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to the new Algerian regulation RPA 2024**

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Dedication

Praise to God alone

This thesis dedicated to

Myself

I have never stopped believing in my dreams. The path to this achievement has been marked by perseverance, discipline, and resilience. Guided by faith in God and determination, I continued to move forward despite the challenges.

My dear mother

Thank you for endless love, patience and support. Your encouragement and sacrifices have been a constant source of strength throughout my journey. With love, respect, and gratitude, I dedicate this work to you.

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Dedication

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ملخص

يتناول هذا البحث دراسة وتحليلاً تقنياً لمنشأ من الخرسانة المسلحة من نوع R+8 (طابق أرضي وثمانية طوابق)، يقع في منطقة ذات نشاط زلزالي مرتفع (المنطقة V) بهدف هذا العمل إلى تقييم السلوك الإنشائي للمنشأ وضمان سلامته تحت تأثير الأحمال الرأسية والتأثيرات الزلزالية.

تم اعتماد نظام إنشائي مختلط يجمع بين الجدران الخرسانية المسلحة لمقاومة القوى الأفقية وتحسين صلابة المنشأ، والإطارات الخرسانية المسلحة لنقل الأحمال الرأسية نحو الأساسات. تم تصميم مختلف العناصر الإنشائية وفق متطلبات 91 BAEI و 93 CBA مع احترام أحكام 2024 RPA.

تمت نمذجة وتحليل المنشأ باستخدام برنامج ETABS 2021 من خلال التحليل الساكن الخطي والتحليل الديناميكي الطيفي. كما تم اعتماد اللبشة المعرّقة كأساس للمنشأ لضمان توزيع مناسب للأحمال وتقليل مخاطر الهبوطات التفاضلية.

أظهرت النتائج أن المنشأ المصمم يحقق شروط المقاومة والاستقرار، مع ضمان أداء زلزالي ملائم وفق المعايير المعتمدة.

الكلمات المفتاحية: R+8، الخرسانة المسلحة، 2024 RPA، ETABS 2021، التحليل الزلزالي، الأساس، النظام المختلط، اللبشة المعرّقة.

Résumé

Ce mémoire de fin d'études porte sur l'étude technique approfondie d'un bâtiment en béton armé de type R+8 (rez-de-chaussée + 8 étages), implanter dans une zone à sismicité élevée (zone V). L'objectif principal est de grandir un comportant structural ductile et stable sous l'effet des charges verticales et des sollicitations sismiques. Le système de contreventement adopté est mixte, associant des voiles en béton armé (pour la reprise des efforts horizontaux) à des portiques (poteaux-poutre) pour le support des charges gravitaires. Le dimensionnement des éléments (poteaux, poutre, voiles, fondations) a été effectué conformément aux règles BAEI 91 (béton armé aux états limites) et au CBA 93, tout en respectant strictement les prescriptions du Règlement parasismique Algérien (RPA 2024). Les analyses statiques linéaire et dynamique modale spectrale ont été réaliser via le logiciel ETABS 2021. Une fondation de type radier nervuré a été privilégiée pour assurer une répartition homogène des charges sur le sol, limitant ainsi les tassements différentiels. Les résultats obtenus montrant que la structure respecte les critères de stabilité, de déplacement inter étages et de résistance imposée par la norme parasismique récent.

Mots clés : R+8, béton armé, RPA 2024, ETABS 2021, analyse sismique, radier, contreventement mixte.

Abstract

This master's thesis presents a comprehensive structural study of an 8-story reinforced concrete residential building (R+8) located in a high seismic zone (zone V). The primary objective is to insure ductile and stable structural system incorporates a mixed lateral load resisting system, combining reinforced concrete shear walls (to resist horizontal forces) and moment resisting frames (columns and beams) for vertical load transfer. The structural design was performed in strict accordance with the Algerian concrete code (CBA 93/BAEI 91) and the updated Algerian seismic design code (RPA 2024). Linear static and dynamic response spectrum analyses were carried out using ETABS 2021. A raft foundation was selected to ensure uniform load distribution onto the soil and minimize differential settlements. The study concludes that the

proposed structure successfully meets the stability, drift, and strength requirements mandated by the latest seismic regulations.

Key words: R+8, reinforced concrete, RPA 2024, ETABS 2021, seismic analysis, mat foundation, shear wall.

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List of notation and symbols

- **A**: zone acceleration coefficient
- α_s : Area of steel section (reinforcement area)
- A_t : Area of transverse reinforcement
- **B**: Area of concrete section
- ϕ : diametere of reinforcement
- φ : Angle of internal friction
- **C**: Cohesion
- **Q**: live load (or exploitation load)
- γ_s : Partial safety factor for steel
- γ_b : Partial safety factor for concrete
- σ_s : Tensil stress in steel
- σ_{bc} : Compressive stress in concrete
- $\overline{\sigma}_s$: Allowable tensile stress in steel
- $\overline{\sigma}_{bc}$: Allowable compressive stress in concrete
- τ_u : Ultimate shear stress
- τ : Tangential shear stress
- β : distribution coefficient (or load distribution factor)
- σ_{sol} : Soil pressure (or soil stress)
- σ_m : Average stress
- **G**: Dead load (Permanent load)
- V_0 : Base shear force
- **U. L. S**: Ultimate limit state
- **S. L. S**: Serviceability limit state
- N_{ser} : Normal force at SLS (Serviceability limit state)
- N_u : Normal force at ULS (Ultimate limit state)
- T_u : Ultimate shear force
- **T**: shear force/period
- S_t : Spacing (of stirrups)
- λ : Slendernes ratio

- **e**: Thickness
- **P**: Concentrated load
- **f**: Deflection
- $\bar{f}$: Allowable deflection
- **L**: Length or span
- **L_f**: Buckling length
- **d**: Effective depth
- **F_e**: Yield strength of steel
- **M_u**: Bending moment at Ultimate limit state (ULS)
- **M_{ser}**: Bending moment at Serviceability limit state (SLS)
- **M_t**: Mid – span bending moment
- **M_a**: Support bending moment
- **M₀**: Isostatic bending moment
- **I**: Moment of inertia
- **f_i**: Instantaneous deflection (due to instantaneous loads)
- **M**: Moment/Mass
- **E_{ij}**: Delayed (or long – term) modulus of elasticity
- **E_s**: Modulus of elasticity of steel
- **f_{c28}**: Characteristic compressive strength of concrete at 28 days
- **f_{t28}**: Characteristic tensile strength of concrete at 28 days
- **f_{cj}**: Characteristic compressive strength of concrete at j days
- **Y**: Position of the neutral axis
- **I₀**: Moment of inertia of the total homogeneous section

Introduction

Introduction

As part of the Master's degree curriculum in Civil Engineering, the preparation of the final year dissertation represents a significant milestone, serving as a crucial link between the strong theoretical knowledge acquired throughout the academic journey and the complex challenges encountered in real-world structural engineering practice. This work aims to demonstrate the candidate's ability to synthesize knowledge, apply advanced computational tools, and adhere to stringent regulatory codes. The present study focuses on the comprehensive structural analysis and seismic design of a residential building, comprising a ground floor and eight stories (R+8).

The dissertation is structured into six logically sequenced chapters:

Chapter I provides a comprehensive presentation of the architectural and geometric characteristics of the project. This includes the site location, the structural system layout, and the definition of the building's dimensions, serving as the foundation reference for subsequent calculations.

Chapter II is dedicated to the preliminary design phase. It details the pre-dimensioning of all primary structural members such as columns, beams, and slabs along with the meticulous evaluation and calculation of the applied loads acting on the structure.

Chapter III focuses on the analysis and design of secondary structural elements. This chapter covers the calculation and reinforcement detailing of non-primary components, including balconies, staircases, and floor slabs, in strict accordance with the BAEL 91 standards.

Chapter IV represents the core and the most critical phase of this research: the seismic study of the structure. This chapter is devoted to the dynamic and static seismic analysis of the building, utilizing the powerful finite element software ETABS 2021, and applying the latest Algerian seismic design code (RPA 2024). This analysis ensures the building's integrity against horizontal seismic forces.

Chapter V addresses the detailed structural design of the primary load bearing elements. It presents the calculation of the required reinforcement (steel detailing) for the columns, main beams, and shear walls, verifying that all members satisfy the limit states and ductility requirements of the codes.

Chapter VI, the final chapter, presents the foundation design. It includes the geotechnical assessment of the soil characteristics, the selection of the appropriate foundation type (a mat foundation / raft foundation is assumed based on load distribution), and the verification of the foundation's stability and safety under the transmitted structural loads.

By integrating the advanced capabilities of ETABS 2021 with the rigorous requirements of the RPA 2024 code, this thesis aims to produce a safe, stable, and economically efficient structural design for a high-rise building (R+8) in a seismic zone.

Chapter I:

Presentation of the structure

I.1 Introduction

The structural design of the reinforced concrete frame for the eight-story residential building in Lesnam, Bouira is governed by the Algerian seismic code RPA 2024 and the reinforced concrete design standard BAEL 91(modified 99). This chapter defines the key material properties, constitutive models, and partial safety factors adopted throughout the analysis and design. All parameters are selected to satisfy the dual requirement of ultimate state performance under everyday loading.

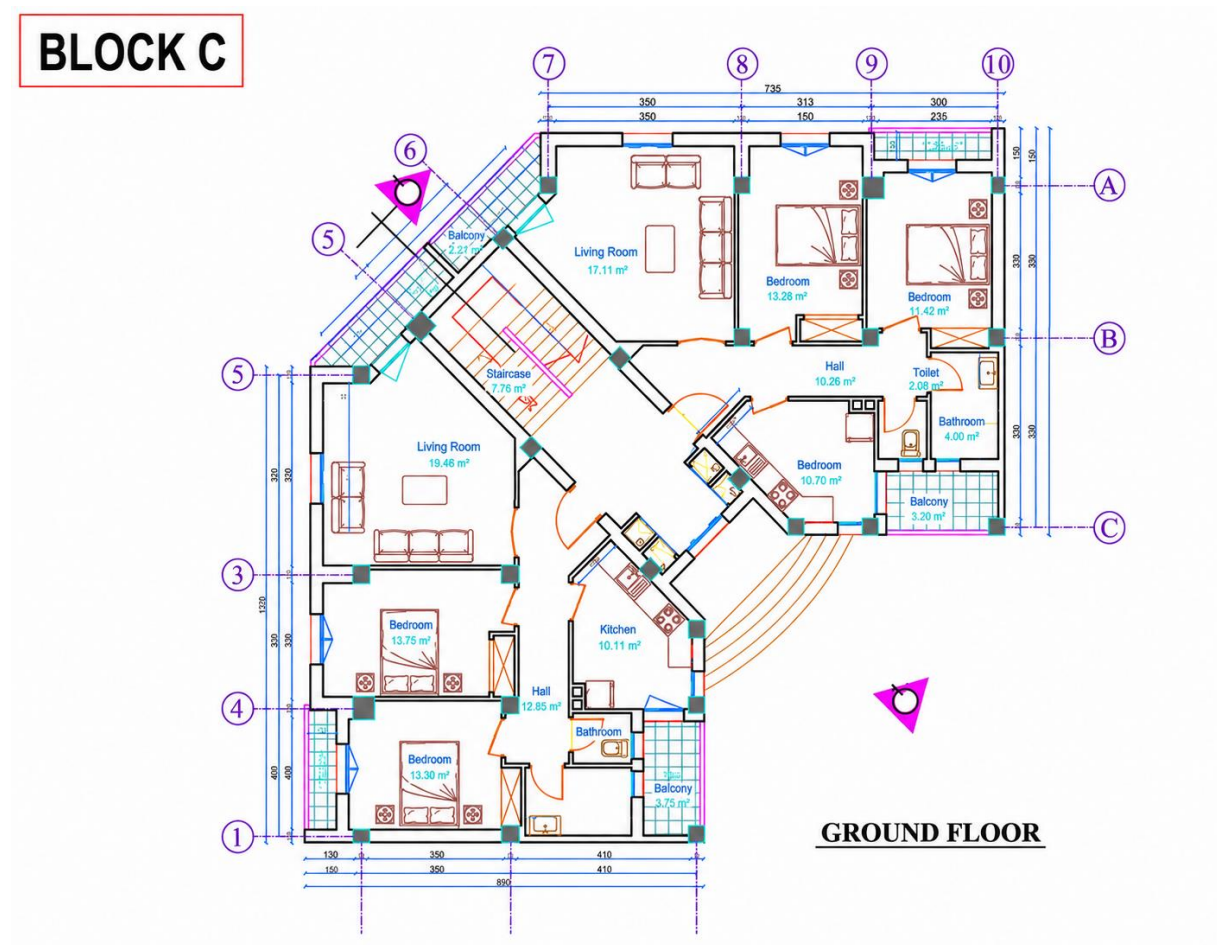


Figure I. 1: Plan view of ground floor.

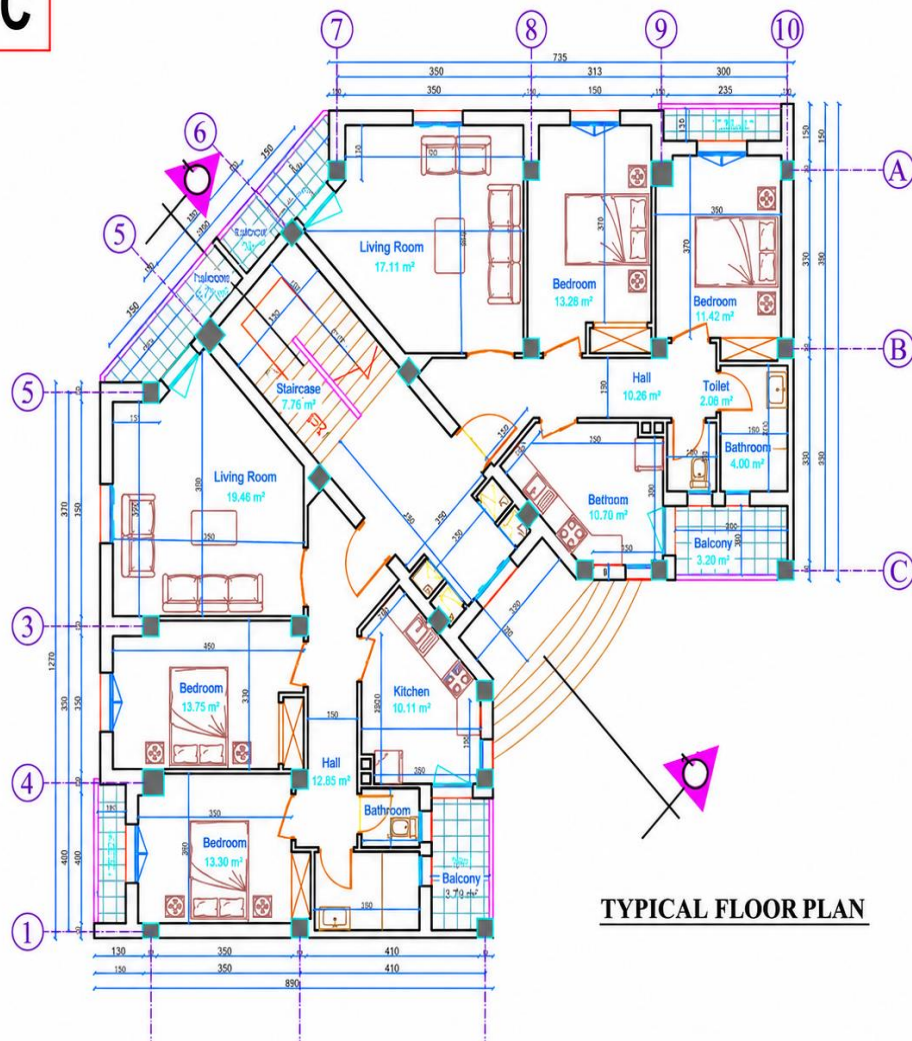
BLOCK C

Figure I. 2: Plan view of typical floors.

BLOCK C

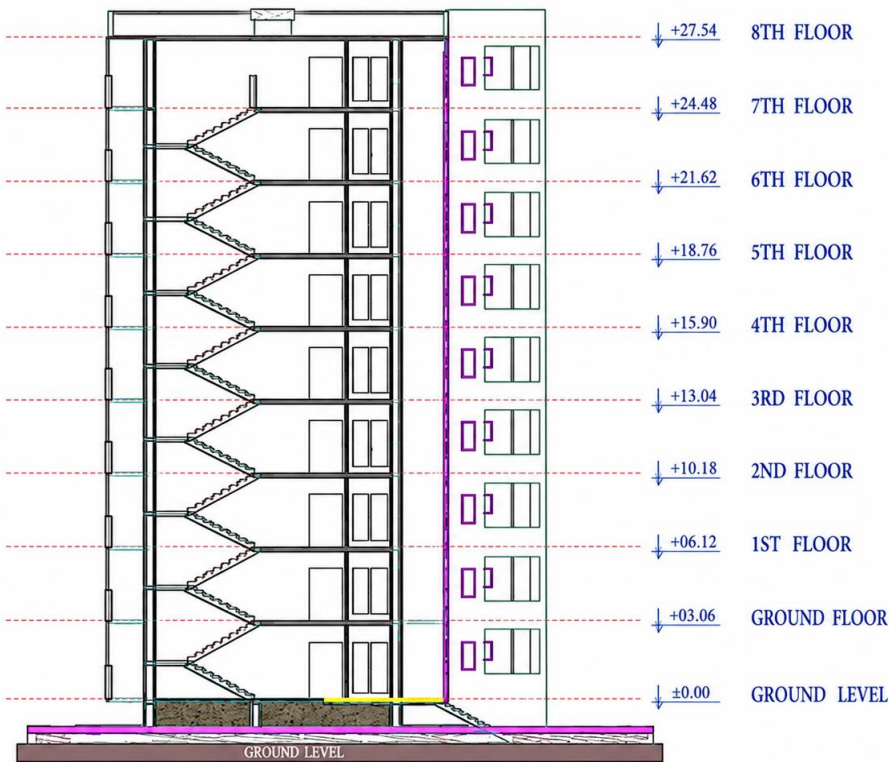


Figure I. 3: Cross-section view of the building.

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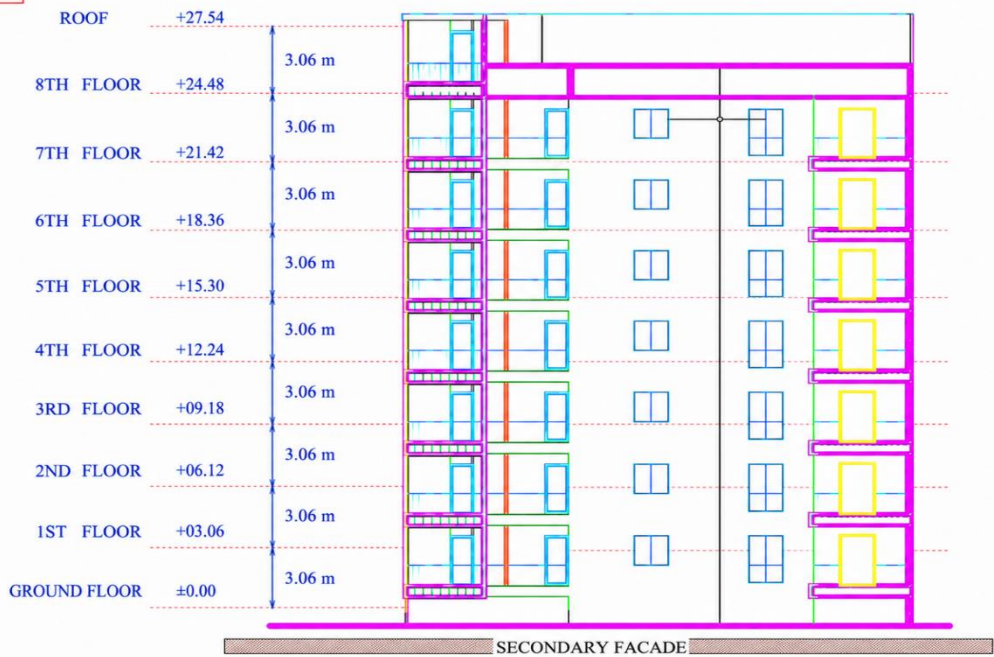


Figure I. 4: Structural elevations of the building

I.2 The geometric characteristics

- Ground floor height: 3.06m
- The height of the current floors: 3.06m
- Total height: 27.54m
- The length: 23.56 m
- The width: 14.70 m

I.3 Material characteristics

I.3.1 Concrete

❖ Compressive strength: (A.2.1.11) BAEL91

The concrete is defined by its compressive strength at 28 days, known as the specified characteristics strength. This value denotes by f_{c28} .

For $f_{c28}=25$ MPa

When load is applied to concrete at an age (during the construction phase) that is less than 28 days, this value denotes f_{cj} .

$$f_{cj} = \frac{j}{4.76+0.83j} \times f_{c28} \quad \text{if: } f_{c28} < 40 \text{ MPa}$$

$$f_{cj} = \frac{j}{1.4+0.95j} \times f_{c28} \quad \text{if: } f_{c28} > 40 \text{ MPa}$$

❖ Tensile strength: (A.2.1.12) BAEL91

The tensile strength of concrete, represented as f_{td} at an age of d days, is determined by this equation: $f_{td}=0.6+0.06(f_{cj})$

$$\text{For } f_{c28} = 25 \text{ MPa} \quad f_{t28}=2.1 \text{ MPa}$$

❖ Longitudinal concrete strains: (A.2.1.2) BAEL91

• The instantaneous longitudinal deformation modulus

When normal stresses are applied less than 24 hours. The instantaneous longitudinal deformation modulus of concrete at j days calculated by this expression:

$$E_{ij}=11000\sqrt[3]{f_{c28}} \quad E_{ij}=32164.19512 \text{ MPa}$$

• Delayed deformation modulus: (A.2.1.21) BAEL91

For stresses of long-term application, the delayed deformation modulus E_{vj} at j :

$$E_{vj}=3700\sqrt[3]{f_{c28}} \quad E_{vj}=10818.86563 \text{ MPa}$$

• The poison's ratio: (A.2.1.3) BAEL91

In stress analyses, the poison's ratio for concrete is assumed to be zero a, whereas of value of 2 is adopted for deformation analysis

$$\nu = \begin{cases} 0 & ULS \\ 0.2 & SLS \end{cases}$$

- **Stress-strain diagram of concrete (A4.3.4) BAEL91**

The parabolic-rectangle diagram is the universal stress-strain model for concrete design. It combines a quadratic arc (from the original to peak stress) with a horizontal segment tangent to the parabola, spanning strains from 2‰ to 3.5 ‰ .

- **Stress-strain diagram of concrete (at ultimate limit state ULS)**

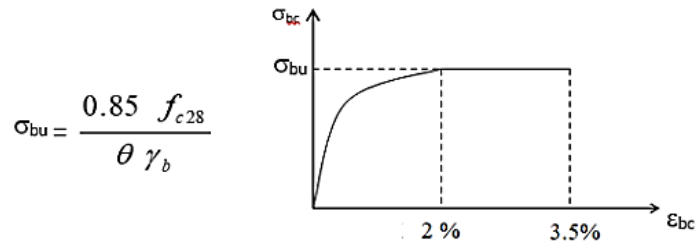


Figure I. 5: Stress-strain diagram of concrete at ULS.

$$\gamma_b = \begin{cases} 1.5 & \text{Persistent design situation} \\ 1.2 & \text{Accidental design situation} \end{cases}$$

$$\theta = \begin{cases} 1 & T > 24h \\ 0.9 & 1h < T < 24h \\ 0.85 & T < 24h \end{cases}$$

- **Stress-strain diagram of concrete (at serviceability limit state SLS):**

For this an analysis:

$$\begin{aligned} \sigma_{bc} &= 0.6 f_{c28} \\ \sigma_{bc} &= 0.6 \times 25 \\ \sigma_{bc} &= 15 \text{ MPa} \end{aligned}$$

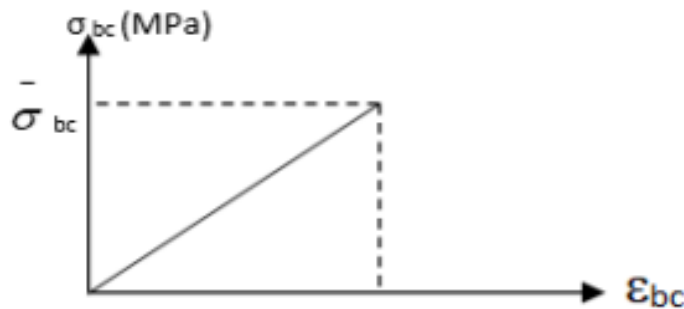


Figure I. 6: Stress-strain diagram of concrete at SLS.

I.3.2 Steel

❖ Definition

Steel is an iron carbon alloy with a low carbon content. Its role is to absorb tensile, shear, and torsional forces. Two types of steel are distinguished:

- Mild or semi-hard steels: 0.15% – 0.25% carbon.
- Hard steels: 0.25% – 0.4% carbon.

The longitudinal modulus of elasticity of steel is taken as:

$$E_s = 200000 \text{ MPA}$$

❖ Mechanical characteristics

Values of the guaranteed yield strength f_e .

Table I. 1: Types and characteristics of steel.

Type	Grade	f_e (MPa)	Use
Plain rounds	FeE22	215	Common use
	FeE24	235	Lifting pin for precast concrete elements
HA bars type 3 (high-adherence)	FeTE40	400	Common use
	FeTE50	500	Common use
HA draws wires type 3	FeTE40	400	Use as a straight bar or lattice bars
	FeTE50	500	Use as a straight bar or lattice bars
Plain drawn wires type 4	TL50	500	Welded mesh-common use only
	TL52	520	Welded mesh-common use only

Note: in our case, high- adherence reinforcement is used.

Steel type: FeE400, $f_e = 400 \text{ MPA}$

❖ Limit stress

The tensile elastic stresses are given by article A.4.5.3 of BAEL 91:

- Ultimate limit state (ULS):

For calculation, the stress-strain diagram for steel at the ULS is used as follows:

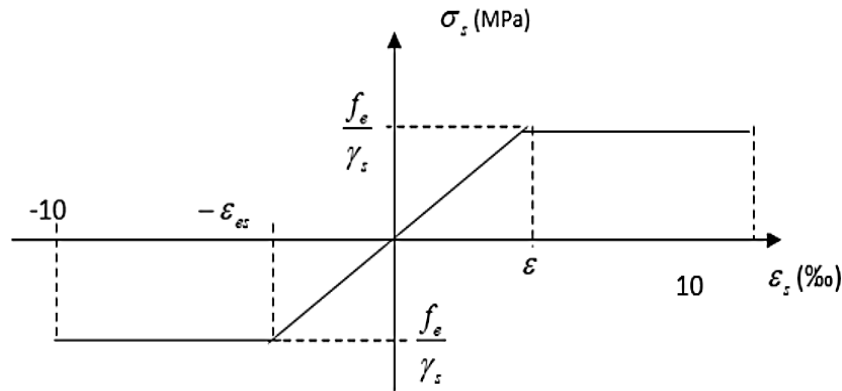


Figure I. 7: Stress-strain diagram of steel at the ULS.

$$\epsilon_{es} = \frac{\sigma_s}{E_s} \text{ with: } E_s = 200000 \text{ MPA}$$

Where E_s is defined as the slope of the line through the origin

γ_s : safety coefficient.

$$\gamma_s = \begin{cases} 1.15 & \text{Presistent design situation} \\ 1 & \text{Accidentel design situation} \end{cases}$$

$$\text{For our case: } \sigma_s = \frac{f_e}{\gamma_s} \rightarrow \begin{cases} 348 \text{ MPA} & S.D.T \\ 400 \text{ MPA} & S.A \end{cases} \quad \text{eq(1.11)}$$

❖ Serviceability limit state (SLS)

Steel stress is not limited except for crack opening limit states:

- Non prejudicial cracking: no limitation $\sigma_s = \frac{f_e}{\gamma_s}$
- Prejudicial cracking: $\sigma_{st} \leq \bar{\sigma}_s = \min(\frac{2}{3}f_e; 110\sqrt{\eta \cdot f_{tj}})$
- Highly prejudicial cracking: $\sigma_{st} \leq \bar{\sigma}_s = \min(\frac{1}{2}f_e; 90\sqrt{\eta \cdot f_{tj}})$

η : cracking coefficient

- ✓ $\eta = 1$: for plain rounds (RL)
- ✓ $\eta = 1.6$: for high adherence reinforcement (HA)

❖ Calculating hypotheses

1. Calculation at the serviceability limit state (SLS)

- Plane sections remain plane, and there is no relative slip between reinforcement and concrete.
- The tensile strength of concrete is neglected.
- Concrete and steel are considered linearly elastic materials.
- The ratio of longitudinal moduli of elasticity of steel to concrete is taken as:

$n = \frac{E_s}{E_b} = 15$; n is called the equivalence coefficient.

2. Calculation at the ultimate limit state of resistance (ULS)

- Plane sections remain plane, and there is no relative slip between reinforcement and concrete.
- Tensioned concrete is neglected.
- The relative elongation of steel is limited to: 10‰ .
- The ultimate shortening of concrete is limited to:

✓ $\varepsilon_{bc} = 3.5\text{‰}$ in bending.

✓ $\varepsilon_{bc} = 2\text{‰}$ in pure compression.

❖ The three-pivot rule

The three-pivot rule assumes that the safety domain is defined by a strain diagram passing through one of the three pivots A, B, C defined by the figure below:

- **Pivot A:** simple or combined tension, bending with the ultimate limit state reached in the steel.
- **Pivot B:** bending with the ultimate limit state reached in the concrete.
- **Pivot C:** simple or combined compression.

Chapter II:
**Preliminary sizing of
structural elements**

II.1 Introduction

Before undertaking a detailed structural design, it is essential to estimate the main dimensions of the structural elements. Current regulations, particularly the RPA 2024, provide recommendations for coherent preliminary sizing that ensures both structural safety and rational use of materials.

II.2 Preliminary Sizing

II.2.1 Floor

A floor slab is a horizontal structural element that separates building levels of a building and supports the loads transmitted during service conditions. A typical floor system is generally composed of a finishing surface, a structural supporting element, and a ceiling layer.

Floors are expected to perform as a rigid horizontal diaphragm that transfer and distribute lateral loads to a vertical resisting members of the structure through sufficient rigidity and continuity.

a) Fire Resistance Condition

- $e = 0.7$ cm: for one (01) hour of fire resistance
- $e = 1$ cm: for two (02) hours of fire resistance.
- $e = 17.5$ cm: for four (04) hours of fire resistance.

b) Acoustic Comfort

To meet the acoustic comfort requirements in buildings, a minimum thickness of 16 cm is generally required for floor slabs.

c) Bending Resistance Condition

❖ Filler block

$$e \geq \frac{\min(L_{x\max}; L_{y\max})}{22.5} \Rightarrow e \geq \frac{\min(474; 416)}{22.5} \Rightarrow e \geq \frac{416}{22.5} = 18.488 \text{ cm} \Rightarrow e = 20 \text{ cm}$$

$$e = 20 \text{ cm} \begin{cases} 16 \text{ cm: height of filler block} \\ 4 \text{ cm: compression slab height} \end{cases}$$

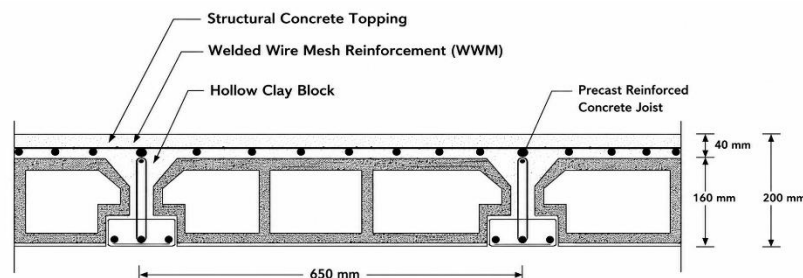


Figure 1. Cross-section of a hollow-core (beam-and-block) floor.

Figure II. 1: Cross-section of a hollow-core floor.

II.2.2 Preliminary Sizing of joists

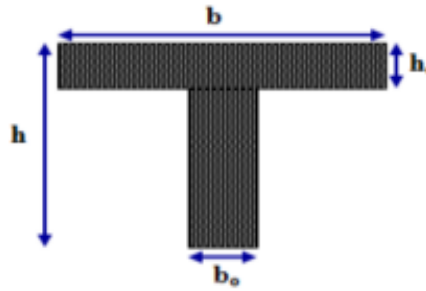


Figure II. 2: Cross-section of joist.

- **h**: Floor height
- **h₀**: Compression slab height $(0.4 \text{ } 0.6) \times h$
- **b₀**: Rib width $(0.4 \text{ } 0.6) \times h$
- **b**: Effective width

Rib width:

$$b_0 = 0.6 \times 20 \Rightarrow b_0 = 12 \text{ cm}$$

According CBA 93(Art 3.1.4) the effective width is determined as follows:

$$\frac{(b-b_0)}{2} \leq \min\left(\frac{L_x}{2}; \frac{L_{ymin}}{10}\right) \Rightarrow \frac{(b-b_0)}{2} \leq \min\left(\frac{65}{2}; \frac{367}{10}\right)$$

$$b \leq \left(\frac{367}{10} \times 2\right) + 12 \Rightarrow b \leq 85.4 \text{ cm} \Rightarrow b = 65 \text{ cm}$$

L_x : Distance between exposed faces of two joints.

L_{ymin} : minimum distance between the exposed faces of the support in the direction of the joints.

II.3 Solid slab

Bending resistance condition: $\alpha = \frac{l_x}{l_y} = \frac{135}{302} = 0.447 > 0.4$

We have a continuous slab: $h \geq \frac{l_x}{40} \Rightarrow h \geq \frac{135}{40} \Rightarrow h \geq 3.375 \text{ cm} \Rightarrow h=15 \text{ cm}$

II.4 Beams

Reinforced concrete beams with a rectangular cross-section are designed to transfer loads to the columns.

The preliminary sizing will be carried out in accordance with BAEL91, and verification in accordance with RPA2024.

L: Span of the beam.

$$\text{BAEL91:} \begin{cases} \frac{L}{15} < h < \frac{L}{10} \\ 0.3h < b < 0.8h \end{cases} \quad h: \text{The height of the beam.}$$

b: The width of the beam.

$$\text{RPA2024: } \begin{cases} b \geq 20 \text{ cm in zones I, II, III.} \\ b \geq 25 \text{ cm in zones IV, V, VI.} \\ h \geq 30 \text{ cm} \end{cases}$$

$$\begin{cases} \frac{h}{b} < 4 \\ b_{\max} \leq (1.5h + b_c) \end{cases} \Rightarrow \begin{cases} \frac{40}{30} = 1.33 < 4 \dots CV \\ 30 \leq (1.5 \times 40 + 30) = 90 \text{ cm} \dots CV \end{cases}$$

b_c : The minimum cross-sectional dimension of the column

The thickness h may be reduced to 20 cm in structures with shear walls.

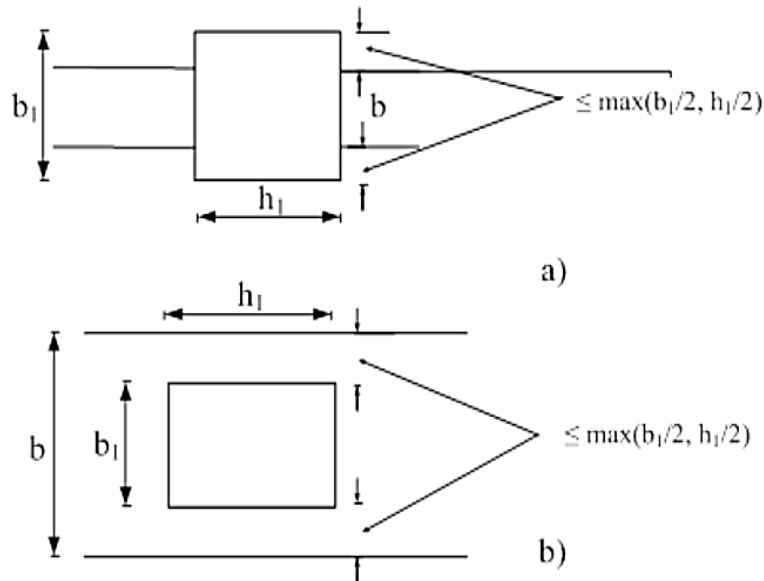


Figure II. 3: Beam dimensions to be respected (RPA2024).

❖ Primary beams

$$\frac{474}{15} \leq h \leq \frac{474}{10} \Rightarrow 31.6 \text{ cm} \leq h \leq 47.4 \text{ cm} \Rightarrow h = 39.5 \text{ cm} \Rightarrow \text{we adopt } h = 40 \text{ cm}$$

$$h = 40 \text{ cm} > 30 \text{ cm} \dots\dots CV$$

$$0.3h \leq b \leq 0.8h \Rightarrow 12 \text{ cm} \leq b \leq 32 \text{ cm} \Rightarrow b = 22 \text{ cm} \Rightarrow \text{we adopt } b = 30 \text{ cm}$$

$$b = 30 \text{ cm} > 25 \text{ cm} \dots\dots CV \text{ (zone V)}$$

$$\frac{40}{30} = 1.33 < 4 \dots\dots CV$$

❖ Secondary beams

$$\frac{416}{15} \leq h \leq \frac{416}{10} \Rightarrow 30 \text{ cm} \leq h \leq 45 \text{ cm} \Rightarrow h = 34.666 \text{ cm} \Rightarrow \text{we adopte } h = 40 \text{ cm}$$

$$h = 40 \text{ cm} > 30 \text{ cm} \dots\dots CV$$

$$0.3h \leq b \leq 0.8h \Rightarrow 12 \text{ cm} \leq b \leq 32 \text{ cm} \Rightarrow h = 22 \text{ cm} \Rightarrow \text{we adopt } h = 30 \text{ cm}$$

$$b = 30 \text{ cm} > 25 \text{ cm} \dots\dots CV$$

$$\frac{h}{b} = \frac{40}{30} = 1.33 < 4 \dots\dots CV$$

We resized the secondary beam from 30×35 cm to 30×40 cm because the initial section did not satisfy the design requirements. Increasing the beam depth improved its load-carrying capacity and stiffness, allowing it to meet the design checks.

II.5 Preliminary sizing of shear walls:

Shear walls stabilized the building by resisting horizontal forces, while also supporting vertical loads. In accordance with RPA 2024, elements are classified as shear walls when they satisfy the condition defined by the following

$$b_w \geq (15 \text{ cm}; \frac{h_e}{20}) \Rightarrow b_w \geq (15 \text{ cm}; \frac{2.66}{20}) \Rightarrow b_w \geq 15 \text{ cm} \Rightarrow \text{we adopt: } b_w = 20 \text{ cm}$$

$$l_w \geq \max(\frac{h_e}{3}; 4b_w; 1\text{m}) \Rightarrow l_w \geq \max(\frac{2.66}{3}; 4 * 0.2; 1\text{m}) \Rightarrow l_w \geq 1\text{m}$$

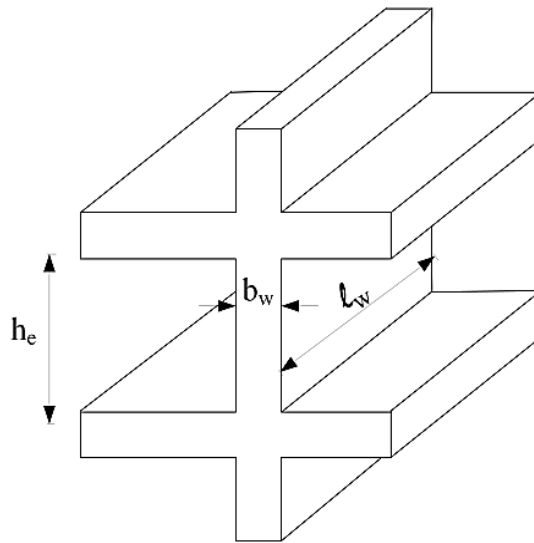


Figure II. 4 : Shear walls dimensions according to the RPA.

