 10^{-3}$

$\Rightarrow\ \mathrm{f_{st} = \frac{f_e}{\gamma_s} = \frac{400}{1.15} = 348MPa}$

$\mathrm{A' = \frac{M_u - M_l}{(d-d')*f_{st}} = \frac{(187.92-181.67)*10^{-3}}{(0.33-0.02)*348} = 0.58cm^2}$

$\mathrm{Z_l = d(1 - 0.4 \propto) = 0.2418m}$

$A = \frac{M_l}{(d-d')*f_{st}} - A' = \frac{0.18167*10^{-3}}{0.2418*348} - (0.58 * 10^{-4}) = 21.01cm^2$

- **Verification of non-fragility condition;**

  $A_{smin} = 1.20cm^2 < A_{cal}$ we reinforce withA_{cal}, we choose $5HA25 = 24.54cm^2$.

- ***Verifications of shear forces;***

$\boldsymbol{\tau_u} = \frac{\boldsymbol{V_u}}{\boldsymbol{b_0 \times d}} = \frac{0.12143}{0.3\times0.33} = 1.23MPa\ < \overline{\tau_u} = 3.33MPa\ \Rightarrow$ No risk of shear failure.

As calculated for the joists, we choose Ties and stirrups **φ 6** spaced at ***25cm***.

Therefore, we opt for spacing of $S_t = 25cm < 29.7cm$

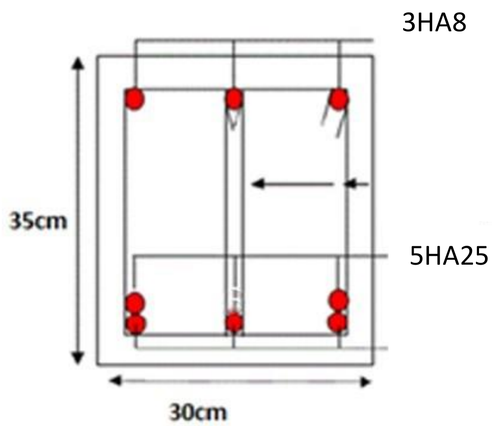


FIGURE III- 6: BEAM 2 AND 3 REINFORCEMENT DIAGRAMS

III.2.2.2 Slab on two supports (slab 5)

$\rho = \frac{L_x}{L_y} = \frac{100}{420} = 0.238 < 0.4 \Rightarrow$ the slab works in one direction as a cantilever

$\mathrm{G = 6.11kN/m^2}$: $\mathrm{G_{wall} = 2.76kN/m^2}$ $\mathrm{Q = 3.5kN/m^2}$;

For 1ml: $\mathrm{q_u = 1.35G + 1.5Q = 13.5kN/m/ml}$

$\mathrm{q_S = G + Q = 9.61kN/m/ml}$

$q_{uwall} = 1.35G_{wall} * h_{wall} = 1.35 * (2.76 * (3.06 - 0.15)) = 10.84kN/ml$

$q_{swall} = G_{wall} * h_{wall} = (2.76(3.06 - 0.15) = 8.0316kN/ml$

$M_u = \frac{-q_u * l_x^2}{2} + q_{uwall} * l_x = -17.59kN.m$

$M_s = \frac{-q_s * l_x^2}{2} + q_{swall} * l_x = -12.84kN.m$

$V_u = q_u * l_x + q_{uwall} = 24.34kN$

The reinforcement calculations are shown in table III.20:

TABLE III 20: REINFORCEMENT CALCULATIONS OF SLAB 5

Location	direction	M_u $(kN.m)$	μ_{bu}	$\propto$	$z(cm)$	A_{cal} (cm^2)	A_{min} (cm^2)	$A_{ad}(cm^2)$	$s_t(cm)$
Span	X-X	17.59	0.086	0.113	11.46	4.41	1.20	4HA12=4.52	25

Distribution and support bars

$A_r = A_y = \frac{A_s}{4} \quad \Rightarrow \frac{4.52}{4} = 1.13cm^2 \quad we\ take\ 4HA8 = 2.01cm^2$

$A_a = \frac{A_r}{4} \quad \Rightarrow \frac{2.01}{4} = 0.5cm^2 \quad we\ take\ 3HA8 = 1.51cm^2$

- **ULS verification**
- ***Shear force verification***

The results of shear force verification are shown in the following table III.21.

TABLE III 21: SHEAR FORCE VERIFICATION.

Location	Direction	$V_u\ (kN)$	$\tau_u \leq \overline{\tau_u}$ MPa		OBS
			τ_u	$\overline{\tau_u}$	
Span	X-X	24.34	0.202	1.17	Verified

- **SLS verification**
- ***Constraints verification***

The constraints verification is shown in the following table,

TABLE III 22: CONSTRAINTS VERIFICATION

Location	M_s	Direction	$Y(cm)$	$I(cm^4)$	$\sigma_{bc} \leq \bar{\sigma}_{bc}(MPa)$		$\sigma_{bc} \leq \bar{\sigma}_{bc}(MPa)$	
					σ_{bc}	σ_{bc}	σ_{bc}	σ_{bc}
Span	12.84	X-X	3.41	6324.56	6.92	15	261.59	201.63

The constraints in SLS in the steel are not verified, so we must recalculate the reinforcement using SLS.

$A_{ser} = \frac{M_{ser}}{d\left(1-\frac{\alpha}{3}\right)*\bar{\sigma}_{st}} \quad \alpha = \sqrt{90 * \beta * \frac{1-\alpha}{3-\alpha}} \qquad \beta = \frac{M_{ser}}{b*d^2*\bar{\sigma}_{st}}$

$\beta = 4.429 * 10^{-3} \qquad \alpha = 0.318 \qquad A_{ser} = 5.95\ cm^2$

Therefore $A_s = 4HA16 = 6.16cm^2 \quad and \quad A_r = 1.54cm^2$

Deflection verification

Verifying the conditions

1. $\frac{h}{L} \geq \frac{3}{80} \Rightarrow \frac{15}{100} = 0.15 > \frac{3}{80} = 0.0375 \Rightarrow verified$
2. $\frac{h}{L} \geq \frac{M_t^S}{20\times*} \Rightarrow 0.15 > \frac{0.01759}{20*0.01759} = 0.05 \rightarrow verified$
3. $\frac{A_t}{b_0*d} \geq \frac{2}{F_E} \Rightarrow \frac{6.16}{100*18} = 3.422 * 10^{-3} < \frac{2}{400} = 5 * 10^{-3} \rightarrow not\ verified$

We must verify deflection for slab D5. The results are shown in the following table,

TABLE III 23: DEFLECTION VERIFICATION OF SLAB D5.

Slab	L(m)	$M_t^g(kN.m)$	$M_t^j(kN.m)$	$M_t^p(kN.m)$	t(cm)	f_{gv} (mm	f_{gi}(mm	f_{ji}(mm	f_{pi}(mm	Δf(mm	$\bar{f}$(mm	OBS
D5	1.00	3.06	2.56	4.81	15	0.22	0.07	0.06	0.12	0.21	2.0	Verified

III.2.2.3 Slab on three supports (slab 7)

For slabs with three supports and $\rho \geq 0.4$

Case 1: $L_x < \frac{L_y}{2}$, moments are calculated using: $M_{0x} = \frac{q*L_x^3*L_y}{2} - \frac{2*q*L_x^3}{3}$ and $M_{0y} = \frac{q*L_y^3}{6}$.

Case 2: $L_x > \frac{L_y}{2}$; moments are calculated using; $M_{0x} = \frac{q\times L_y^4}{24}$

$M_{0y} = \frac{q*L_y^2}{8}\left(L_x - \frac{L_y}{2}\right) + \frac{q*L_y^3}{48}$

$\rho = \frac{L_x}{L_y} = \frac{170}{520} = 0.324 < 0.4 \Rightarrow the\ slab\ works\ in\ X\ direction\ only\ as\ a\ cantilever.$

$G = 7.36kN/m^2\ and\ Q = 5kN/m^2$;

For 1ml: $q_u = 1.35G + 1.5Q = 17.436kN/m$

$q_s = G + Q = 12.36kN/m$

At ULS: $M_u = -\frac{q_u*l_x^2}{2} = -\frac{17.436\times1.7^2}{2} = -25.195kN.m$

At SLS: $M_s = -\frac{q_s*l_x^2}{2} = -\frac{12.36*1.7^2}{2} = -17.86kN.m$

Shear force: $V_u = q_u * L_x = 17.436 * 1.7 = 29.64$kN

The reinforcement calculations are shown in table III.24:

TABLE III 24: REINFORCEMENT CALCULATIONS OF SLAB D7

Location	Direction	$M_u(kN.m)$	μ_{bu}	$\propto$	$z(cm)$	A_{cal} (cm^2)	A_{min} (cm^2)	A_{ad} (cm^2)	$s_t(cm)$
Span	X-X	25.195	0.055	0.0708	17.49	4.14	1.60	4HA12=4.52	25

Distribution and support bars

The same approach was followed as in slab 5. We have,

$A_y = 4HA8 = 2.01cm^2$ and $A_a = 3\text{HA8} = 1.51cm^2$

- **ULS verification**
- ***Shear force verification***

The results of shear force verification are shown in the following table:

TABLE III 25: SHEAR FORCE VERIFICATION OF SLAB D7

Location	Direction	$V_u\ (kN)$	$\boldsymbol{\tau_u} \leq \overline{\tau_u}$		OBS
			$\boldsymbol{\tau_u}$	$\overline{\tau_u}$	
Span	X-X	29.64	0.915	1.17	Verified

- **SLS verification**
- ***Constraints verification***

The constraints verification is shown in the following table,

TABLE III 26: CONSTRAINTS VERIFICATION OF SLAB D7,

Location	M_s (kN.m)	Direction	$Y(cm)$	$I(cm^4)$	$\sigma_{bc} \leq \bar{\sigma}_{bc}(MPa)$	
					σ_{bc}	σ_{bc}
Span	17.86	X-X	4.309	15375.58	5.01	15

- ***Deflection verification***

Verifying the conditions

The conditions for omitting deflection verification are not met; therefore, deflection calculation is required.

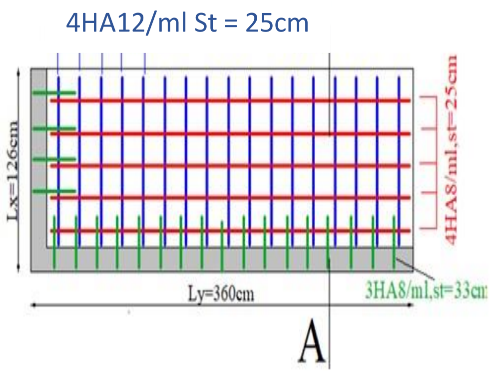
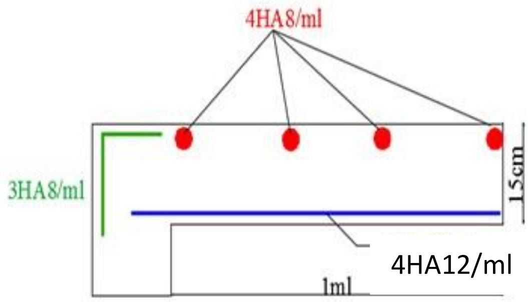
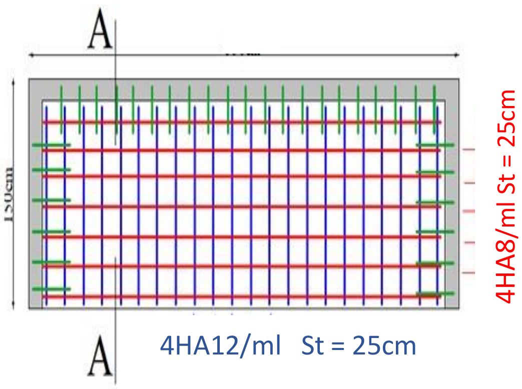
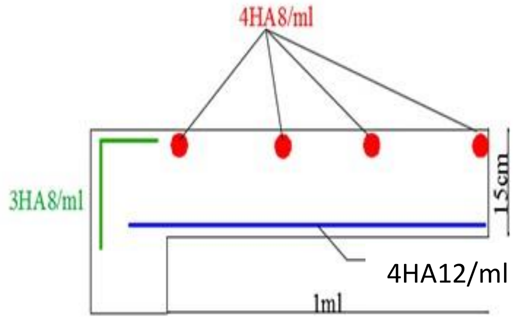
The results are shown in the following table:

TABLE III 27: DEFLECTION VERIFICATION OF SLAB D7

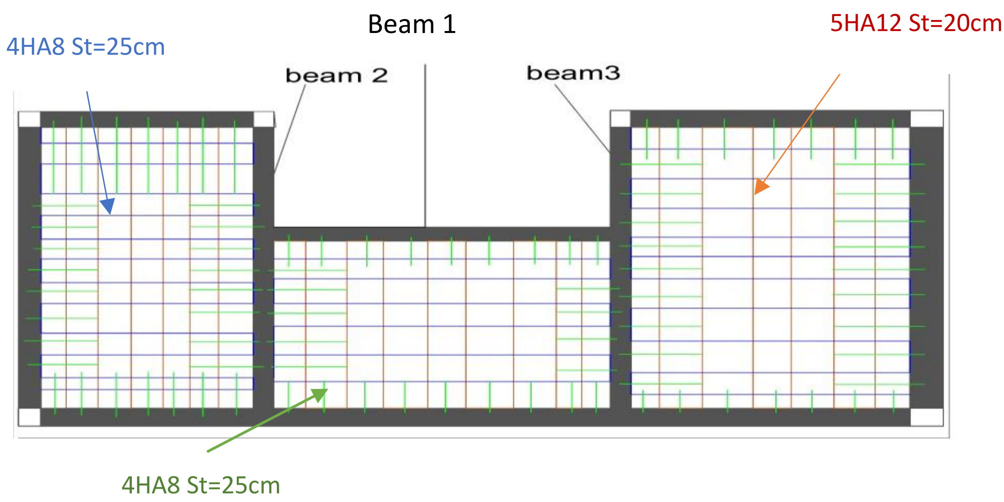
Slab	L(m)	M_t^g(kN.m)	M_t^j(kN.m)	M_t^p(kN.m)	t(cm)	f_{gv} (mm	f_{gi}(mm	f_{ji}(mm	f_{pi}(mm	Δf(mm	$\bar{f}$(mm	OBS
D7	1.70	10.64	8.67	17.86	20	0.91	0.30	0.25	1.29	1.65	3.40	Verified

The slab reinforcement diagrams are shown in the table below:

TABLE III 28: SLABS REINFORCEMENT DIAGRAMS

<table>
<tr><th>Slab</th><th>Diagram</th><th>Section A-A</th></tr>
<tr><td>Slab 5</td><td>
</td><td>
</td></tr>
<tr><td>Slab 7</td><td>
</td><td>
</td></tr>
</table>

Slab 9



III.3 Staircase design

- **Stairs with three flights (Ground floor)**

a) First and third flight

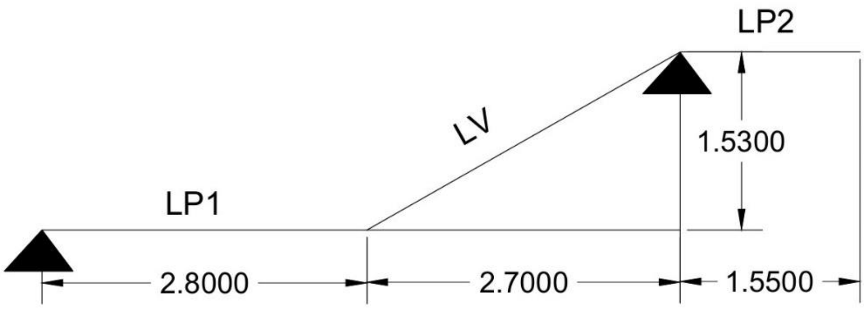


FIGURE III- 7: STAIRCASE DIAGRAM

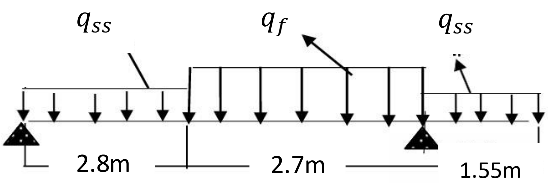


FIGURE III- 8: GROUND FLOOR STAIRS STATIC DIAGRAM

Load combinations:

$G_f = 9.24kN/m^2$; $G_l = 6.52kN/m^2$; $Q = 5.00kN/m^2$

$$At\ ULS \begin{cases} q_f = 1.35 \times 9.24 + 1.5 \times 5 = 19.974kN/m^2 \\ q_{ss} = 1.35 \times 6.52 + 1.5 \times 5 = 16.302kN/m^2 \end{cases}$$

$$At\ SLS \begin{cases} q_f = 9.24 + 5 = 14.24kN/m^2 \\ q_{ss} = 6.52 + 5 = 11.52kN/m^2 \end{cases}$$

- **Moments and shear forces calculations**

The method of sections leads to the following expressions for the bending moment and the shear force.

At ULS=$\begin{cases} R_A = 42.19kN \\ R_B = 82.65kN \end{cases}$ AND At SLS=$\begin{cases} R_A = 29.89kN \\ R_B = 58.67kN \end{cases}$

The bending moments and shear forces values are shown in tables III.28 and III.29.

TABLE III 29: BENDING MOMENTS RESULTS OF FLIGHT 1 AND 2.

Section	Bending moments equations m(x)	Bending moment value (kN.m)		
			ULS	SLS
$0 \leq x \leq 2.8m$	$R_A * x - q_{ss} * \frac{x^2}{2}$	$x = 0$	0	*0*
		$x = 2.8$	*54.23*	*38.53*
$2.8m \leq x \leq 5.5m$	$(R_A \times x) - q_{ss} \times 2.8 \times \left(\frac{2.8}{2} + (x - 2.80)\right) - q_f\left(\frac{x - 2,8^2}{2}\right)$	$x = 2.8$	*54.23*	*41.11*
		$x = 5.5$	*-27.91*	*-14.70*
$5.5 \leq x \leq 7.05m$	$-q_{ss}\frac{(x-5.5)^2}{2} - 2.7q_f\left(\frac{2.7}{2} + (x - 5.5)\right) - 2.8q_{ss}\left(\frac{2.8}{2} + 2.7 + (x - 5.5)\right) + R_A x + R_B(x - 5.5)$	$x = 5.5$	*-31.35*	*-14.70*
		$x = 7.05$	*-8.33*	*0.565*

TABLE III 30: SHEAR FORCES RESULT OF FLIGHT 1 AND 2.

Section	*Shear forces equations T(x)*	*Shear forces value (kN)*	
$0 \leq x \leq 2.8m$	$q_{ss} * x - R_A$	$x = 0$	*-42.18*
		$x = 2.8$	*3.46*
$2.8m \leq x \leq 5.5m$		$x = 2.8$	*3.46*

	$2.8q_{ss}+q_f(x-2.80) - R_A$	$x = 5.5$	*57.39*
$5.5 \leq x \leq 7.05m$	$2.8q_{ss}+q_{ss}(x - 5.50)+2.7q_f - R_A - R_B$	$x = 5.5$	*-25.26*
		$x = 7.05$	*0.0035*

The change of sign in the first equation means that the maximum moment is in the interval [0; 2.8],

$$\frac{\Delta m}{\Delta x} = 0 \Rightarrow \ R_A * x - q_{ss} = 0 \quad therefore\ x = 2.59m$$

We replace the Valeur of x in the first equation,

$$M_{max} = \begin{cases} 54.59kN.m\ (ULS) \\ 41.20kN.m\ (SLS) \end{cases}$$

- ***Maximum stresses***

The moments in the span and at the supports according to BAEL are as follows,

- ✓ $M_{span} = 0.85 * M_{max}$
- ✓ $M_{support} = 0.5 * M_{max}$

The results are shown in the following table:

TABLE III 31: MAXIMUM STRESSES IN SPAN AND SUPPORT.

		ULS (kN.m)	SLS (kN.m)
Moments	Span	46.40	35.02
	Support	27.295	20.6
Shear force		82.65	

- **Stair's reinforcement**:

The reinforcement is done in ULS while taking into consideration low harm cracking.

$b = 100cm \quad h = 20cm \quad f_{c28} = 25MPa \quad d = 18cm \quad fe = 400MPa$

The results of the reinforcement calculations are presented in table III:

TABLE III 32: STAIR'S REINFORCEMENT AT ULS

	Moments(kN.m)	μ_{bu}	$\propto$	$z(cm)$	$A_{cal}(cm^2)$	$A_{min}(cm^2)$	$A_{adopt}(cm^2)$	St
Span	46.40	0.1009	0.133	0.1704	7.82	2.17	4HA16=8.04	25
Support	27.295	0.0593	0.0765	0.1745	4.49		4HA12=4.52	

<u>Distribution bars</u>:

$A_{dist} = \frac{A_{adopted}}{4}$

At span: $A_{dist} = \frac{8.04}{4} = 2.01cm^2/m$ *we take* $A_{dist} = 4HA8 = 2.01cm^2/m \ \Rightarrow St = 25cm$

At support: $A_{dist} = \frac{4.52}{4} = 1.13cm^{2/m}$; *we take* $A_{dist} = 3HA8 = 1.51cm^2/m \ \Rightarrow \ St = 33cm$

- **<u>Verification of shear forces:</u>**

$\tau_u = \frac{V}{b*d} \leq 0.07 * \frac{f_{c28}}{\gamma_b} = 1.17\ MPa$

$\tau_u = \frac{82.65*10^{-3}}{1*0.18} = 0.459MPa < 1.17MPa$

Transversal bars are not necessary.

- **SLS verifications:**
  - **Verification of constraints.**

The verifications are shown in the table III.32.

TABLE III 33: CONSTRAINTS VERIFICATION

Location	$M_{Support}$ (kN.m)	Y (cm)	I (cm^4)	$\sigma_{bc} \leq \bar{\sigma}_{bc}$ *in MPa*		Observation
				σ_{bc}	$\bar{\sigma}_{bc}$	
Span	32.963	5.49	24389.55	7.42	15	verified
Support	19.39	4.31	15375.58	5.44	15	verified

- **Verification of deformation.**

The conditions for omitting deflection verification are not met; therefore, deflection calculation is required.

The calculations carried out have resulted in the findings presented in Table III.33:

TABLE III 34: DEFORMATION VERIFICATION.

M_s^G	M_s^j	M_s^p	$f_{gv}(mm)$	$f_{ji}(mm)$	$f_{pi}(mm)$	$f_{gi}(mm)$	$f(mm)$	$\bar{f}(mm)$	OBS
20.90	16.03	36.93	10.71	3.10	13.94	5.34	16.22	10.50	Not verified

The deformation is not verified hence we increased the section to: 6HA20=18.85cm^2/m with St=16cm

$A_{dist} = \frac{18.85}{4} = 4.71cm^2/m$; *we take* $A_{dist} = 6HA10 = 4.71cm^2/m \ \ with\ St = 16cm.$

The verification of deformation is shown in the table below:

TABLE III 35: CORRECTED DEFLECTION VERIFICATION.

$f_{gv}(mm)$	$f_{ji}(mm)$	$f_{pi}(mm)$	$f_{gi}(mm)$	$f(mm)$	$\bar{f}(mm)$	OBS
7.89	2.19	7.63	3.35	9.98	10.50	No risk of deformation

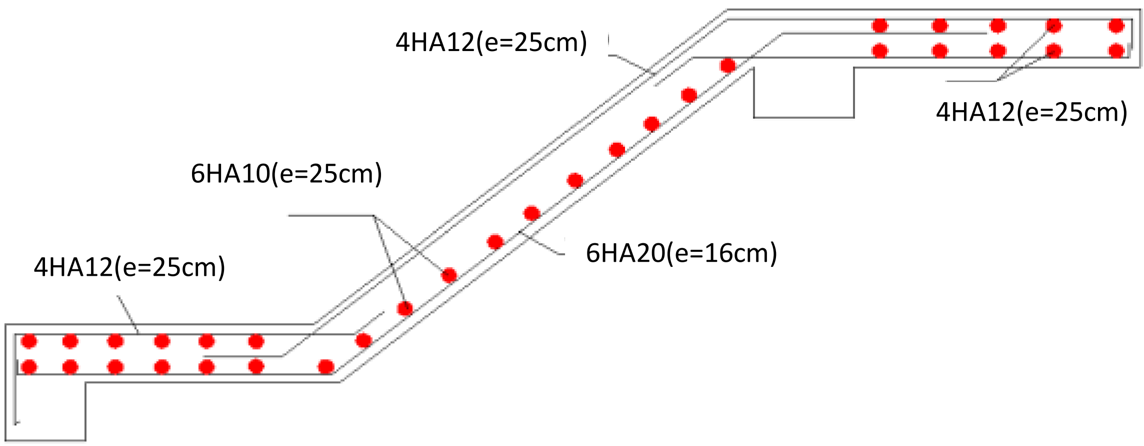


FIGURE III- 9: FLIGHT 1 AND 2 REINFORCEMENT DIAGRAMS

2)Study of the cantilever (flight 2):

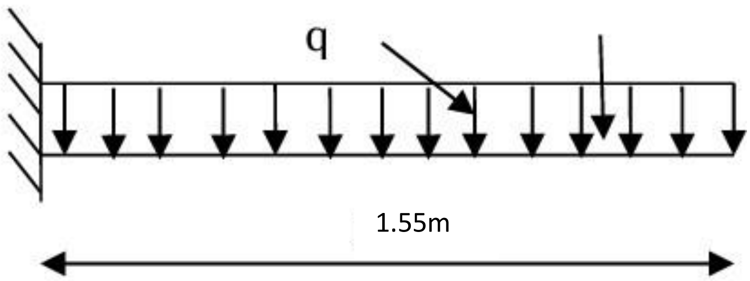


FIGURE III- 10: STATIC DIAGRAM OF THE CANTILEVER

$G_f = 9.24kN/m^2$; $Q_f = 5.00kN/m^2$

At ULS: $q_u^f = 1.35G_f + 1.5Q_f = 19.974kN/m$

At SLS: $q_s^f = G_f + Q_f = 14.24kN/m$

Calculating for the moments and shear forces.

M= $-q_f \times \frac{l^2}{2} \Rightarrow M = \begin{cases} ULS = -\frac{19.974\times1.55^2}{2} = -23.95kN.m \\ SLS = -\frac{14,24\times1.55^2}{2} = -17.11kN.m \end{cases}$

$V_u = q_u \times l = 19.974 \times 1.55 = 30.96kN$

The results of reinforcement calculations are summarized in table III.36:

TABLE III 36: CANTILEVER REINFORCEMENT CALCULATIONS

Mt (kN.m)	μ_{bu}	$\propto$	Z(m)	$A_{cal}(cm^2/ml)$	$A_{min}(cm^2/ml)$	$A_{opt}(cm^2/ml)$	St(cm)
-23.95	0.052	0.0668	17.52	3.93	2.17	5HA10=3.93	20

Secondary bars

Ay=$A_y = \frac{A_x}{4} \Rightarrow A_y = \frac{3.93}{4} = 0.98(cm^2/ml)$ $hence\ A_{adopted} = 3HA8 = \frac{1.51cm^2}{m};\ S_t = 33cm$

Shear force verifications:

$$\tau_u = \frac{V_u}{bd} = \frac{30.96\times10^{-3}}{1\times0.18} = 0.172MPa < 1.17MPa$$

$Transversal\ bars\ are\ not\ not\ necessary.$

- **SLS verifications**
  - **Verification of constraints.**

Table III. 35 Summaries the results of constraints calculations

TABLE III 37: CONSTRAINTS VERIFICATION

$M_{Support}$ (KN.m)	Y (cm)	I (cm^4)	$\sigma_{bc} \leq \bar{\sigma}_{bc}\ in\ MPa$		Observation
			σ_{bc}	$\bar{\sigma}_{bc}$	
-17.11	4.06	13686.16	5.07	15	verified

- **Verification of deformation.**

  The conditions for omitting deflection verification are not met; therefore, deflection calculation is required.

The verification of deformation is shown in the table below:

TABLE III 38: FLIGHT 2 DEFLECTION VERIFICATION

M_s^G	M_s^j	M_s^p	$f_{gv}(mm)$	$f_{ji}(mm)$	$f_{pi}(mm)$	$f_{gi}(mm)$	$f(mm)$	$\bar{f}(mm)$	OBS
11.10	9.15	17.11	0.0796	0.0219	10.0409	0.0265	0.0782	6.2	Verified

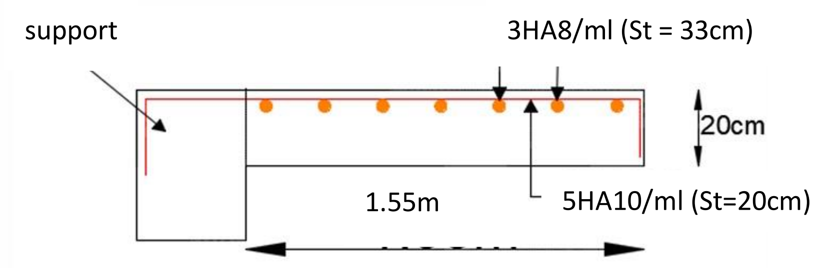


FIGURE III- 11: CANTILEVER REINFORCEMENT DIAGRAM

III.4 Acroterion study

-**Evaluation of loads**

Surface area(S) $= 0.0685m^2$

$G_0 = 25 * S_{acr} = 25 * 0.0685 = 1.715kN/m$

Cement coating = $18 * 2 * 0.02 * 0,60 = 0.432kN/ml$

$Q_{acr} = 1kN/ml$

According to RPA, the acroterion is subjected to a horizontal force due to earthquake:

$F_p = 4 \times A \times C_p \times W_p$ $(RPA2003\ Art\ 6.2.3)$

A: zone acceleration coefficient that depends on the seismic zone and usage group of the structure.

C_p: horizontal force factor varying from 0.3 to 0.8

W_p: weight of the element considered

In our case, we have seismic zone II_a and usage group 2, hence A=0.15 (table 4.1 of RPA2003)

$C_p = 0.8$ (table 6.1 of RPA2003)

$F_p = 4 \times 0.15 \times 0.80 \times 2.1445 = 1.029kN/m$

- **Stresse calculation** :

Calculation of center of gravity of the section G (Y_G and X_G)

$$X_G = \frac{\sum X_i \times A_i}{\sum A_i} = \frac{\left(60\times10\times\frac{10}{2}\right)+\left[7\times10\times\left(10+\frac{10}{2}\right)\right]+\left[3\times\frac{10}{2}\times\left(10+\frac{10}{3}\right)\right]}{(60\times10)+(7\times10)+\left(3\times\frac{10}{2}\right)} = 6.204\text{cm}$$

$$Y_G = \frac{\sum Y_i \times A_i}{\sum A_i} = \frac{\left(60\times10\times\frac{60}{2}\right)+\left[7\times10\times\left(50-\frac{7}{2}\right)\right]+\left[3\times\frac{10}{2}\times\left(57+\frac{3}{3}\right)\right]}{(60\times10)+(7\times10)+\left(3\times\frac{10}{2}\right)} = 33.01\text{cm}$$

The internal forces obtained under the various loads are presented in the table III.39:

TABLE III 39: INTERNAL FORCES UNDER THE DIFFERENT LOADS IN ACROTERION

	N (KN)	M (kN.m)	$V(kN)$
G	2.144	0	0
Q	0	$Q.h = 0.60$	1
F	1.029	$F.Y_G = 0.34$	F

TABLE III 40: INTERNAL FORCES UNDER THE DIFFERENT COMBINATIONS IN ACROTERION

Combination	**ULS**	**SLS**	**AULS**	
stresses	1.35G+1.5Q	G+Q	$G + Q \pm E$	$0.8G + Q \pm$E

N (kN)	2.895	2.1445	2.1445	1.716
M (kN.m)	0.90	0.60	0.94	0.34

- **Reinforcement at compound bending**

The combined bending and axial force design of the parapet under the most critical load combinations resulted in the reinforcement details presented in the table III.39

Calculation example (ULS):

Eccentricity calculation

$e_0 = \frac{Mu}{Nu} = \frac{0.90}{2.895} = 0.311m$

$e_1 = \frac{H}{6} = \frac{0.6}{6} = 0.1m$

$e_0 = 0.311m < e_1 = 0.1m$

Center of pressure is outside the section while Nu is a compression force.

The section is partially compressed

Therefore, the calculations are done by assimilation of simple bending subjected to a moment of **Mua=Nux.**

We replace the actual eccentricity ($e = \frac{Mu}{Nu}$) by a total design eccentricity: $e = e_a + e_1 + e_2$

$e_a = \max\left(2cm; \frac{h}{250}\right) = 2cm$

$e_2 = \frac{3 * l_f^2}{10^4 * h}(2 + \alpha\emptyset);$

$\emptyset$: Ratio of the final strain due to creep to the instantaneous strain under load.

with; α: $\frac{M_G}{M_G + M_Q} = 0$;

l_f: buckling length $\quad l_f = 2 * h = 1.20m$

$e_2 = \frac{3 * 1.2^2 * 2}{10^4 * 0.10} = 8.64 * 10^{-3}m = 0.864cm$

e=2+31.1+0,864=33.964cm

The stresses for calculations: Nu=2.895kN and Mu=2.895*0.33964=0.98 kN.m

Acroterion reinforcement

The position of center of pressure:

$e_g = \frac{M_G}{N_U} = \frac{0.98}{2.895} = 0.339m > y_g = \frac{h}{2} = 0.05m$ ⇒ center of pressure is outside the section

The section is partially compressed.

The reinforcement is done using simple bending under the force of a fictitious moment of

$M_{Ua} = M_{UG} + Nu\left(d - \frac{h}{2}\right) = 1.0379kN.m$

TABLE III 41: ACROTERION REINFORCEMENT RESULTS

M_{Ua} KN.m	μ_{bu}	$\propto$	Z(m)	A_1 (cm^2)	A cm^2	A_{smin}	A_{adopt}	S_t (cm)
1.0379	0.0149	0.0188	0.0647	0.46	0.38	0.85	4HA8=2.01	25

Distribution barres;

$A_{dist} = \frac{A_{adopted}}{3}$

$A_{dist} = \frac{2.01}{3} = 0.67cm^2$; *we take* $A_{dist} = 3HA8 = 1.51cm^2 \quad \Rightarrow S_t = 20cm$

Shear verifications:

We verify that:

$\tau_u = \frac{V}{b*d} \leq \overline{\tau_u} = \min(0.1f_{c28}; 4MPa) = 2.5MPa$

$V_u = Q + F_p = 1 + 1.029 = 2.029kN$

$\tau_u = \frac{V_u}{b*d} = 0.29MPa < 2.5MPa \quad \Rightarrow$ conditon verified

- **Verification at SLS**

For the parapet, the cracking is harmful:

$$\sigma_{bc} = \frac{M_{ser}}{\mu_t} * Y_{ser} \leq \bar{\sigma}_{bc} = 0.6 * f_{c28} = 15MPa$$

$$\sigma_{st} = 15 * \frac{M_{ser}}{\mu_t} * (d - Y_{ser}) \leq \bar{\sigma}_{st} = 201.63MPa$$

Neutral axis position

$Y_{ser}=Y_c + C$ and $C = d - e_1$;

with ;

e_1: distance from the centre of pressure $'c'$ to the most compressed fibre of the section.

