REPUBLIC OF ALGERIA DEMOCRATIC AND PEOPLAR

Ministry of Higher Education and Scientific Research

KASDI Merbah University of OUARGLA

FACULTY OF SCIENCE APPLIED

CIVIL ENGINEERING AND HYDRAULICS DEPARTMENT

Master's thesis

Sector: Engineering Civil

Option: Structures

Theme

Study of a residential building (5 floors) with a mixed bracing system (frames + sails), located in Annaba, a medium seismic zone.

Presented by :

- KOUDIA Belkis
- KOUIRI Rayane

This dissertation was publicly defended on June 15, 2025, before a jury composed of:

Ms BENZRRARI Salma	MAA	President
Mr KHELLASSI Omar	MAA	Examiner
Mr MOKHTARI Abdessamed	MCA	Supervisor

PROMOTION: 2024-2025

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Thanks

We would like to thank above all Allah we have kept in good health in order to lead to well this project of end of study. We thank also our families for the sacrifices they have made for us to finish our studies.

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We thank also all our friends for their support, their patience, their understanding and encouragement,

الاهداء

الحمد لله الذي بنعمته تتمّ الصالحات، وبفضله تكتمل المساعي والغايات،
حمداً طيباً مباركاً فيه كما ينبغي لجلال وجهه وعظيم سلطانه.
إلى نفسي أولًا...

إلى تلك التي صبرت، وتحملت، وثابرت رغم التعب والشكوك،

إلى قلبي الذي طرق باب تخصص جديد بكل شجاعة...

كل الفخر لكِ أيتها النفس، فقد كنتِ أهلًا للتحدي وأهلًا للوصول.

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يا من حملتني بصبرك ودعائك إلى حيث أنا اليوم... لكِ كل الحب والعرفان.

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شكرًا لكم من القلب، فكل قطرة جهد في هذا العمل تحمل بصمتكم.

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أنتم الإلهام الحقيقي...

أنتم الدافع للاستمرار والتعلّم والعطاء...

أهديكم هذا العمل، آملة أن يكون لكم قدوة، وأن تروا فيه انعكاسًا لكل حلم يمكن تحقيقه بالصبر والإيمان.

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....

(وَآخِرُ دَعْوَاهُمْ أَنِ الْحَمْدُ لِلَّهِ رَبِّ الْعَالَمِينَ)

أهدي هذا النجاح الى نفسي الطموحة جدا لقد ظننت انني لا أستطيع ولكن من قال إنها لها نالها و انها ابت رغما عنها اثبت بها وها انا اليوم اختم بحث تخرجي بكل همة ونشاط فالحمد الله اللهم لا تجعله آخر عهدي من العلم واجعلها خير بداية لطريق أعظم اللهم بارك لنا في عملنا وانفعنا بما علمتنا.

وأهدي ثمرة جهدي ودراستي وفرحتي المنتظرة الى نبراس أيامي ووهج حياتي الى التي ظلت دعواتها تضم اسمي دائما الى من افنت عمرها في سبيل ان احقق طموحي واحلق في أعالي الراتب (ماما الغالية حفظها الله).

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الى من ساندوني و كانوا عونا لي في مسيرتي .... الى اساتذتي الكرام شكرا لعلمكم وتوجيهكم ودعمكم المستمر

وإلى ملهمي صناع قوتي صفوة أيامي سلوة اوقاتي الي الشموع التي تنير لي الطريق دوما (اخواتي وصديقاتي).

لكل من كان عونا وسندا في هذا الطريق .... اصحاب الشدائد والأزمات

"ما سلكنا البدايات الا بتيسيره وما بلغنا النهايات الا بتوفيقه وما حققنا الغايات الا بفضله فالحمد لله"

فجزاكم الله خير واليكم خير الجزاء

<u>ريان</u>

Abstract:

This project studies a building in (G+4) for residential use; it is located in the municipality of ANNABA, which is classified in seismic zone IIa according to RPA 99 version 2003. This work is a mixed structure (gantry + sails) and the pre-sizing of the load-bearing elements was done in accordance with BAEL91 modified 99 and RPA99/2003 version. And the seismic analysis of the structure was carried out by the Robot Structural Analysis 2021 software. The drawing was carried out by the AUTOCAD 2021 software.

Keywords: Building, Concrete, ROBOT 2021, RPA99 V2003, BAEL91.

Résumé:

Le présent mémoire étudie un bâtiment en (R+4) à usage d'habitation, implanté à la commune d' ANNABA qui est classée en zone sismique IIa selon RPA 99 version 2003. Cet ouvrage est une structure mixte (portique + voiles) et le prédimensionnement des éléments porteurs a été fait conformément au BAEL91 modifié 99 et RPA99/version 2003. Et l'analyse sismique de la structure a été réalisée par le logiciel Robot Structural Analysis 2021. Le dessin a été réalisé par le logiciel AUTOCAD 2021.

Mots clés : Bâtiment, Béton, ROBOT 2021, RPA99 V2003, BAEL91.

ملخص:

هذا المشروع عبارة عن دراسة تقنية لإنجاز بناية سكنية ذات (طابق أرضي + أربع طوابق) الموجودة بولاية عنابة المصنفة ضمن المنطقة الزلزالية صنف **(2 أ)** حسب القواعد الجزائرية المقاومة للزلزال (RPA99/2003).

هذا العمل عبارة عن هيكل مختلط (أطر + جدران) وتم إجراء التحجيم المسبق للعناصر الحاملة وفقًا لـ BAEL91 المعدل 99 و (RPA99/2003). كما استخدمنا برنامج ROBOT2021 لتحديد السلوك الديناميكي للهيكل أما عملية الرسم فتم بواسطة برنامج .AUTOCAD 2021.

الكلمات المفتاحية: العمارة، الخرسانة، BAEL91, RPA 99 v2003, ROBOT2021

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List of symbols

A : coefficient of acceleration of the box

Aa : section reinforcement in support

Has_{the} : section of the frame longitudinal

A_{min} : section reinforcement minimum determined by the regulations

Ar : section reinforcement of distribution

A_{ser} : section of armature state limit of service

At : section reinforcement of span or cross-sectional

A_u : section reinforcement of state ultimate limit of resistance

Ax : section of the armature in the direction x-x **Ay** : section of the armature in the direction y-y **A'** : section frame compressed

A_1 : section of the reinforcement, the more tense or the less compressed at

A_2 : section of the frame the less tense or the more compressed

Br : reduced cross-section of the concrete

Cp : factor of force horizontal

Cs : factor of safety

Cr : loads of rupture

Crn : charge breaking minimum necessary

D : coefficient of amplification dynamics

E : modulus of deformation , longitudinal

Eij : module of deformation longitudinal instant

Via : modulus of deformation longitudinal deferred

G : action permanent
H : height

HA : frame with high adhesion

I : moment of inertia

If : moment of inertia fictitious

J : action permanent advent putting in place of walls

L : length

The : length in elevation

Ln : between axis of the ribs **Lp** : length in plan

M : time flexing

My : time flexing in support

Mc : bending moment in the centre of mass of the cabin ; the time of the console

Md : time bending to the right

Me : mass of the cable ; the time in the center of the section

Mf : time flexing total

Mg : when flexing under load permanent ; the mass of the winch ; the time due to the guard body

Mj : when flexing under load permanent before setting in place of walls

M_l : mass linear

M_{ser} : time bending state limit of service

Mt : time flexing of span

M_u : time bending state limit ultimate of resistance

Mw : time bending to the left

Mx : moment of bending in the direction x-x

My : time flexing in the direction y-yM_0 : the time of bending of a beam simply supported

M_1 : time for report to reinforcement strained ; coefficient of Pigeaud

M_2 : coefficient of PigeaudM_{22} : the time following the direction 2-2M_{33} : time according to the meaning 3-3

N : effort normal

Do : effort normal to the center of the section

Mpn : effort normal due to the weight of the beams, the

main **Nps** : effort normal due to the weight of the beams

side - **N_{ser}** : normal stress limit state service

N_u : effort normal state limit ultimate of resistance

P : weight of their own ; perimeter

Pr : weight clean the strike

Q : the action variable of any kind ; factor of quality

R **: radius ; coefficient of behavior of the structure**

S : surface

Sr : surface of the slab

T : effort tranchant

T_x : period fundamental in the direction x-

x **T_y** : period fundamental in the direction

y-y **Cpu** : perimeter of the contour

V : action seismic ; effort horizontal

Vt : earthquake force at the base of the structure

W : total weight of the structure

Wp : weight of the item into consideration.

a : length ; distance ; dimension

b : width

b_0 : width of the rib

b_1 : width of column

c : coating

d : useful height ;

e : eccentricity ; spacing e_a : eccentricity additional **f** : arrow

f_c : constraint characteristic of the concrete in the compression

f_e : elastic limit of steel

f_t : constraint characteristic of the concrete to the tensile

g : lap of the walk

h : height

h_c : height of the body hollow

h_d : height of the slab

h_e : height free

h_{avg} : height average

h_t : height total

h' : height of the box nodal

h_1 : the height of the column

i : radius of gyration

j : number of days

l : length ; distance

l_f : length of buckling

l_x : the small dimension of the panel of

the slab **l_y** : the large dimension of the

panel of the slab **l'** : length of the nodal

l_0 : length free

q_b : load linear induced by the stairs

q_{eq} : load linear equivalent

q_l : load linear

q_{ser} : load linear state limit of service

q_u : load linear state limit ultimate of resistance

q_P : load linear of the bearing

s : spacing

T : spacing ; period

x : x coordinate

y : ordered

y_1 : ordinate of the centre of gravity of the section uniform

Introduction general:

In the framework of this project, we have proceeded to the calculation of a building in reinforced concrete for residential use, located in a middle zone seismicity, with a DRC and 4 floors.

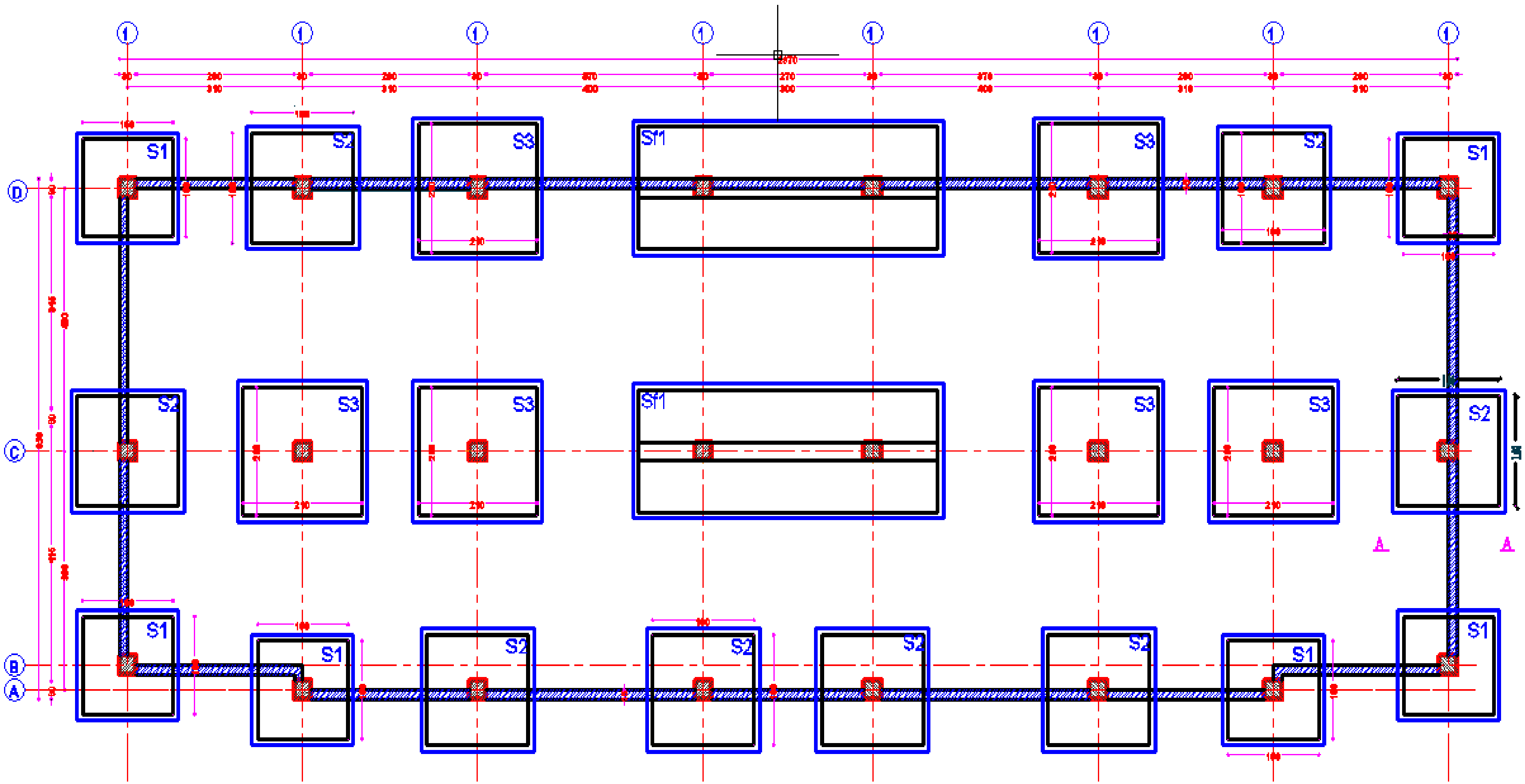
This memory is made up of 6 chapters.

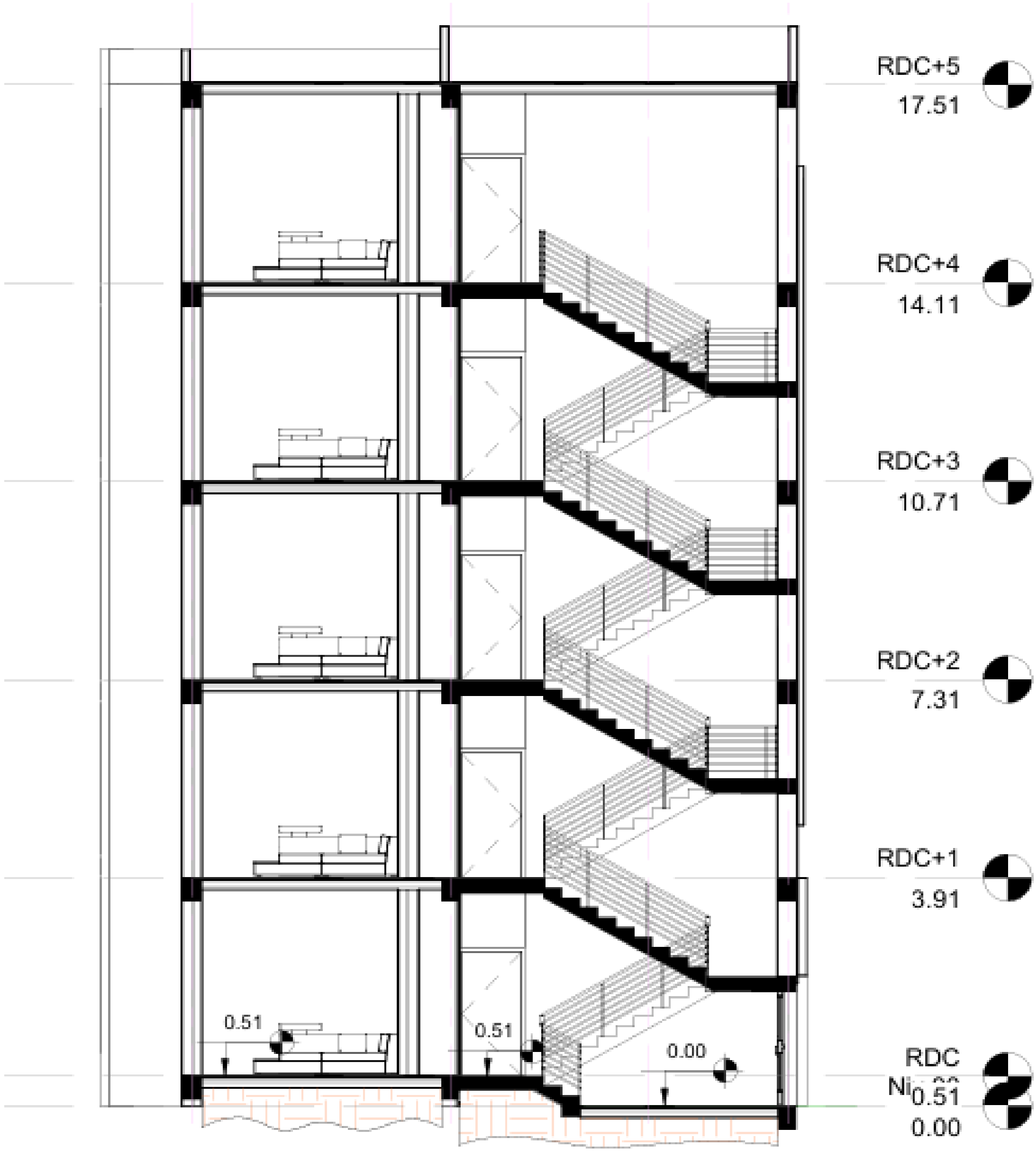
- The First chapter consists in the presentation full of building, the definition of the different elements and the choice of materials to use.
- The second chapter presents the pre - sizing of elements, structural (such as columns, beams and sails).
- The 3rd chapter presents the calculation of the elements non - structural (the acroterion, stairs ......).
- The 4th chapter will focus on the study of the dynamic of the building, the determination of the seismic action and the characteristics of the dynamics of the structure during its vibrations. The study of the building will be made by the analysis of a model of the 3D structure using the calculation software robot structural analysis 2021.
- 5th chapter the calculation of reinforcement of structural elements, based on the results of the software robot structural analysis and presented in the 4th chapter.
- For the last chapter we present the study of the foundations, followed by a general conclusion.

The Plans


21.50
3.10
3.00
3.00
3.00
3.00
3.00
3.10
7.90
4.30
3.30
Chambre 11.20 m²
Chambre 11.31 m²
Sejour 19.07 m²
Hall 10.15 m²
Cuisine 9.90 m²
Rangement 1.01 m²
Sechoir 3.70 m²
WC 1.47 m²
SDB 3.77 m²
GT
F3
UP
Sejour 19.07 m²
Chambre 11.31 m²
Chambre 11.20 m²
Hall 10.15 m²
Rangement 1.01 m²
Cuisine 9.90 m²
Sechoir 3.70 m²
SDB 3.77 m²
WC 1.47 m²
ACCES PRINCIPAL

1 RDC


S1
S2
S3
Sf1
A
B
C
D



RDC+5
17.51
RDC+4
14.11
RDC+3
10.71
RDC+2
7.31
RDC+1
3.91
RDC
0.51
0.00
0.51
0.51
0.00

Chapter I :

Presentation of the structure and Characteristics of materials

I-1-Introduction :

Any study of a project for a building whose structure is in concrete armor has the purpose of ensuring the stability and the resistance of buildings to ensure the safety of the building. We know that the economic development in industrialized countries favours the vertical construction in the interests of the economy of space.

The stability of the structure is a function of the resistance of the different elements of a structural (column, beams, walls,...) to different stresses (compression, flexion, etc.) In which the resistance of these elements is based on the type of materials used and their dimensions and characteristics.

I-2- Therefore, for the calculation of the components of a book, we will follow the rules and procedures known (BAEL91, RPA 99 / version 2003), which is based on the knowledge of materials (concrete and steel) and the sizing and reinforcement elements resistant to the structure.

I-3-Presentation:

The project that we have been studying is a building for residential use (R+4), located in ANNABA, a city located in the middle zone seismic zone II according to the rules of seismic Algerian (RPA 99 / version 2003).

I-4-Purpose :

The good performance of a building depends primarily on the foundations on which it rests. To do this, it is necessary that the soil is well-studied .Seeing that the influence of major resistance and stability of the work, it is the choice of foundations in seismic areas.

I-5-Choice of a structure :

The choice of a construction elevation important is because of the trend that can be explained by the urbanization very dense imposed by population growth, due to theoretical and practical development of the technology of the building.

I-6-Features geometric of the building :

Our project has a form, rectangular, with a setback; the building has the following dimensions:

- Height from floor (h$_e$ = 3.40 m.
- Height of the DRC h_{DRC} = 3.40 m.
- • Height total of building without criterion H = 17.00 m.
- Building width B = 11 m.
- Building length L = 21,5 m.

I-7-Features geometric of the soil :

The ground seat of the construction is a floor cabinet according to the report of the laboratory of soil mechanics

- The stress of the soil is $б_{ground}$ = 2 bars .
- The weight - specific earth γ_h = 1.8 t / m^3.
- The angle of friction internal to the soil φ =30°
- The cohesion C = 0 (ground powder)

I-8-Bone structure and system of constructive adopted :

- **Frame :**

System of bracing joints insured by the sails and porticoes, required by the RPA 99 / version 2003, to ensure the stability of the whole under the effect of the action of vertical and horizontal actions

- **Floors :**

All the floors are made in the body hollow, and a slab of compression type (16+4) cm.

- **Stairs:**

* The building consists of a single type of stairs with two steps and a level of rest.
* The stairs are cast on-site.

- **Masonry :**

* The wall interiors are made of single-partition walls in brick hollow to 10 cm thick.

* The walls of the exterior are carried out in double-wall brick, hollow of (10 cm; 10 cm)
*Separated by a vacuum of 5 cm.

- **Coating :**

*Coated in plaster for the ceiling.
*A coating of cement on the walls outside.
*Coating for tiles on the floors.
*The floor terrace will be covered by a sealing multi-layer waterproof, avoiding the penetration of rainwater.

- **Insulation :**

The insulation acoustic is ensured by the vacuum of body hollow and the mass of the floor; however, the level of wall exterior insulation is ensured by the empty air between the two walls, which is composed of the latter, and by the minimization of the bridges thermal court of achievement. A note that the insulation heat is provided by the layers of cork to the floor terrace.

- **Acrotères** :

The terrace is inaccessible; the last level is surrounded by a criterion in concrete armed with a height equal to 60 cm.

I-9-<u>Feature mechanics of materials :</u>

<u>I-9-1- Concrete :</u>

The concrete is a material formed by the mixture of cement, aggregates (sand, gravel), and water mixed. The reinforced concrete is obtained by introducing in the concrete steel (rebar) arranged so as to balance the effort of traction.
We have used a method of practice so-called <<Method of DREAUX and GORISSE>> to get a normal concrete (D=20mm) with a handled plastic and resistance to compression = 25MPA.
The composition of a metre cube of concrete is the following:

- 350 kg of cement CEM II/ A 42,5
- 400 L of sand $Cg \leq 5$ mm
- 800 L of gravel $Cg \leq 25$ mm
- 175 L of water mixing

<u>I-9-2- Main characteristics and advantages of concrete :</u>

The achievement of an item of work in concrete armor consists of the 4 operations:

A) Execution of a formwork (mold) in wood or in metal.

B) The putting in place of the frames in the formwork.Put in place and «tightening " of the

Concrete in the formwork.

C) Stripping "or release" after hardening enough of the concrete.

The main advantages of concrete reinforced are:

Economics : the concrete is more economical than steel for the transmission of the compression forces, and its association with the frames in steel, it can resist to the efforts of traction.

- **Flexibility of forms :** it is the result of the implementation in the implementation of the concrete in the formwork, which can get all kinds of forms.

- **Resistance to agents air :** it is assured by a coating of correct Frames and a compact design suitable for the concrete.

- **Resistance to fire:** the concrete reinforced resists in good conditions to the effects of fire.

- **Finished the siding:** in reserve to take some precautions in the realization of the forms and in the choice of the aggregates. In return, the risk of cracks constitutes a disability for the reinforced concrete, and that the shrinkage and creep are often drawbacks that it is difficult to bear all the effects.

I-9-3- The cement :

The cements are binders hydraulic consisting of powders, fine, mixed with water to form a dough capable by hydration to jack and harden or end of a time longer , at least long.

I-9-4- Sands :

The sands are composed of grains coming from the disintegration of rocks. The size of its grains is generally inferior to 5mm. A good sand contains grains of any calibre, but shall have advantage of large grains as small.

I-9-5- Gravel

They are formed by the grains of rock which size will generally range between 5 and 25 mm.

I-9-6- Mixing water :

It is an element very important in the hydration of the cement and of the workability of concrete. A deficiency or an excess of water can cause a drop in the resistance, mainly due to the porosity created by the evaporation of the water in excess, or to the non-compactness due to its deficiency.

I-10- Mechanical resistance:

a) Resistance characteristic in the compression :

The characteristic resistance to compression of the concrete **fcj** to *d* - day age is determined from tests on samples normalized to 16 cm in diameter and 32 cm in height.

For a dosage current of 350 Kg/m^3 of cement CEM II/ A 42,5, the characteristic in compression at 28 days is estimated to be 25 MPa (f_{c28} = 25 MPa).

- **for the resistors $f_{c\,28} \leq 40$ MPa :**

$$\begin{cases} f_{cj} = \dfrac{j}{4{,}76+0{,}83j} \cdot f_{c\,28} & \text{If } \quad j < 28 \text{ days} \\ f_{tj} = 1{,}1 .\ f_{c\,28} & \text{If } \quad j > 28 \text{ days} \end{cases}$$

- **for the resistors $f_{c\,28} \geq 40$ MPa :**

$$\begin{cases} f_{cj} = \dfrac{j}{1{,}4+0{,}95j} \cdot f_{c\,28} & \text{If } \quad j < 28 \text{ days} \\ f_{tj} = f_{c\,28} & \text{If } \quad j > 28 \text{ days} \end{cases}$$

b) Resistance to the traction :

The relations conventionally define the resistance characteristic of the traction of the concrete to d - day, denoted as ftj:

$$\begin{cases} f_{tj} = 0{,}6+0{,}06 . f_{cj} & \text{If } f_{c28} \leq 60 \text{ MPa} \\ f_{tj} = 0{,}275 . (f_{c28})^{2/3} & \text{If } f_{c28} > 60 \text{ MPa} \end{cases}$$

To : fc28 = 25 MPA ft28 = 2,1 MPa

I-11- Calculation standards:

The study of structure is carried out in accordance to the regulations below :

<u>I-11-1-</u> <u>The rules of BAEL 91 :</u>

(Concrete Armed to the States Limits) : based on the theory of states limits.