II.6 Stairs

In building, access between floors is provided by staircases or lifts. The structural loads from the stair system are carried to beams and then passed on the columns or shears walls.

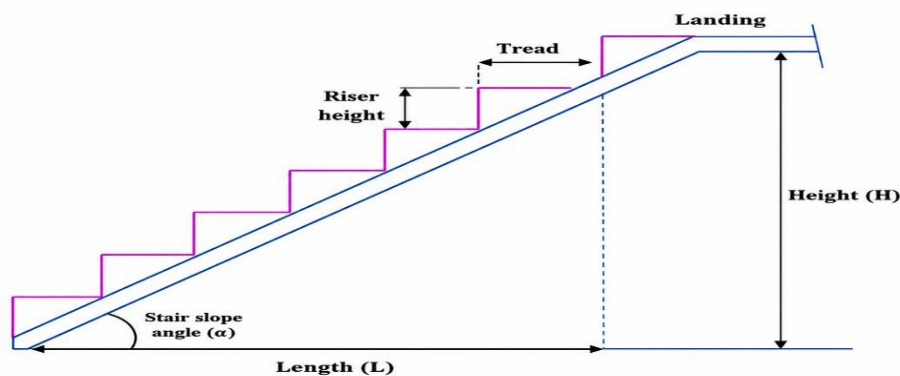


Figure II. 5: Stair dimensions

- The flight height H: $H = \frac{h}{2} = \frac{3.06}{2} \Rightarrow H = 1.53 \text{ m}$
- The riser height h: $h = \frac{H}{n} = \frac{153}{9} \Rightarrow h = 17 \text{ cm}$
- Tread calculation: $g = \frac{L}{(n-1)} = \frac{224}{9-1} \Rightarrow g = 28 \text{ cm}$
- Inclination of the waist slab: $\alpha = \text{tg}\left(\frac{H}{L}\right)^{-1} = \text{tg}\left(\frac{153}{224}\right)^{-1} \Rightarrow \alpha = 34.334^\circ$
- Waist slab length calculation: $L' = \frac{L}{\cos \alpha} = \frac{2.24}{\cos 31.263} \Rightarrow L' = 2.712 \text{ m}$
- Waist slab thickness: $\frac{L}{30} < e < \frac{L}{20}$

$$L = L' + L_p \quad L_p: \text{Landing length}$$

$$L = 2.71 + 1.46 \Rightarrow L = 4.172 \text{ m}$$

$$\frac{417.2}{30} < e < \frac{417.2}{20} \Rightarrow 13.906 \text{ m} < e < 20.86 \text{ cm} \Rightarrow \text{we adopt: } e = 15 \text{ cm.}$$

II.6.1 Landing beam



$$\text{BAEL91:} \begin{cases} \frac{L}{15} < h < \frac{L}{10} \\ 0.3h < b < 0.8h \end{cases} \Rightarrow \begin{cases} \frac{275}{15} < h < \frac{275}{10} \Rightarrow h = 40 \text{ cm} \\ 0.3h < b < 0.8h \Rightarrow b = 30 \text{ cm} \end{cases}$$

$$\text{RPA2024:} \begin{cases} b \geq 20 \text{ cm in zones I, II, III} \\ b \geq 25 \text{ cm in zones IV, V, VI} \\ h \geq 30 \text{ cm} \end{cases} \Rightarrow \begin{cases} b = 30 \text{ cm} > 25 \text{ cm} \dots \text{Verified} \\ h = 40 \text{ cm} > 30 \text{ cm} \dots \text{Verified} \end{cases}$$

$$\begin{cases} \frac{h}{b} < 4 \\ b_{\max} \leq (1.5h + b_1) \end{cases} \Rightarrow \begin{cases} \frac{40}{30} = 1.33 < 4 \dots \text{Verified} \\ 30 \leq (1.5 \times 40 + 30) = 90 \text{ cm} \dots \text{Verified} \end{cases}$$

b_c : The minimum cross-sectional dimension of the column

II.6.2 Parapet

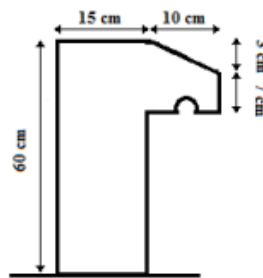


Figure II. 6: Parapet.

Permanent load:

$$S = (0.6 \times 0.15) + \left(\frac{0.1 \times 0.03}{2} \right) + (0.1 \times 0.1) = 0.1015 \text{ m}^2$$

$$G = \gamma \times S = 25 \times 0.1015 = 2.54 \text{ kN/ml}$$

Live load: $Q=1 \text{ kN/ml}$

II.7 Assessment of loads and live loads

II.7.1 Inaccessible roof terrace

❖ Hollow core

Table II. 1: Permanent and live load of the hollow-core at terrace level.

Constituent elements	Density (kg/m^3)	Thickness (m)	Surface density (kg/m^2)
1-Gypsum plaster	10	0.03	0.3
2-Hollow-core slab (16+4)	/	(16+4)	2.8
3-Slope-forming layer	22	0.12	2.64
4-Thermal insulation	0.4	0.04	0.016
5-Multi-layer waterproofing	6	0.02	0.12
6-Kraft paper	/	2 sheets	0.5
7-Protective gravel	16	0.05	0.8
Permanent load G	/	/	7.18
Live load Q	/	/	1

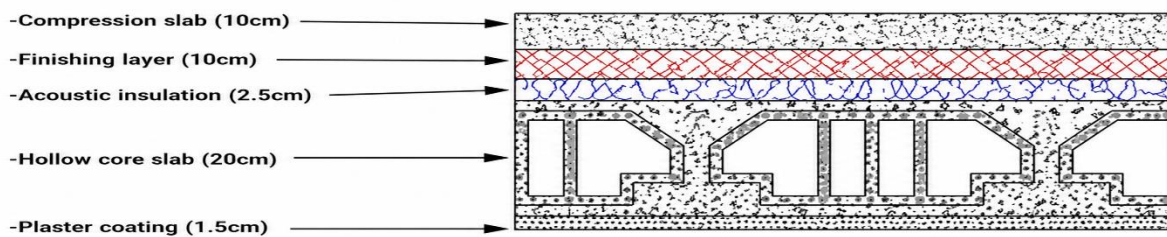


Figure II. 7: Cross-section of an inaccessible roof terrace slab.

❖ Solid Slab

Table II. 2: Permanent and live load of the solid slab at terrace level.

Constituent elements	Density (kg/m^3)	Thickness (m)	Surface density (kg/m^2)
1-Gypsum plaster	10	0.03	0.3
2-Solid slab floor	/	(15cm)	3,75
3-Slope-forming layer	22	0.12	2.64
4-Thermal insulation	0.4	0.04	0.016
5-Multi-layer waterproofing	6	0.02	0.12
6-Kraft paper	/	2 sheets	0.5
7-Protective gravel	16	0.05	0.8
Permanent load G	/	/	8,13
Live load Q	/	/	1

II.7.2 Typical floor

❖ Hollow-core

Table II. 3: Permanent and live load of the hollow-core at typical floor.

Constituent elements	Density (kg/m ³)	Thickness (m)	Surface density (kg/m ²)
1-Gypsum plaster	10	0.03	0.3
2-Hollow core slab (16+4)	/	(16+4)	2.8
3-Sand bedding layer	18	0.03	0.54
4-Cement screed	20	0.02	0.4
5-Granito flooring	22	0.02	0.44
6-Partition walls	/	/	1
Permanent load G	/	/	5.48
Live load Q	/	/	1.5

TYPICAL FLOOR :

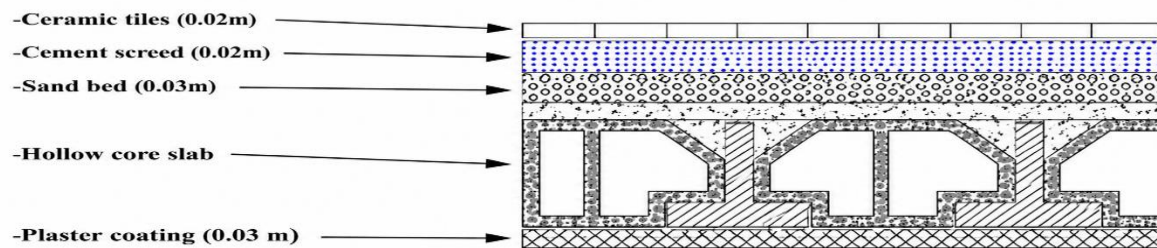


Figure II. 8: Cross-section of a typical story floor

❖ Solid slab

Table II. 4: Permanent and live load of the solid slab at typical floor.

Constituent elements	Density (kg/m ³)	Thickness (m)	Surface density (kg/m ²)
1-Gypsum plaster	10	0.03	0.3
2-Solid slab floor	/	(16+4)	2.8
3-Sand bedding layer	18	0.03	0.54
4-Cement screed	20	0.02	0.4
5-Granito flooring	22	0.02	0.44
6-Partition walls	/	/	1
Permanent load G	/	/	6.43
Live load Q	/	/	3.5

II.7.3 Stair

❖ Waist slab

Table II. 5: Permanent and live load of the waist slab.

Constituent elements	Density (kg/m ³)	Thickness (m)	Surface density (kg/m ²)
1-Cement plaster	18	0.02	0.36
2-Waist slab	25	$0.15/\cos\alpha$	4.30
3-Tread	22	0.17	1.87
4-Sand layer	18	0.03	0.54
5-Cement screed	20	0.02	0.40
6-Granite tile	22	0.02	0.44
7-Railing	/	/	1
Permanent load G	/	/	8.91
Live load Q	/	/	2.50

❖ Landing

Table II. 6: Permanent and live load of the landing.

Constituent elements	Density (kg/m ³)	Thickness (m)	Surface density (kg/m ²)
1-Tiles	22	0.02	0.44
2-Bedding mortar	20	0.025	0.50
3-Bedding layer	18	0.025	0.45
4-Solid slab floor	25	0.15	3.75
5-Plaster coating	10	0.02	0.20
Permanent load G	/	/	5.34
Live load Q	/	/	2.50

❖ Exterior masonry

Table II. 7: Permanent and live load of the exterior masonry.

Constituent elements	Density (kg/m ³)	Thickness (m)	Surface density (kg/m ²)
1-Cement screed	18	0.02	0.36
2- Gypsum plaster	10	0.02	0.2
3-External brickwork	9	0.15	1.35
4-Interior brickwork	9	0.10	0.9
Permanent load G	/	/	2.81



Figure II. 9: Cross-section of external masonry wall.

II.8 Preliminary sizing of columns

The dimensioning of reinforced concrete columns is based on axial compression at the ultimate limit state (ULS). The objective is to provide a concrete section capable of supporting the applied actions while ensuring adequate resistance to buckling.

In accordance with the requirements of the CBA 93 and BAEL 91, the calculation is performed using a reduced concrete section (B_r),

B_r : Reduced cross – section of the column by subtracting 1 cm from outer perimeter of the gross-section in order to consider construction imperfections and possible concrete segregation effects.

$$B_r \geq \frac{K\beta N_u}{\theta \frac{f_{bu}}{0.9} + \frac{0.85}{100} f_{ed}}$$

$$\left\{ \begin{array}{l} K = 1.10 \text{ when more than half of the loads are applied before } 90 \\ K = 1.20 \text{ } f_{c28} \text{ replaced by } f_{cj} \\ K = 1 \text{ for all other cases} \end{array} \right.$$

B_r : Reduced cross-sectional area of the column

$$f_{bu} = \frac{0.85 f_{c28}}{\theta \gamma_b} \quad f_{ed} = \frac{f_e}{\gamma_s}$$

f_{c28} : Characteristic concrete compressive strength at 28 days $f_{c28} = 25 \text{ MPa}$

f_e : Yield stress of the reinforcing steel adopted in design, $f_e = 400 \text{ MPa}$

$\gamma_s = 1.15$

$\gamma_b = 1.5$

θ : Parameter related to the duration T during which the load combinations are applied.

$$\left\{ \begin{array}{ll} \theta = 1 & T > 24h \\ \theta = 0.9 & 1h \leq T \leq 24h \\ \theta & T < 1h \end{array} \right.$$

β : Coefficient depending on the mechanical slenderness ratio (λ) of the columns, defined as follows:

$$\beta = 1 + 0.2 \left(\frac{\lambda}{35} \right)^2 \dots\dots \text{If } \lambda \leq 50$$

$$\beta = \frac{0.85\lambda^2}{1500} \dots\dots \text{If } 50 < \lambda \leq 70$$

$$\lambda = \frac{l_f \sqrt{12}}{a} \quad \text{a: smaller dimension of the column cross-section}$$

When performing centric compression cheches, λ fixed at 35, so that every reinforcement bar contributes to the structural resistance.

β : Coefficient depending on the mechanicalslenderness ratio (λ) of the column

$$\lambda \leq 50 \Rightarrow \beta = 1 + 0.2\left(\frac{\lambda}{35}\right)^2 \Rightarrow \beta = 1 + 0.2 \times 1 \quad \beta = 1.2$$

$$B_r \geq \frac{1 \times 1.2 \times Nu \times 10^{-3}}{1 \times \frac{14.167}{0.9} + \frac{0.85}{100} \times 347.826} \cdot 10^{-4} \quad B_r \geq 0.64Nu \quad a = \sqrt{B_r} + 2$$

$$\text{Non-fragile design criterion : } \nu = \frac{Nu}{B_c \times f_{c28}}$$

❖ Live load

Table II. 8: Live load reduction factors.

Story	Live load
Terrace	Q_0
Story 9	$Q_0 + Q_1$
Story 2	$Q_0 + 0.95(Q_0 + Q_1)$
Story 3	$Q_0 + 0.9(Q_1 + Q_2 + Q_3)$
Story 4	$Q_0 + 0.85(Q_1 + Q_2 + Q_3 + Q_4)$
Story n	$Q_0 + \left(\frac{3+n}{2n}\right) \times (Q_1 + Q_2 + Q_3 + \dots + Q_n)$

❖ Permanent load

$$N_{G \text{ terrace}} = G \times S$$

$$N_{G \text{ typical floor slab}} = G \times S$$

$$N_{G \text{ primary beam}} = \rho_{\text{béton}} \times b \times h \times L$$

$$N_{G \text{ secondary beam}} = \rho_{\text{béton}} \times b \times h \times L$$

$$N_{G \text{ colmun}} = \rho_{\text{béton}} \times a \times a \times (H_{\text{story}} - h_{\text{beam}})$$

$$N_{G \text{ terrace}} = N_{G \text{ terrace}} + N_{G \text{ secondary beam}} + N_{G \text{ secondary beam}}$$

$$N_{G \text{ typical floor slab}} = N_{G \text{ terrace}} + N_{G \text{ secondary beam}} + N_{G \text{ secondary beam}}$$

❖ Central column

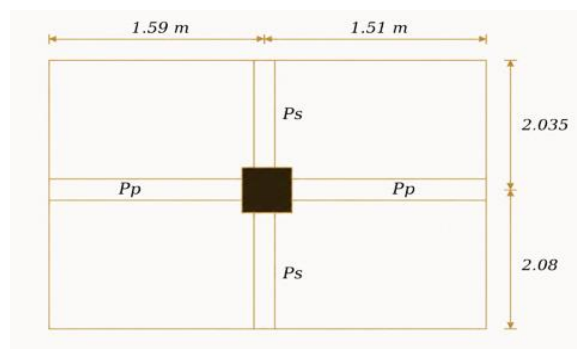


Figure II. 10: Tributary area of the central column.

$$S = (1.51+1.59) \times (2.035+2.08) \Rightarrow S = 12.756 \text{ m}^2$$

$$\text{Amplification of the influence area: } S = 1.1 \times 12.756 = 14.032 \text{ m}^2$$

$$N_{G \text{ terrace}} = 100.749 \text{ KN}$$

$$N_{G \text{ typical floor slab}} = 76.895 \text{ KN}$$

$$N_{G \text{ primary beam}} = 9.3 \text{ KN}$$

$$N_{G \text{ secondary beam}} = 12.345 \text{ KN}$$

$$N_G = N_{G \text{ terrace}} + N_{G \text{ primary beam}} + N_{G \text{ sb}}$$

$$N_{G \text{ typical floor}} = N_{G \text{ typical floor slab}} + N_{G \text{ pb}} + N_{G \text{ sb}} + N_{G \text{ colmun}}$$

Table II. 9: Load transfer to central column.

Story	N_G (kN)	$N_{G \text{ cum}}$ (kN)	N_Q (kN)	$N_{Q \text{ cum}}$ (kN)	N_u (kN)	$1.1N_u$ (kN)	B_r (cm ²)	a (cm)	Section (cm ²)
Terrace	122.394	122.394	14.032	14.032	186.279	204.906	131.139		/
8	104.525	226.919	21.048	35.08	358.960	394.856	252.707	17.896	(30x30)
7	104.525	331.444	21.048	54.023	528.483	581.331	372.052	21.288	(35x35)
6	106.686	438.13	21.048	70.861	697.767	767.543	491.227	24.163	(35x35)
5	106.686	554.816	21.048	85.595	877.394	965.133	617.685	26.853	(40x40)
4	109.18	663.996	21.048	1043.730	1148.103	734.785	29.106	29.106	(40x40)
3	109.18	773.176	21.048	108.748	1206.909	1327.59	849.663	31.910	(45x45)
2	112.006	885.185	21.048	117.167	1370.764	1507.82	965.004	33.064	(45x45)
1	112.006	997.188	21.048	123.481	1531.425	1684.567	1078.122	34.834	(50x50)
Ground floor	115.165	1112.353	21.048	127.691	1693.213	1862.534	1192.021	36.525	(50x50)

❖ **Edge column**

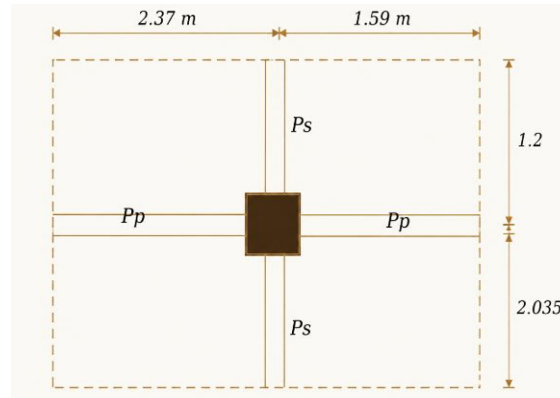


Figure II. 11: Tributary area of the edge column.

$$S = (2.37+1.59) \times (1.2+2.035) \Rightarrow S = 12.81 \text{ m}^2$$

$$\text{Amplification of the influence area: } S = 1.1 \times 12.81 = 14.091$$

$$N_{G \text{ terrace hollow-core}} = 63.643 \text{ KN}$$

$$N_{G \text{ terrace solid slab}} = 34.916 \text{ KN}$$

$$N_{G \text{ typicl floor hollow-core}} = 48.574 \text{ KN}$$

$$N_{G \text{ typicl floor solid slab}} = 28.905 \text{ KN}$$

$$N_{G \text{ primary beam}} = 11.88 \text{ KN}$$

$$N_{G \text{ secondary beam}} = 9.705 \text{ KN}$$

$$N_{G \text{ parapet}} = G \times L \Rightarrow N_{G \text{ parapet}} = 2.54 \times 3.96 \Rightarrow N_{G \text{ parapet}} = 10.058 \text{ KN}$$

$$N_{Q \text{ parapet}} = Q \times L \Rightarrow N_{Q \text{ parapet}} = 1 \times 3.96 \Rightarrow N_{Q \text{ parapet}} = 3.96 \text{ KN}$$

$$N_{G \text{ terrace}} = N_{G \text{ terrace hollow-core}} + N_{G \text{ terrace solid slab}} + N_{G \text{ pb}} + N_{G \text{ ps}} + N_{G \text{ parapet}}$$

$$N_{G \text{ typicl floor}} = N_{G \text{ typicl floor hollow-core}} + N_{G \text{ typicl floor solid slab}} + N_{G \text{ s}} + N_{G \text{ s}}$$

Table II. 10: Load transfer to edge column.

Story	N_G (kN)	$N_{G \text{ cum}}$ (kN)	N_Q (kN)	$N_{Q \text{ cum}}$ (kN)	N_u (kN)	B_r (cm ²)	a (cm)	Section (cm ²)
Terrace	122.145	122.145	17.028	17.028	190.437	121.879	13.039	
8	98.733	220.878	30.054	47.082	368.808	236.037	17.037	(30x30)
7	100.894	321.772	30.054	74.131	545.5888	349.176	20.686	(35x35)
6	100.894	422.666	30.054	98.175	717.861	459.431	23.434	(35x35)
5	103.388	526.054	30.054	119.213	888.992	568.954	25.852	(40x40)
4	103.388	629.442	30.054	137.246	1055.615	675.593	27.992	(40x40)
3	106.214	735.656	30.054	152.273	1221.545	781.788	29.960	(45x45)
2	106.214	841.87	30.054	164.295	1382.967	885.098	31.750	(45x45)
1	109.273	951.243	30.054	173.311	1544.144	988.252	33.436	(50x50)
Ground floor	109.373	1060.616	30.054	179.322	1700.814	1088.520	34.992	(50x50)

❖ **Verification according to the Algerian seismic regulation**

- $\text{Min}(a, b) \geq 30 \text{ cm} \Rightarrow \text{min}(30 : 30) \geq 30 \text{ cm} \dots \text{Verified}$
- $\text{Min}(a, b) \geq \frac{h_e}{20} \Rightarrow 30 \text{ cm} \geq \frac{306}{20} = 15 \text{ cm} \dots \text{Verified}$
- $\frac{1}{4} < \frac{b}{a} < 4 \Rightarrow \frac{1}{4} < 1 < 4 \dots \text{Verified}$

❖ **Stability assessment for buckling:** (According to BAEL91)

$$\lambda = \frac{l_f}{i} \quad i = \sqrt{\frac{I}{B}} \quad L_f = 0.7 h_e$$

I: Minimum section moment of inertia for the column. $\Rightarrow I = \frac{bh^3}{12}$

B: Column-cross section. $\Rightarrow B = b \times h$

l_f : Buckling length. $\Rightarrow L_f = 0.7 h_e$

λ : Column slenderness $\Rightarrow \lambda = \frac{l_f}{i}$

i: Radius of gyration. $\Rightarrow i = \sqrt{\frac{I}{B}}$

h_e : Story height.

b, h: Column dimensions

Table II. 11: Stability assessment of column sections against buckling.

Story	Section (cm^2)	l_f (cm)	I (cm^4)	i (cm)	λ	$\lambda < 50$
Ground floor	(50x50)	182.7	520833.333	14.434	12.657	CV
Story 1 and story 2	(45x45)	182.7	341718.750	12.990	14.064	CV
Story 3 and story 4	(40x40)	182.7	213333.333	11.547	15.822	CV
Story 5 and story 6	(35x35)	182.7	125052.083	10.104	18.081	CV
Story 7 and story 8	(30x30)	182.7	67500	8.660	21.096	CV

II.9 Conclusion

The preliminary design of the building allowed the selection of suitable initial dimensions for the main structural elements, taking into account strength, stability, and serviceability requirements. This phase is a key in the design process, as it provides a reliable basis for detailing structural analysis and final calculations.

The adopted sections for beams, columns, shear walls, and slab were defined in accordance with reinforced concrete design standards and current regulations. The result show that these preliminary choices ensure adequate structural performance under both static and seismic loading conditions, while guaranteeing safety, stability, and design efficiency.

Chapter III:
**Design of structural
secondary elements**

III.1 Introduction

This chapter focuses on the evaluation and design of certain building elements, including staircases, slab, and balconies. The adopted procedure, consists of defining preliminary dimensions, identifying and evaluating the applied loads, determining the most critical internal forces, and finally calculating the reinforcement required to ensure to guarantee sufficient structural resistance. The dimensioning is carried out in accordance with BAEL 91 provisions, while also complying with the provisions of the Algerian Seismic Code (RPA 2024).

III.2 Hollow-core slab system (Hollow concrete)

The building floors are designed as hollow block slab systems of type (16+4), supported by joists. A reinforced concrete topping layer, about 4 cm thick, is poured over the entire surface to provide overall structural continuity and ensure the monolithic behavior of the system. The joists, characterized by a T-shaped section, are responsible for transmitting both uniformly distributed and concentrated loads to the main beams of the structural framework.

III.2.1 Joists design

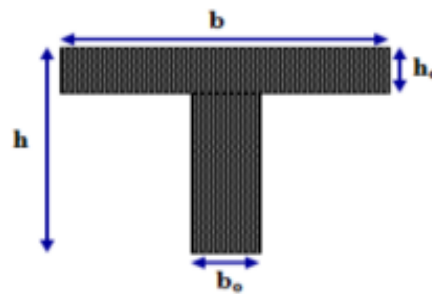


Figure III. 1: Cross-section of joist.

Cross-sectional properties of the joists:

$$b = 65 \text{ cm} \quad b_o = 12 \text{ cm} \quad h = 20 \text{ cm} \quad h_o = 4 \text{ cm}$$

III.2.2 Load assessment

$$\text{ULS: } q_u = (1.35G + 1.5Q) \times b$$

$$\text{SLS: } q_s = (G + Q) \times b$$

Table III. 1: Load assessment.

Floor level	G(kN/m ²)	Q(kN/m ²)	q _u (kN/m ²)	q _s (kN/m ²)
Inaccessible roof terrace	7.18	1	7.275	5.317
Typical floor	5.48	1.5	6.271	4.537

III.2.3 Assessment of structural actions

Joists are designed under simple bending conditions are treated as continuous beams spanning multiple supports. The internal force analysis may be performed using either:

- A simplified method for members subjected to moderate live loads.
- The Caquot method for members subjected to higher live loads.

III.2.4 The simplified method

The use of the simplified method is permitted only if the following conditions are met:

1. The imposed load Q must be greater than the lesser of twice of the permanent load G or 5 KN/m^2 .

$$Q = 1 \text{ KN/m}^2 < \min(2G=14.36 \text{ KN/m}^2; 5 \text{ KN/m}^2) \dots\dots \text{CV.}$$

2. Uniform cross-sectional moments of inertia are assumed for each span to simplify structural analysis. CV

3. The ratio of the length of adjacent spans must be between 0.8 and 1.25.....CV

$$\begin{cases} 0.8 \leq \frac{L_i}{L_{i-1}} \leq 1.25 \\ 0.8 \leq \frac{L_i}{L_{i+1}} \leq 1.25 \end{cases}$$

4. Minor cracking.

The simplified method can be applied.

III.2.5 Application of the simplified method:

❖ Bending moment in the span

$\alpha = \frac{Q}{G+Q}$: The ratio of live loads to the total of permanent and live loads, without applying any weighting factors.

M_0 : Bending moment in statically determinate structure.

M_r : Bending moment at the right support.

M_l : Bending moment at the left support.

M_t : Bending moment in the span.

$$M_0 = \frac{q \times l^2}{8}$$

$$M_t \geq \frac{1.2+0.3\alpha}{2} \times M_0 \dots\dots\dots \text{in the edge span.}$$

$$M_t \geq \frac{1+0.3\alpha}{2} \times M_0 \dots\dots\dots \text{in the interior span.}$$

Verification of bending moments for each span:

$$M_{ti} + \frac{M_r + M_l}{2} \geq \max \begin{cases} (1 + 0.3\alpha)M_{0i} \\ 1.05M_{0i} \end{cases}$$

❖ **Bending moment in the support**

$0.5M_0$: For supports adjacent to the edge supports of a beam with more than two spans.

$0.4M_0$: For interior supports of a beam with more than three span

$0.6M_0$: For interior supports of a two-span beam.

$0.15M_0$: For edge supports in a beam with more than two-spans.

❖ **Shear force**

V_l : Shear force in the left support.

$$V_r: \text{Shear force in the right support.} \quad \begin{cases} V_l = 2 \frac{M_l + M_t}{a} \\ V_r = 2 \frac{M_r + M_t}{b} \end{cases} \Rightarrow \begin{cases} a = L \frac{1}{1 + \sqrt{\frac{M_r + M_t}{M_l + M_t}}} \\ b = L \frac{1}{\sqrt{\frac{M_l + M_t}{M_r + M_t}}} \end{cases}$$

❖ **Type 1**

Table III. 2: Diagrams of bending moments and shear forces at ULS and SLS of joists.

	Internal forces	Maximum value	Diagram
ULS	The maximum bending moment at the support (KN.m)	15.40	
	The maximum bending moment in the span (KN.m)	8.97	
	Maximum shear Force (KN)	18.88	
SLS	The maximum bending moment at the support (KN.m)	11.25	
	The maximum bending moment in the span (KN.m)	6.56	
	Maximum shear Force (KN)	13.75	

❖ Type 2

	Internal forces	Maximum value	Diagram
ULS	The maximum bending moment at the support (KN.m)	12.19	
	The maximum bending moment in the span (KN.m)	9.65	
	Maximum shear Force (KN)	17.79	
SLS	The maximum bending moment at the support (KN.m)	8.87	
	The maximum bending moment in the span (KN.m)	7.02	
	Maximum shear force (KN)	13.00	

III.2.6 The reinforcement design

The longitudinal reinforcement of a reinforced concrete beam is proportioned at the ultimate limit state (ULS) considering a condition of simple bending.

Table III. 3: Geometric and mechanical characteristics of reinforced concrete joists.

b (cm)	h (cm)	d (cm)	$f_{c28}(MPa)$	$f_{bu}(MPa)$	$f_e (MPa)$	$f_{ed}(MPa)$
65	20	18	25	14.167	400	347.826

❖ Longitudinal reinforcement

$$M_{Tu} = b \times h_0 \times f_{bu} \times \left(d - \frac{h_0}{2}\right) \Rightarrow M_{Tu} = 147.336 \text{ kN.ml}$$

$$u_{bu} = \frac{M_u}{b \times d^2 \times f_{bu}} \quad ; \quad u_l = 0.8 \times \alpha_l \times (1 - 0.4\alpha_l)$$

$$\alpha = 1.251 - \sqrt{(1 - 2u_{bu})} \quad ; \quad z = d(1 - 0.4\alpha) \quad ; \quad A_{min} = 0.23 \times b \times d \times \frac{f_{t28}}{f_e}$$

In support: The maximum bending moment: $M_{a \max} = 15.40 \text{ kN.ml}$

$$M_{Tu} = 147.336 \text{ kN.ml} > M_{a \max} = 15.44 \text{ kN.ml}$$

In span: The maximum bending moment: $M_{a \max} = 9.6 \text{ kN.ml}$

$$M_{Tu} = 147.336 \text{ kN.ml} > M_{a \max} = 9.6 \text{ kN.ml}$$

The cross-section is considered rectangular ($b = 65 \text{ cm}$, $h = 20 \text{ cm}$)

Table III. 4: Ultimate limit state reinforcement results for hollow-core slabs.

	M_u (kN.ml)	u_{bu}	u_l	α	Z (cm)	A_s (cm ²)	A_{min} (cm ²)	A_{adop} (cm ²)	Choice
Support	15.40	0.051	0.391	0.065	17.5	2.530	1.412	2.26	2HA12
Span	9.65	0.032	0.391	0.040	17.7	1.559	1.412	2.36	3HA10

❖ Transverse reinforcement

The transverse reinforcement of reinforced concrete joists is performed according to two design codes.

❖ According BAEL91/99

The diameter of reinforcement bars:

ϕ_l : Minimum diameter of longitudinal reinforcement bars.

$$\phi_t \leq \min\left(\frac{h}{35}; \frac{b_0}{10}; \phi_l\right) \Rightarrow \phi_t = \min\left(\frac{20}{35}; \frac{12}{10}; 1\right) \Rightarrow \phi_t \leq 0.571 \text{ cm}$$

$$\text{We adopt: } 2\text{HA6} \Rightarrow A_t = 0.57 \text{ cm}^2$$

Transverse reinforcement spacing:

$$S_t \leq \min(0.9 h; 40 \text{ cm}) \Rightarrow S_t \leq \min(0.9 \times 20; 40 \text{ cm}) \Rightarrow S_t \leq 18 \text{ cm} \Rightarrow S_t = 15 \text{ cm}$$

❖ Ultimate limit state verification

✓ Assessment of ductile behavior

In support: $A_s = 2.53 \text{ cm}^2 > A_{min} = 1.412 \text{ cm}^2 \dots\dots \text{CV}$

In span: $A_s = 1.559 \text{ cm}^2 > A_{min} = 1.412 \text{ cm}^2 \dots\dots \text{CV}$

✓ Shear stress

$$\tau_u = \frac{V}{b \times d} \Rightarrow \tau_u = \frac{18.83 \times 10^{-3}}{0.65 \times 0.12} \Rightarrow \tau_u = 0.871 \text{ MPa}$$

✓ **Verification of shear failure risk**

Non-detrimental cracking:

$$\tau_u = \frac{V}{b \times d} \leq \min \left(\frac{0.2 f_{c28}}{\gamma_b}; 5 \text{ MPa} \right) \Rightarrow \tau_u = \leq \min \left(\frac{0.2 \times 25}{1.5}; 5 \text{ MPa} \right)$$

$$\tau_u = 0.161 \text{ MPa} \leq \min (3.333 \text{ MPa}; 5 \text{ MPa}) \Rightarrow \tau_u = 0.161 \text{ MPa} \leq 3.333 \text{ MPa} \dots \text{CV}$$

✓ **Transverse reinforcement spacing**

$$\tau_0 = 0.3 f_{tj}^* k \quad \text{and} \quad f_{tj}^* = \min (f_{tj}; 3.3 \text{ MPa})$$

$$f_{tj}^* = \min (2.1 \text{ MPa}; 3.3 \text{ MPa}) \Rightarrow f_{tj}^* = 2.1 \text{ MPa}$$

$k = 1$: For simple bending without staged concrete continuity.

$$\tau_0 = 0.3 \times 2.1 \times 1 \Rightarrow \tau_0 = 0.63 \text{ MPa}$$

$$\tau_t = \tau_u - \tau_0 \geq 0.32 \Rightarrow \tau_t = 0.871 - 0.63 \geq 0.32 \Rightarrow \tau_t = 0.241 \text{ MPa}$$

$$S_t \leq \min \left(\frac{0.8 f_e A_t}{b_0 \tau_t}; 0.9d; 40 \text{ cm} \right)$$

✓ **Assessment of the connection between the compression slab and rib**

$$z = 0.9 d \Rightarrow z = 0.9 \times 0.18 \Rightarrow z = 0.162 \text{ m}$$

$$b_1 = \frac{(b-b_0)}{2} \Rightarrow b_1 = \frac{(65-12)}{2} \Rightarrow b_1 = 26.5 \text{ cm}$$

$$\tau_u = \frac{V_u}{z} \times \frac{b_1}{b} \times \frac{1}{h_0} \leq \min \left(\frac{0.2 f_{c28}}{\gamma_b}; 5 \text{ MPa} \right) \Rightarrow \tau_u = 0.394 \text{ MPa} \leq 3.333 \text{ MPa} \dots \text{CV}$$

❖ **Serviceability limit state verification**

a) **Stress verification**

Table III. 5: Serviceability limit state verification.

	$M_s (kN.m)$	$A_s (cm^2)$	$Y (cm)$	$I (cm^4)$	$\sigma_{bc} (MPa)$	$\sigma_{bc} \leq \bar{\sigma}_{bc}$
Support	11.26	2.26	3.833	8023.998	5.378	CV
Span	7.02	2.36	3.916	8323.038	3.326	CV

As cracking is considered non-detrimental, there is no need to verify stresses in the reinforcement.

b) **Deflection verification:**

In compliance with BAEL 91 requirements, deflection verification for a beam or joists connected to hollow-block slab system can be disregarded provided that all required conditions are fulfilled:

$$\left(\frac{h_t}{L} \right) \geq \left(\frac{1}{22.5} \right) \Rightarrow 0.048 \geq 0.044 \dots \text{CV}$$

$$\left(\frac{h_t}{L} \right) \geq \left(\frac{M_{ser span}}{15 M_{0 ser}} \right) \Rightarrow 0.048 \geq 0.040 \dots \text{CV}$$

$$\left(\frac{A_s}{b \times d}\right) \leq \left(\frac{3.6}{f_e}\right) \Rightarrow 0.010 > 0.009 \dots \text{CNV}$$

❖ **Assessment of bending moment**

$$\begin{cases} G = 5.48 \text{ kN / m}^2 \\ P = (5.48 + 5) = 10.48 \text{ kN / m}^2 \\ J = 2.8 \text{ kN/m}^2 \end{cases}$$

$$\text{for } b = 0.65 \text{ m} \begin{cases} G = 5.48 \times 0.65 = 3.56 \text{ kN/m}^2 \\ P = 10.48 \times 0.65 = 6.18 \text{ kN/m}^2 \\ J = 2.8 \times 0.65 = 1.82 \text{ kN/m}^2 \end{cases}$$

$$\begin{cases} M_G = 0.85 \times \frac{G \times L^2}{8} \\ M_P = 0.85 \times \frac{P \times L^2}{8} \\ M_J = 0.85 \times \frac{J \times L^2}{8} \end{cases} \Rightarrow \begin{cases} M_G = 6.54 \text{ kN.ml} \\ M_P = 11.36 \text{ kN.ml} \\ M_J = 3.34 \text{ kN.ml} \end{cases}$$

❖ **Homogenized section**

$$B_0 = b \cdot h_0 + b_0 \cdot (h - h_0) + 15A_s$$

$$B_0 = (65 \times 4) + 12 \times (20 - 4) + 15 \times 2.36$$

$$B_0 = 487.4 \text{ cm}^2$$

❖ **First moment of area**

$$S_{/xx} = \frac{b_0 h^2}{2} + (b - b_0) \frac{h_0^2}{2} + 15A_s d$$

$$S_{/xx} = \frac{12 \times 20^2}{2} + (65 - 12) \frac{4^2}{2} + 15 \times 2.36 \times 18$$

$$S_{/xx} = 3461.2 \text{ cm}^3$$

❖ **Center of gravity**

$$V_1 = \frac{S_{/xx}}{B_0} \Rightarrow V_1 = \frac{3461.2}{487.4} \Rightarrow V_1 = 7.101 \text{ cm}$$

$$V_2 = h - V_1 \Rightarrow V_2 = 20 - 7.101 \Rightarrow V_{A2} = 12.899 \text{ cm}$$

❖ **Moment of inertia**

$$I = \frac{b_0}{3} (V_1^3 + V_2^3) + (b - b_0) \frac{h_0^3}{12} + (b - b_0) h_0 \times (V_1 - 0.5 h_0)^2 + 15A_s (V_2 - c)^2$$

$$I = 14023.957 \text{ cm}^4$$

❖ **The instantaneous longitudinal deformation modulus**

$$E_i = 11000 \sqrt[3]{f_{c28}} \Rightarrow E_i = 11000 \sqrt[3]{25} \Rightarrow E_i = 32164.195 \text{ MPa}$$

❖ **Delayed deformation modulus**

$$E_v = 3700 \sqrt[3]{f_{c28}} \Rightarrow E_v = 3700 \sqrt[3]{25} \Rightarrow E_v = 10818.865 \text{ MPa}$$

❖ Calculation of stress

$$\sigma_{sg} = 15 \times \frac{M_{g\ ser}}{I} (d - y) \Rightarrow \sigma_{sg} = 172.40 \text{ MPa}$$

$$\sigma_{sp} = 15 \times \frac{M_{g\ ser}}{I} (d - y) \Rightarrow \sigma_{sp} = 299.467 \text{ MPa}$$

$$\sigma_{sj} = 15 \times \frac{M_{g\ ser}}{I} (d - y) \Rightarrow \sigma_{sj} = 88.047 \text{ MPa}$$

❖ Calculation of A_t

$$\rho = \frac{A_s}{b_0 \times d} \Rightarrow \rho = \frac{2.36}{12 \times 18} \Rightarrow \rho = 0.011$$

❖ Coefficient for instantaneous deformation

$$\lambda_i = \frac{0.05 f_{t28}}{(2 + \frac{3b_0}{b}) \times \rho} \Rightarrow \lambda_i = \frac{0.05 \times 2.1}{(2 + \frac{3 \times 12}{65}) \times 0.011} \Rightarrow \lambda_i = 3.737$$

❖ Coefficient for delayed deformation:

$$\lambda_v = 0.4 \times \lambda_i \Rightarrow \lambda_v = 0.4 \times 3.737 \Rightarrow \lambda_v = 1.494$$

❖ Calculation of u_g , u_p and u_j

$$u_g = 1 - \left(\frac{1.75 f_{t28}}{4 \cdot \rho \cdot \sigma_{sg} + f_{t28}} \right) \Rightarrow u_g = 0.620$$

$$u_p = 1 - \left(\frac{1.75 f_{t28}}{4 \cdot \rho \cdot \sigma_{sg} + f_{t28}} \right) \Rightarrow u_p = 0.759$$

$$u_j = 1 - \left(\frac{1.75 f_{t28}}{4 \cdot \rho \cdot \sigma_{sj} + f_{t28}} \right) \Rightarrow u_j = 0.384$$

❖ Effective modulus of elasticity

$$I_{fi}^g = \frac{1.1 I_0}{(1 + \lambda_i \cdot u_g)} \Rightarrow I_{fi}^g = 4650.778 \text{ cm}^4$$

$$I_{fv}^g = \frac{1.1 I_0}{(1 + \lambda_v \cdot u_g)} \Rightarrow I_{fv}^g = 8008.364 \text{ cm}^4$$

$$I_{fi}^p = \frac{1.1 I_0}{(1 + \lambda_i \cdot u_p)} \Rightarrow I_{fi}^p = 4021.067 \text{ cm}^4$$

$$I_{fi}^j = \frac{1.1 I_0}{(1 + \lambda_i \cdot u_j)} \Rightarrow I_{fi}^j = 6335.237 \text{ cm}^4$$

❖ Deflection analysis

$$f_{gi} = \frac{M_g \cdot L^2}{10 \cdot E_i \cdot I_{fi}^g} \Rightarrow f_{gi} = 0.43 \text{ cm}$$

$$f_{gv} = \frac{M_g \cdot L^2}{10 \cdot E_v \cdot I_{fv}^g} \Rightarrow f_{gv} = 0.59 \text{ cm}$$

$$f_{pi} = \frac{M_p \cdot L^2}{10 \cdot E_i \cdot I_{fi}^p} \Rightarrow f_{pi} = 0.82 \text{ m}$$

$$f_{ji} = \frac{M_j \cdot L^2}{10 \cdot E_i \cdot I_{fi}^j} \Rightarrow f_{ji} = 0.3 \text{ m}$$

❖ Total deflection calculation

$$\Delta f = f_{gv} + f_{pi} - 2f_{gi} \leq \frac{L}{500} (cm)$$

$$\Delta f = 0.59 + 0.82 - 2 \times 0.43 = 0.55 \text{ cm} < 0.832$$

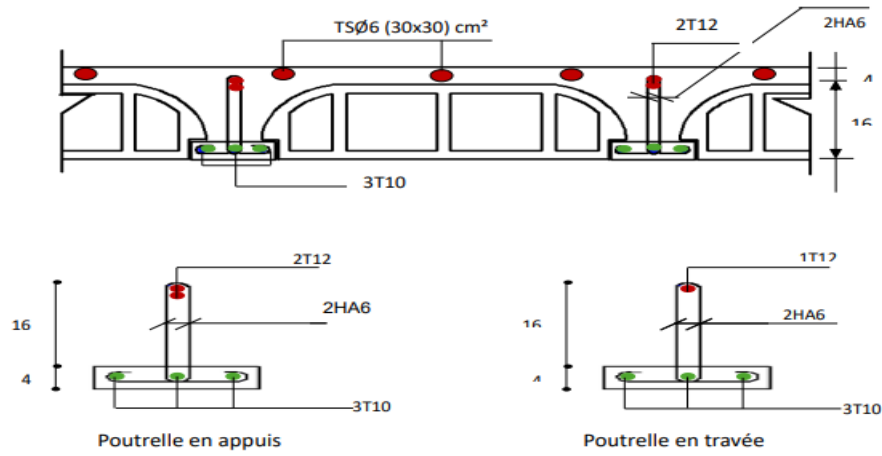


Figure III. 2: Joists reinforcement detailing.