$e_1 = \frac{Ms}{Ns} + \left(d - \frac{h}{2}\right) = \frac{0.60}{2.1445} + \left(0.07 - \frac{0.10}{2}\right) = 0.2998m = 0.30m$>d=0.07m$\Rightarrow$

$e_1 = 0.30m >$ d=0.07m $\Rightarrow$ $'c'$ at exterior of the section. $\quad \Rightarrow C = 0.07 - 0.30 = -0.23m$

$y_c + py_c + q = 0 \ldots\ldots\ldots.1$

$$\begin{cases} p = -3c^2 + \frac{90A}{b}(d-c)^2) = -0.153m^2 \\ q = -2c^3 - \frac{90A}{b}(d-c)^2 = 0.0227m^3 \end{cases}$$

In replacing q and p in 1;

$\Delta = 4p^3 + 27q^2 = -4.135 \times 10^{-3}$.

There are 3 real roots, we keep the suitable one out the following:

$-C \leq Y_c \leq h - C$ which gives $-0.23 \leq y_C \leq 0.30$

$y_{c1} = a\cos(\frac{\emptyset}{3}) = 0.2477$ with $\mathbf{a} = 2\sqrt{\frac{-p}{3}} = 0.4517$; $\emptyset = \cos^{-1}(\frac{3q}{2p}\sqrt{\frac{-3}{p}}) = 170.22°$

$y_{c2} = a\cos\left(\frac{\emptyset}{3} + 120\right) = -0.451$

$y_{c3} = a\cos(\frac{\emptyset}{3} + 240) = 0.2032$

Therefore;

We take: $y_C = 0.25$ and $Y_{ser} = 0.02m$

$\mu_t = \frac{by^2}{2} - 15A(d - y) = 4.925 * 10^{-5}m^3$

$\sigma_{bc} = 0.871MPa < \overline{\sigma_{bc}} = 15MPa$

$\sigma_{st} = 32.657MPa < \overline{\sigma_{st}} = 201.6MPa$

The conditions are verified.

The acroterion reinforcement diagram is shown in figure III-12:

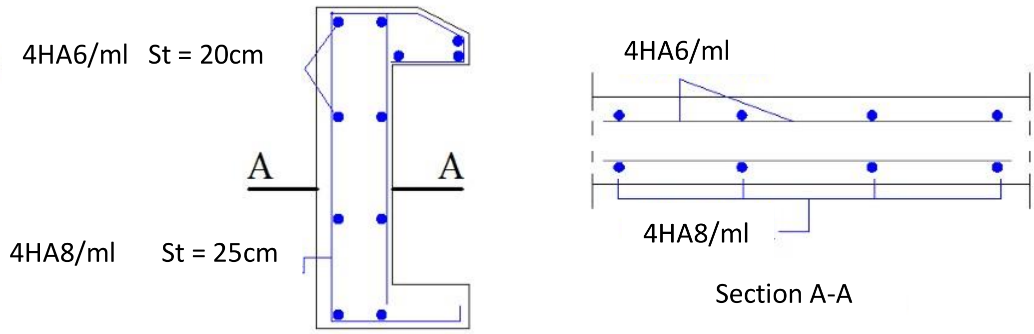


FIGURE III- 12: ACROTERION REINFORCEMENT DIAGRAM

III.5 Landing beam study

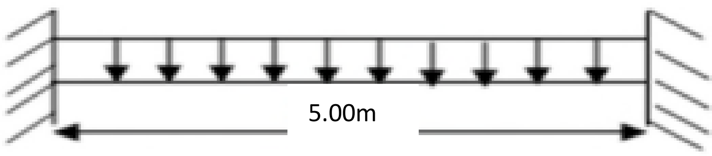


FIGURE III- 13: LANDING BEAM STATIC DIAGRAM

- **Dimensioning:**

According to stiffness condition,

$\frac{L}{15} \leq h \leq \frac{L}{10}$ $\Rightarrow 33.33cm \leq h \leq 50cm \Rightarrow$ $h = 35cm \text{ and } b = 30cm$

- **Verification of the conditions of RPA version 2003.**

$b = 30cm \geq 20cm \Rightarrow verified$: and $h = 35cm \geq 30cm \Rightarrow verified$

$\frac{h}{b} = \frac{35}{30} = 1.17 < 4 \Rightarrow verified$

- **Calculation of the loads**

The maximum loads which were taken directly from ETABS are shown in the table below:

TABLE III 42: MAXIMUM LOADS IN LANDING BEAM.

ULS		SLS		V_u(kN)
M_{Span}(kN.m)	M_{Suport}(kN.m)	M_{Span}(kN.m)	$M_{Support}$(kN.m)	
13.58	-38.64	9.81	-26.96	53.93

- **Longitudinal reinforcement**

The results of longitudinal reinforcement in the span and at the supports are summarized in the following table,

TABLE III 43: REINFORCEMENT CALCULATION AT ULS OF THE LANDING BEAM.

Location	$M_u(kN.m)$	μ_{bu}	$\propto$	$z(cm)$	$A_{cal}(cm^2)$	$A_{min}(cm^2)$	$A_{min}^{RPA}(cm^2)$
Span	13.58	0.0292	0.0370	0.3251	1.20	1.2	4.5
Support	38.64	0.0833	0.1089	0.3156	3.06	1.2	4.5

- **Necessary verifications at ULS**
- **Shear force verification**

$\boldsymbol{\tau_u} = \frac{\boldsymbol{V_u}}{\boldsymbol{b_0 \times d}} \leq \overline{\tau_u} = \min\left(0.2\frac{f_{c28}}{\gamma_s}; 5Mpa\right) = 3.33MPa$

$\tau_u = \frac{V_u}{b_0 \times d} = 0.594MPa$ $\Rightarrow$ There is no risk of shear failure.

- **Longitudinal reinforcement verification**

$\boldsymbol{A} \geq \left(V_u + \frac{M_u}{0.9*d}\right) * \frac{\boldsymbol{\gamma_s}}{\boldsymbol{fe}} =$

- **Transversal reinforcement calculation**

We fix $S_t = 15cm$ in span and support and calculate A_{trans}.

$$\begin{cases} A_{trans} \geq \frac{0.4*b*S_t}{fe} & \rightarrow A_{trans} \geq 0.45cm^2 \\ A_{trans} \geq \frac{b*S_t*(\boldsymbol{\tau_u}-\mathbf{0.3}*\boldsymbol{f_{t28}})}{0.9*fe} & \rightarrow A_{trans} \geq -4.5*10^{-6} \end{cases}$$

We take $A_{trans} = 0.45cm^2$

- **Torsion calculation**

For a solid section, the real section is replaced by an equivalent hollow section whose wall thickness is equal to one-sixth of the diameter of a circle that can be inscribed within the contour of the section.

$M_{tor} = 38.64kNm$

$A_l^{tor} = \frac{M_{tu} * U}{2 * f_{st} * \Omega}$

$U: Perimeter\ of\ the\ hollow\ section = 2 * \left((b-e)+(h-e)\right) = 1.0668m$

Ω: Area of the hollow section $= (\mathrm{b}-\mathrm{e})-(\mathrm{h}-\mathrm{e}) = 0.0705\mathrm{m}^2$

$e = \frac{h}{6} = 0.0583m$

$A_l^{tor} = 8.40cm^2$

- **Global reinforcement**

In span

$A_s = A_l^{fs} + \frac{A_l^{tor}}{2} = 7.65cm^2$

In support

$A_a = A_a^{fs} + \frac{A_l^{tor}}{2} = 7.71cm^2$

- **Shear conditions verification**

$\tau^{tor} = \frac{M_{tu}}{2 * e * \Omega} = 5.71MPa$

$\tau^{tor} = 5.71MPa > \overline{\tau_u} = 3.33MPa \quad \Rightarrow\ condition\ not\ verified$

We must increase the section of the beam. We take a landing beam of section 35*40) therefore,

$\tau^{tor} = 3.067MPa > \overline{\tau_u} = 3.33MPa$

The resulting bending constraints and torsion,

$\tau = \sqrt{\tau_{sb}^2 + \tau_{tor}^2} = 3.124MPa$

The new reinforcement section is shown in the following table,

TABLE III 44: GLOBAL REINFORCEMENT IN LANDING BEAM.

Location	$A_a^{SB}(cm^2)$	$A_l^{SB}(cm^2)$	$A_l^{tor}(cm^2)$	$A(cm^2)$	$A_{min}^{RPA}(cm^2)$	$A_{adopt}(cm^2)$
Span	/	2.90	4.68	7.58	5.25	5HA14
Support	3.00	/	4.68	7.68		

- **Transversal reinforcement**

$U: 2 * \left((b-e)+(h-e)\right) = 1.232m$

$\Omega: (\mathrm{b}-\mathrm{e})-(\mathrm{h}-\mathrm{e}) = 0.094\mathrm{m}^2$

$A_l^{tor} = \frac{M_{tu}*U}{2*f_{st}*\Omega} = 7.28\text{cm}^2$

$A_{trans} = A_{trans}^{SB} + A_{trans}^{tor} = 7.73\text{cm}^2$

Therefore, we take $4HA16 = 8.05\text{cm}^2$

- **SLS verification**

The results of constraints verification are shown in the following table:

TABLE III 45: CONSTRAINTS VERIFICATION

Location	M (KN.m)	Y (cm)	I (cm^4)	σ_{bc}	$\bar{\sigma}_{bc}$	OBS
Span	9.81	16.87	161225.70	1.41	15	Verified
Support	-26.96	16.87	161225.70	2.79	15	//

Landing beam reinforcement diagram (Figure III.14)

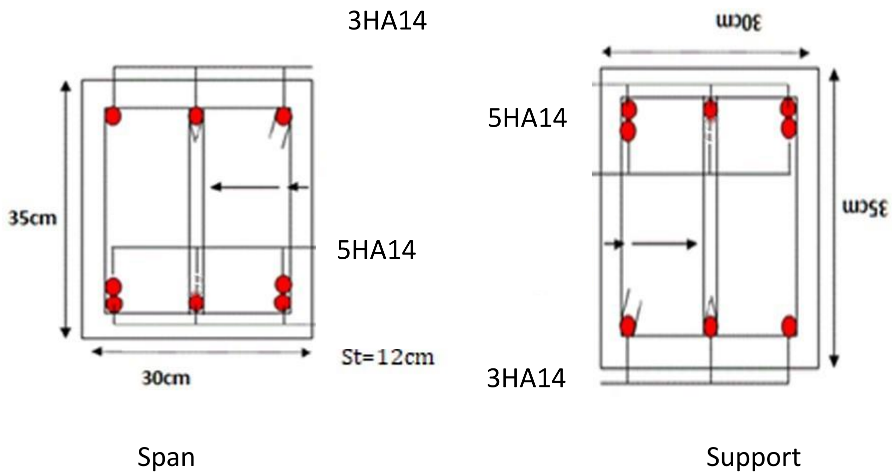


FIGURE III- 14: LANDING BEAM REINFORCEMENT DIAGRAM

III.6 Chain beam study

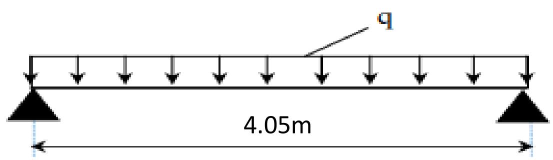


FIGURE III- 15: CHAIN BEAM DIAGRAM

According to RPA99(Art9.3.3), the minimum dimension of chain beam must be greater or equal to 15cm or $\frac{2}{3}$ $of\ the\ support\ element$.The maximum span of the chain beam is:

$L_{max} = 4.35 - 0.3 = 4.05m.$

- **Dimensioning:**

According to stiffness condition,

$\frac{L}{15} \leq h \leq \frac{L}{10} \quad\quad \Rightarrow 30cm \ \leq h \ \leq \ 40.5cm \Rightarrow \quad h = 30cm \ and \ b = 30cm$

- **Verification of the conditions of RPA version 2003.**

$b = 30cm \geq 20cm \Rightarrow verified$: and $h = 35cm \geq 30cm \Rightarrow verified$

$\frac{h}{b} = \frac{30}{30} = 1 < 4 \Rightarrow verified$

- **Calculation of the loads**

The maximum loads which were taken directly from ETABS are shown in the table below:

TABLE III 46: MAXIMUM LOADS IN CHAIN BEAM.

ULS		SLS		V_u(KN)
M_{Span}(KN.m)	M_{Suport}(KN.m)	M_{Span}(KN.m)	$M_{Support}$(KN.m)	
13.495	-19.35	9.70	-13.934	20.66

- **Longitudinal reinforcement**

The results of longitudinal reinforcement in the span and at the supports are summarized in the following table,

TABLE III 47: REINFORCEMENT CALCULATION AT ULS OF THE LANDING BEAM.

Location	$M_u(kN.m)$	μ_{bu}	$\propto$	$z(cm)$	$A_{cal}(cm^2)$	$A_{min}(cm^2)$	$A_{adopt}(cm^2)$
Span	13.495	0.0404	0.0516	0.2742	1.41	1.014	3HA10=2.36
Support	19.35	0.0579	0.0747	0.2716	2.05	1.014	3HA10=2.36

- **Necessary verifications at ULS**
- **Shear force verification**

The same calculations were done as in the landing beam, we found that,
$\tau_u = 0.246 \leq \overline{\tau_u}\ 3.33\text{MPa}$ therefore no risk of shear failure

- **Transverse reinforcement**
  As calculated for the reinforcement beam, we choose Ties and stirrups **φ 8** spaced at ***25cm***.

Therefore, we opt for spacing of $S_t = 25cm < 29.7cm$

- **SLS verification**

The results of constraints verification are shown in the following table:

TABLE III 48: CONSTRAINTS VERIFICATION.

Location	M (KN.m)	Y (cm)	I (cm^4)	σ_{bc}	$\bar{\sigma}_{bc}$	OBS
Span	9.70	7.034	19041.113	3.584	15	Verified
Support	13.934			5.147	15	//

The chain beam reinforcement diagram is shown in (Figure III.16)

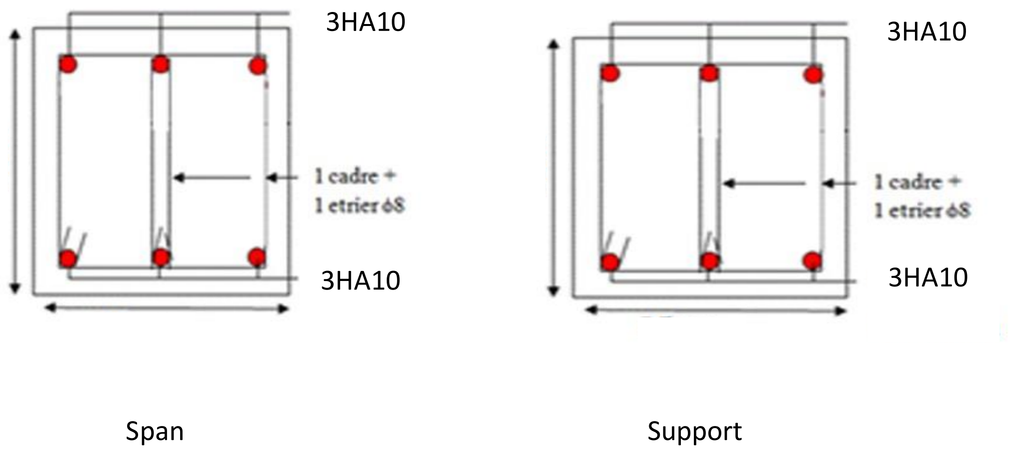


FIGURE III- 16: CHAIN BEAM REINFORCEMENT DIAGRAM

III.7 Cranked beam

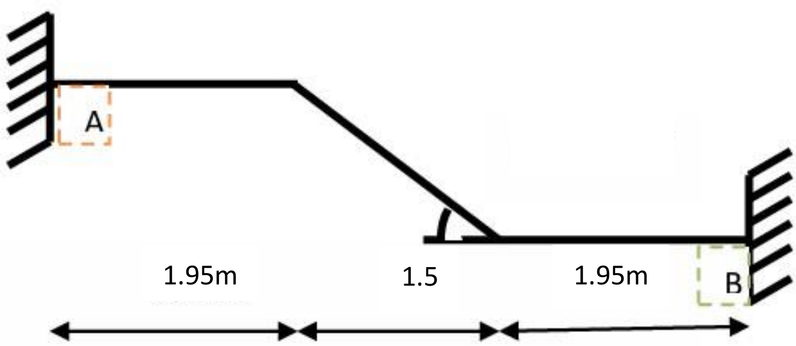


FIGURE III- 17: CRANKED BEAM DIAGRAM

$\frac{L}{15} \leq h \leq \frac{L}{10}$ $\Rightarrow 38.07cm \leq h \leq 5.71m \Rightarrow$ $h = 40cm \; and \; b = 30cm$

As done with the chain beam, all the RPA conditions are verified.

- **Calculation of the loads**

The maximum loads which were taken directly from ETABS are shown in the table below,

TABLE III 49: MAXIMUM LOADS IN CRANKED BEAM.

ULS	SLS	V_u(kN)

M_{Span}(kN.m)	M_{Suport}(kN.m)	M_{Span}(kN.m)	$M_{Support}$(kN.m)	
19.59	-44.04	13.98	-31.63	44.602

- **Longitudinal reinforcement**

The results of longitudinal reinforcement in the span and at the supports are summarized in the following table,

TABLE III 50: REINFORCEMENT CALCULATION AT ULS OF THE CRANKED BEAM.

Location	$M_u(kN.m)$	μ_{bu}	$\propto$	$z(cm)$	$A_{cal}(cm^2)$	$A_{min}(cm^2)$
Span	19.59	0.0318	0.0405	0.3738	1.51	1.38
Support	44.04	0.0716	0.0929	0.3659	3.36	

- **Necessary verifications at ULS**
- **Shear force verification**

As calculated for the chain beam, we found that, $\tau_u = 0.45 \leq \overline{\tau_u}\ 1.17\text{MPa}$ therefore no risk of shear failure

- **Torsion calculation**

The calculations are done in a similar manner to the landing beam. The results are summarized in the following table,

TABLE III 51: TORSION CALCULATION

M_{tu}(kN.m)	e(cm)	$U\ (m)$	$\boldsymbol{\Omega}\ (m^2)$	$A_l^{tor}(cm^2)$	τ^{tor}(MPa)	$A_s(cm^2)$		Choice(cm^2)	
						Span	Support	Span	Support
13.98	5.83	1.10	7.05	3.14	170	3.08	5.03	3HA14=3.39	5HA12=5.65

- **Transversal reinforcement**

The calculations are done in a similar manner to the landing beam. The results are summarized in the following table,

TABLE III 52: TRANSVERSAL REINFORCEMENT

M_{tu}(kN.m)	$U\ (m)$	$\boldsymbol{\Omega}\ (m^2)$	$s_t(cm)$	$A_l^{tor}(cm^2)$	$A_l^{SB}(cm^2)$	$A_{total}(cm^2)$
13.98	1.15	7.05	25	0.71	0.75	1.46

We choose a framework and a stirrup of $4HA8 = 2.01cm^2$.

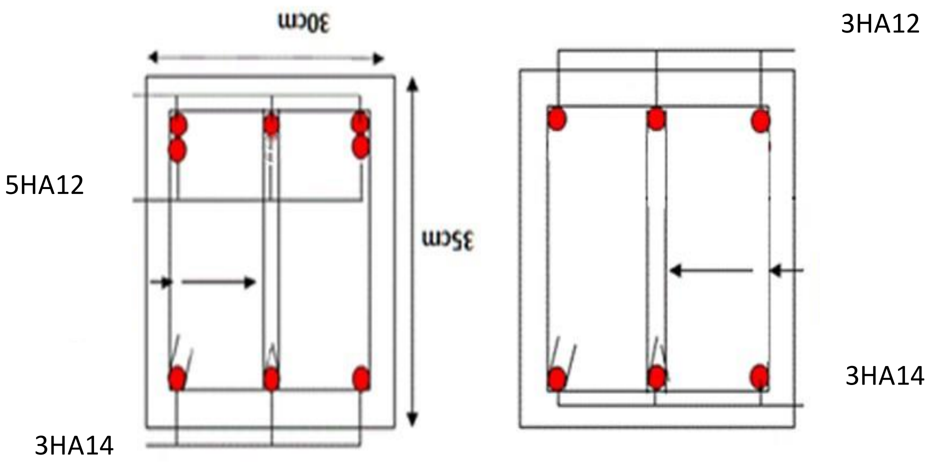
- **SLS verification**

The results of constraints verification are shown in the following table,

TABLE III 53: CONSTRAINTS VERIFICATION

Location	M (KN.m)	Y (cm)	I (cm^4)	σ_{bc}	$\bar{\sigma}_{bc}$	OBS
Span	13.98	9.02	36579.51	3.45	15	Verified
Support	-31.63	12.10	74566.76	5.13	15	//

The cranked beam reinforcement diagram is shown in figure III-18:



Support *Span*

Figure III- 18**:** Cranked beam reinforcement diagram

Chapter IV

Dynamic study

IV.1Introduction

Among the natural disasters that affect the earth's surface, seismic tremors are undoubtedly those that have the most destructive effects in urbanized areas.

Faced with this risk and the impossibility of predicting it, it is necessary to build structures that can withstand such phenomena in order to ensure minimally acceptable protection. Generally, a dynamic analysis of earthquake-resistant constructions is used.

IV.2 Modeling

To determine forces in structural members under vertical and horizontal loads, the structure was modeled in ETABS v16 – a finite element method (FEM)-based software. ETABS generates mass, stiffness, and force matrices at each structural node, then assembles and solves these systems, resolving thousands of matrices within the methodological sequence initiated with structural geometry definition using grid systems and material specification (concrete and steel). Subsequent steps included:

- Element modeling: Frame elements (columns/beams), shell elements (floors/stairs), and wall elements (all designated for meshing)
- Loading: Application of pre-evaluated loads from Chapter II, with supplemental earth pressure on retaining walls
- Boundary conditions: Full fixation of foundation-level nodes
- Diaphragm constraint: Implementation of rigid diaphragms at each floor level to reduce degrees of freedom and enforce uniform horizontal displacements
- Dynamic analysis: Final integration of the RPA2003 response spectrum.

IV.3 Analysis methods

The seismic loads calculation can be performed according to three methods:

- the equivalent static method;
- the modal response spectrum analysis method;
- the time history dynamic analysis method.

IV.3.1 The equivalent static method (ESM)

ESM is a simplified method used to analyze the behavior of structures under seismic loading conditions. It involves calculating the equivalent static force that would produce the same maximum response as the dynamic loads that the structure is expected to experience during an earthquake. This method assumes that the seismic load can be approximated by a single static force that acts on the structure along a particular direction. The use of this method requires the verification of some conditions defined by the Algerian earthquake resistant regulations RPA 2003 (regularity in plan, regularity in elevation, height...).

IV.3.2 Dynamic methods

a. The modal response spectrum analysis method can be used in all cases, and in particular, in the case where the equivalent static method is not permitted.

b. The time history dynamic analysis method can be used in specific cases by qualified personnel which must justify the choice of the seismic inputs (accelerograms) to be used, the behavior relationships of materials, the method of results interpretation and safety criteria to comply with

In our case, the plan regularity condition is not satisfied and the height of our structure (zone IIa, usage group 2) is more than 23 m, so the equivalent static method is inapplicable (RPA99 Version 2003 Art 4.1.2).

According to the RPA, the value of base shear Vdyn, obtained by combination of the modal values, should verify the following condition,

$\mathrm{Vdyn} \geq 0.8\,\mathrm{Vst}$

Where Vst is the resultant of the seismic forces determined by the equivalent static method.

In the case where the condition is not verified, all the responses obtained from the dynamic method must be increased by $\frac{\mathbf{0.8\,Vst}}{\mathbf{Vdyn}}$

IV.4 Calculation of the total seismic load using the equivalent static method

The seismic force applied at the base must be calculated in both X and Y direction using the following formula,

$Vst = \frac{A*D*Q*W}{R}$ (Article 4.2.3 RPA99 v2003)

Such that,

A: Zone acceleration coefficient

D: Average dynamic amplification factor

Q: Quality factor

W: Total weight of the structure

R: Global behavior coefficient of the structure

The parameters listed above depend on the characteristics of the structure.

- $\begin{cases} Building\ group\ 2\ \text{(moderate importance)} \\ Seismic\ zone\ \text{IIa} \end{cases}$ $\quad \mathrm{A} = 0.15$
- For the bracing system, the building is located in a seismic zone (IIa) with more than 4 levels and exceeding a height of 14 m, so it is necessary to introduce shear walls. We will opt for a dual bracing system composed by walls and frames with justification of frame-wall interaction. In our view, it is the most suitable system for this type of building. So, R=5
- $Q = 1 + \sum_1^6 Pq$ RPA99 Version 2003(formula 4.4)

Pq is the penalty to be applied depending on whether the criteria of quality q “is satisfied or not”. The values to be retained are in the following table;

TABLE IV 1: CALCULATION OF THE PENALTY FACTOR

N	Criteria	XX		YY	
		Observation	Pq	Observation	Pq
1	**Minimum conditions on the bracing lines**	Observed	0	Not observed	0.05
2	**Plan redundancy**	Observed	0	Not observed	0.05
3	**Plan regularity**	Not observed	0.05	Not observed	0.05
4	**Elevation regularity**	Observed	0	Not observed	0.05
5	**Material quality control**	Observed	0	Observed	0
6	**Execution checks**	Observed	0	Observed	0
$1+\sum_{1}^{6} Pq$		1.05		1.20	
N.B: We supposed that material quality control and execution checks were verified					

- $\boldsymbol{W = \sum_{i=1}^{n} Wi}$ $\quad with\ Wi = \mathrm{W_{Gi}} + \beta \mathrm{W_{Qi}}$ RPA99 Version 2003(formula 4.5)

$\mathrm{W_{Gi}}$: Weight due to permanent loads and those of any fixed equipment attached to the structure.

$\mathrm{W_{Qi}}$: Operating loads.

β : Weighing coefficient, it depends on the nature and duration of the operating load.

In our case, $\beta = 0.2$

The total weight of our structure $W = 46014.52\ kN$ (from the model)

- **D**: Average dynamic amplification factor, in relation to the site category, the damping correction factor (η), and the fundamental period of the structure (T).

$$D = \begin{cases} 2.5\eta & 0 \le T \le T_2 \\ 2.5\eta\left(\frac{T_2}{T}\right)^{2/3} & T_2 \le T \le 3.0(s) \\ 2.5\eta\left(\frac{T_2}{3.0}\right)^{2/3}\left(\frac{3.0}{T}\right)^{5/3} & T \ge 3.0(s) \end{cases}$$ RPA99/2003 (Formula 4-2)

With $\eta = \sqrt{\frac{7}{2+\xi}} \ge 0.7$ RPA99 / 2003 (Formula 4.3)

ξ : The percentage of critical damping depends on the constituent material, the type of structure and the amount of filling.

For our case, we have a dense and a mixed system,

$\xi\ = 7\%$ which gives $\eta = 0.88$

T_1, T_2, Characteristic periods associated with the site category.

According to the geotechnical report, we have $soft$ site(S3) $\begin{cases} T_1 = 0.15(s) \\ T_2 = 0.50(s) \end{cases}$

- ***Calculation of the fundamental period of the structure***

The value of the fundamental period (T) of the structure can be estimated from empirical formula or can be calculated by numerical or analytic methods.

The empirical formula recommended is the following:

$$T = min\begin{cases} C_T \times H^{3/4} \\ \frac{0.09H}{\sqrt{L}} \end{cases}$$

$H =\ 38.34m\quad Total\ height\ of\ the\ building$

$C_T\ = 0.05$: Coefficient, function of the bracing system used. RPA99⁄2003 (Table 4.6)

L: Maximum building dimension at its base in the calculation direction considered.

$\begin{cases} L_X\ =\ 27.15\text{m} \\ L_Y\ = 17.8m \end{cases}$

Therefore $\begin{cases} T_X\ = \min(0.770{,}0.662) \\ T_Y\ = min(0.770{,}0.818) \end{cases}$ $\quad hence \begin{cases} T_X\ = 0.662\text{s} \\ T_Y\ = 0.770s \end{cases}$

The static periods increased by 30% are $\begin{cases} 1.3 * T_X\ =\ 1.3 * 0.662 = 0.861\ \text{s} \\ 1{,}3 * T_Y\ = 1.3 * 0.770 = 1.001\ s \end{cases}$

From the model: $\begin{cases} Tx\ numerique = 1.214\ s > 0.861s \\ Ty\ numerique = 0.87s < 1.001s \end{cases}$

For the calculation of **D**, we will use **1.3*T** in X direction and **Ty numerique** in Y direction

In the X direction, $T_2\ = 0.5\ \text{s} < T = 0.861s < 3.0s$ therefore $Dx = 2.5\eta\left(\frac{T_2}{T}\right)^{2/3}\ = 1.53$

In the y direction, $T_2\ = 0.5\ \text{s} < T = 0.87s < 3.0s$ therefore $Dy = 2.5\eta\left(\frac{T_2}{T}\right)^{2/3}\ = 1.52$

Therefore, the total static force at the base of the structure is, $\begin{cases} Vstx = 2217.7kN \\ Vsty = 2517.9kN \end{cases}$

IV.5 Modal Response Spectrum Analysis (RSA)

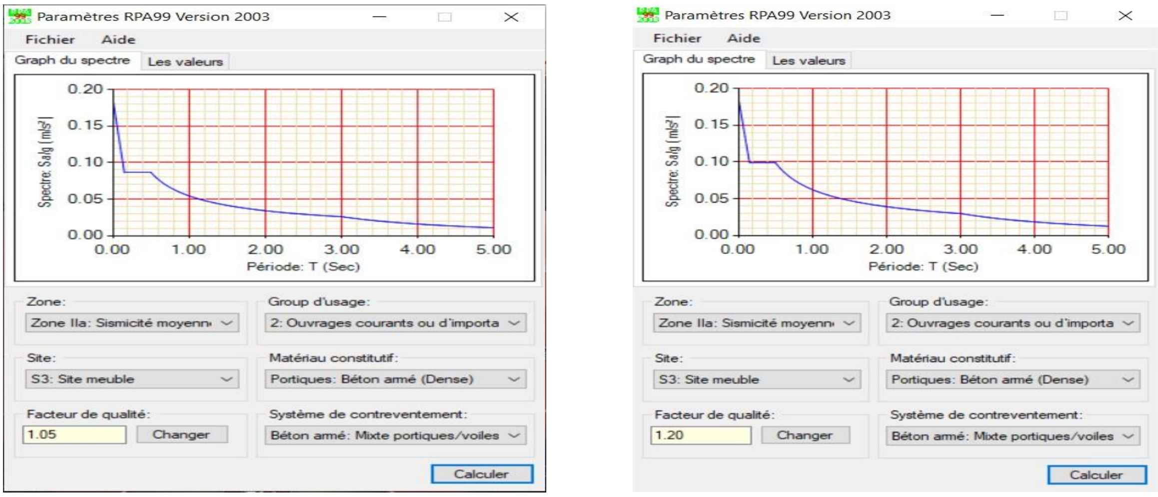
The spectral modal method is undoubtedly the most widely used method for the seismic analysis of the structures. This method seeks for each vibration mode, the maximum effects generated in the structure by the seismic forces represented by a design response spectrum. These effects will then be combined in the most appropriate way to obtain the total response of the structure.

The response spectrum is a curve representing the maximum acceleration induced by seismic vibrations as a function of the natural period of a simple harmonic oscillator (pendulum, string, etc.)

and its critical damping. The spectrum used is that of RPA99⁄2003 (figure IV.1) where the seismic action is represented by the following Design Response Spectrum:

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

RPA99⁄2003 (Formula 4-13)



-a- -b-

FIGURE IV- 1: RPA SPECTRUM

-a- In the x direction

-b- In the y direction

For this method, we used the ETABS, a software used for modelling, analyzing and designing building structures. It helps in 3D modelling, dynamic and static analysis as well as designing structures according to international codes.

IV.5.1 Modal analysis results

Bracing ensures horizontal and vertical stability of the structure during shaking, which has components in all three directions.

The building is located in a seismic zone (IIa) with more than 4 levels and exceeding a height of 14 m, so it is necessary to introduce shear walls. We will opt for a mixed bracing system. In our view, it is the most suitable system for this type of building.

It should be noted here that the column dimensions have been increased. The following dimensions were adopted in the model:

TABLE IV 2: NEW COLUMN DIMENSIONS

Level	Basement	Ground floor and loft	1st floor	2nd and 3rd floor	4th and 5th floor	6th and 7th floor	8th floor	Terrace
Section	55*60	50*60	45*55	40*50	35*45	30*40	30*30	25*25

➢ Arrangement and length of the shear walls

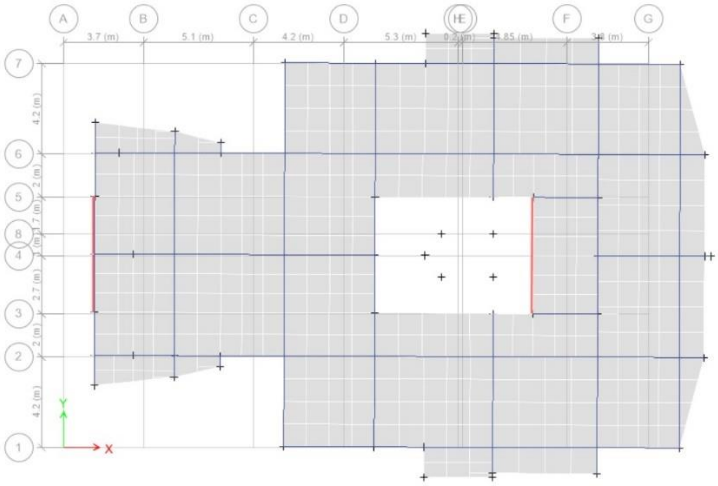
To limit the risk of structural torsion and ensure optimal performance, several arrangements were tested with the aim of achieving a satisfactory mixed bracing system, both good load distribution between frames and compliance with the architectural constraints of the structure.