<u>I-11-2-</u> <u>States limits the ultimate (ULS) :</u>

Correspond to the value maximum of the capacity load of the building, either:

- Balance static.
- Resistance of one of the materials of the structure.
- Stability of form.

-Theory of computation:

- The sections plane before deformation remain plane after deformation.
- Not to drag on between the reinforcement and the concrete.
- The resistance of the concrete to the traction is neglected.
- The shortening of the concrete is limited to :
- ε_{bc}= 2‰ in bending composed.
- ε_{bc}= 3,5‰ in compression simple.
- The elongation of the steel is limited to ε_s=10‰.
- The diagrams deformations, stresses are defined for.
- The concrete in compression.
- The steel in traction and in compression.

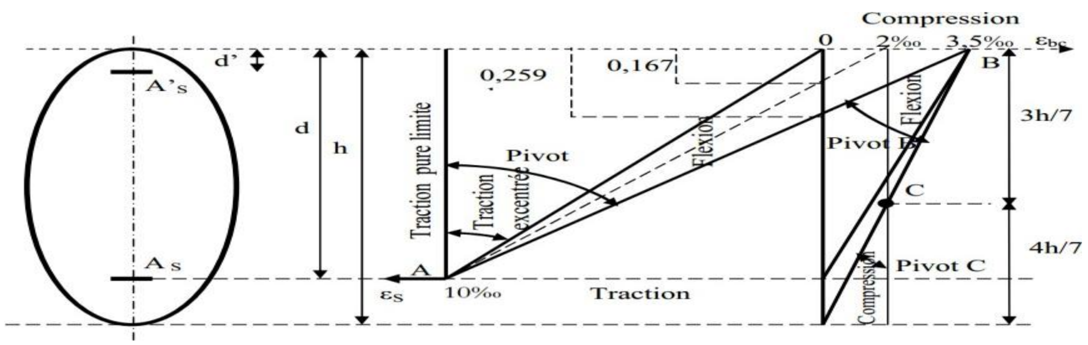


Figure I - 1:Diagram of the deformation limits of the section: rule of three pivots

The limit positions that can take the diagram of the strains are determined from the deformation limits of the concrete and of steel. The deformation is represented by a straight line passing through the peaks of A, B, or C, which are called pivots.

***Pure tension:** all of the fibers lie in the same quantity, the concrete will crack and therefore is not involved in the balance of solicitations, the room will be out of service when the deformation of the steel is worth 10‰, therefore the section will be extended to 10‰.

**The steel must be returned in tent the section ; the limit is on the diagram in the vertical passing through A.

***Traction offset**: to the limit, the fiber the more tense will have an elongation of 10‰, the less tense ε_s< 10‰, the more the eccentring increases the higher the voltage the minimum tends to 0 The rights of deformation rotate around up to the position of AO.

***Bending (simple or compound) :** It may not exceed the position AB, which corresponds to a shortening ε_{bc}= 3,5‰ of the fiber of concrete, the more compressed the limit state ultimate is reached with ε_s= 10‰ and$\varepsilon_{bc} \leq$ 3,5‰.

** The limit position AB corresponds to a neutral axis located a distance y = axAB .d of the fiber, the more compressed with α×AB = 3,5/(10+3,5) = 0,259 ; flexion simple or composed with 0 ≤ α ≤0,259 admits the pivot A.

**The special case where ε_s= 10‰ andε_{bc} = 2‰ corresponds to α =2/(10+2) therefore : α = 0,167

**To increase the area compressed we do can increase ε_{bc} in from there to a 3.5 ‰, it is necessary, therefore, decrease $\sigma_s$the right of the deformation rotates around B until: = 0 ; α
= Y/d varies from 0,259 - to - 1.

**The flexion simple or composed with frame tense with 0,259 ≤ α ≤ 1 admits the pivot B.

** If you just turn right around B, the small part of the section located below the reinforcements will be able to work as part of traction (not of constraint, and the steels will be compressed. It is the bending composed of: the bending composed of steel tablets (section of concrete partially compressed with 1 ≤ α ≤ h/d admits the pivot B.

** Compression: if all the sections of the concrete are compressed in the compression simple, the deformation of the concrete does not exceed εbc= 2‰

**The compression simple it is composed admits the pivot C. 2 ‰ ≤ ε_{bc}≤ 3,5 ‰ on the fibre the more compressed.
ε_{bc}≤ 2 ‰ on the fibre the more less compressed.

In summary :

- Swivel : Traction simple or composed, bending with state limit ultimate reached in the steel.
- Pivot B: Bending with state limit ultimately achieved in concrete.
- Pivot C : Compression simple or composed.

<u>I-11-3-</u> <u>Member limits of service (SLS) :</u>

Are the borders for here which conditions the normal operating and durability of the construction or of its elements are more satisfied are :

- Opening of the cracks.
- Deformation of the elements bearers.
- Compression in the concrete.

Hypothesis of calculation :

- The sections straight and flat before deformation remain straight and flat after deformation.
- Not to drag on between the concrete and the steel.
- The concrete tense is neglected in the calculations.
- The concrete and steel are considered as the materials that are linearly elastic and it is made abstract of the shrinkage and creep of the concrete.
- The module of elasticity longitudinal of the steel is by convention 15 times more large than that of concrete (E_S=15; n =15).

I-12- Characteristics of materials :

The material essentially used for the construction of this book is reinforced concrete, made of concrete and steel.

I-12-1- Concrete :

***.State limit ultimate (SLU) :**

Constraint ultimate of the concrete :

In compression with bending (or induced by the bending stress), the diagram that can be used in all cases and the diagram of calculation of said parabola rectangle.

The deformations of the concrete are:

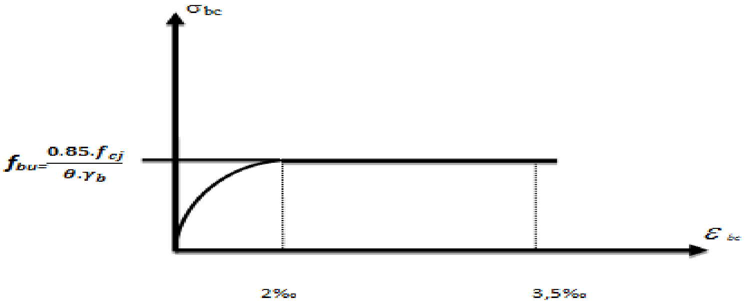


Figure I- 2: Diagram stress-strains of the concrete to The SLU.

f_{bu}: Constraint ultimate of the concrete in compression

$$f_{bu} = \frac{0,85 \times f_{cj}}{\theta \times \gamma_b}$$

γ_b : Coefficient of safety of concrete, it is necessary to 1.5 for the combinations of normal and 1.15 for the accidental combinations.

θ: Coefficient that depends on the duration of application of the load. It is set to :

- **θ =**1 when the term probable application of the combination of actions under consideration is greater than 24 h.
- **θ =**0.9 when this term is understood between 1 h and 24 h.
- **θ =**0.85 when it is lower to 1than.

Diagram rectangular(B. A. E. L91modifie99.p81) :

When the section is partially compressed, we can use a diagram rectangular simplified.

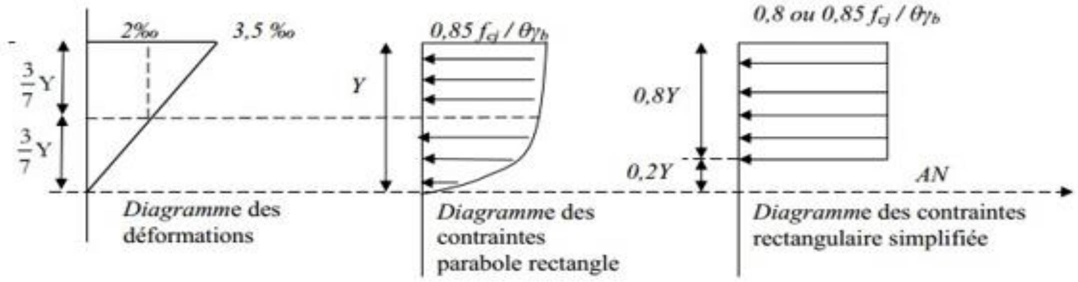


Figure I-3: Simplified Rectangular Diagram

On a remote de 0,2 y counted in from of the axis neutral the constraint is zero.

- On the distance remaining to 0.8 y the constraint to for value

$f_{bu} = \frac{0,85 \times f_{cj}}{\theta \times \gamma_b}$ pour areas compressed whose

the width is increasing or constant to the fibers, the more compressed.

$f_{bu} = \frac{0,85 \times f_{cj}}{\theta \times \gamma_b}$

for the areas compressed that the width is decreasing or constant to these same fibers.

in SLS : is given by $\bar{\sigma}_{bc} = 0,6 f_{c28}$

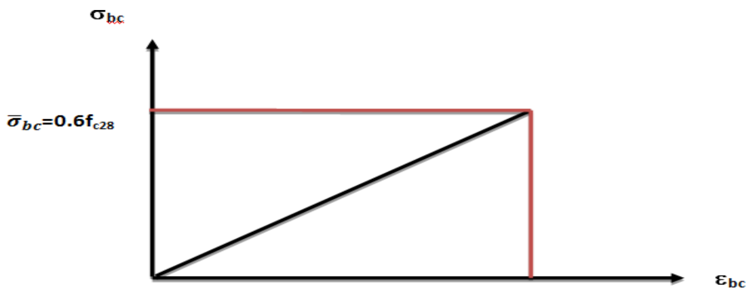


Figure I – 4 Diagramme stress-strains of the concrete to the SLS.

The constraint limit of service in compression in the concrete is limited by: $\sigma_{bc} \leq \bar{\sigma}_{bc}$

I-12-2- Module of deformation longitudinal of the concrete :

They exist two modules of deformation determined according to the BAEL 91.

1. The module of deformation instant : For the expenses of a period of application, less than 24 hours, we have :

$E_{ij} = 11000 \sqrt[3]{f_{cj}}$ Where : E_{i28}= 32164,2 MP

2. The module of deformation deferred : For the expenses of long - term application , one has :

$E_{vj} = 3700 \sqrt[3]{f_{cj}}$ Where: E_{v28}=10818,865 MPa

I. 11.3- Coefficient of POISSON:

This coefficient is the ratio of the deformation cross-sectional, and of the deformations

Longitudinal noted "ν". In accordance with the rules **of BAEL 91**:

to the SLU : ν=0 ⇒ calculation of stresses (concrete cracked).

in the SLS : ν=0,2 ⇒ calculation of deformations in concrete non - cracked).

I. 11.4-Constraint eligible for shear :

$\tau_u \leq$ = Min (0.2 inf_{cj} / γ(b,5MPa) Cracking little detrimental.

$\tau_u \leq$ = Min (0,15f_{cj} / γ(b,4MPa) Cracking harmful or very harmful.

Relation to the shear ultimate τu defines the constraint ultimate of shear in a piece in

concrete: τ_u=Tu/bd

With : **b** : width of the room. ;**d**: height - useful.

I-13- Steel:

Steel is an alloy of iron, carbon in low percentage; its role is to absorb the forces of tensile, shear, and torsion. We distinguish two types of steel:

*Steels, soft or half-hard to 0.15 to 0.25٪of carbon.

*Steels hard to 0.25 to 0.40 ٪of carbon.

The module of elasticity longitudinal of the steel is taken equal to: E_s =200 000 MPa.
The characteristic mechanical the more important of the steels is the limit elastic f_e.

The following table gives a few examples of steels.

Table I - 1:mechanical Characteristics of the steels

Type of steel	Appointed	Symbol	Limit of elasticity Fe [MPa]	Resistance to the Rupture	Elongation relative to the Rupture [‰]	Coefficient of fissuration	Coefficient of [ψ] seals lie
Steels in Bar	Round smooth FeE235	R L	235	410-490	22‰	1	1
	High adhesion FeE400	H Has	400	480	14‰	1,6	1,5
Steels in	Mesh welded (T S)	T S	520	550	8‰	1,3	1

lattice	TL520 (Φ<6)						

Forced limit of the steel :

to be ELECTED **:**

It adopts the diagram stress-strain following, with :

fe : stress elastic limit.

εs : deformation (elongation) relative to the steel.

$$\varepsilon_{es} = \left(\frac{fe}{\gamma_s E_s} \right)$$

σs : stress of the steel.

Constraint eligible for ELS:

This constraint depends on the nature of the cracks in the concrete; it determines:

Cracking a little harmful :

No verification.

Cracking harmful : $\sigma_s \leq \bar{\sigma}_{st} = \min\left(2/3\, f_e\, ;\, 110\sqrt{\eta . f_{c28}}\right)$ in MPa

Cracking very harmful : $\sigma_s \leq \bar{\sigma}_{st} = \min\left(0{,}5 f ;\, 90\sqrt{\eta . f_{c28}}\right)$ in MPa

With η : coefficient of cracking.

η=1 for steel round smooth.

η=1,6 for the steels to high adhesion (HA) Coefficient of equivalence:

The coefficient of equivalence noted η is the report the following :

$$\eta = \frac{E_S}{E_b} = 15,$$

With :

n : coefficient of equivalence.
Es : modulus of deformation of the steel.
E_b : Modulus of deformation of the concrete. Fe= 400 MPa.

- **Load effects considered in relation to limit states**

The solicitations of calculation are determined from the combination of actions

following : 1,35 G + 1,5 Q

Combination of action, following : G + Q

If it is a response to the efforts of the horizontal due to the earthquake, the rules of earthquake in Algeria had planned combinations of the following actions:

G + Q+ E		**G** : permanent load.
G + Q +1,2 E	with	**Q** : operating loads.
0,8 G +E		**E**: earthquake loads.

I.13.Conclusion:

This chapter has provided an overview of the project, stating the nature of the building, its location, and its constructive system, while highlighting the impact of the soil on the choice of the foundations in preparation for the structural analysis to come according to the RPA.

Chapter II :

Pre-sizing and loads descent

II-pre-sizing and load lowering:

II.1- Introduction :

The evaluation of the different sections of the elements of our structure: beams, columns, walls, and other secondary elements necessarily involves a sizing preliminary, called pre-sizing. This pre-sizing serves a purpose: determination of the order of magnitude of the point of view formwork of each element constituting the book.

For this, we evaluate a descent of loads and overloads in order to determine the returns to each element bearer, of all the levels, and at the end up to the foundation.

II.2- Pre - sizing:

II.2.1- Pre - sizing of floors (body hollow) :

The floors are thin plates whose thickness is small compared to other dimensions, and is based on 2, 3, or 4 supports. It adopts the floors to body hollow

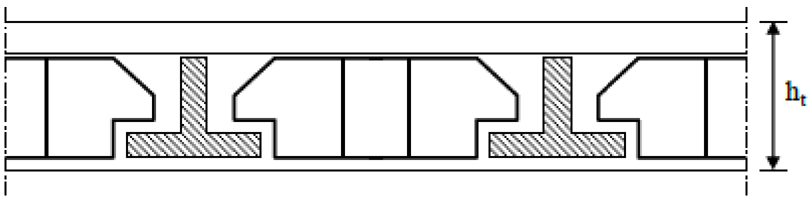


Figure II- 1:floor corp hollow.

The thickness of the hollow floors and the height of the joists (ht), and conditions of use and resistance, we will deduct, therefore, the thickness of the floorfrom onem of the following conditions:

Resistance to fire:

e =7cm for an hour of cup of fire. e =11cm

For two hours of cut-fire.

e =15cm for a firewall to four

hours. We accept: e = 16 cm

Isolation phonetic:

The comfort and isolation phonetics require a thickness minimum of: e = 16 c

Resistance to the bending :

For the pre-sizing of the height of the beams, I will use the following general formula:

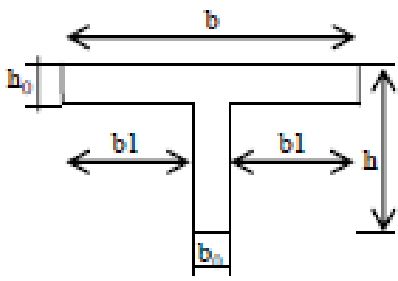
$$\frac{L_{max}}{22.5} \leq h_{,t} \leq \frac{L_{max}}{15} \Longrightarrow \frac{400}{22.5} \leq h_{,t} \leq \frac{400}{15} \Longrightarrow 17.78\ cm \leq h_{,t} \leq 26.66\ cm$$

L: is the widest range of beams, measured between bares of support in the direction of the ribs.

It has: L = 430-30 = 400 cm or 16.88cm < ht< 25.33 cm.

Therefore, we adopt a thickness of : ht = 20 cm (availability of a body hollow of 16+4 at the level of the market

* The table of compression to take into account is chosen in respect to the



Terms and Conditions

the following:

ht =20 cm

B = 65

0.3 (ht) ≤ b_0 ≤ 0.4 (ht)

0.3(20) ≤ b_0 ≤ 0.4(20)

6 cm ≤ b_0 ≤ 10 cm

Therefore, by taking b_0= 10 cm.

$$b1 = \frac{65-(10)}{2} = \frac{55}{2} = 27.5\ cm.$$

So : b1=27.5 cm

II.2.2- Pre - sizing of the floor slab full :

It takes ep= 15 cm

II.2.3- Pre - dimensioning of beams :

The beams will be pre-dimensioned according to the formulas and empirical data by BAEL91 and verified by the result, according to the RPA99/version 2003.

II.2.3.1- Beams (main):

* They receive the loads transmitted by the joists (beams) and then distributed to the columns on which these beams rest.

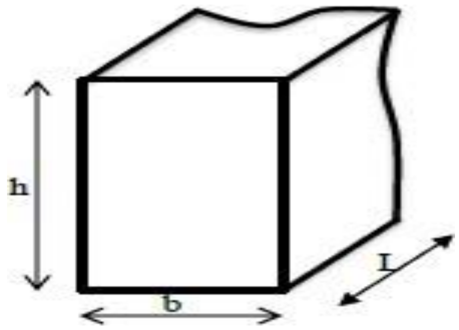
* they connect the columns.

* they Support the slab.

After the BAEL91:

The
$l/15 \leq h \leq l/10$

$0.3\ h \leq b \leq 0.4\ h$



With : L : range maximum of the beam. h : height of thesection.

b : width of the section.

We have : L =410cm
$l/15 \leq h \leq l/10$

$$\frac{430}{15} \leq h \leq \frac{430}{10}$$

$28.67 \leq h \leq 43$ One takes h = 40cm. 0,3 h ≤b ≤ 0,4 h

0.3(40) ≤b ≤ 0,4(40)

12 ≤b ≤ 16 , We take b = 30cm.

2 Condition of the R. P. A 99 :

$$\begin{cases} h \geq 30\ cm \\ b \geq 20\ cm \\ (h/b) \leq 4 \end{cases} \Rightarrow \begin{cases} h = 40 \geq 30\ cm \\ b = 30 \geq 20\ cm \\ (h/b) = 1.33 \leq 4 \end{cases}$$ .... verified

II.2.3.2- Beams (side):

They link the portals between them to not switch.

After the BAEL91:

l/15≤ h ≤l/10

0.3 h ≤ b ≤ 0.4 h

With : L : range maximum of the beam.

h : height of the section.

b : width of the section.

We have : L= 380cm

l/15≤ h ≤l/10

$$\frac{380}{15} \leq h \leq \frac{380}{10}$$

$25 \leq h \leq 38$ One takes h = 35 cm. 0,3 h ≤b ≤ 0,4 h

0.3(35) ≤b ≤ 0,4(35)

10.5 ≤b ≤ 14 , We take b = 30cm.

2 Condition of the R. P. A 99 :

$$\begin{cases} h \geq 30\ cm \\ b \geq 20\ cm \\ (h/b) \leq 4 \end{cases} \Rightarrow$$

$$\begin{cases} h = 35 \geq 30\ cm \\ b = 30 \geq 20\ cm \text{ ....verified} \\ (h/b) = 1{,}16 \leq 4 \end{cases}$$

Table II - 1: section of the beams

Beams	Section cm²
Main	(30x40)
Secondary	(30x35)

II.2.1.3-Columns:

According to the rules RPA 99/ version 2003 (Art 7.4.1) according to the area earthquake, we have: $\min(b_1, h_1) \geq 25cm$, we chose h = b = 40 cm.

$$\min(b_1, h_1) \geq \frac{h_{has}}{20}$$

he: height of the floor, he = 3.4 m.

$$\frac{1}{4} < \frac{b_1}{h_1} < 4$$

Then:

- 40> 25...(C. V)
- 40> $\frac{340}{20} = 17cm$.................................(C. V)
- 0.25<1<4 ....(C. V)

According to the rules of BAEL 91 (Art B. 8.4.1) who deal with the condition of not buckling; we can estimate the formwork preliminary column by the application of the method lump sum If (λ < 70)

$$B_r = \frac{0.9\ \gamma b}{fc28}\left[\alpha Nu - \frac{A.fe}{\gamma s}\right]$$

N_u : Load centered descended to the base of the column. A : section reinforcement contributed

λ...... Elancement geometric.

l_0 : height of the floor.

I_{min}: Moment of inertia minimum of the section of the column in question. B : Area of cross section of the column.

$l_f = 0.7\ l_0$ .... Length of the buckling (BAEL Art-B. 8.3.3.1)

$i = \sqrt{\frac{I_{min}}{B}}$ .... Radius of gyration

$l_f = 0{,}7 \ . \ 3{,}4 = 2.38$ m. $\quad I_{min} = \frac{0.3(0.3)^3}{12} = 6.75.10^{-4}\, m^4$

B = 0,3.A 0.3 = 0.09 m2

$$i = \sqrt{\frac{6.75.10^{-4}}{0.09}} = 0.0866m \qquad \Rightarrow \lambda = \frac{l_f}{i} = \frac{2.142}{0.0866} = 24.73$$

As long as: $\lambda = 24.73 \leq 50 \leq 70 \quad \Rightarrow \alpha = \frac{0.85}{1+0.2(\frac{\lambda}{35})}$

II.3- Determination of permanent and operating loads:

II.3.1-permanent loads :

II.3.1.1- Current floor:

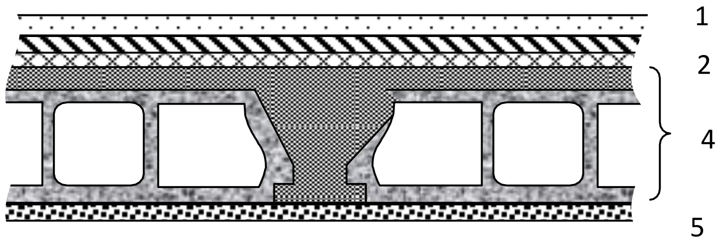


Figure II.2: Current floor

Table II.2 : Components of current floor

N	Designation	e(cm)	Load daN/m2
1	Coating on tiles by cm (20 daN/m2)	e=2cm	40 daN/m2
2	Chappe mortar of cement (mortar of poses) by cm (20 daN/m2)	e=2cm	40 daN/m2
3	Bed of sand, $\gamma = 1700 daN/m^3$	e=2cm	34 daN/m2,
4	Floor body hollow by cm.	e=(16+4)cm	280 daN/m2
5	Coated in plaster by cm (10 daN/m2)	e=1.5 cm	15 daN/m2
6	Partitions	e=1cm	75 daN/m2
		G (Total)	484daN/m2

II.3.1.2-Floor terrace :

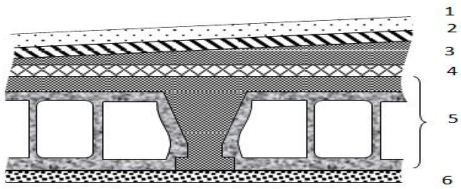


Figure II.3:Floor - floor power

Table II.3 : Components of a floor terrace

N	Designation	e(cm)	Load daN/m2
1	Protection in rubble	e=3cm	60 daN/m2
2	sealing multi-layer	e=2cm	12 daN/m2
3	Concrete slope (ɤ=22KN/ m2)	e=8cm	176 daN/m2
4	Insulation thermal in thin (ɤ=4KN/m2)	e=4cm	16 daN/m2,
5	Floor body hollow by cm.	e=16+4cm	280 daN/m2
6	Coated in plaster by cm (10 daN/m2)	e=1.5 cm	15 daN/m2
		G (Total)	559daN/m2

II.3.2-Loads of exploitation :

Table II.4 : Values of the overload

Item	Overload (daN/m2)
Floor terrace	100
Floor floor current	150

II.4-Descent of the charges :

II.4.1- Introduction:

The descent of the charges is obtained by determining the path of the efforts in the structure since their weight application until the foundations.

A way in general, the charges will distribute in the function of surfaces allocated to each element.

The purpose of the descent of the charges is to check the sections of the elements of the structure, for it determines the loads acting on the columns the most popular.

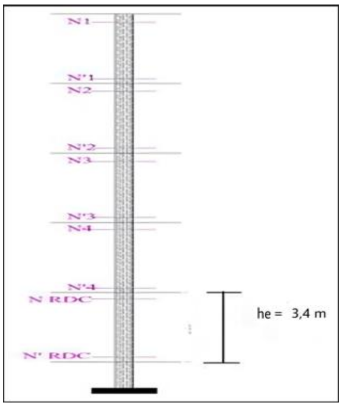


Figure II.4 : Schematic of the descent of the expenses of a column

II.4.2-Steps of calculation:

The steps of pre-dimensioning are:

The choice of the column, the more requested.

The calculation of the surface recovery is done by this column.