III.3 Parapet

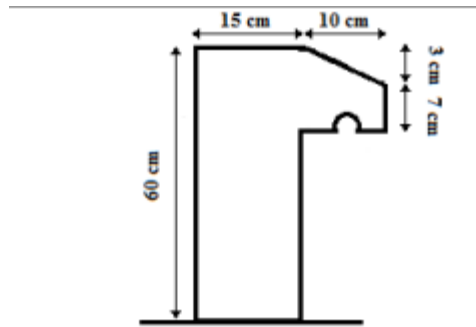


Figure III. 3: Parapet.

III.3.1 Design principle

The calculation is performed for combined bending at the ultimate limit state (ULS) and serviceability limit state (SLS) due to detrimental cracking.

III.3.2 Seismic load verification

All secondary structural elements of the building must be assessed for lateral forces. This verification ensures that horizontal actions, such as seismic load do not compromise their stability or performance. The evaluation is carried out using the following equation, which accounts for the relevant load combinations and structural parameters:

$$F_{pc} = A \cdot I \cdot S \cdot C_p \cdot \left(1 + \frac{h_z}{H}\right) \cdot W_p$$

F_{pc} : Horizontal forces for secondary structural elements of superstructure.

A : Zone-specific acceleration coefficient.

I: Structural importance factor.

S: Site-specific acceleration coefficient.

C_p : Lateral force coefficient.

W: Dead load contribution of the parapet.

Table III. 6: Seismic coefficients.

A	I	S	C_p	H (m)	W (kn)	F_{pc} (kn)
0.25	1	1.3	0.8	0.6	2.54	2.64

$$Q = \max (Q; F_{pc}) \Rightarrow Q = \max (1; 2.64) \Rightarrow Q = 2.64 \text{ kn}$$

III.3.3 Assessment of structural actions

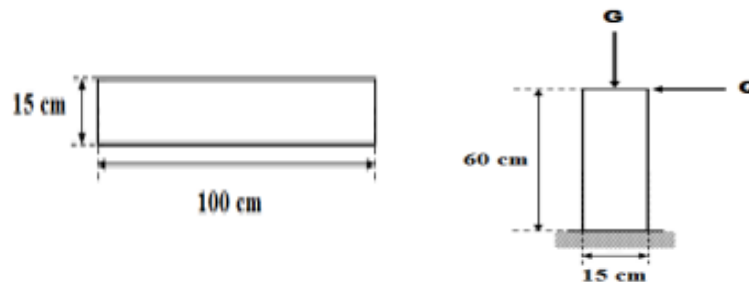


Figure III. 4: Parapet.

❖ Permanent load

$$S = (0.6 \times 0.15) + \left(\frac{0.1 \times 0.03}{2} \right) + (0.1 \times 0.1) = 0.1015 \text{ m}^2$$

$$G = \gamma \times S = 25 \times 0.1015 = 2.54 \text{ kN/ml}$$

❖ Live load: $Q = 2.64 \text{ kN/ml}$

Table III. 7: Internal forces of parapet.

Internal forces	ULS	SLS
Axial force	$N_u = 1.35 G = 3.429 \text{ kN}$	$N_s = G = 2.54 \text{ kN}$
Shear force	$V_u = 1.5Q = 3.96 \text{ kN}$	$V_s = Q = 2.64 \text{ kN}$
Bending moment	$M_u = 1.5Qh = 2.376 \text{ kN.ml}$	$M_s = 0.6 Q = 1.584 \text{ kN.ml}$

III.3.4 The reinforcement design

The reinforcement design is performed under combined bending condition at a section
 $(b \times h) = (15 \times 100) \text{ cm}^2$

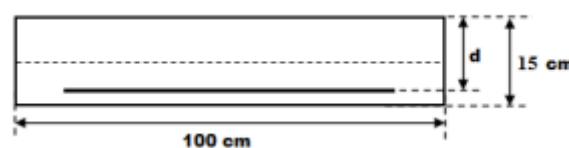


Figure III. 5: Parapet cross-section.

Table III. 8: Geometric and mechanical characteristics of parapet.

b (cm)	h (cm)	d (cm)	f_{c28}(MPa)	f_{bu}(MPa)	f_e (MPa)	f_{ed}(MPa)
100	15	13.5	25	14.167	400	347.826

❖ **Determination of the eccentricity**

$$e = \frac{Mu}{Nu} \Rightarrow e = \frac{2.376}{3.429} \Rightarrow e = 0.692 \text{ m.}$$

$$\frac{h}{2} = \frac{0.15}{2} = 0.075 \text{ m.}$$

$$e > \frac{h}{2} \Rightarrow \text{The pressure center lies outside the cross-section} \Rightarrow \text{Partially compressed section.}$$

❖ **Determination of the equivalent bending moment**

$$M_f = Mu + Nu \left(d - \frac{h}{2}\right) \Rightarrow M_f = 2.581 \text{ kN.m}$$

$$A = A - \frac{Nu}{F_s} \text{ (combined bending)}$$

According to CBA93, the minimum area of longitudinal reinforcement:

$$0.25\% B \quad \text{if: } f_{t28} \leq 2.4 \text{ MPa.} \quad B: \text{Design section.}$$

$$0.25\% B \quad \text{if: } f_{t28} > 2.4 \text{ MPa.}$$

❖ **Longitudinal reinforcement****Table III. 9:** Ultimate limit state reinforcement results for parapet.

M_f (kN.m)	u_{bu}	u_l	α	Z (cm)	A_s (cm²)	A_{min} (cm²)	A_{min} (CBA) (cm²)	A_{adop} (cm²)	Choice
2.581	0.00996	0.391	0.012	13.4	0.467	1.63	3.75	3.93	5HA10

❖ **Repair reinforcement**

$$A_r = \frac{A_s}{4} \Rightarrow A_r = \frac{3.93}{4} = 0.982 \text{ cm}^2 \quad \text{We adopt: 4T8} \Rightarrow A = 2.01 \text{ cm}^2$$

❖ **Spacing**

For longitudinal reinforcement:

$$St \leq \min(3h: 33 \text{ cm}) \Rightarrow St \leq \min(3 \times 15 : 33 \text{ cm}) \Rightarrow St \leq 33 \text{ cm}$$

$$St = \frac{100}{5} = 20 \text{ cm} \Rightarrow St = 20 \text{ cm} < 33 \text{ cm} \dots \text{CV}$$

For repair reinforcement:

$$St \leq \min(4h: 45 \text{ cm}) \Rightarrow St \leq \min(4 \times 15 : 45 \text{ cm}) \Rightarrow St \leq 45 \text{ cm}$$

$$St = \frac{60}{4} = 15 \text{ cm} \Rightarrow St = 15 \text{ cm} < 45 \text{ cm} \dots \text{CV} \quad \textbf{Ultimate limit state verification:}$$

❖ **Assessment of ductile behavior**

$$A_s = 3.93 \text{ cm}^2 > A_{\min} = 1.63 \text{ cm}^2 \dots \dots \text{CV}$$

❖ Verification of shear force

$$\tau_u = \frac{V}{b \times d} \Rightarrow \tau_u = \frac{3.96 \times 10^{-3}}{1 \times 0.135} \Rightarrow \tau_u = 0.029 \text{ MPa}$$

$$\bar{\tau}_u = \leq \min \left(\frac{0.15 \times f_{c28}}{\gamma_s}; 5 \text{ MPa} \right) \Rightarrow \bar{\tau}_u = \leq \min \left(\frac{0.15 \times 25}{1.15}; 4 \text{ MPa} \right) \Rightarrow \bar{\tau}_u = 2.5 \text{ MPa.}$$

$$\tau_u = 0.029 \text{ MPa} < \bar{\tau}_u = 2.5 \text{ MPa} \dots \text{CV.}$$

❖ Serviceability limit state:

Table III. 10: Serviceability limit state verification for parapet.

M_s (kN.ml)	b (cm)	d (cm)	A' (cm ²)	A_s (cm ²)	y (cm)	I (cm ⁴)	σ_{bc} (MPa)	σ_{st} (MPa)	Condition
1.584	100	13.5	0	3.93	3.443	7322.876	0.744	32.631	CV

III.3.5 Parapet reinforcement detailing

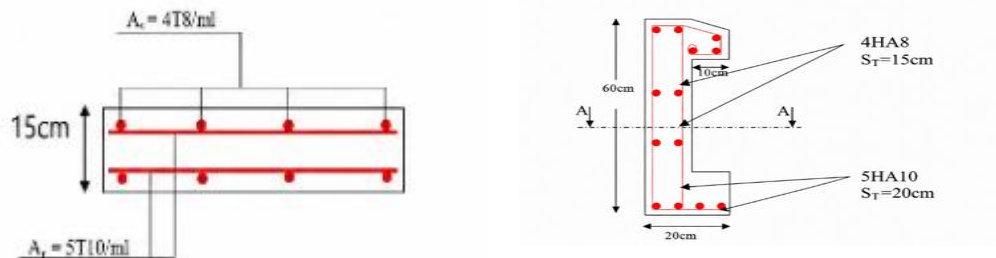


Figure III. 6: Parapet reinforcement detailing.

III.4 Balcony

III.4.1 Design principle

The balcony is treated as a cantilevered element under simple bending. The design process focuses on calculating bending moments at the Ultimate Limit State (ULS) and checking performance at the Serviceability Limit State (SLS). The slab is dimensioned according to ULS then verified for SLS, including assessments of on detrimental cracking.

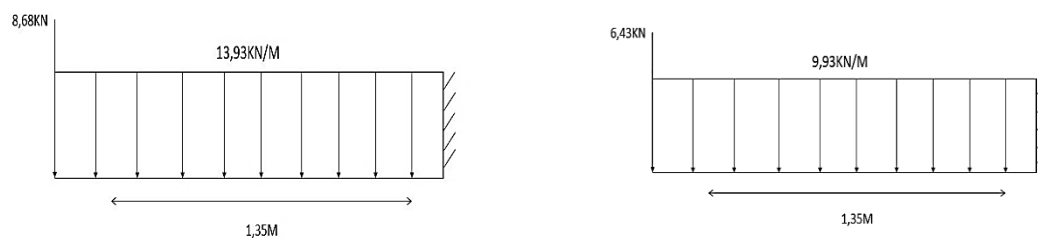


Figure III. 7: Structural system of the balcony at ULS and SLS.

III.4.2 Assessment of structural actions

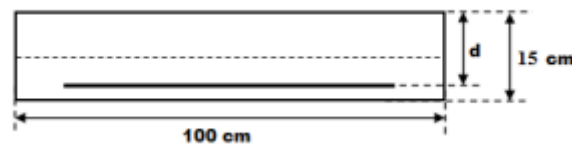
- Permanent load: $G = 6.43 \text{ kN/m}^2$
- Live load: $Q = 3.5 \text{ kN/m}^2$
- Railing load: P

Table III. 11: Internal forces of the balcony.

Internal forces	ULS	SLS
Combined load (kN/ml)	$q_u = 1.35G + 1.5Q = 13.93$	$q_s = G + Q = 9.93$
Railing load (kN)	$P_u = 1.35G = 8.680$	$P_s = G = 6.43$
Bending moment (kN.m)	$M_u = -\frac{q_u \times l^2}{2} - P_u \times l$ $= -24.411$	$M_s = -\frac{q_s \times l^2}{2} - P_s \times l$ $= -17.729$
Shear force (kN)	$V_u = -q_u \times l - P_u =$ -27.485	$V_u = -q_s \times l - P_s =$ -19.835

III.4.3 The reinforcement design

The reinforcement design is performed under simple bending condition at a section (b x h) = (15x100) cm²

**Figure III. 8:** Solid slab cross-section.**Table III. 12:** Geometric and mechanical characteristics of solid slab.

b (cm)	h (cm)	d (cm)	$f_{c28}(\text{MPa})$	$f_{bu}(\text{MPa})$	$f_e(\text{MPa})$	$f_{ed}(\text{MPa})$
100	15	13.5	25	14.167	400	347.826

❖ Longitudinal reinforcement**Table III. 13:** Ultimate limit state reinforcement results for balcony.

M_f (kN. ml)	u_{bu}	u_l	α	z (cm)	A_s (cm ²)	A_{min} (cm ²)	A_{adop} (cm ²)	Choice
24.411	0.094	0.391	0.024	13.3	5.276	1.630	7.7	5HA14

❖ Repair reinforcement

$$A_r = \frac{A_s}{4} \Rightarrow A_r = \frac{5.276}{4} = 1.319 \text{ cm}^2 \quad \text{We adopt: 5T8} \Rightarrow A = 2.51 \text{ cm}^2$$

❖ Spacing

For longitudinal reinforcement:

$$St \leq \min(3h: 33 \text{ cm}) \Rightarrow St \leq \min(3 \times 15 : 33 \text{ cm}) \Rightarrow St \leq 33 \text{ cm}$$

$$St = \frac{100}{5} = 20 \text{ cm} \Rightarrow St = 20 \text{ cm} < 33 \text{ cm} \dots \text{ CV}$$

For repair reinforcement:

$$St \leq \min(4h: 45 \text{ cm}) \Rightarrow St \leq \min(4 \times 15 : 45 \text{ cm}) \Rightarrow St \leq 45$$

$$St = \frac{60}{4} = 15 \text{ cm} \Rightarrow St = 15 \text{ cm} < 45 \text{ cm} \dots \text{ CV}$$

❖ Ultimate limit state verification

• Assessment of ductile behavior

$$A_s = 7.7 \text{ cm}^2 > A_{\min} = 1.63 \text{ cm}^2 \dots\dots \text{CV}$$

• Verification of shear force

$$\tau_u = \frac{V}{b \times d} \Rightarrow \tau_u = \frac{27.485 \times 10^{-3}}{1 \times 0.135} \Rightarrow \tau_u = 0.203 \text{ MPa}$$

$$\bar{\tau}_u = \leq \min \left(\frac{0.15 \times f_{c28}}{\gamma_s}; 5 \text{ MPa} \right) \Rightarrow \bar{\tau}_u = \leq \min \left(\frac{0.15 \times 25}{1.15}; 4 \text{ MPa} \right) \Rightarrow \bar{\tau}_u = 2.5 \text{ MPa.}$$

$$\tau_u = 0.203 \text{ MPa} < \bar{\tau}_u = 2.5 \text{ MPa} \dots\dots \text{CV Serviceability limit state:}$$

Table III. 14: Serviceability limit state verification for balcony.

M_s (kN.m)	A' (cm ²)	A_s (cm ²)	y (cm)	I (cm ⁴)	σ_{bc} (MPa)	σ_{st} (MPa)	Condition
17.729	0	7.7	4.547	12391.714	6.505	192.137	CV

III.4.4 Balcony reinforcement detailing

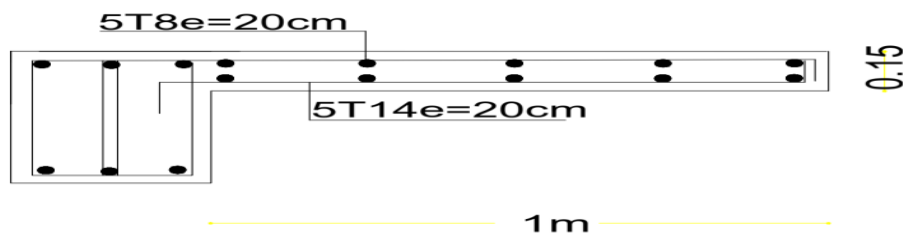


Figure III. 9: Balcony reinforcement detailing.

III.5 Stairs

III.5.1 Design principle

The design of stairs is based on simple bending analysis focusing on determining moments at the Ultimate Limit State (ULS) and checking performance at the Serviceability Limit State (SLS). The slab designed at ULS and then checked at SLS, considering non detrimental cracking.

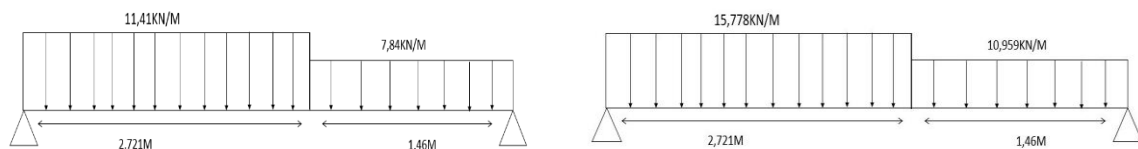


Figure III. 10: Structural system of the stair at ULS and SLS.

III.5.2 Assessment of structural actions

- Waist slab: $G = 7.55 \text{ kN/m}^2$

- Landing: $G = 5.34 \text{ kN/m}^2$
- Live load: $Q = 3.5 \text{ kN/m}^2$

Table III. 15: Load combination of stair.

Load combination	ULS ($q_u=1.35G+1.5Q$)	SLS ($q_s=G+Q$)
Waist slab	15.778 kN	11.41 kN
Landing	10.959 kN	7.84 kN

Table III. 16: Internal forces of stair.

Reactions	ULS	SLS
R_A (kN)	31.67	22.89
R_B (kN)	27.09	19.50
M (kN.ml)	31.78	22.93
V (kN)	31.67	22.89

❖ Determination of bending moments

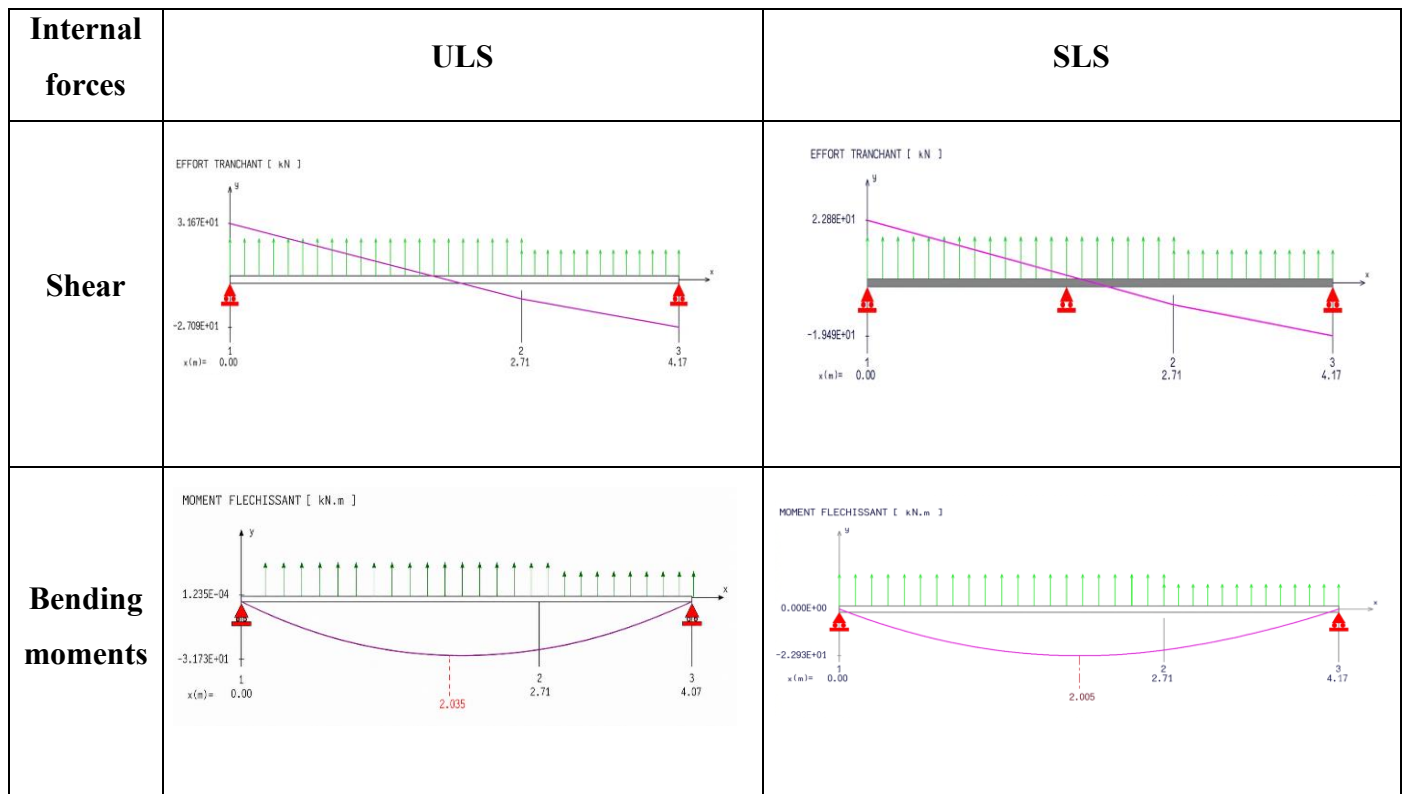
In support: $M_a = 0.85 M_{max}$

In span: $M_t = -0.3 M_{max}$

Table III. 17: Bending moment in support and span of stair.

	Moments	ULS	SLS
Support	M_a (kN.ml)	- 9.534	-6.879
Span	M_t (kN.ml)	27.013	19.49

Table III. 18: Diagrams of bending moments and shear forces at ULS and SLS of stair.



III.5.3 The reinforcement design

The reinforcement design is performed under simple bending condition at a section
 $(b \times h) = (15 \times 100) \text{ cm}^2$

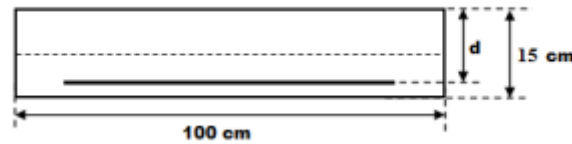


Figure III. 11: Stair cross-section.

Table III. 19: Geometric and mechanical characteristics of stair.

b (cm)	h (cm)	d (cm)	$f_{c28}(\text{MPa})$	$f_{bu}(\text{MPa})$	$f_e(\text{MPa})$	$f_{ed}(\text{MPa})$
65	20	18	25	14.167	400	347.826

❖ Longitudinal reinforcement

Table III. 20: Ultimate limit state reinforcement results for stairs.

	M_u (kN.m)	u_{bu}	u_l	α	z (m)	A_s (cm ²)	A_{min} (cm ²)	A_{adop} (cm ²)	Choice
Support	9.534	4.991×10^{-3}	0.391	6.254×10^{-3}	13.4	2.047	1.630	4.71	6HA10
Span	27.013	0.014	0.391	0.017	13.4	5.801	1.630	6.79	6HA12

❖ Repair reinforcement

$$\text{In support: } A_r = \frac{A_s}{4} \Rightarrow A_r = \frac{4.71}{4} = 1.177 \text{ cm}^2 \Rightarrow \text{We adopt: 4T8} \Rightarrow A = 2.01 \text{ cm}^2$$

$$\text{In span: } A_r = \frac{A_s}{4} \Rightarrow A_r = \frac{6.79}{4} = 1.697 \text{ cm}^2 \Rightarrow \text{We adopt: 4T8} \Rightarrow A = 2.01 \text{ cm}^2$$

❖ Spacing

For longitudinal reinforcement:

$$St \leq \min(3h: 33 \text{ cm}) \Rightarrow St \leq \min(3 \times 15 : 33 \text{ cm}) \Rightarrow St \leq 33 \text{ cm}$$

$$St = \frac{100}{6} = 16.666 \text{ cm} \Rightarrow St = 15 \text{ cm} < 33 \text{ cm} \dots \text{ CV}$$

For repair reinforcement:

$$St \leq \min(4h: 45 \text{ cm}) \Rightarrow St \leq \min(4 \times 15 : 45 \text{ cm}) \Rightarrow St \leq 45 \text{ cm}$$

$$St = \frac{100}{4} = 25 \text{ cm} \Rightarrow St = 25 \text{ cm} < 45 \text{ cm} \dots \text{ CV}$$

❖ Ultimate limit state verification

• Assessment of ductile behavior

$$\text{In support: } A_s = 4.71 \text{ cm}^2 > A_{min} = 1.63 \text{ cm}^2 \dots \text{ CV}$$

$$\text{In span: } A_s = 3.835 \text{ cm}^2 > A_{min} = 1.63 \text{ cm}^2 \dots \text{ CV}$$

- **Verification of shear force**

$$\tau_u = \frac{V}{b \times d} \Rightarrow \tau_u = \frac{31.68 \times 10^{-3}}{1 \times 0.135} \Rightarrow \tau_u = 0.234 \text{ MPa}$$

$$\bar{\tau}_u \leq \min \left(\frac{0.15 \times f_{ct28}}{\gamma_b}; 5 \text{ MPa} \right) \Rightarrow \bar{\tau}_u \leq \min \left(\frac{0.15 \times 25}{1.5}; 4 \text{ MPa} \right) \Rightarrow \bar{\tau}_u = 2.5 \text{ MPa.}$$

$$\tau_u = 0.234 \text{ MPa} < \bar{\tau}_u = 2.5 \text{ MPa} \dots\dots \text{CV.}$$

- **Effect on bottom longitudinal reinforcing bars**

$$V_{u \max} = 31.68 \text{ kN} \quad ; \quad M_{\text{support}} = 9.543 \text{ kN}$$

$$A_{s \text{ support}} = 6.79 \text{ cm}^2 \geq \frac{1.15}{f_e} \left(T_u + \frac{M_{\text{support}}}{0.9d} \right) = 3.168 \text{ cm}^2 \dots\dots \text{CV}$$

- ❖ **Serviceability limit state**

Table III. 21: Serviceability limit state reinforcement results for stairs.

	$M_s (\text{kN. ml})$	$A_s (\text{cm}^2)$	$y (\text{cm})$	$I (\text{cm}^4)$	$\sigma_{bc} (\text{MPa})$	$\sigma_{bc} \leq \bar{\sigma}_{bc}$
Support	6.888	4.71	3.717	8473.519	3.021	CV
Span	19.49	6.79	4.323	11270.523	7.485	CV

As cracking is considered non-detrimental, there is no need to verify stresses in the reinforcement.

- ❖ **Stair reinforcement detailing**

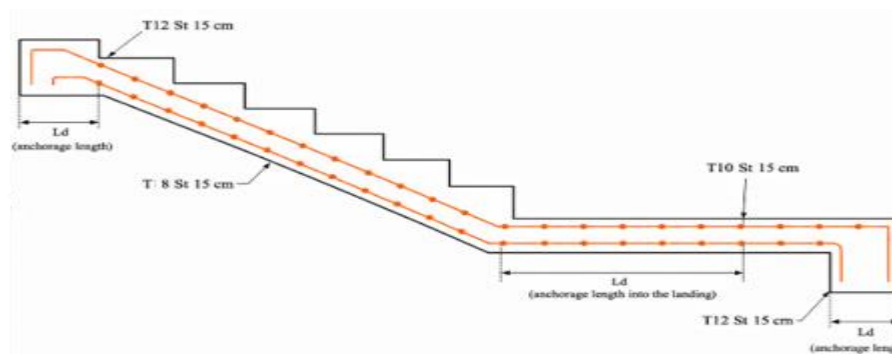


Figure III. 12: Stair reinforcement detailing.

III.6 Landing beam

III.6.1 Design principle

The design of landing beam is based on simple bending analysis focusing on determining moments at the Ultimate Limit State (ULS) and checking performance at the Serviceability Limit State (SLS). The slab designed at ULS and then checked at SLS, considering non detrimental cracking.

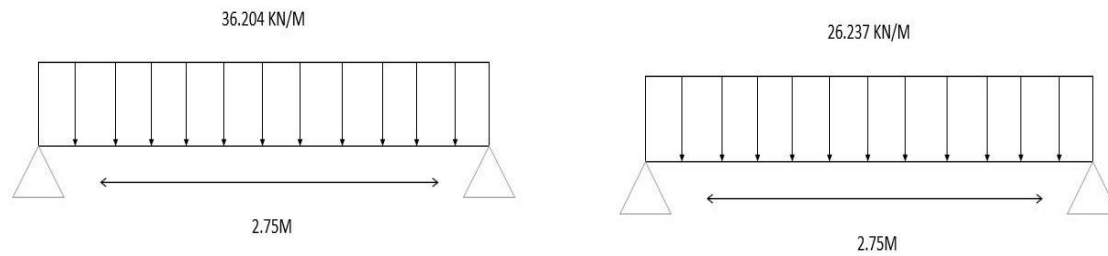


Figure III. 13: Structural system of the landing beam at ULS and SLS.

III.6.2 Assessment of structural actions

- $g_p = 25 \times b \times h \Rightarrow g_p = 25 \times 0.3 \times 0.4 \Rightarrow g_p = 3 \text{ kN.}$
- $g_m = G_{\text{mas}} \times h_{\text{mas}} \Rightarrow g_m = 2.81 \times \frac{(3.06 - 0.4)}{2} \Rightarrow g_p = 3.737 \text{ kN.}$

Landing support reaction at ULS: $R_u = 27.11 \text{ kN}$

Landing support reaction at SLS: $R_s = 19.50 \text{ kN}$

Table III. 22: Load combination of landing beam.

Load combination	ULS	SLS
$q_u = 1.35(g_p + g_m) + R_u$	36.204	/
$q_s = (g_p + g_m) + R_s$	/	26.237

Table III. 23: Internal forces of landing beam.

Internal forces	ULS	SLS
Bending moment M (kN.ml)	34.22	24.80
Shear force V (kN)	49.78	36.08

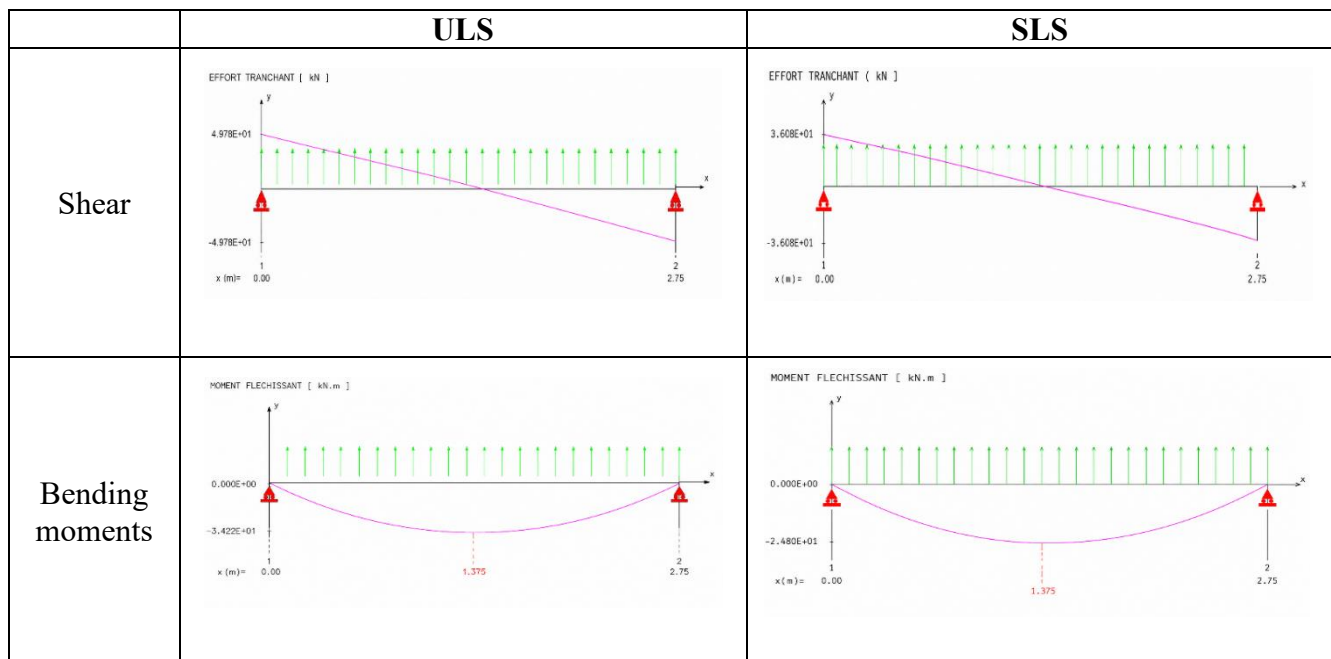
❖ Determination of bending moments

In support: $M_a = 0.85 M_{\text{max}}$

In span: $M_t = -0.3 M_{\text{max}}$

Table III. 24: Bending moment in support and span of landing beam.

	Moments	ULS	SLS
Support	M_a (kN.ml)	-10.266	-7.44
Span	M_t (kN.ml)	29.087	21.08

Table III. 25: Diagrams of bending moments and shear forces at ULS and SLS of landing beam.

III.6.3 The reinforcement design

Table III. 26: Geometric and mechanical characteristics of stair.

b (cm)	h (cm)	d (cm)	$f_{c28}(\text{MPa})$	$f_{bu}(\text{MPa})$	$f_e(\text{MPa})$	$f_{ed}(\text{MPa})$
30	40	36	25	14.167	400	347.826

❖ Longitudinal reinforcement

	M_u kN.m	u_{bu}	u_l	α	z (cm)	A_s (cm ²)	A_{min} (cm ²)	A_{adop} (cm ²)	Choice
Support	10.266	0.018	0.391	0.022	35.683	1.163	1.304	4.62	3HA14
Span	29.087	0.052	0.391	0.066	35.049	3.835	1.304	4.62	3HA14

❖ Ultimate limit state verification

• Assessment of ductile behavior

In support: $A_s = 1.163 \text{ cm}^2 < A_{min} = 1.304 \text{ cm}^2$ CNV

In span: $A_s = 3.835 \text{ cm}^2 > A_{min} = 1.304 \text{ cm}^2$ CV

• Verification of shear force

$$\tau_u = \frac{V}{b \times d} \Rightarrow \tau_u = \frac{49.78 \times 10^{-3}}{0.3 \times 0.36} \Rightarrow \tau_u = 0.461 \text{ MPa}$$

$$\bar{\tau}_u = \leq \min \left(\frac{0.15 \times f_{c28}}{\gamma_s}; 5 \text{ MPa} \right) \Rightarrow \bar{\tau}_u = \leq \min \left(\frac{0.15 \times 25}{1.15}; 4 \text{ MPa} \right) \Rightarrow \bar{\tau}_u = 2.5 \text{ MPa.}$$

$$\tau_u = 0.461 \text{ MPa} < \bar{\tau}_u = 2.5 \text{ MPa} \text{ CV.}$$

• Transverse reinforcement

✓ According BAEL91/99

$$\phi_t \leq \min\left(\frac{h}{35}; \frac{b_0}{10}; \phi_l\right)$$

ϕ_l : Minimum diameter of longitudinal reinforcement bars $\phi_l = 1.4$ cm

We adopt: 4HA8 $\Rightarrow A_t = 2.02 \text{ cm}^2$

Transverse reinforcement spacing:

$$S_t \leq \min(0.9 h; 40 \text{ cm}) \Rightarrow S_t \leq \min(0.9 \times 36; 40 \text{ cm}) \Rightarrow S_t \leq 32.4 \text{ cm} \Rightarrow S_t = 10 \text{ cm}$$

✓ According RPA2024

Maximum spacing of transverse reinforcement:

In critical zone

$$s = \min\left(\frac{h}{4}; 24\phi_t; 17.5 \text{ cm}; 6\phi_l\right) \Rightarrow s = 7.2 \text{ cm on adopt } s = 10 \text{ cm}$$

ϕ_l : Minimum diameter of longitudinal reinforcement bars.