To do this, we choose the arrangement shown in figure IV.2

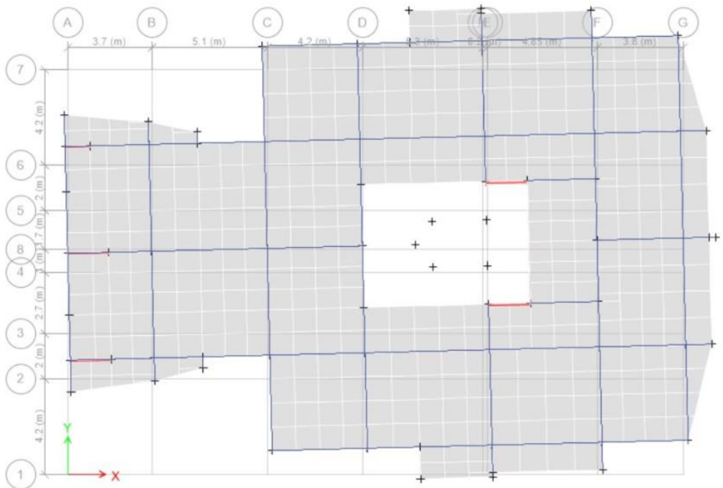
$vx1 = vx2 = 1.80m$ $\quad vx3 = 1.10m$ $\quad vx4 = vx5 = 1.85m$ $\quad vy1 = vy2 = 5.40m$

FIGURE IV- 2: DISPOSITION OF SHEAR WALLS

➢ Vibration modes of the building



-a-



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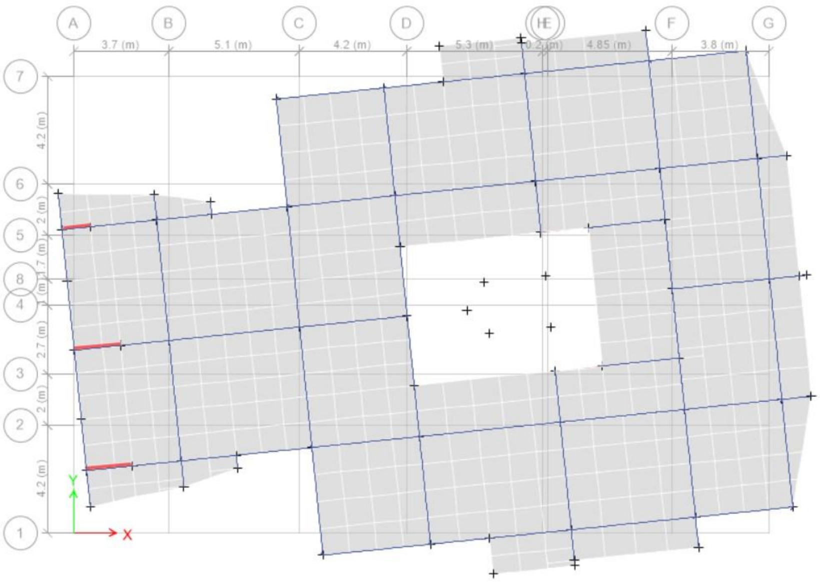


-c-

FIGURE IV- 3: VIBRATING MODE DIAGRAMS

-a- First mode, Translation in X direction

-b- Second mode, Translation in Y direction

-c- Third mode, Rotation in the Z direction

Comments:

Modal analysis indicates predominant X-direction translation in Mode 1 and Y-direction translation in Mode 2 while in the third mode exhibit a torsional rotation. These results are satisfactory due to the non-symmetric form of our building.

➢ Vibration Periods and Mass Participation

The mass participation coefficient corresponds to the i^{eme}vibration mode and represents the percentage of the seismic energy absorbed in this mode by the building. The sum of these coefficients represents the total amount absorbed by the building.

For structures represented by the plane model in two orthogonal directions, the number of modes to be retained must be such that the sum of the total effective masses for the retained modes is equal to at least 90%of the total mass of the structure. Table IV.3 present the period and the mass participating ratio of the building.

TABLE IV 3: PERIOD AND MASS PARTICIPATION RATE OF THE STRUCTURE

Mode	Period	UX	UY	SUM UX	SUM UY
1	**1.214**	**0.7157**	0.0001	0.7157	0.0001
2	**0.87**	2.051E-05	**0.6624**	0.7157	0.6625
3	0.809	0.0004	0.0274	0.7162	0.6899
4	0.399	0.1177	5.738E-07	0.8339	0.6899
5	0.322	0	0.0466	0.8339	0.7365
6	0.288	0.0381	0	0.872	0.7365
7	0.276	0.0002	0.0009	0.8721	0.7374
8	0.217	0	0.1419	0.8721	0.8793
9	0.21	4.45E-05	0.0054	0.8722	0.8847
10	0.17	0.0503	0	**0.9224**	0.8847
11	0.108	3.302E-06	0.0387	0.9224	**0.9234**
12	0.101	0.027	1.247E-05	0.9495	0.9235

Comments:

From the results obtained in the table above, we can clearly see that the mass participation rate along the X axis reaches 90% at the 10th mode and along the Y axis at the 11th mode.

IV.5.2 Verification of RPA requirements

a) Verification of the shear force at the base

$$Vdyn \geq 0.8Vst$$

The results are shown in the following table,

TABLE IV 4: SHEAR FORCE VERIFICATION AT THE BASE.

Seismic force at the base	V_{dyn}	$0.8V_{st}$	Observation	$\frac{0.8Vst}{Vdyn}$
X-X	1367.12	1774.14	Not Verified	**1.29**
Y-Y	1813.92	2014.33	Not Verified	**1.11**

The condition is not verified in the two directions; all the response parameters will be increased by $\frac{0.8Vst}{Vdyn}$.

b) Justification of the wall-frame interaction

The RPA99/2003 Art 3.4 requires the following verifications for mixed systems:

- **Under vertical loads**

$$\frac{\sum Fframes}{\sum Fframes+\sum Fsails} \geq 80\% \text{ (Percentage of vertical loads absorbed by the frames)}$$

$$\frac{\sum Fsails}{\sum Fframes+\sum Fsails} \leq 20\% \text{ (Percentage of vertical loads absorbed by the shear walls)}$$

- **Under horizontal loads**

$$\frac{\sum Fframes}{\sum Fframes+\sum Fsails} \geq 25\% \text{ (Percentage of horizontal loads absorbed by the frames)}$$

$$\frac{\sum Fframes}{\sum Fframes+\sum Fsails} \leq 75\% \text{ (Percentage of horizontal loads absorbed by the shear walls)}$$

The results obtained are presented in the following tables:

TABLE IV 5: VERIFICATION OF VERTICAL INTERACTION ON THE BASEMENT

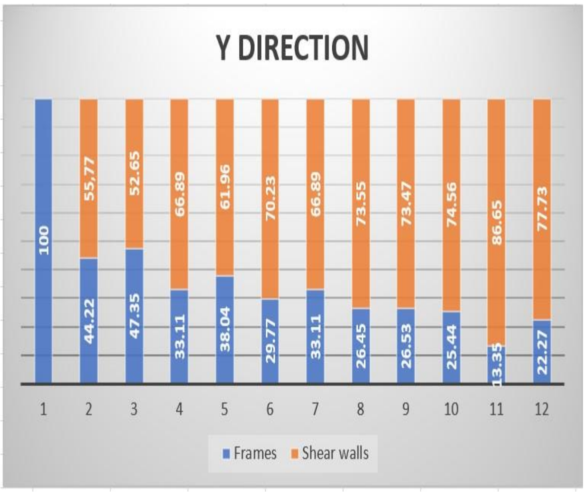
Level	Loads absorbed in (kN)		Percentage absorbed (%)		Observation
	Frames	Shear walls	Frames	Shear walls	
Basement	44796.64	10327.62	81.26	18.64	Verified

TABLE IV 6: VERIFICATION OF THE HORIZONTAL INTERACTION IN THE X-X DIRECTION

Level	Loads absorbed in (kN)	
	Frames	Shear walls
Terrasse	203.6515	
8th floor	315.1256	229.472
7th floor	380.5633	225.5322
6th floor	368.891	377.1255
5th floor	565.5806	346.7135
4th floor	509.6174	531.2736
3rd floor	694.0622	477.0633
2nd floor	613.9757	675.0337
1st floor	702.4282	693.4438
Loft	783.8809	705.4188
Ground floor	458.356	1135.4426
Base	505.8872	1259.7988

TABLE IV 7: VERIFICATION OF THE HORIZONTAL INTERACTION IN THE Y-Y DIRECTION

Level	Loads absorbed in (kN)	
	Frames	Shear walls
Terrasse	201.634	
8th floor	234.4736	295.7275
7th floor	390.8264	434.5466
6th floor	348.935	704.8811
5th floor	475.7962	774.8652
4th floor	420.7378	992.6439
3rd floor	516.1623	1042.5483
2nd floor	444.1361	1234.7177
1st floor	473.3675	1310.974
Loft	476.8515	1397.9033
Ground floor	256.5348	1665.6133
Base	453.4277	1582.5582



Comments:

The results in the table above show that the interactions are verified at all levels in the X direction. In the Y direction, the interaction is not verified only on the basement and the ground floor. The results are therefore acceptable.

c) Verification of inter-storey displacement

According to RPA99/2003 Art 5.10, the relative displacement between two adjacent stories (inter storey drift) must not exceed 1.0% of the storey height.

The relative displacement at level k in relation to level K-1 is equal to:

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

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

$\Delta k \leq 1\% \, he$

δ_k: Horizontal displacement at each level K :

δ_{ek}: Elastic displacement at level K

R: $Dynamic\ behavior\ coefficient\ (R = 5)$

The results are summarized in the following table,

TABLE IV 8: VERIFICATION OF INTER-STOREY DISPLACEMENTS

Level	h_k (m)	$X\ direction$			$Y\ direction$		
		Δ_e (m)	Δk(mm)	$1\%h_k(mm)$	Δ_e(m)	Δk(mm)	$1\%h_k(mm)$
Terrasse	3.06	0.001523	7.61	30.6	0.001714	8.57	30.6
8th floor	3.06	0.000831	4.155	30.6	0.000771	3.86	30.6
7th floor	3.06	0.000863	4.315	30.6	0.000783	3.92	30.6
6th floor	3.06	0.000913	4.565	30.6	0.000807	4.04	30.6
5th floor	3.06	0.000944	4.72	30.6	0.000802	4.01	30.6
4th floor	3.06	0.000975	4.875	30.6	0.000798	3.99	30.6
3rd floor	3.06	0.000975	4.875	30.6	0.000762	3.81	30.6
2nd floor	3.06	0.000963	4.815	30.6	0.00072	3.60	30.6
1st floor	3.06	0.000904	4.52	30.6	0.000647	3.24	30.6
Loft	2.89	0.00081	4.05	28.9	0.000559	2.80	28.9
Ground floor	3.74	0.000644	3.22	37,4	0.00045	2.25	37,4
Base	3.57	0.000289	1.445	35.7	0.000245	1.23	35.7

Comments:

The inter-storey displacements are less than one hundredth of the storey height in both directions. the structure is stable and has no instability risk.

d) Justification with respect to the $P - \Delta$ effect

Second order effects are the effects due to vertical loads after displacement. They can be neglected in the case of buildings if the following condition is satisfied at all levels:

$$\theta = \frac{P_k * \Delta k}{V_k * H_k} \leq 0.1 \quad RPA99/2003 \text{ Art } 5.1.9$$

P_K: Total weight of the structure and associated live loads above the level (k)

$$P_k = \sum_{i=k}^{n}(W_{Gi} + \beta W_{Gi})$$

$V_k = \sum_{i=k}^{n} F_i$: Storey shear force

Δk: Relative displacement of level k with respect to level k-1

h_k: Storey height

If $0.1 < \theta_k < 0{,}2$, the $P - \Delta$ effects can be taken into account approximately by amplifying the effects of seismic action by the factor $\frac{1}{1-\theta_k}$.

If $\theta_k \geq 0.2$, the structure is potentially unstable and must be redesigned.

The results are summarized in the following table IV.9:

TABLE IV 9: VERIFICATION OF SECOND-ORDER EFFECTS

Level	h_k (m)	P_k(kN)	X-X Direction			Y-Y Direction		
			Δk (mm)	V_k(kN)	θ_k	Δk (mm)	V_k(kN)	θ_k
Terrasse	3.06	1305.3646	7.61	203.6515	0.016	8.725	201.634	0.018
8th floor	3.06	5256.4811	4.155	480.0075	0.015	3.915	502.3814	0.013
7th floor	3.06	9073.7899	4.315	708.9497	0.018	3.98	803.8031	0.014
6th floor	3.06	12917.5128	4.565	901.1674	0.021	4.105	1054.7329	0.016
5th floor	3.06	16846.5388	4.72	1075.4271	0.024	4.08	1259.7841	0.018
4th floor	3.06	20775.5648	4.875	1227.6902	0.027	4.055	1434.3883	0.019
3rd floor	3.06	24802.1339	4.875	1363.5863	0.029	3.87	1584.553	0.019
2nd floor	3.06	28828.703	4.815	1487.723	0.030	3.66	1712.7359	0.020
1st floor	3.06	32965.0553	4.52	1590.0122	0.031	3.29	1821.4318	0.019
Loft	2.89	37163.6165	4.05	1670.9634	0.031	2.845	1913.0176	0.019
Ground floor	3.74	41577.7919	3.22	1735.9627	0.021	2.295	1983.1468	0.013
Basement	3.57	46014.5156	1.445	1763.5905	0.011	1.25	2013.4531	0.008

The calculation results indicate that the second-order effects are less than 0.1, they may then be neglected.

e) Verification of the reduced axial force in columns

To avoid (or limit) the risk of brittle fracture under seismic actions, the reduced axial force in the column must comply with the following condition: $\vartheta \leq 0.30 \quad with\ \vartheta = \frac{N}{B * fc28}$ RPA99/2003 Art IV.4.3.1

N: *Design normal force acting on a concrete section under seismic combinations.*

B: *The column area (gross section)*

f_{cj} *The characteristic strength of the concrete*

The results are given in table IV.10:

TABLE IV 10: VERIFICATION OF THE REDUCED AXIAL FORCE FOR EACH FLOOR

Level	Section	B (cm^2)	Combination	N (kN)	ϑ	Observation
Basement	55*60	3300	G+Q+EY Min	2338.302	0.283	No risk of brittle fracture
Ground floor	50*60	3000	G+Q+EY Min	2079.414	0.277	//
Loft	50*60	3000	G+Q+EY Min	1817.825	0.242	//
1st Floor	45*55	2475	G+Q+EY Min	1611.825	0.260	//
2nd Floor	40*50	2000	G+Q+EY Min	1408.669	0.282	//
3rd Floor	40*50	2000	G+Q+EY Min	1212.147	0.242	//
4th Floor	35*45	1575	G+Q+EY Min	1019.826	0,259	//
5th Floor	35*45	1575	G+Q+EY Min	839.938	0.213	//
6th Floor	30*40	1200	G+Q+EY Min	663.973	0.221	//
7th Floor	30*40	1200	G+Q+EY Min	500.113	0.167	//
8th Floor	30*30	900	G+Q+EY Min	368.967	0.164	//
Terrasse	25*25	625	G+Q+EY Min	260.472	0.167	//

f) Overturning verification

To avoid the risk of the building overturning:

$$\frac{M_s}{M_r} \geq 1.5$$

M_s: The stabilizing moment, equal to: $Mx_s = W * Yccm\ and : My_s = W * Xccm$

Yccm, Xccm: The cumulative center of mass coordinates.

M_r: Overtuning moment is equal to: $= \sum fi \times hi$

It can be taken directly from ETABS under seismic actions.

Table IV.11 gives the verification results:

TABLE IV 11: THE OVERTURNING VERIFICATION

$X\ direction$					
W	$Yccm$	M_s	M_r	M_s/M_r	Observation
46014.515	8.8892	409032.23	42873.764	9.54	No risk of overturning
$Y\ direction$					
W	$Xccm$	M_s	M_r	M_s/M_r	Observation
46014.515	15.2472	701592.51	49987.27	14.04	No risk of overturning

Conclusion

After several tests on the arrangement of the shear walls and on increasing the dimensions of the structural elements, and by balancing the resistance and economic criteria's, we were able to satisfy all the conditions, which allows us to keep our model and move on to the calculation of the structural elements.

The final dimensions of the structural elements are shown in the following table:

TABLE IV 12: FINAL DIMENSIONS OF STRUCTURAL ELEMENTS

<table>
<tr><th>Level</th><th>Basement</th><th>Ground floor</th><th>Loft</th><th>1st floor</th><th>2nd and 3rd floors</th><th>4th and 5th floors</th><th>6th and 7th floor</th><th>8th floor</th><th>terrasse</th></tr>
<tr><td>Columns</td><td>55*60</td><td>50*60</td><td>50*60</td><td>45*55</td><td>40*50</td><td>35*45</td><td>30*40</td><td>30*30</td><td>25*25</td></tr>
<tr><td>Shear walls</td><td colspan="2">19cm</td><td colspan="6">16cm</td><td>/</td></tr>
<tr><td>Principal beams</td><td colspan="8">30*35</td><td>30*40</td></tr>
<tr><td>Secondary beams</td><td colspan="9">30*30</td></tr>
</table>

Chapter V

Study of principal elements

Introduction

Columns and shear walls are subjected to normal forces, bending moment and shear forces. They will therefore be calculated in compound bending. Beams, on the other hand, are subjected to bending moments and shear forces, and are therefore calculated in simple bending.

V.1 Column analysis

The columns are calculated under the action of the most unfavorable stresses resulting from the combination actions given by the CBA 93 and the RPA99/2003.

$1.35G + 1.5Q$ ULS

$G + Q \pm E$ $AULS$

$0.8G \pm E$ $AULS$

$G + Q$ SLS

The reinforcement adopted will be the maximum between those given by the following stresses,

N^{max}……………..$M^{Corresponding}$
N^{min}……………...$M^{Corresponding}$
M^{max}……………$N^{Corresponding}$

V.1.1 RPA99/2003 Recommendations

a) Longitudinal reinforcement

Longitudinal reinforcement must be high-bond, straight, and without hooks. Their percentage is limited by:

- ✓ $A_{min} = 0.8\%\ of\ the\ concrete\ section\ in\ a\ medium\ seismicity\ zone$
- ✓ $A_{min} = 4\%\ of\ the\ concrete\ section (in\ the\ standard\ zone)$
- ✓ $A_{min} = 6\%\ of\ the\ concrete\ section (in\ the\ lapping\ zone)$
- ✓ $\phi_{min} = 12mm (minimum\ diameter\ used\ for\ longitudinal\ bars)$
- ✓ $40\phi = minimum\ overlap\ length\ (l_{min})$
- ✓ $The\ spacing\ between\ two\ vertical\ bars\ in\ a\ column\ face\ must\ not\ exceed\ 25cm$
- ✓ Lap joints should be made, if possible, outside the nodal zones. The nodal zone (figure V.1) is defined by l' and h' such that

$l' = 2h$

$h' = \max\left(\frac{he}{6}; b_1; h_1; 60cm\right)$

$b_1\ and\ h_1$ section of the column considered

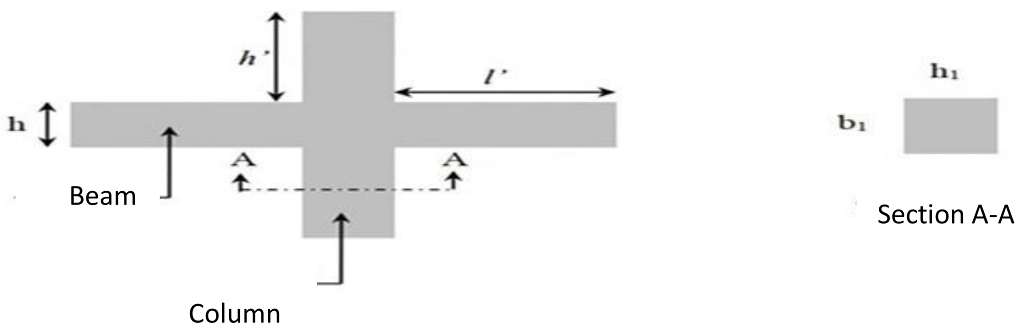


FIGURE V- 1: NODAL ZONE

The limit quantities of longitudinal reinforcement relative to the requirements of RPA99/2003 concerning our project are illustrated in table V.1:

TABLE V 1: MINIMUM LONGITUDINAL REINFORCEMENT IN COLUMNS

	Column section(cm^2)	A_{min}(cm^2)	A_{max}(cm^2)	
			Main zone	lapping zone
Basement	55*60	26.4	132	198
Ground floor and loft	50*60	24.0	120	180
1st floor	45*55	19.8	99	148.5
2nd and 3rd floor	40*50	16.0	80	120
4th and 5th floor	35*45	12.6	63	94.5
6th and 7th floor	30*40	9.6	48	72
8th floor	30*30	8.2	36	54
Terrasse	25*25	5.0	25	37.5

b) Transverse reinforcement

According to the RPA 2003, the cross-section of the transverse reinforcement is given by the following formula:

$$\frac{At}{t} = \rho * \frac{V^{max}}{h1 * fe}$$

V^{max} : Maximum shear force in the column.

ρ : Coefficient that takes into account the brittle failure mode by shear force:

$$\rho \begin{cases} 2.5 \; if \; \lambda \geq 5 \\ 3.75 \; if \; \lambda < 5 \end{cases} \qquad with \quad \lambda_g = (\frac{lf}{a} \; or \; \frac{lf}{b})$$

Such that **a** and **b** are the dimensions of the cross-section of the column in the direction of the deformation considered.

To calculate A, we simply fix the spacing(t) while taking into account the following conditions:

In the nodal zone: $\boldsymbol{t \leq \min\,(15cm; \phi_{lmin})}$

In the standard zone: $\boldsymbol{t \leq 15\phi_{lmin}}$

$$A_t^{min} = \begin{cases} 0.3\%(b * t) \; or \; 0.3\%(h * t) & if \quad \lambda_g \geq 5 \\ 0.8\%(b * t) \; or \; 0.8\%(h * t) & if \quad \lambda_g \leq 3 \end{cases}$$

Ties and stirrups must be secured by $135°$ hooks with a straight length of $10\phi_{lmin}$

V.1.2. Design loads

The design loads resulting from the most unfavorable combinations are taken directly from ETABS V16 software. The results are summarized in the table V.2 and table V.3.

TABLE V 2: DESIGN LOADS FOR THE COLUMNS NOT ASSOCIATED WITH SHEAR WALLS

Level	b*h	N_{max} (kN)	M_{corr} (kN.m)	N_{min} (kN)	M_{corr} (kN.m)	M_{max} (kN.m)	N_{corr} (kN)	V (kN)
Basement	55*60	3134.243	9.546	149.587	5.948	116.022	773.706	134.524
		ULS		$0.8G + Ex^{max}$		ULS		ULS
Ground floor and loft	50*60	2749.463	21.842	66.026	24.896	132.15	1439.298	84.099
		ULS		$0.8G + Ex^{max}$		$G + Q + Ex^{min}$		$G + Q + Ex^{min}$
1st floor	45*55	2106.332	12.361	74.760	60.389	98.664	1237.911	68.9
		ULS		$0.8G + Ex^{min}$		$G + Q + Ex^{min}$		//
2nd and 3rd floor	40*50	1854.992	11.366	86.934	52.350	114.226	859.656	77.069
		ULS		$0.8G + Ex^{max}$		$G + Q + Ex^{min}$		//
4th and 5th floor	35*45	1366.715	8.482	100.682	51.529	102.178	541.477	68.948
		ULS		$0.8G + Ex^{min}$		$G + Q + Ex^{min}$		//
6th and 7th floor	30*40	887.412	6.422	55.368	1.887	77.284	278.327	50.823
		ULS		$0.8G + Ex^{max}$		$G + Q + Ex^{min}$		//
8th floor	30*30	477.075	23.316	22.756	18.916	67.601	159.593	47.923
		ULS		$0.8G + Ex^{max}$		$G + Q + Ex^{min}$		//
Terrace	25*25	344.541	9.912	22.150	10.724	40.332	154.593	26.576
		ULS		$0.8G + Ey^{max}$		$G + Q + Ex^{min}$		$G + Q + Ey^{min}$

TABLE V 3: DESIGN LOADS FOR THE COLUMNS ASSOCIATED WITH SHEAR WALLS

Level	b*h	N_{max} (kN)	M_{corr} (kN.m)	N_{min} (kN)	M_{corr} (kN.m)	M_{max} (kN.m)	N_{corr} (kN)	V (kN)
Basement	55*60	2478.126	20.476	430.708	25.390	48.353	237.850	76.745
		$G+Q+Ey^{min}$		$0.8G+Ex^{max}$		$G+Q+Ey^{max}$		ULS
Ground floor and loft	50*60	2151.798	55.486	432.710	9.397	52.151	590.366	69.619
		$G+Q+Ey^{min}$		$0.8G+Ex^{max}$		$G+Q+Ey^{max}$		$G+Q+Ey^{min}$
1st floor	45*55	1432.902	38.586	81.513	36.711	48.031	1139.817	53.677
		$G+Q+Ey^{min}$		$0.8G+Ey^{min}$		$G+Q+Ey^{min}$		$G+Q+Ey^{min}$
2nd and 3rd floor	40*50	1051.578	40.490	6.625	33.572	49.085	815.216	47.391
		$G+Q+Ey^{min}$		$0.8G+Ey^{max}$		$G+Q+Ey^{min}$		$G+Q+Ey^{min}$
4th and 5th floor	35*45	560.368	38.712	18.281	29.271	45.950	402.497	36.578
		$G+Q+Ey^{min}$		$0.8G+Ey^{min}$		$G+Q+Ey^{min}$		$G+Q+Ey^{min}$
6th and 7th floor	30*40	304.008	18.638	556.763	18.444	40.152	110.986	25.402
		$G+Q+Ex^{min}$		$0.8G+Ex^{max}$		$G+Q+Ey^{min}$		$G+Q+Ey^{max}$
8th floor	30*30	76.032	20.71	19.147	1.846	27.742	49.676	19.917
		$G+Q+Ex^{min}$		$0.8G+Ey^{min}$		$G+Q+Ey^{min}$		$G+Q+Ey^{min}$

V.1.3 Reinforcement calculation

- **Longitudinal reinforcement**

Calculation example Basement (55*60) cm²

1st combination ULS: $\boldsymbol{N_{max}}$; $\boldsymbol{M_{corr}}$

Low harmful cracking e=2cm

b=55cm; h=60cm; d=58cm

Common situation $\gamma_b = 1.5 \quad and\ \gamma_s = 1.15$

$$e_G = \frac{M}{N} = \frac{9.546}{3134.243} = 3.04cm < \frac{h}{2} = 30cm$$

Nu(compression) and C inside the section, with the following condition,

$$N_u(d-d') - M_{UA} \leq (0.337H - 0.81d')b * h * f_{bu} \quad with \quad M_{UA} = M_{UG} + N_u(d - \frac{h}{2})$$

$$M_{UA} = 9.546 + 3134.243(0.58 - 0.3) = 887.1kN.m$$

$$N_u(d-d') - M_{UA} = 3134.243(0.58 - 0.02) - 887.1 = 868kN.m$$

$(0.337H - 0.81d')b * h * f_{bu} = (0.337 * 0.6 - 0.81 * 0.02) * 0.55 * 0.6 * 14.2 = 872kN.m$

$868kN.m < 872kN.m \quad Therefore\ the\ section\ is\ partially\ compressed$

$U_{bu} = \frac{M_{UA}}{b*d^2*f_{bu}} = \frac{0.8871}{0.55*0.58^2*14.2} = 0.378 < 0.392 \quad therefore\ A' = 0$

$\propto = 1.25(1 - \sqrt{1 - 2 * U_{bu}} = 0.538$

$Z = d * (1 - 0.4 * \propto) = 0.4552$

$A_1 = \frac{M_{UA}}{Z*fst} = 55.99cm^2$

We return to compound flexion $A = A_1 - \frac{N_u}{fe} = -22.36cm^2$ The section is negative so the concrete does not require reinforcement steel.

$A_{min} = 0.23 * b * d * \frac{f_{t28}}{fe} = 3.85cm^2$

2nd combination

$N_{min} = 149.5876kN \qquad M_{corr} = 5.9483\text{kN.m}$

$e_G = \frac{M}{N} = 0.04m > \frac{h}{2} = 0.3$

N_u (traction) and C inside the section, therefore the section is fully tensioned

$fs10 = \frac{fe}{\gamma s} = 400MPa$

$e_1 = \left(\frac{h}{2} - d'\right) + e_G = 0.32\ m \qquad e_2 = (d - d') - e_1 = 0.24m$

$A_1 = \frac{Nu*e_2}{fs10(d-d')} = 1.60cm^2 \qquad A_2 = \frac{Nu*e_1}{fs10(d-d')} = 2.14cm^2$

$A_{min}^{BAEL} = \frac{B*f_{t28}}{fe} = \frac{55*60*2.1}{400} = 17.33cm^2$

$A_1 + A_2 = 3.74cm^2 < A_{min}^{BAEL} = 17.33cm^2 \qquad We\ reinforce\ with\ \frac{A_{min}^{BAEL}}{2} = 8{,}66cm^2\ /face$

3rd combination

$M_{max} = 116.022kN\ .m \qquad N_{corr} = 773.706\text{kN}$

$e_G = \frac{M}{N} = 0.015m > \frac{h}{2} = 0.3$

Nu(compression) and C inside the section, with the following condition,

$M_{UA} = 0.3327Mn.m$

$N_u(d - d') - M_{UA} = 0.433Mn.m \leq (0.337H - 0.81d')b * h * f_{bu} = 0.872Mn.m$

The section is partially compressed.

$U_{bu} = 0.127$ $\propto = 0.170$ $Z = 0.54m$ $A_1 = \frac{M_{UA}}{Z*fst} = 17.69\ cm^2$

$A = A_1 - \frac{N_u}{fe} = -4.54cm^2$ The section is negative so the concrete alone is enough.

We reinforce with $A_{min} = 3.85cm^2$

Therefore $A_{cal} = \max(A_{1,}A_2, A_3) = 8{,}66cm^2\ /face$

Tables V.4 and V.5 summarize the reinforcement results for columns at different levels:

TABLE V 4: LONGITUDINAL REINFORCEMENT OF COLUMNS NOT ASSOCIATED WITH SHEAR WALLS.

Level	Section	$A_{cal}(cm^2)$	$A_{min}^{BAEL}(cm^2)$	$A_{min}^{RPA}(cm^2)$	$A^{adopted}(cm^2$
Basement	55*60	2.14	8.66	26.4	4HA20+8HA16=28.65
Ground floor and loft	50*60	0.23	3.50	24.0	12HA16=24.13
1st floor	45*55	1.98	2.88	19.8	4HA16+8HA14=20.36
2nd and 3rd floor	40*50	1.67	2.32	16.0	8HA16=16.08
4th and 5th floor	35*45	1.82	1.82	12.6	4HA16+4HA12=14.20
6th and 7th floor	30*40	2.22	1.38	9.6	4HA14+4HA12=10.68
8th floor	30*30	4.92	1.01	8.2	8HA12=9.05
Terrasse	25*25	3.25	0.69	5.0	8HA12=9.05

TABLE V 5: LONGITUDINAL REINFORCEMENT OF COLUMNS ASSOCIATED WITH SHEAR WALLS.

Level	Section	$A_{cal}(cm^2)$	$A_{min}^{BAEL}(cm^2)$	$A_{min}^{RPA}(cm^2)$	$A^{adopted}(cm^2$
Basement	55*60	0.903	3.85	26.4	4HA20+8HA16=28.65
Ground floor	50*60	2.52	3.50	24.0	12HA16=24.13
1st floor	45*55	0.71	2.88	19.8	4HA16+8HA14=20.36
2nd and 3rd	40*50	1.68	2.32	16.0	8HA16=16.08
4th and 5th	35*45	0.97	1.82	12.6	4HA16+4HA12=14.20
6th and 7th	30*40	1.34	1.38	9.6	4HA14+4HA12=10.68
8th floor	30*30	1.94	1.01	8.2	8HA12=9.05

- **Transversal reinforcement**

Calculation example (basement)

$l_f = 0.7 l_0$ $\quad l_0 = 3.74m$ $\quad l_f = 2.618m$

$\lambda_g = \frac{l_f}{a} = 4.76$ $\quad \lambda_g < 5 \quad therefore \quad \rho = 3.75$

In nodal region: $t \leq \min(15cm; \emptyset_{lmin})$ $\quad t \leq 15cm \quad therefore\ t = 15cm$

In main zone: $t \leq \min 15\emptyset_{lmin}$ $\quad t \leq 24cm \quad therefore\ t = 20cm$

$\frac{A_t}{t} = \frac{\rho * V_u}{a * fe}$ $\quad A_t = \frac{3.75*0.1345*0.15}{0.60*400} = 3.15cm^2$

$A_{min} = x\% * t * b$

$3 \leq \lambda_g = 4.76 \leq 5 \quad therefore\ \ A_{min}\ will\ be\ calculated\ by\ interpolation$

$A_{min} = 0.46\% * 55 * 15 = 3.80cm^2$

We take 3 ties of $\emptyset 10 = 6HA10 = 4.71cm^2$

Table V.6 and V.7 summarizes the transversal reinforcement for the different columns at different levels:

TABLE V 6: TRANSVERSAL REINFORCEMENT OF COLUMNS NOT ASSOCIATED WITH SHEAR WALLS

Level	Basement	Ground floor	loft	1st floor	2nd and 3rd floor	4th and 5th floor	6th and 7th floor	8th floor	Terrace
$\emptyset_{lmin}(cm)$	1.6	1.6	1.6	1.4	1.6	1.4	1.2	1.2	1.2
$l_f(cm)$	249.9	261.8	202.3	214.2	214.2	214.2	214.2	214.2	214.2
λ_g	4.165	4.36	2.998	3.89	4.284	4.76	5.355	7.14	8.568
$V(kN)$	134.52	84.099	84.099	68.9	77.07	68.95	50.82	47.92	26.58
t_{nz}	15	15	15	10	15	10	10	10	10
t_{mz}	20	20	20	20	20	20	15	15	15
ρ	3.75	3.75	3.75	3.75	2.5	2.5	2.5	2.5	2.5
$A_t(cm^2)$	0.62	1.96	1.96	1.42	1.80	1.23	1.06	1.00	0.66
A_{tmin}	3.8	3.19	3.19	1.62	1.8	1.05	0.9	0.9	0.75
A_t^{adopt}	6HA10	6HA10	6HA10	6HA8	4HA8	4HA8	4HA8	4HA8	4HA8

TABLE V 7: TRANSVERSAL REINFORCEMENT OF COLUMNS ASSOCIATED WITH SHEAR WALLS.