The determination of the efforts taken by this column to the CHOSEN one and one fact check at RPA 99 version 2003& BAEL 91

II.4.3- Column : (10- A)

S= (1.4+1.35)x(1.5+2)=9.62m^2

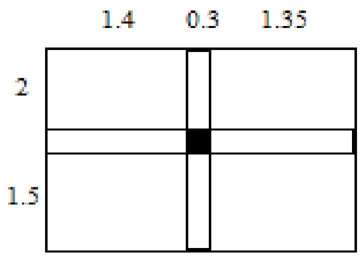


Figure II.5: Surface afferent -Column (y - 10- A)

Table II.5 : Values of the descent of the charges-Column (y - 10- A)

Level	Section	Element		N_G (KN)	N_Q (KN)
04	N01	Floor terrace	9.62×6.32	60.798	9.62
		Beams main	$(0,4 \times 0,3 \times 3,5) \times 25$	10,500	
		Beams side	$(0,3 \times 0,35 \times 2,75) \times 25$	7,218	
				78.516	
	N'01	Weight of N01		78.516	
		Columns	$(0,4\times0,4\times3,4)\times25$	7,650	
		The sum		86.166	
03	N02	Weight N01		86.166	24,050
		Floor floor	9.62×5.53	53.198	
		Beams main	$(0,4 \times 0,3 \times 3,5) \times 25$	10,500	
		Beams side	$(0,3 \times 0,35 \times 2,75) \times 25$	7,218	
		partitions	$(0,75 \times 2,75\times 3,5)$	7,218	

				164.300	
	N̓02	Weight of N02		164.300	
		columns	$(0,4 \times 0,4 \times 3,4) \times 25$	7,650	
		The sum		171.95	
02	N03	Weight of N02		171.95	48,100
		Floor floor	$9,62 \times 5.53$	53.198	
		Beams main	$(0,4 \times 0,3 \times 3,5) \times 25$	10,500	
		Beams side	$(0,3 \times 0,35 \times 2,75) \times 25$	7,218	
		partitions	$(0,75 \times 2,75 \times 3,5)$	7, 218	
				250.084	
	N̓03	Weight of N03		250.084	
		columns	(0,4×0,4×3,4)×25	7,65	
		The sum		257.734	
01	N04	Weight of N03		257.734	96,200
		Floor to floor	in 9.65×5.53	53.198	
		Beams main	$(0,4 \times 0,3 \times 3,5) \times 25$	10,5	
		Beams side	$(0,3 \times 0,35 \times 2,75) \times 25$	7,218	
		partitions	$(0,75 \times 2,75 \times 3,5)$	7,218	
				335.868	
	N'04	Weight of N04		335.868	

		columns	(0,4×0,4×3,4)×25	7,65	
		The sum		343.518	
DRC	NRDC	Weight N04		343.518	192,400
		Floor floor	9,62 × 5.53	53.198	
		Beams main	(0,4 × 0,3 × 3,5) × 25	10,5	
		Beams side	(0,3 × 0,35 × 2,75) × 25	7,218	
				414.434	
	Don DRC	Weight of NRDC		414.434	
		columns	(0,4 × 0,4 × 3,4) × 25	7,65	
		The sum		422.084	

$N_u = 1,35 N_G + 1,5 N_Q$

$= 1,35(422.084) + 1,5(192,400) \Rightarrow N_u = 858,413 KN$

$N_u = 858,413 \leq 1185,83$ CV

II.4.4- Verification of the buckling :

+According BAEL91 :

A min. = max. [0,2B/100 ; 4% du périmètre]

A min. = Section min. des aciers en cm²

B = Aire de la section droite en cm²

+RPA 99 version 2003 :

Percentage minimum is :

- 0.8% in Zone II Percentage maximum :
- 4% in Zone current
- 6% in Zone recovery

*the diameter is a minimum of 12 mm

*the length of the minimum of recoveries is to:

-40 ϕ in Area II

* the distance between the bars vertical on one side of the column must not exceed

-25 cm in Area II

+Operating the Socotec

II-5- Verification of the stability of the form :

II.5.1- Verification of the section of colum (40x40) of the 2nd up to 5 th floor : (BAEL.99.B.8.4.1)

$N_G = 335.868KN$ $N_Q = 96.8$ KN

***/ Combinations of loads :**

ELU : $N_U = 1.35\ N_G + 1.5\ N_Q = 1.35\ (308.932) + 1.5\ (96.8) =$ **598.621 KN**

***/ Verification of structural form stability:**

N_U = 598.621 KN

- Moment of inertia : $I = \frac{(b\times h^3}{12} + \frac{40\times 40^3}{12} = 12505.08\ cm^4$
- A = b×h = 1225 cm2
- The radius of gyration : $i = \sqrt{\frac{125052.08}{35\times 35}} = 10.10$
- Length of flambements : $If = 0.7 \times L_0 = 0.7 \text{ x } 300 = 210$ cm
- The slenderness of the post : $\lambda = Lf/i = 210/8.66 = 20.79 < 50$
- $\alpha = 0.79$

- The reduced surface : Br = 38×38 = 1089 cm2
- A = max $* \frac{0.2\, b\times h}{100} = \frac{0.2\times 40\times 40}{100} = 2.45$ cm2

  $* \frac{8(b+h)}{100} = \frac{8(40+40)}{100} = 5.6\text{ cm2} \rightarrow A = 5.6cm^2$

$$Nu < \alpha\left(\frac{Br \times f_{c28}}{1.35} + \frac{A \times F_{(e}}{\gamma_s}\right) = 0.79\left(\frac{1089 \times 2.5}{1.35} + \frac{5.6 \times 400}{\gamma_s}\right) = 1747KN$$

II-6 Pre - sizing of sails:

Pre-dimensioning of walls in concrete armed justified by article 7.7 of RPA 99.

The sails are used, on the one hand, to contravene the building from horizontal forces (earthquake and/or wind), and on the other hand, to take up the vertical forces (own weight and others), which they transmit to the foundations.

The charges vertical: loads permanent and overloads.

The horizontal actions: effects of an earthquake and/or wind.

The sail ensuring the bracing is assumed to be full.

Only the efforts of translation will be taken into account if those of the rotation do not know the part of this pre-sizing.

According to the **RPA 99** article 7.7.1 ,considered as the sails of the elements to be satisfactory to the condition :($L \geq 4e$). In the opposite case, items considered for linear elements

With :

L : length of sail.

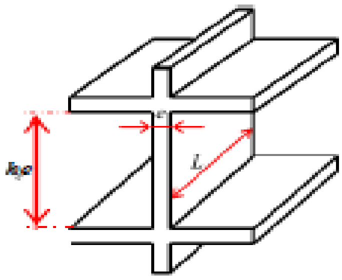
e : thickness of the sail.

The thickness minimum is 15 cm. In addition,the thickness should be determined in function of the height of the free room *he* and conditions of rigidity at the ends as shown in c. to.d. The sails are walls in concrete armed justifying to section 7.7.1 of RPA99 : e_{min} = 15cm.

Sailing floor and the DRC :

et = 3.40 m and condition of rigidity in the extremities

following :



For the sails about free : a≥ max [et/20; 15 cm]

$e \geq h/20 \Rightarrow e \geq 340/20 \Rightarrow e \geq 1$to 7 cm. We shall adopt: (**e=20 cm**

II.7-Conclusion:

This chapter has introduced the various structural elements in the main constituting the building, highlighting the role of each element in the transfer of loads and the stability of the structure. The system adopted has been analysed through the detailed study of the columns, beams, slabs and foundations, in accordance with the technical standards in force. The mechanical characteristics of the materials used have been defined, constituting the basis of the calculations for sizing and verification of structural safety. This theoretical overview is an essential support to address the steps of calculating depth to come.

Chapter III :

Calculate Secondary elements

III-Calculates Secondary elements

III-1- Introduction:

This chapter deals with the methodology of calculation of structural elements adopted for the design and the reinforcement elements of the work studied. The calculations are based on the regulations in force, in particular, the RPA 99 version 2003 and the BAEL, in order to ensure the resistance of the structure in the face of different stresses, in particular, the seismic loads. This chapter also introduces the steps in the calculation of the loads, the determination of the internal forces as well as the use of specialized software in the design and verification.

III.2-The Acroterion:

III.2.1-Pre-sizing of the acroterion:

The acroterion is an element in concrete reinforced that ensures the security, total to the level of the terrace.

It is likened to a console built into the floor terrace of which the reinforcement is calculated under the effect of the two efforts, and will be determined in flexion combined with compression.

The acroterion is requested by:
An effort normal (G) due to its weight of clean, a horizontal force (Q) due to the main current, generating a time reversal (M). The calculation is made for a band of 1 m in the section of the bezel.
The surface of the acroterion is :

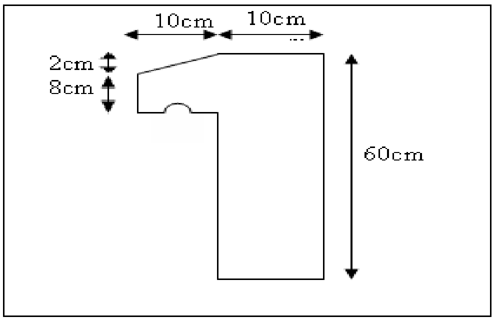


Figure III.1 :schematic of the acroterion.

S = (0,02+0,1)/2 + (0,08x0, 1) + (0,1+0,6) S = 0,069 m^2

The weight of own of the acroterion is :

P= (0,0x25) =KN/m

G =1.72 KN/m

It is undergoing a surge in horizontal hand current Of : **Q= 1KN/m**

III.2.2-Calculation of the acroterion :

The acroterion is an element in concrete reinforced that ensures the security, total to the level of the terrace.

It is likened to a console built into the floor terrace of which the reinforcement is calculated under the effect of the two efforts, Q and G, and will be determined in flexion combined with compression.

The calculation will be for a band of 1 m in the section bezel

G = 25 [(0,6*x*0, 1) + (0,15*x*0,1) - (0,03*x*0,15) / 2] =1,819*KN /ml*

Weight clean of the acroterion: G = 1,819 KN/ml

Overload operation: Q =1,00 KN /ml

Normal stress due to the weight own G: N = Gx1ml =1,819 KN

shear force : T = Qx1ml = 1,00 KN

Time flexing max had to have the overload Q: M = T x H = Qx1mlxH = 0,60 KN m

III.2.3-Combination of loads :

Has **the CHOSEN one**:

Nu = G = 1,819 KN

Mu = 1.5Q = 0,9 KNm

Has The ULS:

Ns = 1.819 KN

Ms = 0.60 KNm

III.2.4-Reinforcement of the acroterion:

The reinforcement of the acroterion will be determined in bending composed and will be given by meter linear; for the calculation, we consider a section :

Thickness of the section : 10 cm

b : width of the section : 100 cm

c and c' : Coating : 2 cm

d = h – c : Height useful

(b*h) cm^2 subject to bending composed.

M -$_f$: Time fictional calculated by report to the C. D. G of reinforcement tense. Calculation of frames with The CHOSEN one :

Position of the center of pressure to the ELU :

$$e_u = \frac{M_u}{N_u} = \frac{0.6 \times 10^2}{1.819} = 32.98 \approx 33$$

$\frac{h}{2} - c = 10 - 2 = 3cm$

$\frac{h}{2} - c < e_u$

The center of pressure is located outside of the section limited by the frames of where the section is partially compressed.

Therefore, the acroterion will be calculated in flexion simple under the effect of the time dummy (M), and then in flexion composed where the section of reinforcement will be determined in function of the already calculated.

III.2.5-Calculation in simple bending and composed:

a) calculation in simple bending :

- **The time dummy:**

$$M_f = N_{(u} \times e_f = N_u \left(e_u + \frac{h}{2} - c\right) = 1.819 \times \left(0.37 + \left(\frac{0.1}{2}\right) - 0.02\right) = 0.727\ KN.m$$

$$\mu_u = \frac{M_f}{bd^2 f_{bu}} = \frac{0.727 \times 10^2}{100 \times 8^2 \times 14.2} = 0.0079$$

With : $f_{bu} = \frac{0.85 f_{c28}}{\theta.\gamma_b} = \frac{0.85 \times 10^4}{1 \times 1.5} = 142 \frac{kgf}{cm^2} = 14.2\ MPa > 0.392$

$The\ \mu_u < \mu_l\ of\ the\ section\ is\ simply\ army, therefore, Has_s = 0$

$\mu_u = 0.0108 \Rightarrow \beta = 0.995$

- **Reinforcement calculation:**

- $A_f = \frac{M_f}{ßd\frac{fe}{\gamma_b}} = \frac{0.727 \times 10^5}{0.995 \times 8 \times 348 \times 10^2} = 0.262cm^2$

b) Calculation in bending comprising :

The section real of reinforcement is :

$$A_s = A_{(f} - \frac{N_u}{\sigma_{st}} = 0.26 - \frac{1.819 \times 10^3}{348 \times 10^2} = 0.21cm^2$$

III .2.6- Check with ULS:

a) Condition of non - fragility :

$A_{min} = 0.23\ bd \times \frac{f_{t28} \times e_s - 0.455\ d}{f_e \times e_s - 0.185\ d} = 0.9cm^2$

$A_{min} = 0.9cm^2$

$With: e_s = \frac{M_s}{N_s} = 0.330cm^2$

$f_{tj} = 0.6 + 0.06 f_{c28} = 0.6 + 0.06 \times 25 = 2.1\ MPa$

Conclusion :

The reinforcement checking the condition of non-fabricity is superior to those calculated for the CHOSEN one, so we will adopt:$\boldsymbol{A_s}$=A_{min} =0.9cm2

Either $\boldsymbol{A_s}$=5HA8 /ml=2,51 cm2 with a spacing : **S**$_t$= 20 cm

Reinforcement of distribution:

$$\boldsymbol{A_r} = \frac{A_s}{4} = \frac{2.51}{4} = 0.62\ cm/ml$$

Either : 4HA8/ml= 2,01cm2repartie about 60 cm of height.
Check the shear :

$$\tau_u = \frac{T_{max}}{b_0 d} = \frac{1.5 \times 10^3}{1000 \times 80} = 0.01875\ MPa$$

$$\overline{\tau_u} \leq Min\left(\frac{1.5 \times f_{c28}}{\gamma_{(b}}, 4MPa\right)$$

$$\overline{\tau_u} \leq Min\left(\frac{1.5 \times 25}{1.5}, 4MPa\right) = 2.5\ MPa$$

$$\tau_u < \overline{\tau_u} \Rightarrow (conditon\ verified)$$

b) Length of sealing right according to BAEL99 :

l_s=40 **ϕ** =40x0,8=32 cm.

III.2.7-Verification in the SLS:

The acroterion is exposed to the weather. Therefore, the cracking is regarded as Detrimental, we must verify:

$\sigma_{bc} \leq \overline{\sigma_{bc}}$ = 0,6f_{28}= 15 MPa

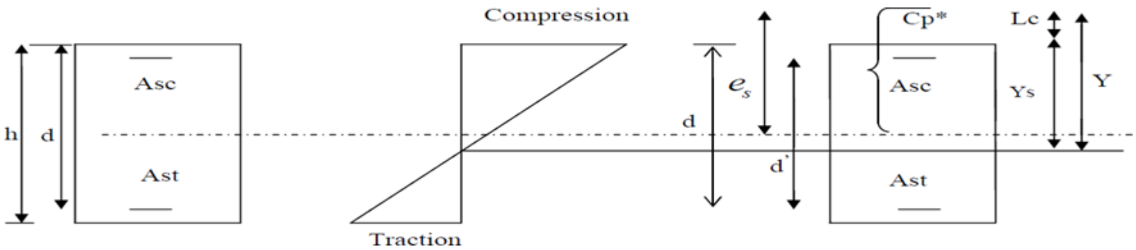
$$\sigma_s = \bar{\sigma}_s = min\left\{\frac{2}{3}; f_e; max\{0.5f_{,e}; 110\sqrt{nf_{t28}}\}\right\} = 201.63\ MPa$$

⇒n=1,6 : Cracking harmful, (acierHA) $\emptyset \geq 6mm$

One has , **The$_c$**: Distance of the center of pressure to the fiber, the more compressed of the section.

$$L_c = \frac{h}{2} - e_s = \frac{10}{2} - 33 = -28cm$$

$$L_c = -28cm$$



y³+Pʏ+q =0 ....(*)

$$\boldsymbol{p} = -3(-28)^2 - 90(2.51)\frac{(-28-6)}{100} + 90(2.51)\left(\frac{8+28}{100}\right)$$

A_s=A_{st}=2.01 cm^2

$$q = -2{L_c}^3 - 90A_s\frac{(L_c d -)^2}{(b} + 90\frac{(d - L_c)^2}{b}$$

$$q = -2(-28)^3 - 90(2.51)\frac{(-28-6)^2}{100} + 90\frac{(8+28)^2}{100}$$

q = $\mathbf{38364, 93}$

⇒ y^3-2193,87 y +38364,93 =0

y_1 = 30,78.**y_2** = -53,90.**y_3**

=23,12. Condition : ≤ **y_{ser}** ≤ d

y_{ser} =**y** + **L_C**

y_{1SER}= 30,78-28 =2,78 cm..OK

y_{2SER}= -53,90-28 =-81,90 cm... non

y_{3SER}= 23,12-28 =-4,88 cm .... non

y_{SER}=**y_{1SER}** = $\mathbf{2, 78cm}$

y_{1SER}= 30,78-28 =2,78 cm

With **y** =**y_{SER}** **-L_C** =28+2,78= 30,78 cm

$$I = \frac{b{y_{ser}}^3}{3} + n[A_{st}(d - y\ \ -_{ser})^2 + A_s(y_{ser} - d')^2] : n = \frac{E_s}{E_b} = 15$$

n : equivalent Coefficient

$$I = \frac{100\times2.78^3}{3} + 15[2.51(8 - 2.78)^2 + 2.51(2.78 - 6)^2]\ = 2132.43cm^4$$

a) Verification of the constraints of compression in the concrete :

$\bar{\sigma}_{bc}$= 0,6 fc28 =15 MPA

$$\sigma_{bc} = \frac{y.N_{ser}}{I}y_{ser} = \frac{0.3078\times1.81\times10^3}{2132.43\times10^{-8}} = 2.78\times10^{-8}MPa$$

σ_{bc} =0,73 MPA<$\bar{\sigma}_{bc}$ = $15MPA$..(Condition true).

b) Verification of the stress in the steel :

$\bar{\sigma}_s$=201,63 MPA (already calculated)

$$\sigma_s = \frac{15\times0.3078\times10^3}{2132.43\times10^{-8}}(0.08 - 2.78\times10^{-2})\times10^{-6}$$

σ_s =20,55 MPA <$\bar{\sigma}_s$=201,63 MPA CV

III.2.8-Verification of the acroterion on the earthquake: (RPA99. Art 6.2.3)

The acroterion is calculated under the action of forces seismic following the formula as follows :

F_P=4 .A.C_P.W_P

A : coefficient of acceleration of the area, in our case (box II.a group of use 2)

A = 0,15

(RPA99, art 4.2.3 table 4-1)

Cp : Factor of force horizontal (Cp = 0,8)

Wp : Weight of the acroterion =1,819 kN/ml

where : *Fp*= 4x 0,15x0,8x1,819 = 0,873 /ml < Q = 1 KN/ml ⟹ (Condition true).

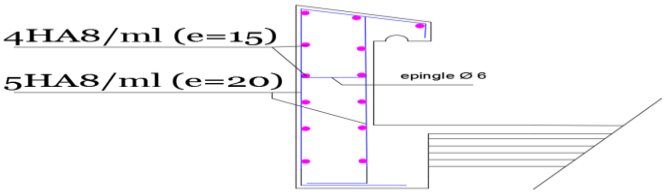


Figure III.2 : reinforcement of the acroterion

III.3. Study of stairs

III.3.1-Definition :

A staircase is a suite of plans horizontally arranged in tiers, in order to allow one to move from one level to another.

III.3.2-Dimensions

-height of walk : $14 \leq h \leq 18$, it takes: h= 17 cm

-number of steps: (2 flocks)

$$n = \frac{H}{h} = \frac{340}{17} = 20 \text{ steps}$$

For a single fly: n = 10

-width of works: $24 \leq g \leq 32cm$, we take: g= 30 cm.

-Verification of the formula de BLONDEL :

$59 \leq g + 2h \leq 66 \Leftrightarrow 59 \leq 30 + 2\times 17 \leq 66 ....CV$.

-Length horizontal of the fly :

$The = (n-1)\times g \Rightarrow The = (10-1)\times 30 = 270\ cm.$

-Length of the bearing : $the = 4\ .2 - 2,70 = 1,50\, m$.

-Thickness (bench and level) :

(L : scope of stairs between bare L= 5.5 m)

$\frac{L}{40} \leq e \leq \frac{L}{30}$ $\frac{456}{40} \leq e \leq \frac{456}{30} \Rightarrow 13.7 \leq e \leq 18.3$

it takes: e = 15cm.

<u>Note :</u>

The calculation which is based on the thickness (15cm) has shown that the condition to check the load arrow is not filled can be changed (e=17cm) in the calculation below.

-Angle of inclination :

tag α= h/g =17/30 =0.567

α = 29.53°

Table III.1::Level of rest (floor and current DRC):

Materiaux		Thickness e (cm)	(KN/m³)	G (KN/m²)
1	Tile	2	20	0.4
2	Mortar of poses	3	20	0.6
3	self-Weight of the bearing	15	25	3.75
4	Coated in plaster	2	10	0.20
Total				4.95

Load : Q=2.5 KN/m². Load

permanent : G = 4.95 KN/m²

Table III- 2:Bench floor power and the DRC.

Materiaux		Thickness e (cm)	d (KN/m³)	G (KN/m²)
1	Tile	2	20	0.4
2	Mortar of poses	3	20	0.6
3	Weight of stairs	/	0.15x25x0.5	1.87
4	self-Weight of the bench-top	15	25/cos29.05	.3.43
5	Coated in plaster	2	0.02x10/cos29.05	0.23
6	Gard corp	/	/	0.5
Total				7.03

Supported os: Q=2.5 KN/m².

Load permanent: G = 7.03 KN/m²

Combinations :

a-) level :

G = 4.95 KN/m2, Q= 2.5 KN/m2

SLU :

that= 1, 35G+1.5 Q= 10.42 KN / m

SLS :

qu= G +Q=4,95+2.5=7,45 KN/m2

(b) bench :

G = 7.03 KN/m2, Q= 2.5 KN/m2

SLU :

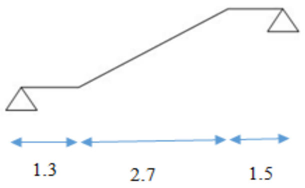
that = 1, 35G+1.5 Q= 13.24 with regard to KN / m

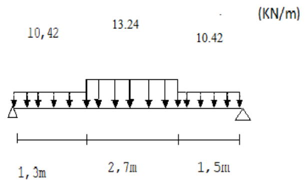
SLS :

qu= G +Q=4,95+2.5=9.53 KN/m2

Reinforcement :

SLU :





$$Q_{\text{equivalent}} = \frac{10.42(1.5)+13.24(2.7)+10.42(1.3)}{1.5+2.7}$$

$Q_{\text{equivalent}}$ =11.80 KN/m

With : SLU : Pu = 1,35 G+1,5 Q SLS : Ps = G+Q Mt = 0,95M_0 Ma = 0.3 M_0

Table III.3:Calculation of the time and of the effort tranchant

State		ELU	ELS
P (KN/ml)	Bench	13.24 with regard to	9.53
	Bearing	10.42	7.45
M_0 (KN.ml)		44.62	32.07
T (KN)		32.45	23.29
M_t (KN.ml)		35.70	25.62
$M_{(a}$ (KN.ml)		13.50	9.60

Reinforcement :

On Span :

SLU :

Diagram of reinforcement: the calculation is made at the bending simple

with :(b x h =100*15). d=h-c.

h=15 cm; C=C'= 2cm; d=13 cm

$\mu = \frac{35.70*10^4}{100*13^2*142} = 0{,}148 < \mu_R = 0{,}392 \Rightarrow A\ s = 0$

$\alpha = 1,25\left(1 - \sqrt{1 - 2\mu}\right) = 0,201$

$\beta = 1 - 0,4\alpha = 0,919$

Z = d (1-0,4 α) = 13*(1-0,4*0,201)

Z = 11.95 cm

$$A_s = \frac{M_t}{Z \times \delta_s} = \frac{35.70 \times 10^4}{12.47 \times 3480}$$

Has$_s$ = 8.58 cm2 ⇒ **is adopt6HA12 (8HA12 = 9.05cm2) Calculation of the spacing:**

St ≤ min(3h,33cm), We adopt a spacing of

16cm Frames distribution :

$$\boldsymbol{Ar = \frac{A}{4} = \frac{9.05}{4} = 2.26cm^2}$$

Has$_s$ = 2.26 cm2 ⇒ **we adopt 3HA10 (3HA10=2.36cm2)** St ≤ min (4h, 40cm), We adopt a spacing of **25cm Checking the shear**:

Has$_s$ = 2.36 cm2

It has: Amin =0,23 xbxd (ft28/fe)

Amin= 1,57 cm2, As ≥ Amin...Condition checked

<u>SLS reinforcement</u>

Diagram of reinforcement: the calculation is made at the bending simple with: (b x h =100*15).

d=h-c.

h=15 cm; C=C'= 2cm; d=13 cm

$$u = \frac{13.39*10^4}{100*13^2*142} = 0.55 < \mu_R = 0.392 \Rightarrow \text{ A's}$$

$$\propto = 1.25 \times \left(1 - \sqrt{2\mu}\right) = 0.07$$

$\beta = 1 - 0,4\alpha = 0,971$

Z = d (1-0,4 α) = 13*(1-0,4*0,055)

Z = 12.63 cm

$A_s = \frac{M_t}{Z*\delta_s} = \frac{13.40*10^4}{12.63*3480}$ H_{as} = 3.04 cm2 ⇒ is adopt4HA10 (4HA10 =3.14cm2)

Calculation of the spacing:

St ≤ min(3h,33cm), We adopt a spacing of 25cm

Reinforcement of distribution :

Ar= $\frac{A}{4} = \frac{3.14}{4}$ = 0.79 cm2.

Has_s = 0.79 cm2 ⇒is adopt3HA10 (3HA10 = 2.36cm2) St ≤ min(4h,40cm), We adopt a spacing of

30cm, **Checking the shear** :

Has_s = 2.36 cm2

It has: Amin=0,23 xbxd (ft28/fe)

Amin= 1,57 cm2, As ≥ Amin .... Condition checked

Check the shear:

You must verify that $\tau_u \leq \tau_{ad}$

fc28 = 25MPa, γ_b = 1,5

$\overline{tu}$ ≤ = Min (0,2 fcj / yb ; 5MPa) Cracking little detrimental.