Non critical zone

$$s' = \min\left(\frac{h}{4}; 12\phi_l\right) \Rightarrow s' = 10 \text{ cm}$$

$$s' \leq \frac{h}{2} \Rightarrow s' = \leq \frac{h}{2} = \frac{20}{2} = 10 \text{ cm}$$

$$s' \leq 10 \text{ cm} \dots \text{ CV}$$

$$A_t = (0.003) \times s \times b_0 \Rightarrow A_t = (0.003) \times 10 \times 30 \Rightarrow A_t = 1.08 \text{ cm}^2$$

$$A_t = 2.02 \text{ cm}^2 \Rightarrow 4\text{HA8}$$

✓ Effect on bottom longitudinal reinforcing bars

$$V_{u \max} = 49.78 \text{ kN} \quad ; \quad M_{\text{support}} = 10.266 \text{ kN.m}$$

$$A_{s \text{ support}} = 6.79 \text{ cm}^2 \geq \frac{1.15}{f_e} \left(V_u + \frac{M_{\text{support}}}{0.9d} \right) = 2.342 \text{ cm}^2 \dots \dots \text{ CV}$$

✓ Serviceability limit state

	$M_s (\text{kN.m})$	$A (\text{cm}^2)$	$Y (\text{cm})$	$I (\text{cm}^4)$	$\sigma_{bc} (\text{MPa})$	$\sigma_{bc} \leq \bar{\sigma}_{bc}$
Support	7.44	4.62	10.791	56605.365	4.018	CV
Span	21.08	4.62	10.791	56605.365	4.018	CV

As cracking is considered non-detrimental, there is no need to verify stresses in the reinforcement.

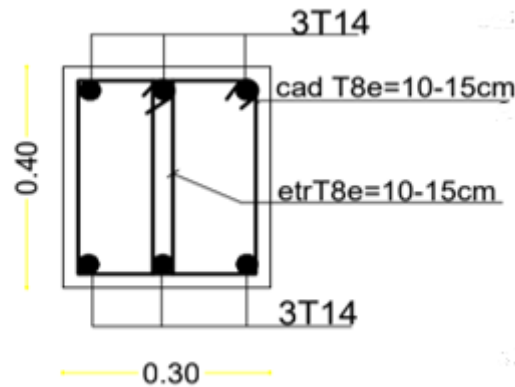


Figure III. 14: Landing beam reinforcement detailing.

III.7 Conclusion

Secondary elements play an essential role in maintaining the stability and functional performance of the R+8 structure. Even if they are not part of the main resisting system, they contribute to efficient load and distribution and enhance the overall behavior of the building. For this project, the study and design of slabs, stairways, and parapet were performed according to reinforced concrete requirements and current design regulation. The different structural checks demonstrated that stresses, deflections, and reinforcement arrangements remain within the permissible limits.

The final results show that these components provide adequate resistance and stiffness under both static and seismic effects. Their proper design therefore contributes to the safety, durability, and long-term performance of the structure.

Chapter IV:

Seismic analysis

IV.1 Introduction

In the seismic design of multi-story reinforced concrete structures, the accurate quantification of seismic demands is paramount to ensuring life safety and structural integrity. As buildings deviate from ideal regularity in both plan and elevation, the response to seismic excitation becomes increasingly complex, requiring a rigorous analytical approach. This chapter addresses the seismic analysis of the subject building, which is designed within the regulatory framework of the Algerian seismic code (RPA2024) and situated in the highly demanding seismic zone 5 on a soft soil site (site class S3).

The chapter begins by establishing the fundamental structural geometry and the loading criteria (dead and live loads) that serve as inputs for the analysis. Recognizing the complexity introduced by the structure's inherent irregularities, this study adopts a dual analytical methodology. While the equivalent static method (ESM) is briefly utilized as a reference, the primary analytical engine is the modal response spectrum analysis (MRSRA). This choice is necessitated by RPA 2024 (clause 4.1.3), which mandates dynamic analysis for the structures exhibiting plan irregularities to accurately capture torsional response and higher mode effects. Central to this chapter is the systematic verification of the structure's compliance with the numerous criteria mandated by the RPA 2024 regulation, this includes a detailed assessment of regularity in both plan and elevation, the calculation of torsional radii to mitigate torsional instability, and the determination of the building's fundamental period. Furthermore, the lateral force-resisting system is characterized, confirming the dominant role of structural walls. The analysis concludes with the comprehensive safety verification including base shear correction, P-Delta effects, inter-story drifts, damage limitation, and overall stability against overturning to ensure that the structural configuration meets the strict performance criteria of modern seismic codes.

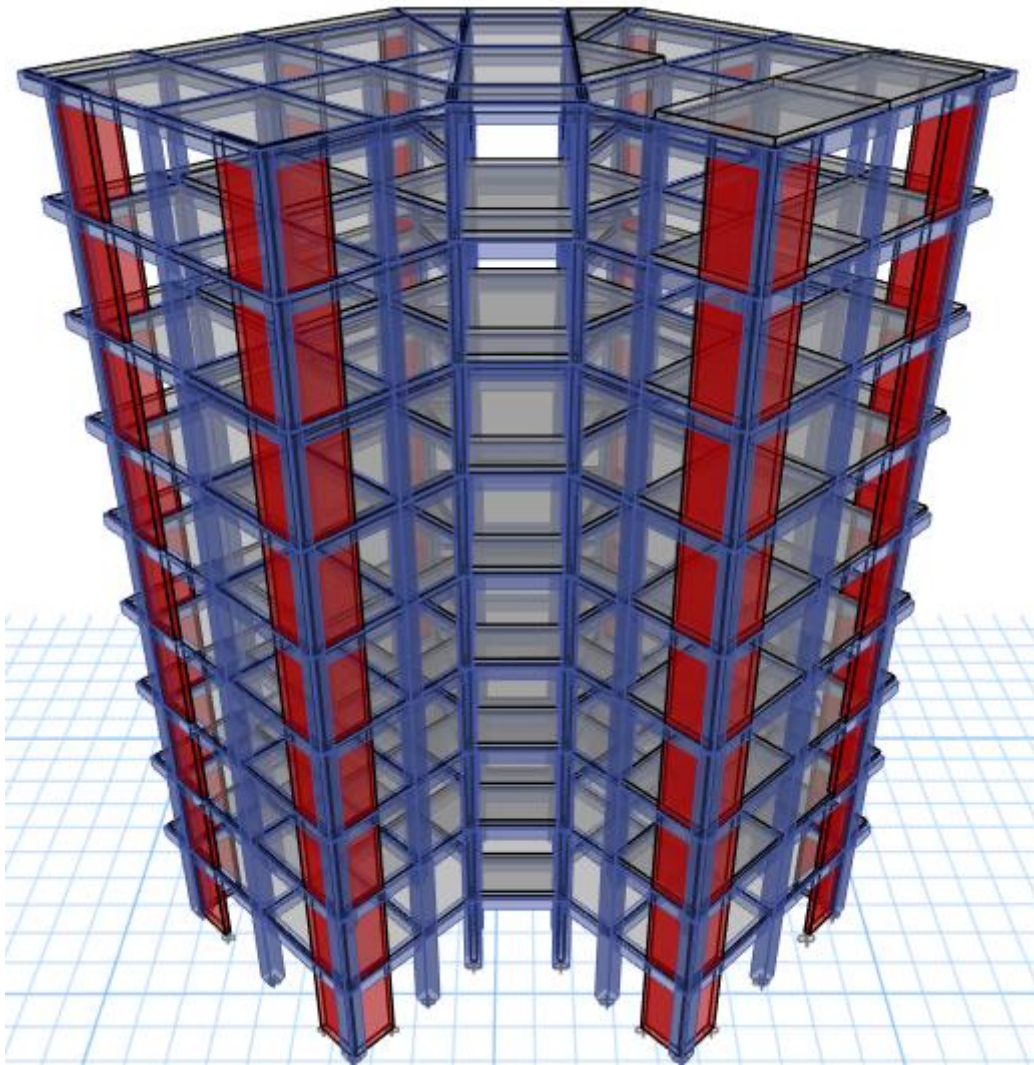


Figure IV. 1: Three-dimensional view.

IV.2 Quality factory (table 3.18 RPA24)

$$Qf = 1 + \sum_{1}^n Pq$$

❖ Regularity in plan

If either of the two conditions is not met, the structure is considered irregular in plan.
Calculation of r_x (torsion radius) and I_s (Radius of Gyration)

- The building configuration should remain compact, with a floor plan length-to- width ratio not exceeding 4.
- The total extent of recessed or projecting sections of the building in a given direction shall not exceed 25% of the overall building dimension measured along the same direction. (Art a3 3.7.1 RPA 2024)

$$0.25 \leq \frac{L_x}{L_y} \leq 4 \Rightarrow \frac{L_x}{L_y} = \frac{23.56}{14.60} = 1.61$$

$$\frac{l_x}{L_x} \leq 0.25 \Rightarrow \frac{10.204}{23.56} = 0.43 \dots \text{CNV}$$

$$\frac{l_y}{L_y} \leq 0.25 \Rightarrow \frac{9.26}{14.60} = 0.63 \text{CNV}$$

According to article 3.7.2 of the RPA, the building is classified as irregular in plan

- **Regularity in elevation**

For any two adjacent story, the variation of the masse-to-stiffness ratio shall be limited to 25% in each seismic analysis direction. (Art b3 3.7.1 RPA 2024)

Table IV. 1: Verification of regularity in elevation.

STORY	Mass (kg)	K_x (kN/m)	K_y (kN/m)	M/ K_x	M/ K_y	$\frac{M_{i-1}/K_{x i-1}}{M_i/K_{x i}}$	$\frac{M_{i-1}/K_{y i-1}}{M_i/K_{y i}}$	0.25
story 9	225749,71	237024,817	252420,804	0,952	0,894	0,616	0,621	CNV
story 8	210725,52	358612,718	379297,455	0,587	0,555	0,858	0,821	CNV
story 7	214466,43	425019,658	469777,411	0,504	0,456	0,902	0,879	CNV
story 6	222110,25	487658,913	553161,389	0,455	0,401	0,935	0,901	CNV
story 5	230456,45	540638,209	637004,065	0,426	0,361	0,903	0,865	CNV
story 4	234082,58	607864,706	747426,375	0,385	0,313	0,866	0,816	CNV
story 3	238286,43	713918,927	931773,266	0,333	0,255	0,726	0,688	CNV
story 2	242562,14	1000272,55	1377637,21	0,242	0,176	0,508	0,499	CNV
story 1	247379,15	2005388,60	2810796,55	0,123	0,088	0	0	CNV

Note: According to article 3.7.3 of RPA2024, the building is classified as irregular in elevation.

Conclusion: The structure is classified as irregular.

Table IV. 2: Values of the quality factor Q.

category	Criterion q	Values	x-x dir	y-y dir
			q	q
a	Regularity in plan	0.05		
	Regularity in elevation	0.2		
	Minimum conditions on the number of stories	0.2		
	Minimum conditions on the number of bays	0.1		
b	Regularity in plan	0.05	✓	✓
	Regularity in elevation	0.02	✓	✓
	Redundancy in plan	0.05		
Pq			0.25	0.25
Q			1.25	1.25

IV.3 Shear wall layout

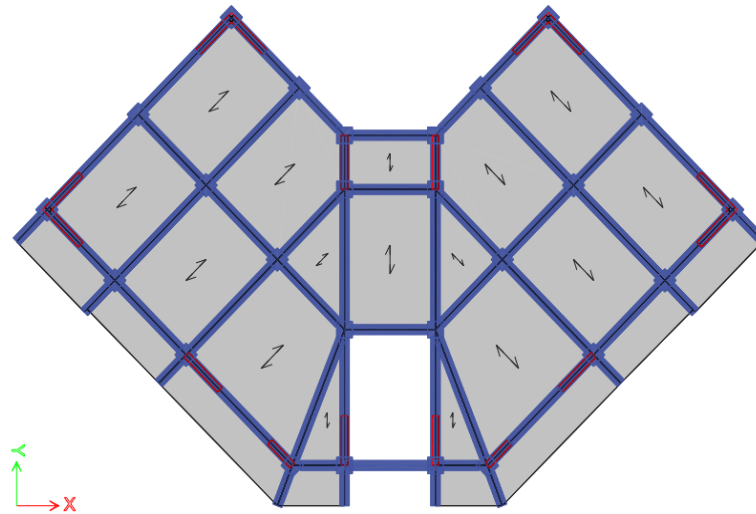


Figure IV. 2: Shear wall layout.

IV.4 The geometric and mass characteristics of the structure

In this step, the center of mass, the center of rigidity are calculated, and the accidental eccentricity condition of RPA 2024 art 4.2.7 is verified.

$$\text{Eccentricity: } \begin{cases} e_x = [X_{CM} - X_{CG}] \\ e_y = [Y_{CM} - Y_{CG}] \end{cases} \quad \text{Eccentricity: } \begin{cases} e_{ax} = 0.05L_x \\ e_{ay} = 0.05L_y \end{cases}$$

$$L_x = 23.56m \quad L_y = 14.70m$$

Table IV. 3: Geometric and mass characteristics of the structure.

Story	Mass (Kg)	Center of Mass		Center of rigidity		Eccentricity		Accidental eccentricity		$e_{x,y}$
		X-X	Y-Y	X-X	Y-Y	$e_x(m)$	$e_y(m)$	$e_{ax}(m)$	$e_{ay}(m)$	$< e_{ax,y}$
						0.0095	0.4982	1.162	0.734	✓
Story1	247379,15	11,773	7,628	11,783	8,0167	0.0106	0.366	1.162	0.734	✓
Story2	242562,14	11,773	7,619	11,7839	7,75	0.0125	0.2205	1.162	0.734	✓
Story3	238286,43	11,771	7,613	11,7843	7,3833	0.0132	0.1084	1.162	0.734	✓
Story4	234082,58	11,771	7,601	11,7843	7,1069	0.0138	0.1235	1.162	0.734	✓
Story5	230456,45	11,770	7,592	11,7842	6,8945	0.0138	0.2598	1.162	0.734	✓
Story6	222110,25	11,769	7,567	11,784	6,7255	0.0136	0.3902	1.162	0.734	✓
Story7	214466,43	11,769	7,542	11,7837	6,5844	0.0137	0.611	1.162	0.734	✓
Story8	210725,52	11,769	7,546	11,7833	6,4466	0.0158	0.448	1.162	0.734	✓
Story9	225749,71	11,767	7,211	11,7831	6,3063	0.0095	0.4982	1.162	0.734	✓

IV.5 Results of the dynamic analysis

IV.5.1 Modal participating mass ratios

Table IV. 4: Modal participating mass ratio.

Mode	Period (sec)	Modal Mass UX	Mode Mass UY	Mode Mass UZ	Cumulated Mass UX	Cumulated Mass UY	Cumulated Mass UZ	RZ
1	0.687	0.6915	0	0	0.6915	0	0	0.0333
2	0.631	0	0.7072	0	0.6915	0.7072	0	2.774×10^{-6}
3	0.535	0.0403	2.29×10^{-3}	0	0.7317	0.7072	0	0.6873
4	0.212	0.1322	0	0	0.8639	0.7072	0	0.0029
5	0.185	0	0.1478	0	0.8639	0.8551	0	0
6	0.16	0.0012	5.85×10^{-7}	0	0.8651	0.8551	0	0.138
7	0.105	0.0559	0	0	0.921	0.8551	0	1.09×10^{-6}
8	0.086	0	0.0625	0	0.921	0.9176	0	0

IV.5.2 Nature of the first three modes

Based on the participating mass ratio:

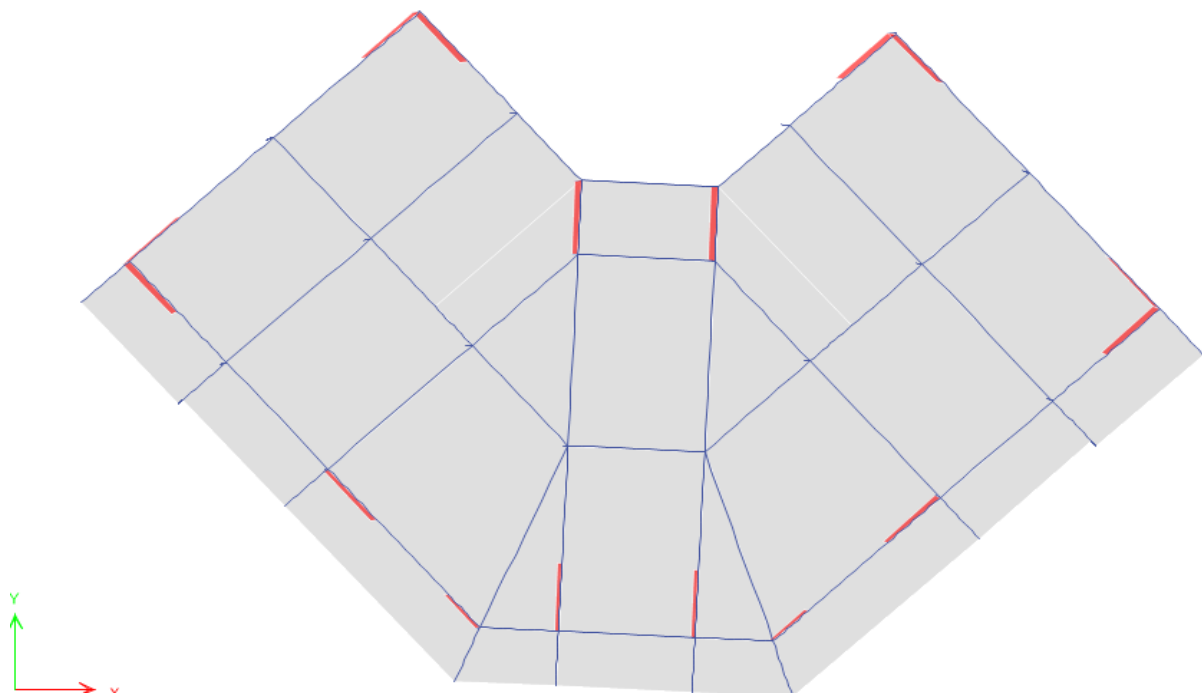


Figure IV. 3: 1st mode (T=0.68 s) translation along the X-axis.

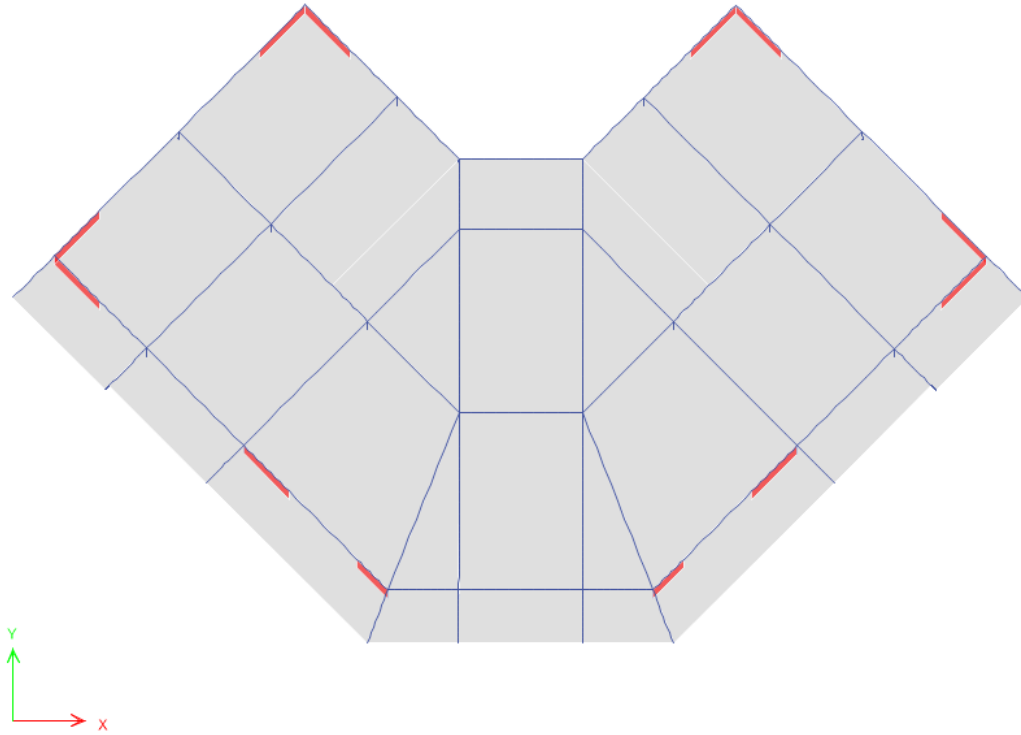


Figure IV. 4: 2nd mode (T=0.624 s) translation along the Y-axis.

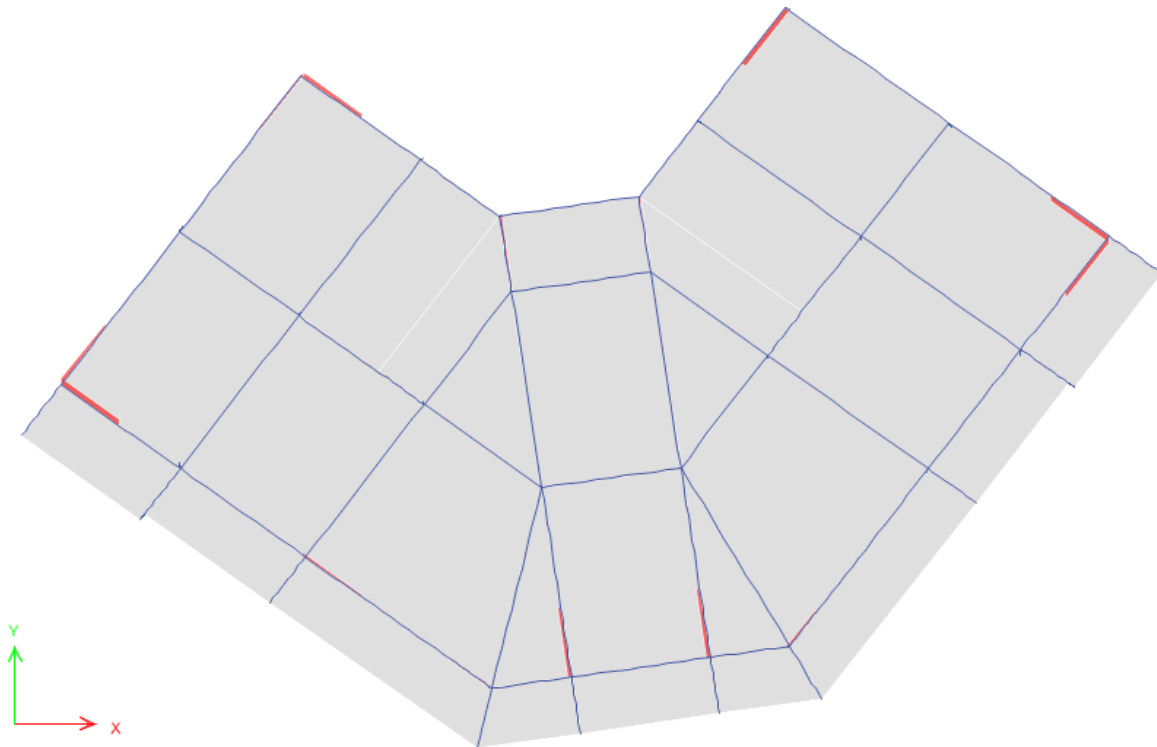


Figure IV. 5: 3rd mode (T=0.58) rotation.

IV.6 Behavior factor

IV.6.1 Verification of torsional behavior (core effect) (art 3.5 RPA2024)

$$r_x \leq l_s \quad \text{and} \quad r_y \leq l_s$$

Torsional radii in both X-X and Y-Y directions:

$$r_x = \sqrt{\frac{K_M}{K_{Fy}}} ; r_y = \sqrt{\frac{K_M}{K_{Fx}}}$$

With: K_M, K_{Fx} , and K_{fy} ; The coefficients were evaluated the horizontal displacements U_x and U_y , as well as the rotation R , produced by forces and a moment applied at the center of gravity. As these quantities may be assigned arbitrary values, the following

$$F_x = 10^6 kN ; F_y = 10^6 kN ; M = 10^6 kN.m$$

$$K_{Fx} = \frac{1}{U_{Fx}} ; K_{Fy} = \frac{1}{U_{Fy}} ; K_M = \frac{1}{R_M}$$

Table IV. 5: Torsional radii and verification of torsional behavior.

story	$U_x(m)$	$U_y(m)$	$R_z(rad)$	$K(F_x)$	$K(F_y)$	$K(F_{zx})$	r_x	r_y
terrace	66,6662	58,8178	0,903003	0,0150	0,0170	1,1074	8,0707	8,5923
Story8	61,8152	53,4614	0,826136	0,0162	0,0187	1,2105	8,0444	8,6501
Story7	55,8059	47,4278	0,738846	0,0179	0,0211	1,3535	8,0120	8,6909
Story6	48,6592	40,5615	0,638155	0,0206	0,0247	1,5670	7,9725	8,7321
Story5	40,4596	32,9684	0,524748	0,0247	0,0303	1,9057	7,9264	8,7808
Story4	31,3090	24,8503	0,400837	0,0319	0,0402	2,4948	7,8737	8,8379
Story3	21,6689	16,6473	0,272623	0,0461	0,0601	3,6681	7,8143	8,9153
Story2	12,1019	8,9654	0,149163	0,0826	0,1115	6,7041	7,7527	9,0073
Story1	3,9914	2,8415	0,048005	0,2505	0,0170	20,8312	7,6936	9,1184

IV.6.2 Mass radius of gyration

$$I_s = \sqrt{\frac{I_z}{m_{plancher}}}$$

Table IV. 6: Justification of the lateral load resisting system.

Story	Mass-X(kg)	Mass-Y (kg)	$l_s(x)$	$l_s(y)$
Story9	230289,79	230289,79	6,840	6,840
Story8	219805,69	219805,69	7,027	7,027
Story7	223546,59	223546,59	7,024	7,024
Story6	226650,33	226650,33	7,022	7,022
Story5	230456,45	230456,45	7,019	7,019
Story4	234082,58	234082,58	7,017	7,017
Story3	238286,43	238286,43	7,015	7,015
Story2	242562,14	242562,14	7,012	7,012
Story1	247379,15	247379,15	7,008	7,008

Conclusion: The condition $r_x \leq l_s$ and $r_y \leq l_y$ is not verified for all stories in both directions.

In accordance with RPA 2024 requirements, the lateral load-resisting system of the building

Table IV. 7: Fundamental period.

direction	F (kN)				
	Total base shear (kN)	Shear on column (kN)	Shear on walls (kN)	On columns %	On walls %
X-X	2943,1033	408,6883	2443,0379	13.886	83.008
Y-Y	3270,3148	344,4752	2891,0926	10.533	88.404

According to these results, the walls resist more than 65% of the total base shear. The system is considered a wall system (system 5), or as a mixed system equivalent to walls with $R=4.5$.

IV.7 Calculated of the fundamental period of the structure (art 4.2.4 RPA2024)

Table IV. 8: Fundamental period.

T num (s)	C_t	$h_N(m)$	$T_{emp}(s)$	$1.3 \times T_{num}$
0.687	0.05	30.77	0.65	0.893

IV.8 Seismic analysis

According to article 4.1.2b of RPA024, our building does not fulfil the conditions for use of the equivalent static method, therefor, the seismic force calculation will be performed using the modal response spectrum analysis. The seismic base shear is calculated by the following formula:

$$V = \lambda \times \frac{S_{ad}}{g}(T_0) \times W$$

$$\lambda = \begin{cases} 0.85 & \text{If } T_0 \leq 2 \times T_2 \text{ and building has more than 2 levels} \\ 1 & \text{Other case} \end{cases}$$

IV.8.1 Correction coefficient

- Seismic zone: **V**
- Spectrum type: **Type 1**
- Soil type: **S3**
- Importance group: **2**
- Zone acceleration coefficient [RPA2024, Table 3.3]: **A=0.25**
- Site coefficient [RPA2024, Table 3.4 and 3.5]: **T1=0.15s, T2=0.6s, T3=2s**
- Importance factor [RPA2024, Table 3.11]: **I=1**

- Structural behavior factor [RPA2024, Table 3.18]: For a mixed lateral load-resisting system equivalent to shear walls: $R=4.5(b)$.

Table IV. 9: The weight of each floor level.

$T_{num}(s)$	Type	$T_2(s)$	$2 \times T_2$	λ
0.687	Type 1	0.6	1.2	0.85

Table IV. 10: Correction coefficient.

Story	G (kN)	Q (kN)	ψ	$\Psi.Q$ (kN)	Pk=G+ $\psi.Q$ (kN)
Story 9	2517.163	212,818	0.3	63,845	2581.009
Story 8	2127.363	361,517	0.3	108,455	2235.819
Story 7	2127.414	361,545	0.3	108,463	2235.878
Story 6	2195.150	361,530	0.3	108,459	2303.609
Story 5	2195.177	361,545	0.3	108,463	2303.641
Story 4	2272.986	361,601	0.3	108,480	2381.467
Story 3	2273.396	356,768	0.3	107,030	2380.427
Story 2	2361.048	361,824	0.3	108,547	2469.596
Story 1	2361.048	361,824	0.3	108,547	2469.596
Total	20430.748	3100.977	/	930.293	21361.041

IV.8.2 Design spectrum and calculation base shear force

$$\frac{S_{ad}}{g}(T_0) = A.I.S. \left[2.5 \times \frac{Q_F}{R} \right] \cdot \left[\frac{T_2}{T_0} \right] \quad \text{If: } T_2 \leq T_0 \leq T_3$$

Table IV. 11: Seismic base shear value (static equivalent).

Direction	λ	W(kN)	Spectrum S_{ad}	V(KN)
X-X	0.85	21361.04	0.197	3576.906
Y-Y	0.85	21361.04	0.214	3885.573

IV.8.3 Modal response spectrum analysis method

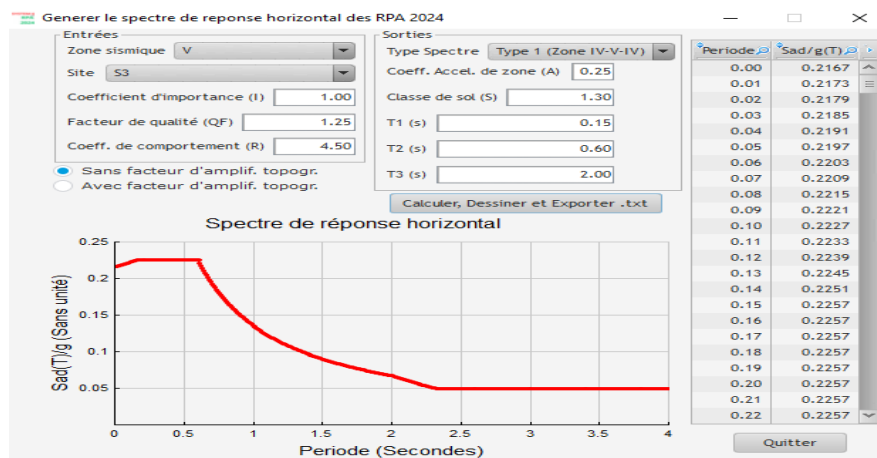


Figure IV. 6: The design spectrum used.

IV.8.3.1 Verification of the base shear

Table IV. 12: Verification of the base shear.

Direction	V_{static}	$0.8 \times V_{static}$	$V_{dynamic}$	Verification
X-X	3576.906	2861.524	2943.103	✓
Y-Y	3885.573	3108.458	3270.314	✓

IV.8.3.2 Distribution of seismic forces

$$V = F_t + \sum_{i=K}^n F_i \quad \text{with } F_t = \begin{cases} 0 & \text{If } T_0 < 0.7 \text{ s} \\ 0.07 T_0 \cdot V & \text{If } T_0 > 0.7 \text{ s} \end{cases}$$

$$\text{In our case: } \begin{cases} T_x = 0.687 \\ T_y = 0.631 \end{cases} \Rightarrow F_t = 0$$

Table IV. 13: Distribution of seismic forces.

Story	$V_x(\text{kN})$	$F_x(\text{kN})$	$V_y(\text{kN})$	$F_y(\text{kN})$
Story 8	690,9165	690,9165	788,6386	788.6386
Story 7	1206,5542	515,6377	1374,1576	585.519
Story 6	1642,2848	435,7306	1866,5745	492.4169
Story 5	2013,6428	371,3580	2281,8886	415.3141
Story 4	2325,7493	312,1065	2625,2246	343.336
Story 3	2580,1985	254,4492	2898,8697	273.6451
Story 2	2771,2081	191,0096	3098,2289	199.3592
Story 1	2894,7637	123,5556	3222,9619	124.733
Ground floor	2943,1033	48,3396	3270,3148	47.3529
	Sum	2943.1033	Sum	3270.3148

IV.9 Verification of the P- Δ effect

Second-order effects (or P-Δ effects) may be neglected in building if the following condition is satisfied at all stories levels:

$$\theta = \frac{P_k \cdot \Delta_k}{V_k \cdot h_k} \leq 0.1$$

P_k : Represents the total vertical load of the structure, including dead load and imposed loads above level (k):

$$\sum_{i=1}^n (G + \psi Q_i)$$

V_k : Story shear at level (k): $V_k = \sum_{i=1}^n F_i$

Δ_k : Relative lateral displacement between level (k) and the level just below it (k-1).

h_k : The height of story (k).

Depending on the value of θ_k , it is recommended that:

- When $0.10 \leq \theta_k \leq 0.20$, second-order (P- Δ) effects can be considered approximately by increasing the seismic response obtained from a first-order elastic analysis using the amplification factor: $(\frac{1}{1-\theta})$.
- When $\theta_k \geq 0.20$, the structure is considered unstable or unsafe, and a new structural design is required.

Table IV. 14: Verification of the P- Δ effect in the X-X direction.

Story	Δ_{Kx} (m)	P_K (kN)	V_K (kN)	h_k (m)	θ_x	Verification ($\theta_x \leq 0.1$)
Story 8	0.0137	2581,00881	690,9165	3.06	0,0167	✓
Story 7	0.0152	4816.828	1206,5542	3.06	0,0092	✓
Story 6	0.0170	7052.706	1642,2848	3.06	0,0075	✓
Story 5	0.0183	9356.315	2013,6428	3.06	0,0068	✓
Story 4	0.0191	11659.956	2325,7493	3.06	0,0061	✓
Story 3	0.0188	14041.423	2580,1985	3.06	0,0056	✓
Story 2	0.0172	16421.850	2771,2081	3.06	0,0048	✓
Story 1	0.0134	18891.446	2894,7637	3.06	0,0037	✓
Ground floor	0.0060	21361.041	2943,1033	3.06	0,0016	✓

Table IV. 15: Verification of the Δ delta effect in Y-Y direction.

Story	Δ_{Ky} (m)	P_K (kN)	V_K (kN)	h_k (m)	θ_y	Verification ($\theta_y \leq 0.1$)
Story 8	0.0113	2581,00881	788,6386	3.06	0.0121	✓
Story 7	0.0125	4816.828	1374,1576	3.06	0.0066	✓
Story 6	0.0137	7052.706	1866,5745	3.06	0.0054	✓
Story 5	0.0146	9356.315	2281,8886	3.06	0.0048	✓
Story 4	0.0148	11659.956	2625,2246	3.06	0.0043	✓
Story 3	0.0143	14041.423	2898,8697	3.06	0.0038	✓
Story 2	0.0127	16421.850	3098,2289	3.06	0.0032	✓
Story 1	0.0096	18891.446	3222,9619	3.06	0.0024	✓
Ground floor	0.0042	21361.041	3270,3148	3.06	0.0010	✓

IV.10 Displacement analysis: (Art 4.5.2 RPA 2024)

Seismic forces generate horizontal deformation that could compromise the stability of the structure. The limitation of relative displacement at each level is carried out in accordance with the new Algerian seismic regulation (RPA2024), which stipulated the following condition: The horizontal displacement at each level “k” of the structure is calculated according to equation (Eq 7.1):

$$\delta_k = \frac{R}{Q_F} \cdot \delta_{ek}$$

The relative displacement at level “k” with respect to level “k-1” is equal to:

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

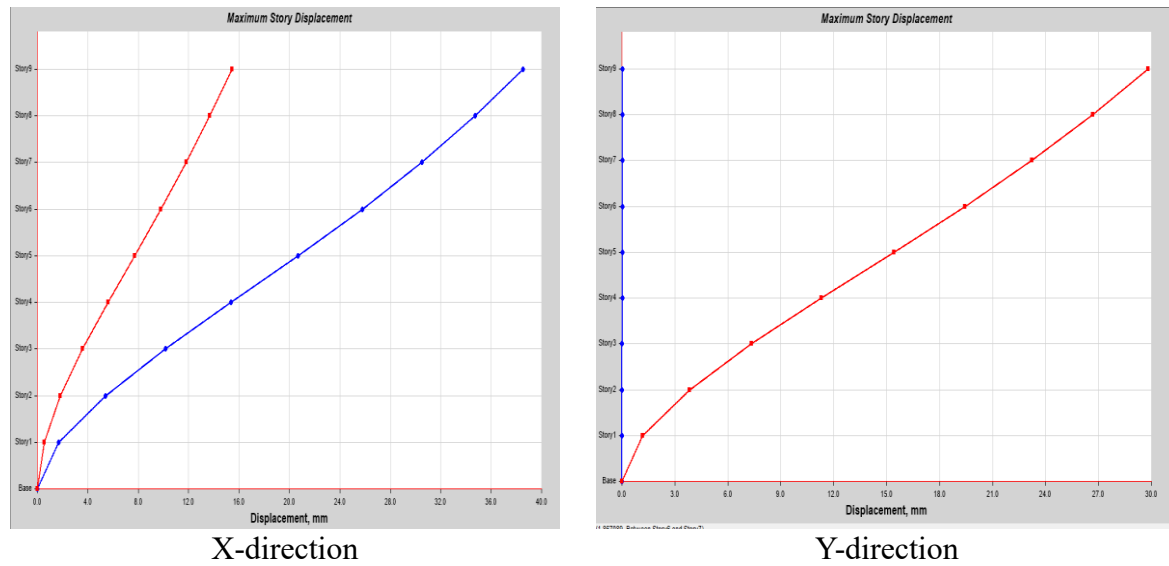


Figure IV. 7: Displacement diagram.

Table IV. 16: Inter story displacement drift

Story	δ_{ek}, x (mm)	δ_{ek}, y (mm)	R/Q_F	δ_x (mm)	δ_y (mm)	Δx (mm)	Δy (mm)
Story 8	38,549	29,876	3,6	138,7764	107,5536	13,7268	11,2716
Story 7	34,736	26,745	3,6	125,0496	96,282	15,21	12,4632
Story 6	30,511	23,283	3,6	109,8396	83,8188	17,0172	13,7232
Story 5	25,784	19,471	3,6	92,8224	70,0956	18,324	14,5584
Story 4	20,694	15,427	3,6	74,4984	55,5372	19,1304	14,8464
Story 3	15,38	11,303	3,6	55,368	40,6908	18,7704	14,2596
Story 2	10,166	7,342	3,6	36,5976	26,4312	17,208	12,6648
Story 1	5,386	3,824	3,6	19,3896	13,7664	13,356	9,5616
Ground floor	1,676	1,168	3,6	6,0336	4,2048	6,0336	4,2048

IV.11 Verification of Inter-story drifts

IV.11.1 Verification against structural collapse:(Art 5.10.1 RPA 2024)

The relative lateral displacement of one story with respect to the story below, as calculated according to article 4.5.2, must not exceed the limits $\bar{\Delta}_k$ given in table 5.2 and Eq. (5.11):

$$\Delta_k \leq \bar{\Delta}_k$$

For reinforces concrete buildings: $\bar{\Delta}_k = 0.015h_k = 0.015 \times 3060 = 45.9$ mm.

Table IV. 17: Verification of inter story drifts.

Story	Δx (mm)	Δy (mm)	$\bar{\Delta}_k$ (mm)	Verification on X-X	Verification on Y-Y
Story 9	13,7268	11,2716	49.5	✓	✓
Story 8	15,21	12,4632	49.5	✓	✓
Story 7	17,0172	13,7232	49.5	✓	✓
Story 6	18,324	14,5584	49.5	✓	✓
Story 5	19,1304	14,8464	49.5	✓	✓
Story 4	18,7704	14,2596	49.5	✓	✓
Story 3	17,208	12,6648	49.5	✓	✓
Story 2	13,356	9,5616	49.5	✓	✓
Story 1	6,0336	4,2048	49.5	✓	✓

IV.11.2 Justification of damage limitation

For building having non-structural elements composed of brittle materials fixed to the structure:

$$v_A \cdot \Delta_k \leq h_k 0.005$$

The reduction coefficient v_A is defined in article 5.12 it is taken equal to 0,5.