Level	Basement	Ground floor	loft	1st floor	2nd and 3rd floor	4th and 5th floor	6th and 7th floor	8th floor
$\emptyset_{lmin}$	1.6	1.6	1.6	1.4	1.6	1.4	1.2	1.2
l_f	249.9	261.8	202.3	214.2	214.2	214.2	214.2	214.2
λ_g	3.89	4.36	4.50	3.89	4.28	4.76	5.35	7,14
$V(kN)$	76.75	69.62	69.62	53.68	47.39	36.58	25.40	19.92
t_{nz}	15	15	15	10	15	10	10	10
t_{mz}	20	20	20	20	20	20	15	15
ρ	3.75	3.75	3.75	3.75	2.5	2.5	2.5	2.5
$A_t(cm^2)$	1.11	1.6	1.6	0.9	0.88	0.51	0.4	0.42
A_{tmin}	3.8	3.44	3.19	2.59	2.87	1.26	0.9	0.9
A_t^{adopt}	6HA10	6HA10	6HA10	6HA8	4HA8	4HA8	4HA8	4HA8

V.1.4. Verifications

- **Minimum diameter of the transversal reinforcement**

According to CBA93 Art.7.7.3; $\emptyset_t \geq \frac{\emptyset_{lmax}}{3}$

$12mm \geq \frac{20}{3} = 6.67mm \qquad the\ condition\ is\ verified$

- **Buckling verification**

The columns are subjected to combined bending; CBA93 Art B.8.2.1 requires justification with respect to the ultimate limit state of shape stability. The verified relationship is as follows,

$$Br \geq Brcal = \frac{Nu}{\alpha} * \frac{1}{\frac{fc28}{0.9*\gamma_b} + \frac{fe}{\gamma_s}}$$

With $Br(b-2)*(h-2)$: $reduced\ section\ of\ the\ column$

Buckling verification of the columns will be done in the same way as the calculations set out in chapter 2. The results are summarized in the table V.8:

TABLE V 8: BUCKLING VERIFICATION OF DIFFERENT COLUMNS

Level	Nu(kN)	$i(cm)$	λ	α	$Br(cm^2)$	Br^{cal} (cm^2)	OBS
Basement	3134.24	0.173	14.45	0.725	0.307	0.196	No buckling risk
Ground floor	2749.46	0.173	15.13	0.720	0.278	0.173	//
Loft	2749.46	0.173	11.69	0.747	0.278	0.167	//
1st floor	2106.33	0.158	13.56	0.732	0.227	0.131	//
2nd and 3rd floor	1854.99	0.144	14.87	0.722	0.184	0.116	//
4th and 5th floor	1366.71	0.129	16.60	0.709	0.141	0.087	//
6th and 7th floor	887.41	0.115	18.63	0.694	0.106	0.058	//
8th floor	477.07	0.086	24.73	0.652	0.078	0.033	//
Terrace	344.64	0.072	29.75	0.621	0.052	0.025	//

- **Verification of shear stresses**

According to RPA99/2003 Art 7.4.3.2, the shear stress in concrete must be less than or equal to the ultimate shear stress:

$$\tau_{bu} = \frac{Vu}{b*d} < \tau_{adm} = \rho_d * f_{c28}$$

$$with\ \rho_d = \begin{cases} 0.075\ if \quad \lambda_g \geq 5 \\ 0.040\ if \quad \lambda_g < 5 \end{cases}$$

The results are illustrated in table V.9.

TABLE V 9: VERIFICATION OF SHEAR FORCES

Level	Section	l_f	λ_g	ρ_d	$Vu(kN)$	$\tau_{bu}\ (Mpa)$	τ_{adm}	OBS
Basement	55*60	249.9	4.165	0.04	134.52	0.42	1	no shear risk
Ground	50*60	261.8	4.36	0.04	84.099	0.29	1	//
Loft	50*60	202.3	2.998	0.040	84.099	0.29	1	//
1st and 2nd	45*55	214.2	3.89	0.040	68.9	0.29	1	//
2nd and 3rd	40*50	214.2	4.284	0.040	77.07	0.40	1	//
4th and 5th	35*45	214.2	4.76	0.040	68.95	0.46	1	//
6th and 7th	30*40	214.2	5.355	0.075	50.82	0.45	1.875	//
8th floor	30*30	214.2	7.14	0.075	47.92	0.57	1.875	//
Terrace	25*25	214.2	8.568	0.075	26.58	0.46	1.875	//

- **Verification of stresses at SLS**

Since the cracking is not harmful, the verification will be done for the concrete compressive stress only. This verification will be done for the most stressed column at each level.

$$\begin{cases} \sigma_{bc1} = \frac{N_{ser}}{S} + \frac{M_{serG}}{I_{yy'}} * V \leq \sigma_{adm} \\ \sigma_{bc2} = \frac{N_{ser}}{S} - \frac{M_{serG}}{I_{yy'}} * V' \leq \sigma_{adm} \end{cases}$$

With $S = b * h + 15(A + A')$

$$M_{serG} = M_{ser} - N_{ser}(\frac{h}{2} - V)$$

$$I_{yy'} = \frac{b}{3}(V^3 + V'^3) + 15A'(V - d') + 15A(d - V)^2$$

$$V = \frac{h}{2} \quad and \quad V' = h - v \ (a\ symetrical\ section)$$

Table V.10 summarizes the stress verification results:

TABLE V 10: STRESS VERIFICATION IN CONCRETE

Level	Basement	Ground floor and loft	1st floor	2nd and 3rd floors	4th and 5th floors	6th and 7th floors	8th floor	Terrace
Section	55*60	50*60	45*55	40*50	35*45	30*40	30*30	25*25
d(cm)	58	58	53	48	43	38	28	23
A(cm^2)	28.65	24.13	20.36	16.08	14.20	10.68	9.05	9.05
V(cm)	30	30	27.5	25	22.5	20	15	12.5
V'(cm)	30	30	27.5	25	22.5	20	15	12.5
$I_{yy}(m^4)$	0.0164	0.0118	0.0056	0.00544	0.00355	0.00211	0.00081	0.000475
S(cm^2)	0.3729	0.3361	0.2781	0.224	0.1788	0,1360	0.1035	0.0760
$N_{ser}(kN)$	2263.87	1991.19	1530.88	1348.27	993.63	645.63	349.01	252.37
$M_{ser}(kN)$	6.66	15.32	9.03	8.21	6.13	4.64	16.93	7.16
$M_{serG}(kN)$	6.66	15.32	9.03	8.21	6.13	4.64	16.93	7.16
$\sigma_{bc1}(MPa)$	6.19	7.24	7.12	6.36	7.28	6.95	6.50	5.20
$\sigma_{bc2}(MPa)$	5.95	4.65	3.89	3.34	3.83	2.54	0.23	1.43
$\sigma_{bc}(MPa)$	15	15	15	15	15	15	15	15
OBS	Verified	//	//	//	//	//	//	//

V.2 Beam analysis

The beam study will be conducted with reference to the internal forces extracted from ETABS 2016, taking into account the most unfavorable stresses required by RPA99/2003 Art 5.5,

$1.35G + 1.5Q$ ULS

$G + Q \pm E$ $AULS$

$0.8G \pm E$ $AULS$

$G + Q$ SLS

V.2.1. RPA99/2003 Recommendations

- **Longitudinal reinforcement (art 7.5.2.1):**

The minimum total percentage of longitudinal steels over the entire length of the beam is **0.5%** in any section.

The maximum total percentage of longitudinal steels is:

$A_{max} = 4\%$ in current zone

$A_{max} = 6\%$ in lapping zone

The minimum recovery length is 40φ in zone IIa.

- **Transversal reinforcement (art 7.5.2.2):**

The minimum area of transverse reinforcement is given by the expression:

$$A_t^{min} = 0.003 * \mathrm{St} * \mathrm{b}$$

St: Maximum spacing between the transversal reinforcements given as follows:

$St \leq \min\left(\frac{h}{4}\ 12\varphi_l^{min}\right)$ in nodal zone.

$St \leq \frac{h}{2}$ *outside* the nodal area

V.2.2. load effect and reinforcement design of beams

a) Design loading

The most unfavorable design loadings are shown in the table 11:

TABLE V 11: MOST UNFAVORABLE LOADS IN BEAMS

Level	Principal Beams 1		Secondary Beams		Principal Beam 2	
	M_{span}	$M_{support}$	M_{span}	$M_{support}$	M_{span}	$M_{support}$
Basement	75.00	-103.35	14.79	-31.24	/	/
ground	80.55	-116.34	14.48	-54.01	/	/
Loft	52.50	-126.61	15.83	-65.08	/	/

1^{st} floor	53.80	-135.73	15.77	-72.68	/	/
2^{nd} floor	54.00	-143.19	15.86	-76.94	/	/
3^{rd} floor	54.71	-143.11	15.99	-77.80	/	/
4^{th} floor	56.86	-140.56	16.16	-76.92	/	/
5^{th} floor	57.22	-134.56	16.34	-74.32	/	/
6^{th} floor	59.49	-125.89	16.58	-70.75	/	/
7^{th} floor	59.78	-120.81	15.24	-66.64	/	/
8^{th} floor	77.44	-94.02	15.33	-56.66	/	/
Terrace	/	/	15.67	-36.87	142.57	-108.65

Note:

Principal beam 2 are found on the inaccessible terrace only as shown in chapter 2 page 16.

Since the loads are almost the same in many floors, their reinforcement calculations will be conducted as a single beam in similar floors as shown in table V.14.

- **Longitudinal reinforcement**

It is done in simple bending for a rectangular section

Calculation example,

Principal beams Ground floor (30*35)

- ***Span reinforcement***

$$U_{bu} = \frac{M_U}{b*d^2*f_{bu}} = \frac{0.08055}{0.3*0.33^2*14.2} = 0.174 < 0.392 \quad therefore\ A' = 0$$

$$U_{bu} < 0.186 \implies Pivot\ A$$

$$\propto = 1.25(1 - \sqrt{1 - 2 * U_{bu}} = 0.24$$

$$Z = d * (1 - 0.4 * \propto) = 0.2983$$

$$A = \frac{M_{UA}}{Z*fst} = \frac{0.08055}{0.2983*348} = 7.76cm^2$$

$$A_{min}^{BAEL} = 0.23 * b * d * \frac{ft28}{fe} = 1.2cm^2$$

$$A_{min}^{RPA} = 0.5\% * (b * h) = 5.25cm^2$$

- ***Support reinforcement***

The results are resumed in the table V.12:

TABLE V 12: SUPPORT REINFORCEMENT FOR PRINCIPAL BEAMS ON GROUND FLOOR.

location	$M_a(\mathrm{kN.m})$	U_{bu}	$\propto$	Z (m)	A (cm²)	$A_{min}^{BAEL}(cm^2)$	$A_{min}^{RPA}(cm^2)$
Support	116.34	0.251	0.367	0.2816	11.87	1.2	5.25

The results of the other beams are summarized in the following tables:

TABLE V 13: LONGITUDINAL REINFORCEMENT FOR PRINCIPAL BEAMS IN SPAN.

Type	Levels	M (kN.m)	$A_{cal}(cm^2)$	$A_{min}^{BAEL}(cm^2)$	$A_{min}^{RPA}(cm^2)$	$A_{adop}(cm^2)$
Beam 1 (30*35)	Ground	80.55	7.76	1.2	5.25	3HA14+3HA12=8.01
	Basement	77.44	7.43	//	//	5HA14 =7.70
	Loft to 7th	59.78	5.59	//	//	5HA12 = 5.65
Beam 2 (30*40)	Terrace	142.57	12.45	1.38	6.00	3HA20+2HA14=12.50

TABLE V 14: LONGITUDINAL REINFORCEMENT FOR PRINCIPAL BEAMS IN SUPPORT.

Type	Levels	M (kN.m)	$A_{cal}(cm^2)$	$A_{min}^{BAEL}(cm^2)$	$A_{min}^{RPA}(cm^2)$	$A_{adop}(cm^2)$
Beam 1 (30*35)	Basement and 7th floor	-120.81	10.32	1.2	5.25	3HA16+3HA14=10.65
	Ground floor, loft,1st, 5th and 6th floor	-135.73	11.81	//	//	6HA16=12.06
	2nd ,3rd and 4th floors	-143.19	12.58	//	//	3HA20+3HA12=12.81
	8th floor	-94.02	7.79	//	//	3HA14+3HA12=8.01
Beam 2 (30*40)	Terrace	-108.65	9.11	1.38	6.00	3HA16+2HA14=9.11

TABLE V 15: LONGITUDINAL REINFORCEMENT FOR SECONDARY BEAMS.

Location	Levels	M (kN.m)	$A_{cal}(cm^2)$	$A_{min}^{BAEL}(cm^2)$	$A_{min}^{RPA}(cm^2)$	$A_{adop}(cm^2)$
Support	Basement and terrace	-36.87	3.44	1.01	4.5	4HA12=4.52

	Ground floor and 8th floor	-56.66	5.44	//	//	5HA12=5.65
	Loft,6th and 7th floor	-70.75	6.94	//	//	3HA14+3HA10=6.98
	1st and 5th floor	-74.32	7.33	//	//	3HA16+2HA10=7.60
	2nd ,3rd and 4th floor	-77.80	7.71	//	//	4HA16=8.04
Span	All floors	16.58	1.75	//	//	4HA12=4.52

According to RPA99/2003, the midspan reinforcement of secondary beams must satisfy the following condition:

$A_{span} \geq \frac{A_{support}}{2}$

This condition is verified in our case.

- **Transversal reinforcement**

- $\emptyset_t \quad calculation$

It is given by the following relation,

$\emptyset_t \ \leq \min\left(\emptyset_t^{min}, \frac{h}{35}, \frac{b}{10}\right)$

$\emptyset_t \ \leq \min(12\ , 10\ ,30) \quad \Rightarrow \ \emptyset_t \ \leq 10 \quad \Rightarrow we\ take\ \emptyset_t = 8mm$

We therefore opt for $4HA8 = 2.01cm^2 \quad (a\ stirrup\ and\ a\ tie\ )$

- *Spacing calculation*

According to RPA99/2003(Art 7.5.2.2),

<u>*In nodal region*</u> $s_t \leq min\left(\frac{h}{4}, 12\emptyset_{lmin}\right)$

$$s_t = \begin{cases} 7cm\ for\ secondary\ beams \\ 8cm\ for\ principal\ beam \\ 10cm\ for\ principal\ beam\ 2(roof) \end{cases}$$

<u>*In current zone*</u> $s_t \leq \frac{h}{2}$

$$s_t = \begin{cases} 15cm\ for\ secondary\ beams \\ 17cm\ for\ principal\ beam \\ 20cm\ for\ principal\ beam\ 2(roof) \end{cases}$$

- *Verification of the minimum transversal reinforcement section*

$A_t^{min} = 0.3\% * (t * b) \quad \Rightarrow \quad A_t^{min} = 0.003 * 20 * 30 = 1.8cm^2 \ < 2.01cm^2$

➢ ULS verifications

- ***Shear stress***

$\tau_u = \frac{Vu}{b*d} < \tau_{adm} = \min\left(\frac{0.2*f_{c28}}{\gamma_b}\ ; 5MPa\right) \quad Low\ Harm\ Cracking$

Table V.16 shows the shear verifications:

TABLE V 16: SHEAR VERIFICATION

Beam	Section (cm^2)	$V_{max}(kN)$	$\tau_u\ (MPa)$	$\tau_{adm}\ (MPa)$	OBS
Secondary beam	30*30	31.41	0.374	3.33	Verified
Principal beam 1	30*35	142.57	1.357	3.33	//
Principal beam 2	30*40	130.82	1.090	3.33	//

- ***Shear verifications for longitudinal reinforcement***

For edge supports: $A_l\ \geq\ A_l^{edge} = \frac{V_{max}*\gamma_s}{fe}$

For intermediate supports: $A_l\ \geq\ A_l^{int} = \left(V_{max} - \frac{M_a}{0.9*d}\right) * \frac{\gamma_s}{fe}$

The results are shown in the table V.16,

Table V.16: Shear verifications for longitudinal reinforcement

Beam	$V_{max}(kN)$	$M_a(kN.m)$	$A_l(cm^2)$	$A_l^{edge(cm^2)}$	$A_l^{int}(cm^2)$	OBS
Secondary beam	31.41	77.80	8.04	0.90	-7.94	Verified
Principal beam 1	142.57	143.19	12.81	4.10	-9.76	//
Principal beam 2	130.82	108.65	9.11	3.76	-5.37	//

➢ SLS verifications

- ***Ultimate compressive limit state of concrete verification***

$\sigma_{bc} = \frac{M_{ser}}{S} * Y\ \leq\ \sigma_{adm} = 0.6f_{c28}\ = 15MPa$

The results are shown in the table V.17.

TABLE V 17: ULTIMATE COMPRESSIVE LIMIT STATE VERIFICATION

Beam	Location	$M_{ser}(kN.m)$	$Y(cm)$	$I(cm^4)$	Constraints (MPa)		OBS
					σ_{bc}	σ_{adm}	
Secondary beams	Span	11.84	10.07	37457.32	3.184	15	Verified
	Support	-27.0	12.49	55748.85	6.049	15	//
Principal beam 1	Span	56.73	15.13	95995.71	8.941	15	//
	Support	-82.04	16.14	107921.8	12.268	15	//
Principal beam 2	Span	104.23	16.57	133739.31	12.915	15	//
	Support	-79.62	16.57	133739.31	9.866	15	//

V.3 Nodal zone verification

According to article (7.6.2) of RPA99/2003, the following condition must be verified,

$\lfloor M_n \rfloor + \lfloor M_s \rfloor \ \geq 1.25 * (\lfloor M_w \rfloor + \ \lfloor M_e \rfloor)$

Figure V.2 shows the moments distribution in nodal zones,

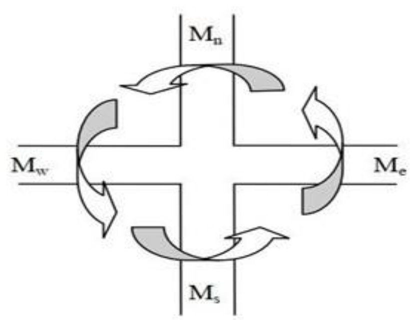


FIGURE V- 2: MOMENTS DISTRIBUTION IN NODAL ZONES

This verification ensures that the plastic joints form in the beams rather in the columns. According to RPA, this verification is optional for the two top levels of buildings above R+2. We will not verify the last two levels.

V.3.1. Determination of the moment resistance in columns and beams

The resistant moments of a concrete section depend on the following parameters;

- Dimensions of the concrete section
- Quantity of reinforcement in the section
- Yield stress in the section

$$M_r = z * A_s * \sigma_s \quad with\, z = \; h - 2 * d' \quad and \quad \sigma_s = \frac{fe}{\gamma_s}$$

The results are illustrated in the tables V.18, V.19, V.20 and V.21:

TABLE V 18: RESISTANT MOMENTS IN COLUMNS

Level	Section (cm^2)	Z(cm)	$A_s(cm^2)$	$\sigma_s\ (MPa)$	$M_r\ (kN.m)$
Basement	55*60	56	10.30	400	230.72
Ground floor and loft	50*60	56	8.04	//	180.10
1st floor	45*55	51	7.10	//	144.84
2nd and 3rd floor	40*50	46	6.03	//	110.95
4th and 5th floor	35*45	41	5.56	//	91.18
6th and 7th floor	30*40	36	4.21	//	60.62
8th floor	30*30	26	4.21	//	43.78

TABLE V 19: RESISTANT MOMENTS IN BEAMS

Level	Beam	Location	h(cm)	Z (cm	$A_s(cm^2)$	$\sigma_s\ (MPa)$	$M_r\ (kN.m)$
Basement	Principal	Span	35	31	7.70	348	83.08
		Support			10.65	348	114.89
	Secondary	Span	30	26	4.52	348	40.89
		Support			4.52	400	47.01
		Span			8.01	348	86.41

Ground floor	Principal	Support	35	31	12.06	348	130.01
	Secondary	Span	30	26	4.62	348	41.80
		Support			5.65	400	58.76
Loft	Principal	Span	35		5.65	348	60.95
		Support		31	12.06	400	149.54
	Secondary	Span	30	26	4.62	348	41.80
		Support			6.98	400	72.59
1st floor	Principal	Span	35	31	5.65	348	60.95
		Support			12.06	400	149.54
	Secondary	Span	30	26	4.62	348	41.80
		Support			7.60	400	79.04
2nd floor	Principal	Span	35	31	5.65	348	47.76
		Support			12.81	400	158.84
	Secondary	Span	30	26	4.52	348	40.89
		Support			8.04	400	83.62
3rd floor	Principal	Span	35	31	5.65	348	60.95
		Support			12.81	400	158.84
	Secondary	Span	30	26	4.52	348	40.89
		Support			8.04	400	83.62
4th floor	Principal	Span	35	31	5.65	348	60.95
		Support			12.81	400	158.84
	Secondary	Span	30	26	4.52	348	40.89
		Support			8.04	400	83.62
5th floor	Principal	Span	35	31	5.65	348	60.95
		Support			12.06	400	149.54
	Secondary	Span	30	26	4.62	348	41.80
		Support			7.60	400	79.04
6th floor	Principal	Span	35	31	5.65	348	60.95
		Support			12.06	400	149.54
	Secondary	Span	30	26	4.62	348	41.80
		Support			6.98	400	72.59
		Span			5.65	348	60.95

7th floor	Principal	Support	35	31	10.65	400	132.06
	Secondary	Span	30	26	4.62	348	41.80
		Support			6.98	400	72.59

TABLE V 20: VERIFICATION OF THE NODAL ZONE OF PRINCIPAL BEAMS IN DIFFERENT LEVELS

Level	M_s $(kN.m)$	M_n $(kN.m)$	$M_n + M_s(kN.m)$	$M_E(kN.m)$	$M_W(kN.m)$	$1.25*(M_E + M_W(kN.m)$	OBS
Basement	230.72	180.10	410.82	83.067	114.89	247.45	Verified
Ground floor	180.10	180.10	360.2	86.41	130.01	270.53	//
Loft	180.10	144.84	324.94	60.95	149.54	263.11	//
1st floor	144.84	110.95	255.79	60.95	149.54	263.11	Not verified
2nd floor	110.95	110.95	221.9	47.76	158.84	258.25	//
3rd floor	110.95	91.18	202.13	60.95	158.84	274.77	//
4th floor	91.18	91.18	182.36	60.95	158.84	274.77	//
5th floor	91.18	60.62	151.8	60.95	149.54	263.15	//
6th floor	60.62	60.62	121.24	60.95	149.54	263.15	//
7th floor	60.62	43.78	104.4	60.95	132.06	241.3	//

TABLE V 21: VERIFICATION OF THE NODAL ZONE OF SECONDARY BEAMS IN DIFFERENT LEVELS

Level	M_s $(kN.m)$	M_n $(kN.m)$	$M_n + M_s(kN.m)$	$M_E(kN.m)$	$M_W(kN.m)$	$1.25*(M_E + M_W(kN.m)$	OBS
Basement	230.72	180.10	410.82	40.89	47.01	109.88	Verified
Ground floor	180.10	180.10	360.2	41.80	58.76	125.7	//

Loft	180.10	144.84	324.94	41.80	72.59	142.99	//
1st floor	144.84	110.95	255.79	41.80	79.04	151.05	//
2nd floor	110.95	110.95	221.9	40.89	83.62	155.64	//
3rd floor	110.95	91.18	202.13	40.89	83.62	155.64	//
4th floor	91.18	91.18	182.36	40.89	83.62	155.64	//
5th floor	91.18	60.62	151.8	41.80	79.04	151.05	//
6th floor	60.62	60.62	121.24	41.80	72.59	142.99	Not verified
7th floor	60.62	43.78	104.4	41.80	72.59	142.99	//

NOTE

We note that the condition is not verified from 1st to 7th floor in principal beams and 6th and 7th floor in secondary beams. the reinforcement area in columns must be increased.

The new reinforcement areas are summarized in the table V.22:

TABLE V 22: NEW COLUMN REINFORCEMENT

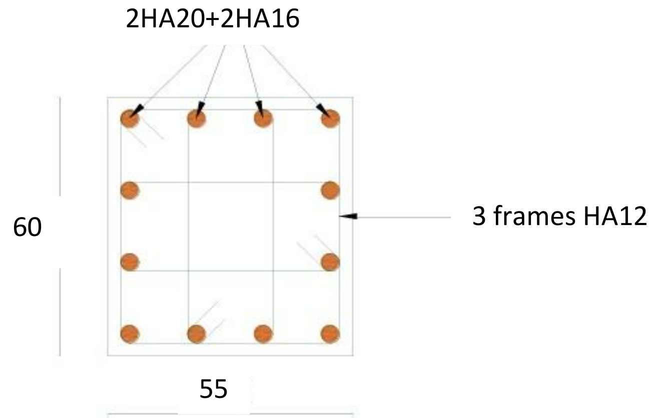
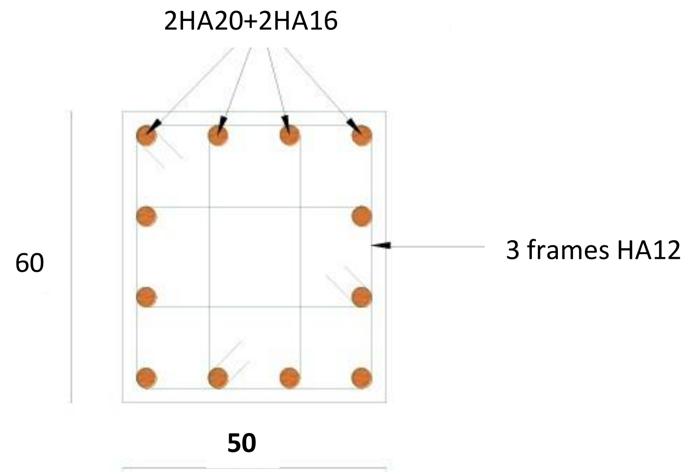
Level	Section (cm^2)	Z(cm)	$A_{adopt}(cm^2)/face$	$M_r\ (kN.m)$
Basement	55*60	56	2HA20+2HA16=10.3	230.72
Ground floor and loft	50*60	56	2HA20+2HA16=10.3	230.72
1st floor	45*55	51	2HA20+2HA16=10.3	210.12
2nd and 3rd floor	40*50	46	2HA20+2HA16=10.3	189.52
4th and 5th floor	35*45	41	2HA20+2HA16=10.3	168.92
6th and 7th floor	30*40	36	2HA20+2HA16=10.3	148.32
8th floor	30*30	27	3HA20=9.42	101.74

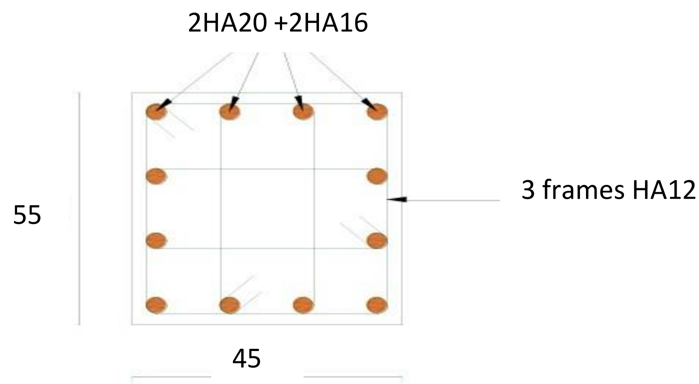
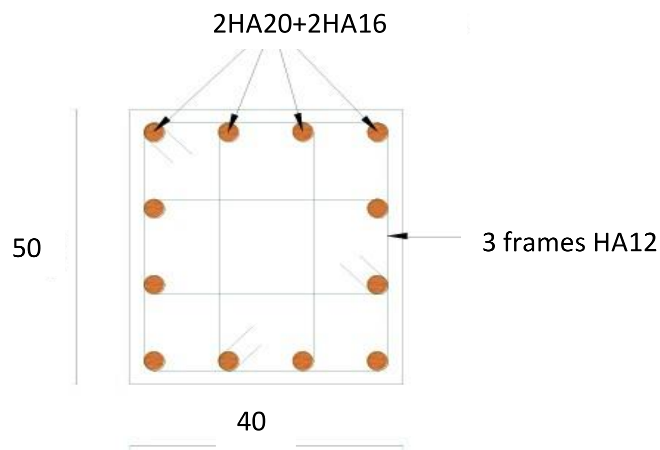
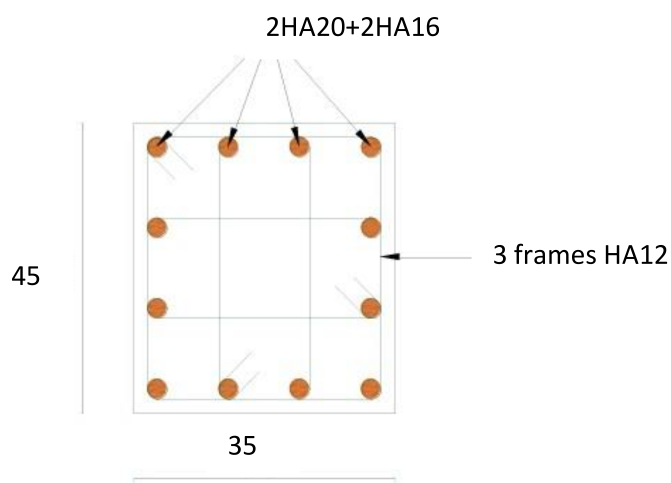
TABLE V 23: CORRECTED NODAL ZONES CHECKS

Level	Type	M_s $(kN.m)$	M_n $(kN.m)$	$M_n + M_s(kN.m)$	$M_E(kN.m)$	$M_W(kN.m)$	$1.25*(M_E + M_W(kN.m)$	OBS
Basement	PB	230.42	230.42	460.84	83.067	114.89	247.45	Verified
	SB				40.89	47.01	109.88	//
Ground floor	PB	230.42	230.42	460.84	86.41	130.01	270.53	//
	SB				41.80	58.76	125.7	//
Loft	PB	230.42	210.12	440.54	60.95	149.54	263.11	//
	SB				41.80	72.59	142.99	//
1st floor	PB	210.12	189.52	389.64	60.95	149.54	263.11	//
	SB				41.80	79.04	151.05	//
2nd floor	PB	189.52	189.52	379.04	47.76	158.84	258.25	//
	SB				40.89	83.62	155.64	//
3rd floor	PB	189.52	168.92	358.44	60.95	158.84	274.77	//
	SB				40.89	83.62	155.64	//
4th floor	PB	168.92	168.92	337.84	60.95	158.84	274.77	//
	SB				40.89	84.63	156.9	//
5th floor	PB	168.92	148.32	317.24	60.95	149.54	263.15	//
	SB				41.80	79.04	151.05	//
6th floor	PB	148.32	148.52	296.64	60.95	149.54	263.15	//
	SB				41.80	72.59	142.99	//
7th floor	PB	148.52	101.74	250.26	60.95	132.06	241.3	//
	SB				41.80	72.59	142.99	//

V.4. Reinforcement diagrams

TABLE V 24: REINFORCEMENT DIAGRAM OF COLUMNS

Level	diagram
Basement	
Ground floor and loft	

1st floor	
2nd and 3rd floor	
4th and 5th floor	

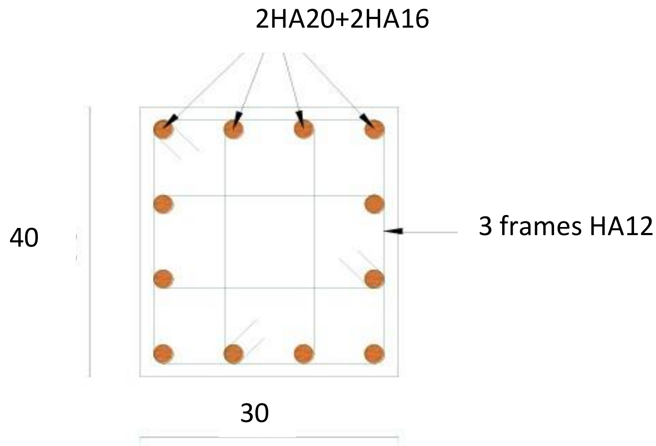
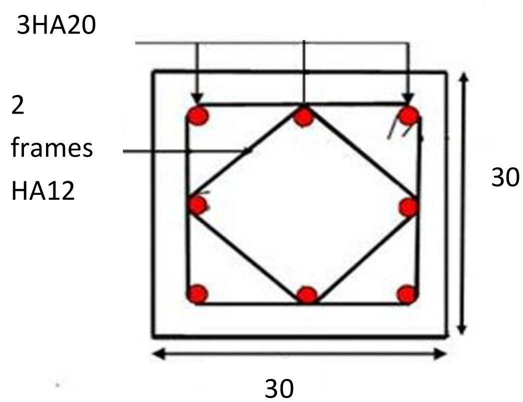
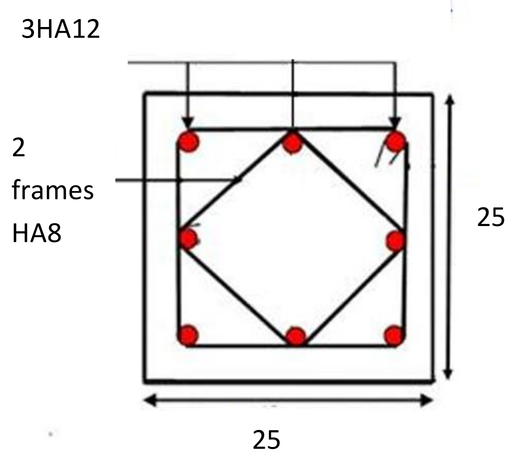
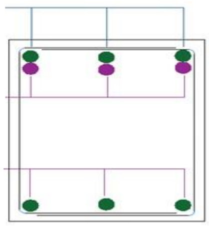
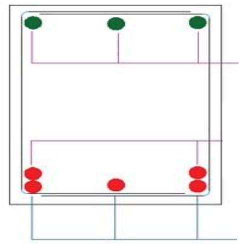
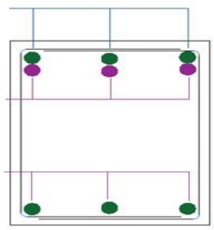
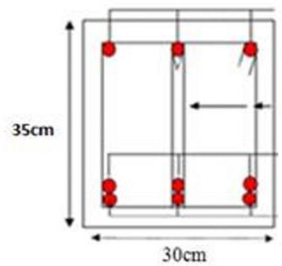
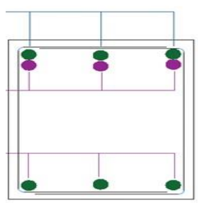
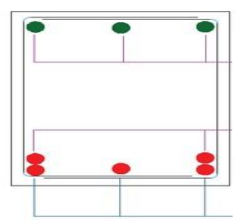
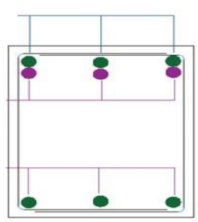
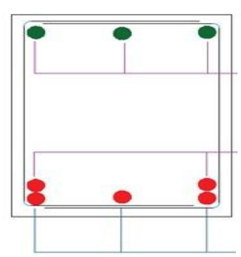
6th and 7th floor	
8th floor	
Terrace	