$\overline{tu}$≤ = Min (3.33 a ; 5MPa)

$\tau_u = \frac{Yo}{u\,b\,d} = \frac{3245}{1000 \times 130}$

τ_u = 0.02 KN/ cm2 = 0.2 MPa < 3.3 MPa

0.2 MPA < 1,16 MPA ' So no rebar cross-sectional'

Element	*As* (*cm*)	*Amin* (*cm*)			*Hasr* (*cm*)	
Span	**8.58**	**1.57**	**8HA12**	**9.05**	**2.26**	**3HA10**
Support	**3.04**	**1.57**	**4HA10**	**3.14**	**0.79**	**3HA10**

III.3.2- Study of the landing beam:

III.3.2.1-Pre-sizing:

The beam landing has a section (h x b)

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

L: distance between the bare beam landings.

L=2.70 m

Where

$\frac{270}{15} \leq$ h $\leq \frac{270}{10}$

It takes: h=30 cm

0.3 h$\leq b \leq$0.5 h → $9cm \leq b \leq$15cm

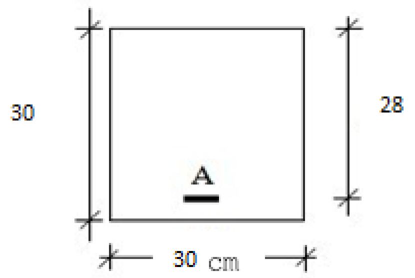
We take: b=30 cm

III.3.2.2- Verification (RPA 99/V2003 Art-7.5.1) **:**

b = 30*cm* ≤ 20*cm* .... *CV.*

h =30*cm* ≤ 30*cm*........................... *CV*

$\frac{h}{b}$ = 1≤ 4 CV



Therefore, the beam landing is in section (30x30) cm²

III.3.2.3-Evaluation of the loads :

The beam landing supports :

- its own weight : ....PP = 0,3.0.3 inches. 25 = 2.25 KN/ml
- the weight of the wall it supports : g_{wall} = 4.17 KN/ml
- G_{total}= PP + g_{wall}= 2.25 + 4.17 = 6.42 KN/ml
- the reaction of the staircase runs along beam bearing :
- SLU : $R_b = \frac{T_u}{b} = \frac{35.45}{1.10}$ = 32.27 KN/ml
- SLS : $R_b = \frac{T_u}{b} = \frac{23.29}{1.10}$ = 21.17 KN/ml

III.3.2.4-Combinations of loads :

- SLU:1,35 G + T_u
- SLS: G + T_s

q_u=1,35.6.42+35.45= 44.117KN / ml

q_s= 6.42+ 23.29 = 29.71 KN/ ml

III.3.2.5-Effect of bending :

$M_U = \frac{q_{u.L^2}}{8} = \frac{44.117\times(2.70)^2}{8}$ = 40.20 KN.ml

$T_U = \frac{q_{u.l}}{2} = \frac{44.117\times(2.70)}{2}$ = 59.558 kn /ml

Span

$M_t = 0.8 \times M_0 = 40.20 \times 0.8 = 32.16 \text{ KN. ml}$

On support

$M_t = 0.3 \times M_0 = 40.20 \times 0.3 = 12.06 \text{ KN. ml}$

III.3.3-Reinforcement :

h = 30 cm, b =30 cm ; d = 28 cm ; (Fe400) ; f_{bu}= 14.17 MPA

Table III.4: Reinforcement of the beam landing

Element	M ($KN.ml$)	μ	μ_r	α	β	$Has_{SCalculé}$ $(cm\ )$	$Has_{adopted}$	$A_s\,(cm)$
Span	32.16	0,096	0,392	0,126	0,949	3,47	4HA12	4,52
Support	12.06	0,036	0,392	0,046	0,981	1,30	2HA12	2,26

III.3.3-Verification of the condition of non - fragility (BAEL 91 Art-B. 6.4) :

It is necessary that the following condition is verified:

Span :

III.3.3.1-minimum Percentage :

$Has_S \geq A_{min}$ =(0.23× $f_{tj} \times b \times d)/f_e$= 1.014

Has_S= 4.52 cm^2 ≥1.014 cm^2...................................CV

On support :

III.3.3.2-minimum Percentage :

Has_S= 2.26 cm^2 ≥1.014 cm^2.................................CV

III.3.4-transverse Reinforcement :

Constraint tangent due to the shear force

The shade of the transverse reinforcement is of type $f_e \rightarrow feE\ 235\ Mpa$ according to BAEL(art A. 7.2.2)

$\tau_u = \frac{T_U}{(b \times d} = \frac{5956 \times 10}{300 \times 280} = 0{,}706 MPa$

The cracking is a little detrimental

III.4.Floor - to - body hollow

III.4.1-Introduction :

The floor is an area horizontally divided between two levels; it provides the functions of the following:

- Bears the loads and the overload of the buildings.
- Provides insulation, thermal and soundproofing between the different levels.
- Participates in the resistance of structures to the horizontal efforts

In our study, the floor is composed of the body being hollow and a slab of compression there including the ribbing (beams).

III.4.2-Characteristics of beams :

- the thickness of the body–hollow: e = 16cm
- the thickness of the slab of compression: h_0 = 4cm
- the width of the beam: $b = b_0 + 2b_1 = 10 + 2.(27{,}5) = 65$cm
- the body hollow used for the floor is to: L_0 = 55cm.

III.4.3-Evaluation of loads and overloads :

a). Floor terrace :

G = 0,65.559 = 363,35 daN/ml

Q = 0,65.100 = 65 daN/ml

Combination with the S. L. U

q_u = 1,35 G + 1,5 Q =588,02 daN/ml

Combination with the S. L. S

q_s = G + Q = 428,36 daN/ml

b). Floor floor current :

$G = 0{,}65 \times 484 =$

$314{,}6 daN / ml\ Q =$

$0{,}65 \times 150 =$

$97{,}5 daN / ml$

Combination with the S. L. U

$q_u = 1{,}35G + 1{,}5Q = 570{,}96 daN / ml$

Combination with the S. L. S

$q_S = G + Q = 412{,}10 daN / ml$

Table III.5:Evaluation of loads and overloads (Floor)

Type of flooring	G (daN/ml)	Q (daN/ml)	q_u (daN/ml)	q_s (daN/ml)
Floor terrace	363,35	65,0	588,02	428,35
Floor floor current	314,60	97,5	570,96	412,10

Not : it is preferred to estimate the loads (M and T) for the beams continuous by using a few methods to enrich the study ;

III.4.4-Different types of beams to study by the method :flat

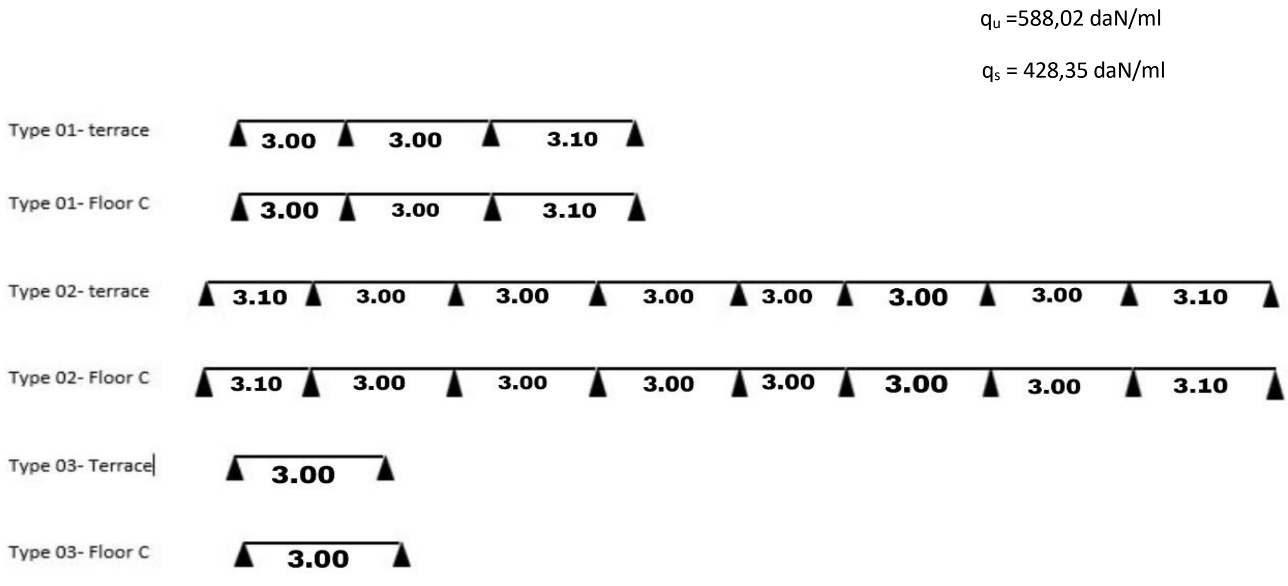


Figure III.3: schema static room current and terrace

Note: ***In this project, three types of floors are identified; however, the study will be limited to one specific type, which will be examined in detail.***

III.4.5-Area of validity of the method flat (BAEL 91 Art-B. 6.2.2) **:**

- the charge of operating Q≤ $\begin{Bmatrix} 2G \\ 5KN/m^2 \end{Bmatrix}$
- -the moment of inertia is constant in all the bays.
- -the relation between the spans of successive is understood between the 0.8 and 1.25 a
- The cracking is not harmful

- Q_t = 1,00 KN/m2 ≤ max (Gt = 7,26 KN/m2 5 KN/m2) = 7,26 KN/m2 ....(CV)
- Inertia constant in the different bays in continuity ....(CV)
- The worn successive are in a report included between the 0.8 and 1.25 a
- $\frac{Li}{Li+1} = \frac{3,10}{3,00}$ = 1,033 → 0,8≤ $\frac{Li}{Li+1}$ ≤1,25 .... (cv)
- $\frac{Li}{Li+1} = \frac{3,00}{3,00}$ = 1,00 → 0,8≤ $\frac{Li}{Li+1}$ ≤1,25 .... (cv)
- The cracking is not harmful... (CV)

III.4.5.1-Presentation of the method BAEL 91 :

It means each span by :

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

$$M_t + \frac{M_w+M_e}{2} \geq M_{max}\begin{Bmatrix} 1.05M_0 \\ (1+0.3\,\alpha)M_0 \end{Bmatrix}$$

$$M_t \geq M_{max}\begin{Bmatrix} \frac{1.2+0.3\alpha}{2}\ M_0\ if\ the\ travee\ is\ for\ bank \\ \frac{1+0.3\alpha}{2} M_0\ if\ the\ travee\ is\ of\ intermediate \end{Bmatrix}$$

The level of support :

$M_{has} \geq 0,6.M_0$...Beam with two spans

$M_{has} \geq$ of $0.5.M_0$..Support neighbor of the support of shore

$M_{has} \geq 0,4M_0$...Support through

$M_{has} \geq 0,2M_0$..Support of the bank (according to the conditions of the flush -)

With:

M_t: time in span

M_w: time to max on the support left M_e: time max on the support right M_a: time on support

M_0: the time isostatic

-Effort tranchant :

$T_w = T_0 + \frac{M_W-M_e}{L} = \frac{ql}{2} + \frac{M_w-M_e}{L}$

$T_w = -T_0 + \frac{M_W-M_e}{L} = -\frac{ql}{2} + \frac{M_w-M_e}{L}$

With:

T_w: effort - edged to the left of span Te: shearing force to the right of span

III.4.6-Determination of solicitations :

- **Type 02 – Terrace**

q_u = 588,02 daN/ml q_s = 428,35 daN/ml

$\alpha = \frac{Q}{G+Q} = \frac{65}{363.35+65} = 0.153$

- **Span 1-2 :**

1) $M_t + \frac{M_{w+}M_e}{2} \geq M_{max}\begin{Bmatrix} 1.05M_0 \\ (1+0.3\ \alpha)M_0 \end{Bmatrix}$,$M_t + \frac{(0.2+0.5)M_0}{2} \geq M_{max}\begin{Bmatrix} 1.05M_0 \\ (1+0.3*0.152)M_0 \end{Bmatrix}$

$M_t \geq M_{max}\begin{Bmatrix} 0.7M_0 \\ 0.695M_0 \end{Bmatrix} \rightarrow M_{,t} \geq 0.7M_0$

2) $M_t \geq \frac{(0.2+0.3)\alpha}{2}.M_0$, $M_t \geq \frac{(0.2+0.3)0.153}{2}.M_0 \rightarrow M_{,t} \geq 0.523.M_0$

We take the maxdes moments : $M_t = 0.7M_0$

- **Span 2-3 :**

1) $M_t + \frac{M_{w+}M_e}{2} \geq M_{max}\begin{Bmatrix} 1.05M_0 \\ (1+0.3\ \alpha)M_0 \end{Bmatrix}$, $M_t \geq M_{max}\begin{Bmatrix} 0.55M_0 \\ 0.545M_0 \end{Bmatrix} \rightarrow M_{,t} \geq 0.55M_0$

2) $M_t \geq \frac{(1+0.3)\alpha}{2}.M_0$, $M_t \geq \frac{(1+0.3)0.153}{2}.M_0 \rightarrow M_{,t} \geq 0.523.M_0$

It takes the max of moments : $M_t = 0.6M_0$

- **Span 3-4 :**

1) $M_t + \frac{M_w+M_e}{2} \geq M_{max}\begin{Bmatrix} 1.05M_0 \\ (1+0.3\,\alpha)M_0 \end{Bmatrix}$, $M_t \geq M_{max}\begin{Bmatrix} 0.55M_0 \\ 0.545M_0 \end{Bmatrix} \rightarrow M_{,t} \geq 0.55M_0$

2) $M_t \geq \frac{(1+0.3)\alpha}{2}.M_0$, $M_t \geq \frac{(1+0.3)0.153}{2}.M_0 \rightarrow M_{,t} \geq 0.523.M_0$

It takes the max of moments : $M_t = 0.6M_0$

- **Span4-5 :**
- $M_t + \frac{M_w+M_e}{2} \geq M_{max}\begin{Bmatrix} 1.05M_0 \\ (1+0.3\,\alpha)M_0 \end{Bmatrix}$, $M_t \geq M_{max}\begin{Bmatrix} 0.7M_0 \\ 0.695M_0 \end{Bmatrix} \rightarrow M_{,t} \geq 0.55M_0$
- $M_t \geq \frac{(1+0.3)\alpha}{2}.M_0$, $M_t \geq \frac{(1+0.3)0.153}{2}.M_0 \rightarrow M_{,t} \geq 0.623.M_0$

It takes the max of moments : $M_t = 0.7M_0$

III.4.6.1-Moments and efforts sharp isostatic :

- Span 1-2

L= 3.10 ; $M_{01} = \frac{qu.L^2}{8} = \frac{588,02.(3.10)^2}{8} = 706,359$ dan/m

$T_{01} = \frac{qu.L}{8} = \frac{588,02(3.10)}{2} = 911,431$ dan/m

-Span 2-3-4-5

L= 3.00 ; $M_{02} = \frac{qu.L^2}{8} = \frac{588,02.(3.00)^2}{8} = 661,523$ dan/m

$T_{02} = \frac{qu.L}{8} = \frac{588,02(3.00)}{2} = 882,030$ dan/m

III.4.6.2-Moments in spans (ELU)

- **Span 1-2:** $M_t = 0.7\ M_{01}$ =494,451 daN.m
- **Span 2-3:** $M_t = 0.6\ M_{02}$ =396,913 daN.m
- **Span 3-4:** $M_t = 0.6\ M_{03}$ = 396,913 daN.m
- **Span 4-5:** $M_t = 0.7\ M_{04}$ =463,066 daN.m

III.4.6.3-Times on support (ELU)

- **Support 1 :** $M_1 = 0.2\ M_1$ = 0,2.706,35 = 141,27 daN.m
- **Support 2 :** $M_2 = 0,5.\max\{M_1, M_2\}$ = 0,5.706,35 = 353,175 daN.m
- **Support 3 :** $M_3 = 0,5.\max\{M_2, M_3\}$ = 0,5.661,523 = 330,761 daN.m
- **Support 4 :** $M_3 = 0,5.\max\{M_3, M_4\}$ = 0,5. 661,523 = 330,761 daN.m
- **Support 5 :** $M_4 = 0.2\ M_{04}$ = 0,2 .661,523 = 132,304 daN.m

III.4.6.4-Efforts - edged (SLU) :

- **Span 1-2**

$T_e = -911,431 + \frac{141.270 - 353,175}{3,10} = 843.074 daN$

$T_e = -911,431 + \frac{141.270 - 353,175}{3,10} = -979.78 daN$

- **Span 2-3 :**

$T_e = 882.03 + \frac{353,175 - 330.761}{3,00} = 889.501 daN$

$T_e = -882.03 + \frac{353,175 - 330.761}{3,00} = 874.559 daN$

- **Span 3-4 :**

$T_e = 882.03 + \frac{330.761 - 330.761}{3,00} = 882.03 daN$

$T_e = -882.03 + \frac{330.761 - 330.761}{3,00} = -882.03 daN$

Table III.6: Calculation of stresses to the ELU

Span	Moment in span (daN/m)	L(m)	Effort edge(daN)		Support (daN/m)	Time to support
			T_w	T_e		
1-2	494,451	3,10	843,074	-979,78	M_1	141,270
2-3	396,913	3,00	889,501	-874,559	M_2	353,175
3-4	396,913	3,00	882,013	-882,013	M_3	330,761
4-5	463,066	3,00	948,182	-815,878	M_4	330,761
					M_5	132,304

To calculate the loads to the state limit of service, it is sufficient to multiply the results obtained by the ratio : $(q_s / q_u) = (428,35/588,02) = 0,728$

III.4.6.5-Time in the bays and on the support and the effort sharp (ELS) :

Span: $M_{ts} = 0,728 \times M_{tu}$

$T_{ws} = 0,728 \times T_{wu}$

$T_{es} = 0,728 \times T_{eu}$

Support : $M_{as} = 0,728 \times M_{at}$

$T_{ws} = 0,728 \times T_{wu}$

Table III.7:Calculation of stresses in SLE

Span	Moment in span (daN/m)	L(m)	Effort sharp (daN)		Support (daN/m)	Time to support
			T_w	T_e		
1-2	360,180	3,10	613,757	-713,270	M_1	102,8440

2-3	288,952	3,00	647,557	-636,679	M_2	257,1100
3-4	288,952	3,00	642,118	-642,118	M_3	178,0760
4-5	337,325	3,00	690,275	-593,959	M_4	265,4720
					M_5	106,18899

III.4.7-Reinforcement :

- **In span :**

The beam is calculated as a section in the "T" submitted to the flexion simple solicited by a time max: M_{max} = 494,451 daN.m

- Coating : C = 2 cm.

III.4.7.1- Times bending M_t balanced by the table :

M_{table}= 494.451 daN.m

Therefore:

$M_{table} > M_{max}$, then a part of the table is only compressed and the section in the " T " will be calculated as a rectangular section whose width b = 65cm and the useful height d =18cm subjected to simple bending.

$\mu = \frac{M_t}{b*d^2*fbc} = \frac{494.451*10}{65*18^2*14.17} = 0.016 \leq \mu_R = 0.392 \rightarrow A'_s = 0$

$\alpha = 1,25(1 - \sqrt{1-2\mu}) = 0,0201$

$\beta = 1 - 0,4\alpha = 0,991$

$\sigma s = \frac{fe}{ys} = \frac{400}{1.15} = 347.826$ MPA

$As = \frac{M_t}{\beta*d*\sigma s} = \frac{510.524*10}{0.199*18*347.826} = 0.82\ cm^2$

III.4.7.2- Condition of non - fragility :

$A_{min} = 0,23.b.d.\frac{fc28}{fe} = 0,23.65.18.\frac{2.1}{400} \rightarrow A_{min} = 1.412\ cm^2 > A_s$ .... (cv)

Then it takes: **A_s=2HA10=1,57 cm2**

-On support

Ma = - 353,175 daN.m

The table is located in the area tense (when negative), the section in the " T " will be calculated as a rectangular section of width b = 10cm and a useful height d = 18 cm

Element	M (daN.m)	μ	α	β	A_s (cm^2)	(A'$_s$ (cm^2)
Support	**353,175**	**0,0769**	**0,100**	**0,959**	**0,588**	**0**

III.4.7.3-Condition of non - fragility

A_{min} =0,23.b.d.$\frac{fc28}{fe}$ = 0,23.10.18.$\frac{2.1}{400}$ → A_{min} =0.22 cm^2>A_s ....(cv)

Then in takes: **A=1HA12=1,131 cm**

III.4.8-Verifications :

In order to generalize the checks to the different types of beams, it will be taken as a solicitation maximum value:

Table III.8: the values of maximum of solicitations

State	ELU	ELS
$M_{t\,max}(daN.m)$	494,451	360, 180
$M_{a\,max}(daN.m)$	353,175	265,472
$T_{max}(daN)$	948,182	690,275
$M_{0\,max}(daN.m)$	706,359	514,555

III.4.8.1-Verification of constraints :

For a cracking little harmful, it there was no audit to perform in this that relates to σ_s (steel).

When the constraints of compression of the concrete and of traction of the frames are checked, the calculation of The E. L. S is not necessary.

III.4.8.2-Stresses tangential :

T_{max}=948,182 daN

$$Tu = \frac{948,182*10^{-5}}{0.10*0.18} = 0.553 \text{ MPA}$$

Cracking Peupréjudiciable : τ_{ul}=min $\left\{0.20 \times \frac{fcj}{yb}\ ; 5MPA\right\}$= 3.33 MPA

Tu=0.20 × $\frac{fcj}{yb}$ = 3.33 MPA=0.553 MPa

III.4.8.3-Armatures transversales :

Diameter of the reinforcement cross-sectional (BAEL 91 Art-A. 7.2.2)

$$\emptyset_t \leq \min \left\{ \begin{array}{c} \frac{h}{35} = 5.71\ mm \\ \emptyset_{min} = 10\ mm\ it\ takes\ , \emptyset = 6mm \\ \frac{b_0}{12} = 8.33\ mm \end{array} \right\}$$

$$\frac{A_t}{b_0.S_t} \cdot \frac{fe}{ys} \frac{\tau_{ul-0.3*k*f_{t28}}}{0.9(\text{sina}+\text{cosa})}$$

K=1 (coefficient of concreting)

$\alpha = 90°$ (α : Inclination of the steel cross-sectional)

III.4.8.4-Percentage minimum :

$\frac{A_t}{b_0 . S_t} . \frac{fe}{ys} > 0.4\ MPA \rightarrow \frac{A_t}{S_t} > \frac{0.4*10}{235} = 0.017\ cm^2$

A_t/S_t= max (-0,0034,0,017) = 0,017 cm2/cm

III.4.8.5-Spacing of stirrups (BAEL Art-A. 5.2) :

$S_t \leq (0{,}9\ d = 16.2\ cm, 40\ cm) = 16.2\ cm$

It takes: **S_t=15 cm**

$0{,}017\ cm2/cm \Rightarrow A_t = 0{,}017 . 15 = 0{,}255\ cm2$

$\frac{A_t}{S_t} = 0{,}017\ cm2/cm \Rightarrow A_t = 0{,}017 . 15 = 0{,}255\ cm2$

It takes: **A_t=2ϕ 6=0,57cm2**

III.4.9-Verification of the deformation (BAEL Art-B. 6.5.1) :

We may admit that it is not necessary to process the calculation of the arrow if the conditions below are met:

1) $\frac{h}{L} > \frac{1}{22.5}$
2) $\frac{h}{L} > \frac{1}{15} . \frac{M_{max}}{M_0}$
3) $\frac{A_s}{b_0 . d} < \frac{3.6}{fe}$

Where,

1) $\frac{20}{310} = to\ 0.064 > \frac{1}{22.5} = 0{,}04$ ....(cv)
2) $\frac{20}{310} = \text{to } 0.064 > \frac{1}{15} \frac{514{,}555}{706{,}359} = 0{,}04$ ....(cv)
3) $\frac{1{,}57}{18.10} = 0{,}0087 < \frac{3{,}6}{400} = 0{,}009$ ....(cv)

Therefore, the calculation of the arrow is not necessary

III.5.Conclusion:

Through this chapter, a rigorous methodology was applied for the analysis and dimensioning of the structural elements of the building, ensuring its stability in accordance with the standards in force. The study has highlighted the importance of a precise breakdown of the expenses and the use of computer software to optimize the reliability of the results. These results are a key step towards developing detailed implementation plans.

Chapter IV :

Seismic Study of the Structure

IV Seismic Study of the Structure

IV.1 Introduction

The earthquake is the natural phenomenon of the most destruction and the most difficult to predict when it hits, and with what intensity it is going to shake our constructions. The earthquake is the vibration of the ground caused by a sudden strain of energy accumulated in the crust of the earth or in the underlying layer called the mantle. This natural phenomenon can dig up the human and material losses which round the study of behavior of structure sounds the effect of the activities dynamic due to the earthquake is mandatory and must be justified according to the rules of earthquake in Algeria. Our work consists of the study and design of a building (R+4) residential brace by the sail and portals with justification of interaction portals, located in Annaba. This is an area classified by the RPA 99/version 2003 come zone seismicity and a site closes (S3).

Goals of the dynamic analysis:

Determination of the characteristics dynamic equity of the structure.

To determine the modes and periods. **ROBOT** considers a model skewer inserted into the base, where the masses are considered concentrated at the level of each floor.

The mass of the floors is calculated in a way to include a part of the overload operation...... (β=0,2) Tab.4.5. RPA99-v2003.

IV.2 Presentation of the different methods of estimation of the seismic forces:

SelonRPA99/version2003 the calculation of the forces seismic can be carried out following three methods: The method static equivalent.

The method of _analyse modal spectral.

The method of _analyse dynamic by accelero-gram.