Limit value: $0.005 h_k = 0.005 \times 3060 = 15.3 \text{ mm}$

Table IV. 18: Justification of damage limitation.

Story	$\Delta_{k,x}$ (mm)	$\Delta_{k,y}$ (mm)	v_A	$v_A \cdot \Delta_{k,x}$	$v_A \cdot \Delta_{k,y}$	$0.005 h_k$	Verification X-X	Verification Y-Y
8	13,7268	11,2716	0.5	6,8634	5,6358	15.3	✓	✓
7	15,21	12,4632	0.5	7,605	6,2316	15.3	✓	✓
6	17,0172	13,7232	0.5	8,5086	6,8616	15.3	✓	✓
5	18,324	14,5584	0.5	9,162	7,2792	15.3	✓	✓
4	19,1304	14,8464	0.5	9,5652	7,4232	15.3	✓	✓
3	18,7704	14,2596	0.5	9,3852	7,1298	15.3	✓	✓
2	17,208	12,6648	0.5	8,604	6,3324	15.3	✓	✓
1	13,356	9,5616	0.5	6,678	4,7808	15.3	✓	✓
Base	6,0336	4,2048	0.5	3,0168	2,1024	15.3	✓	✓

IV.12 Evaluation of the structural center of gravity

Table IV. 19: Evaluation of the structural center of gravity.

Story	Mass (kg)	Center (m)	Center (m)	Mi*Xi	Mi*Yi
Story9	230289.79	11.767	7.234	2709889.05	1666008.46
Story8	219805.69	11.769	7.579	2587025.05	1666105.15
Story7	223546.59	11.77	7.575	2631143.36	1693365.42
Story6	226650.33	11.770	7.583	2667697.05	1718848.11
Story5	230456.45	11.770	7.592	2712564.6	1749671.46
Story4	234082.58	11.771	7.601	2755409.46	1779308.51
Story3	238286.43	11.771	7.613	2805060.2	1814074.59
Story2	242562.14	11.773	7.619	2855756.84	1848177.97
Story1	247379.15	11.773	7.628	2912518.42	1887206.06
Σ	2093059.15	105.937	68.027	24637064	15822765.7

IV.13 Verification of overall stability against overturning: (Art 5.5 RPA 2024)

The building stability against overturning shall be assessed at the foundation level. The overturning moment is obtained by combining the lateral loads acting on each story with their respective elevations measured from the base of the foundations.

A safety coefficient not less than 1.3 shall be ensured against overturning.

$$M_{\text{overturning}} = \sum_i^n F_i \times d_i$$

The stabilizing moment shall be evaluated from the vertical loads considered in the calculation of the lateral seismic action. The weight of the foundation and the overlying soil shall be included in these vertical loads.

The stabilizing moment is determined at the bottom level of the foundation with respect to the extreme edge: $M_{\text{stabilizing}} = W \times b$

Table IV. 20: Overturning moment and stabilizing moment calculation in the X-X direction.

Story	F_x (kN)	Height (m)	W (kN)	X_g (m)	Mr (kN.ml)	Ms (kN.ml)
Story 9	690,9165	27,54	21361.04	11,770	19027,84041	161480,9271
Story 8	515,6377	24,48			12622,8109	
Story 7	435,7306	21,42			9333,349452	
Story 6	371,358	18,36			6818,13288	
Story 5	312,1065	15,3			4775,22945	
Story 4	254,4492	12,24			3114,458208	
Story 3	191,0096	9,18			1753,468128	
Story 2	123,5556	6,12			756,160272	
Story 1	48,3396	3,06			147,919176	
				Sum	58349,36887	161480,9271

Verification: $\frac{M_{s,x}}{M_{r,x}} = 2.77 \geq 1.3 \rightarrow \text{Satisfied}$

Table IV. 21: Overturning moment and stabilizing moment calculation in the Y-Y direction.

Story	F_y (kN)	Height (m)	W (kN)	Y_g (m)	Mr (kN.ml)	Ms (kN.ml)
Story 9	788,638	27,54	21361.04	7,559	21719,10704	251436,5438
Story 8	585,519	24,48			14333,50512	
Story 7	492,416	21,42			10547,57	
Story 6	415,314	18,36			7625,166876	
Story 5	343,336	15,3			5253,0408	
Story 4	273,645	12,24			3349,416024	
Story 3	199,359	9,18			1830,117456	
Story 2	124,733	6,12			763,36596	
Story 1	47,352	3,06			144,899874	
				Sum	65566,18915	251436.5438

✓ **Verification:** $\frac{M_{s,y}}{M_{r,y}} = 3.83 \geq 1.3 \rightarrow \text{Satisfied}$

IV.14 Diaphragm verification: (Art 6.1.2 RPA 2024)

In accordance with article 6.1.2 of RPA 2024, diaphragms and horizontal bracing systems shall be designed to resist the seismic forces determined by equation (6.1):

$$F_{pk} = \frac{F_i + \sum_{i=k}^n F_i}{\sum_{i=k}^n W_i} \cdot W_{pk}$$

The seismic force acting on the diaphragm shall be bounded as expressed by equation (6.2):

$$0.35 \leq \frac{F_{pk}}{(A.I.S) \cdot W_{pk}} \leq 0.7$$

Table IV. 22: Diaphragm verification in the X-X direction.

Story	$\sum W_i$ (kN)	F_{pkx} (kN)	AIS	$\frac{F_{pkx}}{(A.I.S) \cdot W_{pk}}$	Observation
Story 9	2581.01	83.481	0.325	0.099	capped
Story 8	4816.83	126.287		0.173	capped
Story 7	7052.71	171.899		0.236	capped
Story 6	9356.32	217.154		0.290	capped
Story 5	11659.96	250.815		0.335	capped
Story 4	14041.43	287.657		0.371	verified
Story 3	16421.86	308.817		0.399	verified
Story 2	18891.46	334.669		0.416	verified
Story 1	21361.04	340.258		0.423	verified

Table IV. 23: Diaphragm verification in the Y-Y direction.

Story	$\sum W_i$ (kN)	F_{pky} (kN)	AIS	$\frac{F_{pky}}{(A.I.S) \cdot W_{pk}}$	Observation
Story 9	2581.01	95.289	0.325	0.113	capped
Story 8	4816.83	143.830		0.197	capped
Story 7	7052.71	195.375		0.268	capped
Story 6	9356.32	246.082		0.328	capped
Story 5	11659.96	283.112		0.378	verified
Story 4	14041.43	323.184		0.417	verified
Story 3	16421.86	345.259		0.446	verified
Story 2	18891.46	372.613		0.464	verified
Story 1	21361.04	378.088		0.471	verified

Table IV. 24: Rigid diaphragm verification.

Story	Ux rigid (cm)	Uy rigid (cm)	Ux extreme (cm)	Uy extreme (cm)	Ratio X	Verification X-X	Ratio Y	Verification Y-Y
Story 9	0.399	0.284	0.398	0.284	0.29	Semi-rigid	0.08	Semi-rigid
Story 8	1.210	0.897	1.185	0.897	2.09	Semi-rigid	0.08	Semi-rigid
Story 7	2.167	1.665	2.086	1.666	3.76	Semi-rigid	0.08	Semi-rigid
Story 6	3.131	2.485	2.972	2.487	5.09	Semi-rigid	0.08	Semi-rigid
Story 5	4.046	3.297	3.796	3.299	6.18	Semi-rigid	0.08	Rigid
Story 4	4.866	4.056	4.520	4.059	7.12	Rigid	0.07	Rigid
Story 3	5.581	4.743	5.135	4.746	7.99	Rigid	0.07	Rigid
Story 2	6.182	5.346	5.634	5.350	8.86	Rigid	0.06	Rigid
Story 1	6.667	5.882	6.043	5.885	9.35	Rigid	0.06	Rigid

IV.15 Conclusion

This chapter has successfully executed a detailed and multi-faceted seismic analysis of the reinforced concrete structure, verifying its behavior and stability under the rigorous requirement of the Algerian seismic code (RPA 2024). The analytical journey moved from fundamental geometric definition to complex dynamic assessment, yielding several critical insights into the building's structural integrity. The modal response spectrum analysis verified that the lateral load-resisting system where shear walls absorb over 65% of the total base shear justifies a behavior factor of $R=4.5$.

Crucially, all deformation and stability checks were satisfied. Inter story drifts remained below the 45.9 mm limit, and P-Delta effects were negligible, ensuring no second order instability. Overturning verification yielded high safety margins (2.77 in X-X and 3.83 in Y-Y), and the floor slabs exhibit confirmed rigid diaphragm behavior. In summary, the analysis demonstrates that the proposed structural configuration is robust, code compliant, and capable of safely resisting the severe seismic demands of its zone.

Chapter V:

Analysis and design of structural elements

V.1 Introduction

This chapter describes the analysis and design of the structural members after calculation of Internal forces resulting from the analysis. The idea here is to ensure the necessary strength, stability and ductility of the structural member when subjected to the most severe conditions, the process of design of the structural element is based on the provisions of the design code such as CBA93, BAEL91/99, and the Algerian Seismic Regulations (RPA 2024).

This set of codes outlines the basic principles of reinforced concrete design based on material behavior and safety factors. The design of the structural elements is made on the basis of the type and function of the structural element:

- Columns are designed to resist the combined effect of axial force and bending moment and are thus analyzed as axially loaded beams. (flexure)
- Beams resist the action of bending moment and shear force and are analyzed as simple beams in flexure.
- Shear wall members, resist the horizontal loads.

The evaluation of internal forces is carried out using ETABS 21, which reliable simulation of the structural response under various load combinations. Based on these results, reinforcement is designed by considering the governing load cases, including both ultimate state and accidental conditions, in accordance with the relevant design code.

V.2 Column

Columns are subjected to axial forces and bending moments about two principles axes, and thus exhibit biaxial bending. Longitudinal reinforcement is designed according to ultimate limit state considering the most critical loading combinations.

V.2.1 Verification of the reduced axial force

Checking of the reduced axial force is important to minimize the probability of brittle failure of concrete structures. This check, which was initially performed under static loading conditions, should also be performed for dynamic loading conditions as follows:

According to the RPA:
$$v = \frac{N_d}{B_c \times f_{c28}} \leq 0.35$$

N_d : Design of the compressive axial force considering seismic loads.

B_c : Cross cross-sectional area of the structural element.

f_{c28} : Compressive strength of the concrete after 28 days.

Table V. 1: Verification of the reduced axial force.

Story	Section (cm^2)	N_d (kN)	ν	$\nu \leq 0.35$
8	(30× 30)	150.546	0.066	CV
7	(35× 35)	271.831	0.088	CV
6	(35× 35)	395.486	0.129	CV
5	(40× 40)	525.526	0.131	CV
4	(40× 40)	659.346	0.164	CV
3	(45× 45)	802.593	0.158	CV
2	(45× 45)	975.206	0.192	CV
1	(50× 50)	1267.922	0.202	CV
Ground floor	(50× 50)	1840.819	0.294	CV

V.2.2 Verification of shear stresses

In the seismic loading combination, the designed shear stress in concrete τ_{bu} shall remain lower than or equal to a limiting value defined as follows: $\tau_{bu} = \rho_d \times f_{c28}$

$$\tau_u = \frac{V_u}{b_0 d}$$

ρ_d : Coefficient of the geometric slenderness in the considered direction.

- $\rho_d=0.075$ for a slenderness ratio greater than or equal to 5.
- $\rho_d=0.04$ when the slenderness ratio less than 5.

Table V. 2: Verification of shear stresses.

Section	V_u (kN)	λ_g	ρ_d	τ_u (MPa)	τ_{bu} (MPa)	$\tau_u \leq \tau_{bu}$
(30x30)	48.135	6.206	0.075	0.594	1.875	CV
(35x35)	76.022	5.32	0.075	0.689	1.875	CV
(40x40)	114.976	4.655	0.04	0.798	1	CV
(45x45)	136.227	4.137	0.04	0.747	1	CV
(50x50)	110.444	3.724	0.04	0.490	1	CV

V.2.3 Result of internal forces and reinforcement design:

The internal forces used for the designed the reinforcement based on different loads summarized below:

$$N_{max} \Rightarrow |M_y|_{\text{corresponding}}; |M_z|_{\text{corresponding}}$$

$$N_{min} \Rightarrow |M_y|_{\text{corresponding}}; |M_z|_{\text{corresponding}}$$

$$|M_{y \max}| \Rightarrow N_{\text{corresponding}}$$

$$|M_{z \max}| \Rightarrow N_{\text{corresponding}}$$

For a square column that a square in section, the design of the reinforcement is carried out considering the the most critical bending moment selected from the two principal directions.

$$M_{adopted} \Rightarrow \max (M_y; M_z)$$

❖ Load combination

Depending on the nature of the applied actions, the following combination are considered:

- Combination according to the RPA 2024: $\begin{cases} G + \psi Q + E_1 \\ G + \psi Q + E_2 \end{cases} \begin{cases} E_1 = \pm E_x \pm 0.3E_y \\ E_2 = \pm 0.3E_x \pm E_y \end{cases}$
- Combination based on the BAEL 91/99: $\begin{cases} ULS: 1.35G + 1.5Q \\ SLS: G + Q \end{cases}$

- ✓ **Note:** In ETABS, compressive forces are designed using a negative sign, whereas tension load is indicated with a positive sign. However, according SOCOTEC convention

Table V. 3: Result of internal forces and column reinforcement design.

Column (50x50)					
Combination	N (kN)	M ₂ (kN.ml)	M ₃ (kN.ml)	Structural action	A (cm ²)
$N_{max}(G + 0.3Q + E_2)$	1840.819	59.109	31.391	FCS	0
$N_{min}(G + 0.3Q + E_1)$	-929.974	31.612	18.042	FTS	13,38
$M_{2max}(G + 0.3Q + E_1)$	704.457	178.116	40.630	PCS	1,67
$M_{3max}(G + 0.3Q + E_1)$	704.773	40.705	178.157	PCS	1,67
Column (45x45)					
Combination	N (kN)	M ₂ (kN.ml)	M ₃ (kN.ml)	Structural action	A (cm ²)
$N_{max}(G + 0.3Q + E_2)$	975.206	125.967	33.074	FCS	0
$N_{min}(G + 0.3Q + E_2)$	-349.073	2.809	3.959	FTS	4,27
$M_{2max}(G + 0.3Q + E_1)$	490.591	238,52	25.739	PCS	10,28
$M_{3max}(G + 0.3Q + E_1)$	490.753	25.794	238,52	PCS	10,28
Column (40x40)					
Combination	N (kN)	M ₂ (kN.ml)	M ₃ (kN.ml)	Structural action	A (cm ²)
$N_{max}(ULS)$	659.346	2.276	2.359	FCS	0
$N_{min}(G + 0.3Q + E_2)$	-98.763	2.684	3.273	FTS	1,54
$M_{2max}(G + 0.3Q + E_1)$	271.829	171.773	12.913	PCS	9,76
$M_{3max}(G + 0.3Q + E_1)$	271.871	12.957	171.797	PCS	9,76
Column (35x35)					
Combination	N (kN)	M ₂ (kN.ml)	M ₃ (kN.ml)	Structural action	A (cm ²)

$N_{max}(ULS)$	395.486	-2.209	2.984	FCS	0
$N_{min}(G + 0.3Q + E_2)$	-75.385	2.806	26.945	FTS	3,15
$M_{2max}(G + 0.3Q + E_1)$	145.791	114.669	1.243	PCS	8.2
$M_{3max}(G + 0.3Q + E_1)$	145.778	1.280	114.661	PCS	8,2
Column (30x30)					
Combination	N (kN)	M₂ (kN.m)	M₃ (kN.m)	Structural action	A (cm²)
$N_{max}(ULS)$	150.546	2.460	2.450	FCS	0
$N_{min}(G + 0.3Q + E_2)$	-54.443	1.023	17.573	FTS	2.4
$M_{2max}(G + 0.3Q + E_1)$	91.987	69.318	3.217	PCS	5.97
$M_{3max}(G + 0.3Q + E_1)$	91.987	3.187	69.301	PCS	5.97

V.2.4 Longitudinal reinforcement

❖ According to RPA 2024

The structure is classified within seismic zone V, implying that it's a region of high seismicity:

- The critical zone length l_{cr} : $l_{cr} = \max\left(1.5h_c; \frac{l_{cl}}{6}; 60cm\right)$.
- Minimum longitudinal reinforcement area: $A_{min} = 1\%(b \times h)$.
- Maximum reinforcement area for non-critical zone: $A_{max} = 4\%(b \times h)$.
- The maximum reinforcement area for lap splice zone: $A_{max} = 8\%(b \times h)$.
- Minimum diameter for reinforcing bar $\Phi_{min}=12$ mm.
- Minimum length of lap splice: $l_{min} = 60\Phi_l$.
- Maximum spacing between reinforcing bars: 15cm.

❖ According to BAEL91

- Requirement of non-brittleness behavior: $A_{min} = 0.23 \times b_0 \times d \times \frac{f_{t28}}{f_e}$

Table V. 4: Longitudinal reinforcement of columns.

Section	A_{min}^{BAEL} (cm ²)	A_{min}^{RPA} (cm ²)	Choice	$A_{s\ adop}$ (cm ²)
(30x30)	0.978	9	8HA16	16.08
(35x35)	1.331	12.25	4HA20+4HA16	20.60
(40x40)	1.738	16	4HA20+8HA16	28.65
(45x45)	2.200	20.25	4HA20+8HA16	28.65
(50x50)	2.716	25	8HA20+8HA16	41.21

Table V. 5: Recommendations of the Algerian Seismic Regulations.

Section	l_{min} (cm)	l_{cr} (cm)	$A_{max RPA}$		$A_{adop} < A_{max RPA}$
			4%(b × h)	8% (b × h)	
(30x30)	96 ≈ 100	100	36	72	Verified
(35x35)	96 ≈ 100	100	49	98	Verified
(40x40)	96 ≈ 100	100	64	128	Verified
(45x45)	96 ≈ 100	100	81	162	Verified
(50x50)	96 ≈ 100	100	100	200	Verified

V.2.5 Transverse reinforcement

The transverse reinforcement in column is determined using the following expression:

$$\frac{A_t}{t} = \frac{\rho_a V_u}{h_l f_e}$$

- A_t : Cross-section area of transverse reinforcement legs.
- V_u : Design shear force.
- h_l : Overall depth of the gross section in the considered direction.
- f_e : Yield strength of the transverse reinforcement steel.
- ρ_a : Modification factor accounting for brittle shear failure behavior.

This coefficient ρ_a depends on the geometric slenderness ratio in the considered direction:

- $\rho_a = 2.5$ when $\lambda_g \geq 5$
- $\rho_a = 3.75$ when $\lambda_g \leq 3$
- Joint region: $t \leq \min(\frac{b_0}{3}; 10cm; 6\Phi_l)$
- Non-critical region: $t \leq \min(\frac{b_1}{2}; \frac{h_l}{2}; 10\Phi_l)$

The required minimum of transverse reinforcement, expressed as a percentage is determined according to the following expression: $\frac{A_t}{t \times b_1}$

$$\begin{cases} \lambda_g \geq 5 & 0.3\% \\ \lambda_g \leq 3 & 0.8\% \\ 3 < \lambda_g < 5 & \text{Interplation between the limit values} \end{cases}$$

- λ_g : The geometric slenderness of the column. $\lambda_g = \frac{l_f}{a}$ or $\frac{l_f}{b}$
- **a** and **b**: The column's cross-section dimensions in the direction of deformation considered.
- l_f : The column's effective buckling length.

$$l_f = 0.71 \Rightarrow l_f = 0.7 \times (3.06 - 0.4) \Rightarrow l_f = 1.862 \text{ m}$$

❖ Joint region

Table V. 6: Transverse reinforcement in joint region.

Section	Vu (kN)	t (cm)	λ_g	ρ_a	A_t (cm^2)	Choice	$A_{s \text{ adop}}$ (cm^2)	A_{cal} %	A %	$A_{\text{cal}} < A$
(30x30)	48.135	10	6.206	2.5	1.002	4HA8	2.01	0.67	0.3	CV
(35x35)	76.022	10	5.32	2.5	1.357	4HA8	2.01	0.573	0.3	CV
(40x40)	114.976	10	4.655	3.75	2.694	6HA8	3.02	0.755	0.	CV
(45x45)	136.227	10	4.137	3.75	2.838	6HA8	3.02	1.046	0.668	CV
(50x50)	110.417	10	3.724	3.75	2.070	6HA8	4.02	0.942	0.893	CV

❖ Non-critical region

Table V. 7: Transverse reinforcement in non-critical region.

Section	Vu (kN)	t(cm)	λ_g	ρ_a	A_t (cm^2)	Choice	$A_{s \text{ adop}}$ (cm^2)	A_{cal} %	A %	$A_{\text{cal}} < A$
(30x30)	48.135	15	6.206	2.5	1.504	4HA8	2.01	0.446	0.3	CV
(35x35)	76.022	15	5.32	2.5	2.036	4HA8	2.01	0.382	0.3	CV
(40x40)	114.976	15	4.655	3.75	4.042	6HA8	3.02	0.503	0.386	CV
(45x45)	136.227	15	4.137	3.75	4.257	6HA10	4.71	0.697	0.51	CV
(50x50)	110.417	15	3.724	3.75	3.105	6HA10	4.71	0.640	0.62	CV

V.2.6 Verification of stresses at the serviceability limit state (SLS)

Cracking is considered detrimental in columns column regions. The result of the stress checks at the serviceability limit state (SLS) are presented in the table below:

Table V. 8: Verification of stresses at (SLS) of columns.

Section	N_{ser} (kN)	M_{ser} (kN.m)	σ_{bc} (MPa)	$\bar{\sigma}_{bc}$ (MPa)	$\sigma_{bc} \leq \bar{\sigma}_{bc}$	σ_{st} (MPa)	$\bar{\sigma}_{st}$ (MPa)	$\sigma_{st} \leq \bar{\sigma}_{st}$
(30x30)	110.621	1.761	0,99	15	Verified	14,3	201.63	CV
(35x35)	288.74	2.215	1,72	15	Verified	25,4	201.63	CV
(40x40)	480.806	1.743	2,03	15	Verified	30,3	201.63	CV
(45x45)	693.48	1.746	2,46	15	Verified	36,8	201.63	CV
(50x50)	931.990	1.607	2,53	15	Verified	37,9	201.63	CV

V.2.7 Columns reinforcement detailing

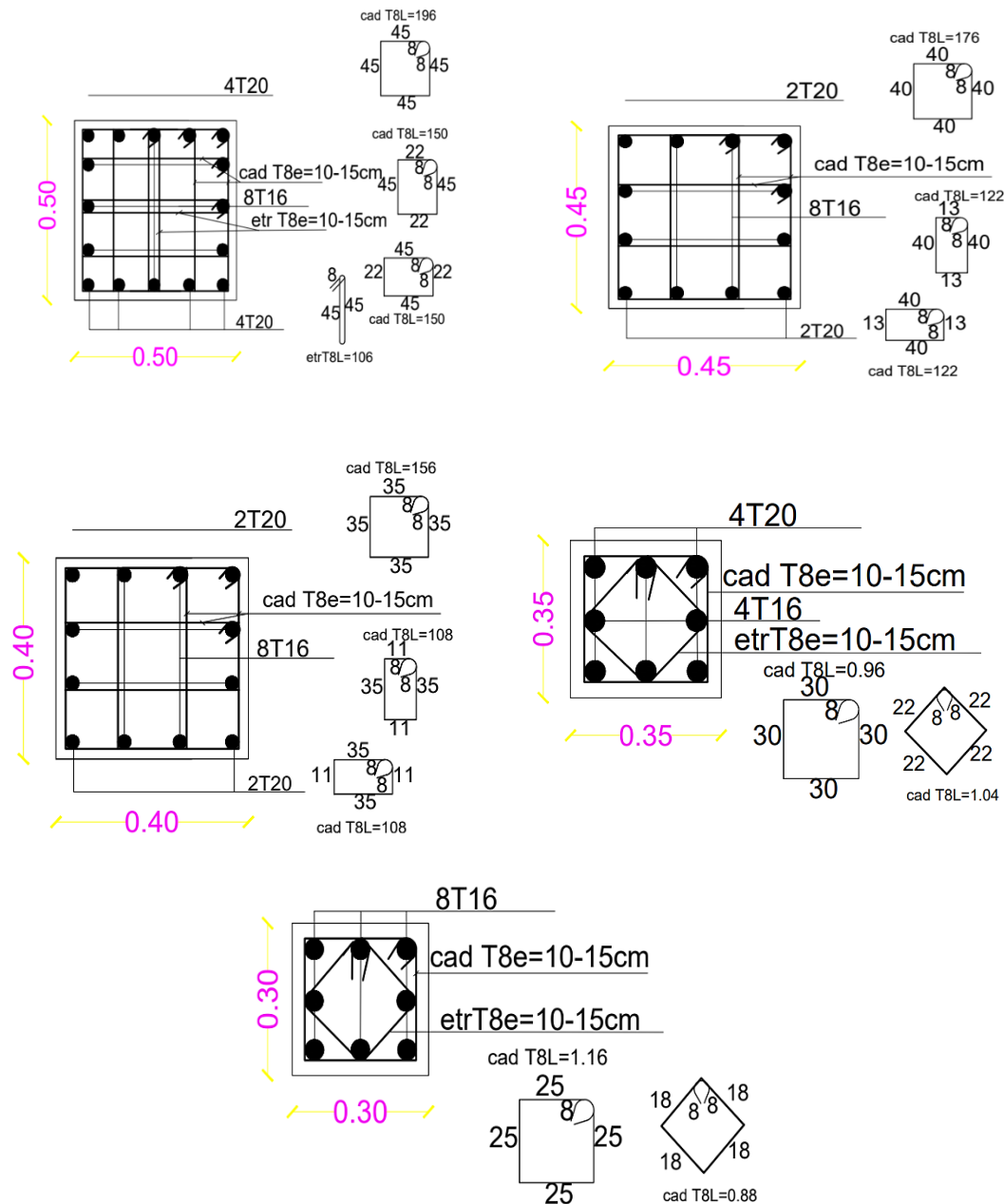


Figure V. 1: Columns reinforcement detailing.

V.2.8 Beams

Beams are mainly subjected to simple bending due to the combined action of bending moments and shear forces. The longitudinal reinforcement is designed based on bending moments, whereas the transverse reinforcement is determined from shear forces. After evaluation the load effects, the reinforcement design is carried out in accordance with RPA, based on the results obtained from the analysis software and considering the most unfavorable load combination.

❖ Load combination

- Combination according to the RPA 2024:
$$\begin{cases} G + \psi Q + E_1 \\ G + \psi Q + E_2 \end{cases} \quad \begin{cases} E_1 = \pm E_x \pm 0.3E_y \\ E_2 = \pm 0.3E_x \pm E_y \end{cases}$$
- Combination based on the BAEL 91/99:
$$\begin{cases} ULS: 1.35G + 1.5Q \\ SLS: G + Q \end{cases}$$

V.2.9 Longitudinal reinforcement

❖ According to RPA 2024

The structure is classified within seismic zone V, implying that it's a region of high seismicity:

- Maximum reinforcement area for non-critical zone: $A_{max} = 4\%(b \times h)$
- The maximum reinforcement area for lap splice zone: $A_{max} = 6\%(b \times h)$
- Minimum length of lap splice: $l_{min} = 60\Phi_l$.
- A least two 14 mm diameter high-bond longitudinal reinforcement bars (for zones IV, V, and VI) must be placed on the upper and lower faces of the beam, running continuously along the entire length of the beam.
- Longitudinal reinforcement (top and bottom) is anchored in edge and corner columns using 90° hooks.
- Beams under mainly seismic loads are symmetrically reinforced, with midspan steel at least half of the support reinforcement.

❖ According to BAEL91

- The non-brittle behavior requirement: $A_{min} = 0.23 \times b_0 \times d \times \frac{f_{t28}}{f_e}$

Table V. 9: Longitudinal reinforcement of primary beams connected to shear wall.

Section (30x40)							
Position	M (kN.m)	$A_s (cm^2)$	$A_{min}^{BAEL} (cm^2)$	$A_{min}^{RPA} (cm^2)$	Choice	$A_{s\ adop} (cm^2)$	
Span	24.325	1,93	1.304	6	3HA20+ HA16	11.43	11.43
Upper Support	-251.104	20.82	1.304	6	3HA20+ 6HA16	21.48	42.96
Lower support	245.942	20.28	1.304	6	3HA20+ 6HA16	21.48	

Table V. 10: Longitudinal reinforcement of secondary beams connected to shear wall.

Section (30x40)							
Position	M (kN.m)	A_s (cm ²)	$A_{min\ BAE L}$ (cm ²)	$A_{min\ RPA}$ (cm ²)	Choice	$A_{s\ adop}$ (cm ²)	
Span	36.955	2,97	1.304	6	3HA20	9.42	9.42
Upper support	-233.718	19.01	1.304	6	3HA20+5HA16	19.47	38.31
Lower support	220.463	17,68	1.304	6	3HA20+3HA20	18.84	

Table V. 11: Longitudinal reinforcement of primary beams non connected to shear wall.

Section (30x40)							
Position	M (kN.m)	A_s (cm ²)	$A_{min\ BAE L}$ (cm ²)	$A_{min\ RPA}$ (cm ²)	Choice	$A_{s\ adop}$ (cm ²)	
Span	28.920	2,31	1.304	6	3HA16	6.03	6.03
Upper Support	-154.291	11,63	1.304	6	3HA16+3HA16	12.06	24.12
Lower support	142.746	10,65	1.304	6	3HA16+3HA16	12.06	

Table V. 12: Longitudinal reinforcement of secondary beams non connected to shear wall.

Section (30x40)							
Position	M (kN.m)	A_s (cm ²)	$A_{min\ BAE L}$ (cm ²)	$A_{min\ RPA}$ (cm ²)	Choice	$A_{s\ adop}$ (cm ²)	
Span	25.131	1,97	1.304	6	3HA16	6.03	6.03
Upper support	-112.797	8,24	1.304	6	3HA16+2HA14	9.11	18.22
Lower support	98.488	7.1	1.304	6	3HA16+2HA14	9.11	

Table V. 13: Recommendations of the Algerian Seismic Regulations.

		l' (m)	Φ_l (mm)	60 Φ_l (m)	$A_{\max RPA}$ (cm ²)		$A_{s\ adop}$ < $A_{\max RPA}$
Primary beams connected to shear wall					4% (b× h)	6% (b× h)	CV
Span	0.6	HA16	CV	11.43 < 48	/		
Support				/	42.96 < 72		
Secondary beams connected to shear wall					4% (b× h)	6% (b× h)	CV
Span	0.6	HA16	CV	9.42 < 48	/		
Support				/	38.31 < 72		
Primary beams not connected to shear wall					4% (b× h)	6% (b× h)	CV
Span	0.6	HA16	CV	6.03 < 48	/		
Support				/	43.48 < 72		
Secondary beams not connected to shear wall					4% (b× h)	6% (b× h)	CV
Span	0.6	HA16	0.9≈1	6.03 < 48	/		
Support				/	43.48 < 72		

V.2.10 Transverse reinforcement

❖ According CBA93

$$\phi_t \leq \min\left(\frac{h}{35}; \frac{b_0}{10}; \Phi_l\right) \Rightarrow \phi_t \leq 1.14 \text{ cm}$$

Φ_l : Minimum diameter of longitudinal reinforcement bars $\Phi_l = 1.4 \text{ cm}$

We adopt: $\phi_t = 8 \text{ mm}$

Transverse reinforcement spacing:

$$S_t \leq \min(0.9 h; 40 \text{ cm}) \Rightarrow S_t \leq \min(0.9 \times 36; 40 \text{ cm}) \Rightarrow S_t \leq 32.4 \text{ cm} \Rightarrow S_t = 10 \text{ cm}$$

❖ According RPA2024

Maximum spacing of transverse reinforcement:

• In critical zone

$$s = \min\left(\frac{h}{4}; 24\phi_t; 17.5 \text{ cm}; 6\Phi_l\right) \Rightarrow s = 8.4 \text{ cm} \Rightarrow \text{we adopt } s = 10 \text{ cm}$$

Φ_l : Minimum diameter of longitudinal reinforcement bars.

• Non critical zone

$$s' \leq \frac{h}{2} \Rightarrow s' \leq 20 \text{ cm} \Rightarrow s' = 15 \text{ cm} \dots \text{Verified}$$

$$A_t = (0.003) \times s \times b_0 \Rightarrow A_t = 0.003 \times 15 \times 30 \Rightarrow A_t = 1.35 \text{ cm}^2$$

$$A_t = 2.02 \text{ cm}^2 \Rightarrow 4\text{HA}8$$

V.2.11 Verification of shear force

The shear stress is expressed by the following expression: $\tau_u = \frac{Vu}{b_0 d}$

Cracking is considered detrimental: $\bar{\tau}_u = \min(0.15f_{c28}/\gamma_b; 4 \text{ MPa})$

$$\gamma_b = \begin{cases} 1.5 & \text{Persistent design situation} \\ 1.2 & \text{Accidental design situation} \end{cases}$$

Table V. 14: Verification of shear force of beams.

V_u (MPa)	τ_u (MPa)	$\bar{\tau}_u$ (MPa)	$\tau_u \leq \bar{\tau}_u$
Primary beams connected to shear wall			
303.152	2.806	3.125	CV
Secondary beams connected to shear wall			
167.590	1.551	3.125	CV
Primary beams not connected to shear wall			
186.237	1.724	3.125	CV
Secondary beams not connected to shear wall			
86.881	0.804	3.125	CV

V.2.12 Verification of stresses at the serviceability limit state (SLS)

Cracking is considered detrimental in beam regions. The result of the stress checks at the serviceability limit state (SLS) are presented in the table below: 11.43

Table V. 15: Verification of beams stresses at (SLS).

Position	M_{ser} (kN.m)	Y (cm)	I (cm ⁴)	σ_{bc} (MPa)	$\bar{\sigma}_{bc}$ (MPa)	σ_{st} (MPa)	$\bar{\sigma}_{st}$ (MPa)	Check
Primary beams connected to shear wall								
Span	17.558	15.359	109278.132	2.467	15	49.746	201.63	CV
Support	-31.383	23.403	231672.298	3.170	15	25.596	201.63	CV
Secondary beams connected to shear wall								
Span	26.899	14.298	95778.824	4.013	15	91.423	201.63	CV
Support	-43.985	23.403	231672.298	4.443	15	35.874	201.63	CV
Primary beams not connected to shear wall								
Span	21.147	12.023	69378.941	3.664	15	109.624	201.63	CV
Support	-26.641	19.779	172574.317	3.053	15	37.561	201.63	CV
Secondary beams not connected to shear wall								
Span	18.326	12.023	69378.941	3.175	15	95.001	201.63	CV
Support	-23.966	18.073	146864.865	2.949	15	43.881	201.63	CV

V.2.13 Deflection verification

The deflection check is not required provided that the following three conditions are satisfied:

$$\left(\frac{h_t}{L}\right) \geq \left(\frac{1}{16}\right) \quad ; \quad \left(\frac{h_t}{L}\right) \geq \left(\frac{M_{ser \text{ span}}}{10M_{0 \text{ ser}}}\right) \quad ; \quad \left(\frac{A_s}{b \times d}\right) \leq \left(\frac{4.2}{f_e}\right)$$

Where:

L: Span length measured between the face of the support (L=4.74 m)

$M_{ser \text{ span}}$: is the maximum span moment.

$M_{ser \text{ span}}$: is the isostatic bending moment

A_s : is the required area of the tensile reinforcement.

Primary beams connected to shear wall:

$$\left(\frac{h_t}{L}\right) \geq \left(\frac{1}{16}\right) \Rightarrow 0.084 \geq 0.0625 \dots CV$$

$$\left(\frac{h_t}{L}\right) \geq \left(\frac{M_{ser span}}{10M_{0 ser}}\right) \Rightarrow 0.084 \geq 0.078 \dots CV$$

$$\left(\frac{A_s}{b \times d}\right) \leq \left(\frac{4.2}{f_e}\right) \Rightarrow 0.0018 < 0.0105 \dots CV$$

- **Secondary beams connected to shear wall**

$$\left(\frac{h_t}{L}\right) \geq \left(\frac{1}{16}\right) \Rightarrow 0.084 \geq 0.0625 \dots CV$$

$$\left(\frac{h_t}{L}\right) \geq \left(\frac{M_{ser span}}{10M_{0 ser}}\right) \Rightarrow 0.084 \geq 0.064 \dots CV$$

$$\left(\frac{A_s}{b \times d}\right) \leq \left(\frac{4.2}{f_e}\right) \Rightarrow 0.0018 < 0.0105 \dots CV$$

- **Primary beams no connected to shear wall**

$$\left(\frac{h_t}{L}\right) \geq \left(\frac{1}{16}\right) \Rightarrow 0.084 \geq 0.0625 \dots CV$$

$$\left(\frac{h_t}{L}\right) \geq \left(\frac{M_{ser span}}{10M_{0 ser}}\right) \Rightarrow 0.084 \geq 0.060 \dots CV$$

$$\left(\frac{A_s}{b \times d}\right) \leq \left(\frac{4.2}{f_e}\right) \Rightarrow 0.0018 < 0.0105 \dots CV$$

- **Secondary beams connected to shear wall**

$$\left(\frac{h_t}{L}\right) \geq \left(\frac{1}{16}\right) \Rightarrow 0.084 \geq 0.0625 \dots CV$$

$$\left(\frac{h_t}{L}\right) \geq \left(\frac{M_{ser span}}{10M_{0 ser}}\right) \Rightarrow 0.084 \geq 0.052 \dots CV$$

$$\left(\frac{A_s}{b \times d}\right) \leq \left(\frac{4.2}{f_e}\right) \Rightarrow 0.0018 < 0.0105 \dots CV$$

V.214 Beams reinforcement detailing

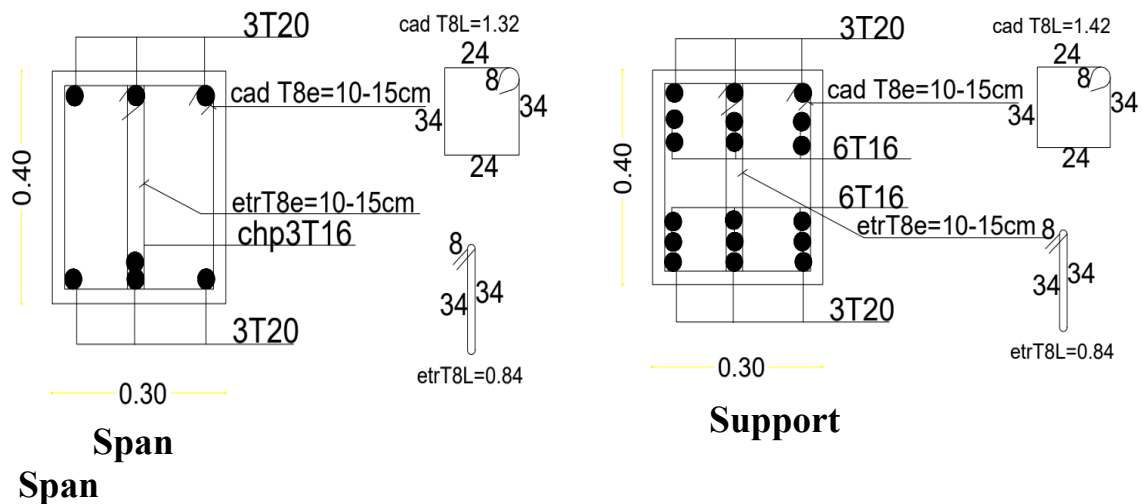


Figure V. 2: Reinforcement detailing of primary beams connected to shear walls.