TABLE V 25: REINFORCEMENT DIAGRAMS OF PRINCIPAL BEAMS

Level	Support diagram	Span diagram
Basement	3HA16 3HA14 3HA14 	 3HA16 5HA14
Ground floor	6HA16 3HA14 	3HA16  3HA14+ 3HA12
Loft ,1st 5th and 6th floor	6HA16 3HA12 	 3HA16 5HA12
2nd 3rd and 4th floor	3HA20 3HA12 3HA12 	 3HA20 5HA12

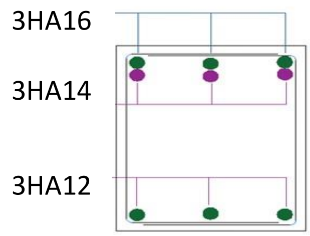
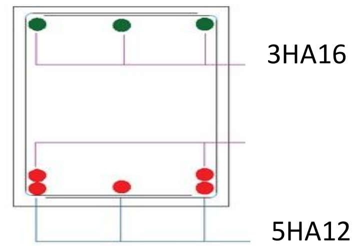
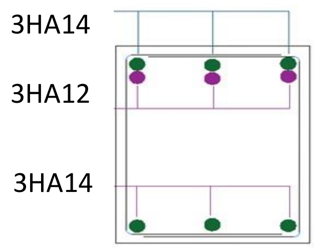
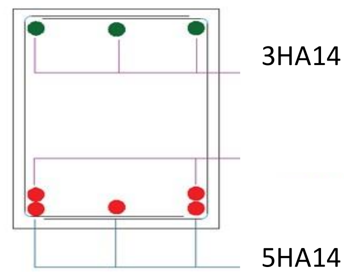
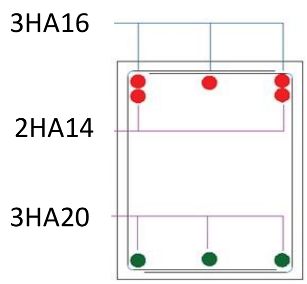
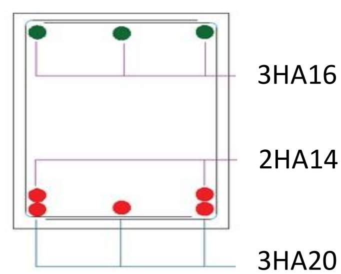
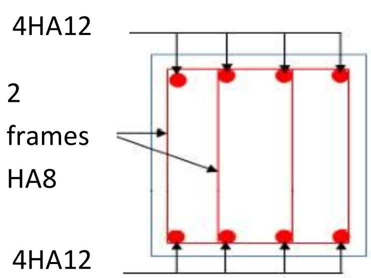
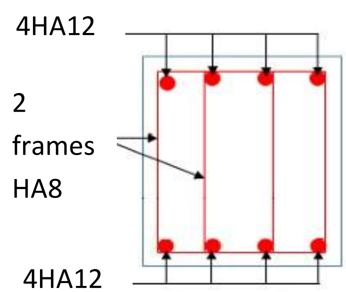
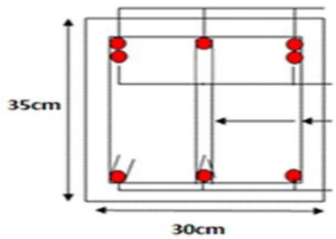
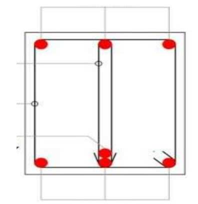
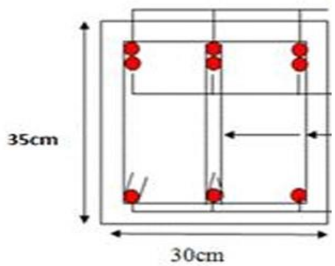
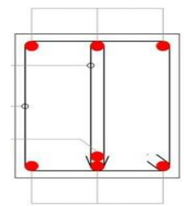
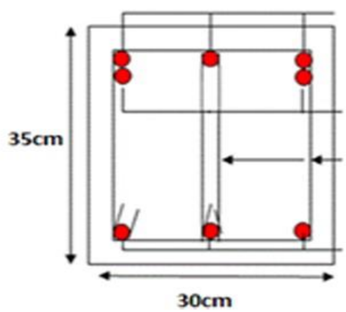
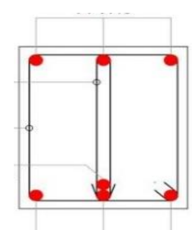
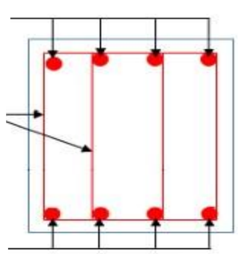
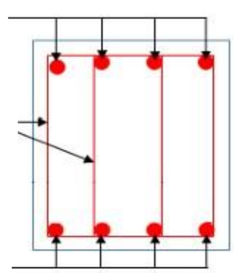
7th floor		
8th floor		
Terrace		

TABLE V 26: REINFORCEMENT DIAGRAMS OF SECONDARY BEAMS

Level	Support diagram	Span diagram
Basement and terrace		

Ground floor and 8th floor	 5HA12 3HA14	3HA12  4HA14
Loft 6th and 7th floor	 3HA14 3HA10 3HA12	3HA14  4HA12
1st and 5th floor	 3HA16 2HA10 3HA12	3HA16  4HA12
2nd 3rd and 4th floor	4HA12 2 frames HA8 4HA12 	4HA12 2 frames HA8 4HA12 

V.5 Shear walls study

A shear wall is a structural bracing element subjected to vertical and horizontal forces due to the earthquake. Its reinforcement was carried out according BAEL 91 and the verifications according to RPA99/2003.

The walls are considered as consoles embedded at their bases; their failure models are;

- ✓ Bending failure
- ✓ Bending failure by shear force
- ✓ Failure by crushing or tension of the concrete.

The calculation will be carried out in the same way as for the columns, with the most unfavorable stresses.

V.5.1. RPA99/2003 Recommendations

a) Vertical reinforcement

Longitudinal rebars are arranged in two layers parallel to the wall faces intended to absorb bending moments and must comply with the following requirements:

- ✓ In the outermost zone, the vertical bars must be tied by horizontal ties.
- ✓ The spacing of the horizontal ties must not exceed the thickness of the wall.
- ✓ In the tension zone, the maximum spacing is 15cm and a percentage of 0.2% of the concrete section $A_{min} = 0.2\% * l_t * e \ \ with\ l_t; length\ of\ the\ tension\ zone$
- ✓ At the end of the wall, the spacing of the bars must be reduced by half over $1/_{10}$ of its length.

b) Horizontal reinforcement

Horizontal bars must be arranged in two layers toward the outside of the vertical reinforcements. They are intended to absorb shear forces, maintain the vertical steel bars and prevent them from buckling.

c) Transverse reinforcement

They are perpendicular to the cross-section faces. Their role is to prevent the vertical bars from bucking under the action of compression. Their number must be at least $4bars/m^2$.

d) Common rules (vertical and horizontal reinforcement) (RPA99/2003 ART 7.7.4.3)

- The minimum percentage of reinforcement (vertical and horizontal) is:

0.15% $\quad$ $Overall\ in\ the\ shear\ wall\ section$

0.10% $\quad$ $In\ the\ main\ zone$

- The two reinforcement layers must be connected with at least 4pins per square meter.
- The spacing of the horizontal and vertical bars is:$s_t \ \leq \min(1.5e; 30cm)$.
- The diameter of the vertical and horizontal bars (except for the abutment zones) should not exceed $1/_{10}$ of the wall thickness.
- The overlap lengths must e equal to;
    - $40\emptyset$ for the bars located in areas where the reversal of the sign of the forces is possible.
    - $50\emptyset$ for bars located in areas compressed under the action of all possible load combinations.
- ✓ Along the casting joints, the shear force must be absorbed by the seam steels, the cross section of which must be calculated using the formula;

$$A_{i,j} = \frac{V}{fe} \quad with\ V = 1.4 * V_u$$

V.5.2. Stresses in the walls

The design loads are extracted directly from the model designed using ETABS2016. We will present the calculation of one shear wall in each direction (Table V.27 and V.28). The design loads of the other walls are presented in Annex 3.

TABLE V 27: MAXIMUM STRESSES IN THE SHEAR WALL VX5

Levels	N_{min} (kN)	M_{corr} (kN.m)	$comb$	N_{max} (kN)	M_{corr} (kN.m)	$comb$	M_{max} (kN.m)	N_{corr} (kN)	$comb$
Basement	289.05	178.91	*0.8G+E*$_y$	-2700.87	-179.48	*G+Q-*E_Y	-252.92	-1055.57	*0.8G-*E_X
Ground floor	262.49	193.33	//	-2619.33	-581.35	//	-1002.82	-1541.72	*G+Q-*E_x
Loft	86.87	221.68	//	-1931.84	-460.32	//	-622.39	-1167.84	//
1st floor	-25.73	231.93	//	-1735.31	-454.30	//	-505.38	-1109.28	//
2nd floor	-131.10	177.42	//	-1532.42	-410.90	//	-438.98	-1057.07	//
3rd floor	-201.54	128.27	//	-1237.88	-335.03	//	-335.03	-1237.88	*G+Q-*E_Y
4th floor	-256.39	87.56	//	-1064.68	-196.79	ELU	-323.37	-1042.24	//
5th floor	-256.94	38.77	//	-857.43	-183.17	//	-258.28	-790.27	//
6th floor	-233.57	-5.22	//	-687.98	-209.71	//	-270.39	-593.56	*G+Q-*E_x
7th floor	-151.42	-3.60	*0.8G-+*E_y	-441.03	-237.85	//	-280.22	-392.49	//
8th floor	-76.39	165.99	//	-192.03	-142.76	//	-204.22	-113.11	*G+Q+*E_x

TABLE V 28: MAXIMUM STRESSES IN THE SHEAR WALL VY2

Levels	N_{min} (kN)	M_{corr} (kN.m)	$comb$	N_{max} (kN)	M_{corr} (kN.m)	$comb$	M_{max} (kN.m)	N_{corr} (kN)	$comb$
Basement	-489.88	145.04	$0.8G + E_y$	-5380.96	-59.74	$G + Q - E_X$	2253.76	-3422.83	G+Q+E_Y
Ground floor	-223.63	386.76	//	-5236.25	-405.54	//	-4791.73	-2294.73	$0.8G - E_y$

Loft	-837.03	293.93	//	-4500.40	-285.36	//	3822.25	-2047.30	$0.8G + E_y$
1st floor	-1097.22	262.85	//	-4026.58	-19.98	ELU	3300.29	-2867.36	$G + Q + E$
2nd floor	-1216.58	-235.05	//	-3631.95	-21.97	//	2831.52	-2604.17	//
3rd floor	-1170.91	196.07	//	-3232.08	24.86	//	2248.10	-2319.40	//
4th floor	-963.65	163.98	//	-2756.41	20.79	//	1837.74	-1974.47	//
5th floor	-726.20	130.83	//	-2289.30	20.56	//	1305.04	-1638.80	//
6th floor	-487.29	111.37	//	-1785.98	7.16	//	1009.19	-1271.87	//
7th floor	-325.87	134.46	//	-1314.01	96.91	//	635.93	-933.2	//
8th floor	-144.68	22,36	//	-632.02	-76.62	//	-419.35	-472.51	$G + Q - E$

V.5.3. Reinforcement calculation

Calculation example ; Vx5(Basement)

1st combination

$N_{min} = 289.05kN$ $\quad M_{corr} = 178.91\text{kN.m}$ $\quad l = 1.85m$ $\quad d' = 2cm$ $\quad d = 1.83m$ $\quad e = 19cm$

$e_G = \frac{M}{N} = 0.62m > \frac{h}{2} - d' = 0.905m$

N_u (tension) and C inside the section, therefore the section is fully tensioned

$fs10 = \frac{fe}{\gamma s} = 400Mpa$

$e_1 = \left(\frac{h}{2} - d'\right) + e_G = 1.525m$ $\qquad e_2 = (d - d') - e_1 = 0.285m$

$A_1 = \frac{Nu*e_2}{fs10(d-d')} = 1.14cm^2$ $\qquad A_2 = \frac{Nu*e_1}{fs10(d-d')} = 6.09cm^2$

$A_{min}^{BAEL} = \frac{B*ft28}{fe} = 18.45cm^2$

$A = A_2 = 6.09cm^2$

$2A_2 = 12.18cm^2 < A_{min}^{BAEL} = 18.45cm^2$ $\qquad We\, reinforce\, with\ \frac{A_{min}^{BAEL}}{2} = 9.23cm^2\ /face$

- ***Determination of lengths (tension and compression)***

Tension zone

Tension zone

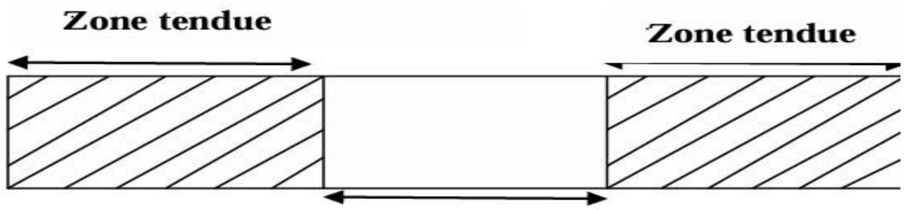


FIGURE V- 3: COMPRESSION AND TENSION ZONE IN SHEAR WALLS

We have $\begin{cases} l_t = \frac{\sigma_{min}*L}{\sigma_{max}+\sigma_{min}} \\ l_c = l_s - 2*l_t \end{cases}$ $\quad with \quad \sigma_{1,2} = \frac{N}{S} \pm \frac{M}{I}*\frac{h}{2}$

$l_t; Tension\ zone\ length$

$l_c\ ; Compression\ zone\ length$

In this case $l_t = l_s = 1.85m$

- ***Minimum reinforcement***

We have $A_{min}^{Tz} = 0.2\% * (e * l_t) \quad 0{,}002 * 19 * 185 = 7.039.23cm^2$

- ***Spacing of vertical bars***

$S_t \leq \min(1.5e\ ; 30cm) \quad \Rightarrow \quad S_t \leq 28.5 \qquad We\ take\ S_t = 20cm$

- ***Horizontal reinforcement***

$A_h = \frac{\tau_u*e*s_t}{0.8*fe} \qquad with \quad \tau_u = \frac{1.4*V_u}{e*d} = 1.12MPa$

- ***Spacing of horizontal bars***

$S_t \leq \min(1.5e\ ; 30cm) \quad \Rightarrow \quad S_t \leq 28.5 \qquad We\ take\ S_t = 20cm$

Therefore $A_h = 1.33cm^2$

$A_h^{min} = 0.15\% * e * S_t = 0.57cm^2$

2nd combination

$N_{max} = 2700.87kN\ .m \qquad M_{corr} = 179.48\text{kN}$

$e_G = 0.066m > \frac{h}{2} - d' = 0.905m$

Nu(compression) and C inside the section, with the following condition,

$M_{UA} = 2.651Mn.m$

$N_u(d - d') - M_{UA} = 2.238Mn.m \leq (0.337H - 0.81d')b * h * f_{bu} = 2.651Mn.m$

The section is partially compressed.

$U_{bu} = 0.225 \qquad \propto = 0.3230 \qquad Z = 1.59m \qquad A_1 = \frac{M_{UA}}{Z*fst} = 46.68\ cm^2$

$A = A_1 - \frac{N_u}{fe} = -25.84cm^2$ The section is negative so the concrete alone is enough.

We reinforce with $A_{min} = 4.20cm^2$

3rd combination

$M_{max} = 252.92kN\ .m$ $N_{corr} = 1055.57\text{kN}$

$e_G = 0.24m > \frac{h}{2} - d' = 0.905$

Nu(compression) and C inside the section, with the following condition,

$M_{UA} = 1.208Mn.m$

$N_u(d - d') - M_{UA} = 0.703Mn.m \leq (0.337H - 0.81d')b * h * f_{bu} = 3.945Mn.m$

The section is partially compressed.

$U_{bu} = 0.103$ $\propto = 0.103$ $Z = 1.73m$ $A_1 = \frac{M_{UA}}{Z*fst} = 17.46\ cm^2$

$A = A_1 - \frac{N_u}{fe} = -8.93cm^2$ The section is negative so the concrete alone is enough.

We reinforce with $A_{min} = 4.20cm^2$

The first combination gives the maximum section.

The tables V.29 and V.30 summarize all the reinforcement results for shear walls Vx5 and Vy2:

TABLE V 29: REINFORCEMENT RESULTS FOR SHEAR WALL VX5

Level	Basem-ent	Ground floor	Loft	1st floor	2nd floor	3rd floor	4th floor	5th floor	6th floor	7th floor	8th floor
e(cm)	19	19	16	16	16	16	16	16	16	16	16
L(m)	1.85	//	//	//	//	//	//	//	//	//	//
Section type	FTS	FTS	PCS	//	//	//	//	//	//	//	//
N(kN)	289.05	262.49	86.87	-25.73	-131.10	-201.54	-256.39	-256.94	-233.57	-392.49	-113.11
M (kN.m)	178.91	193.33	221.68	231.93	177.42	128.27	87.56	38.77	-5.22	-280.22	204.22
V (kN)	277.94	434.57	344.72	297.39	260.32	224.1	209.54	179.29	177.32	224.45	129.54
τ (MPa)	1.12	1.75	1.65	1.42	1.24	1.07	1.0	0.86	0.85	1.07	0.62
$\bar{\tau}$ (MPa)	5.00	//	//	//	//	//	//	//	//	//	//

A_{cal} (cm^2)	6.09	5.93	-2.38	2.89	0.83	-0.72	-1.90	-2.67	-2.85	-0.83	1.43
A_{min} (cm^2)	18.45	18.45	4.44	4.44	//	//	//	//	//	//	//
l_t(m)	1.85	1.85	0.82	0.89	0.71	0.48	0.09	0.47	0.85	0.53	0.77
l_c(m)	/	/	0.21	0.07	0.43	0.89	1.67	0.91	0.15	0.79	0.31
A_{min}^{Tz} (cm^2)	7.03	7.03	2.62	2.85	2.27	1.54	0.29	1.50	2.72	1.70	2.46
Choice	24HA10	24HA10	20HA8	24HA8	20HA8	16HA8	4HA8	16HA8	24HA8	16HA8	24HA8
A_z^T (cm^2)	18.85	18.85	10.05	12.06	10.05	8.04	2.01	8.04	12.06	8.04	12.06
St(cm)	20	20	20	18	18	//	//	//	//	//	//
$st\ on\ \frac{l}{10}$ (cm)	10	10	10	9	9	//	//	//	//	//	//
A_{min}^c (cm^2)	/	/	0.34	0.17	0.69	1.42	2.67	1.46	0.24	1.26	0.50
Choice	/	/	4HA8	2HA8	6HA8	10HA8	22HA8	10HA8	2HA8	10HA8	2HA8
A_z^c (cm^2)	/	/	2.01	1.01	3.02	7.85	11.06	7.85	1.01	7.85	1.01
A_h^{cal} (cm^2)	1.33	1.33	1.49	1.28	1.12	0.96	0.90	0.77	0.77	0.96	0.56
A_h^{min} (cm^2)	0.54	0.54	0.54	0.43	0.43	//	//	//	//	//	//
Choice	2HA10	2HA10	//	//	//	2HA8	//	//	//	//	//
A_h^{adopt} (cm^2)	1.58	1.58	//	//	//	1.01	//	//	//	//	//
St(cm)	20	20	20	18	//	//	//	//	//	//	//

Table V 30: Reinforcement results for shear wall Vy2

Level	basement	Ground floor	Loft	1st floor	2rd floor	3rd floor	4th floor	5th floor	6th floor	7th floor	8th floor

e(cm)	19	19	16	16	//	//	//	//	//	//	//
L(m)	5.40	//	//	//	//	//	//	//	//	//	//
Section type	PCS	PCS	//	//	//	//	//	//	//	//	//
N(kN)	-489.88	-2294.73	-2047.30	-373.72	-1216.58	-1170.91	963.65	-726.20	487.29	325.87	144.68
M (kN.m)	145.04	-4791.24	-3822.25	-1097.22	235.05	196.07	163.98	130.83	111.37	134.36	22.36
V (kN)	363.58	1207.50	1179.11	1060.54	1020.24	868.39	826.10	630.63	572.83	366.44	254.81
τ (MPa)	0.50	1.65	1.92	1.72	1.65	1.41	1.34	1.03	0.93	0.60	0.41
$\bar{\tau}$ (MPa)	5	5	//	//	//	//	//	//	//	//	//
A_{cal} (cm^2)	-5.40	-3.45	-5.27	-8.96	-13.83	-13.47	-11.11	-8.38	-5.54	-3.43	-1.71
A_{min} (cm^2)	15.39	15.39	//	//	//	//	//	//	//	//	//
l_t(m)	1.82	1.84	1.40	1.08	2.13	2.19	2.18	2.15	2.01	1.44	2.22
l_c(m)	1.76	1.72	2.60	3.24	1.14	1.02	1.04	1.10	1.38	2.52	0.96
A_{min}^{Tz} (cm^2)	6.92	6.99	4.48	3.46	6.82	7.01	6.98	6,88	6.43	4.61	7.10
Choice	36HA8	36HA8	44HA8	36HA8	60HA8	60HA8	60HA8	60HA8	56HA8	44HA8	60HA8
A_z^T (cm^2)	18.10	18.10	22.12	18.10	30.16	30.16	30.16	30.16	28.16	22.12	30.16
St(cm)	28	28	19	//	//	//	//	//	//	//	//
$st\ on\ \frac{l}{10}$ (cm)	14	14	9.5	//	//	//	//	//	//	//	//
A_{min}^{c} (cm^2)	3.34	3.26	4.16	5.18	1.82	1.63	1.66	1.76	2.21	4.03	1.54
Choice	12HA8	12HA8	26HA8	34HA8	10HA8	1OHA8	10HA8	10HA8	14HA8	26HA8	10HA8
A_z^c (cm^2)	6.03	6.03	13.06	17.10	5.03	5.03	5..03	5.03	7.041	13.06	5.03
A_h^{cal} (cm^2)	0.83	2.74	1.82	1.63	1.57	1.34	1.27	0.98	0.88	0.57	0.39

A_h^{min} (cm^2)	0.80	0.80	0.46	//	//	//	//	//	//	//	//
Choice	2HA8	4HA10	2HA12	//	2HA10	//	//	2HA8	//	//	//
A_h^{adopt} (cm^2)	1.O1	3.14	2.26	//	1,57	//	//	1.01	//	//	//
St(cm)	28	28	19	//	//	//	//	//	//	//	//

The shear wall reinforcement diagram is shown in figure V-4:

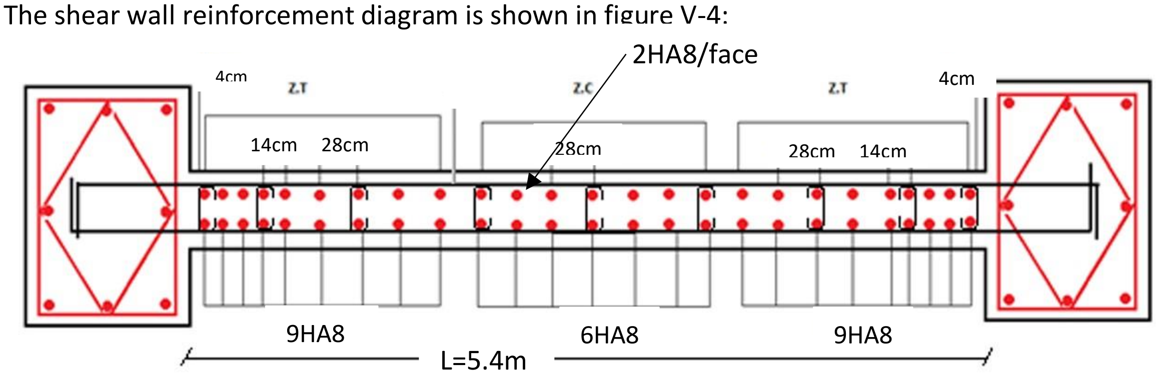


FIGURE V- 4: SHEAR WALL REINFORCEMENT DIAGRAM OF VY2(BASEMENT)

Chapter V

Infrastructure study

Introduction

The substructure transfers all structural loads to the foundation, either directly (e.g., footings on soil, raft foundations) or indirectly (e.g., footings supported on piles). This chapter covers the design of these foundation elements as reinforced concrete, determining their dimensions and reinforcement.

VI.1 Foundation Type Selection

The selection of the foundation type primarily depends on the following factors:

- ✓ The bearing capacity of the soil.
- ✓ The distance between column centers.
- ✓ The loads transmitted to the soil.
- ✓ The depth to competent soil.

According to the geotechnical report, it is recommended to adopt an allowable bearing capacity of 1.5 bar.

Foundation design shall comply with the load combination requirements of both BAEL (French Concrete Code) and RPA (Algerian Seismic Regulations).

Note

The design is carried out at the SLS with the permissible soil pressure obtained at the SLS, or at the ULS but increasing the soil pressure by the coefficient of 3/2.

Generally, the total surface area of the foundation must satisfy the following condition;

$\frac{N}{S_f} \leq \sigma_{soil} \quad with;$

N: The total force at the base of the building (ULS and SLS), determined by Etabs.

S_f: Support surface on the ground.

σ_{soil}:The bearing capacity of the soil.

$$S_f \geq \begin{matrix} \frac{55981.35}{150} & SLS \\ \frac{77292.89}{1.5*150} & ULS \end{matrix} \quad \Rightarrow \; S_f \geq 373.21m^2 \; (SLS)$$

Total ground area: $S_b = 436.695m^2$

$$\frac{S_f}{S_b} = \frac{373.21}{436.695} = 86.15\%$$

The foundation area covers >80% of the footprint, leaving minimal space for isolated footings. Design calculations showed that strip footings would overlap due to the 3.7m minimum column spacing. Consequently, a full raft foundation was selected.

VI.2 Raft Foundation Design Calculation

VI.2.1 Geometric characteristics of the raft

The raft is considered infinitely rigid, so the following conditions must be satisfied;

- **Formwork conditions**

- The thickness of **the raft slab** is calculated from the condition:

$$\boldsymbol{h_r \geq \frac{l}{20}}$$

l: The longest span between two successive load-bearing elements (face-to-face distance).

$$l = 5.50 - 0.3 = 5.20m$$

$$h_r \geq \frac{520}{20} = 26cm \quad \Rightarrow \; h_r = 30cm$$

- The thickness of **the raft ribs** is calculated from the condition:

$$h_r \geq \frac{l}{10}$$

l: The longest span of the raft ribs (face-to-face distance).

$$l = 5.50 - 0.55 = 4.95m$$

$$h_t \geq \frac{495}{10} = 49.5m \quad \Rightarrow \; h_t = 50cm$$

- **Rigidity condition**

For a raft to be rigid, it is necessary that;

$$l_{max} \leq \frac{\pi}{2} * l_e$$

$$l_e \; \geq \; \sqrt[4]{\frac{48*E*I}{K*b}}$$

With,

l_e: The elastic length, which allows us to determine the nature of the raft (rigid or flexible)

K: Soil stiffness coefficient K=4*10^4 kN/m^3 (average soil)

E: Concrete elasticity modulus E=3.2*10^7 kN/m^2

b: Footing width

$I = \frac{b*h_t^3}{12}$: Footing inertia

$$h_t \geq \sqrt[3]{\frac{48*l_{max}^4*K}{\pi^4*E}} \quad \Rightarrow h_t \geq \sqrt[3]{\frac{48*4*10^4*4.95^4}{\pi^4*3.2*10^7}} = 0.7178m$$

$$h_t = 90\text{cm}$$

❖ **The width of the rib**

We choose b=60cm

❖ **Calculating the surface area of the foundation slab**

$N' = N_s + G_{raft} + G_{rib} + G_{column\ sheet}$

$N_s = 55981.35kN$

$G_{raft} + G_{rib} = 25 * 0.30 * (436.695 + 0.60 * (0.90 - 0.30)) = 3277.91kN$

$G_{column\ sheet} = 25 * 0.60 * 0.55 * 1.5 * 35 = 445.5kN$

$\boldsymbol{N' = 59704.76kN}$

$S_{raft} \geq \frac{N'}{\sigma_{soil}} = \frac{59704.76}{150} = 398.03m^2$

Therefore, no overhang is necessary

We take $S_{raft} = S_{building} = 436.695m^2$

❖ **Shear condition**

$\tau_u = \frac{V_{max}}{\mathrm{b} \times d} \leq 0.07 * \frac{f_{c28}}{1.5} = 1.17MPa \Rightarrow d \geq \frac{V_{max}}{b*1.17}$

$b = 1m\ , \qquad d = 0.9 * 0.27m$

$V_{max} = \dfrac{N_u * l * b}{2 * s} \quad with\ N_u = 1.35 * N_{raft} + N_{ULS}$

$N_u = 1.35 * 3277.91 * 77292.89 = 81718.07kN$

$V_{max} = \dfrac{81718.07 \ * 5.5 * 1}{2 * 436.695} = 518.48kN$

$d \geq \dfrac{0.51848}{1 * 1.17} = 0.44 \quad \Rightarrow d \geq 0.45m$

We take $h_r = 50cm$

Therefore;$h_t = 90cm\ ,\ b = 50cm,\ h_r = 50cm,\ d' = 5cm,\ S_{raft} = 436.695m^2$

Therefore $\boldsymbol{N' = 61888.54kN}$

VI.2.2. Necessary verifications

a) Soil Stress Verification

The average stress under the foundation must not exceed the allowable soil bearing pressure:

$\sigma = \frac{N}{S_{raft}} \pm \frac{M_x * Y_G}{I_x}$

$X_G = \frac{\sum A_i * X_i}{\sum A_i} = 15.355m$ $\quad\quad$ $Y_G = \frac{\sum A_i * Y_i}{\sum A_i} = 9.05m$

M_x et M_y are the bending moments caused by vertical loads with respect to the center of gravity of the raft G.

$M_x = N_u * y$ $\quad\quad$ $M_y = N_u * x$

$y = Y_G - Y_{ccm} = 0.16m$ $\quad\quad$ $x = X_G - X_{ccm} = 0.105m$ $\quad\quad$ $N_u = 77292.89kN$

Y_{ccm} and X_{ccm} are the center of mass coordinates of the structure in the Y and X direction respectively.

Therefore $M_x = 12366{,}86kN \quad and \quad M_y = 8115.75kN$

From Socotec: $\begin{cases} I_x = 23169.58m^4 \\ I_y = 9885.099m^4 \end{cases}$

X-X axis

$\sigma_{max,min} = \frac{N}{S_{raft}} \pm \frac{M_x * Y_G}{I_x} = \frac{55.98135}{436.695} \pm \frac{12.366.86*9.05}{23169.58}$

$\sigma_{max} = 0.133MPa$ $\quad\quad$ $\sigma_{min} = 0.123MPa$

$\sigma_{moy} = \frac{3*\sigma_{max} + \sigma_{min}}{4} = 0.131MPa < 0.15MPa$

Y-Y axis

$\sigma_{max,min} = \frac{N}{S_{raft}} \pm \frac{M_y * X_G}{I_y} = \frac{55.98135}{436.695} \pm \frac{8.11575*15.355}{9885.10}$

$\sigma_{max} = 0.141MPa$ $\quad\quad$ $\sigma_{min} = 0.116MPa$

$\sigma_{moy} = \frac{3*0.141 + 0.116}{4} = 0.135MPa < 0.15MPa$

b) Punching shear verification

According to BAEL99 (Art A.5.2.4.2), slabs resistance to punching by shear force must be verified. It is carried out as follows;

$N_u \leq 0.045 * U_c * h_t * \frac{f_{c28}}{\gamma_b}$

N_u: Normal design force at ULS

U_c: Perimeter of the contour at the level of the middle layer

Under the most stressed column

The most stressed column is $(55 * 60)\ cm^2$, the impact perimeter U_c is given by:

$U_c = 2(A + B)$

$\begin{cases} A = a + h_t = 0.55 + 0.90 = 1.45m \\ B = b + h_t = 0.60 + 0.90 = 1.50\text{m} \end{cases} \rightarrow U_c = 5.9m$

$From\ Etabs, N_u = 3134.24kN < 0.045 * 5.9 * 0.9 * \frac{25000}{1.5} = 3345.3kN$

The condition is verified.

c) Hydrostatic uplift verification

The condition to be verified is as follows:

$N \geq f_s * H * S_{raft} * \gamma_w$

With;

$f_s: 1.15(security\ factor)$

$\gamma_w: 10kN/m^2(volume\ weight\ of\ water)$

$S_{raft}: 436.695m^2(raft\ surface)$

$H = 2m(height\ of\ the\ anchored\ part\ of\ the\ building)$

$N = 55981.31kN > 2 * 10 * 1.15 * 436.695 = 10043.985kN \quad \Rightarrow Verified$

d) Verification of overturning stability

According to the RPA (Art 10.1.5), to avoid the risk of foundation overturning, it is required to ensure that:

$e = \frac{M}{N} \leq \frac{B}{4}$

M: the overturning moments due to earthquake forces

From Etabs, Mx = 51366.83kN

; My =43037.68kN

X-X axis

$e = \frac{51366.83}{55981.35} = 0.91m \leq \frac{18.1}{4} = 4.53m$

Y-Y axis

$e = \frac{43037.68}{55981.35} = 0.768m \leq \frac{28.2}{4} = 7.05$

The conditions are verified; there is no overturning risk.

VI.2.3 Mat foundation reinforcement

The raft is calculated as an overturned floor, subjected to simple bending caused by the ground reaction. The reinforcement will be done for one panel as an example.

a) ULS calculation

$Q_U = \frac{\boldsymbol{N_u}}{S_{raft}}$

N_u= N +1.35($P_{raft} + P_{rib} + P_{column\ sheet}$)

$N_u = 77292.89 + 1.35(5461.69 + + 445.5) = 85267.59$kN

$Q_U = \frac{85267.59}{436.695} = 195.26\ kN/m^2$

Calculation example

$l_y = 5.1 - 0.5 = 4.6m$; $l_x = 4.7 - 0.5 = 4.2m$

$\rho = \frac{l_x}{l_y} = \frac{4.2}{4.6} = 0.91 > 0.4 \quad \Rightarrow the\ slab\ works\ in\ two\ directions$

$\begin{cases} M_{ox} = \mu_x * Q_U * l_x^2 = 164.30kN.m \\ M_{oy} = \mu_y * M_{ox} = 132.03kN.m \end{cases} \quad with \quad \begin{cases} \mu_x = 0.0447 \\ \mu_y = 0.8036 \end{cases}$

$\begin{cases} M_{sx} = 0.75 * M_{ox} = 123.23kN.m \\ M_{sy} = 0.85 * M_{oy} = 112.23kN.m \\ M_{ax} = M_{ay} = -0.5 * M_{ox} = -82.15kN.m \end{cases}$

The reinforcement is done for a section of $(b * h) = (1 * 0.50)\ m^2$

The results are shown in the following table:

TABLE VI 1: REINFORCEMENT SECTION OF THE RAFT.