IV.2.1 Method static equivalent : (Article 4.2 of the RPA99/2003)

a) Principle :

The real forces dynamics that developed in the construction were replaced by a system of forces, static mock the effects of which considered those of the seismic action. The seismic forces horizontal equivalent will be considered applied successively in two directions of the principal axes of the horizontal plane; only the fundamental mode of vibration of the structure is to be considered in the calculation of the seismic force total.

b) Conditions of application :

b.1) The building or the block studied met the conditions of regularity in plan and in elevation prescribed in chapter III, paragraph 3.5 (RPA99/Version 2003) with a height at most equal to 65 m in zones I and II and to 30 m in areas III.

b.2) The building or block studied presents an irregular configuration, while respecting, in addition to the conditions of the height stated in (a), the following additional terms:

- **Area I:** All groups.
- **Area II:** Group use 3 - Group usage 2: If the height is less than or equal to 7 levels or 23m. Group use 1B: If the height is less than or equal to 5 levels or 17m. Group use 1A: If the height is less than or equal to 3 levels or 10m.
- **Area III:** Groups, use 3 and 2: If height is less than or equal to 5 levels or 17m. Group use 1B: If the height is less than or equal to 3 levels or 10m. Group use 1A: If the height is less than or equal to 2 levels or 8m.

c) Method of modeling :

- The building model to use in each of the two directions of calculation is plan with the masses concentrated at the center of gravity of the floors, and a single degree of freedom in the horizontal translation by level, subject to the bracing systems in the two (2) directions can be decoupled.
- The lateral stiffness of the bearing elements of the system of bracing is calculated from sections of non-cracked reinforced concrete structures or masonry.
- Only the fundamental mode of vibration of the structure is considered in the calculation of the seismic force total.

IV.2.2 Method dynamic modal spectral :

a) Principle of the method of dynamic modal spectral :

According to the RPA, the method of modal analysis spectral can be used in all cases, and in particular, in the case where the static method equivalent is permitted. By this method, it searched for each mode of vibration, the maximum of effects generated in the structure by the seismic forces represented by a response spectrum calculation.

These effects are the result combined to obtain the response of the structure.

b) Modeling :

b.1) For the structures and regular in plan with floors rigid, the analysis was done separately in each of the two main directions of the building. This is represented in each of the two directions of calculation by a model plan, recessed at the base and where the masses are concentrated at the level of the centres of gravity of the floors with a single DDL in horizontal translation.

b.2) For irregular structures in the plan, subject to torsion and with floors rigid, they are represented by a three-dimensional model, embedded in the base and where the masses are concentrated at the level of the centres of gravity of the floors with three (03) DDL (2 translations and horizontal rotation around the vertical axis).

b.3) For the structures, regular or non-regular with the floors of flexible, they are represented by

b.4) Three-dimensional models built at the base and several DDL by floor.

b.5) The deformability of the foundation soil must be taken into account in the model, at all times when the response of the structure depends significantly.

b.6) The model of the building to be used should best represent the distributions of stiffnesses and masses in a way to take into account all modes of deformation that are significant in the calculation of the inertia forces, seismic (ex: the contribution of nodal areas and nonstructural components to the rigidity of the building).In the case of buildings, reinforced concrete or masonry stiffness of the bearing elements should calculated by considering the sections not cracked.

b.7) If the displacements are critical, especially in the case of structures associated with high values of the coefficient of behavior, an estimate the most accurate of the stiffness becomes necessary by taking into account sections cracked.

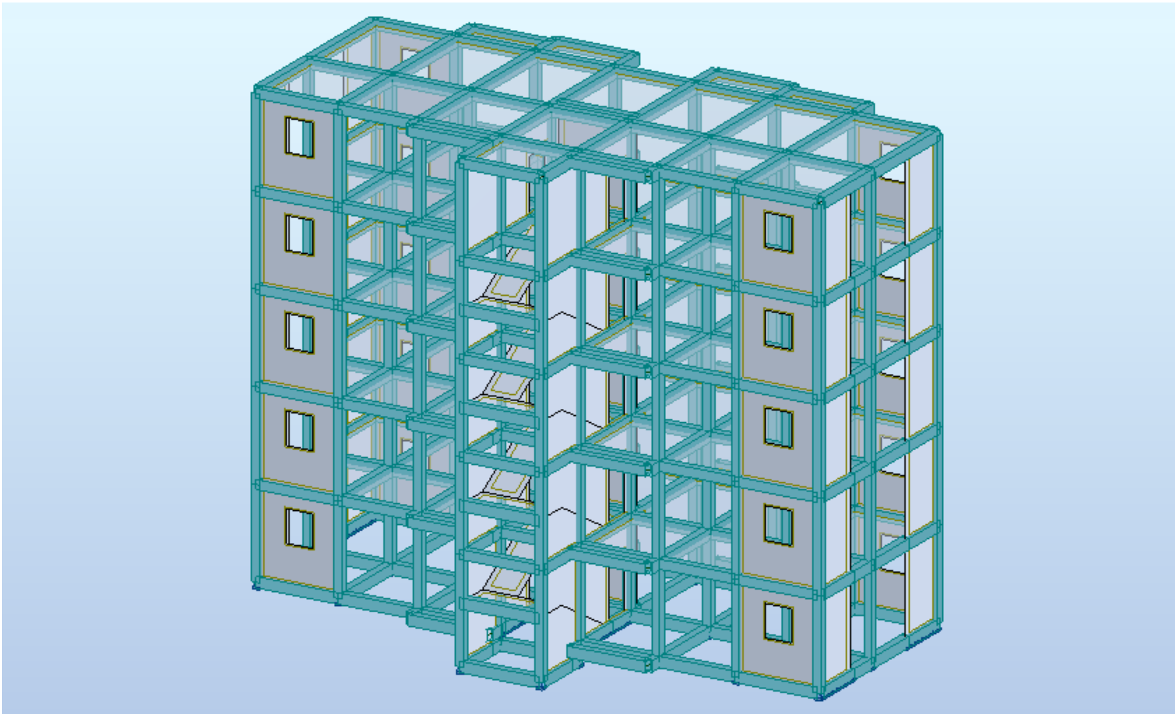


Figure IV.1 : View 3D of structure

IV.3 Calculation of the stress-edged sword with the method static equivalent :

$$V = \frac{A \times D \times Q}{R} \times W$$ ……………………………………………RPA 99 [formula 4-1]

…ent of acceleration of the area (table 4 -1 of RPA 99).

D: Amplification Factor dynamic way, depending on the site category, the correction factor damping (ξ) and the fundamental period of the structure (T) (table 4-2 RPA 99).

R: Coefficient of behavior (table 4 -3 RPA 99).

Q: Factor of quality (table 4 – 4 of RPA 99).

W: Total Weight of the structure.

Definitions of the value of the coefficients:

- **Coefficient of acceleration of area (A):** Given by table 4.1 (APR.99/03) following the seismic zone and the use of the building.

Table IV.1: Coefficient of Acceleration of Zone A.

	ZONE			
Group	**I**	**IIa**	**IIb**	**III**
1A	0.15	0.25	0.30	0.40
1B	0.12	0.20	0.25	0.30
2	0.10	**0.15**	0.20	0.25
3	0.07	0.10	0.14	0.18

Our building is located in the region of Mila (**zone IIa**) and belongs to the **group 2**. for our case we have: **A= 0.15**

- **Coefficient of behavior global of structure (R):**

Its value is unique; is given by the table 4.3. RPA99 in function of the system of bracing.

Table IV.2: Values of the coefficient of behavior A.

Concrete armed.	Coefficient of behavior (R) .
Porticos self-stable without fills in masonry rigid.	5
Porticoes cars-tables with fillings masonry rigid.	3.5
Sails holders.	3.5
Kernel.	3.5
Mixed gantry cranes/sails with interaction.	**5**
portal frames braced by the sails.	4
Console vertical to the masses divided.	2
Clock reverse.	2

System of bracing mixed gantry cranes/sails with interaction : **R= 5**

- **Factor of quality Q :**

Q: Factor of quality is a factor related to the quality of the system of bracing of the structure. It penalizes the poor designs earthquakes (see RPA99/2003).

The factor of quality of the structure is based on :

The redundancy and the geometry of the elements which form. The regularity in plan and elevation.

The quality control of the construction.

The value of Q is determined by the formula:

$$Q = 1 + \sum_{q=1}^{6} p_q$$ .... Section 4.4 (RPA99/2003).

Table IV.3 : Value of factor of quality.

Criterion: q	**Pq -**
1. Conditions minimum on the files of bracing.	0.05
2. Redundancy in the plan.	0.05
3. Regularity in plan.	0
4. Regularity in elevation.	0
5. Control the quality of the materials.	0.05
6. Control the quality of the execution.	0
$\sum P_{q.}$	0.15

Q=1+0.05+0.05+00+00+0.05+0.10 =1.2

- **Dynamic amplification factor:**

D: mplification factor dynamic way, depending on the site category, the factor of correction damping **(ξ)** and the fundamental period of the structure (T).

$$D = \begin{cases} 2.5\eta & 0 \leq T \leq T_2 \\ 2.5\eta (T_2/T)^{2/3} & T_2 \leq T \leq 3s \\ 2.5\eta (T_2/3)^{2/3}(3/T)^{5/3} & T \geq 3s \end{cases}$$

T_2:A period feature, associated withe the category of the site and given by (Table 4.7 of the RPA99/2003).

η:Factor of correction of depreciation given by the formula:

$\eta = \sqrt{\frac{7}{2+\xi}}$...RPA99/V2003 (Formula 4.3)

a.1) Percentage of amortization critical :

The percentage of depreciation is critical in the function of material constitutive, of the type of structure ,and the importance of fillings.

Table IV.4: Values of the coefficient of damping according to the system structure.

Filling	Gantry		Sails or walls
	Concrete reinforced	**Steel**	**Concrete reinforced/masonry**
Lightweight	6	4	**10**
Dense	**7**	5	

Therefore : 6 %

$$\eta = \sqrt{\frac{\mathbf{7}}{\mathbf{2} + \boldsymbol{\xi}}} = \mathbf{0.93}$$

IV.4 Modeling and results :

IV.4.1 Modeling :

One of the most important steps during a dynamic analysis of the structure is modeling appropriate to the latter. View of the complexity and volume calculation that requires the analysis of our structure, the necessity of the use of the tool is necessary. The software used to model our structure is Autodesk Robot Structural Analysis Professional 2021. The modelled structure is represented in figure IV.1.

IV.4.2 Analysis of the structure :

a) Interpretation :

According to the table above, one finds that which follows:

- The 1st mode is a mode of translation according to the axis Y.
- 2nd mode is a mode of torsion.
- 3rd is a mode of translation according to the x-axis.

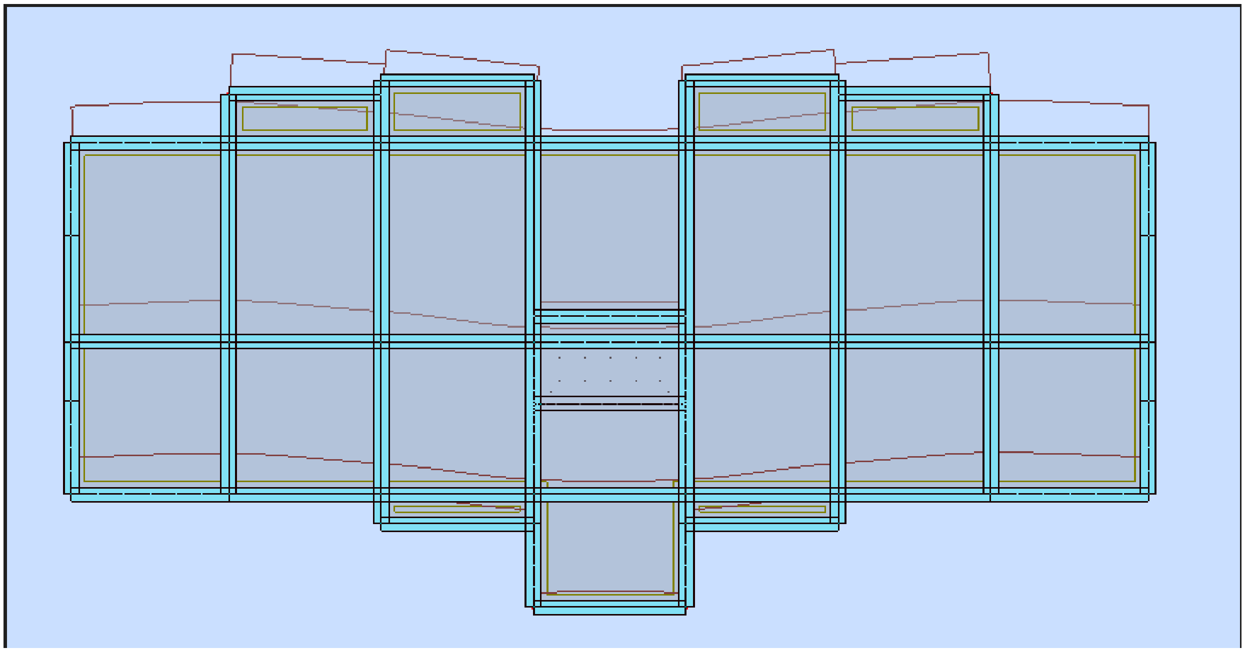


Figure IV.2: First mode of vibration, View 3D and view in plan

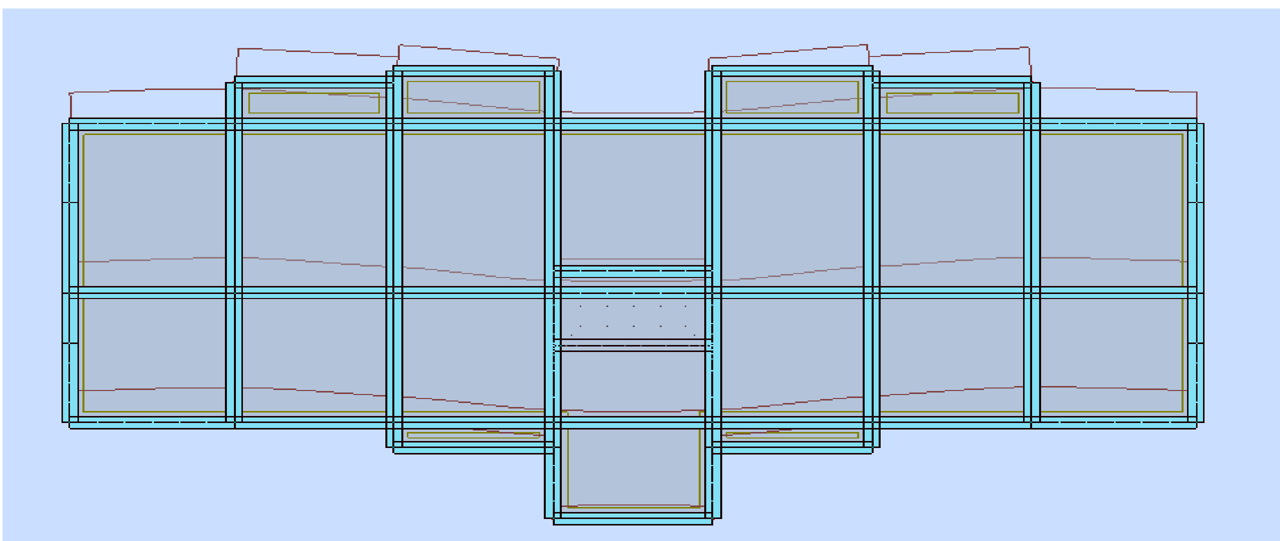


Figure IV.3: Second mode of vibration. View 3D view and in plan.

b.1) Results of the analysis of dynamics for ROBOT :

Table IV.5: typical Picture of the period and the mass participant found by software.

Mode	Period [sec]	Mass Modal UX [%]	Masses Cumulative UX [%]	Mass Modal UY [%]	Mass Accumulated UY [%]	Tot. mas. UX [kg]
1	**0,37**	0,00	0,00	**75,85**	75,85	1444506,79
2	**0,31**	0,06	0,06	0,00	75,85	1444506,79
3	**0,26**	**73,61**	73,67	0,00	75,85	1444506,79
4	0,10	0,00	73,67	14,84	90,69	1444506,79
5	0,08	0,03	73,70	0,00	90,69	1444506,79
6	0,07	18,62	92,31	0,00	90,69	1444506,79
7	0,06	0,01	**92,32**	0,11	**90,80**	1444506,79
8	0,06	0,00	**92,32**	0,11	**90,91**	1444506,79
9	0,06	0,00	**92,32**	0,00	**90,91**	1444506,79

b.2) Interpretation :

According to the table above, one finds that which follows :

- The 1st mode is a mode of translation according to the axis **Y (75.85%)** of participation of mass modal.
- 2nd mode is a mode of torsion.
- 3rd is a mode of translation along the axis **X (73.61%)** of mass participation modal

It finds that it takes 6 modes to wait for **90%** of participation of the masses, modal required by the RPA 99 VERSION 2003 ART 4.3.4.a.

We see that the majority of the modes are independent modes of translation that are not coupled with the torsion).

c) Conclusion :

For this reason, we adopt this structural design that gives the best performance of our structure vis-à-vis the earthquake.

IV.5 Calculation of the Shares of Seismic :

According to the Method Static Equivalent **(RPA99/version 2003 art 4.3.6):**

The resultant of the forces seismic base (**V_t**) obtained by combination of the values of the modal must not be less than **80%** of the resultant force of seismic determined by the static method, the equivalent of **V** for a value of the fundamental period given by the empirical formula appropriate.

If **$V_{T\text{-software}} \leq 0{,}80\ V_{MSE}$**, it will increase all the response parameters (forces, displacements, moments, ...) in the ratio of **$0.8\ V/V_t$**

The period empirical " T " :

The period finding with software should be between the period of empirical and one plus.

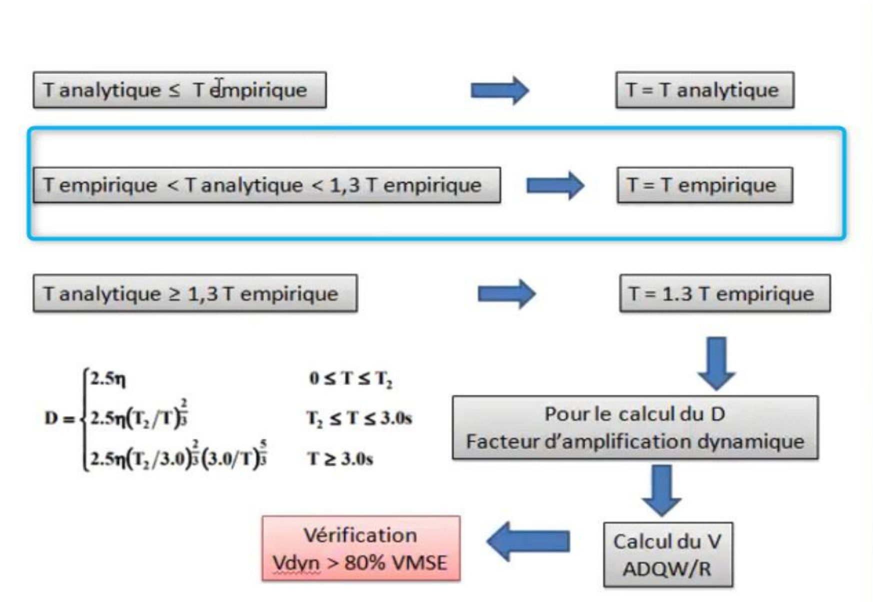


Figure IV.4 : Relationship between the period of empirical and analytical.

Table IV.6 : Values of the coefficient C_t

1	Gantry auto-stable concrete reinforced without filling in masonry.	0.075
2	Gantry auto-stable in steel without filling in masonry.	0.085
3	Porticos self-stable in concrete armed or in steel with filler in masonry.	0.050
4	Bracing insured partially or totally by the sails in Concrete armed, of pale triangulated and the walls in masonry.	**0.050**

- **Meaning longitudinal**

The formula empirical to use according to the case is the following:

1. $T = C_t . h_n^{3/4}$ =0.05*(17)$^{3/4}$ = 0.42s

With : l_x =21. 5m $T = \frac{0.09h}{\sqrt{lx}}$

$$T = \frac{0.09 \times 17}{\sqrt{29.65}} = 0.32s$$

$T_{emperique}$ = min (0.42s ; 0.32s) = 0.32s

$1.3 \times T_{emperique} = 0.41s$

$T_{analytical} = 0.26s$ ...... (Table IV.5)

Transverse direction. The empirical formula to use according to the case is the following:

1 . $\boldsymbol{T = C_t \times h_n^{\frac{3}{4}}}$

T==0.05*(17)$^{3/4}$ = 0.42s

2. l_y=11m $T = \frac{0.09h}{\sqrt{ly}}$

$$T = \frac{0.09 \times 17}{\sqrt{11}} = 0.46$$

$T_{emperique}$ = min (0.37;0.46) = 0.37 s

$1.3 \times T_{emperique} = 0.48s$

$T_{analytical} = 0.37s$ ..... (Table IV.5)

calculation of the dynamic amplification factor D :

T_1 and T_2: period characteristics are related to the category of site **(table 4.7).**

Table IV.7: period characteristics related to the category of site.

Site	S1	S2	S3	S4
T1	0.15	0.15	**0.15**	0.15
T2	0.30 To	0.40	**0.50**	0.70

$S_3 \Rightarrow$ **T_1=0.15** and **T_2=0.5.** One obtains

- **Meaning longitudinal :**

The condition: $0 \le T \le T_2$

Therefore: $D = 2.5\eta = 2.33$

- **Meaning transversal :**

The condition: $0 \leq T \leq T_2$

Therefore: $D = 2.5\eta = 2.33$

Calculation of the total seismic force "V":

x Direction :

$$V_x = \frac{H.D.Q}{R} W = \frac{0.15 \times 2.33 \times 1.2}{5} \times 1444506.79 \Rightarrow V_x = 1211.65KN$$

0.8V$_x$= 969.82 KN

The Meaning Is :

$$V_y = \frac{H.D.Q}{R} W = \frac{0.15 \times 2.33 \times 1.2}{5} \times 1444506.79 \Rightarrow V_y = 1211.65KN$$

0.8V$_y$= 969.82 KN

Results of the action of seismic to the base found by **software ROBOT** in two senses:

$V_X = 1066.60KN$

$V_Y = 1012.63KN$

- **Summary of results :**

Table IV.8: Summary of results

Parameters	A	Q	R	W (KN)	D	V(KN)	0.8 V(KN)
SensX	0.15	1.2	5	1444506.79	2.33	1016.64	969.32
SensY	0.15	1.2	5	1444506.79	2.33	972.44	969.32

- **Verification of the shear force at the base:**

Table IV.9: Verification of the shear force at the base.

The sense	0.8 V_{static} (KN)	$V_{dynamic}$ (KN)	$V_{dynamic}$> 80% V_{static}
Next X	969.32	1066.60	Condition checked
Following Are	969.32	1012.63	Condition checked

- **Verification of mass participating :**

This helps us to know the percentage of the masses acting on the supporting elements to the last mode. According to the table (Table IV.5), the table is representative of the period and the mass participant found by software.

Comparison of results: the values found for the Masses Cumulative UX [%], the Masses of Accumulated UY [%] are higher than 90%; therefore, the condition of mass participant is checked.
Min (Masses Cumulative UX [%], the Masses of Accumulated UY [%]) ≥ 90%. Min (92.18; 90.69≥ 90%<=>90.69≥ 90% =>**Condition is verified**.

- **Verification of the interaction sails-gantry** :

Article (3-4-4-(a) the RPA99/version 2003 requires that for the construction bracing mixed with justification of the interaction, the sails of bracing must take at most 20% of the stresses due to vertical loads; the horizontal loads are taken jointly by the sails and the gantry (at least 25% of the effort-edged floor).

Table IV.10: effort Normal.

	Effort normal to the SAILS	**Effort normal of COLUMNS**	**Effort normal TOTAL**
Floor	**N**	**N**	**N**
1	-2494,95	-15084,82	-17579.77
2	-2064,05	-12177,28	-4241.33
3	-1624,49	-9285,42	-10909.91
4	-1170,06	-6420,6	-7590.66
5	-678,44	-3587,81	-4266.25

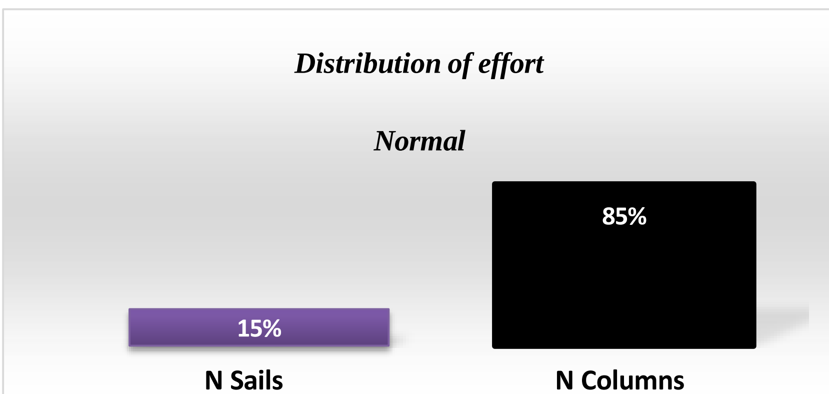


Figure IV.5: Distribution of the effort of Normal

Table IV.11: effort Normal and stress on the Column.

	Columns		
	Effort tranchant		Effort normal
Floor	Tx	Ty	N
1	205,28	344,02	-15084,82
2	193,41	427,57	-12177,28
3	227,62	455,59	-9285,42
4	225,41	408,41	-6420,6
5	249,47	424,74	-3587,81

Table IV.12: Normal stress and the shear force on the sails of bracing.