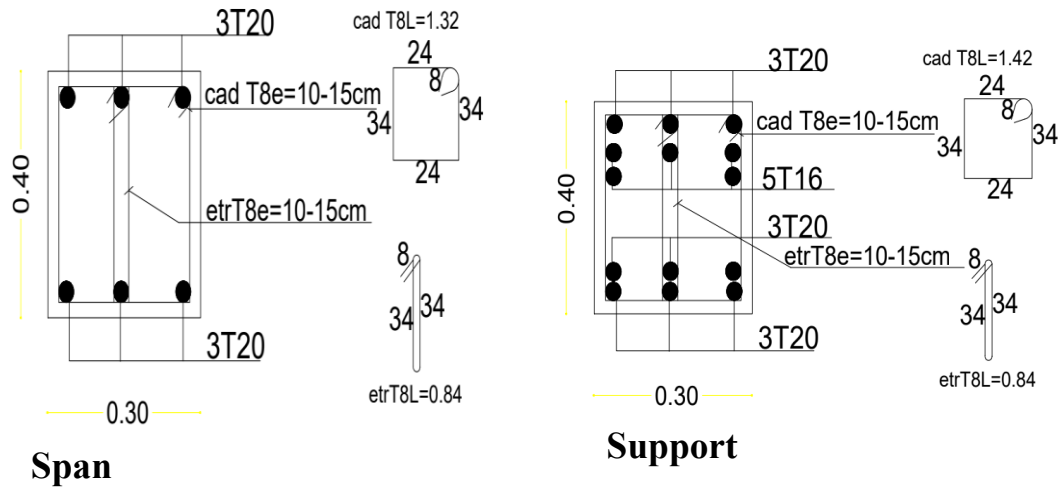


Figure V. 3: Reinforcement detailing of secondary beams connected to shear walls.

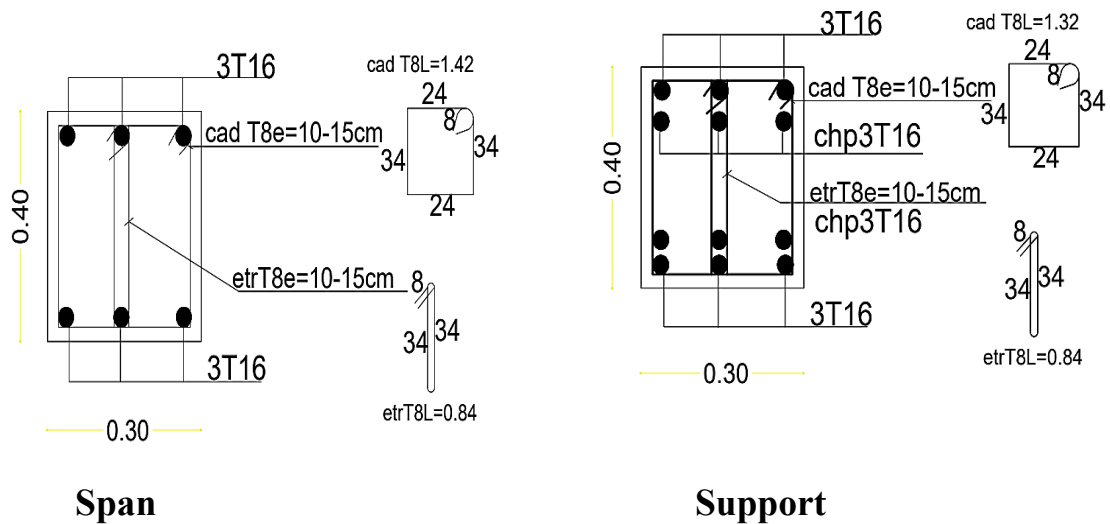


Figure V. 4: Reinforcement detailing of primary beams no connected to shear walls.

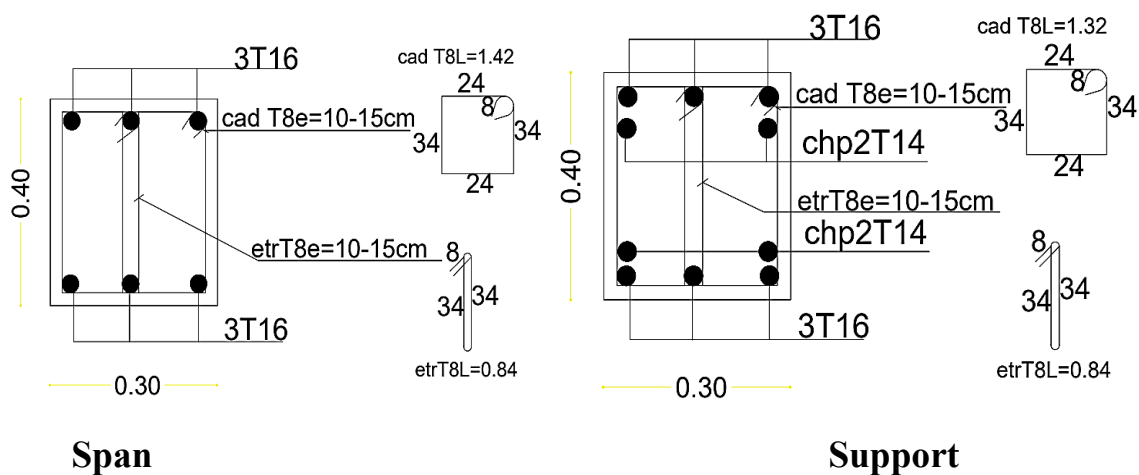


Figure V. 5: Reinforcement detailing of secondary beams no connected to shear walls.

V.3 Shear walls

In reinforced concrete buildings, the shear walls provide for vertical loading capacity as well as lateral stability, particularly during seismic conditions, due to their rigidity. They are exposed to the effects of combined axial force and bending and shearing actions due to gravity and seismic conditions. Their design should take into consideration the interaction of the above-mentioned forces so as not to cause brittle failure. It's crucial that appropriate reinforcement be provided, such as vertical reinforcement for axial and flexural actions, horizontal reinforcements for shear, and confinement in wall boundaries.

❖ Load combination

- Combination according to the RPA 2024:
$$\begin{cases} G + \psi Q + E_1 \\ G + \psi Q + E_2 \end{cases} \quad \begin{cases} E_1 = \pm E_x \pm 0.3E_y \\ E_2 = \pm 0.3E_x \pm E_y \end{cases}$$
- Combination based on the BAEL 91/99:
$$\begin{cases} ULS: 1.35G + 1.5Q \\ SLS: G + Q \end{cases}$$

Zone 1: Spanning from the ground floor to the third floor.

Zone 2: Spanning from the third floor to the sixth floor.

Zone 3: Spanning from the sixth floor to the ninth floor.

- ✓ **Note:** In ETABS, compressive forces are expressed with negative values, while tensile forces are represented by positive value. However, according to SOCOTEC conventions, these signs must be reversed to ensure coherent result.

V.3.1 Verification of the reduced axial force

In addition to the checks required by the CBA code, the design compressive axial force in shear walls must be limited in order to reduce the risk of brittle failure under combined bending effects induced by seismic actions. This requirement is expressed by the following condition:

- According to the RPA:
$$v = \frac{N_d}{B_c \times f_{c28}} \leq 0.4$$

N_d : The design axial compressive force under seismic load combinations.

B_c : The gross cross-sectional area of the structural element.

f_{c28} : The characteristic compressive strength of concrete at 28 days.

Table V. 16: Verification of the reduced axial force of shear walls.

Section (m)	N_d (kN)	f_{c28}	v	$v \leq 0.4$
(1 × 0.2)	986.309	25	0.197	CV
(1.5 × 0.2)	2201.027	25	0.293	CV
(1.65 × 0.2)	1598.675	25	0.193	CV

V.3.2 Verification of shear stresses

❖ According RPA2024

The shear stress in concrete is limited in accordance with the following expression

$$\tau_b \leq 0.2f_{c28} \quad \text{and} \quad \tau_b = \frac{\bar{V}}{b_w.d}$$

$$\bar{V} = 1.4V_u$$

b_w : Thickness of the wall

d : Effective depth, with $d=0.9h$.

h : Total depth of the gross section.

Table V. 17: Verification of shear stresses of shear walls (RPA2024).

Section	$V_u(kN)$	$\bar{V}(KN)$	$\tau_b(MPa)$	$0.2f_{c28} (MPa)$	check
(1× 0.2)	190.542	266.758	1.482	5	CV
(1.5× 0.2)	391.231	547.723	2.028	5	CV
(1.65 × 0.2)	398.799	558.318	1.879	5	CV

- According BAEL91:

The shear stress is expressed by the following expression: $\tau_u = \frac{V_u}{b_0 d}$

Cracking is considered detrimental: $\bar{\tau}_u = \min (0.2f_{c28}/\gamma_b ; 4 \text{ MPa})$

$$\gamma_b = \begin{cases} 1.5 & \text{Under serviceability conditions} \\ 1.2 & \text{Under accidental conditions} \end{cases}$$

Table V. 18: Verification of shear stresses of shear walls (BAEL 91).

Section	$V_u(kN)$	$\tau_b(MPa)$	$\bar{\tau}_u (MPa)$	Check
(1× 0.2)	190.542	1.058	4.16	CV
(1.5× 0.2)	391.231	1.449	4.16	CV
(1.65 × 0.2)	398.799	1.342	4.16	CV

V.3.3 Recommendations of the RPA for shear walls

a) Vertical reinforcement: Vertical reinforcing bars are intended to resist bending effects.

They are arranged in two layers distributed along the two faces of the shear walls.

Their detailing must comply with following provisions:

- A minimum reinforcement ratio of 0.2% shall be ensured within the entire tensile zone.

- In the boundary elements, vertical bars must be confined by transverse ties, with spacing not exceeding the wall thickness.
- In the edge regions, the spacing between vertical bars shall be reduced by half over a distance equal 1/10 of the wall height, with the maximum spacing limited to 150mm.
- At the top story, vertical bars must be provided with hooks at their upper ends, while at other levels, connections are achieved by lap splices without hooks.

b) Horizontal reinforcement: Horizontal reinforcement is placed parallel to the wall faces on both sides between vertical bars and the formwork. It includes 135° hooks with a 10 Φ anchorage and is designed mainly to resist shear force.

According to RPA 2024: $\frac{A_h}{s} \geq \frac{\bar{V}}{z.f_e}$

c) Transverse reinforcement: These bars must be perpendicular to the wall surface where they connect the two sections together. Minimum of four pins per square meter is required.

d) Stitching reinforcement: Along construction joints, shear forces must be transferred through specific stitching reinforcement. The required steel area is determined according to the following expression: $A_{vj} = 1.1 \frac{V}{f_e}$ $\bar{V} = 1.4 V_u$

V_u : Shear force evaluated at the specified section.

This contribution is to be included in the required tensile reinforcement area to ensure resistance to the tensile forces generated by the overturning moment.

V.3.4 Reinforcement design

The reinforcement design is carried out using stress-based approach and is then verified according to RPA 2024, considering the following load cases:

$$N_{max} \Rightarrow M_{\text{corresponding}}$$

$$N_{min} \Rightarrow M_{\text{corresponding}}$$

$$M_{max} \Rightarrow N_{\text{corresponding}}$$

❖ Presentation of the method:

The adopted procedure aims to evaluate the stress distribution diagram under loading conditions that promote tensile forces, using the following expressions:

$$\sigma_{min,max} = \frac{N}{\Omega} \pm \frac{M.V}{I}$$

N: Applied normal force.

M: Applied bending moment.

Ω : Cross-sectional area of the shear wall.

V : Lever arm.

I : Moment of Inertia of the wall section.

Three different situations can be identified:

❖ Case 01

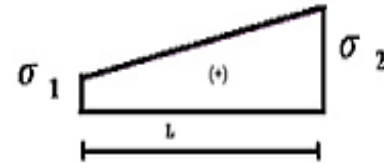
When σ_{max} and $\sigma_{min} > 0$ are positive, the wall section is subjected to compressive stresses, with no tensile zone present.

$$F = \frac{\sigma_{max} + \sigma_{min}}{2} \times L_c \times e$$

L_c : Refers to the length of the compressed zone.

$$L_c = L$$

$$A_v = \frac{F - B f_{bc}}{f_e}$$



❖ Case 02

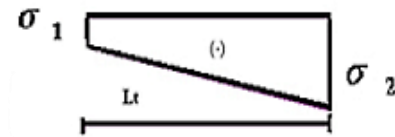
When σ_{max} and $\sigma_{min} < 0$ are negative, the wall section is under tensile stresses, with no compression zone present.

$$F = \frac{\sigma_{max} + \sigma_{min}}{2} \times L_t \times e$$

L_t : Refers to the length of the tensile zone.

$$L_t = L$$

$$A_v = \frac{F}{f_e}$$



❖ Case 03

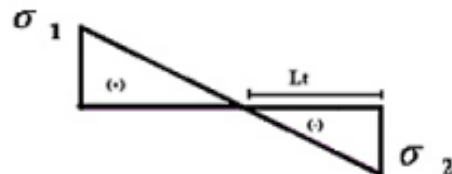
When σ_{max} and σ_{min} have opposite signs, the wall section presents a combined behavior, with zones subjected, is partially in compression and partially in tension. In this case, the stress analysis is mainly focused on the tensile region.

$$F = \frac{\sigma_{min}}{2} \times L_t \times e$$

L_t : Refers to the length of the tensile zone.

$$L_t = \frac{\sigma_{min}}{\sigma_{max} + \sigma_{min}} L$$

$$A_v = \frac{F}{f_e}$$



- If $A_v < A_{min}$ the reinforcement is designed using A_{min}
- If $A_v > A_{min}$ the reinforcement is designed using A_v

Minimum reinforcement according BAEL:

Partially compressed section: $A_{v\ min} \geq \max(0.23B \frac{f_{t28}}{f_e}; 0.005B)$

Fully tensioned section: $A_{v\ min} \geq \max(\frac{N_u}{2\sigma_s} B \frac{f_{t28}}{f_e}; 0.005B)$

Table V. 19: Evaluation of stresses in shear wall W1 (1x0.2) m.

Zone	N (kN)	M (kN.m)	Ω (m ²)	I (m ⁴)	V (m)	σ_{max} (kN/m ²)	σ_{min} (kN/m ²)	L_t (m)	F (kN)
1	986.309	384.212	0.2	0.016	0.5	12096.556	-11947.506	0.49	585.427
	-17.963	375.5866							
	-14.905	384.705							
2	162.32	275.658	0.2	0.016	0.5	11280.59	-10220.66	0.47	480.371
	-11.179	202.339							
	105.993	344.02							
3	395.328	143.0299	0.2	0.016	0.5	6050.395	-6007.105	0.49	294.348
	155.501	156.906							
	4.329	192.92							

Table V. 20: Evaluation of stresses in shear wall W1 (1.5x0.2) m.

Zone	N (kN)	M (kN.m)	Ω (m ²)	I (m ⁴)	V (m)	σ_{max} (kN/m ²)	σ_{min} (kN/m ²)	L_t (m)	F (kN)
1	2201.127	1077.916	0.3	0.056	0.75	21773.46	-7099.285	0.37	262.673
	-1478.931	1002,726							
	-1478.931	1002,726							
2	835.793	47.548	0.3	0.056	0.75	12277.292	-11281.546	0.71	800.989
	-16.475	445.293							
	-149.362	879.53							
3	512.967	266.805	0.3	0.056	0.75	8012.164	-6665.871	0.68	453.279
	121.718	52.771							
	201.944	547.98							

Table V. 21: Evaluation of stresses in shear wall W1 (1.65x0.2) m.

Zone	N (kN)	M (kN.m)	Ω (m ²)	I (m ⁴)	V (m)	σ_{max} (kN/m ²)	σ_{min} (kN/m ²)	L_t (m)	F (kN)
1	1598.67	535.24	0.33	0.074	0.825	13218.189	-10246.304	0.72	737.733
	-14.905	384.705							
	490.361	1052.34							
2	649.610	3.049	0.33	0.074	0.825	10616.311	-8343.972	0.72	600.765
	291.701	130.308							
	374.936	850.34							
3	420.145	14.9196	0.33	0.074	0.825	6738.189	-5190.195	0.71	368.503
	101.045	97.948							
	255.419	534.97							

V.3.5 Shear walls classification

$$\frac{h_w}{l_w} > 2 \Rightarrow \text{Slender shear wall}$$

$$\frac{h_w}{l_w} \leq 2 \Rightarrow \text{Short shear wall}$$

$$\frac{h_w}{l_w} \leq \frac{2}{3} \Rightarrow \text{Large – scale shear wall}$$

Table V. 22: Shear walls classification.

Section	$l_w(m)$	$h_w(m)$	h_w / l_w	Note
W1	1	27.54	27.54	Slender shear wall
W2	1.5	27.54	18.36	Slender shear wall
W3	1.65	27.54	11.12	Slender shear wall

Slender shear walls require the determination of a critical height to assess their structural behavior.

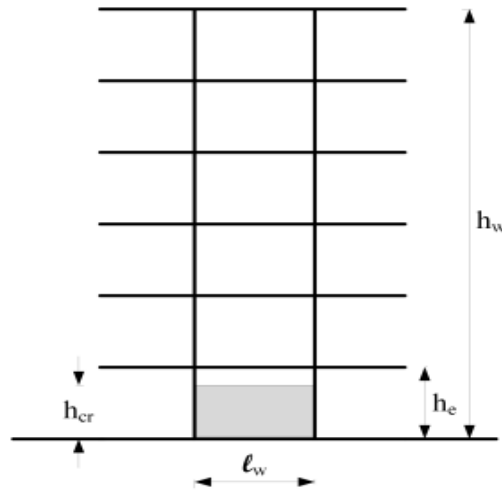


Figure V. 6: Critical region at the base of the shear wall.

$$\text{Critical height: } h_{cr} = \max \left(l_w; \frac{h_w}{6} \right) \quad h_{cr} \leq \begin{cases} 2l_w \\ h_e \Rightarrow n \leq 6 \quad \text{level} \\ 2h_e \Rightarrow n > 6 \quad \text{level} \end{cases}$$

❖ Boundary element

$$\text{Confined length: } l_c \geq \max(0.15l_w; 1.5b_w)$$

$$\text{Confined depth: } \begin{cases} l_c \leq \max(2b_w; 0.2l_w) \Rightarrow b_c \geq \frac{h_e}{15} \\ l_c > \max(2b_w; 0.2l_w) \Rightarrow b_c \geq \frac{h_e}{10} \end{cases}$$

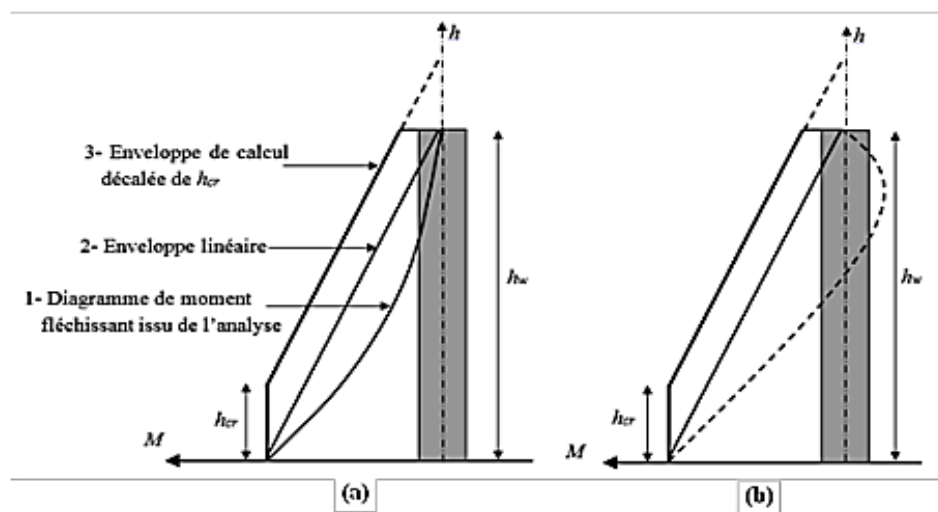
$$\text{Minimum confined area: } A_c \geq 0.5\% l_c \times b_c$$

Table V. 23: Dimensions of confined boundary elements and critical heights for shear walls.

Section	$l_w (m)$	$h_{cr}(m)$	$l_c (m)$	$b_c (m)$
W1	1	2	0.3	0.2
W2	1.5	3	0.4	0.2
W3	1.65	3.3	0.5	0.5

V.3.6 Shear wall reinforcement: (Art 7.7.4 RPA 2024)

For slender shear walls characterized by a ratio $(h_w/l_w) > 2$, the design bending moment diagram along the elevation is represented by a linearized envelope. This envelope derived from flexural moment diagram obtained through structural analysis and translated vertically by a tension shift equal to the critical wall zone height (h_{cr}).

**Figure V. 7:** Bending moment envelope.

- First, the initial shear force diagram is increased by 40%.
- Second, the amplified diagram is kept constant from the base of the wall up to a height equal to $\frac{h_w}{3}$.
- Finally, a linear transition envelope is adopted between $\frac{h_w}{3}$ and the top of the wall.

The modified design envelope for shear forces is applicable only to dual structural systems. This modification is introduced to account for uncertainties associated with higher-mode effects.

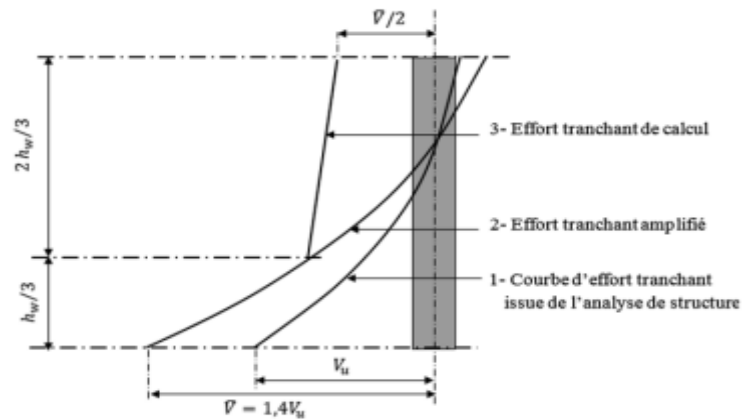


Figure V. 8: Shear force envelope analysis of slender walls in dual lateral load-resisting systems.

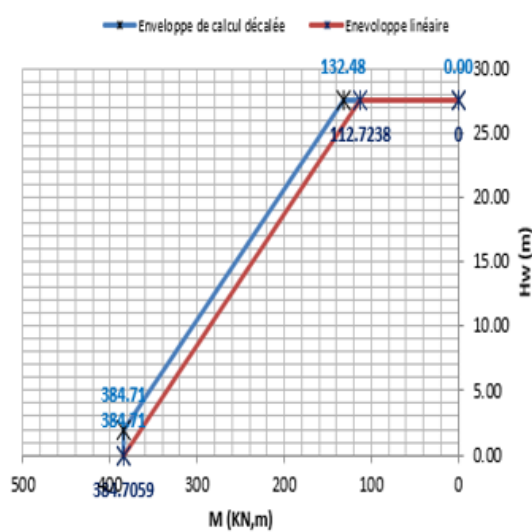


Figure V. 9: Bending moment envelope for wall W.

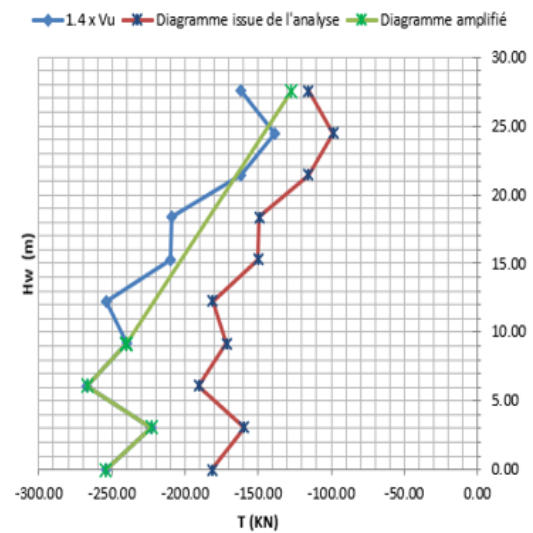


Figure V. 10: Shear force envelope for wall W1.

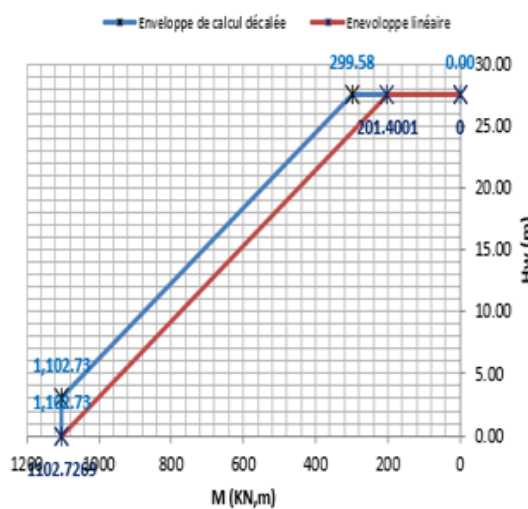


Figure V. 11: Bending moment envelope for shear wall W2.

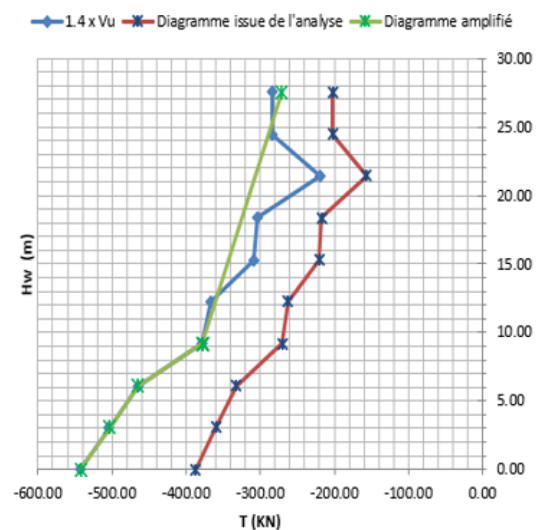


Figure V. 12: Shear force envelope for shear wall W2.

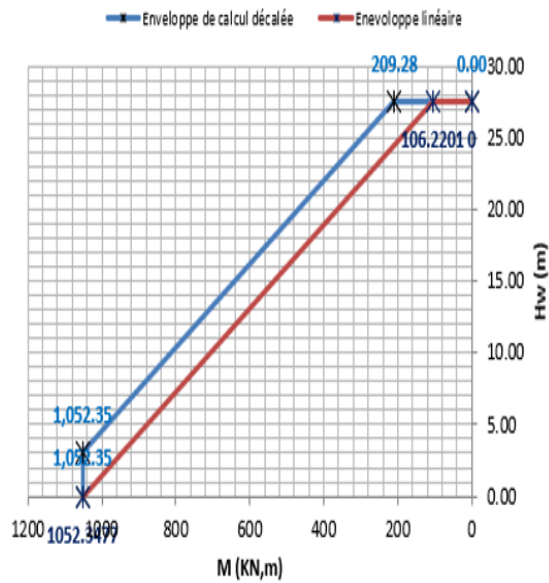


Figure V. 13: Bending moment envelope for shear wall W3.

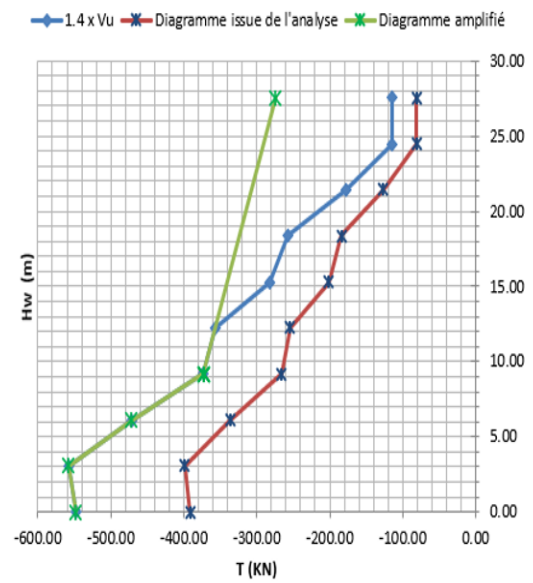


Figure V. 14: Shear force envelope for shear wall W3.

Table V. 24: Design forces and reinforcement requirements for shear walls.

Zone	Wall	N (kN)	M (kN.m)	A_s (cm^2)	A_c min RPA (cm^2)	A_c min BAEL (cm^2)	A_{adop} (cm^2)	Choice	A_h (cm^2)	Choice	A_{vj} (cm^2)
1	(1x0.2)	-14.905	384.705	10.69	3	10	8HA14	12.31	0.529	2HA10	7.335
	(1.5x0.2)	-1478.931	1002,726	37.49	4	15	12HA20	37.70	0.645	2HA10	15.062
	(1.65x0.2)	490.361	1052.347	12.08	5	16.5	12HA14	18.47	0.606	2HA10	15.071
2	(1x0.2)	105.993	344.02	8.16	3	10	6HA14+2HA12	11.50	0.288	2HA8	6.990
	(1.5x0.2)	-149.362	879.53	17.56	4	15	12HA14	18.47	0.450	2HA8	10.401
	(1.65x0.2)	374.936	850.34	9,61	5	16.5	8HA14+4HA12	16.84	0.406	2HA8	10.253
3	(1x0.2)	4.329	192.92	5.05	3	10	6HA14+2HA12	11.50	0.237	2HA8	4.785
	(1.5x0.2)	201.944	547.98	12.09	4	15	8HA14+4HA12	12.09	0.343	2HA8	8.347
	(1.65x0.2)	255.419	534.97	5,49	5	16.5	8HA14+4HA12	16.84	0.293	2HA8	8.894

- The vertical reinforcement of the boundary element shall be confined by transverse ties and/or horizontal stirrups, with a vertical spacing complying with the following requirement (art 7.7.4 RPA2024):

$$s_t \leq \min\left(\frac{b_0}{3}; 12.5 \text{ cm}; 6\Phi_l\right) \Rightarrow s_t = 5$$

Φ_l : The minimum diameter required for the longitudinal reinforcing bars in the boundary elements $\Phi_l \geq 12 \text{ mm}$.

- The horizontal spacing between two tied vertical reinforcement bars shall not exceed 20 cm.
- The area of the confinement reinforcement in the boundary elements (A_t), evaluated in the direction parallel to the wall thickness, shall extend over the height (h_{cr}) and complying with the following requirements.21

$$A_t = \max \begin{cases} 0.09 \cdot s_t \cdot b_0 \cdot \frac{f_{c28}}{f_e} \\ 0.09 \cdot s_t \cdot b_0 \cdot \left(\frac{A_g}{A_c} - 1\right) \cdot \frac{f_{c28}}{f_e} \end{cases} \quad \begin{cases} A_g = l_c \times b \\ A_c = (l_c - 2e) \times b_0 \end{cases}$$

b_0 : Refers to the width of the confined core within the boundary element.

$\frac{A_g}{A_c}$: Corresponds to the ratio of the gross area of the boundary element to the confined core area.

Table V. 25: Transverse reinforcement requirements in confined boundary elements of shear walls.

Wall	$A_g(\text{cm})$	$A_c(\text{cm})$	$A_t(\text{cm}^2)$	$A_{t \text{ adopted}}(\text{cm}^2)$
(1x0.2)	600	500	0.562	6T8 = 3.01
(1.5x0.2)	800	700	0.562	4T8 = 2.01
(1.65x0.2)	2500	2250	1.406	6T8 = 3.01

❖ Wall web reinforcement

The maximum diameter, $\Phi_{l \max}$ of the web reinforcement bars shall be satisfy the following requirement:

$$8\text{mm} \leq \Phi_{l \max} \leq \frac{b_w}{8} \quad \Rightarrow \quad 8\text{mm} \leq \Phi_{l \max} \leq \frac{200}{8}$$

$$S_t \leq \min(250 \text{ mm}; 25 \Phi_{l \min})$$

Wall W1: (1x0.2) m $\Rightarrow \Phi_{l \max} = 14 \text{ mm}$

Wall W2: (1.5x0.2) m $\Rightarrow \Phi_{l \max} = 16 \text{ mm}$

Wall W3: (1.65x0.2) m $\Rightarrow \Phi_{l\max} = 12\text{ mm}$

V.3.7 Local ductility requirements: (Art 7.7.5 RPA 2024)

In rectangular wall sections, the volumetric mechanical reinforcement ratio of the confinement reinforcement required in the boundary elements, denote $\omega_{\omega d}$, must comply with the following condition:

$$\alpha\omega_{\omega d} = 30 \cdot \mu_{\Phi} \cdot (v_d + \omega_v) \cdot \varepsilon_{syd} \cdot \frac{b_c}{b_0} - 0.035$$

$$\omega_{\omega d} = \frac{\text{Volumetric confinement reinforcement}}{\text{Volume of the concrete core}} \cdot \frac{f_{yd}}{f_{ed}}$$

$$\frac{f_{yd}}{f_{ed}} = \frac{f_e}{f_{c28}} \cdot \frac{\gamma_b}{\gamma_s}$$

Where:

μ_{Φ} : Required curvature ductility factor.

ε_{sy} : Steel strain at yield, defined as follows

$$\varepsilon_{sy} = \frac{f_e}{E_s}$$

E_s : Modulus of elasticity of steel.

v_d : Normalized axial force section for a rectangular section.

$$v_d = \frac{N_d}{B_c \cdot f_{c28}}$$

ω_v : Normalized percentage of vertical web reinforcement.

$$\omega_v = \frac{A_{sv}}{(l_w - 2l_c) \cdot b_w} \cdot \frac{f_{yd}}{f_{ed}}$$

A_{sv} : Reinforcement area associated with the wall web section.

f_{yd} : Design value of the steel yield strength.

f_{ed} : Design value of the concrete compressive strength.

α : Confinement efficiency factor, equal to (α_n, α_s) , where:

$$\begin{cases} \alpha_n = 1 - \sum_n \left(\frac{b_i^2}{6b_0 \cdot h_0} \right) \\ \alpha_n = \left(1 - \frac{t}{2b_0} \right) \left(1 - \frac{t}{2h_0} \right) \end{cases}$$

n : Total number of longitudinal bars laterally restrained by confinement reinforcement or cross-ties.

b_i : Spacing between consecutive restrained bars.

b_0, h_0, t : Geometric parameters.

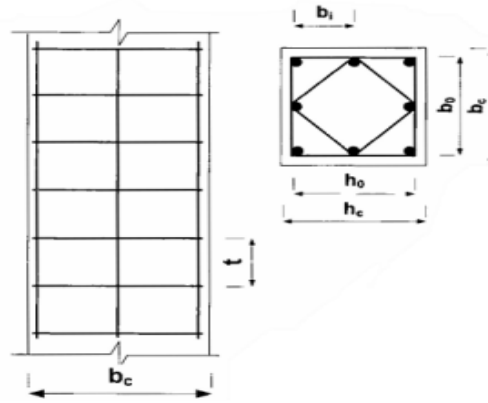


Figure V. 15: Longitudinal reinforcement details.

A minimum value of $\omega_{\omega d}$ equal to 0.12 should be provided within the critical region at the base of the wall.

The curvature ductility coefficient μ_{ϕ} can be determined from this equation, based on the fundamental vibration period T_0 , the spectral parameter T_2 , the behavior factor $\frac{R}{Q_F}$, and the ratio $\frac{M_{ED}}{M_{RD}}$.

$$\mu_{\phi} = \begin{cases} 2 \cdot \left(\frac{R}{Q_F} \cdot \frac{M_{ED}}{M_{RD}} \right) - 1 & T_0 \geq T_2 \\ 1 + 2 \cdot \left(\left(\frac{R}{Q_F} \cdot \frac{M_{ED}}{M_{RD}} \right) - 1 \right) \cdot \frac{T_2}{T_0} & T_0 < T_2 \end{cases}$$

M_{ED} : Corresponds to the bending moment obtained from structural analysis (design envelope shifted by h_{cr}).

M_{ED} : Refers to the design flexural strength.

The neutral axes depth, x_u corresponding to the ultimate curvature after crushing of the concrete outside the confined core of the boundary elements, may be estimated as follows:

$$x_u = (v_d + \omega_v) \cdot \frac{l_c \cdot b_c}{b_o}$$

The evaluation of the compressed length of the boundary elements requires the determination of the ultimate compressive strain $\varepsilon_{cu,c}$, of confined concrete at the failure state.

$$\varepsilon_{cu,c} = 0.0035 + 0.1 \alpha \omega_{\omega d} l_c \text{ calcul} = x_u \cdot \left(1 - \frac{\varepsilon_{cu}}{\varepsilon_{cu,c}} \right) \quad \text{with: } \varepsilon_{cu} = 0.0035$$

The calculated compressed length $l_c \text{ calcul}$ shall be greater than the max $(0.15l_w; 1.5b_w)$.

Table V. 26: Verification parameters for wall boundary element confinement.

Wall	$L_{tie}(\text{cm})$	V_{st}	V_{nc}	$\omega_{\omega d}$	α_n	α_s	α	$\alpha \omega_{\omega d}$
(1x0.2)	195	3136.566	90000	0.669	0.861	0.75	0.645	0.431
(1.5x0.2)	319.3	8024.884	180000	0.855	0.870	0.773	0.672	0.574
(1.65x0.2)	355	5174.831	742500	0.133	0.898	0.902	0.809	0.107

Table V. 27: Flexural capacity verification and reinforcement ratio of structural walls.

Wall	R	Q_f	M_{ED}	M_{RD}	μ_ϕ	A_{sv}	ω_v
(1x0.2)	4.5	1.25	384.705	1 414	0.958	6.16	0.1478
(1.5x0.2)			1002.726	1115	5.475	16.08	0.2205
(1.65x0.2)			1052.347	3 169	1.390	13.57	0.2

Table V. 28: Verification of ductility requirements for structural walls.

Wall	$\alpha\omega_{od}$	$30.u_0.(v_d + \omega_v) \cdot \varepsilon_{sy,d} \cdot \frac{b_c}{b_0} - 0.035$	Condition
(1x0.2)	0.431	-0.023	CV
(1.5x0.2)	0.431	0.314	CV
(1.65x0.2)	0.107	-9.7×10^{-3}	CV

Table V. 29: Verification of confined length in shear wall.

Wall	x_u	$\varepsilon_{cu,d}$	$L_{c,cal}$	$\max(0,15l_w; 1.5b_w)$	Condition
(1x0.2)	34.48	0.044	31.73	30	CV
(1.5x0.2)	102.7	0.046	94.885	40	CV
(1.65x0.2)	82.16	0.014	61.62	50	CV

V.3.8 Shear walls reinforcement detailing

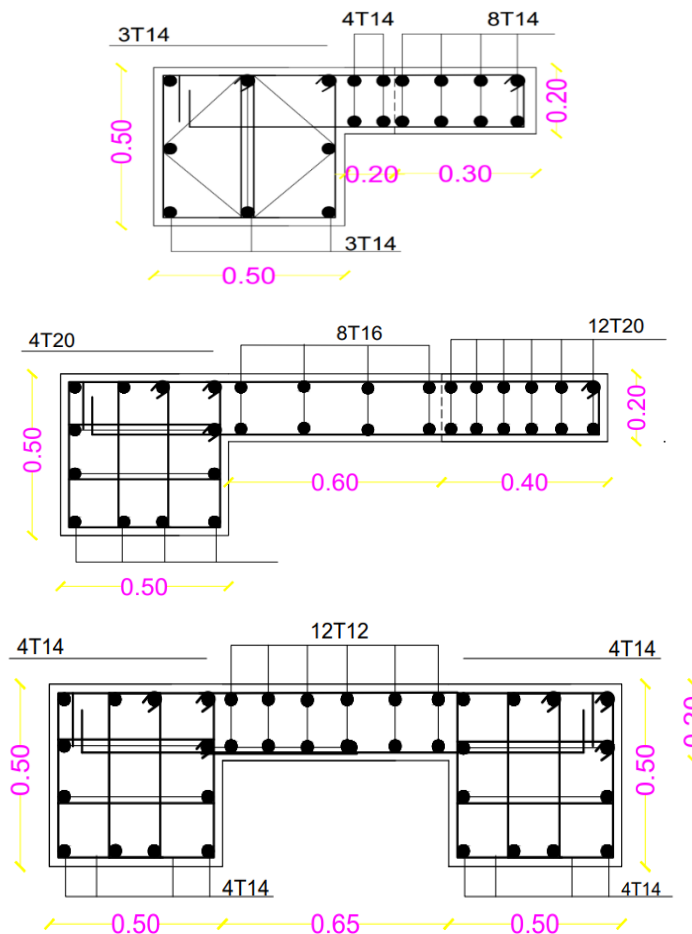


Figure V. 16: Shear walls reinforcement detailing.