Position		$M_u(kN.m)$	A_{cal} $(cm^2)/ml$	A_{min} $(cm^2)/ml$	A_{adopt} $(cm^2)/ml$	Number of bars	St
Span	X-X	123.23	8.05	4.18	9.05	8HA12	12.5
	Y-Y	112.23	7.31	4.00	7.70	5HA14	20
Support		−82.15	5.32	4.18	5.65	5HA12	20

Minimum reinforcement criterion

We have $e = 50cm > 12cm\ and\ \rho = 0.92 > 0.4$

$\begin{cases} A_{minx} = \rho_0 * \frac{3-\rho}{2} * b * h_r = 4.18cm^2 \\ A_{miny} = \rho_0 * b * h_r = 4.0cm^2 \end{cases}$

b) SLS verification

$Q_s = \frac{N_s}{S_{raft}}$

$N_s^{cal} = \mathbf{61888.54kN}$

$Q_s = 141.72kN/m^2$

$\begin{cases} M_{ox} = 129.5kN.m \\ M_{oy} = 111.96kN.m \end{cases}$ *with* $\begin{cases} \mu_x = 0.0518 \\ \mu_y = 0.8646 \end{cases}$

$\begin{cases} M_{sx} = 97.12kN.m \\ M_{sy} = 95.17N.m \\ M_{ax} = M_{ay} = -64.56kN.m \end{cases}$

The results of constraints verification are shown in the table below:

TABLE VI 2: CONSTRAINTS VERIFICATION AT SLS.

Position		M_s $(kN.m)$	Y(cm)	I (cm^4)	$\sigma_{bc} \leq \underline{\sigma_{bc}}$ MPa	OBS	$\sigma_{st} \leq \underline{\sigma_{st}}$	OBS
Span	X-X	97.12	9.78	199572.25	$4.76 < 15$	Verified	$257.09 > 201.63$	N. V
	Y-Y	95.17	9.11	161375.60	$5.37 < 15$	//	$317.49 > 201.63$	N. V
Support		-64.56	7.92	124804.90	$4.1 < 15$	//	$287.72 < 201.63$	N. V

We note that the constraints in steel in span and support are not verified; therefore, the reinforcement must be recalculated at the serviceability limit state (SLS).

The results are shown in the following table:

TABLE VI 3: REINFORCEMENT SECTION OF THE RAFT AT SLS

Position		$M_s(kN.m)$	$B(10^{-3})$	α	A_{cal} (cm^2)	A_{adopt} (cm^2)	St
Span	X-X	97.12	2.379	0.242	11.64	6HA16=12.06	16
	Y-Y	95.17	2.331	0.240	11.40	6HA16=12.06	16
Support		-64.56	1.581	0.201	7.63	5HA14=7.70	20

1) **The raft reinforcement diagram is shown on the figure Vi.1**

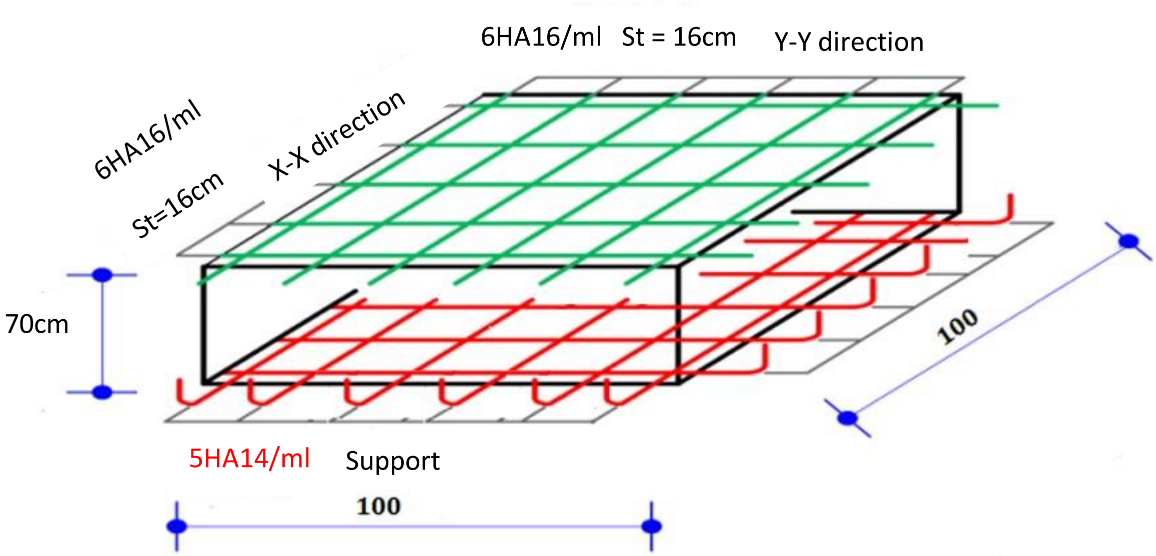


FIGURE VI- 1: RAFT REINFORCEMENT DIAGRAM

VI.3 Ribs analysis

VI.3.1 Loads carried by the ribs

The ribs serve as supports for the slab and work like the beams, that is, in simple bending. The loadings on the ribs in our case can be triangular or trapezoidal. These loads can be replaced by equivalent uniform loads.

1. **Triangular loads**

$$q_m = q_v = \frac{\rho}{2} * \frac{\sum l_{xi}^2}{\sum l_{xi}} \text{ ; } case\ of\ multiple\ triangular\ loads\ on\ the\ same\ span.$$

$$\begin{cases} q_m = \frac{2}{3} * p * l_x \\ q_v = \frac{1}{2} * p * l_x \end{cases} \text{ ; } in\ the\ case\ of\ a\ triangular\ loads\ on\ two\ sides\ \ per\ span.$$

NOTE

These expressions are developed for beams supporting triangular loads on both sides, therefore, for beams receiving a triangular load on one side, these expressions should be divided by two.

2. **Trapezoidal loads**

$$q_m = \frac{p}{2}(\left(1 - \frac{\rho_g^2}{3}\right) * l_{xg} + \left(1 - \frac{\rho_d^2}{3}\right) * l_{xd})$$

$q_v = \frac{p}{2}(\left(1 - \frac{\rho_g}{3}\right) * l_{xg} + \left(1 - \frac{\rho_d}{3}\right) * l_{xd})$

With;

q_m; *equivalent load that gives the same maximum moment as the actual load.*

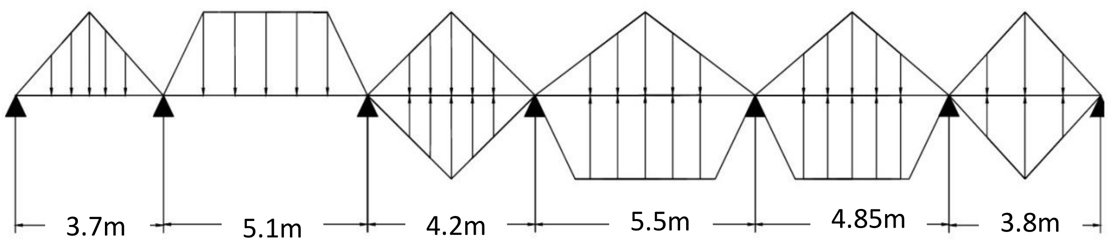
q_v; *equivalent load that gives the same maximum shear force as the actual load.*

3. Calculation of stresses

For each direction, the calculation is performed for one rib in each direction as a calculation example.

To determine the moments, the Caquot method will be used for the X and Y direction.

X-X direction



Y-Y direction

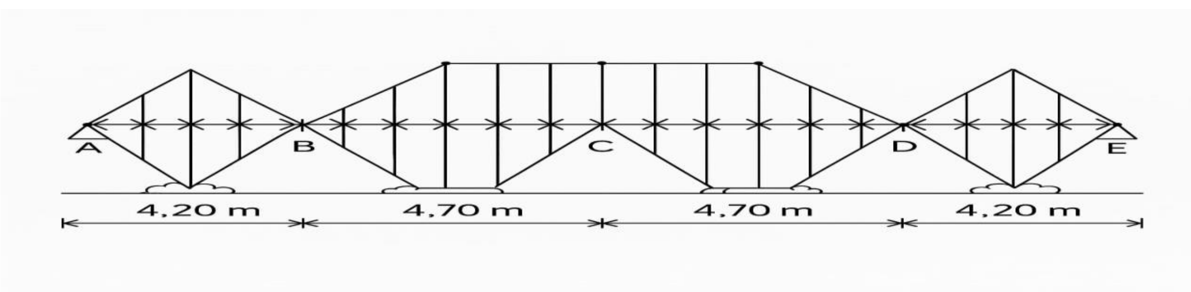


FIGURE VI- 2: LOAD DISTRIBUTION ON THE DESIGNED RIBS

VI.3.2 Calculation of internal forces

$ULS: P_u = 195.26\ kN/m^2$

$SLS: P_s = 141.72kN/m^2$

The results are summarized in the tables below,

TABLE VI 4: LOADS ON SPANS IN X-X DIRECTION

Loads	Span AB	Span BC	Span CD	Span DE	Span EF	Span FG
q_m^u (kN.m)	201.77	289.79	468.62	607.96	543.44	416.55
q_m^s (kN.m)	146.44	210.33	340.13	441.26	394.43	302.34
q_v^u (kN.m)	151.33	218.15	351.47	462.38	409.56	312.42
q_v^s (kN.m)	109.83	158.34	255.10	335.59	297.26	226.75

TABLE VI 5: LOADS ON SPANS IN Y-Y DIRECTION

Loads	Span AB	Span BC	Span CD	Span DE
q_m^u (kN.m)	468.62	569.1	569.1	468.62
q_m^s (kN.m)	340.13	413.05	413.05	340.13
q_v^u (kN.m)	351.47	369.04	369.04	351.47
q_v^s (kN.m)	255.1	267.85	267.85	255.1

The application of the Caquot method led to the results summarized in the following Tables.

TABLE VI 6: RIB LOADS IN X DIRECTION

Span		M_l (kN.m)	M_r (kN.m)	X_0 (m)	M_{span} (kN.m)	V_l^u (kN)	V_R^u (kN)
AB	ULS	0	-452.17	1.24	156.203	157.75	-402.17
	SLS	0	-328.182		113.366		
BC	ULS	-452.17	-686.519	2.39	376.478	510.33	-602.23
	SLS	-328.182	-498.281		273.248		
CD	ULS	-686.519	-1118.264	2.21	455.866	635.29	-840.88
	SLS	-498.281	-811.643		330.878		
DE	ULS	-1118.264	-1186.86	2.73	1146.414	1259.07	-1284.02
	SLS	-811.643	-861.43		832.071		
	ULS	-1186.86	-836.34		591.059		

EF	SLS	-861.43	-607.06	2.56	428.988	1065.46	-920.91
FG	ULS	-836.34	0	2.43	391.826	373.51	813.69
	SLS	-607.06	0		284.398		

TABLE VI 7: RIB LOADS IN Y DIRECTION

Span		M_l (KN.m)	M_l (KN.m)	X_0 (m)	M_{span} (KN.m)	V_l^u (kN)	V_l^u (kN)
AB	ULS	0	-960.26	1.612	608.952	509.45	-966.72
	SLS	0	-696.96		441.988		
BC	ULS	-960.26	-946.55	2.355	618.029	870.16	-864.33
	SLS	-696.96	-687.00		448.559		
CD	ULS	-946.55	-960.26	2.345	618.029	864.33	870.16
	SLS	687.00	-696.96		448.559		
DE	ULS	-960.26	0	2.588	608.952	966.72	-509.45
	SLS	-696.96	0		441.988		

TABLE VI 8: MAXIMUM STRESS ON RIBS

Direction	Location	M_{max} (KN.m)		V_{max}^u (kN)
		ULS	SLS	
X-X	Span	1146.414	832.071	--1284.02
	Support	--1186.86	--861.43	
Y-Y	Span	618.029	448.559	-966.72
	Support	-960.26	-696.96	

VI.3.3 Rib reinforcement

The rib reinforcement will be done by simple bending like inverted T sections.

- **Determination of the width b in both directions**

$$\begin{cases} h = 0.90m\ ; \quad h_0 = 0.50m \\ b_0 = 0.60m\ ; \quad d = 0.85m \end{cases}$$

<u>**X-X Direction**</u>

$\frac{b-b_0}{2} \leq min\left(\frac{l_x}{2}\,;\,\frac{l_{ymin}}{10}\right)$ $\qquad CBA(Art\ 4.1.3)$

$\frac{b-0.6}{2} \leq min\left(\frac{3.7}{2}\,;\,\frac{3.2}{10}\right) = 0.32m$

$\frac{b-0.6}{2} \leq 0.32m$

$b \leq 1.24m$

Therefore, we take **b = 1.20m.**

Y-Y Direction

$\frac{b-b_0}{2} \leq min\left(\frac{l_x}{2}\,;\,\frac{l_{ymin}}{10}\right)$ $\qquad CBA(Art\ 4.1.3)$

$\frac{b-0.6}{2} \leq min\left(\frac{3.2}{2}\,;\,\frac{3.7}{10}\right) = 0.37m$

$\frac{b-0.6}{2} \leq 0.37m$

$b \leq 1.34m$

Therefore, we take **b = 1.30m.**

The calculation results are shown in the table below:

TABLE VI 9: REINFORCEMENT CALCULATION IN RIBS

Location		$M_{tu}(kN.m)$	$M_u(kN.m)$	A_{cal} (cm^2)	A_{min} (cm^2)	A_{adopt} (cm^2))
X-X	Span		1146.414	43.25	6.16	4HA32+4HA20=44.74
	Support	3940.89	--1186.86	42.27	12.32	4HA32+4HA20=44.74
Y-Y	Span		618.029	22.06	6.16	4HA25+2HA14=22.72
	Support	4430.4	-960.26	33.72	13.34	4HA32+2HA12=34.43

- **Shear force verification at ULS**

$\tau_u = \frac{Vu}{b_o * d} < \tau_{adm} = \min(0.1 * f_{c28}\,; 4MPa) = 2.5MPa$

$$\begin{cases} X-X\ direction : \tau_u = \frac{1284.02*10^{-3}}{0.6*0.85} = 2.52MPa\ > 2.5MPa \\ Y-Y\ direction : \tau_u = \frac{966.727*10^{-3}}{0.6*0.85} = 1.9MPa\ < 2.5MPa \end{cases}$$

The condition is not verified in the X direction. Therefore, we increase the ribs dimensions.

We take $\boldsymbol{b_o = 65cm}$

Therefore,

$$\begin{cases} X - X\ direction : \tau_u = 2.32MPa\ < 2.5MPa \\ Y - Y\ direction : \tau_u = 1.75MPa\ < 2.5MPa \end{cases}$$

- **Verification of the ribs-table junction**

$$\tau_u = \frac{Vu*(\frac{b-b_o}{2})}{b_o*d} < \tau_{adm}$$

$$\begin{cases} X - X\ direction : \tau_u = \frac{1284.02*10^{-3}*(\frac{1.10-0.70}{2})}{0.65*0.85} = 0.64MPa\ < 2.5MPa \\ Y - Y\ direction : \tau_u = \frac{966.727*10^{-3}*(\frac{1.20-0.70}{2})}{0.65*0.75} = 0.57MPa\ < 2.5MPa \end{cases}$$

➢ **SLS verification**

The verification of constraints is shown in the table below:

TABLE VI 10: CONSTRAINTS VERIFICATION AT SLS

Location		$M_s(kN.m)$	Y (cm)	I (m^4)	$\sigma_{bc} \leq \overline{\sigma_{bc}}$ (MPa)	OBS		$\sigma_{st} \leq \overline{\sigma_{st}}$ (MPa)	OBS
X-X	Span	832.071	32.82	0.0259	10.54< 15	V	251.42< 201.63		N. V
	Support	--861.43	25.74	0.0304	7.29< 15	V	251.88< 201.63		N. V
Y-Y	Span	448.559	25.07	0.0176	6.39< 15	V	229.11< 201.65		N. V
	Support	-696.96	22.32	0.0251	6.20< 15	V	261.07< 201.63		N. V

The tensile stress is not verified, so the reinforcement must be recalculated at SLS. The results are shown in the following table:

TABLE VI 11: REINFORCEMENT CALCULATION AT SLS

Location		$M_s(kN.m)$	$B(10^{-3})$	α	A_{cal} (cm^2)	A_{adopt} (cm^2)
X-X	Span	832.071	8.79	0.421	56.47	6HA32+2HA25=58.07
	Support	--861.43	4.93	0.333	56.54	
Y-Y	Span	448.559	4.74	0.327	29.37	6HA25=29.45
	Support	-696.96	3.68	0.293	45.07	6HA32=48.25

VI.3.4 Transversal reinforcement

$\emptyset_t \ \leq \min\left(\frac{h_t}{35}\ ;\ \frac{b_o}{10};\emptyset_l^{min}\right) \ = \ [25.71\ ;\ 65\ ;\ 25\ ]$

$\emptyset_t \ \leq 25mm \qquad \Rightarrow we\ take\ \ \emptyset_t = 10mm$

$A_t = 6HA10 = 4.71cm^2 \qquad\quad 3\ frameworks\ of\ \ \emptyset 10$

- ***Spacing of transversal bars***

4. $S_t \leq (0.9d; 40)\ cm = min = (0.9 * 8540)cm \Rightarrow S_t \leq 40cm$
5. $S_t \leq \frac{A_t\times}{0.4\times b_0} = \frac{4.71\times 400}{0.4\times 65} = 72.46cm$
6. $S_t \leq \frac{A_t\times f_e\times 0.8\times(sin \propto + cos \propto)}{b_0\times(\tau_u - 0.3\times f_{t28})} \Rightarrow S_t \leq 13.72cm$

Therefore, we opt for spacing of $\boldsymbol{S_t = 10cm}$**.**

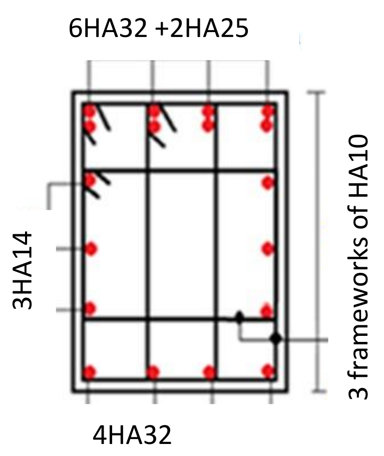
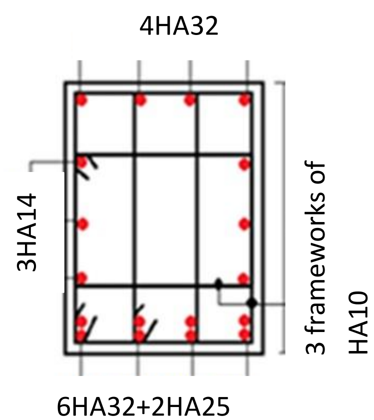
VI.3.5 Skin reinforcement (surface bars to control cracking)

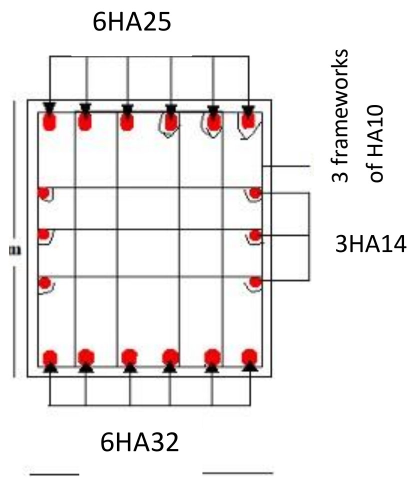
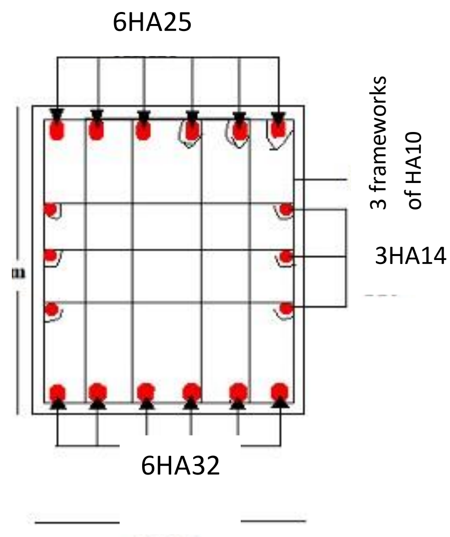
$A_t = 3 * h = 3 *\ 90\ \ = 2.72cm^2$

We take $A_t\ = 3HA14 = 3.39cm^2$

VI.3.6 Reinforcement diagrams

TABLE VI 12: RIB REINFORCEMENT DIAGRAMS

Direction	Diagram	
	Span	Support
X-X		

Y-Y		

VI.4 Perimeter retaining wall analysis

VI.4.1 Introduction

According to RPA99/2003, frames below the base level must have a peripheral wall between the foundation level and the base level. The wall must have the following characteristics;

- ✓ A minimum thickness of 15cm.
- ✓ The minimum percentage of reinforcement is 0.1%in both directions.
- ✓ The frames are made up of two layers.
- ✓ Openings in the veil must not significantly reduce its rigidity.

- **Perimeter retaining wall characteristics**
- Height: 3.57m
- Thickness: 19cm
- Length between supports: 5m
- Anchorage: 2m
- **Soil characteristics**
- Specific weight $\gamma_h: 21.1 kN/m^3$
- Friction angle: 4°
- Cohesion C: 0.5bars

VI.4.2 Evaluation of loads and overloads

a) **Earth pressure**

The retaining wall attached to the building is in limit equilibrium without significant displacement, so we take rest pressure coefficient which gives us predominant effects.

$G = (1 - \sin(\varphi)) * h * \gamma_h$

$G = (1 - \sin 4) * 3.57 * 21.1 = 70.07kN/m^2$

b) Possible surcharge

In view of mass plan attached to the plan of our structure, there is land that may be usable in future. To take into account possible loads that may generate, we plan the calculation of our wall taking into account an overload.

We have;

$q = 10kN/m^2$

$Q = q * (1 - \sin 4) = 9.302kN/m^2$

VI.4.3 Perimeter retaining wall reinforcement

The perimeter retaining wall will be calculated like a solid slab on four supports with a variable distributed load, with the embedment provided by the floor, columns and the foundation.

Figure VI.3 Shown the stress distribution on a perimeter retaining wall.

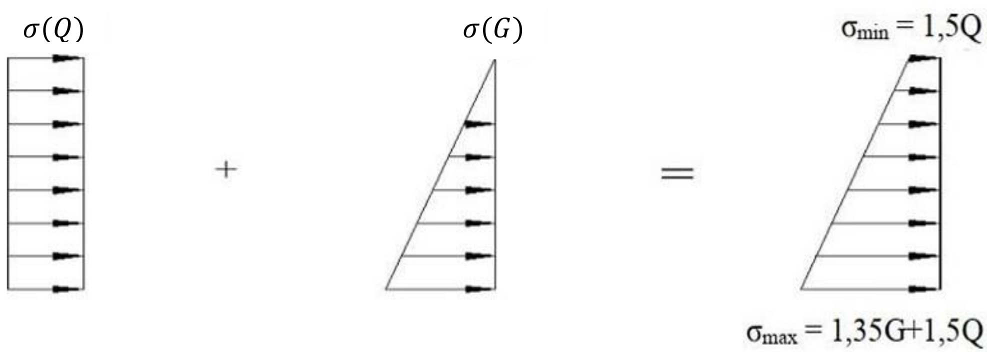


FIGURE VI- 3: STRESS DISTRIBUTION ON A PERIPHERAL WALL.

$\sigma_{min} = 1.5 * Q = 13.953kN/m^2$

$\sigma_{max} = 1.35 * G + 1.5 * Q = 108.55kN/m^2$

$\sigma_{moy} = \frac{3*\sigma_{max} + \sigma_{min}}{4} = 84.90kN/m^2$

$q_u = \sigma_{moy} * 1ml = 84.90kN/ml$

Calculation example

$\begin{cases} l_x = 3.22m \\ l_y = 5.0m \end{cases} \quad with \quad \begin{cases} b = 1m \\ t = 19cm \end{cases}$

$\rho = \frac{l_x}{l_y} = 0.64 > 0.4 \qquad \Rightarrow The\ wall\ works\ in\ two\ directions$

Therefore, we have;

$\begin{cases} M_{ox} = 67.34kN.m \\ M_{oy} = 23.38kN.m \end{cases} \quad with \quad \begin{cases} \mu_x = 0.0765 \\ \mu_y = 0.3472 \end{cases}$

$M_{spanx} = 0.85 * M_{ox} = 57.24kN.m$

$M_{spany} = 0.85 * M_{oy} = 24.12kN.m$

$M_{supportx} = M_{supporty} = -0.5M_{ox} = -33.67kN.m$

$A_{min} = 0.1\%(b * h) \qquad RPA99/2003(Art. 10.1.2)$

The reinforcement sections are summarized in the table below,

TABLE VI 13: PERIMETER RETAINING WALL REINFORCEMENT AT ULS

Location		$M_u(kN.m)$	u_{bu}	α	Z(cm)	A_{cal} (cm^2/m)	A_{min} (cm^2/m)	A_{adopt} (cm^2/m)
Span	X-X	57.24	0.1678	0.2311	14.07	11.69	1.9	6HA16=12.06
	Y-Y	24.12	0.0707	0.0917	14.98	4.64	//	6HA10=4.71
Support		-33.67	0.0987	0.1301	14.69	6.58	//	6HA12=6.79

- **Spacing**

$X\ direction: S_t \leq min(2e\ ; 25cm) \quad \Rightarrow S_t = 16cm$

$Y\ direction: S_t \leq min(2e\ ; 25cm) \quad \Rightarrow S_t = 16cm$

VI.4.4 Verifications

- **Shear force verification**

We must verify that,

$\tau_u = \frac{V_u}{bd} \leq 0.07 \frac{f_{c28}}{\gamma_b}$

Were,

$$V_x = \frac{q_u L_x}{2} * \frac{L_y^4}{L_x^4 + L_y^4} = 116.63kN$$

$$V_y = \frac{q_u L_y}{2} * \frac{L_x^4}{L_x^4 + L_y^4} = 31.15kN$$

$\tau_{ux} = 0.223MPa < 1.17MPa \qquad \Rightarrow Verified$

$\tau_{uy} = 0.833MPa \; < 1.17MPa \quad \Rightarrow Verified$

- **SLS verifications**

$\sigma_{min} = Q = 9.302kN/m^2 \qquad \sigma_{max} = G + Q = 79.374kN/m^2$

$\sigma_{moy} = 61.856kN/m^2 \qquad q_s = 61.856kN/ml$

$\rho = 0.64 > 0.4$ Therefore, we have;

$$\begin{cases} M_{ox} = 52.53kN.m \\ M_{oy} = 26.88kN.m \end{cases} \quad with \quad \begin{cases} \mu_x = 0.0819 \\ \mu_y = 0.5117 \end{cases}$$

$M_{spanx} = 44.65kN.m \qquad M_{spany} = 22.85kN.m$

$M_{support} = -26.27kN.m$

- **Constraints verifications**

$$\sigma_{bc} = \frac{M_{ser}}{I} * y \le \bar{\sigma}_{bc} = 0.6 * f_{c28} = 15MPa$$

The results are summarized in the following table:

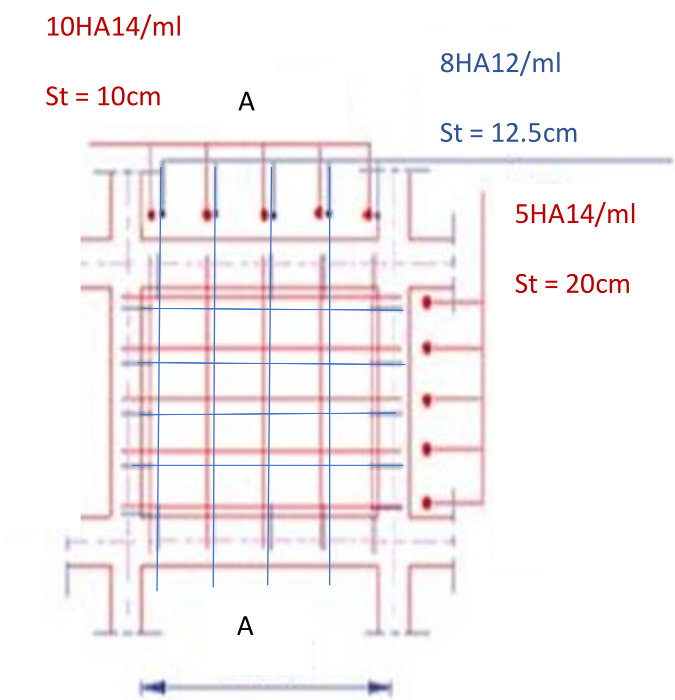
TABLE VI 14: CONSTRAINTS VERIFICATION

Location		$M_s(kN.m)$	Y(cm)	I (cm^4)	$\sigma_{bc} \le \underline{\sigma_{bc}}$ (MPa)	OBS	$\sigma_{st} \le \underline{\sigma_{st}}$ MPa	OBS
Span	X-X	44.65	5.89	20112.10	$13.08 < 15$	V	$213.35 < 201.63$	N. V
	Y-Y	22.85	4.03	10385.62	$8.87 < 15$	V	$252.36 < 201.63$	N. V
Support		-26.27	4.69	13621.16	$9.05 < 15$	V	$208.48 < 201.63$	N. V

The tensile stress is not verified, so the reinforcement must be recalculated at SLS. The results are shown in the following table:

TABLE VI 15: REINFORCEMENT CALCULATION AT SLS

Location		$M_s(kN.m)$	$\mu(10^{-3})$	α	A_{cal} (cm^2/m)	A_{adopt} (cm^2/m)	S_t (cm)
Span	X-X	44.65	1.429	0.1923	15.27	10HA14=15.39	10
	Y-Y	22.85	0.731	0.1461	7.69	5HA14=7.70	20
Support		-26.27	0.841	0.1502	8.89	8HA12=9.05	12.5



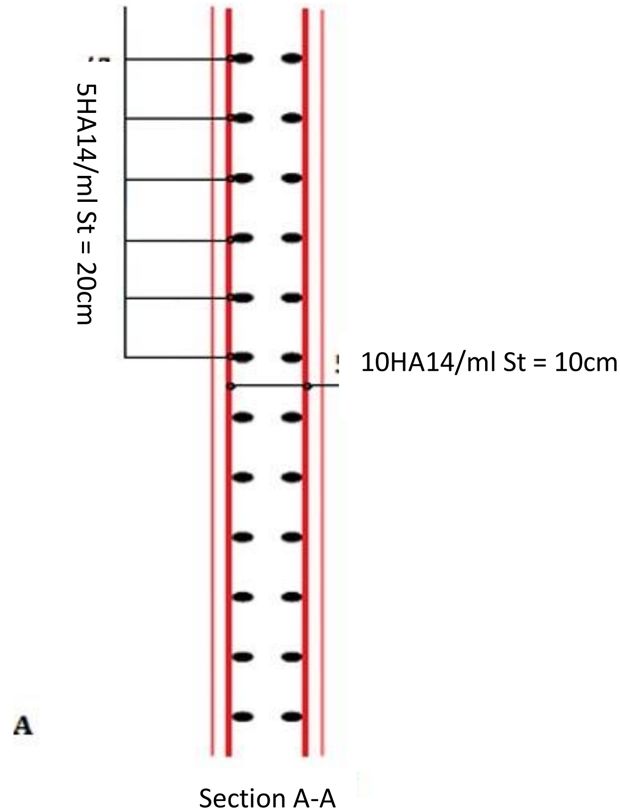


FIGURE VI- 4: PERIPHERAL WALL REINFORCEMENT DIAGRAM.

General conclusion

General Conclusion.

Throughout the study, several key challenges were encountered, each requiring thoughtful analysis and engineering judgment. One of the primary difficulties was the dispositioning of shear walls, which demanded careful consideration to ensure both structural efficiency and architectural compatibility. Improper placement could lead to torsional effects and reduced lateral resistance, making this a critical aspect of the design.

Additionally, joists dispositioning posed a challenge in terms of achieving both structural effectiveness and construction practicality. It was essential to find an optimal arrangement that balanced load distribution while aligning with the architectural layout and service routes, requiring close coordination between different design elements.

The calculation of moment resistance in beams and columns also proved to be a complex task. This step demanded precise evaluation to guarantee that all structural members could safely resist applied loads and moments, ensuring the overall stability and safety of the building under various loading scenarios.

Lastly, the reinforcement of the foundation was a fundamental concern, particularly because it serves as the primary load-transferring element to the ground. Determining the appropriate type, size, and distribution of reinforcement was vital to prevent settlement issues and to adequately support the superstructure.

In conclusion, addressing these challenges required a multidisciplinary approach, combining structural principles, practical design considerations, and adherence to building codes to achieve a safe, efficient, and durable structure.

In summary, this culminating project achieved two primary objectives: Firstly, it enabled a **comprehensive synthesis of the theoretical knowledge** accumulated throughout our academic journey. Secondly, it facilitated the **acquisition of significant new practical expertise** in structural calculation methodologies and the holistic process of building design and analysis. This experience has been invaluable in preparing us for the complexities of professional civil engineering practice, particularly in the critical domain of reinforced concrete structures.

References

References.