	Sails of bracing		
	Effort tranchant		Effort normal
Floor	Tx	Ty	N
1	861,36	668,61	-2494,95
2	800,88	532,1	-2064,05
3	632,36	381,87	-1624,49
4	475,83	260,67	-1170,06
5	223,83	10,46	-678,44

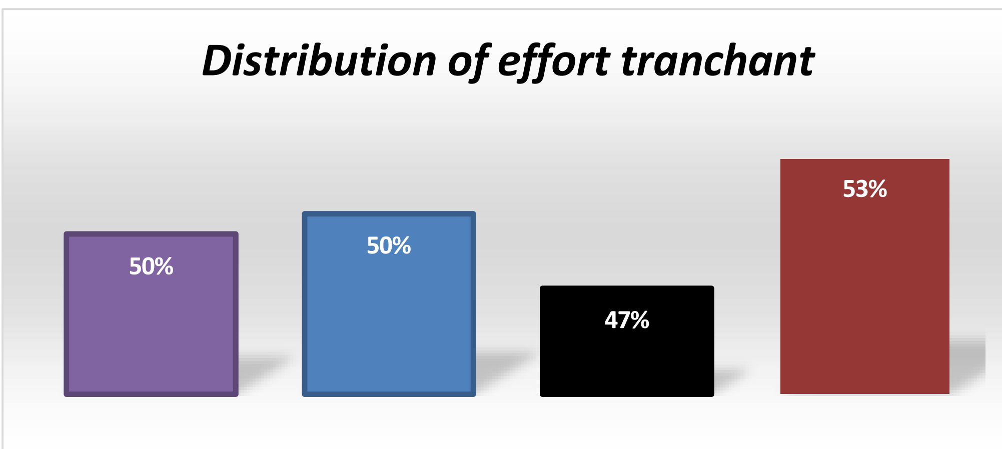


Figure IV.6: Distribution of the effort - edged sword.

- **Verification of the displacement :**

The moving horizontally at each level " k " of the structure is calculated as follows:

$\delta k = R \cdot \delta ck$. δck: Displacement due to the forces of seismic - Fi.

R: coefficient of behavior =5.

ΔK: the displacement relative to the level of " k " by report at the level of " k-1 " is equal to $\Delta K = \delta k - \delta k-1$

Cas/Etage	UX [cm]	UY [cm]	dr UX [cm]	dr UY [cm]	d UX	d UY	Max UX [cm]	Max UY [cm]	Min UX [cm]	Min UY [cm]
4/ 1	0,116	0,014	0,116	0,014	0,00	0,00	0,132	0,041	0,0	0,0
4/ 2	0,322	0,040	0,206	0,026	0,00	0,00	0,371	0,084	0,101	0,002
4/ 3	0,572	0,070	0,250	0,030	0,00	0,00	0,658	0,148	0,276	0,003
4/ 4	0,835	0,099	0,263	0,030	0,00	0,00	0,958	0,211	0,490	0,006
4/ 5	1,093	0,123	0,258	0,024	0,00	0,00	1,246	0,270	0,718	0,024

Figure. IV.7: Summary of the results of the ROBOT.

Table IV.13: Verification of the displacement in the direction x-x.

	dr. UX [cm]	**1/100 h[cm]**	**Verification**
DRC	**0.116**	**3. 4**	**CV**
1	**0.206**	**3.4**	**CV**
2	**0.250**	**3. 4**	**CV**
3	**0.263**	**3.4**	**CV**
4	**0.258**	**3.4**	**CV**

Cas/Etage	UX [cm]	UY [cm]	dr UX [cm]	dr UY [cm]	d UX	d UY	Max UX [cm]	Max UY [cm]	Min UX [cm]	Min UY [cm]
5/ 1	0,012	0,228	0,012	0,228	0,00	0,00	0,044	0,260	0,0	0,0
5/ 2	0,037	0,653	0,025	0,426	0,00	0,00	0,066	0,752	0,000	0,192
5/ 3	0,066	1,128	0,029	0,475	0,00	0,00	0,109	1,306	0,001	0,552
5/ 4	0,095	1,579	0,029	0,450	0,00	0,00	0,157	1,834	0,002	0,949
5/ 5	0,122	1,975	0,027	0,397	0,00	0,00	0,199	2,301	0,002	1,323

Figure IV.8: Summary of the results of the ROBOT.

Table IV.14: Verification of displacements in the y-y direction.

	dr. UY [cm]	1/100 h[cm]	Verification
DRC	**0.228**	**3.4**	**CV**
1	**0.426**	**3.4**	**CV**
2	**0.475**	**3.4**	**CV**
3	**0.450**	**3.4**	**CV**
4	**0.397**	**3.4**	**CV**

Based on the above table, we find that the displacement relative levels are lower than the hundredth of the storey height.

- **Checking for accidental eccentricity: (RPA art 4.3.7)** [**1**]

- In our three-dimensional analysis, the effects of accidental torsion of the vertical axis must be taken into account ±0.05 L (L is the dimension of the floor perpendicular to the direction of the seismic action); this value must be applied at the level of the floor that is considered to be following each direction.Meaning of X : accidental = 0.05 x L_{xi}

- Sens Y : accidental = 0,05 x L_{yi}

The calculation of the centers of mass of each element in the structure makes it possible to determine the coordinates of the eccentricity mass.

The coordinates of the center of mass are data by:

$$X_G = \frac{\sum M_i \times X_i}{\sum M_i}$$

$$Y_G = \frac{\sum M_i \times Y_i}{\sum M_i}$$

With :

Mi: mass of the element "i" of the considered level. Xi, Yi-Coordinates of the center of gravity of the element "i" relative to the global coordinates.

The automatic analysis by the software **ROBOT** has given results which are illustrated in the following table:

Cas/Etage	Nom	Masse [kg]	G (x,y,z) [m]	R (x,y,z) [m]	Ix [kgm2]	Iy [kgm2]	Iz [kgm2]	ex0 [m]	ey0 [m]	ex2 [m]	ey2 [m]
5/ 1	Etage 1	275823,71	14,83 8,48 2,99	14,82 8,39 2,25	6565381,92	19209264,73	25701279,84	0,00	0,10	1,48	0,90
5/ 2	Etage 2	275823,71	14,83 8,48 6,05	14,82 8,39 5,31	6565381,92	19209264,73	25701279,84	0,00	0,10	1,48	0,90
5/ 3	Etage 3	275575,71	14,83 8,49 9,11	14,82 8,39 8,37	6553733,03	19203364,36	25683733,17	0,01	0,10	1,48	0,90
5/ 4	Etage 4	275823,71	4,83 8,48 12,17	4,82 8,39 11,43	6565381,92	19209264,73	25701279,84	0,00	0,10	1,48	0,90
5/ 5	Etage 5	341459,97	4,83 8,37 15,30	4,83 8,30 14,53	8321305,71	24282720,52	32604026,23	0,00	0,07	1,48	0,90

Figure IV.9: Summary of the results of the **ROBOT**.

Table IV.15: Verification of the eccentricity.

e_x (m)	e_y (m)	0.05 L_X(m)	0.05 L_Y(m)
0.00	0.10	1.07	0.55
0.00	0.10	1.07	0.55
0.01	0.10	1.07	0.55
0.00	0.10	1.07	0.55
0.00	0.07	1.07	0.55

- **overthrow check: (RPA99/V2003 Art.5.5.) [1]:**

The moment of a reversal that may be caused by the action of seismic shall be calculated by comparison with the level of contact soil-foundation. The moment stabilizer will be calculated by taking into account the total weight equivalent to the weight of the construction, the weight of the foundation and possibly the weight of the embankment. $M_s > M_r$.

With :

M_s : stabilizer moment; **M_r :** overturning moment.

This equilibrium condition refers to the stability of the whole of the building or of the book. Subject to the effects of reversal and/or to slip.

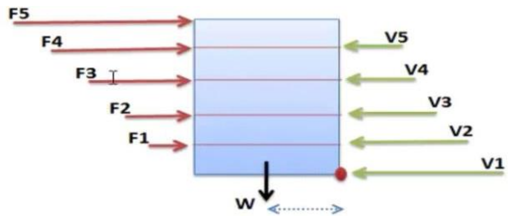


Figure IV.10: Time stabilizing and moments of reversals.

$$M_{\text{reversal}} = \sum_{n-1}^{n} F_i \times d_i$$

$$M_{\text{stabilizer}} = W \times b$$

Table IV.16: Checking the reversal in the direction x-x

FLOOR	Fx	Vx (KN)	h (m)	W (KN)	Mr (KN)	Ms (KN)	Audit
1	72,35	1066,64	03.4		221,391		**OK**
2	134,31	994,29	6.8		821,9772		**OK**
3	158,73	859,98	10.2	14165.77	1457,1414	212486.5 5	**OK**
4	227,95	701,25	13.6		2790,108		**OK**
5	473,3	473,3	17		7241,49		**OK**

Verification:

Mr= 12532.10 KN

Ms= 212486.55 KN

Ms /Mr = 16.95 > 1.5........... CV

Table IV.17: Checking the reversal in the direction y-y.

FLOOR	Fy	Vy (KN)	h (m)	W (KN)	Mr (KN)	Ms (KN)	Audit
1	52,96	1012,63	03.06		162,0576		**OK**
2	122,21	959,67	6.12		747,9252		**OK**
3	168,38	837,46	9.18	14165.77	1545,7284	119842.4142	**OK**
4	233,88	669,08	12.24		2862,6912		**OK**
5	435,2	435,2	15.3		6658,56		**OK**

Verification:

Mr=11976.96 KN

MS=119842.4142 KN

Ms /Mr = 10 > 1.5........... CV

- **Effort normal reduced :**

Article (7-4-3-1) of the EPS (version 2003) requires the verification of the effort of normal reduction to avoid the brittle failure of the section of concrete.

$$V = \frac{N}{B.f_{c28}} \leq 0,3$$

The audit is done by the formula as follows:
Where :
N_d: refers to the effort normal of computation operating on a section of concrete (obtained by ROBOT).
$B_{,c}$: is the area (section gross) of the latter.

f_{cj}: is the characteristic resistance of the concrete to calculate the normal stress "N_d" according to **the BAEL** (the ArticleB.8.2.2) for studs subjected to loads due to gravity and earthquakes: "The combinations of action to consider."

Table IV.18: Verification of reduced axial force in columns**.**

Column	**Section(mm)**	N_d (N)	Fc28 (MPa)	V	Observation
DRC-1-4	400*400	624806,14	25	0.16	CV

- **Justification of the effect of P-Δ :**

The effect of the second order (or the effect of P-Δ) can be neglected when the condition following is satisfying on all levels. Section 5.9 of the RPA99/version2003-P40.

$$\theta = \frac{P_K \times \Delta_K}{V_K \times h_K} \leq 0.10$$

With:

P_K: Weight total of the structure and of the expenses of operating associated to the top of level k.

V_K: Effort - edged floor at level k.

Δ_K: Displacement relative level k compared to the level k-1.

h_K: height of the floor k.

Table IV.19: Verification of the effect of P-Δ in the sense that y-y**.**

Floor	P	Δ_Y	V_Y	H	θ_Y	Verification
1	-14165,77	0,228	1012,63	3,4	0,010423242	OK
2	-11455,11	0,426	959,67	3,4	0,016617494	OK
3	-8750,2	0,475	837,46	3,4	0,016219075	OK
4	-6047,73	0,45	669,08	3,4	0,013292462	OK
5	-3348,58	0,397	435,2	3,4	0,009982536	OK

Table IV.20: Verification of the effect of P-Δ in the direction x-x

Floor	P	Δ_X	V_X	H	θ_X	Verification
1	-14165,8	0,116	1066,64	3,4	0,005034529	OK
2	-11455,11	0,206	994,29	3,4	0,007755896	OK
3	-8750,2	0,25	859,98	3,4	0,008312817	OK
4	-6047,73	0,263	701,25	3,4	0,007412315	OK
5	-3348,58	0,258	473,3	3,4	0,005965165	OK

IV.7 Conclusion :

Among the methods used for modeling, we use the method static equivalent that checks all the criteria to use.

The exploitation of the results given by the software **Autodesk Robot Structural Analysis Professional 2021,** was used to verify several criteria, namely:

- ✓ Determine the modes' own in such a way that the 1era and 3rd shifts, the 2nd torsion for more security.
- ✓ Audit of the period, fundamental to the structure according to the requirements of the RPA.
- ✓ Check the effort - edged at the base obtained by the approach static equivalent is specified as the effort sharp minimum at the base (=0,8.VMSE).
- ✓ To verify the effect of torsion additionally.
- ✓ Check the travel inter-floor, which is an index of damage to the floor.
- ✓ To verify the effect of P-Δ on the stability of structure.
- ✓ Verification of the condition of the effort reduced

Chapter V :

Calculate structural elements

V-Calculates structural elements:

V-1- Introduction :

The calculation of the sections will be conducted according to the rules of calculation of reinforced concrete (BAEL 99 and RPA 99/V2003)

The rules BAEL 99 " Rules for the design and calculation of reinforced concrete structures " have to object to specify the principles and methods, most current to preside at, and be used in the design and calculations of the audit of structures and reinforced concrete structures, and apply more especially to the buildings common.

The rules of design have come to address by working with materials in the plastic domain and by adopting combinations of action that take into account both the variation possible in the unfavourable case of the intensities of the shares, on the other hand the probability which the shares sentaient their values.

The rules RPA 99/Ver2003 "Rules Seismic Algerian" are for the purpose of setting the standards for the design and calculation of reinforced concrete structures in seismic zones.

The objectives and targets are to ensure reasonable protection of human lives and structures with respect to the effect of seismic actions by a design and sizing appropriate.

Beams: are subject to the moments of flexing and efforts to sharpen, and therefore, they are calculated in the simple bending.

Combinations of the loads to the beams:
In function of the type of solicitation, it distinguishes the different combinations of the following :

- **According to BAEL 91**(situation sustainable)

ELU 1,35 G + 1,5 Q

ELS G + Q

- **According to RPA 99:** (the status accidental)

G + Q ± E 0.8 G ± E

The Column: are subject to the efforts normal, of the efforts sharp and to the flexural moment, and will therefore be calculated composed of bending.

combinations of expenses for the Columns:

In function of the type of stress, we distinguished the different combinations of the following:

- **According to BAEL 91** (situation sustainable)
- ELU ...... 1,35 G + 1,5 Q

ELS .... G + Q

- **According to RPA 99:** (the status accidental) G+Q±1.2 E

G + Q ± E 0.8 G ± E

- **characteristics Constraints of the concrete and the steel :**

Table V - 1Contraintes characteristics of the concrete and of steel.

Situation	Concrete			Steel		
	γ_β	f_{c28} (MPa)	f_{bu}(MPa)	γ_s	f_e (MPa)	f_{ed}(MPa),
Sustainable	1.5	25	14.20	1.15	400	348
Accidental	1.15	25	18.48	1	400	400

V-2- The study of Beams major is secondary :

We will take moments to max on each floor and one computes the reinforcement of all the beams on each floor together.

.Recommendations of the RPA 99(V2003):

a) longitudinal Reinforcement:

The percentage of the total minimum steels that are longitudinal on the entire length of the beam is 0.5% in any section.

The percentage of total maximum of steel dividers is to:

4% in Area common.

6% in the Area of recovery.

The length of a minimum of recovery is to:

40∅: In Area IIa.

The anchorage of reinforcement longitudinal upper and lower columns of the bank angle must be made with hooks to **90°.**

The beams supporting the low expenses vertical and solicited primarily by the lateral forces on the seismic must have reinforcements symmetrical with a section in span at least equal to the half of the section on support.

b) Transverse Reinforcements:

- The amount of reinforcement cross-sectional minimum is to: **At = 0.003.S.b**
- The spacing maximum between the reinforcement cross-cutting is determined as follows :

- In the area of nodal and in - span, if the frames are compressed are required : a minimum of (h/4, 12ϕ) ;
- In the outside of the box nodal: s ≤ h/2.

The value of the diameter $\emptyset_l$ of reinforcement longitudinal to take is the most a small diameter used

- The first transverse reinforcement should be placed at 5 cm from the most nude to the support or flush.

V-2-1- Reinforcement of beams major (30×40):

V-2-1-1-ELECTED : (1,35 G + 1,5 Q) :

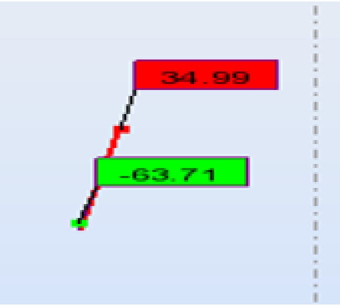


Figure V- 1 : secondary beam in SLU

- **In spans:**

$\mu = \frac{M_t}{b*d^2*f_{bc}}$

$f_{bc} = \frac{0.85*f_{28}}{\theta_b*\gamma_b}$

$\mu = \frac{34.99*10^4}{30*37.5^2*14.16} = 0.058$

$\boldsymbol{\varepsilon_{si}} = \frac{fe}{\sigma s*ys} = \frac{400000}{2*10^8*1.15}$ **= 0.00173 FeE400**

$\alpha_R = \frac{3.5}{3.5+1000\varepsilon_{\delta S}} = \frac{3.5}{3.5+1000*0.0017} = 0.669$

$U_R = 0.8\,\alpha R\,(1 - 0.4\,\alpha R) = 0.8 * 0.673 * (1 - 0.4 * 0.669) = 0.392$

$\boldsymbol{U} < \boldsymbol{U_R}$ → yes domain → $\boldsymbol{A'_S} = \mathbf{0}$ → $\boldsymbol{pivot\ A}$

$\alpha = \frac{(1-\sqrt{1-2U})}{0.8}$

$\alpha = \frac{(1-\sqrt{1-2U})}{0.8} = 0.0747$

$\boldsymbol{U} < 0.186$ → yes domain

$es = 10\%$

$\sigma s = \frac{f_e}{ys} = \frac{400000}{1.5} = 347826.08$

$Z = (1 - 0.4\alpha) = 37.5(1 - 0.4 * 0.0747) = 36.37$ cm

$A_s = \frac{M_t}{z*\sigma} = \frac{34.99*10^4}{36.37*8480} = 3.5$ cm^2

Has$_S$ = 3.5 cm^2

On support :

$\mu = \frac{M_t}{b*d^2*f_{bc}}$

$f_{bc} = \frac{0.85*f_{28}}{\theta_b*\gamma_b}$

$\mu = \frac{63.71*10^4}{30*37.5^2*14.16} = 0.104$

$\boldsymbol{\varepsilon_{si} = \frac{fe}{\sigma s*ys} = \frac{400000}{2*10^8*1.15} = 0.00173}$ **FeE400**

$\alpha_R = \frac{3.5}{3.5+1000\varepsilon_{\delta S}} = \frac{3.5}{3.5+1000*0.0017} = 0.669$

$U_R = 0.8\alpha_R (1 - 0.4\alpha_R) = 0.8 * 0.673 * (1 - 0.4 * 0.669) = 0.392$

$\boldsymbol{U} < \boldsymbol{U_R}$ → yes domain → $\boldsymbol{A's} = \boldsymbol{0}$ → $\boldsymbol{pivot\ A}$

$\alpha = \frac{(1-\sqrt{1-2U})}{0.8}$

$\alpha = \frac{(1-\sqrt{1-2U})}{0.8} = 0.137$

$\boldsymbol{U} < 0.186$ → yes domain

$es = 10\%$

$\sigma s = \frac{f_e}{ys} = \frac{400000}{1.5} = 347826.08$

$Z = (1 - 0.4\alpha) = 37.5(1 - 0.4 * 0.137) = 35.44$ cm

$A_s = \frac{M_t}{z*\sigma} = \frac{63.71*10^4}{35.44*8480} = 5.16$ cm^2

H$_{as}$= 5.16 cm^2

V-2-1-2- SLA:

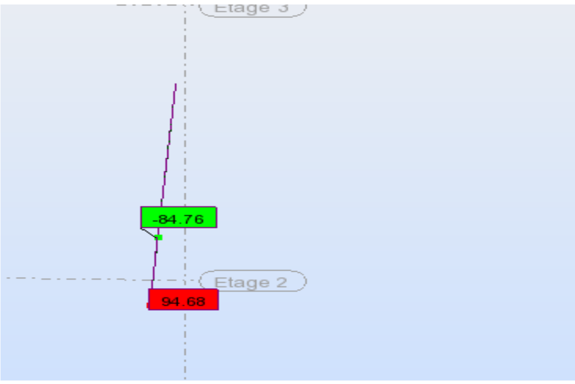


Figure V - 2: secondary beam SLA.

- **In spans :**

$\mu = \frac{M_t}{b * d^2 * f_{bc}}$

$f_{bc} = \frac{0.85 * f_{28}}{\theta_b * \gamma_b}$

$\mu = \frac{94.68 * 10^4}{30 * 37.5^2 * 14.16} = 0.121$

$\varepsilon_{si} = \frac{fe}{\sigma s * \gamma s} = \frac{400000}{2 * 10^8 * 1.15} = 0.00173$ FeE400

$\alpha_R = \frac{3.5}{3.5 + 1000\varepsilon_{\delta S}} = \frac{3.5}{3.5 + 1000 * 0.0017} = 0.636$

$U_R = 0.8\,\alpha R\,(1 - 0.4\alpha R) = 0.8 * 0.673 * (1 - 0.4 * 0.669) = 0.372$

$U < U_R$ → yes domain → $A's = 0$ → $pivot\ A$

$\alpha = \frac{(1 - \sqrt{1 - 2U})}{0.8}$

$\alpha = \frac{(1 - \sqrt{1 - 2U})}{0.8} = 0.161$

$U < 0.186$ → yes domain

$es = 2‰$

$\sigma s = \frac{f_e}{\gamma s} = \frac{400000}{1.5} = 347826.08$

$Z = (1 - 0.4\alpha) = 37.5(1 - 0.4 * 0.0747) = 35.08$cm

$A_s = \frac{M_t}{z*\sigma} = \frac{34.99*10^4}{35.08*8480} = 6.96\ cm^2$

$H_{as} = 6.96\ cm^2$

<u>On support:</u>

$\mu = \frac{M_t}{b*d^2*f_{bc}}$

$f_{bc} = \frac{0.85*f_{28}}{\Theta_b*\gamma_b}$

$\mu = \frac{84.76*10^4}{30*37.5^2*14.16} = 0.109$

$\varepsilon_{si} = \frac{fe}{\sigma s*ys} = \frac{400000}{2*10^8*1.15} = 0.00173$

$\alpha_R = \frac{3.5}{3.5+1000\varepsilon_{\delta S}} = \frac{3.5}{3.5+1000*0.0017} = 0.636$

$U_R = 0.8\alpha_R\,(1 - 0.4\alpha_R) = 0.8 * 0.673 * (1 - 0.4 * 0.669) = 0.379$

$U < U_R$ → yes domain → $A'_s = 0$ → $pivot\ A$

$\alpha = \frac{(1-\sqrt{1-2U})}{0.8}$

$\alpha = \frac{(1-\sqrt{1-2U})}{0.8} = 0.144$

$U < 0.186$ → yes domain

$es = 2‰$

$\sigma s = \frac{f_e}{ys} = \frac{400000}{1.5} = 347826.08$

$Z = (1 - 0.4\alpha) = 37.5(1 - 0.4 * 0.137) = 35.34cm$

$A_s = \frac{M_t}{z*\sigma} = \frac{94.68*10^4}{35.34*8480} = 6\ cm^2$

$A_S = 6\ cm^2$

Condition of non-fragility:

Span:

BAEL (Art A4.2.1):

$A_{min} \geq 0.23$ in. $b.\ d.\frac{f_t}{f_e}$ $=0,23 \times 30 \times 37.5 \times \frac{2.1}{400}$

$A_{min} = 1.34cm^2 \leq 6.96cm^2$ .... (CV)

RPA (Art7.5.2.1) :

$A_{min} \geq 0.005.\ b.\ h = 0,23 \times 30 \times 40$

A_{min}= **6 cm²** $\leq 6.96cm^2$ Check

Support:

BAEL (Art A4.2.1) :

$A_{min} \geq 0.23$ in. $b.\ d.\frac{f_t}{f_e}$ $= 0, 23 \times 30 \times 37.5 \times \frac{2.1}{400}$

$A_{min} = 1.34cm^2 \leq 6.96cm^2$ .... (CV)

RPA (Art7.5.2.1):

$A_{min} \geq 0.005.\ b.\ h = 0,23 \times 30 \times 40$

A_{min}= **6 cm²** $\leq 6cm^2$ Check

Therefore, we adopt**: 3HA14+3HA12 = 8.01 cm² (in span)**

3HA14+3HA12 = 8.01 cm² (in support)

V-2-1-3- Verification The SLS:

The calculation is done according to the rule of B. A. E. L 91, cracking is considered harmful :

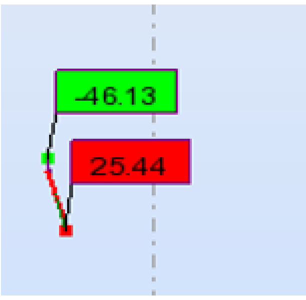


Figure V - 3Diagramme of the time of the beam the SLS

a) In spans :

Ms = 25.44 kn.m As=8.01 cm^2

➢ **neutral axis position (y) :**

bx2/2+15 (As+A's)-15(A s *c'+As.(d)=0

=> 30x2/2 + 15×(8.01)x -15(8.01×(37.5)) =0

15x2 + 120.15 x – 4505.62 =0

Δ = 284773.22 => $\sqrt{\Delta}$= 533.64

x = (-120.15+ 533.64)/30 =13.78 cm

x = 13.78 cm

➢ **Moment of inertia (I) :**

I= (bx^3/3) +15As (d-x) 2 + 15A's (x-c)^2

I = (30× (13.78) ^{3}/3) + 15×8.01 (37.5-13.78)2

I= 29016.57 cm^4

V-2-1-4- constraint verification:

In concrete :

$\sigma_{bc} = \frac{M_S.x}{I} = \frac{25.44*13.78.10^{-2}}{29016.57} = 12081.48 kN/m^2$

$\sigma_{bc} \leq 0.6\ f_{cj}$= 0.6*25 = 15 MPA

σ_{bc} =12081.48$kN/m^2 \leq \sigma'_{bc}$ =150000kN/m^2 CV

In the steel:

$\sigma s = 15 * M_s (\frac{d-x}{I})$

$= 15 * 25.44 (\frac{37.5-13.78}{2901657.10^{-8}}$ =308656.64 KN/m

σ_s = 308656.46 $kN / m^2 \leq \sigma'_s$ =2000000 kN / m^2 CV

b)On support:

M_S = 46.13 $kN.m$

Has_S = 8.01 cm^2

- **neutral axis position :**

x =13.78 cm

- **Moment of inertia (I) :**

I= 29016.57 cm^4

Check constraints :

In the concrete

$\sigma_{bc} = \frac{M_S.x}{I} = \frac{46.13*13.78.10^{-2}}{29016.57*10^{-8}}$ = 21907.18kN / m^2

$\sigma_{bc} \leq 0.6 f_{cj}$= 0.6*25 = 15 MPA

σ_{bc} =21907.18$kN / m^2 \leq \sigma'_{bc}$ =150000kN / m^2 CV

In the steel:

$\sigma s = 15 * M_s (\frac{d-x}{I})$

$= 15 * 25.44 (\frac{0.375-0.137}{2901657.10^{-8}}$)= 559682.5 KN/ m^2

σ_s = 559682.5 $kN / m^2 \leq \sigma'_s$ =2000000 kN / m^2 CV

Shear force verification

T=Vu=84.64 kn $\tau_u < \tau_u$ lim $\tau_u = \frac{YOU}{b*d} = \frac{84.64}{30*37.5)10^{-2}}$=$752.35\ KN/m^2$

τ_u' = {3.33MP , 5MPA} = (3330KN/m^2 ,5000) KN/m^2,

τ'_u = 3330KN/m^2

Then $\tau_u < \tau'_u$ condition checks

V-2-1-2- **transverse reinforcement determination:**

$$\Phi \leq \min\left\{\frac{l}{35}\ ;\Phi\ ;\ \frac{l}{10}\right\} = \min\{1{,}14;\,1{,}4;\,3\}$$

$\Phi_t = 1{,}14$ **Either :** $(A = 4T8 = 2{,}01cm^2$

Spacing according to the BAEL :

$S_{t1} \leq \min(0.9\times d;\ 40\ cm) = 33.75$ cm

$$St1 \leq \frac{At.fe}{0.4*b} = \frac{2{,}01*400}{0.4*30} = 67\ cm$$

St = min (St1 ; St2) =33.75 cm

- **Spacing required by the RPA : II**

In the area of nodal : $S_t \leq \min(h/4\ ,12\phi) = \min(40/4\ ;\ 12\times1{,}4)$ **= 10 cm**

In stock area : $S_t \leq h/2 = 40/2 =$ **20 cm**

Therefore, we shall adopt :

S_t = 10 cm..................................**In area nodal**.