V.4 Conclusion

The study of the load-bearing elements of the building ensured the stability, strength and satisfactory structural behavior of the project under the applied loads. The design of beams, columns, and shear walls was carried out in accordance with reinforced concrete design standards and current seismic regulations.

The different verification checks confirmed that tresses, deflections, and reinforcement detailing remain within the allowable limits. The obtained results demonstrate that the load-bearing elements provide sufficient capacity to resist both static and seismic actions, while ensuring the safety, stability, and durability of the structure throughout its service life.

Chapter VI:
**Substructure analysis and
design**

VI.1 Introduction

The primary function of a foundation is to safely transmit the loads from the superstructure to the ground. In the context of seismic design, the foundation must move in tandem with the soil during an earthquake, acting as a single unit. A rational design requires a precise knowledge of two fundamental parameters:

- The total load to be transferred to the soil.
- A thorough analysis of mechanical characteristic of the soil.

Based on these soil properties, two main foundation typologies are distinguished:

- ❖ Shallow foundation: suitable for competent soils found at shallow depths ($D < 3$)
- Mechanism: direct transmission of the loads via contact between the structure and the soil.
- Standard types: isolated footing (for point loads), continuous strip footing (for linear loads), and raft/mat foundation (for distributed loads).
- ❖ Deep foundation: applied when soils are compressible or where the competent bearing stratum is found at greater depths ($D > 5 \text{ m}$)
- Mechanism: load transfer occurs through lateral skin friction and/or end-bearing resistance.
- Standard types: driven or bored piles, piers, and micropiles.
- ❖ **Load combination**
- Combination according to the RPA 2024:
$$\begin{cases} G + \psi Q + 1.4E_1 \\ G + \psi Q + 1.4E_2 \end{cases} \quad \begin{cases} E_1 = \pm E_x \pm 0.3E_y \\ E_2 = \pm 0.3E_x \pm E_y \end{cases}$$
- Combination based on the BAEL 91/99:
$$\begin{cases} ULS: 1.35G + 1.5Q \\ SLS: G + Q \end{cases}$$

VI.2 Soil investigation and characterization

The allowable bearing capacity of the soil is a critical parameter determined experimentally. This value can be estimated either from experience gained on similar sites supporting existing structures, or from the results of laboratory tests and geotechnical site investigations. In the context of this preliminary study, the allowable soil bearing capacity is evaluated at 0.2 MPa.

VI.3 Selecting the foundation type

The appropriate foundation system is selected based on analysis of the following key parameters:

- Structural characteristic: the architectural configuration and structural typology, the density of constitutive materials, and the height to width ratio (slenderness) of the building.
- Mechanical loads: the magnitude of permanent (G) and variable (Q) loads, the spatial distribution of the forces (concentric or eccentric), and the nature of dynamic actions (seismic loads).

For this specific project, two feasible solutions are examined: continuous strip footing or a general raft foundation.

VI.4 Feasibility study for strip footing

Regulation require that the total footprint of the strip footing does not exceed 50% of the total building footprint. this limit is defined by the following condition:

$$\text{The footing area: } \frac{N}{S_{footing}} \leq \bar{\sigma}_{soil} \rightarrow S_{footing} \geq \frac{N}{\bar{\sigma}_{soil}}$$

$$N_s = 931.990 \text{ KN} \quad \bar{\sigma}_{soil} = 2 \text{ bars} = 200 \text{ kN/m}^2$$

$$S_1 \geq \frac{931.990}{200} = 4.659 \text{ m}^2$$

$$- \quad S_{tot} = S_1 \times n_{column} \Rightarrow S_{tot} = 4.659 \times 26 \Rightarrow S_{tot} = 121.134 \text{ m}^2$$

$$\text{Building footprint area } S_b = 218.87 \text{ m}^2$$

❖ Verification

$$\frac{S_{tot}}{S_b} = \frac{121.134}{218.87} = 0.5534 = 55.34\% > 50\%$$

❖ Conclusion

The required area for strip footing represents over 50% of the building's footprint, which implies significant geometrical overlapping. Consequently, strip footings are deemed unfeasible, leading to the adoption of a raft foundation.

Advantage of the raft foundation:

- The increased contact area with soil significantly reduces the applied bearing stress ($\sigma = N/S$), improving geotechnical stability.
- It effectively limits differential settlements between different parts of the structure.
- It facilitates simpler and faster execution on the construction site.

VI.5 Raft foundation

A raft foundation consists of a continuous concrete slab, with or without ribs, extending over the whole building footprint in order to support the structure uniformly.

Although this foundation system is generally more expensive than isolated or strip footings, it is commonly adopted when shallow foundations systems cannot be effectively adopted.

VI.5.1 Preliminary sizing

❖ Simplified conditions

The raft thickness must comply with the following condition:

$$\frac{L_{max}}{8} \leq h_r \leq \frac{L_{max}}{5}$$

$L_{max} = 4.74 \text{ cm}$: The maximum span between supports

$$59.25 \text{ cm} \leq h_r \leq 94.8 \text{ cm}$$

❖ Rigidity condition

The raft must be sufficiently rigid to limit differential settlements. This is ensured by the following condition: $L_{max} \leq \frac{\pi}{2} \cdot Le$ where $Le = \sqrt[4]{\frac{4EI}{bK}}$

$$Le = 4.85 \text{ m} \quad ; \quad L_{max} = 4.74 \text{ cm} \leq 7.62 \text{ cm} \dots CV$$

- E: young's modulus of concrete (32164.195 MPA)
- b: Raft foundation width, considering a 1m wide strip.
- K: Coefficient of soil subgrade reaction (40000 KN/m³)
- I: moment of inertia of the raft cross section ($bh^3/12$)

By substituting I and rearranging the terms. The expression is resolved to provide a lower bound for the raft thickness (h_r):

$$h_r \geq \sqrt[3]{\frac{48KL_{max}^4}{E\pi^4}} \Rightarrow h_r \geq 62.54 \text{ cm}$$

❖ Adopted thickness and optimization

A raft thickness of $h_r = 90 \text{ cm}$ was adopted. This dimension satisfies the recommended empirical limits ($58.875 \text{ cm} < h_r < 94.2 \text{ cm}$) as well the rigidity requirement $h_r \geq 62.54 \text{ cm}$. Considering the relatively large structural depth, the use of a full solid raft would lead to excessive self-weight and increased construction cost. For this reason, a ribbed raft system was selected as a more efficient structural solution in order to:

- Improve the global bending stiffness of the foundation.
- Reduce the risk of differential settlement and deformation.

The rib dimensions are determining using the following expressions:

$$h_{rib} \geq \frac{L_{max}}{2} \Rightarrow h_n \geq \frac{474}{2} \Rightarrow h_n \geq 47.4 \text{ cm} \Rightarrow h_n = 90 \text{ cm}$$

- $b_{rib} \geq \frac{h_{rib}}{2} \Rightarrow b_n \geq \frac{90}{2} \Rightarrow b_n \geq 45 \text{ cm} \Rightarrow b_n = 60 \text{ cm}$

Note: The width of the ribs must be greater than the largest dimension of the column base so that the column is fully enclosed within the rib. This configuration ensures an efficient transfer of loads to the ribs and facilitates the reinforcement arrangement during construction.

The slab depth is determining using the following expression:

- $h_s \geq \frac{L_{max}}{20} \Rightarrow h_s \geq \frac{474}{20} \Rightarrow h_s \geq 23.7 \text{ cm} \Rightarrow h_s = 40 \text{ cm}$

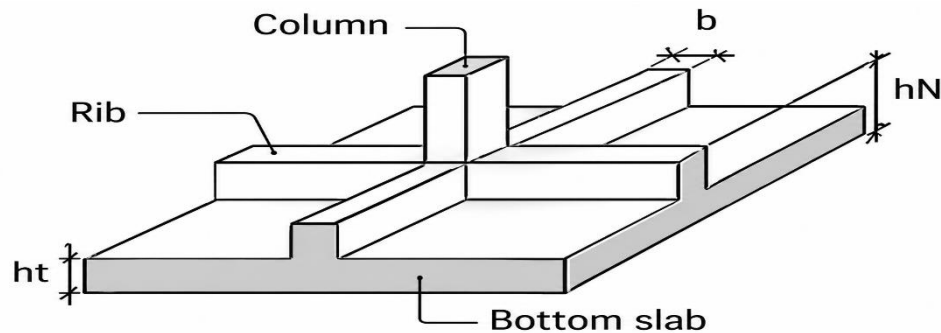


Figure VI. 1: Rib arrangement.

VI.5.2 Minimum required surface area

The area of the raft foundation must satisfy the following requirement:

$$G_{raft} = 4924.575 \text{ kN}$$

$$Q_{raft} = 547.175 \text{ kN}$$

$$G_{superstructure} = 20430.748 \text{ kN}$$

$$Q_{superstructure} = 3100.977 \text{ kN}$$

$$G_{total} = 25355.323 \text{ kN}$$

$$Q_{total} = 3648.152 \text{ kN}$$

$$N_u = 1.35G_{total} + 1.5Q_{total} \Rightarrow N_u = 39701.910 \text{ kN}$$

$$N_s = G_{total} + Q_{total} \Rightarrow N_s = 29003.472 \text{ kN}$$

The building footprint area is equal to $S_{building} = 220.87 \text{ cm}^2$

The area of the raft foundation must satisfy the following requirement:

❖ At ULS

$$S_{rad} \geq \frac{N_u}{1.33\sigma_{adm}} = 149.255 \text{ m}^2 \dots\dots\dots \text{CV}$$

❖ At SLS

$$S_r \geq \frac{N_s}{\sigma_{adm}} = 145.017 \text{ m}^2 \dots\dots\dots \text{CV}$$

❖ Conclusion

It is observed that the building footprint area ($S_{building} = 218.87 \text{ cm}^2$) exceeds the minimum area required for the raft foundation. Therefore, the design requirement is satisfied and no raft overhang is needed.

VI.6 Verification of the raft foundation

VI.6.1 Punching shear verification (CBA93. Art. A.5.2.4)

Punching shear occurs due to concentrated loads (from columns or walls) and manifests as a critical rupture surface forming a truncated cone at a 45° angle. The non-punching condition aims to prevent local failure of the raft and is expressed by:

$$N_u \leq 0.045 \times u_c \times h_r \times \frac{f_{c28}}{\gamma_b}$$

- N_u : design load at the ULS.
- μ_c : perimeter of the critical contour.
- h : total thickness of the raft.

❖ Under structural walls

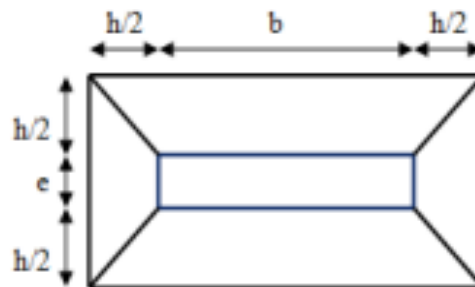


Figure VI. 2: Load transfer mechanism under shear wall.

$$N_{u,wall} = 1153.6028 \text{ kN}$$

$$u_c = 2(e + L + 2h_r) \Rightarrow u_c = 13.48 \text{ m}$$

$$h \geq \frac{N_u}{0.045 \times u_c \times \frac{f_{c28}}{\gamma_b}} \Rightarrow h \geq 0.087 \text{ m} \dots \text{CV}$$

❖ Under columns

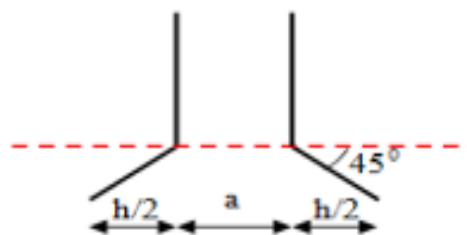


Figure VI. 3: Load transfer mechanism under column.

$$N_{u,column} = 1278.661 \text{ kN}$$

$$\mu_c = 4(a + h_r) = 5.6 \text{ m}$$

$$h \geq \frac{N_u}{0.045 \times u_c \times \frac{f_{c28}}{\gamma_b}} \Rightarrow h \geq 0.233 \text{ m} \dots \text{CV}$$

VI.6.2 Shear stress condition

The thickness of the raft must also satisfy shear resistance criteria according to CBA93(article 5.1) and RPA 2024, the condition is:

$$\tau_u = \frac{V_u}{b \cdot d} \leq \frac{0.06 f_{cj}}{\gamma_b} = 1 \text{ MPa}$$

Where:

- V_u : design shear force at ULS.
- b : width of the strip (1m).
- $d = 0.9h$: effective depth.
- $\gamma_b: 1.5$

❖ Calculation

The shear force V_u is derived from the ultimate soil pressure (q_u):

$$q_u = \frac{N_u}{S_{rad}} \Rightarrow q_u = \frac{39701.910}{218.87} \Rightarrow q_u = 181.394 \text{ kN}$$

$$V_u = q_u \times \frac{L_{max}}{2} \Rightarrow V_u = 181.394 \times \frac{4.74}{2} = 429.903 \text{ kN}$$

$$\tau_u = \frac{V_u}{d \times b} \Rightarrow \tau_u = \frac{429.903 \times 10^{-3}}{0.9 \times 0.9 \times 1} = 0.530 \text{ MPa} < 1 \text{ MPa} \dots \dots \dots CV$$

VI.6.3 Verification of stability against hydrostatic uplift pressure

To ensure the structure's stability against groundwater pressure, the following condition must be checked to prevent buoyancy:

$$W \geq F_s \times \gamma \times Z \times S$$

Where:

- W : total weight of the structure at the raft bas.
- $W_{tot} = W_{superstructure} + W_{raft} \Rightarrow W_{tot} = 26285.616 \text{ kN}$
- F_s : safety factor against uplift pressure ($F_s = 1.5$).
- γ : unit weight of water ($\gamma = 10 \text{ kN/m}^3$).
- Z : depth of the infrastructure ($Z = 2.75 \text{ m}$).
- S : surface area of the raft ($S = 218.87 \text{ m}^2$).

Calculation:

$$F_s \times \gamma \times Z \times S = 1.5 \times 10 \times 2.75 \times 218.87$$

$$26285.616 \text{ kN} > 9028.387 \text{ kN} \dots \dots \dots CV$$

VI.6.4 Geometric properties of the raft foundation

a) Centroid of the raft foundation

$$X_{G,r} = \frac{\sum X_i S_i}{\sum S_i} = 11.770 \text{ m} ; \quad Y_{G,r} = \frac{\sum Y_i S_i}{\sum S_i} = 7.559 \text{ m}$$

b) Moment of inertia of the raft:

$$I_x = 15910.999 \text{ cm}^4 ; \quad I_y = 6236.573 \text{ cm}^4$$

c) Eccentricity

$$X_{G,r} = X_{G,str} = 11.770 \text{ m} ; \quad Y_{G,r} = Y_{G,str} = 7.559 \text{ m}$$

$$e_x = |X_{G,str} - X_{G,r}| = 0 \text{ m} ; \quad e_y = |Y_{G,str} - Y_{G,r}| = 0 \text{ m}$$

VI.6.5 Assessment and verification of contact stress under the raft foundation

The contact pressures exerted by the raft foundation on the supporting soil must be not exceed the allowable bearing capacity of the ground. As the result the vertical load coincide with the centroid of the raft area, the soil pressure distribution is non-uniform and may be triangular or trapezoidal.

N: Axial load due to vertical actions.

M: Eccentric moment due to load eccentricity.

$$M = N \times e$$

The calculation of the average stress is performed using the following formulation:

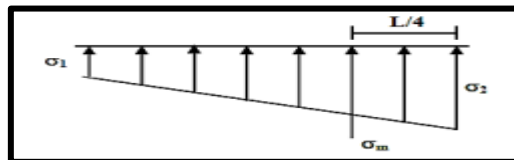


Figure VI. 4: Stress diagram under the raft.

$$\sigma_{1,2} = \frac{N}{S} \pm \frac{M \times V}{I}$$

$$\sigma_{moy} = \frac{3\sigma_1 + \sigma_2}{4} < 1.5\sigma_{allowable}$$

Table VI. 1: Stress verification under the raft.

	ULS		SLS	
	Longitudinal	Transverse	Longitudinal	Transverse
N (kN)	39701.910	39701.910	29003.472	29003.472
M (kN.m)	0	0	0	0
V (m)	11.770	7.559	11.770	7.559
I (cm⁴)	15910.999	6236.573	15910.999	6236.573
S (cm²)	218.87	218.87	218.87	218.87
σ_1 (kN/m²)	181.394	181.394	132.514	132.514
σ_2 (kN/m²)	181.394	181.394	132.514	132.514
σ_{moy} (kN/m²)	181.394	181.394	132.514	132.514
$1.5\sigma_{allowable}$ (kN/m²)	300	300	300	300
Condition	CV	CV	CV	CV

VI.6.6 Overturning stability verification of the raft foundation:

The structural system must maintain adequate stability when subjected to horizontal forces. This is assessed by comparing the stabilizing moment with the overturning to ensure sufficient safety against overturning and conformity with seismic design provisions.

Condition of stability: $\frac{M_s}{M_r} > 1.3$

Were

M_s : Stabilizing moment (due to self-weight). $\begin{cases} M_{s,x} = W \times x \\ M_{s,y} = W \times y \end{cases}$

$M_{overturninig}$: Total overturning moment at the base, defined as:

$$M_{overturninig} = M_0 + h.V$$

M_0 : seismic base moment (refer to chapter 4).

h : height of application (taken here as the raft thickness =2.75 m).

V : base shear force.

Table VI. 2: Overturning moment at the base of the raft foundation.

Direction	W (kN)	b (m)	M_s (kN.m)
X-direction	26285.616	11.770	309381.700
Y-direction	26285.616	7.559	198692.971

Table VI. 3: Stabilizing moment.

Direction	M_0 (kN.m)	h (m)	V (kN)	$M_{overturninig}$ (kN.m)
X-direction	58349.368	2.75	2943.101	66442.901
Y-direction	65566.189	2.75	3270.314	74559.552

$$\frac{M_{sx}}{M_{overturninig,x}} = 4.656 > 1.3 \dots \text{Verified}$$

$$\frac{M_{sy}}{M_{overturning,y}} = 2.664 > 1.3 \dots \text{Verified}$$

Table VI. 4: Overturning verification.

	G+ψQ+1.4E	
	G+ψQ+1.4E₁	G+ψQ+1.4E₂
N (kN)	43185.931	48440.222
M (kN.m)	66442.901	74559.552
V (m)	11.770	7.559
I (cm⁴)	15910.999	6236.573
S (cm²)	218.87	218.87
σ₁ (kN/m²)	246.463	311.689
σ₂ (kN/m²)	148.162	130.950
σ_{moy} (kN/m²)	221.887	266.504
1.5σ_{allowable} (kN/m²)	300	300
Condition	CV	CV

VI.6.7 Overturning stability verification of the raft foundation:

According to the provisions of the Algerian Seismic Code (RPA 2024)

$$e = \frac{M}{N} < \frac{L}{4}$$

e: Eccentricity of the resultant vertical load.

M: Global overturning moment of structure.

N: Total axial force acting on the structure.

Table VI. 5: Overturning stability verification of the raft foundation.

	G+ψQ+1.4E	
	G+ψQ+1.4E₁	G+ψQ+1.4E₂
N (kN)	43185.931	48440.222
M (kN.m)	66442.901	74559.552
e (m)	1.538	1.539
L/4 (m)	5.89	3.675
Verification	CV	CV

VI.7 Reinforcement of the raft panels

The raft is analyzed as an inverted slab supported by the building's shear walls. Each panel is designed as a slab on four edges and subjected to soil pressure loads. Bending moments are distributed according to the recommendations of BAEL 91, while the reinforcement is designed for flexure action under detrimental cracking conditions resulting from the presence of groundwater.

VI.7.1 Calculation of solicitations

The raft foundation is composed of slab panels supported along all four edges and subjected to uniformly distributed soil pressure.

- The structural behavior of each rectangular panel depends on its aspect ratio: $\alpha = \frac{L_x}{L_y}$

Where:

L_x : is the shorter span. ($L_x=4.07$ m)

L_y : is the longer span. ($L_y=4.74$ m)

- If $\alpha < 0.4$: the panel acts in one direction.
- If $0.4 \leq \alpha \leq 1$: the panel acts in two directions.

$$\alpha = \frac{4.07}{4.74} = 0.85 \rightarrow \text{the panel acts in two directions.}$$

❖ The design bending moments per unit width are evaluated using:

- In the short span direction: $M_x = \mu_x \cdot q \cdot L_x^2$
- In the long span direction: $M_y = \mu_y \cdot M_x$

The coefficients μ_x and μ_y depend on the aspect ratio $\alpha = \frac{L_x}{L_y}$.

❖ The panel under consideration is located between adjacent supports and is therefore treated as a continuous element. Based on this boundary conditions, the bending moment can be distributed between the span and the support.

- In the short span direction:

$$\begin{cases} \text{Support: } M_{ax} = -0.5M_x \\ \text{Span: } M_{tx} = 0.75M_x \end{cases}$$

- In the long span direction:

$$\begin{cases} \text{Support: } M_{ay} = -0.5M_y \\ \text{Span: } M_{ty} = 0.75M_y \end{cases}$$

❖ Evaluation of loads

$$q_u = \frac{N_u}{S_{raft}} = 181.394 \text{ kN.m} \quad ; \quad q_s = \frac{N_s}{S_{raft}} = 132.514 \text{ kN.m}$$

Table VI. 6: The design bending moments per unit width at ULS and SLS.

ULS			SLS		
α	μ_x	μ_y	α	μ_x	μ_y
0.85	0.0506	0.6864	0.85	0.0567	0.7794
$M_x = \mu_x \cdot q \cdot L^2$		$M_y = \mu_y \cdot M_x$	$M_x = \mu_x q L^2$		$M_y = \mu_y M_x$
152.041		104.361	124.461		97.005

The moment at span and supports are derived using the coefficients for an intermediate panel with fully fixed supports.

Table VI. 7: Bending moment in span and support at ULS and SLS.

	Short span direction		Long span direction	
	Support	Span	Support	Span
	$M_{tx} = -0.5M_x$	$M_{ax} = 0.75M_x$	$M_{ty} = -0.5M_y$	$M_{ay} = 0.75M_y$
ULS	76.020	114.030	52.180	78.270
SLS	62.230	93.345	48.502	72.753

Maximum spacing (prejudicial cracking):

$$S_{tx} \leq \min(33 \text{ cm}; 3h) \rightarrow S_{tx} = 15 \text{ cm}$$

$$S_{ty} \leq \min(33 \text{ cm}; 3h) \rightarrow S_{ty} = 15 \text{ cm}$$

VI.7.2 Reinforcement calculation at ULS

Table VI. 8: Reinforcement dimensioning of the raft at ULS.

Panel section	Direction X-X		Direction Y-Y	
	Support	Span	Support	Span
M_u (kN.m)	76.020	114.030	52.180	78.270
A_s' (cm ² /ml)	0	0	0	0
A_s (cm ² /ml)	6,03	9,15	4.11	6,21
$A_s \text{ min}$ (cm ² /ml)	4.347	4.347	4.347	4.347
Selected bars	7HA16	7HA16	7HA16	7HA16
$A_s \text{ adopted}$ (cm ² /ml)	14.07	14.07	14.07	14.07
Spacing (cm)	15	15	15	15

VI.7.3 Verification of the stresses at (SLS)

Table VI. 9: Stress verification at the raft foundation level.

Direction		M_s (kN.m)	Y (m)	I (m ⁴)	σ_{bc} (MPa)	σ_{st} (MPa)	Verification
X-X	Support	62.230	10.395	185809.175	3.487	128.839	CV
	Span	93.345			5.230	193.259	CV
Y-Y	Support	48.502			2.717	100.417	CV
	Span	72.753			4.076	150.626	CV

VI.7.4 Reinforcement layout of the raft slab

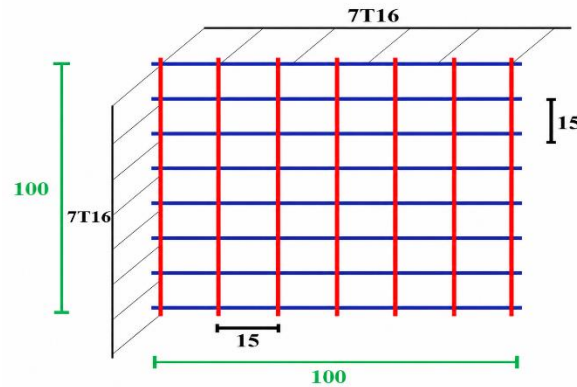


Figure VI. 5: Reinforcement layout of the raft foundation slab schematic.

VI.8 Study of the rib (ribbed slab)

The ribs are considered as doubly fixed beams.

$h=90$ cm ; $d=81$ cm ; $b= 60$ cm

VI.8.1 Evaluation of applied load and resulting bending moments

❖ Direction X-X

- Ultimate load: $q_u = \frac{N_u}{s} = 181.394$ kN.ml
- Service load : $q_s = \frac{N_s}{s} = 132.514$ kN.ml
- Ultimate isostatic bending moment: $M_{0u,x} = \frac{q_u \times l^2}{8} = 375.596$ kN.ml
- Serviceability isostatic bending moment: $M_{0s,x} = \frac{q_s \times l^2}{8} = 274.385$ kN.ml

For the reinforcement design, the maximum moments at supports and spans are used:

- Span moment (M_t) = $0.85M_0$
- Support moment (M_a) = $-0.5M_0$

❖ Ultimate limit state (ULS)

- Span bending moment $M_t = 319.256$ kN.ml
- Support bending moment (M_a) = -187.798 kN.ml

❖ Serviceability limit state (SLS)

- Span bending moment $M_t = 233.227$ kN.ml
- Support bending moment $M_a = -137.192$ KN.ml

❖ Direction Y-Y

- Ultimate load: $q_u = \frac{N_u}{s} = 181.394$ kN.ml

- Service load : $q_s = \frac{N_s}{s} = 132.514 \text{ kN.m}$
- Ultimate isostatic moment $M_{0u,y} = \frac{q_u \times l^2}{8} = 509.435 \text{ kN.m}$
- Service isostatic moment $M_{0s,y} = \frac{q_s \times l^2}{8} = 372.159 \text{ kN.m}$

For the reinforcement design, the maximum moments at supports and spans are used:

- Span moment (M_t) = $0.85M_0$
- Support moment (M_a) = $-0.5 \times M_0$
- ❖ **Ultimate limit state (ULS)**
- Span moment $M_t = 433.020 \text{ kN.m}$
- Support moment $M_a = -254.718 \text{ kN.m}$
- ❖ **Serviceability limit state (SLS)**
- Span moment (M_t) = 316.335 kN.m
- Support moment (M_a) = -186.079 kN.m

VI.8.2 Longitudinal reinforcement of the rib at (ULS)

The following table presents the calculated reinforcement sections for the raft slab panels based on the bending moments obtained at the ULS.

Table VI. 10: Rib longitudinal reinforcement at ULS.

	Zone	$M_u(\text{kN.m})$	$A_{\min}(\text{cm}^2)$	$A'_s(\text{cm}^2)$	$A_s(\text{cm}^2)$	$A_{\text{adopted}}(\text{cm}^2)$	selection
X_X	support	187.798	5.868	0	6.3	12.57	4HA20
	span	319.256	5.868	0	10.83	18.85	4HA20+2HA20
Y_Y	support	254.718	5.868	0	8.6	12.57	4HA20
	span	433.020	5.868	0	14.84	25.13	4HA20+4HA20

VI.8.3 Transverse reinforcement of the rib

$$\phi_t \leq \min\left(\frac{h}{35}; \frac{b}{10}; \phi_l\right) \Rightarrow \phi_t \leq 20 \text{ mm} \Rightarrow \text{We adopt: } \phi_t = 10 \text{ mm}$$

❖ Minimum transverse reinforcement (According to RPA 2024)

the minimum area: $A_t = 0.003 \times S_t \times b$

Spacing limit in nodal zone:

$$S_t \leq \min\left(\frac{h}{4}; 24\phi_t; 17.5 \text{ cm}; 6\phi_L\right) \Rightarrow \text{We adopt: } S_t = 10 \text{ cm}$$

Spacing limit in current zone:

$$s' \leq \frac{h}{2} \Rightarrow s' = 15 \text{ cm} \leq \frac{h}{2} = 45 \text{ cm} \dots \text{CV}$$

Resulting calculation remains: $A_t = 0.003 \times 15 \times 60 = 2.7 \text{ cm}^2$

Reinforcement bar selection: 4HA10..... $A_t = 3.14 \text{ cm}^2$

VI.8.4 Verification of shear force:

The shear stress is expressed by the following expression: $\tau_u = \frac{Vu}{b_0 d}$

Cracking is considered detrimental: $\bar{\tau}_u = \min (0.15 f_{ct28} / \gamma_b ; 4 \text{ MPa})$

$$V_u = q_u \times \frac{L}{2}$$

Table VI.12: Shear force verification

Direction	V_u (kN)	τ_u (MPa)	$\bar{\tau}_u$ (MPa)	Verification
X-X	369.136	0.759	2.5	CV
Y-Y	429.903	0.884	2.5	CV

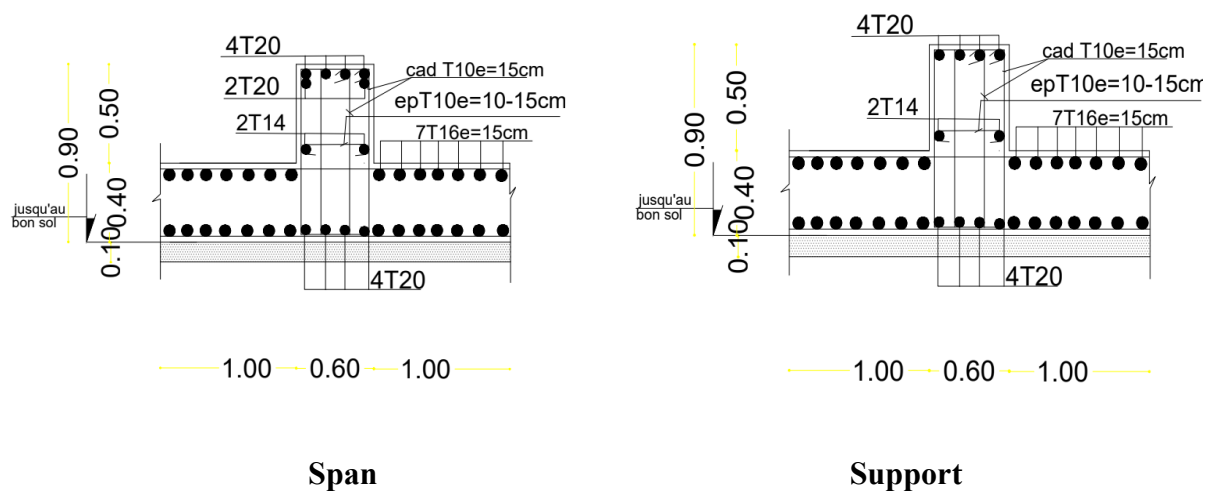
VI.8.5 Verification of the stresses at (SLS) of the rib

Table VI.12: Stress verification of the rib at (SLS).

Direction		M_s (kN.m)	Y (cm)	I (cm^4)	σ_{bc} (MPa)	σ_{st} (MP)	Verification
X-X	Support	137.192	19.638	861414.684	3.127	146.590	CV
	Span	233.227	23.316	1194343.02	4.553	168.964	CV
Y-Y	Support	186.079	19.638	861414.684	4.242	198.827	CV
	Span	316.335	26.232	1491688.41	5.562	174.215	CV

VI.8.6 Raft reinforcement detailing

X-X Direction



Y-Y Direction

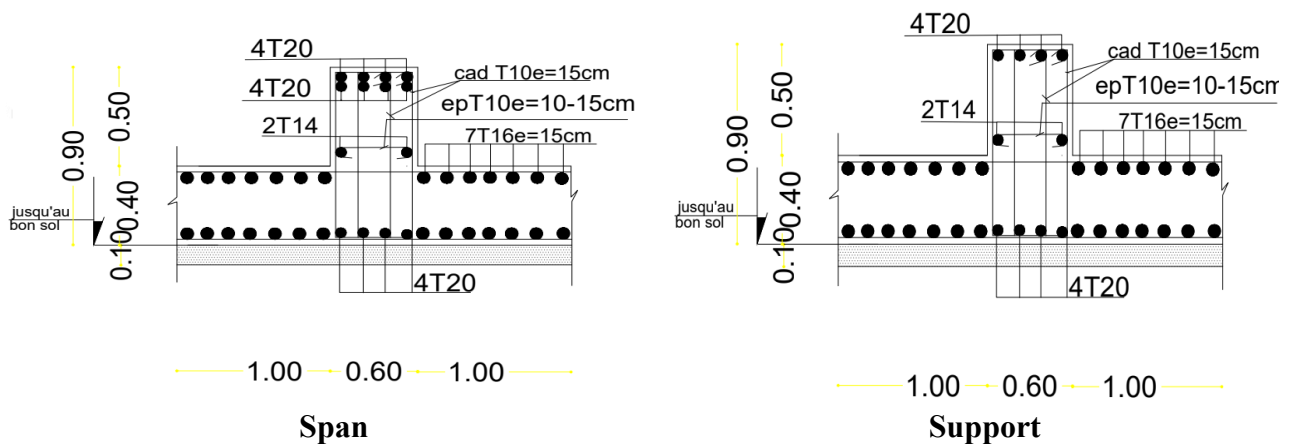


Figure VI. 6: Raft reinforcement detailing.

VI.9 Conclusion

This chapter presented the analysis and design of the reinforced concrete foundation system. The loads transferred from the superstructure were evaluated, and the required geotechnical and structural verifications were carried out to ensure the safety and stability of the foundation. The adopted foundation solution was addressed with respect to soil bearing capacity, contact pressure distribution, and overall structural performance under the applied loading conditions. The reinforcement design was performed in accordance with the relevant design provisions, ensuring compliance with both the Ultimate Limit State (ULS) and the Serviceability Limit State (SLS) requirements.

The obtained results demonstrate that the designed foundation system provides adequate strength, stability, and durability for structure under consideration. Consequently, the selected foundation solution is considered suitable for the safe transfer of structural loads to the so

General conclusion

General conclusion

This final year project represents far more than a mere academic requirement, it stands as the culmination of five years of rigorous education in civil engineering, marking the symbolic transition from the student to professional engineer. Through the comprehensive study of an R+8 residential building, we have successfully bridged the gap between theoretical knowledge and the complex realities of the structural analysis and seismic design.

One of the significant challenges of this work lay in mastering advanced computational tools. The decision to utilize ETABS 2021 proved to be highly effective, allowing us to accurately model the dynamic behaviour of the structure under combined vertical and horizontal loads. The modelling of the shear walls, columns, and beams was carried out with strict adherence to the RPA 2024 regulation a modern code that imposes stringent requirement regarding ductility and rigidity, surpassing earlier versions.

Our mythological approach followed an unwavering logic: from preliminary sizing to foundation design. The analysis of the obtained results reveals three fundamental conclusions:

1-Safety as the absolute priority: the dimensioning of all primary load-bearing elements (columns, shear walls, and the general mat foundation) was driven by the ultimate objective of preserving human life. All regulatory checks (BAEL 91, CBA 93, and RPA 2024) were rigorously satisfied, ensuring a ductile and stable structural response in the event of a major seismic event.

2-the rigor of seismic analysis: the strategic placement of the shear walls constituted a key technical challenge. Several configurations were tested, however, only the final selected layout allowed for the optimization of the building's natural period and the minimization of inter-story drift. This confirms that a mixed structural system (moment resisting frames combined with shear walls) is the most appropriate solution for an R+8 building located in a seismic zone.

3-economic rationality: the optimization of the steel reinforcement detailing and the selection of a general mat foundation (over isolated footings) demonstrated that a safe design does not necessarily equate to excessive costs. The delicate balance between mechanical performance and economic feasibility was maintained throughout the dimensioning process.

In essence, this project has enabled us to master the entire design process from load estimation and dynamic analysis to foundation verification. It demonstrates that the modern civil engineer must not only possess the ability to perform calculation but also the foresight to anticipate potential failure and innovate in the selection of structural systems.