Codes and Standards:

- BAEL91 – *Reinforced Concrete at Ultimate Limit States*
- CBA 93 – *(Reinforced Concrete Code)*
- DTR BC 2.41 (1993) – *Technical Regulatory Document on the Design and Calculation of Reinforced Concrete Structures*
- DTR BC 2.48 – *Technical Regulatory Document for Construction in Seismic Zones*
- RPA99 (version 2003) – *Algerian Seismic Design Code*
- DTR BC 2.2 – *Permanent Loads and Live Loads*

Other Documents Consulted:

- Final year project reports
- Previous academic courses:
  - CES (*Structural Element Analysis*)
  - RDM (*Strength of Materials*)
  - MDS (*Structural Modeling*)
  - DDS (*Structural Dynamics*)
  - Reinforced Concrete

Software Used:

- ETABS version 16
- AutoCAD 2010
- Socote**c**
- Microsoft excels
- Microsoft word

Appendices

Annex 1

Rectangular slabs uniformly loaded simply supported along their edges

$\rho = \frac{L_x}{L_y}$ $\rho = \frac{L_x}{L_y}$	$ULS(\upsilon = 0)$		$SLS\ (\upsilon = 0.2)$		$\rho = \frac{L_x}{L_y}$	$ULS\ (\upsilon = 0)$		$SLS\ (\upsilon = 0.2)$	
	μ_X	μ_y	μ_X	μ_y		μ_X	μ_y	μ_X	μ_y
0.40	0.1101	0.2500	0.1121	0.2854	0.71	0.0671	0.4471	0.0731	0.5940
0.41	0.1088	0.2500	0.1110	0.2924	0.72	0.0658	0.4624	0.0719	0.6063
0.42	0.1075	0.2500	0.1098	0.3000	0.73	0.0646	0.4780	0.0708	0.6188
0.43	0.1062	0.2500	0.1087	0.3077	0.74	0.0633	0.4938	0.0696	0.6315
0.44	0.1049	0.2500	0.1075	0.3155	0.75	0.0621	0.5105	0.0684	0.6447
0.45	0.1036	0.2500	0.1063	0.3234	0.76	0.0608	0.5274	0.0672	0.6580
0.46	0.1022	0.2500	0.1051	0.3319	0.77	0.0596	0.5440	0.0661	0.6710
0.47	0.1008	0.2500	0.1038	0.3402	0.78	0.0584	0.5608	0.0650	0.6841
0.48	0.0994	0.2500	0.1026	0.3491	0.79	0.0573	0.5786	0.0639	0.6978
0.49	0.0980	0.2500	0.1013	0.3580	0.80	0.0561	0.5959	0.0628	0.7111
0.50	0.0966	0.2500	0.1000	0.3671	0.81	0.0550	0.6135	0.0617	0.7246
0.51	0.0951	0.2500	0.0987	0.3758	0.82	0.0539	0.6313	0.0607	0.7381
0.52	0.0937	0.2500	0.0974	0.3853	0.83	0.0528	0.6494	0.0596	0.7518
0.53	0.0922	0.2500	0.0961	0.3949	0.84	0.0517	0.6678	0.0586	0.7655
0.54	0.0908	0.2500	0.0948	0.4050	0.85	0.0506	0.6864	0.0576	0.7794
0.55	0.0894	0.2500	0.0936	0.4150	0.86	0.0496	0.7052	0.0566	0.7932
0.56	0.0880	0.2500	0.0923	0.4254	0.87	0.0486	0.7244	0.0596	0.8074
0.57	0.0865	0.2582	0.0910	0.4357	0.88	0.0476	0.7438	0.0546	0.8216
0.58	0.0851	0.2703	0.0897	0.4462	0.89	0.0466	0.7635	0.0537	0.8358
0.59	0.0836	0.2822	0.0884	0.4565	0.90	0.0456	0.7834	0.0528	0.8502

0.60	0.0822	0.2948	0.0870	0.4672	0.91	0.0447	0.8036	0.0518	0.8646
0.61	0.0808	0.3075	0.0857	0.4781	0.92	0.0437	0.8251	0.0509	0.8799
0.62	0.0794	0.3205	0.0844	0.4892	0.93	0.0428	0.8450	0.0500	0.8939
0.63	0.0779	0.3338	0.0831	0.5004	0.94	0.0419	0.8661	0.0491	0.9087
0.64	0.0765	0.3472	0.0819	0.5117	0.95	0.0410	0.8875	0.0483	0.9236
0.65	0.0751	0.3613	0.0805	0.5235	0.96	0.0401	0.9092	0.0474	0.9385
0.66	0.0737	0.3753	0.0792	0.5351	0.97	0.0392	0.9322	0.0465	0.9543
0.67	0.0723	0.3895	0.0780	0.5469	0.98	0.0384	0.9545	0.0457	0.9694
0.68	0.0710	0.4034	0.0767	0.5584	0.99	0.0376	0.9771	0.0449	0.9847
0.69	0.0697	0.4181	0.0755	0.5704	1.00	0.0368	1.0000	0.0441	1.0000
0.70	0.0684	0.4320	0.0743	0.5817					

Annex 2

Actual reinforcement sections

Section in cm^2

Reinforcement in mm

Φ	5	6	8	10	12	14	16	20	25	32	40
1	0.2	0.28	0.5	0.79	1.13	1.54	2.01	3.14	4.91	8.04	12.57
2	0.39	0.57	1.01	1.57	2.26	3.08	4.02	6.28	9.82	16.08	25.13
3	0.59	0.85	1.51	2.36	3.39	4.62	6.03	9.42	14.73	24.12	37.7
4	0.79	1.13	2.01	3.14	4.52	6.16	8.04	12.57	19.64	32.17	50.27
5	0.98	1.41	2.51	3.93	5.65	7.7	10.05	15.71	24.55	40.21	62.83
6	1.18	1.7	3.01	4.71	6.78	9.24	12.06	18.85	29.46	48.25	75.4
7	1.37	1.98	3.52	5.5	7.91	10.77	14.07	21.99	34.37	56.3	87.96
8	1.57	2.26	4.02	6.28	9.04	12.31	16.08	25.13	39.28	64.34	100.53
9	1.77	2.54	4.52	7.07	10.17	13.85	18.09	28.27	44.18	72.38	113.1
10	1.96	2.83	5.03	7.85	11.3	15.39	20.1	31.42	49.09	80.42	125.66
11	2.16	3.11	5.53	8.64	12.43	16.93	22.11	34.56	53.99	88.47	138.23
12	2.36	3.39	6.03	9.42	13.56	18.47	24.12	37.7	58.9	96.51	150.8
13	2.55	3.68	6.53	10.21	14.69	20.01	26.13	40.84	63.81	104.55	163.36
14	2.75	3.96	7.04	11.0	15.82	21.55	28.14	43.98	68.72	112.59	175.93
15	2.95	4.24	7.54	11.78	16.95	23.09	30.15	47.12	73.63	120.64	188.5
16	3.14	4.52	8.04	12.57	18.08	24.63	32.16	50.27	78.54	128.68	201.06
17	3.34	4.81	8.55	13.35	19.21	26.17	34.17	53.41	83.45	136.72	213.63

18	3.54	5.09	9.05	14.14	20.34	27.7	36.18	56.55	88.36	144.76	226.2
19	3.73	5.37	9.55	14.92	21.47	29.24	38.19	59.69	93.27	152.81	238.76
20	3.93	5.65	10.05	15.71	22.6	30.78	40.2	62.83	98.17	160.85	251.33

Annex 3

Shear walls load

Shear wall V_{y1}.

Floors	N_{min} (kN/M)	$M_{corr}(kN/M)$	Combo	$N_{max}(kN/M)$	$M_{corr}(kN/M)$	Combo	$M_{max}(kN/M)$	$N_{corr}(kN/M)$	Combo
Basement	-338.50	3231.91	$0.8G + E_X$	-2897.64	-2742.68	$G+Q-E_X$	5661.39	-1776,88	$G+Q+E_Y$
Ground floor	-247.23	-1214.24	//	-2706.15	-1102.66	//	4773.22	-1687.94	//
Loft	-307.03	697.03	//	-2277.07	-654.94	//	3099.91	-1466.95	//
1st	-373.72	562.81	//	-1995.55	-526.26	//	2361.58	-1347.4	//
2nd	-413.92	413.55	//	-1732.40	-382.26	//	1749.35	-1223.35	//
3rd	-418.32	273.44	//	-1513.36	9.111	ELU	1320.17	-1070.6	//
4th	-394.69	235.35	//	-1285.96	15.92	//	1060.68	-910.47	//
5th	-337.33	239.97	//	-1029.93	22.56	//	793.51	-729.61	//
6th	-253.81	257.10	//	-759.90	31.58	//	613.95	-537.72	//
7th	-153.76	221.79	//	-465.80	44.26	//	474.28	-329.06	//
8th	-72.54	115.42	//	-198.70	28.27	//	256.05	-140.25	//

Shear wall V_{X1}.

Floors	N_{min} (kN/M)	$M_{corr}(kN/M)$	Combo	$N_{max}(kN/M)$	$M_{corr}(kN/M)$	Combo	$M_{max}(kN/M)$	$N_{corr}(kN/M)$	Combo
Basement	/	/	$0.8G + E_Y$	/	/	/	/	/	/
Ground floor	-119.60	128.56	//	-1057.37	-54.52	$G+Q-E_Y$	921.34	-446.27	$G+Q+E_X$

Loft	-104.57	23.90	//	-882.43	-85.96	//	-273.22	-699.19	$G+Q-E_X$
1st	-103.18	6.60	//	-831.08	-88.52	//	-309.12	-688.64	//
2nd	-102.22	4.07	//	-773.78	-103.04	//	-281.78	-688.85	//
3rd	-96.77	4.57	//	-668.57	-98.40	//	-193.41	-613.15	//
4th	-93.37	-10.59	//	-586.39	-107.44	//	-212.92	-549.98	//
5th	-83.63	5.81	//	-461.74	-102.64	//	-145.59	-455.28	//
6th	-75.15	-11.84	//	-370.17	-102.97	//	-162.90	-364.58	//
7th	-58.09	33.73	//	-268.25	-130.98	$G+Q-E_X$	-130.98	-268.25	//
8th	-46.32	50.77	//	-146.97	-18.53	ULS	-73.85	-135.23	//

Shear wall V_{X2}.

Floors	N_{min} (kN/M)	$M_{corr}(kN/M)$	Combo	$N_{max}(kN/M)$	$M_{corr}(kN/M)$	Combo	$M_{max}(kN/M)$	$N_{corr}(kN/M)$	Combo
Basement	-45.61	257.26	$0.8G+E_X$	-1070.43	-271.63	$G+Q-E_X$	-271.63	-1070.43	$G+Q-E_X$
Ground floor	-79.48	598.20	//	-1078.16	-794.96	//	-794.96	-1078.16	//
Loft	-142.05	286.11	//	-725.70	-396.06	//	-396.06	-725.70	//
1st	-204.03	222.15	//	-641.60	-114.17	ULS	-357.16	-591.34	//
2nd	-202.66	177.23	//	-580.63	-142.62	//	-344.72	-516.80	//
3rd	-133.21	93.22	//	-496.18	-146.39	//	-265.66	-481.32	//
4th	-74.98	84.92	//	-460.07	-283.71	$G+Q-E_X$	-283.71	-460.07	//
5th	-17.91	20.73	//	-419.08	-219.77	//	-219.77	-419.08	//
6th	-5.38	22.82	//	-370.54	-237.03	//	-237.03	-370.54	//

7th	-1.95	0.73	//	-305.85	-204.49	//	-204.49	-305.85	//
8th	-33.50	18.61	//	-176.11	-154.59	//	-154.59	-176.11	//

$Shear\ wall\ V_{X3}.$

Floors	$N_{min}\ (kN/M)$	$M_{corr}(kN/M)$	Combo	$N_{max}(kN/M)$	$M_{corr}(kN/M)$	Combo	$M_{max}(kN/M)$	$N_{corr}(kN/M)$	Combo
Basement	-30.61	93.64	$0.8G + E_y$	-744.85	-105.57	$G + Q - E_Y$	-161.02	-565.01	$G + Q - E_X$
Ground floor	-67.47	32.62	//	-652.49	-94.22	//	-293.64	-587.01	//
Loft	-40.19	9.14	//	-592.51	-46.05	//	-91,72	-424.07	//
1st	-38.58	-20.58	//	-557.21	-59.60	//	-156.25	-404.90	//
2nd	-36.97	-15.01	//	-536.42	-59.54	//	-139.78	-418.33	//
3rd	-37.67	-13.70	//	-462.65	-62.52	//	-111.95	-372.76	//
4th	-39.35	-19.87	//	-412.87	-66.43	//	-130.70	-343.32	//
5th	-38.09	-12.65	//	-331.90	-67.17	//	-92.23	-286.93	//
6th	-38.01	-19.95	//	-265.27	-68.17	//	-110.39	-234.05	//
7th	-32.86	-12.24	//	-183.67	-63.31	//	-73.60	-169.28	//
8th	-35.92	9.53	//	-110.65	-64.92	//	-87.91	-98.71	//

$Shear\ wall\ V_{X4}.$

Floors	$N_{min}\ (kN/M)$	$M_{corr}(kN/M)$	Combo	$N_{max}(kN/M)$	$M_{corr}(kN/M)$	Combo	$M_{max}(kN/M)$	$N_{corr}(kN/M)$	Combo
Basement	-274.30	203.66	$0.8G + E_y$	-2750.72	-134.99	$G + Q - E_Y$	399.74	-1409.40	$G + Q - E_X$
Ground floor	-269.45	320.91	//	-2589.83	-288.08	//	804.49	-1288.92	//

Loft	-83.65	296.76	//	-1930.58	-254.69	//	572.31	-1041.28	//
1^{st}	-21.75	329.94	//	-1733.86	-300.01	//	486.40	-991.87	//
2^{nd}	-124.44	293.55	//	-1529.74	-262.86	//	426.83	-931,76	//
3^{rd}	-191.51	263.83	//	-1235.00	-196.47	//	334.41	-797.59	//
4^{th}	-244.64	232.59	//	-1051.31	49.00	ULS	312.22	-696.18	//
5^{th}	-245.19	198.65	//	-842.03	73.19	//	243.84	-543.49	//
6^{th}	-227.56	159.45	//	-670.55	54.70	//	224.94	-406.65	//
7^{th}	-152.57	182.27	//	-432.39	114.25	//	217.14	-250.20	//
8^{th}	-72.76	208.48	//	-183.83	221..88	//	268.64	-107.31	//

$Shear\ Force\ \ (V_U)\ on\ the\ shear\ walls.$

Floors	$V_U(kN)$									
	V_{Y1}	Combo	V_{X1}	Combo	V_{X2}	Combo	V_{X3}	Combo	V_{X4}	Combo
Basement	659.06	$G+Q+E_X$	/	/	-266.12	$G+Q-E_X$	-116.13	$G+Q-E_X$	386.88	$G+Q+E_X$
Ground floor	576.43	$G+Q+E_Y$	-311.32	$0.8G+E_X$	-280.25	//	-116.91	//	390.67	//
Loft	468.94	//	123.84	$G+Q-E_X$	-198.4	//	-52.11	//	424.66	//
1^{st}	442.70	//	-14.69	//	-181.94	//	-88.53	//	388.89	//
2^{nd}	403.62	//	-149.21	//	-194.32	//	-84.10	//	394.03	//
3^{rd}	334.24	//	-113.45	//	-162.47	//	-71.28	//	347.64	//
4^{th}	322.24	//	-131.22	//	-181.37	//	-84.46	//	357.71	//

5th	-260.90	//	-91.83	//	-145.11	//	-62.64	//	308.71	//
6th	242.93	//	-106.38	//	-159.03	//	-75.17	//	301.31	//
7th	82.53	//	-84.03	//	-134.20	//	-52.69	//	299.40	//
8th		//	-60.65	//	-116.53	//	-66.79	//	162.70	//

Geotechnical report

Chapitre I : GENERALITES :

1. Introduction :

Faisant suite à la demande de l'Entreprise de Promotion du Logement Familial EPLF de Bejaia par bon de commande n°024/DT/829/DG/2005 du 25/ 04/ 2005 , le Laboratoire National de l'Habitat et de la Construction (LNHC - U.Bejaia) a procédé à l'étude de sol d'un terrain devant recevoir une promotion immobilière sis au boulevard KRIM BELKACEM à Ihaddaden . (Siège ex-Jute) – Bejaia .

2. Situation régionale et géologie locale :

Le terrain devant recevoir le projet sus-cité se trouve à l'enceinte du siège ex-Jute .Il est plat et limité comme suit :

- au Nord par la Rue BOUMDAOUI .
- au Sud par la BNA .
- à l'Ouest par le boulevard Krim Belkacem .
- à l'Est par la route des Aurès .

Du point de vue géologique , nous classons ce site dans les alluvions anciennes , basses terrasses , dépôts limoneux , sables et cailloutis d'âge quaternaire .

Chapitre II : ESSAIS IN SITU :

Le programme d'investigation établi par notre laboratoire consiste en une prospection générale du site, la réalisation de onze (11) essais au pénétromètre statique lourd de type Gouda ainsi que la réalisation de huit sondages carottés .

Un plan de masse remis par le client nous a permis d'implanter et de réaliser nos essais in-situ .

1. Essai de pénétration statique lourd :

L'essai de pénétration statique consiste à faire pénétrer dans le sol par vérinage à vitesse constante (1cm /s) une pointe cônique portée par un train de tiges et à mesurer à intervalles déterminés (20 cm), la résistance à la pénétration du cône q_c ainsi que le frottement latéral. Il permet de dresser une coupe de sol lorsque le contexte géologique est bien connu, apprécier l'homogénéité d'un horizon et également la détermination de l'épaisseur et la capacité des couches portantes traversées en plus de la cohésion non drainée Cu des sols purement cohérents.

Les résultats sont consignés sous forme de pénétrogrammes donnés en annexe.
Dans le tableau suivant ,nous récapitulons les résultats obtenus :

* pour une semelle rigide de (B x L) = (10 x 20 m) de largeur et ancrée à 2.00 m de profondeur, on a :

$$q\ adm - \gamma.D = [\,5.14.(\,1 + 0.2\ B/L\,)\ Cu\,]\ /\ 3\ .$$

A.N : *q adm = 1.29 bar* .

b) - Calcul de la contrainte admissible à partir des résultats de laboratoire :

Le taux de travail peut être aussi calculé à partir des résultas de laboratoire selon les formules de Terzaghi ,comme suit :

- Pour une semelle filante de 1.00 m de largeur et ancrée à 2.00 m de profondeur :

$$q\ adm = \gamma\ .\ D + 1/3\ [\ 0.8\ .\ N\gamma\ .\ B\ /2\ + \gamma\ .\ D.\ (\ Nq - 1) + C\ .\ Nc\ ]\ .$$

Avec :
$N\gamma = 0.546$, $Nq = 1.43$, $Nc = 6.19$ et $\gamma\ h = 2.11\ T/m^3$.

(En considérant un angle de frottement interne $\varphi = 4°$ et une cohésion de $C = 0.5$ bar) .

A.N : q adm = 1.56 bar.

- Pour une semelle rigide de 10 x 20 m , ancrée à 2.00 m de profondeur :

$$q\ adm = \gamma\ D + [\ 0.8\ .\ N\gamma\ .\ B\ /2\ + \gamma\ D\ (\ Nq - 1) + CN\ c\ ]\ /\ 3\ .$$

A.N : q adm = 1.16 bar

La contrainte admissible retenue pour le calcul de fondations est de 1.50 bar.

VI- Conclusions :

VI- Conclusions :

Le terrain réservé pour la réalisation d'une promotion immobilière à Ihaddaden (Ex –Jute) est essentiellement constitué d'une couche de terre végétale allant jusqu'à 0.80 m reposant sur une couche de limon marneux plastique à compact ; puis on retrouve une couche de marne plastique sableuse assez compacte de couleur grise .Le tout repose sur une couche de marne très compacte légèrement coquillée de couleur grise .

Le sol en place présente des résistances de pointe élevées aux deux mètres .Au –delà de cette profondeur , le il détient un bon pouvoir portant .

A cet effet , on suggère :

- L'emploi de **fondations superficielles** de type **semelles filantes** .

- Le taux de travail à adopter pour le calcul des fondations sera de **1.5 bar** .

- La sous face des fondations devrait être à au moins **2.00 m** de profondeur par rapport à la côte du terrain naturel .

- Les analyses chimiques effectuées montrent un environnement d'agressivité nulle ; aucune mesure particulière de protection n'est recommandée néanmoins le béton fabriqué suivant les règles de l'art doit être compact par ses qualités intrinsèques .

- Les tassements calculés sous une semelle filante ancrée à 2.00 m de profondeur , de 1.00 m de largeur , soumise à une charge de 1.80 bar ; s'élèvent à 4.00 cm .

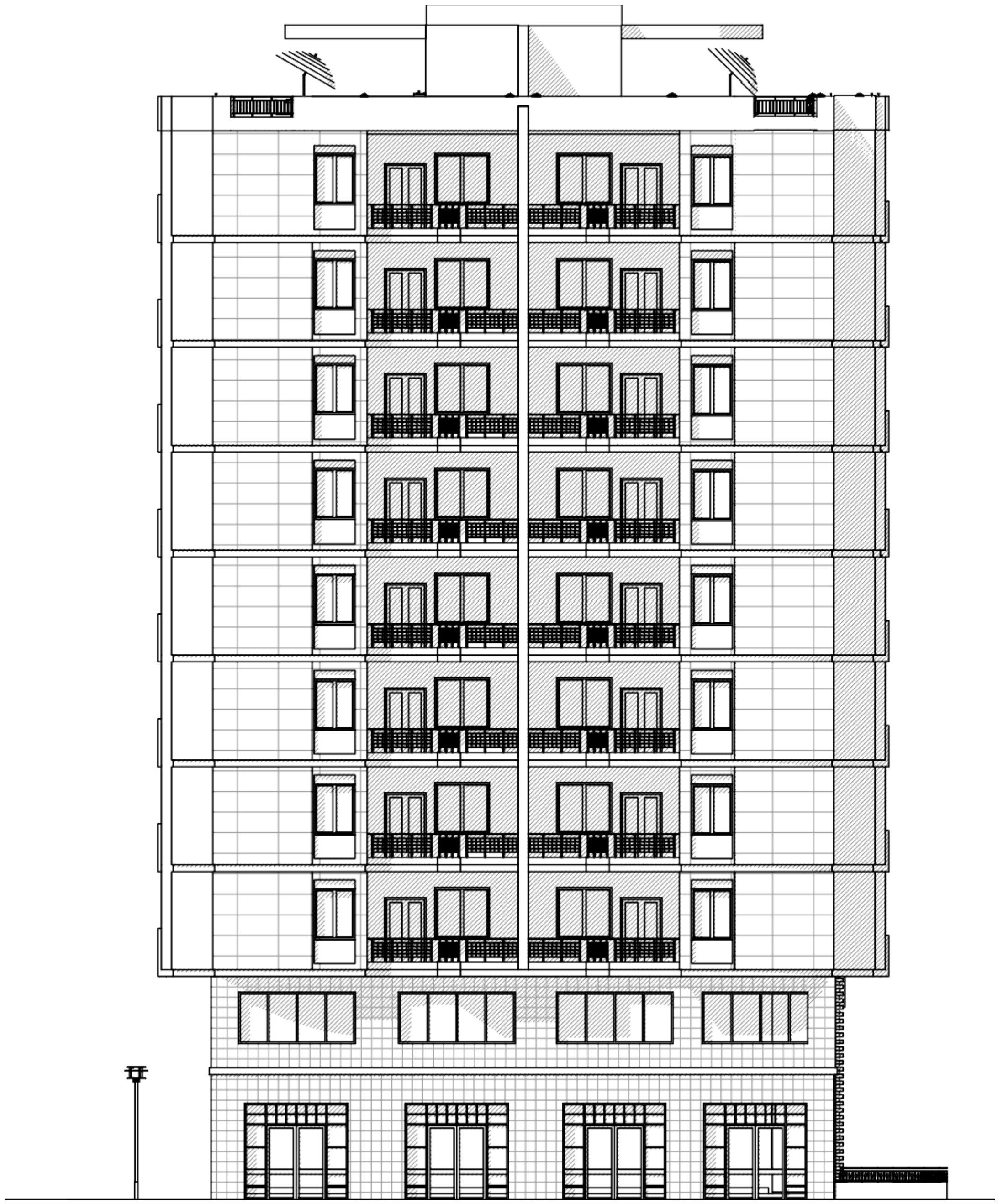
مدير الوكالة
لعزيزي صالح

Annex 5

Structural plans

FAÇADE PRINCIPALE

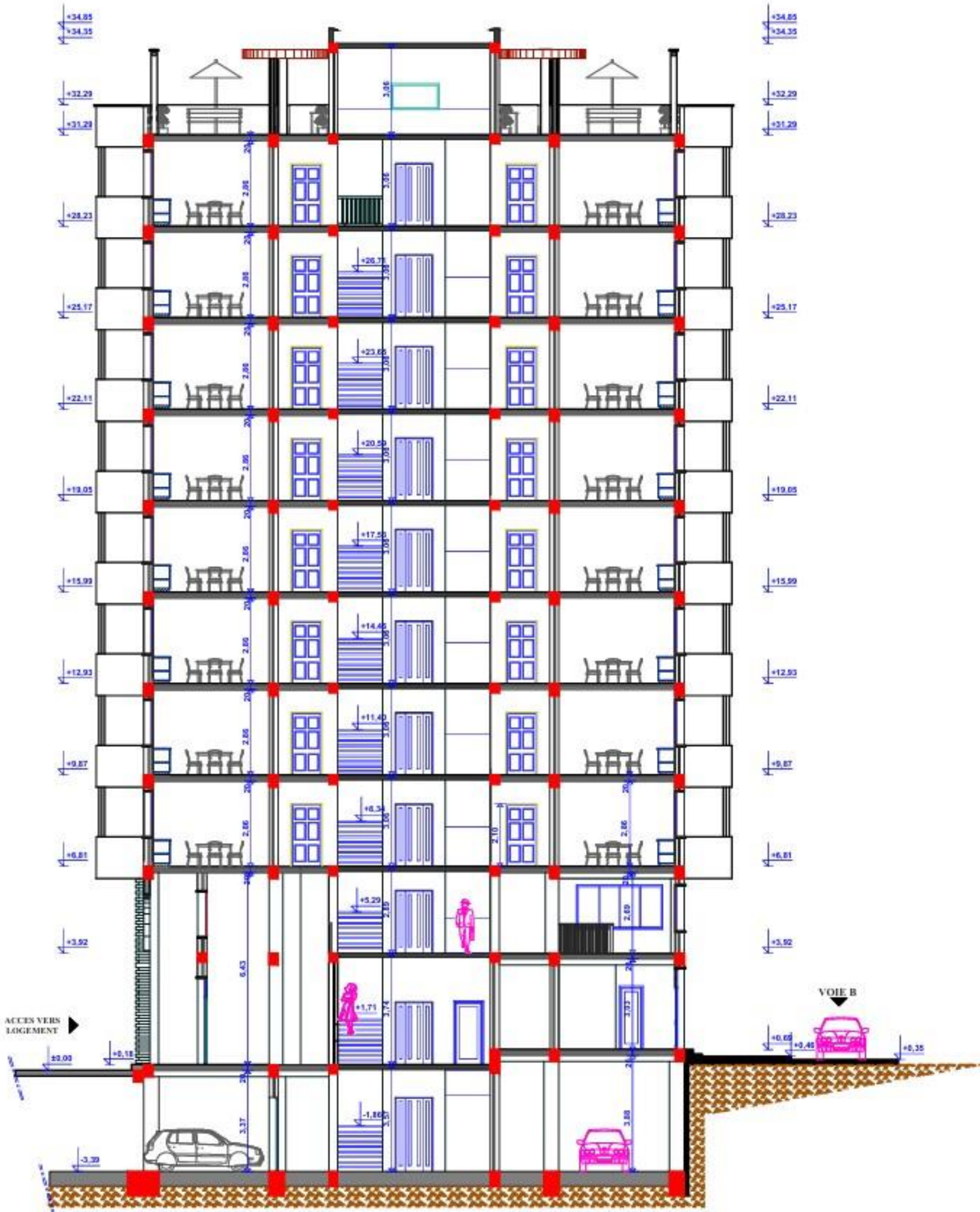
REAR FACADE



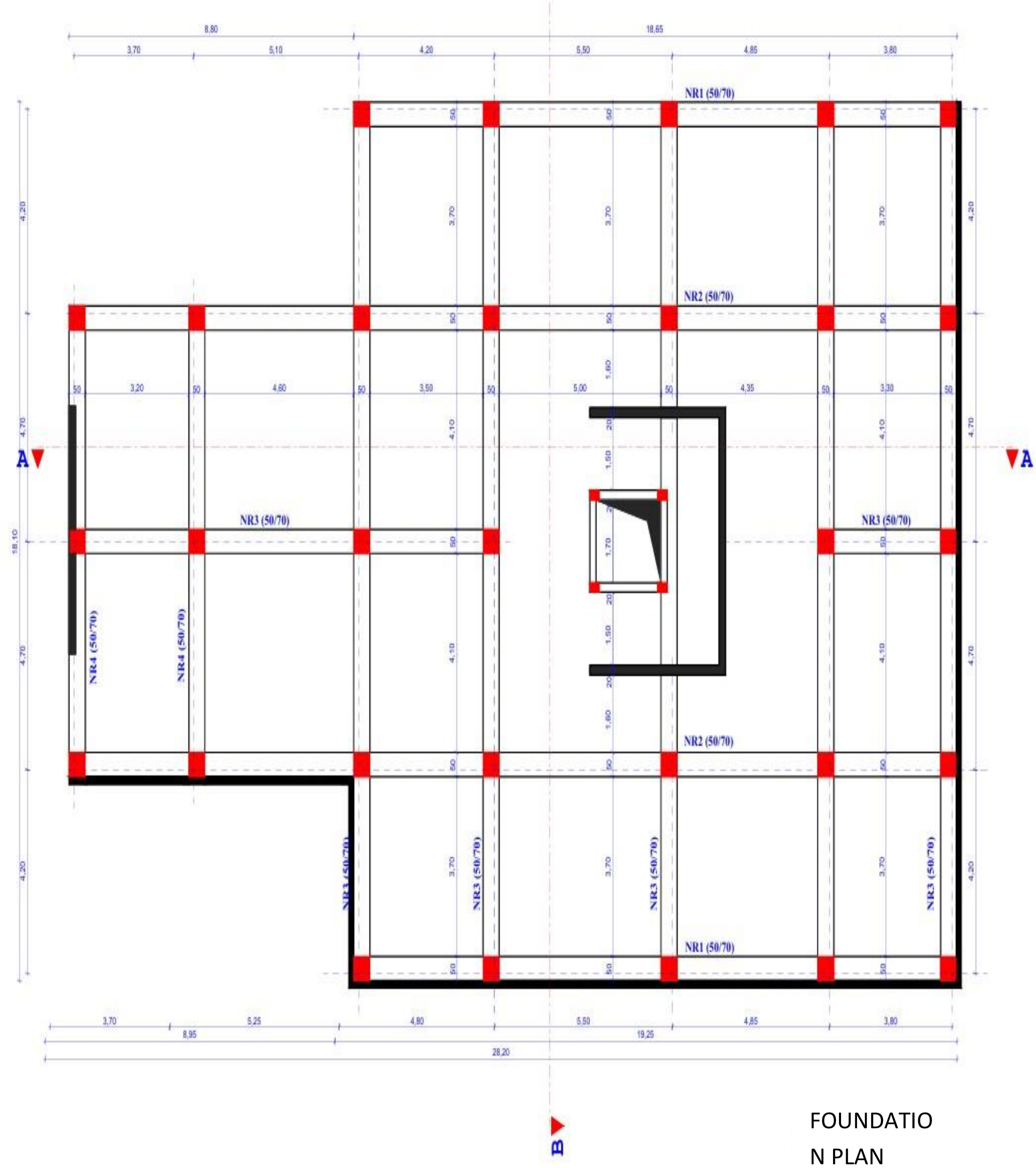
SIDE FACADE


+34,35
+31,29
+28,23
+25,17
+22,11
+19,05
+15,99
+12,93
+9,87
+6,81
+3,92
+0,48
+0,31
+0,69
+0,18
-3,39

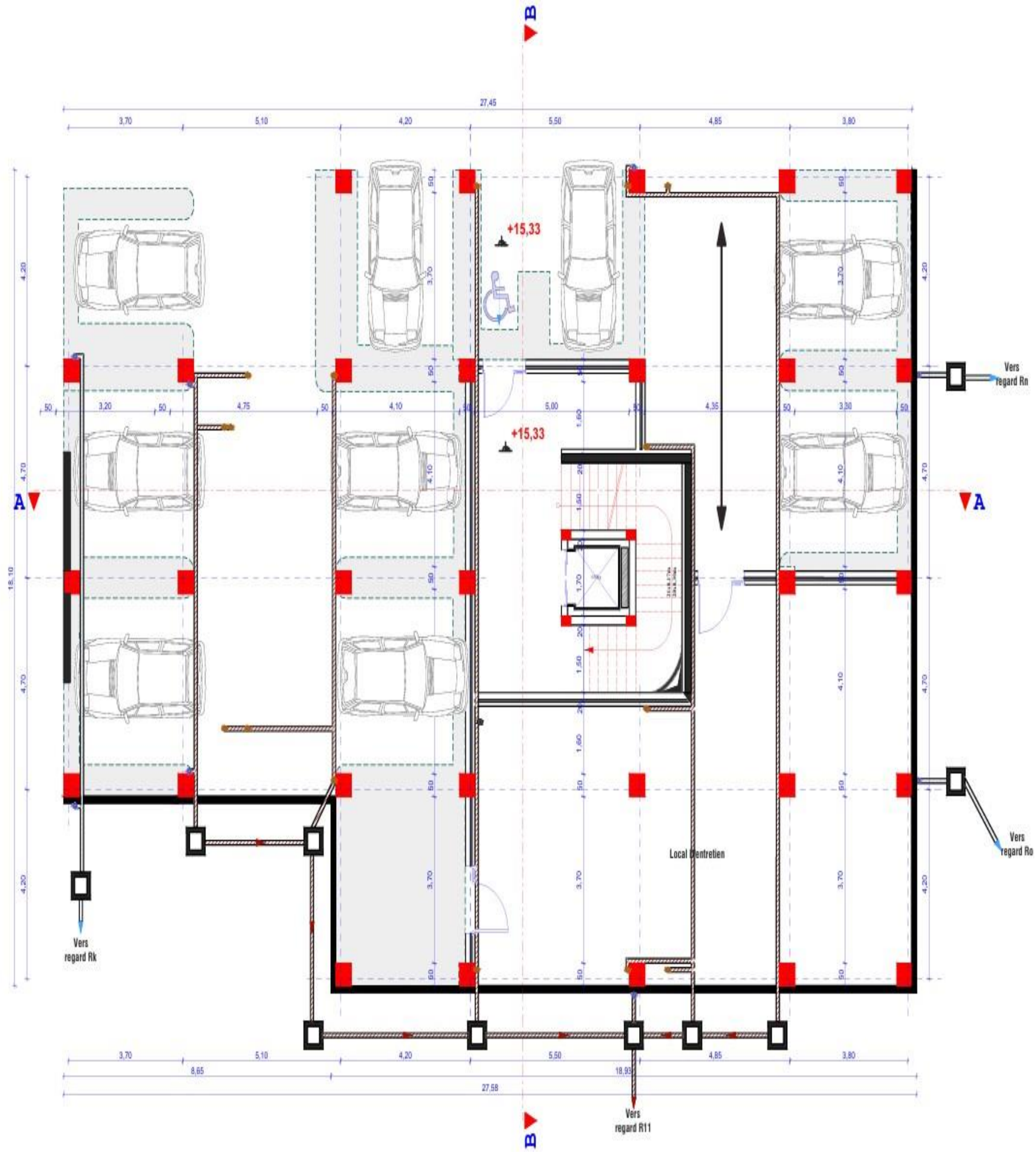
SECTION A-A


+34,85
+34,35
+32,29
+31,29
+28,23
+25,17
+22,11
+19,05
+15,99
+12,93
+9,87
+6,81
+3,92
ACCES VERS
LOGEMENT
±0,00
+0,18
-3,39
VOIE B
+0,69
+0,46
+0,35