S_t = 20 cm.......................... **In the area of the current one.**

- **According to RPA99V2003 : calculation of the amount of reinforcement cross-sectional mini :**

$A_t = 0.003\times S\times b$

- **Area nodal**

$A_t = 0.003\times10\times30 = 0.9\ cm^2 < 2.01\ cm^2$

- **Area current**

$A_t = 0.003\times20\times30 = 1.8\ cm^2 < 2.01\ cm^2$

- **Percentage total max of steel long :**
  - **4% in zone current :** $A_{s1} = 4/1000 \times 30 \times 40 = 48\ cm^2$
  - **6% in the area of recovery :** $A_{s2} = MAX = 6/100 \times 30 \times 40 = 72\ cm^2$
  - **Length of the box nodal** : L' = 2h = 80 cm

- **Length of recovery :** L=40∅ $= 1.4 \times 40 = 56\ cm$

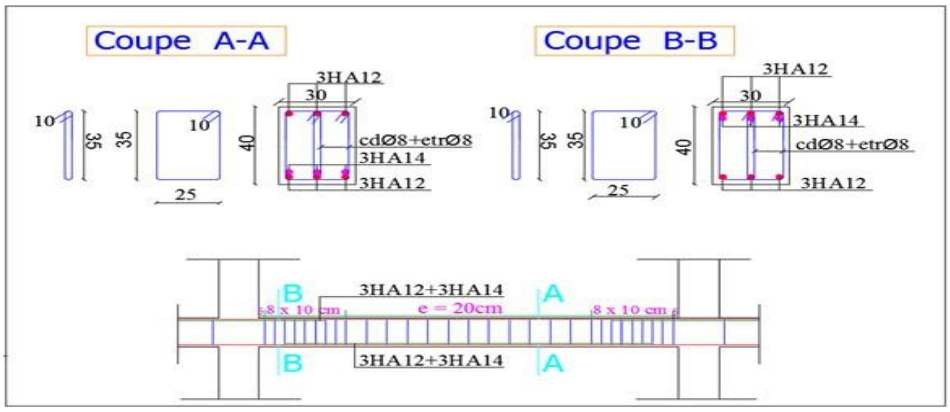


Figure V - 4Ferraillage of the main beam.

V-2-2- calculation of beam reinforcement (30×35):

V-2-2-1-**ELECTED : (1,35 G + 1,5 Q) :**

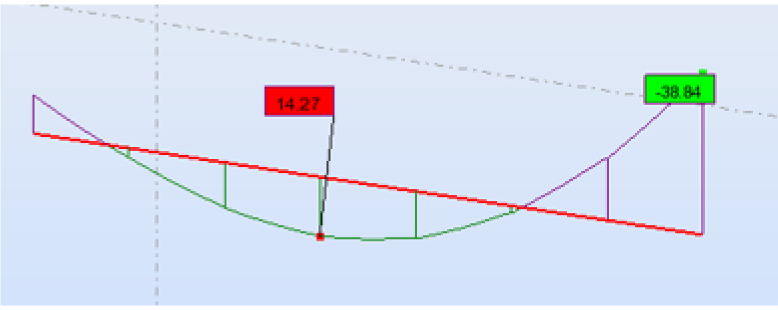


Figure V - 5 : principal beam to the SLU

- **In spans :**

$$\mu = \frac{M_t}{b*d^2*f_{bc}}$$

$$f_{bc} = \frac{0.85*f_{28}}{\theta_b*\gamma_b}$$

$$\mu = \frac{14.27*10^4}{30*37.5^2*14.16} = 0.0312$$

$$\varepsilon_{si} = \frac{fe}{\sigma s*ys} = \frac{400000}{2*10^8*1.15} = 0.00173$$

$$\alpha_R = \frac{3.5}{3.5+1000\varepsilon_{\delta S}} = \frac{3.5}{3.5+1000*0.0017} = 0.669$$

$$U_R = 0.8\,\alpha R\,(1 - 0.4\alpha R) = 0.8 * 0.673 * (1 - 0.4 * 0.669) = 0.392$$

$U < U_R$ → yes domain → $A's = 0$ → *pivot A*

$\alpha = \frac{(1-\sqrt{1-2U})}{0.8}$

$\alpha = \frac{(1-\sqrt{1-2U})}{0.8}$ =**0.0396**

U < 0.186 → yes domain
$es = 2‰$

$\sigma s = \frac{f_e}{ys} = \frac{400000}{1.5} = 347826.08$

$Z = (1 - 0.4\alpha) = 37.5(1 - 0.4 * 0.112 = 31.68$cm

$A_s = \frac{M_t}{z*\sigma} = \frac{34.99*10^4}{35.08*8480} = 1.29$ cm^2

Hass = 1.29 cm^2

On support :

$\mu = \frac{M_{(a}}{(b*d^2*f_{bc}}$

$f_{bc} = \frac{0.85*f_{28}}{\theta_b*\gamma_b}$ = 14166.66 kn/m^2

$\mu = \frac{38.84*10^4}{30*32.5^2*14.16} = 0.067$

$\varepsilon_{si} = \frac{fe}{\sigma s*ys} = \frac{400000}{2*10^8*1.15} = 0.00173$

$\alpha_R = \frac{3.5}{3.5+1000\varepsilon_{\delta S}} = \frac{3.5}{3.5+1000*0.0017} = 0.669$

$U_R = 0.8\alpha_R\ (1 - 0.4\alpha_R) = 0.8 * 0.673 * (1 - 0.4 * 0.669) = 0.392$

$U < U_R$ → yes domain → $A's = 0$ → $pivot\ A$

$\alpha = \frac{(1-\sqrt{1-2U})}{0.8}$

$\alpha = \frac{(1-\sqrt{1-2U})}{0.8}$ =**0.086**

U < 0.186 → yes domain
$es = 10‰$

$$\sigma s = \frac{f_e}{ys} = \frac{400000}{1.5} = 347826.08$$

$$Z = (1 - 0.4\alpha) = 37.5(1 - 0.4 * 0.086) = 31.38\text{cm}$$

$$A_s = \frac{M_t}{z*\sigma} = \frac{38.84*10^4}{31.38*3480} = 3.09\text{cm}^2$$

Hass = 3.09 cm^2

<u>In spans :</u>

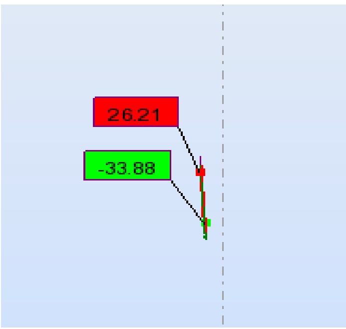


Figure V - 6 : principal beam to the SLA

$$\mu = \frac{M_{(a}}{(b*d^2*f_{bc}}$$

$$f_{bc} = \frac{0.85*f_{28}}{\theta_b*\gamma_b} = 18478.26 \text{ kn/m2}$$

$$\mu = \frac{38.84*10^4}{30*32.5^2*14.16} = 0.0447$$

$$\varepsilon_{si} = \frac{fe}{\sigma s*ys} = \frac{400000}{2*10^8*1.15} = 0.00173$$

$$\alpha_R = \frac{3.5}{3.5+1000\varepsilon_{\delta S}} = \frac{3.5}{3.5+1000*0.0017} = 0.636$$

$$U_R = 0.8\alpha_R (1 - 0.4\alpha_R) = 0.8 * 0.673 * (1 - 0.4 * 0.669) = 0.379$$

$U < U_R$ → **yes domain** → $A's = 0$ → ***pivot A***

$$\alpha = \frac{(1-\sqrt{1-2U})}{0.8}$$

$$\alpha = \frac{(1-\sqrt{1-2U})}{0.8} = 0.0571$$

$U < 0.186$ → **yes domain**

$es = 2‰$

$\sigma s = \frac{f_e}{ys} = \frac{400000}{1} = 400000$

$Z = (1 - 0.4\alpha) = 37.5(1 - 0.4 * 0.0571) = 31.75$ cm

$A_s = \frac{M_t}{z*\sigma} = \frac{26.21*10^4}{31.75*3480} = 6.96$ cm^2

$H_{as} = 6.96$ cm^2

- **On support :**

$\mu = \frac{M_{(a}}{(b*d^2*f_{bc}}$

$f_{bc} = \frac{0.85*f_{28}}{\theta_b*\gamma_b} = 18478.26$ kn/m2

$\mu = \frac{50.5*10^4}{30*37.5^2*184.78} = 0.086$

$\varepsilon_{si} = \frac{fe}{\sigma s*ys} = \frac{400000}{2*10^8*1.15} = 0.00173$

$\alpha_R = \frac{3.5}{3.5+1000\varepsilon_{\delta S}} = \frac{3.5}{3.5+1000*0.0017} = 0.636$

$U_R = 0.8\alpha_R (1 - 0.4\alpha_R) = 0.8 * 0.673 * (1 - 0.4 * 0.669) = 0.379$

$U < U_R$ → yes domain → $A's = 0$ → $pivot\ A$

$\alpha = \frac{(1-\sqrt{1-2U})}{0.8}$

$\alpha = \frac{(1-\sqrt{1-2U})}{0.8} = 0.112$

$U < 0.186$ → yes domain

$es = 2‰$

$\sigma s = \frac{f_e}{ys} = \frac{400000}{1} = 400000$

$Z = (1 - 0.4\alpha) = 37.5(1 - 0.4 * 0.112) = 31.01$cm

$A_s = \frac{M_t}{z*\sigma} = \frac{50.5*10^4}{31.01*4000} = 4.07$ cm^2

$Ha_{SS} = 4.07$ cm^2

Condition of non - fragility:

In span:

BAEL (Art A4.2.1) :

$A_{min} \geq 0.23 \text{ in. } b.\, d.\frac{ft}{fe} = 0.23 \text{ in. } b.\, d.\frac{2.1}{400}$

Amin = 1.17cm2 ≤ 4.07cm2 Check

RPA (Art7.5.2.1) :

$A_{min} \geq 0.005.\, b.\, h = 0{,}23 \times 30 \times 35$

A_{min}= **5.25 cm²** ≤ 4.07 cm² non - Check

Therefore, we adopt: **3HA12+3HA12 = 6.79 cm²** **in span**

3HA12+3HA12 = 6.79 cm² **in support**

V-2-2-2- Verification of the section of steel SLS :

The calculation done according to the rule of B. A. E. L 91, the cracking considered as detrimental:

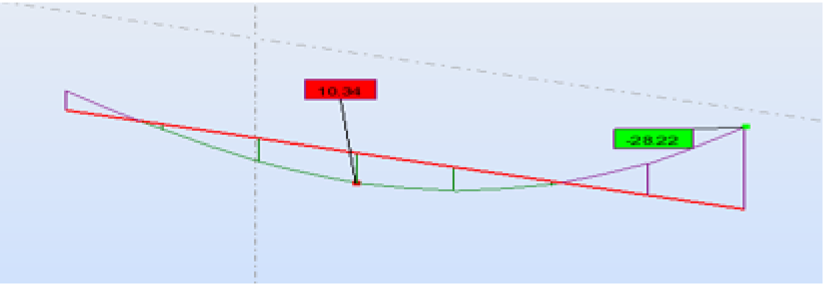


Figure V - 7 Chart of the moment of the beam at the SLS

a) In spans :

Ms = 10.34 kn.m As=6.79 cm²

➢ **The Position of the neutral axis (y) :**

bx2/2+15(As+A's)-15(A s *c'+As.(d)=0

=> 30x2/2 + 15×(6.79)x -15(6.79×(32.5)) =0

15x2 + 101.85 x – 3310.12 =0

Δ = 208980.622 => $\sqrt{\Delta}$ = 457.14

x = (-101.85+ 457.14)/30 =13.78 cm

x = 11.84 cm

- **When inértie (I) :**

I= $(bx^3/3) + 15As\ (d-x)\ 2 + 15A's\ (x-c)^2$

I = $(30\times (11.84)^3/3) + 15\times 6.79\ (32.5-11.84)2$

I= 18702.2 cm^4

Constraints verification:

In concrete :

$$\sigma_{bc} = \frac{M_S.x}{I} = \frac{10.34*11.84.10^{-2}}{18702{,}2*10^{-8}} = 6476.41kN / m^2$$

$$\sigma_{bc} \leq 0.6\ f_{cj} = 0.6*25 = 15\ \text{MPA}$$

σ_{bc} =6476.41$kN / m^2 \leq \sigma'_{bc}$ =150000kN / m^2 CV

In the steel:

$$\sigma s = 15 * M_s\ (\frac{d-x}{I})$$

$$= 15 * 25.44\ (\frac{0.325-0.1184}{18702{,}2.10^{-8}}) = 169513.58\ \text{KN/m}$$

σ_s = 169513.58 KN/m$\leq \sigma'_s$ =2000000 kN / m^2 CV

b) On support :

$M_S = -28.22\ kN.m$

$H_s = 6.79\ cm^2$

- **Position of the axis neutral : x =11.84 cm**
- **Moment of inertia (I) : I= 169531.58 cm^4**

Verification of constraints:

In the concrete :

$$\sigma_{bc} = \frac{M_S.x}{I} = \frac{28.22*11.84.10^{-2}}{18702.2*10^{-8}} = 17865.53kN / m^2$$

$$\sigma_{bc} \leq 0.6\ f_{cj} = 0.6*25 = 15\ \text{MPA}$$

σ_{bc} =17865.53$kN / m^2 \leq \sigma'_{bc}$ =150000kN / m^2 CV

In the steel:

$\sigma s = 15 * M_s \left(\frac{d-x}{I}\right)$

$= 15 * 28.22 \left(\frac{0.325-0.1184}{18702.2.10^{-8}}\right) = 467612.25$ KN/m^2

σ_s = 467612.25 KN/m^2 $\leq \sigma'_s$ =2000000 kN/m^2 CV

Verification of the shear force

T=Vu=77.92 kn $\tau_u < \tau_u$ *lim* $\tau_u = \frac{YOU}{b*d} = \frac{77.92}{30*32.5)10^{-2}}$ =*799.18 KN/m²*

τ_u' = {3.33MP , 5MPA} = (3330KN/m^2 ,5000) KN/m^2,

τ'_u = 3330KN/m^2

Then τ_u < τ'_u condition checks

V-2-1-3- **Determination of transverse reinforcement :**

$\Phi \leq \min \left\{\frac{l}{35} ; \Phi ; \frac{l}{10}\right\}$ = min {1; 1,2; 3}

$\Phi_t = 1$ **Either :** *(A = 4T8 = 2,01cm²*

Spacing according to the BAEL :

$S_{t1} \leq$ min (0.9×d; 40 cm) = 29.25 cm

$St1 \leq \frac{At.fe}{0.4*b} = \frac{2,01*400}{0.4*30}$ = 67 cm

St = min (St1 ; St2) = 29.25 cm

- **Spacing required by the RPA : II**

In the nodal area: $S_t \leq$ min (h/4 ,12ϕ) = min (35/4 ; 12×1,2) = min (8.75 ; 14,4) =8.75 cm
In stock area : $S_t \leq$ h / 2 = 35 / 2 = **17.5 cm**
Therefore, we shall adopt :

S_t = 10 cm..................................**In area nodal**.

S_t = 20 cm.......................... **In the area of the current one.**

- **According to RPA99V2003 : calculation of minimum cross-sectional reinforcement area :**

$A_t = 0.003 \times S \times b$

- **Area nodal**

$A_t = 0.003 \times 10 \times 30$ = 0 .9 cm^2< 2.01 cm^2

- **Area current**

$A_t = 0.003 \times 20 \times 30$ $=1.8\ cm^2 < 2.01\ cm^2$

- **maximum percentage of reinforcement:**

- **4% In the area common:**

$A_{s1}=4/1000 \times 30 \times 35 = 42\ cm^2$

- **6% in the recovery zone:**

$A_{s2}=MAX=6/100 \times 30 \times 35 = 63\ cm^2$

- **Length of the box nodal :** L' = 2h = 70 cm
- **Length of recovery :** $L=40\emptyset\ = 1.2 \times 40 = 48\ cm$

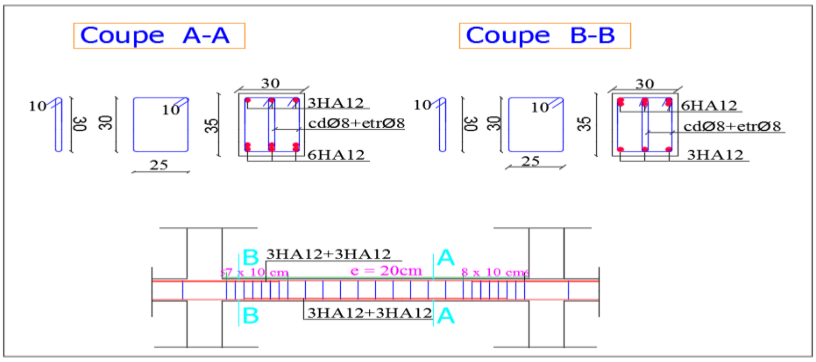
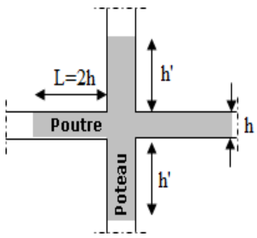


Figure V- 8: Reinforcement of secondary beam.

V-3- **The Columns:**

(a) Recommendations of the RPA 99/version2003:

- The reinforcement longitudinal needs to be on high grip, straight and without hooks.
- The percentage of the minimum reinforcement longitudinal will be of **0.8%**(zone IIa).
- Length minimum of recovery is to **40Φ** (zone IIa)
- The distance between the bars vertically on a face the column does should not exceed **25cm** (zone IIa).
- The junctions for collection must be made, if possible, outside of areas nodal (critical areas).
- $h' = max(\frac{h_e}{6}, b_1, h_1, 60cm)$
- The diameter minimum is 12 mm Calculation, a zone 1 :



Calculates The Columns ' Zone 1' :

Transverse reinforcements

The reinforcement cross-sectional of the columns are calculated with the help of the formula:

$$\frac{At}{t} = \frac{pVu}{h1.fe}$$

- $\mathbf{V_u}$ is the effort cutting edge of computing

- $\mathbf{h_1}$ height total of the section gross

- $\mathbf{f_e}$ constraint limit elastic steel rebar cross

- ρ_a is a weighting factor which takes into account the brittle mode of failure due to shear; it is taken equal to 2.50 if the slenderness geometric λ_g in the direction considered is greater than or equal to 5 and 3.75 in the opposite case.

- **t** is the spacing of the transverse reinforcement the value of which is determined by the formula (7.1) and the maximum value of this spacing is determined as follows:

* **In the nodal area:**

$t \leq \text{Min}(10\varnothing_l, 15\text{cm})$ in zone IIa

* **current zone:**

$t' \leq 15\ \varnothing_l$ II

where $\varnothing_1$ is the diameter of the minimum of the frames and longitudinal of the column

- The amount **of reinforcement cross-sectional minimum $Is_t/t.\ b_1$ in %** is given

as follows: If $\lambda_g \geq 5$: 0,3%

If $\lambda_g \leq 3$: 0.8%

If $3 < \lambda_g < 5$: interpolate between the values limits the previous

λ_g is the slenderness geometric the column

$\lambda_g = \left(\frac{a\ Fi}{a} or\ \frac{Fi}{b}\right)$

with a and b, the dimensions of the right-hand section of the column in the direction of deformation are considered, and the buckling length of the column.

Frames and brackets must be closed by hooks to 135° with a straight length of 10 $\varnothing_t$ minimum ;

Frames and brackets should leave chimneys vertical in number and diameter sufficient (∅ fireplaces > 12cm) to allow a vibration of the correct concrete over the entire height of the columns.

Spacing of the frames:

In the nodal area:

t ≤ Min (10$\varnothing_l$, 15cm) = (10x1,6 cm,15 cm) = 15 cm. It adopts 10 cm.

current zone:

t'≤ 15 $\varnothing_l$ = 15*16= 24 cm. It adopts =15 cm

calculation of λ_g :

Lf = 0,7×4.50 a = 3,15 m =315 cm

$\lambda_g = \frac{If}{(a} = \frac{315}{55}$ = 5.72 ; ρ_a= 2.5 ; Vu=216.87 KN

At $= \frac{pa*Seen*t}{h1.fe} = \frac{2.5*216.87*10}{55*10}$ = 2,64 cm2 it adopts = 6 HA8 = 3,014 cm2

Check the section minimum of the frames:

At /t ∗ b1 (%) = 0.3% ⇒ if λg ≥ 5

⇒ if λg ≤ 3

λg = 2.50 ⇒ At /t∗b1 = 0.55% ≥ 0.3% ... ... condition checked

►Collection :

The width of recovery minimum given by the RPA 99 / version 2003 (page 61) is of:

40 Φ zone IIa= 40*16 = 64 cm.

Length of the box nodal :

h' = Max (h_e/6;b1 ; h1 ; 60) cm= (67,5,55,55,60) =70 cm

the length of the right side of the frame equal to 10 $\varnothing_t$=10*0,8=8 cm it adopts of 15 cm.

Closed by hooks to 135°.

V-3-1- Reinforcement of columns (40×40):

V-3-1-1- SLU:

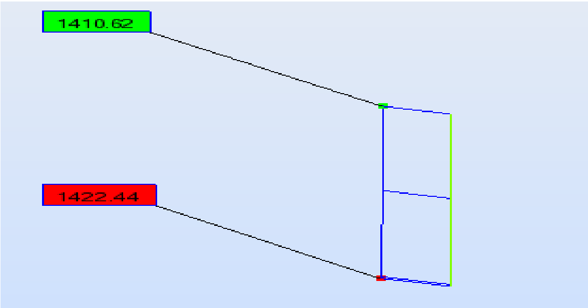


Figure V- 9 Column at the SLU.