We sincerely hope that this work will serve as a reliable technical reference and a source of inspiration for future cohorts of civil engineering students. This is not an end, but rather a promising beginning to our professional careers. May this dissertation contribute to the construction of a more resilient national infrastructure, capable of withstanding seismic hazard

Appendices

Section Partiellement Comprimée (SPC)	
Section Rectangulaire à l'ELU	Section en « Tè » à l'ELU
- N_u (traction) et C à l'extérieur de la zone entre A et A' : - N_u (compression) et C en dehors de la section : - N_u (compression) et C à l'intérieur de la section, avec : $N_u(d-d') - M_{u1} \leq (0,337 \times h - 0,81 \times d')b \times h \times f_{cu}$ Le calcul se fait par assimilation à la F.S sous M_{u1} avec : $M_{u1} = N_u \times e_s = M_{u0} + N_u \left(d - \frac{h}{2} \right)$ N_u pris avec son signe : Compressions (+) / Traction (-)	
$\mu_{br} \leq \mu_l$ $\Rightarrow A'_1 = 0 \Rightarrow A' = 0$ $A_1 = \frac{M_{u1}}{z \times f_c}$ $\mu_{br} = \frac{M_{u1}}{b \times d^2 \times f_{cu}}$ On revient à la F.C : $A = A_1 = \frac{N_u}{f_c}$	$\mu_{br} > \mu_l \Rightarrow A'_1 \neq 0 \Rightarrow A' \neq 0$ $A'_1 = A' = \frac{M_{u1} - M_{l1}}{(d-d')f_c}$ $A = \frac{1}{f_c} \left(\frac{M_{u1} - A'_1 f_c (d-d')}{d(1-0,4\alpha)} + A'_1 f_c - N_u \right)$ $(N_u \text{ est pris avec son signe})$
- N_u (traction) et C à l'intérieur de la section, avec : $N_u(d-d') - M_{u1} \leq (0,337 \times h - 0,81 \times d')b_0 \times h \times f_{cu}$ $\begin{cases} N_u = N_u - (b-b_0)b_0 \times f_{cu} \\ M_{u1} = M_{u1} - (b-b_0)b_0 \times f_{cu} \left(d - \frac{b_0}{2} \right) \\ M_{u1} = N_u \times e_s = M_{u0} + N_u(d-y_0) \\ M_{u1} = b \times b_0 \times f_{cu} \left(d - \frac{b_0}{2} \right) \end{cases}$ - SI : $M_{u1} \leq M_{l1} \Rightarrow$ Calcul d'une section rectangulaire ($b \times b$) - SI : $M_{u1} > M_{l1} \Rightarrow$ Calcul d'une section en Tè. Ce calcul revient à celui d'une section ($b_0 \times b$) soumise à N_u et M_u $A' = A'_1$ et $A = A_1 - \frac{N_u}{f_c}$ (N_u pris avec son signe) SI : $A' = 0 \Rightarrow A = A_1 - \frac{N_u}{f_c} = \frac{M_{u1}}{d(1-0,4\alpha)f_c} - \frac{N_u}{f_c}$	
Condition de non fragilité : $A \geq A_{min} = 0,23 \times b \times d (f_{ct}/f_c)$	
Calcul à l'ELS	
- N_{ue} (traction) et C hors de la zone entre A et A' : ($e > 0$ et $y_c < 0$) - N_{ue} (compression) et C hors de la section : ($e < 0$ et $y_c > 0$) : - N_{ue} (compression) et C à l'intérieur de la section mais en dehors du noyau central : $e_0 = M_{ue}/N_{ue} > h/6$: ($e > 0$ et $y_c > 0$). Vérifier : $\sigma_{ue} = \frac{N_{ue}}{\mu_i} \times y \leq \overline{\sigma_{ue}}$ (N_{ue} avec son signe) $\sigma_{ue} = 15 \times \frac{N_{ue}}{\mu_i} (y-d') \leq \overline{\sigma_{ue}}$ SI : $A' \neq 0$ $\sigma_{ue} = 15 \times \frac{N_{ue}}{\mu_i} (d-y) \leq \overline{\sigma_{ue}}$	
$\mu_i = \frac{b}{2} y^2 + 15 d' (y-d') - 15 A (d-y)$ Calcul de y_c : ($y=y_c+e$) $\left\{ \begin{array}{l} N \text{ (Traction)} \Rightarrow e = e_0 + (h/2) \text{ avec : } e > 0 \text{ et } y_c < 0 \\ N \text{ (Compression)} \Rightarrow e = e_0 - (h/2) \text{ avec : } e < 0 \text{ et } y_c > 0 \end{array} \right.$ $y_c^2 + p y_c + q = 0$ $\begin{cases} p = -3e^2 - 90 \frac{d'}{b} (e-d') + 90 \frac{d}{b} (d-e) \\ q = -2e^3 - 90 \frac{d'}{b} (e-d')^2 - 90 \frac{d}{b} (d-e)^2 \end{cases}$ $\begin{cases} -e \leq y_c \leq h-e & \text{si } e > 0 \\ +e \leq y_c \leq h+e & \text{si } e < 0 \end{cases}$ SI : $\sigma_y > \overline{\sigma_y}$, on recalcule à l'ELS : $A_{ue1} = \frac{M_{ue1}}{d \left(1 - \frac{\alpha}{3} \right) \sigma_y}$ et $M_{ue1} = M_{ue0} + N_{ue} \left(d - \frac{h}{2} \right)$ On revient à la F.C avec : $A_{ue} = A_{ue1} - \frac{N_{ue}}{\sigma_y}$	<u>Position de l'axe neutre</u> $\begin{cases} E_1 = (b-b_0)(3e-2b_0)b_0^2 + 90 \left[A'(e-d')(d'-A(d-e)d) \right] \\ E_2 = 15b_0^2(b_0-3e) + 90 \left[A'(e-d')(d'-b_0) - A(d-e)(d-b_0) \right] \end{cases}$ - SI E_1 et E_2 sont de signes contraires $\Rightarrow$ Calcul d'une section ($b \times b$) - SI E_1 et E_2 sont de même signe $\Rightarrow$ Calcul d'une section en Tè $\mu_i = \frac{b}{2} y^2 - \frac{1}{2} (b-b_0)(y-b_0)^2 + 15 A' (y-d') - 15 A (d-y)$ Calcul de y_c : ($y=y_c+e$) $e = e_0 - y_{c1}$ (N_{ue} = compression) et $e = e_0 + y_{c1}$ (N_{ue} = traction) $y_c^2 + p y_c + q = 0$ $\begin{cases} p = -3 \frac{b}{b_0} e^2 + 3 \left(\frac{b}{b_0} - 1 \right) (e-b_0)^2 - 90 \frac{d'}{b_0} (e-d') + 90 \frac{d}{b_0} (d-e) \\ q = -2 \frac{b}{b_0} e^3 + 2 \left(\frac{b}{b_0} - 1 \right) (e-b_0)^3 - 90 \frac{d'}{b_0} (e-d')^2 - 90 \frac{d}{b_0} (d-e)^2 \end{cases}$ SI : $\sigma_y > \overline{\sigma_y}$, on recalcule à l'ELS : $A_{ue1} = \frac{M_{ue1}}{z \times \sigma_y}$ $M_{ue1} = M_{ue0} + N_{ue}(d-y_{c1})$ et en F.C : $A_{ue} = A_{ue1} - \frac{N_{ue}}{\sigma_y}$

Section Entièrement Tendue (SET)	
Calcul à l'ELU	
Condition	- N_u est un effort de traction ; - Le centre de pression « C » ($e_0 = \frac{M_{ed}}{N_u}$) est entre les armatures (à l'intérieur de la section).
Calcul des armatures	$\begin{cases} A_1 = \frac{N_u \times e_2}{f_{t10}(d-d')} \\ A_2 = \frac{N_u \times e_1}{f_{t10}(d-d')} \end{cases} \quad \text{Avec : } \begin{cases} f_{t10} = \frac{f_c}{\gamma_s} \\ e_1 = \left(\frac{h}{2} - d'\right) + e_0 \\ e_2 = (d - d') - e_1 \end{cases} \quad \text{Et : } A_{\min} = \frac{B \times f_{t21}}{f_c}$ Si : A_1 et $A_2 \geq A_{\min} \Rightarrow$ Ferrailage avec A_1 et A_2 Si : A_1 et $A_2 < A_{\min} \Rightarrow$ Ferrailage avec $A_{\min}$
Cas d'armatures symétriques	$e_1 = e_2 \Rightarrow A_1 = A_2 = \max \left(\frac{N_u}{2f_{t10}} ; \frac{B \times f_{t21}}{f_c} \right)$ Si A_1 et A_2 sont connues, il faut vérifier que : $N_u \leq \frac{A_1 \times f_{t10}(d-d')}{e_2} \quad \text{et} \quad N_u \leq \frac{A_2 \times f_{t10}(d-d')}{e_1}$
Calcul à l'ELS	
Condition	- N_{ser} est un effort de traction ; - Le centre de pression « C » ($e_0 = \frac{M_{ser}}{N_{ser}}$) est entre les armatures (à l'intérieur de la section).
Calcul des armatures	$\begin{cases} A_1 = \frac{N_{ser}}{\sigma_s} \left(\frac{v_2}{v_1 + v_2} \right) \\ A_2 = \frac{N_{ser}}{\sigma_s} \left(\frac{v_1}{v_1 + v_2} \right) \end{cases} \quad \text{Avec : } \begin{cases} v_1 = \frac{h}{2} - d' \\ v_2 = d - v_1 - d' = \frac{h}{2} - d'' \end{cases}$ Les armatures tendues doivent aussi vérifier la CNF : A_1 et $A_2 \geq A_{\min} = \frac{B \times f_{t21}}{f_c}$
Cas d'armatures symétriques	$v_1 = v_2 \Rightarrow A_1 = A_2 = \max \left(\frac{N_{ser}}{2\sigma_s} ; \frac{B \times f_{t21}}{f_c} \right)$
Vérification des contraintes	$\begin{cases} \sigma_{s1} = \frac{N_{ser}}{A_1 + A_2} + \frac{M_{ser}}{A_1(v_1 + v_2)} < 0 \\ \sigma_{s1} = \frac{N_{ser}}{A_1 + A_2} - \frac{M_{ser}}{A_2(v_1 + v_2)} < 0 \end{cases} \quad (N_{ser} \text{ pris avec son signe})$

Section Entièrement Comprimée (SEC)	
Section Rectangulaire – Calcul à l'ELU	
N_v (Pris avec son signe) est un effort de compression et le centre de pression "C" est à l'intérieur de la section, avec :	
$N_u(d-d')-M_{ed} \geq (0,337 \times h - 0,81 \times d')b \times h \times f_{bc}$ avec : $M_{ed} = M_{ui} + N_u \left(d - \frac{h}{2} \right)$	
Si : $N_u(d-d')-M_{ed} < (0,5 \times h - d')(b \times h \times f_{bc})$ $\rightarrow A=0$	Si : $N_u(d-d')-M_{ed} \geq (0,5 \times h - d')(b \times h \times f_{bc})$ $\rightarrow A \neq 0$
$A' = \frac{N_u - (\psi \times b \times h \times f_{bc})}{f_{td}}$ $\psi = \frac{0,357 + \left(\frac{N_u(d-d')-M_{ed}}{b \times h^2 \times f_{bc}} \right)}{0,857 - \frac{d'}{h}}$ $\epsilon'_i = 2\text{‰} \left(1 + \left(1,719 - 4,010 \frac{d'}{h} \right) \sqrt{1-\psi} \right)$	$A' = \frac{M_{ed} - (b \times h \times f_{bc})(d - 0,5 \times h)}{(d-d')f_{ctm}}$ $A = \frac{N_u - (b \times h \times f_{bc})}{f_{ctm}} - A'$ $\psi = 1$ et $\begin{cases} \epsilon_i < 2\text{‰} \Rightarrow f_{ctm} = \frac{f_{ct}}{\gamma_c} \\ \epsilon_i \geq 2\text{‰} \Rightarrow f_{ctm} = 2\text{‰} \times E_s \end{cases}$
Calcul à l'ELS	
N_{se} (Pris avec son signe) est un effort de compression et "C" à l'intérieur de la section, avec $e_0 < h/6$	
$\sigma_{s1} = \frac{N_{se}}{S} + \frac{M_{se}}{I_{se}} v \leq \overline{\sigma}_s$ $\sigma_{s2} = \frac{N_{se}}{S} - \frac{M_{se}}{I_{se}} v' \geq 0$ $\sigma_c = 15 \left[\frac{N_{se}}{S} + \frac{M_{se}}{I_{se}} (v-d') \right] \leq \overline{\sigma}_c$ $\sigma_c = 15 \left[\frac{N_{se}}{S} - \frac{M_{se}}{I_{se}} (d-v) \right] \leq \overline{\sigma}_c$	Avec : $M_{se} = M_{ui} - N_{se} \left(\frac{h}{2} - v \right)$ $I_{se} = \frac{b}{3} (v^3 + v'^3) + 15(A(d-v)^2 + A'(v-d')^2)$ $S = (b \times h) + 15(A + A')$ $v = \frac{\frac{b \times h^2}{2} + 15(A \times d + A' \times d')}{(b \times h) + 15(A + A')}$ $v' = h - v$
Section en Tê – Calcul à l'ELU	
N_v (Pris avec son signe) est un effort de compression et le centre de pression "C" est à l'intérieur de la section, avec :	
$N_u(d-d')-M_{ed} \geq (0,337 \times h - 0,81 \times d')b_0 \times h \times f_{bc}$	
avec : $N_u = N_u - (b-b_0)h_0 \times f_{bc}$; $M_{ed} = M_{ui} - (b-b_0)h_0 \times f_{bc} \left(d - \frac{h_0}{2} \right)$ et $M_{ed} = M_{ui} + N_u(d-y_0)$	
Le calcul des armatures d'une section en Tê revient à déterminer celles d'une section ($b_0 \times h$) soumise à N_{se} et M_{se}	
Si : $N_u(d-d')-M_{ed} \leq (0,5 \times h - d')(b_0 \times h \times f_{bc})$ $\rightarrow A=0$	Si : $N_u(d-d')-M_{ed} \geq (0,5 \times h - d')(b_0 \times h \times f_{bc})$ $\rightarrow A \neq 0$
$A' = \frac{N_u - (\psi \times b_0 \times h \times f_{bc})}{f_{td}}$ $\psi = \frac{0,357 + \left(\frac{N_u(d-d')-M_{ed}}{b_0 \times h^2 \times f_{bc}} \right)}{0,857 - \frac{d'}{h}}$ $\epsilon'_i = 2\text{‰} \left(1 + \left(1,719 - 4,010 \frac{d'}{h} \right) \sqrt{1-\psi} \right)$	$A' = \frac{M_{ed} - (b_0 \times h \times f_{bc})(d - 0,5 \times h)}{(d-d')f_{ctm}}$ $A = \frac{N_u - (b_0 \times h \times f_{bc})}{f_{ctm}} - A'$ $\psi = 1$ et $\begin{cases} \epsilon_i < 2\text{‰} \Rightarrow f_{ctm} = \frac{f_{ct}}{\gamma_c} \\ \epsilon_i \geq 2\text{‰} \Rightarrow f_{ctm} = 2\text{‰} \times E_s \end{cases}$

Section en « Tê »		
Calcul à l'ELU		
$M_{Te} = b \times b_0 \times f_{bu} \left(d - \frac{b_0}{2} \right) \quad \text{avec} \quad f_{bu} = \frac{0,85 \times f_{c28}}{\gamma_k} ; \begin{cases} \gamma_k = 1,50 \rightarrow \text{situation courante} \\ \gamma_k = 1,35 \rightarrow \text{situation accidentelle} \end{cases}$		
Si : $M_{Te} \geq M_e \Rightarrow$ Calcul d'une section rectangulaire ($b \times h$)		
Si : $M_{Te} < M_e \Rightarrow$ Calcul d'une section en T		
$M_1^* = M_e - M_2^*$	$M_2^* = \frac{b-b_0}{b} M_{2e}$	$\mu_{bal} = \frac{M_1^*}{b_0 \times d^2 \times f_{bu}} \quad z_j = d(1 - 0,4\alpha_1)$
Calcul de la section ($b_0 \times h$)		Calcul des débords
Si : $\mu_{bal} < \mu_l \rightarrow A' \neq 0$	Si : $\mu_{bal} > \mu_l \rightarrow A' = 0$	
$A_1 = \frac{M_1^*}{d(1 - 0,4\alpha_1)f_{ce}}$	- Si : $z_j > d - \frac{b_0}{2} \Rightarrow$ Calcul d'une section ($b \times h$) avec d' - Si : $z_j < d - \frac{b_0}{2} \Rightarrow$ Calcul d'une section en Tê avec d'	$A_2 = \frac{M_2^*}{\left(d - \frac{b_0}{2} \right) f_{ce}}$
$\alpha = 1,25 \left(1 - \sqrt{1 - 2\mu_{bal}} \right)$	$d' = \frac{M_e - M_1^* - M_2^*}{(d - d'')f_{ce}}$ $A_3 = \left(\frac{M_1^*}{z_j} + \frac{M_e - M_1^* - M_2^*}{d - d'} \right) \frac{1}{f_{ce}}$	
Condition de non fragilité : $A + A_1 + A_2 \geq A_{min} = 0,23 \times b_0 \times d \times \frac{f_{c28}}{f_t}$		
Vérification à l'ELS		
Localisation de l'axe neutre : $H = \frac{b \times h_0^3}{2} - 15A(d - b_0) \quad$ Si : $\begin{cases} H > 0 \Rightarrow \text{Vérification pour une section } (b \times h) \\ H < 0 \Rightarrow \text{Vérification pour une section en T} \end{cases}$		
Vérification des contraintes pour une section en T		Calcul des armatures si : $\sigma_{st} < \overline{\sigma_{st}}$
$\begin{cases} \sigma_{st} = \frac{M_{2e}}{I} \times y \leq \overline{\sigma_{st}} = 0,6 \times f_{ctk} \\ \sigma_{sc} = 15 \times \sigma_{st} \left(\frac{d-y}{y} \right) = 15 \times \frac{M_{2e}}{I} (d-y) \leq \overline{\sigma_{sc}} \end{cases}$ - FN : $\overline{\sigma_{st}} = \min \left[\frac{2}{3} f_{te} ; 110 \sqrt{\eta \times f_{te}} \right]$ - FTN : $\overline{\sigma_{st}} = \min \left[\frac{1}{2} f_{te} ; 90 \sqrt{\eta \times f_{te}} \right]$ - Calcul de y et I $\frac{b_0}{2} y^2 + (15A + 15A' + (b-b_0)b_0)y = 15(A'd + A'd') = (b-b_0) \frac{h_0^2}{2} = 0$ $I = \frac{b}{3} y^3 + (b-b_0) \frac{(y-b_0)^3}{3} + 15A(d-y)^3 + 15A'(d'-y)^3$		Remarque : Si $\sigma_{sc} > \overline{\sigma_{sc}}$, la section des armatures tendue (A) doit être recalculer à l'ELS en utilisant la méthode approchée ci-dessous : $A_{2e} = \frac{M_{2e}}{l \times \sigma_{st}}$ Avec : $z = d - \frac{b_0}{2} \quad \text{ou bien} \quad z = 0,93 \times d$

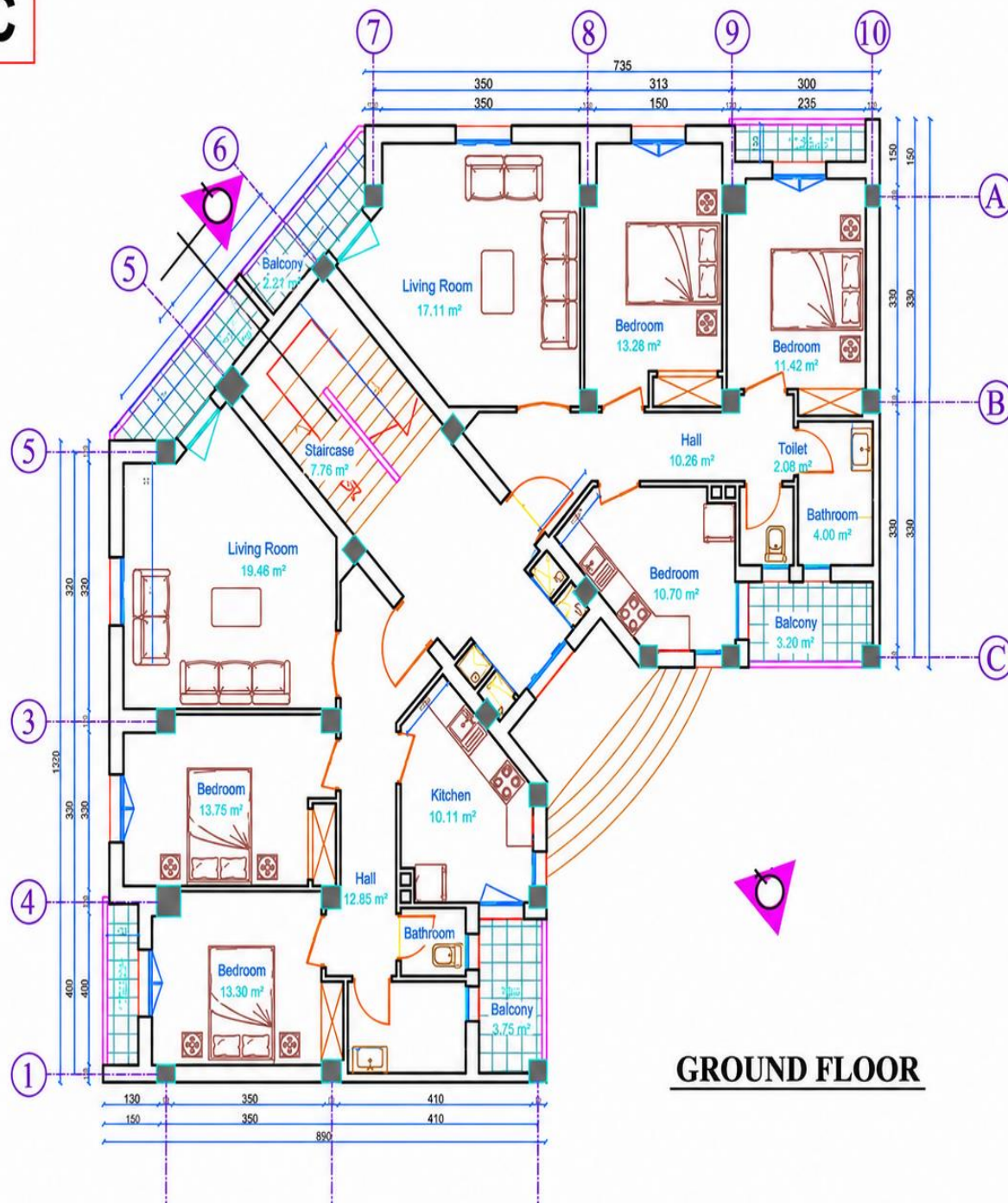
Section Rectangulaire			
Calcul à l'ELU			
$\mu_{bs} = \frac{M_s}{b \times d^2 \times f_{bs}} \quad \text{avec} \quad f_{bs} = \frac{0,85 \times f_{ctk}}{f_k} ; \begin{cases} f_k = 1,50 \rightarrow \text{situation courante} \\ f_k = 1,15 \rightarrow \text{situation accidentelle} \end{cases}$			
$\mu_s = 0,5\alpha_1(1-0,4\alpha_1) \quad \text{avec} \quad \alpha_1 = \frac{3,5}{3,5+(1000 \times \varepsilon_1)} \quad \text{et} \quad \varepsilon_1 = \frac{f_s}{f_k \times E_s} ; \begin{cases} f_k = 1,15 \rightarrow \text{situation courante} \\ f_k = 1 \rightarrow \text{situation accidentelle} \end{cases}$			
Si : $\mu_{bs} > 0,186 \Rightarrow$ Poutre à B s		Si : $\mu_{bs} < 0,186 \Rightarrow$ Poutre à A s	
Si : $\mu_{bs} < \mu_1 \rightarrow A' \neq 0$	Si : $\mu_{bs} > \mu_1 \rightarrow A' = 0$	Si : $\mu_{bs} < \mu_1 \rightarrow A' \neq 0$	Si : $\mu_{bs} > \mu_1 \rightarrow A' = 0$
$A = \frac{M_s}{z \times f_{cs}}$ $\begin{cases} z = d(1-0,4\alpha) \\ \alpha = 1,25(1-\sqrt{1-2\mu_{bs}}) \\ f_{cs} = \frac{f_{ctk}}{f_k} \rightarrow \text{si } \varepsilon_{cs} > \varepsilon_1 \\ f_{cs} = E_s \times \varepsilon_1 \rightarrow \text{sinon} \\ \text{Avec : } \varepsilon_1 = \frac{1,5}{1000} \left(\frac{1-\alpha}{\alpha} \right) \end{cases}$	$A' = \frac{M_s - M_1}{(d-d')f_{cs}}$ $M_1 = \mu_1 \times b \times d^2 \times f_{bs}$ $\begin{cases} f_{cs} = \frac{f_{ctk}}{f_k} \rightarrow \text{si } \varepsilon_{cs} > \varepsilon_1 \\ f_{cs} = E_s \times \varepsilon_1 \rightarrow \text{sinon} \\ \text{Avec : } \varepsilon_1 = \left(\frac{1,5}{1000} + \varepsilon_1 \right) \left(\frac{d-d'}{d} \right) = \varepsilon_1 \end{cases}$ $A = \left(\frac{M_1}{z_1} + \frac{M_s - M_1}{d-d'} \right) \frac{1}{f_{cs}}$ $z_1 = d(1-0,4\alpha_1) \text{ et } f_{cs} = \frac{f_{ctk}}{f_k}$	$A = \frac{M_s}{z \times f_{cs}}$ $\begin{cases} z = d(1-0,4\alpha) \\ \alpha = 1,25(1-\sqrt{1-2\mu_{bs}}) \\ f_{cs} = \frac{f_{ctk}}{f_k} \end{cases}$	$A' = \frac{M_s - M_1}{(d-d')f_{cs}}$ $M_1 = \mu_1 \times b \times d^2 \times f_{bs}$ $f_{cs} = \frac{f_{ctk}}{f_k}$ $A = \left(\frac{M_1}{z_1} + \frac{M_s - M_1}{d-d'} \right) \frac{1}{f_{cs}}$ $z_1 = d(1-0,4\alpha_1)$ $f_{cs} = \frac{f_{ctk}}{f_k}$
Condition de non fragilité : $A_{min} = 0,23 \times b \times d \times \frac{f_{ctk}}{f_s}$		Condition de non fragilité : $A_{min} = 0,23 \times b \times d \times \frac{f_{ctk}}{f_s}$	
Vérification à l'ELS			
Vérification des contraintes		Calcul des armatures si : $\sigma_{st} > \sigma_{st1}$	
$\begin{cases} \sigma_{cs} = \frac{M_{ss}}{I} \times y \leq \overline{\sigma_{cs}} = 0,6 \times f_{ctk} \\ \sigma_{st} = 15 \times \sigma_{cs} \left(\frac{d-y}{y} \right) = 15 \times \frac{M_{ss}}{I} (d-y) \leq \overline{\sigma_{st}} \end{cases}$ - Fissuration visible (FN) $\overline{\sigma_{st}} = \min \left[\frac{2}{3} f_{ctk} ; 110 \sqrt{0,5 \times f_{ctk}} \right]$ - Fissuration très visible (FTS) $\overline{\sigma_{st}} = \min \left[\frac{1}{2} f_{ctk} ; 90 \sqrt{0,5 \times f_{ctk}} \right]$ - Calcul de y et I $\frac{b}{2} y^3 + 15 \times A' (y-d')^2 + 15 \times A (d-y)^2 = 0$ $I = \frac{b}{3} y^3 + 15 \times A' (y-d')^2 + 15 \times A (d-y)^2$		<u>Remarque</u> : Si $\sigma_{st} > \overline{\sigma_{st}}$, la section des armatures tendue (A) doit être recalculée à l'EELS en utilisant la formule : $A_{st} = \frac{M_{ss}}{d \left(1 - \frac{\sigma}{3} \right) \overline{\sigma_{st}}}$ $\begin{cases} \alpha = \sqrt{90 \times \beta \frac{1-\alpha}{3-\alpha}} \rightarrow \alpha \text{ calculé par itération} \\ \beta = \frac{M_{ss}}{b \times d^2 \times \sigma_{st}} \end{cases}$	

	ELU $\nu = 0$		ELS $\nu = 0.2$	
l_x/l_y	μ_x	μ_y	μ_x	μ_y
0.40	0.1101	0.2500	0.1121	0.2854
0.45	0.1036	0.2500	0.1063	0.3234
0.50	0.0966	0.2500	0.1000	0.3671
0.55	0.0894	0.2500	0.0936	0.4150
0.60	0.0822	0.2948	0.0870	0.4672
0.65	0.0751	0.3613	0.0805	0.5235
0.70	0.0684	0.4320	0.0743	0.5817
0.75	0.0621	0.5105	0.0684	0.6447
0.80	0.0561	0.5959	0.0628	0.7111
0.85	0.0506	0.6864	0.0576	0.7794
0.90	0.0456	0.7834	0.0528	0.8502
0.95	0.0410	0.8875	0.0483	0.9236
1.00	0.0368	1.0000	0.0441	1.0000

Section (en cm²) de 1 à 20 armatures “HA” de diamètre “Ø” (en mm)

		Diamètre des barres Ø (en mm)										
		5	6	8	10	12	14	16	20	25	32	40
Nombre de barres	1	0.20	0.28	0.50	0.79	1.13	1.54	2.01	3.14	4.91	8.04	12.57
	2	0.39	0.57	1.01	1.57	2.26	3.08	4.02	6.28	9.82	16.08	25.13
	3	0.59	0.85	1.51	2.36	3.39	4.62	6.03	9.42	14.73	24.13	37.70
	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	7.70	10.05	15.71	24.54	40.21	62.83
	6	1.18	1.70	3.02	4.71	6.79	9.24	12.06	18.85	29.45	48.25	75.40
	7	1.37	1.98	3.52	5.50	7.92	10.78	14.07	21.99	34.36	56.30	87.96
	8	1.57	2.26	4.02	6.28	9.05	12.31	16.08	25.13	39.27	64.34	100.53
	9	1.77	2.54	4.52	7.07	10.18	13.85	18.10	28.27	44.18	72.38	113.10
	10	1.96	2.83	5.03	7.85	11.31	15.39	20.11	31.42	49.04	80.42	125.66
	11	2.16	3.11	5.53	8.64	12.44	16.93	22.12	34.56	54.00	88.47	138.23
	12	2.36	3.39	6.03	9.42	13.57	18.47	24.13	37.70	58.91	96.51	150.80
	13	2.55	3.68	6.53	10.21	14.70	20.01	26.14	40.84	63.81	104.55	163.36
	14	2.75	3.96	7.04	11.00	15.83	21.55	28.15	43.98	68.72	112.59	175.93
	15	2.95	4.24	7.54	11.78	16.96	23.09	30.16	47.12	73.63	120.64	188.50
	16	3.14	4.52	8.04	12.57	18.10	24.63	32.17	50.27	78.54	128.68	201.06
	17	3.34	4.81	8.55	13.35	19.23	26.17	34.18	53.41	83.45	136.72	213.63
	18	3.53	5.09	9.05	14.14	20.36	27.71	36.19	56.55	88.36	144.76	226.20
	19	3.73	5.37	9.55	14.92	21.49	29.55	38.20	59.69	93.27	152.81	238.76
	20	3.93	5.65	10.05	15.71	22.62	30.79	40.21	62.83	98.17	160.85	251.33

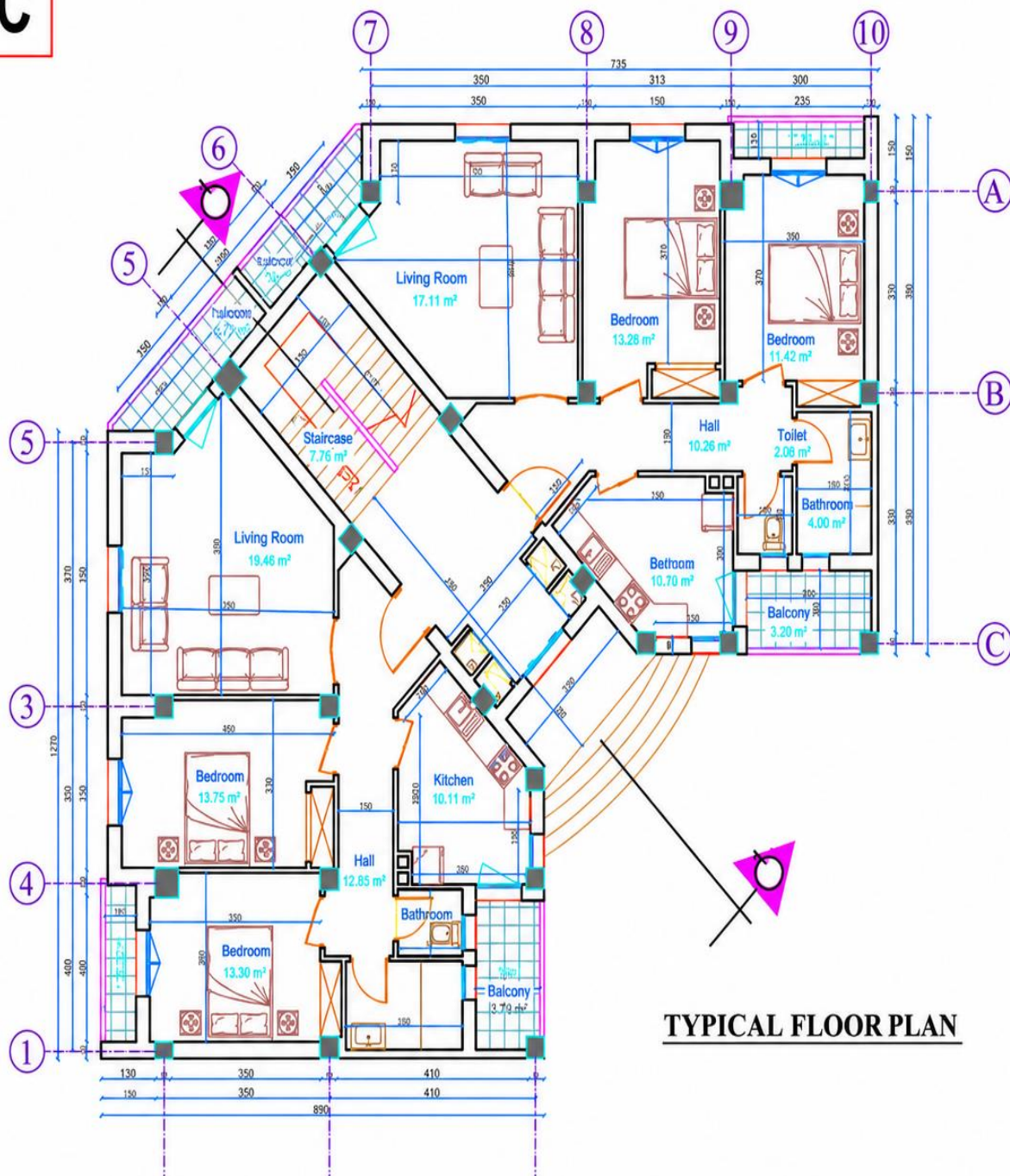
BLOCK C



GROUND FLOOR

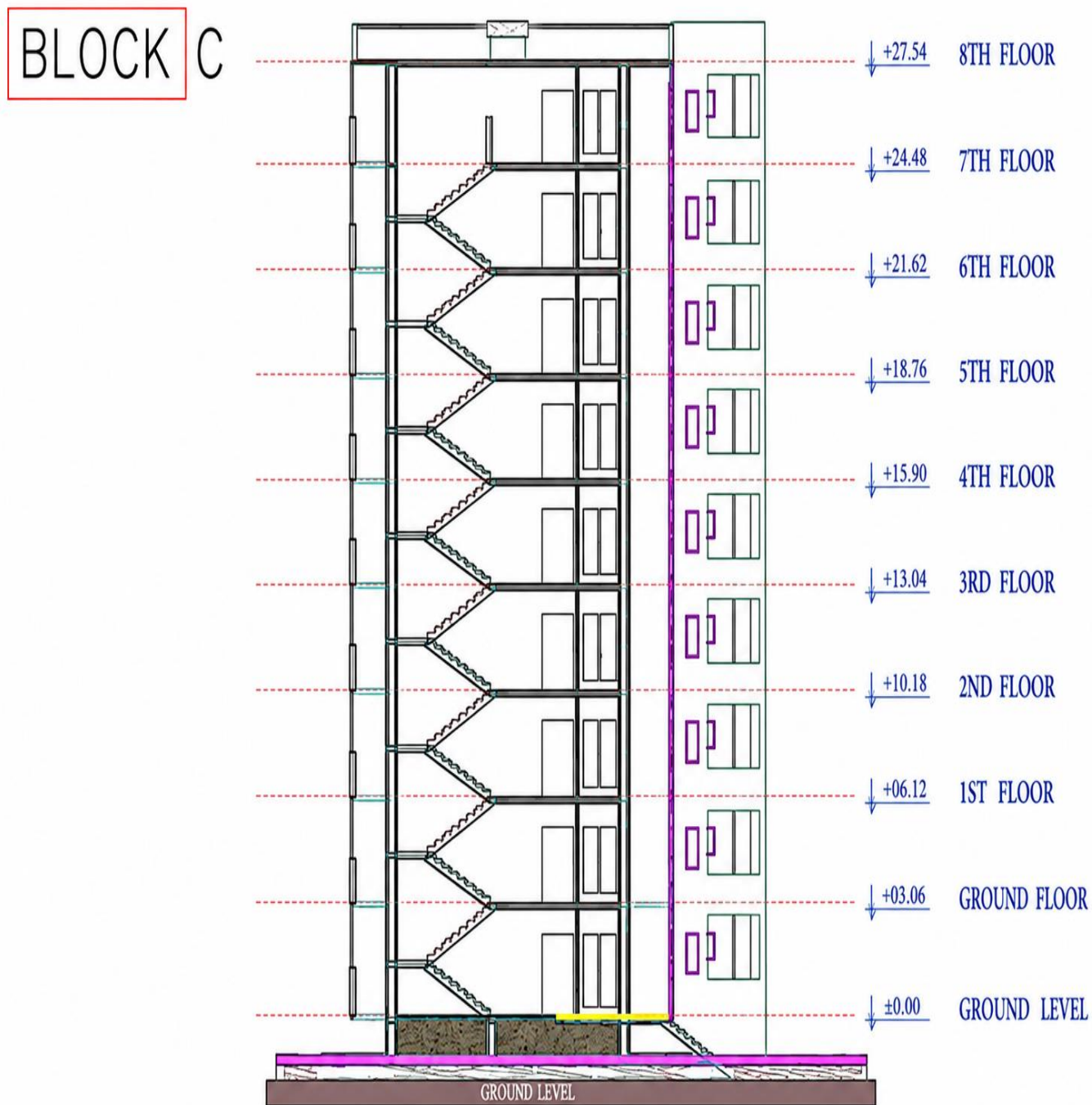
Plan view of ground floor

BLOCK C



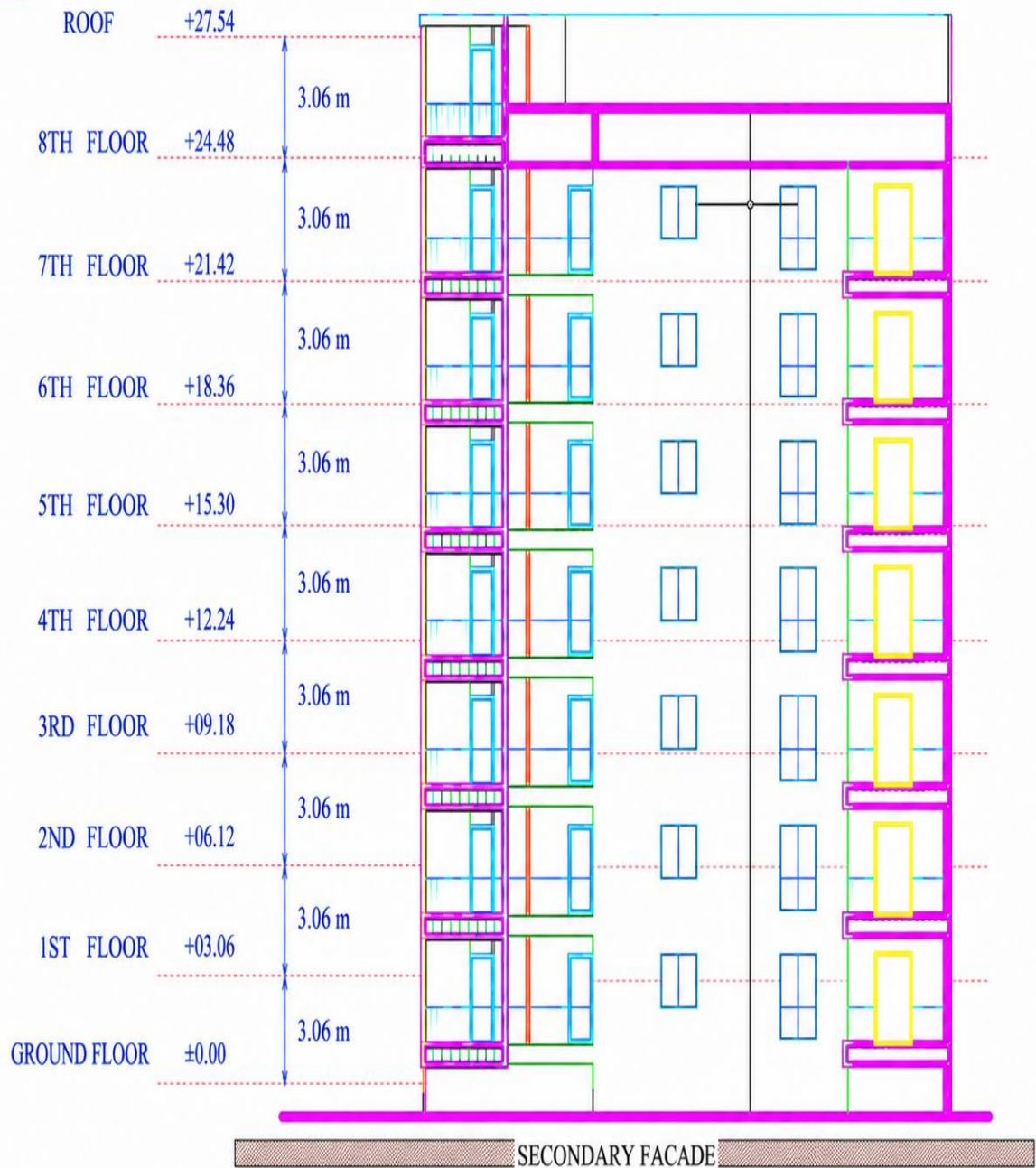
TYPICAL FLOOR PLAN

Plan view of typical floor



Cross-section view of the building

BLOCK C



Structural elevations of the building

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