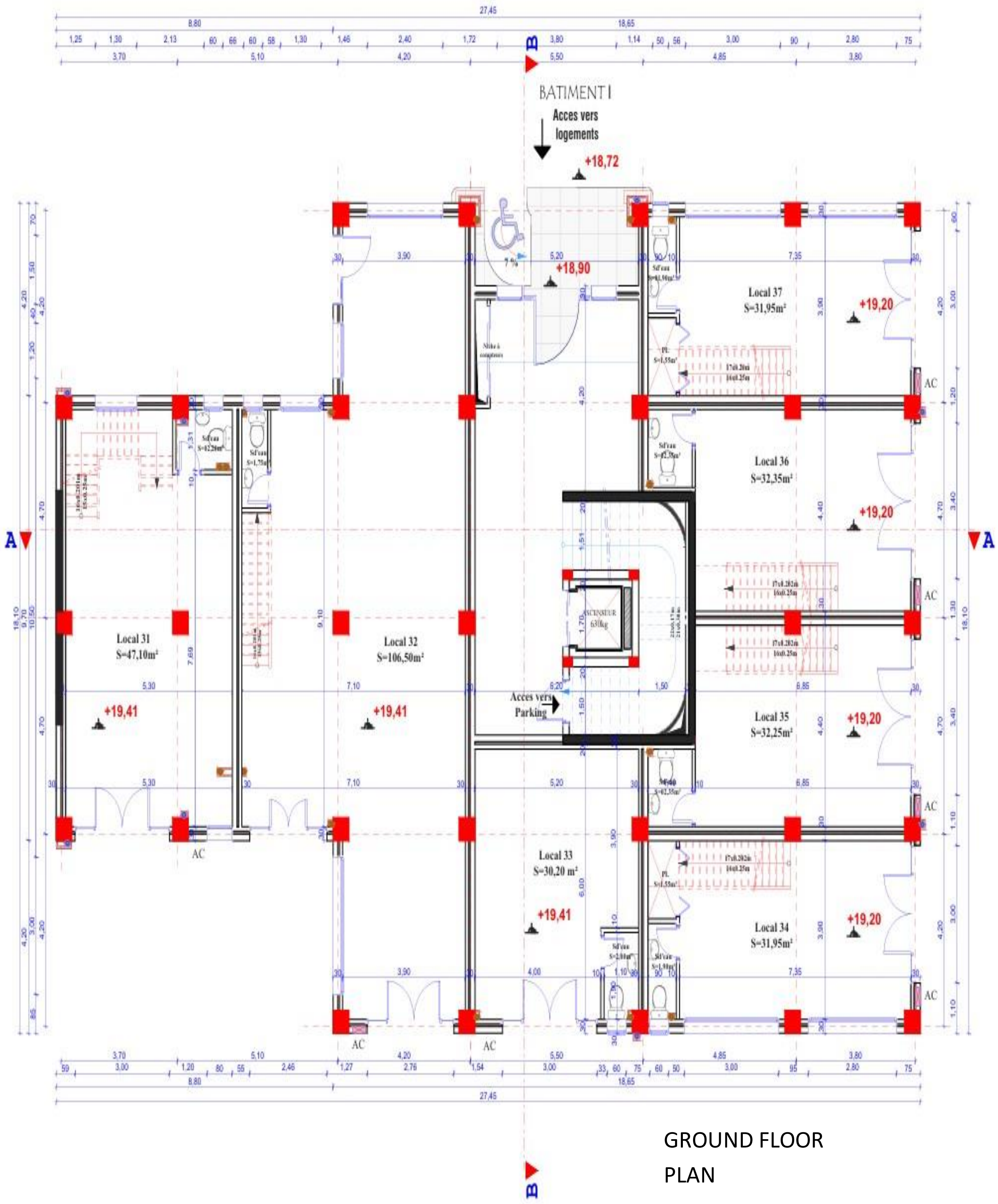
SECTION B-B



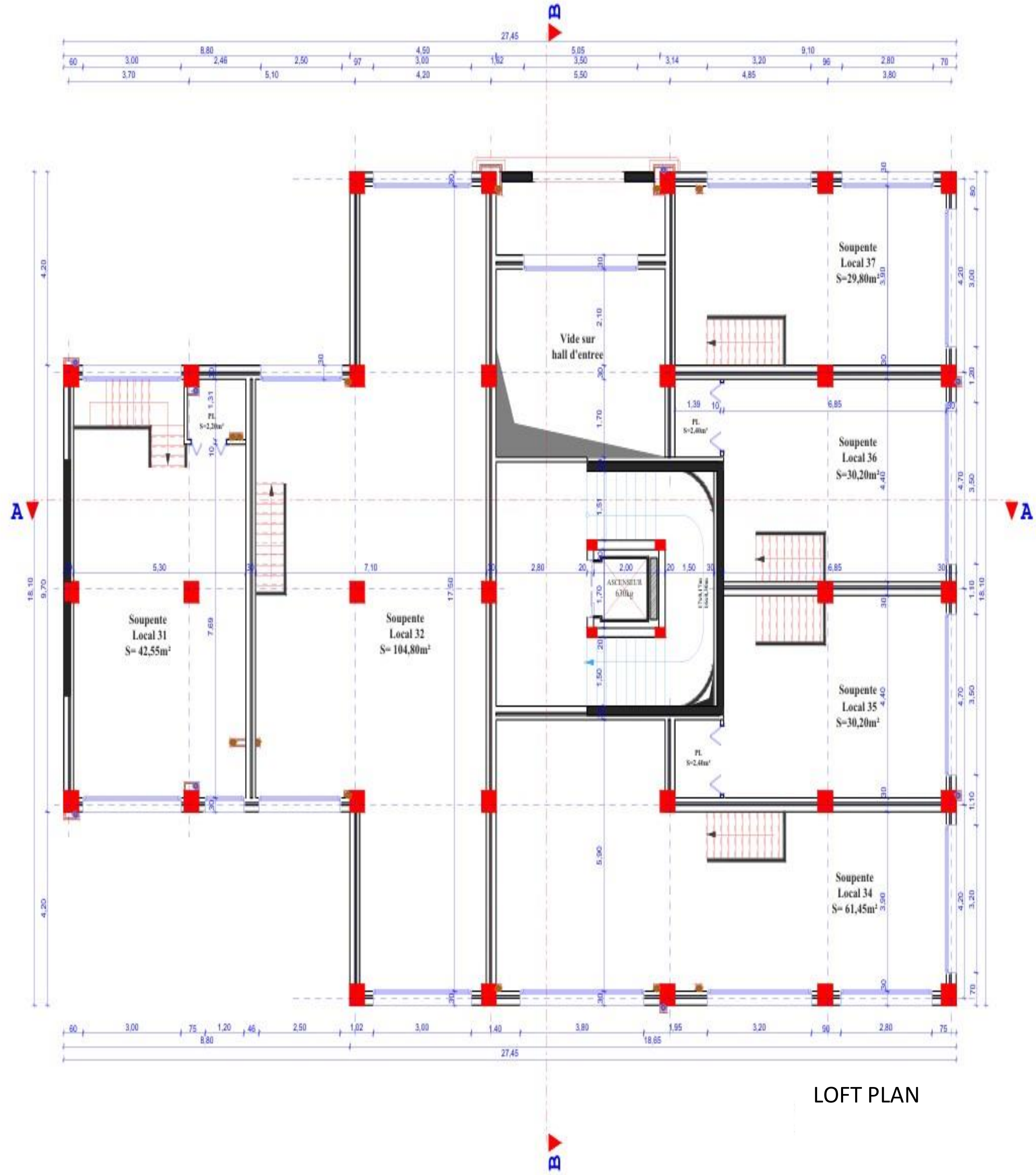
FOUNDATIO
N PLAN



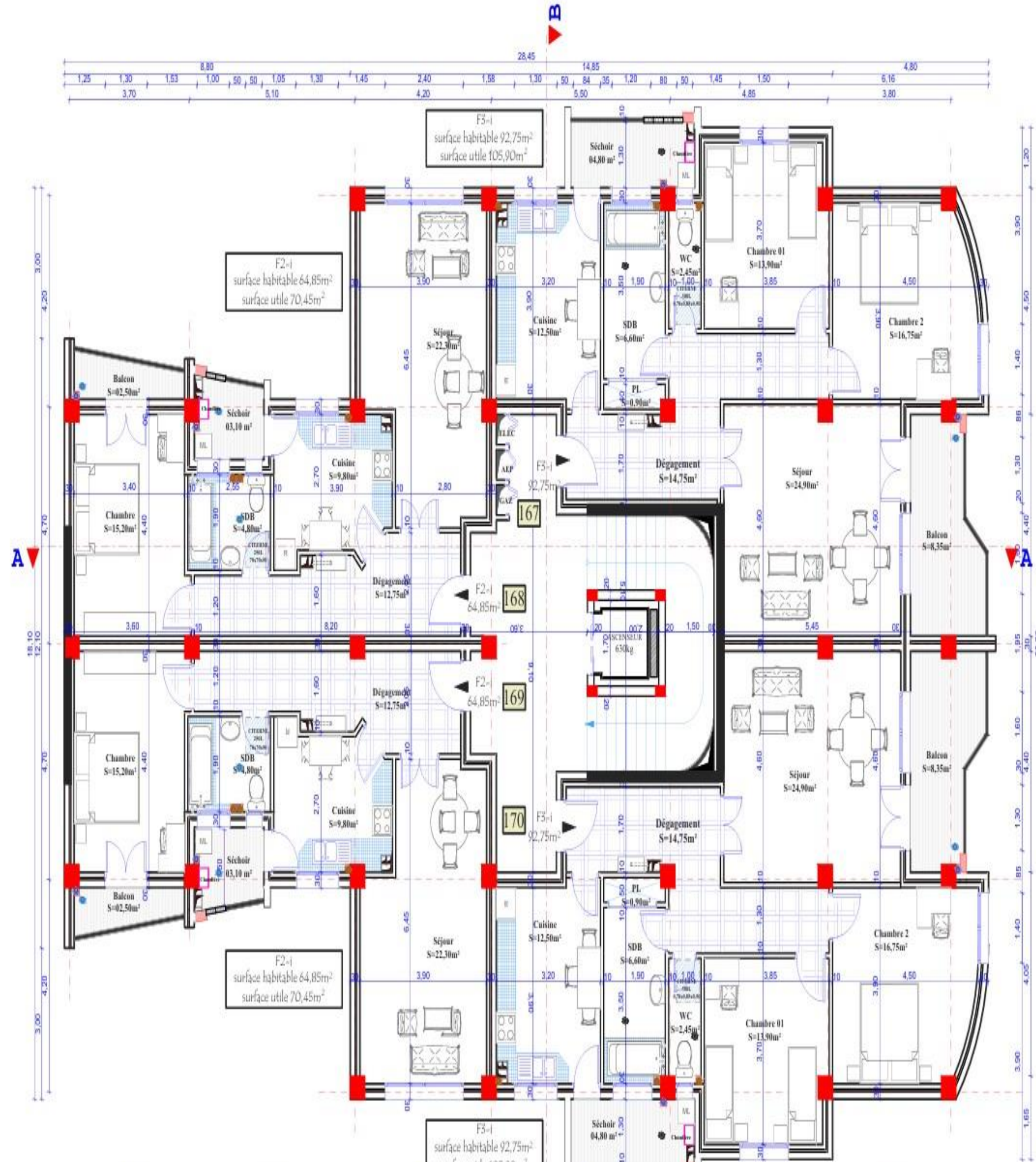
BASEMENT
PLAN



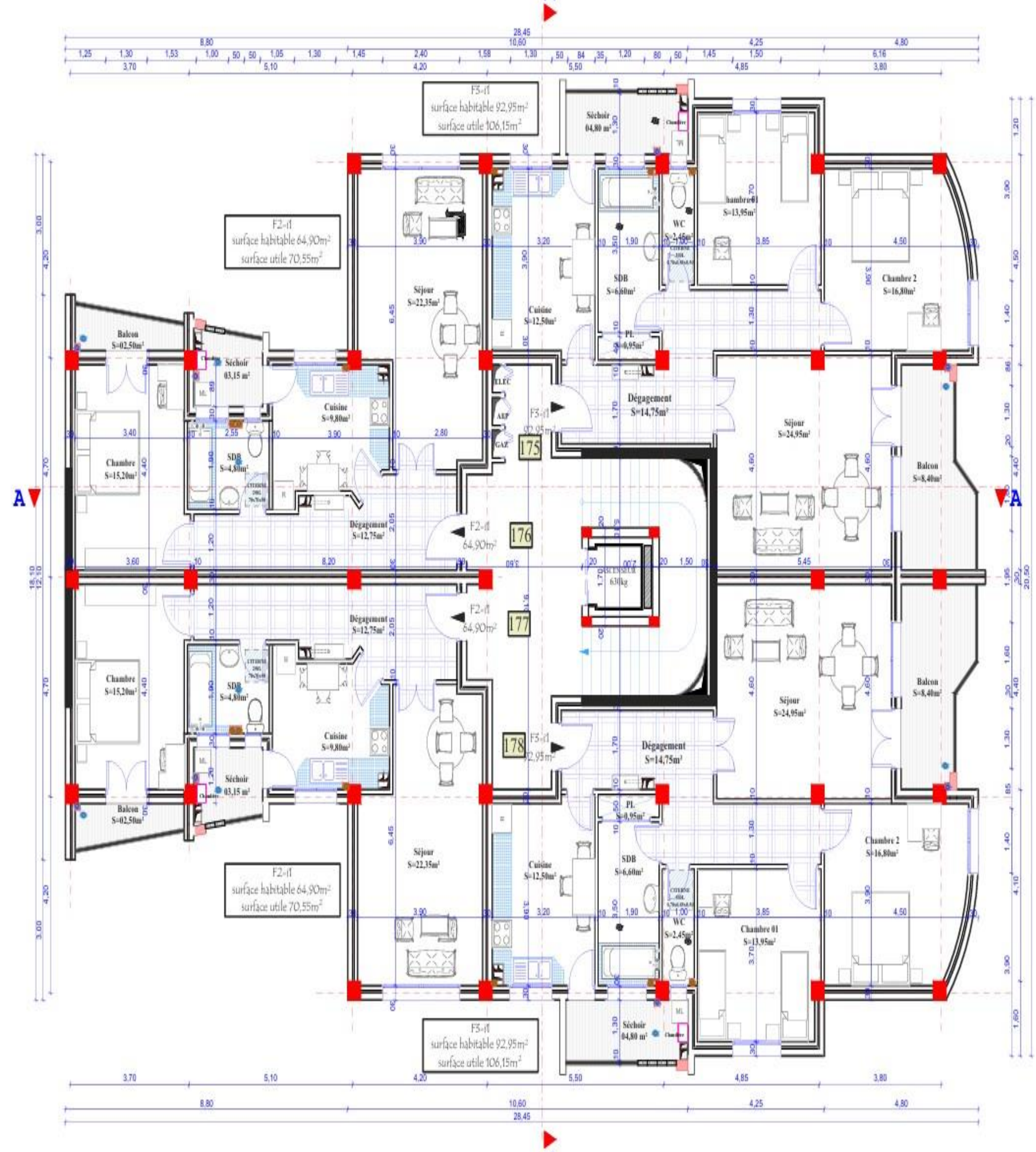
GROUND FLOOR
PLAN


Soupente
Local 37
S=29,80m²
Vide sur
hall d'entree
Soupente
Local 36
S=30,20m²
ASCENSEUR
630kg
Soupente
Local 31
S= 42,55m²
Soupente
Local 32
S= 104,80m²
Soupente
Local 35
S=30,20m²
Soupente
Local 34
S= 61,45m²
27,45
A
B

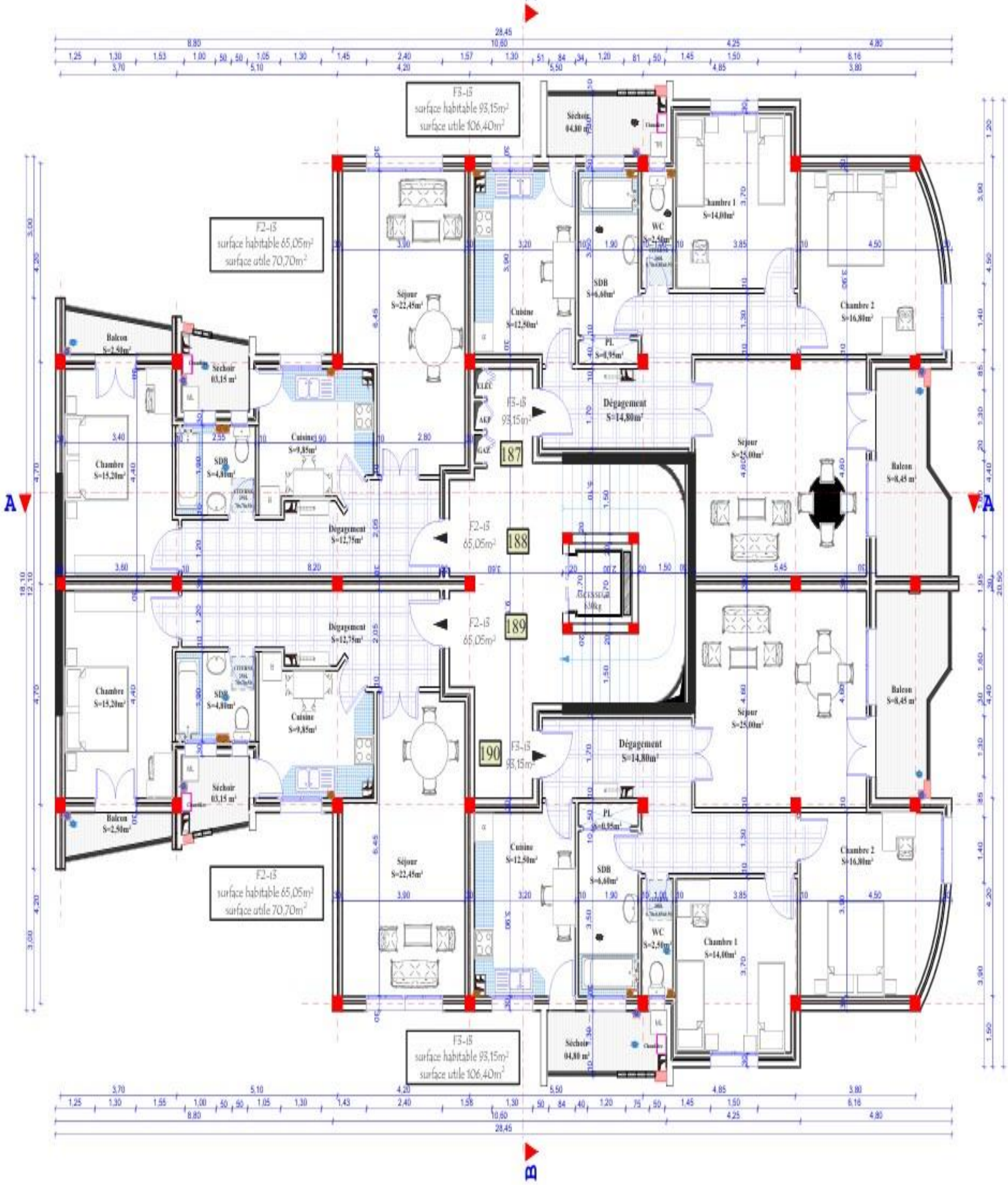
LOFT PLAN



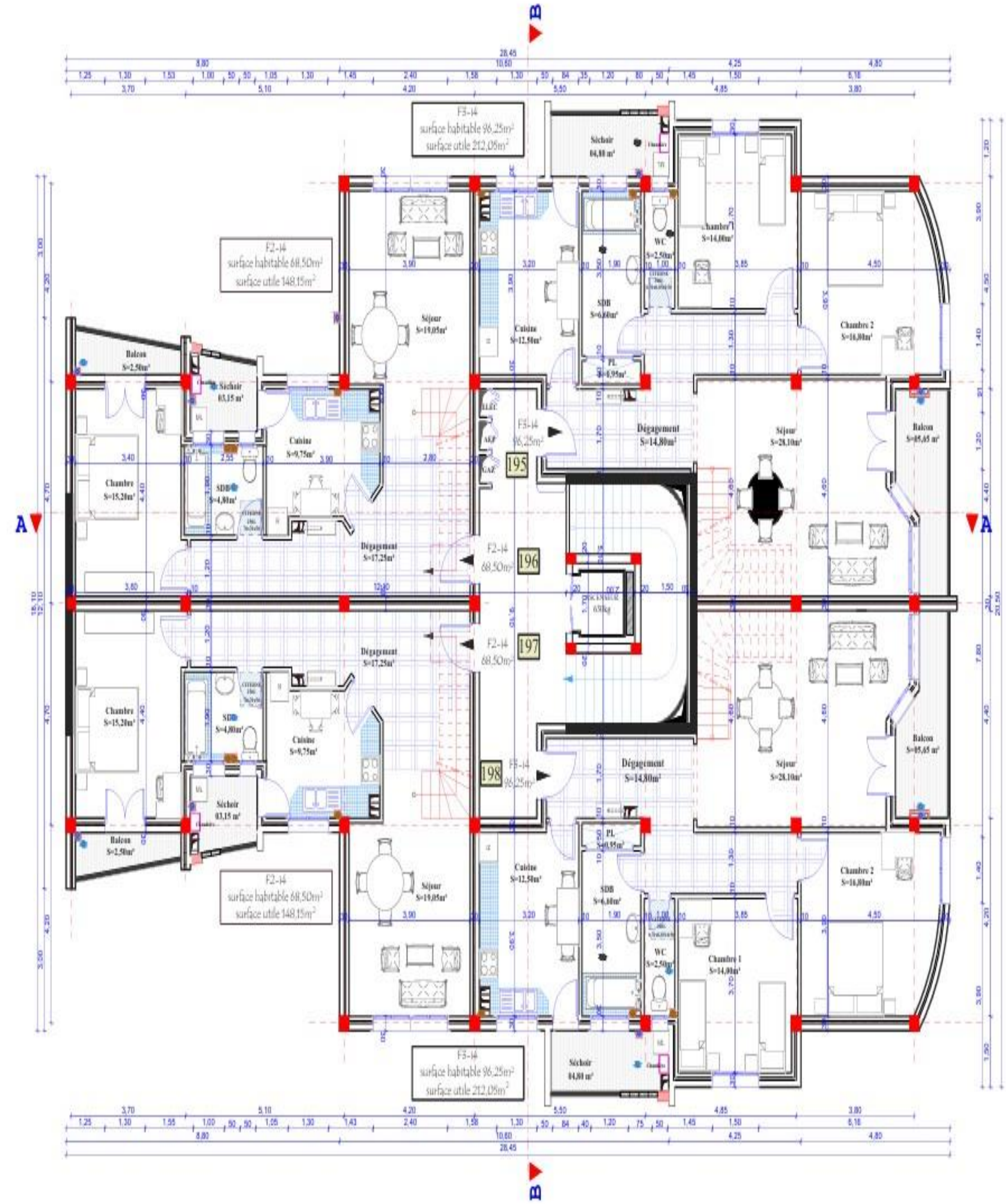
1st AND 2nd
FLOOR PLAN



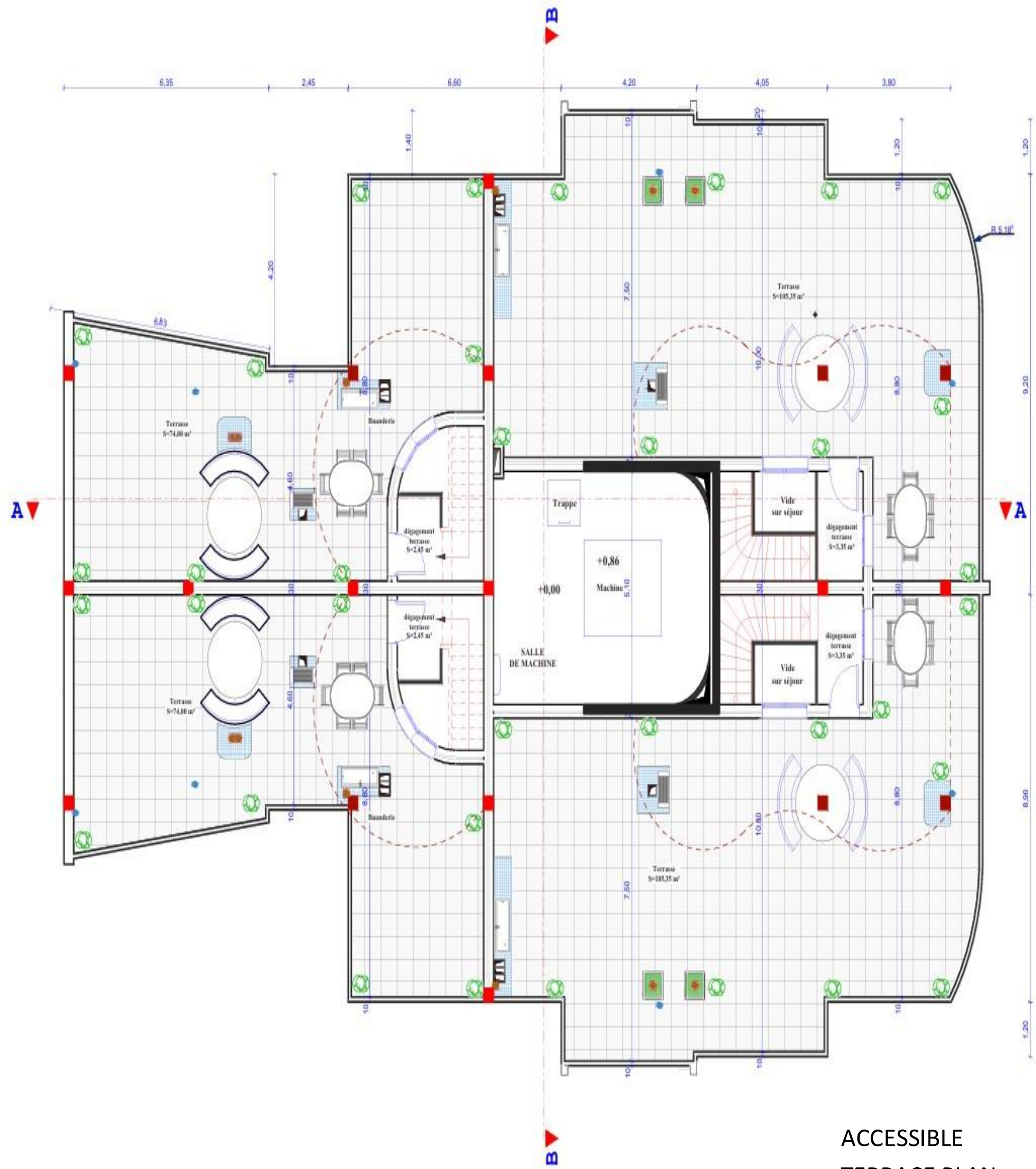
3rd ,4th AND 5th
FLOOR PLAN



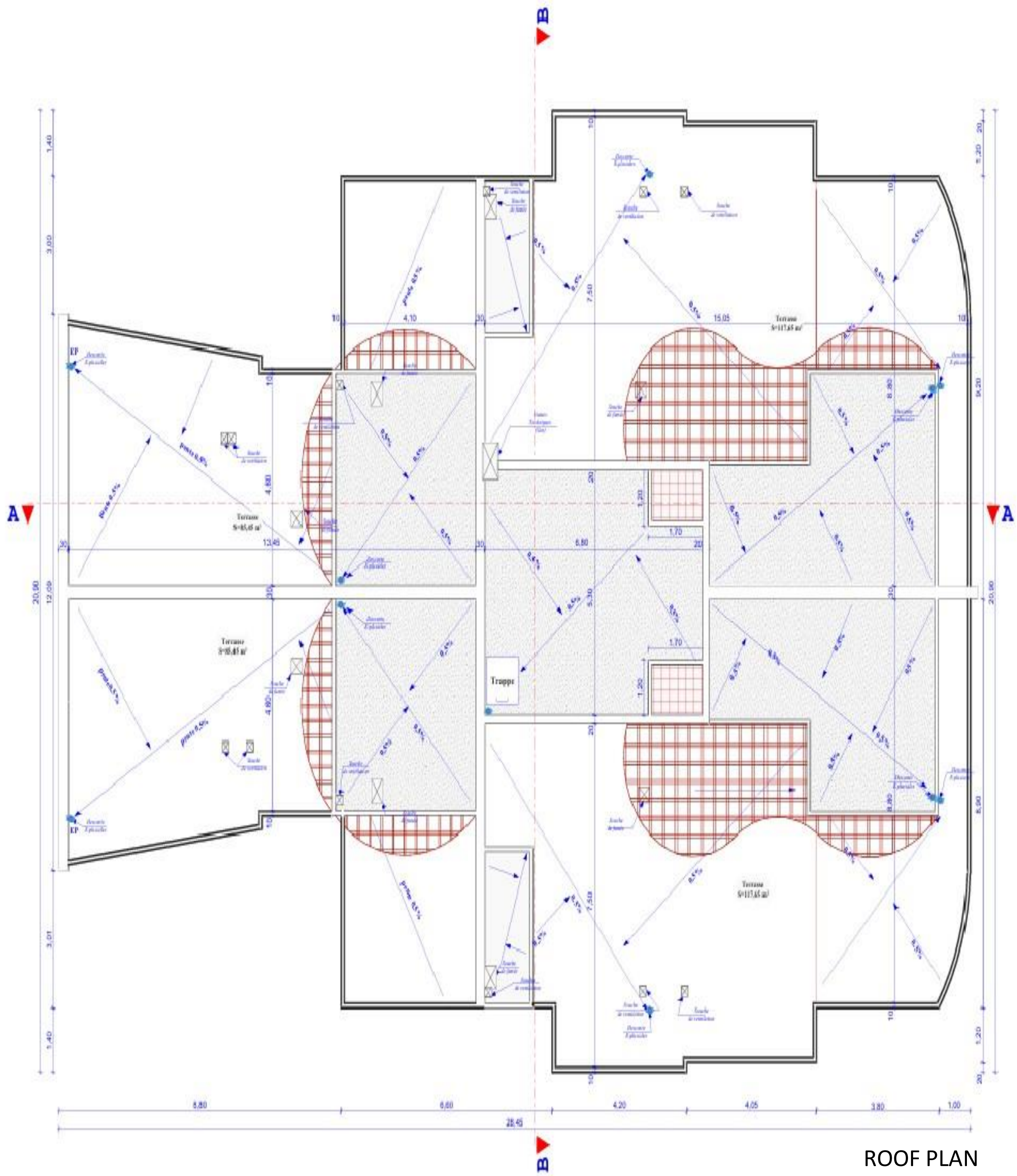
6th AND 7th FLOOR PLAN



8th FLOOR PLAN



ACCESSIBLE TERRACE PLAN


B
A
A
B
Terrasse
S=85,65 m²
Terrasse
S=117,65 m²
Trappe
8,80
6,60
4,20
4,05
3,80
1,00
28,45
20,90
15,05
13,45
6,80
4,10
7,50
5,30
1,70
1,20
0,5%

ROOF PLAN

Abstract

This study, representing the final phase of our training, focused on analyzing and designing a multi-use building structure reinforced by a mixed bracing system. The building, which integrates commercial and residential functions, required a robust structural system to ensure safety, stability, and adaptability. A combination of reinforced concrete frames and shear walls was adopted to form the mixed bracing system, effectively resisting both vertical and lateral loads. Using ETABS software, we evaluated the structural behavior under gravity and seismic forces, ensuring compliance with relevant codes and standards, including the DTR, RPA 99 (2003 version) and BAEL 99. This project allowed us to apply and deepen the theoretical knowledge gained during our course while familiarizing ourselves with current regulations. The findings demonstrate the effectiveness of the mixed bracing system in improving the overall performance and resilience of high-rise, multi-purpose structures.

Key words: Reinforced concrete – Bracing - Shear walls – Mixed bracing system – Etabs - Response spectrum - RPA - Modeling - Rebar design.

Résumé

Cette étude, qui représente la phase finale de notre formation, a porté sur l'analyse et la conception d'une structure de bâtiment à usage mixte renforcée par un système de contreventement mixte. Le bâtiment, qui intègre des fonctions commerciales et résidentielles, nécessitait un système structural robuste pour garantir la sécurité, la stabilité et l'adaptabilité. Une combinaison des portiques en béton armé et de murs voiles a été adoptée pour former le système de contreventement mixte, résistant efficacement aux charges verticales et latérales. À l'aide du logiciel ETABS, nous avons évalué le comportement structurel sous l'effet des charges gravitationnelles et sismiques, en veillant au respect des normes et réglementations en vigueur, notamment le DTR, le RPA 99 (version 2003), et le BAEL 99. Ce projet nous a permis d'appliquer et d'approfondir les connaissances théoriques acquises durant notre formation, tout en nous familiarisant avec les réglementations actuelles. Les résultats démontrent l'efficacité du système de contreventement mixte dans l'amélioration des performances globales et de la résilience des structures de grande hauteur à usage multiple.

Mots clés : Béton armé - Contreventement – Voiles – Contreventement mixte – Etabs – Spectre de réponse – RPA – Modélisation – Ferraillage.

ملخص

تناولت هذه الدراسة، التي تمثل المرحلة النهائية من مشوارنا الدراسي، تحليل وتصميم هيكل مبنى متعدد الاستخدامات معزز بنظام تثبيت مختلط. كان المبنى، الذي يضم وظائف تجارية وسكنية، بحاجة إلى نظام هيكلي قوي لضمان السلامة والاستقرار والقدرة على التكيف. تم اعتماد مزيج من الأعمدة الخرسانية المسلحة و الجدران القصية لتشكيل نظام التثبيت المختلط، الذي يقاوم بشكل فعال الأحمال الرأسية والجانبية. باستخدام برنامج ETABS، قمنا بتقييم السلوك الهيكلي تحت تأثير الأحمال الجاذبية والزلزالية، مع ضمان الامتثال للمعايير واللوائح المعمول بها، بما في ذلك DTR و RPA 99 (إصدار 2003) و BAEL 99. سمح لنا هذا المشروع بتطبيق وتعميق المعرفة النظرية التي اكتسبناها خلال تدريبنا، مع التعرف على اللوائح الحالية. تظهر النتائج فعالية نظام التثبيت المختلط في تحسين الأداء العام ومرونة الهياكل الشاهقة متعددة الاستخدامات.

الكلمات المفتاحية : الخرسانة المسلحة – التثبيت الجانبي – الجدران القصية – نظام التثبيت المختلط – إيتابس – طيف الاستجابة – RPA – النمذجة– تصميم حديد التسليح" -