Mu=13.01 kn.m	Nu=1442.44 kn	b= 40 cm
Ng=873.84 kn	Nq=161.83 kn	h=40 cm
d=37.5 cm	c=2.5 cm	Lo=3.28 m

e0 = **Mu / Nu=**13.01 /1442.44 = 0.009 m = 0.9 cm

I beam = b*h3/12 = $(30*40^3)/12$ →**I beam = 160000 cm4**

I column = b*h3/12 = $(40*40^3)/12$→ **I column =213333.33 cm4 I column < I beam Lf = 0.7l0→**

Lf = 0.7*3.28=2.29 m

ea = max (2 cm ; l/250) = max (2 cm ; 3.68/250=1.84 cm)

ea = 2 cm

e_1 = e0 + ea = 0.9 + 2 = **2.9 cm.**

a) The simplified method is applicable:

α**= Ng/Ns** = 873.84 / 1035.67=0.87

α **= 0,85**

Φ = 2

$e_2 = \frac{3l\,F^3}{10^4 h}(2+\alpha\Phi) = \frac{3*2.29^3}{10^4*0.4}(2+0.85\times2) = 0.0333\text{ m} = 3.33\text{ cm}$

We have: **e = e1 + e2→ e = 6.23 cm**

Mu = Nu*e→ Mu = 1422.44×0.0623 → Mu = 88.61 kn.m

e0 < **0,5(h-c)** → e0 < 0,5(40-2.5) → e**0** < 18.75 cm

6.23 cm < 18.75 cm................**C. V**
b) compound bending of:
Mu = 88.61 kn.m e =6.23 cm

e < h/2→ e < 40/2→ **6.23 cm < 20 cm.**

The section is fully or partially compressed:

Nu (d-c) - M1 < (0,337 - 0,81 C/h) b h2 fbc

M1 = Mu + Nu (d - h/2) →**M1** = 88.61 + 1422.44(0,375 - 0,4/2)= **337.53**

kn.m Nu (d-c) - M1 = 1422.44(0,375-0,025)- 337.53=**160.324 kn.m**

(0,337 - 0,81 C/h) b h2 fbc = (0,337-0,81×0,025/0,4)×0,4×(0,4)^2×14200

= **260.25 kn.m**
160.324 kn.m < 260.25 kn.m .... **C. V.**

The section is partially compressed.

c) calculation of section rectangular in flexion

simple with : Under M1:

M1 = **337.53 kn.m**

$\mu = \frac{M_t}{b * d^2 * f_{bc}}$

$f_{bc} = \frac{0.85 * f_{28}}{\theta_b * \gamma_b}$

$\mu = \frac{337.53 * 10^4}{30 * 37.5^2 * 14.16} = 0.472$

$\boldsymbol{\varepsilon_{si} = \frac{fe}{\sigma s * ys} = \frac{400000}{2 * 10^8 * 1.15} = 0.00173}$

$\alpha_R = \frac{3.5}{3.5 + 1000\varepsilon_{\delta S}} = \frac{3.5}{3.5 + 1000 * 0.0017} = 0.669$

$U_R = 0.8\,\alpha R\,(1 - 0.4\,\alpha R) = 0.8 * 0.673 * (1 - 0.4 * 0.669) = 0.392$

$\boldsymbol{U} < \boldsymbol{U_R}$ → yes domain → $\boldsymbol{A's} = \boldsymbol{0}$ → $\boldsymbol{pivot\ A}$

$\varepsilon'_s = (3.5\times10^{-3} + \varepsilon_{es}) * (\frac{d-C}{d}) - \varepsilon_{es} = 3.15\times 10^{-3}$

$\varepsilon'_s > \varepsilon_{es}$

.σ'**s**= 3480 kg/cm^2

MR = $0.8\alpha_R\,(1-0.4\alpha_R)\,bd^2 f_{bc}$ = 0.8×0.669×(1-0.4×0.669)40×37.5×142

= 3.13×10^6 kg/cm^2

ZR = d (1-0.4α_R)= 27.47 cm

AS = $\left(\frac{Mf - MR}{d-c} + \frac{MR}{ZR}\right)\frac{1}{\sigma S}$ =34.75 cm^2

A's = $\frac{Mf - MR}{ZR * \sigma'_s}$ = 2.56 cm^2

H_{sTOT}= As + A's = 34.75+2.56 = 37.31 cm^2

Hsfc = AS - $\frac{NU}{fe}$ = 37.31 - $\frac{142244}{400}$ = 1.75 cm^2

Percentage minimum reinforcement according to the RPA 99:

Area current : 0.4 percent (b x h) = 0,4% 40) = 6.4 cm^2

Zone recovery: 0,6% (b x h) = 0,6% (40x40) = 9.6 cm^2

SLA :

G +Q±E

0.8 G±E

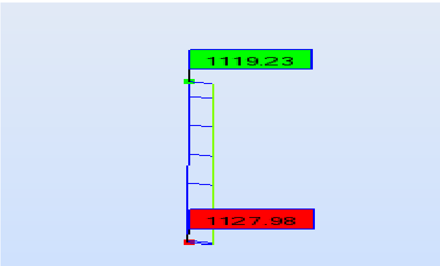


Figure V - 10 : Column to the SLA.

Ma=31.36 kn.m Na=1127.98 kn **e0** = **Ma Na=**31.36 /1127.98 = 0.027 m = 2.7 cm **e1 = e0 + ea** = 2.7 + 2 = **4.7 cm.**

The simplified method is applicable :

α**= Ng/Ns** = 873.84 / 1035.67=0.87

α **= 0,85**

Φ = 2

$e_2 = \frac{3l_F^3}{10^4 h}(2+\alpha\Phi) = \frac{3*2.29^3}{10^4*0.4}(2+0.85\times2)$ = 0.0333 m = 3.33 cm

We have: **e = e1 + e2**→ **e = 8.03 cm**

Mu = Nu*e→ Mu =1127.98×0.0803 → Mu = 90.57 kn.m

e0 < 0,5(h-c) → e0 < 0,5(40-2.5) → e**0** < 18.75 cm

8.03 cm < 18.75 cm   C. V

Composed Bending:

Mu = 90.57 kn.m e =8.03 cm

e < h/2→ e < 40/2→ **8.03 cm < 20 cm.**

The section is fully or partially compressed:

Nu (d-c) - M1 < (0,337 - 0,81 C/h) b h2 fbc

M1 = Mu + Nu (d - h/2) →M1 = 90.57 + 1127.98(0,375 - 0,4/2) = **287.95 kn.m Nu (d-c) - M1** = 1127.98(0,375-0,025)- 287.95=**141.84 kn.m**

(0,337 - 0,81 C/h) b h2 fbc = (0,337-0,81×0,025/0,4)×0,4×(0,4)^2×14200

= **260.25 kn.m**

141.84 kn.m < 260.25 kn.m C. V.

The section is partially compressed.

calculation of rectangular section in bending simple : Under M1:

M1 = **287.95 kn.m**

$\mu = \frac{M_{(a}}{(b * d^2 * f_{bc}}$

$f_{bc} = \frac{0.85 * f_{28}}{\theta_b * \gamma_b}$ **= 18478.26 kn/m2**

$\mu = \frac{287.95 * 10^4}{40 * 37.5^2 * 184.78}$ **= 0.360**

$\varepsilon_{si} = \frac{fe}{\sigma s * ys} = \frac{400000}{2 * 10^8 * 1.15}$ **= 0.00173**

$\alpha_R = \frac{3.5}{3.5 + 1000\varepsilon_{\delta S}} = \frac{3.5}{3.5 + 1000 * 0.0017} = 0.669$

$U_R = 0.8\alpha_R\,(1 - 0.4\alpha_R) = 0.8 * 0.673 * (1 - 0.4 * 0.669) = 0.332$

$U < U_R$ → yes domain → $A's = 0$ → $pivot\ A$

$\alpha = \frac{(1 - \sqrt{1 - 2U})}{0.8}$

$\alpha = \frac{(1 - \sqrt{1 - 2U})}{0.8}$ **=0.588**

$\sigma s = \frac{f_e}{ys} = \frac{400000}{1} = 400000$

$Z = (1 - 0.4\alpha) = 37.5(1 - 0.4 * 0.588) = 28.26$ **cm**

$A_s = \frac{M_t}{z * \sigma} = \frac{287.95 * 10^4}{28.26 * 4000} = 35.47$ **cm²**

Rebar bending composed of:

Has**fc** = AS - $\frac{NU}{fe}$ = 25.47 - $\frac{112798}{400}$ = 2.65 cm^2

Percentage minimum reinforcement according to the RPA 99:

Amin ≥ 0.8 percent (b x h) = 0,8% (40x40) = 12.8 cm²

Adopts : 4HA14 + 4HA16 = 14.2 cm²

<u>SLS :</u>

Verification SLS

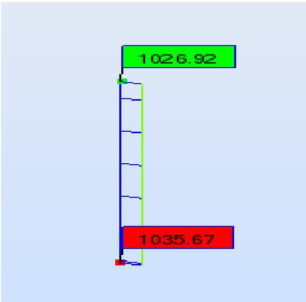


Figure V – 11Column to the SLS.

Ms =9.5 kn.m Ns = 1035.67 kn   Ng = 873.84 kn e0 = Ms / Ns= 9.5/1035.67 = 0.009 m = 0.9 cm

Homogeneous Section:

B =b (h+15(As+A's)

B = (40×40)+15 (14.2+0)

B = 1813 cm2

V1 and V2 will be determined by the equations of the static moment compared to the more tablets

$V1 = \frac{1}{g}\left[\frac{bh^3}{2} - 15\ (A1.c' + A2.(d)\right]$

V2 = 22.35 cm

V**2**= h - V**1**= 40-22.35 = 17.65 cm

$I_1 = \frac{1}{g}\ [\ (V1^3 + V2^3) - 15\ (\ A1(v1-c)^2 + A2(v2-c)^2)]$

I**1**= 354984.21 cm⁴

Boundary Condition 1

σb< σ(b**'** =0,6. Fc28

$\sigma b = \frac{NS}{B1} + N_S \times e_1(\frac{V1,}{I1})$

$\sigma b = \frac{1035.67}{18.13} + 1035.67 \times e_1(\frac{22.35}{354984.21})$ =0.7016 kn/cm2 = 70.16 kg/cm2

σb =70.16 kg/cm2 < σ̄ **'**= 0.6fc28 = 150kg/cm2... cv

Constraint of steel :

σs = $\frac{NS}{B1}$ + N_S × e $_1$($\frac{V1}{I1}$)

σs = $\frac{1035.67}{18.13}$ +1035.67 × e $_1$($\frac{22.35-3}{354984.21}$) = 8.6816 kn/cm2 = 868.16 kg/cm2

σs = min $\{400; 110\sqrt{1.6 \times 1.8}\}$ ={26 ; 186.68} =2000 kg/cm2

σs=868.16 kg/cm2 < σ '= 2000 kg/cm2... cv

Determination of Transverse Reinforcements:

RPA

Φ_t ≤ min $\left\{\frac{l}{35}\ ; \Phi\ ; \frac{l}{10}\right\}$ = min {1.14; 1.6; 4}

Φ_t =1 **Is :** $A = 2T\,8 = 1{,}01cm^2$

BAEL

Ø ≥ 0.3 Ø = 0.3 × 1.6 = 0.48 cm

One chooses Ø = 0.8 cm = 8 mm Is adopted : **2HA8**

Condition of non-fragility :

BAEL (Art A4.2.1) :

A_{min} ≥ 0.23 in. $b.\ d.\frac{ft}{fe}$ = 0.23 in. $b.\ d.\frac{2.1}{400}$

A_{min} = 1.81cm2 ≤ 14.2 cm^2 ....Check

Verification of the shear force

T=Vu=77.92 kn $\tau_u < \tau_u\ lim\ \tau_u = \frac{YOU}{b*d} = \frac{30.57}{40*37.5)10^{-2}}$=2.038 KN/m^2

τ_u' = {3.33MP , 5MPA} = (3330KN/m^2 ,5000) KN/m^2,

τ'_u = 3330KN/m^2

Then $\boldsymbol{\tau_u} < \boldsymbol{\tau'_u}$ ....condition checks

Spacing required by the RPA :

Zone's hub : St ≤ min (15cm ,10) = min (15cm; 16) = 15 cm

In stock area : St ≤ 15Ø = 15×1.6 = 24 cm

So we shall adopt :

St = 10 cm In the nodal area.

St = 20 cm , In the regular area

Cover length:

L=40Ø = 1.6 × 40 = 65 cm

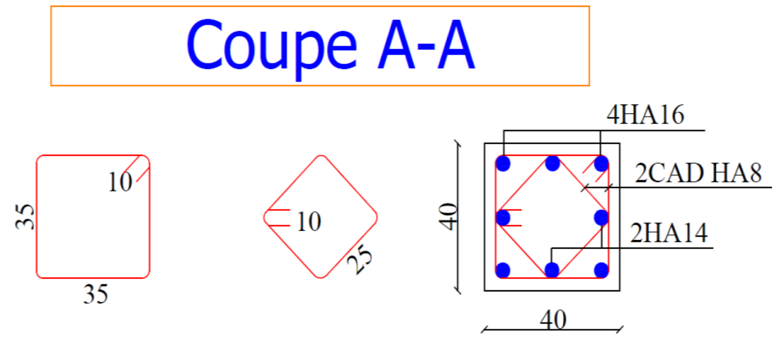


Figure V - 12Coupe A-A (Clumn).

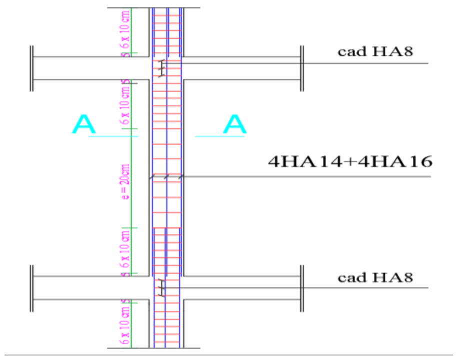


Figure V - 13 column reinforcement.

V-4- **Reinforcement of Sails:**

V-4-1- Introduction :

The sails will be solicited by :

When flexing and effort cutting edge caused by the action of seismic forces.

Normal stress of the combination of loads, permanent, operating as well as the load seismic forces.

In our case the efforts caused by the earthquake have given the band fully compressed and the bands partly stretched under the effect of horizontal loads and vertical by a result, the reinforcement is will be by report to the conditions minimum imposed by the RPA.

V-4-1-1- Steel vertical : (section 7.7.4.1.RPA99/2003).

When a part of the sail is stretched under the action of forces, vertical and horizontal, the tractive effort must be made in full by the reinforcement, the minimum percentage of vertical frames on the entire area tense is 0.20%.

It is possible to focus the reinforcement of traction at the end of the wing or the pier, the section total reinforcement vertical area stretched in front to stay at least equal to 0.20% of the horizontal section of the concrete in tension.

The bars vertical of the areas' extremes should be ligated with horizontal frames in which the spacing should not be greater than the thickness of the sail.

If the efforts are important to compression acting on the tip, the bars are vertical must comply with the conditions imposed on the columns

The vertical bars at the last level must be fitted with hooks at the top. All the other bars don't have hooks (junction by recovery).

Has each end of the sailing spacing of bars should be reduced by half, about 1/10 of the length of the sail? This spacing of the end should be at most equal to 15 cm.

V-4-1-2- Steel horizontal : (section 7.7.4.2.RPA99/2003).

The bars horizontal must be of hooks to 135^0 with a length of 10Ø. In the event that there are heels stiffness, the bars horizontal will need to be rooted sound brackets if the mentioned heels allow the realization of a right anchor.

Rules in common :

The percentage of minimum reinforcement horizontally and vertically of sails is given as follows:

Overall, in the section of the sail 0.15%. In stock area is 0.10%.

The spacing of the horizontal bars and verticals must be less than or smaller than

the two following values:

$S \leq 1.5\ a$

$S \leq 30cm$

The two tablecloths' reinforcement must be connected with 4 pins at a meter square, in each table, the horizontal bars should be arranged toward the outside.

The diameter of the bars vertical and horizontal of the sails (with the exception of areas of about) should not exceed 1/10 of the thickness of the sail.

The lengths of the collection must be equal to:

40Ø for the bars located in the zones, or the reversal of the sign of the efforts is possible.

20Ø to the bars located in the areas compressed under the action of all the possible combinations of loads.

Along the joints of recovery of casting, effort tranchant must be taken by the steel of seam in which the section must be calculated with the formula: $A_{vj}= 1.10.V/f_e$.

This amount must be added to the section of steel strained necessary to balance the effort of traction due to moments of confusion.

V-4-1-3- Method of calculation :

The calculation of the reinforcement will be made by the method of constraints and verified according to R. P. 99 under the efforts N_{max} and M_{cor},

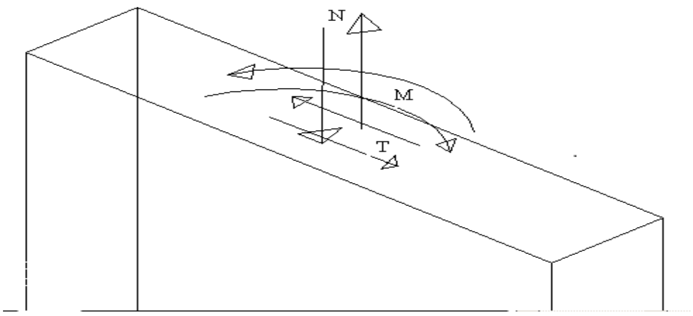


Figure V - 14 Sail subjected to combined bending

-Study of the section subjected to the bending comprising:
It determines the constraints by the formula of NAVIER –BERNOULLI :

$$\sigma_{g,d} = N/_S \pm M \times Y/_I$$

With :

N : effort normal acting on the recasts considered.

M : moment of bending acting on the recasts considered.

I : moment of inertia of the recasts considered.

Y : center of circle of the section of the wing in the direction of the plan means.

Note :

Ifσ_g and σ_d are negative signs, there will be a section fully stretched (SET). Ifσg andσd are signs of positive there will be a section fully compressed (SEC).

If σ_g andσ_d are the signs to the contrary there will be a section partially compressed (SSC).

Combinations of calculation :

According to the RPA 99 the combinations of actions to consider for the determination of stresses and deformations of the calculation are:

G+Q $\pm$ E

0.8 G$\pm E$

V-4-2- Calculating reinforcement of the sails solid :

The reinforcement of the sails are made according to :the BAEL99 :
(document unified technique) .

V-4-2-1- Sailing ⇒ direction (X) :

L= 270 cm, e = 25 cm, h = 468cm ,28 = 25 MPA in The horizontal section of wing :

S = e.$\boldsymbol{L}$ = 25x270 = 6750 cm2

I = $\frac{e.L^3}{12}$ = $\frac{4.1*.10^3}{12}$= 4.1.10^7 $\boldsymbol{cm^4}$

V and V' : lever arm

V=V'= L/2 = 270 /2 = 135cm

C'= C = $\frac{I}{S.V}$ =$\frac{4.1*10^7}{6750*135}$ = 44.08 cm

Length of buckling l:

l_f = 0,85.L = 0,85.468 =397.8 cm

Elancement:

$\boldsymbol{\lambda} = \frac{lf.\sqrt{12}}{e} = \frac{397.8.\sqrt{12}}{of\ 25} = 55.12 > 50$

$\boldsymbol{\alpha} = 0{,}6.\frac{50}{\lambda}^2 = 0.49$

The section reduced Br :

Br = d.(a-2)= 270(25-2) =6210 cm2

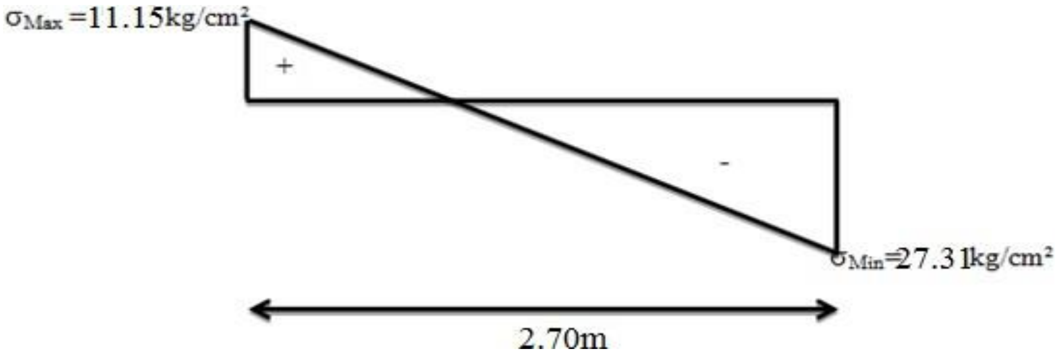
- **A SLA :**

$$\begin{Bmatrix} \boldsymbol{NA} = -545{,}55 \text{ KN} \\ \\ \boldsymbol{MY} = -584{,}2KN.m \end{Bmatrix}$$

T = 424.21 KN

$\boldsymbol{\sigma}_{\max} = \frac{\boldsymbol{NA}}{\boldsymbol{S}} + \frac{\boldsymbol{M.has\ .V}}{\boldsymbol{I}} = \frac{-545.55}{6750} + \frac{584.2*135*10^2}{\mathbf{4.1.10^7}}$ **= 0.1115 KN / cm2 = 11.15 kg/cm2**

$\boldsymbol{\sigma}_{\max} = \frac{\boldsymbol{NA}}{\boldsymbol{S}} + \frac{\boldsymbol{M.has\ .V}}{\boldsymbol{I}} = \frac{-545.55}{6625} + \frac{584.2*132.5*10^2}{\mathbf{3.87.10^7}}$ **= 0.2731 KN / cm2 = -27.31 kg/cm2**



Lt = $\frac{L.\sigma_{min}}{\sigma_{min}+\sigma_{max}} = \frac{270\times27.31}{27.31+11.15}$ = 191.72 cm

Lc = 265 –lt =270 – 191.72 = 78.28 cm

Na $_{\text{lim}}$ = $\propto (\frac{b.cf28}{0.9yb}) = \frac{0{,}49.(6095.250)}{0.9*1.15}$ = 721388.88 kg = 7213.88 KN

σ $_{\text{bna}}$ = $\frac{MA\ min}{e.l}$ = = $\frac{8166.66}{25.300}$ = 108.88 Kg /cm2

σ_{bna} = 108.88 kg /cm2 >σ_{Max} =11.15 KN/cm2

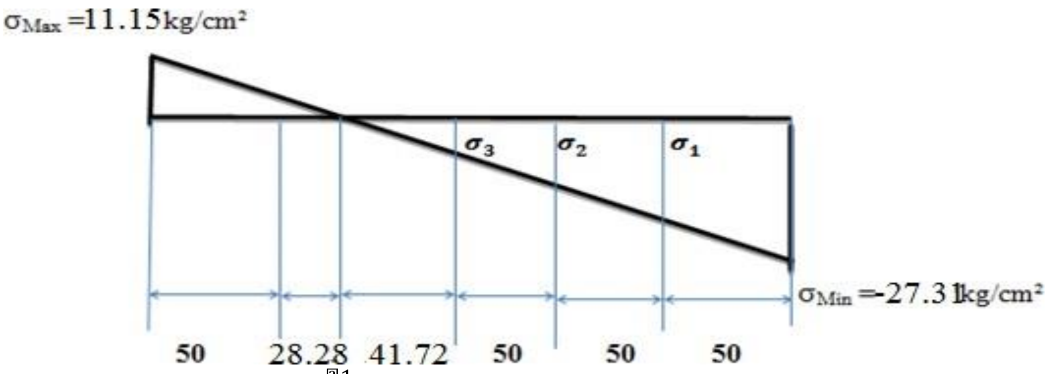
. Area compressed concrete enough

Tenses Area :

The calculation is done in this case for the stripes vertical of width **d :**

d ≤ min ($\frac{heb}{2}$; $\frac{2L,}{3}$) = min (230; 52.14)

Therefore, we adopt a strip 50 cm



Band (1):

tan α = $\frac{-27.31}{191.72}$ = $\frac{}{141.72}$ = $\sigma1$ = $\frac{-27.31\times141.72}{191.72}$ = - 20,18 kg/cm2

σ max = $\frac{-27.31\times20.18}{2}$ = - 23.75 kg/cm2

As = $\frac{\text{f.γs}}{\text{fe}}$ =$\frac{23.75.50.25}{4000}$ = 7,42 cm2

We adopt 8HA 12 /2face = 9,04 cm 2 ⇒ 4HA 12 /front Spacing = 10 cm

$Amin$ = 0,002.50.25= 2.5 cm2

As= 9,04 cm2 >$Amin$ CV

Band(2) :

tan α = $\frac{-20.18}{141.72}$ = $\frac{\sigma1}{95.72}$ = $\sigma1$ = - 13.62 kg/cm2

σ max = $\frac{-13.62\times20.18}{2}$ = - 16.9 kg/cm2

As = $\frac{\text{f.ys}}{\text{fe}}$ =$\frac{16.9.50.25}{4000}$ = 5.28 cm2

It adopts 6HA 12 /2face = 6.78 cm2 ⇒ 3 HA 12 /side

Escapement = 15 cm

$Amin$ = 0,002.50.25= 2.5 cm2

As= 6.78 cm2 >$Amin$ CV

Band (3):

tan $\alpha = \frac{-13.62}{95.72} = \frac{\sigma 1}{95.72} = \sigma 1$ = - 6.5 kg/cm2

$\sigma_{max} = \frac{-13.62 \times 6.5}{2}$ = - 10.06 kg/cm2

As = $\frac{f.\gamma s}{fe} = \frac{10.06.50.25}{4000}$ = 3.14 cm2

We adopt 4HA 12 /2fac =4,52 cm2 ⇒

2HA 12 /side Escapement = 15 cm

A_{min} = 0,002.50.25= 2,5 cm2

A_s= 4.52 cm2 >A_{min} CV

For the other bands we adopt a reinforcement of how

symmetrical Spacing of the bars by the RPA :

S ≤ min(1.5.a ; 30 cm) ;

S ≤ min (35 ; 30cm) , we adopt the 20 cm spacing end = S/2, So we choose :

St = 10 cm in area about St = 20 cm in area current

The length of the area of about = l /10 = 270/10 = 27cm one adopts

30 cm, **Check the shear stress** :

$\tau b \le \tau b = \frac{0,07.fc28}{yb}$ = 15,22 kg/ cm2

$\tau b = \frac{\nabla}{b0*d} = \frac{424.21}{25.0.9.270}$ = 0,0771 KN/cm2 = 7.71 Kg / cm2............ CV

State limit of stability of the form :

$$\left\{ \begin{matrix} \boldsymbol{MU} = 76.83 \text{ KN.m} \\ \boldsymbol{NU} = 1368.04 \text{ KN} \\ \boldsymbol{MS} = 56.32 \text{ KN.m} \end{matrix} \right\}$$

e $_0 = \frac{MU,}{NU} = \frac{70.05}{1368.04}$ = 0,0561 m = 4.61 cm

$\boldsymbol{ea}$= max {2cm ; $\frac{l}{250}$ } =2cm

$\boldsymbol{\alpha}$ = 10.[1-(M_U /1,5. M_S)] =10.[1-(76.83 /1,5.56,32)] = 0.9 a

.Φ= 2

$\boldsymbol{e_1}$ =(3.lf 2 /10000.(h)(2 +$\boldsymbol{\alpha}$.**Φ**) = (3.(374.4)2 /10000.25).(2 + 0,9.2) = 6,39 cm
$\boldsymbol{e_y}$= $\boldsymbol{e_a}$+$\boldsymbol{e_1}$= 8,39 cm

$$\sigma = \frac{NU}{S} + \frac{NU.e_{x.V}}{I} \pm \frac{NUDE.e_{y\,V}}{I}$$

$$\begin{bmatrix} \sigma1 = 27.73 \text{ kg /cm2} \\ \sigma2 = 19.32 \text{ kg /cm2} \rightarrow< \sigma bna = 108.88 \text{ kg /cm2} \\ \sigma3 = 21.99 \text{ kg /cm2} \\ \sigma4 = 13.94 \text{ kg /cm2} \end{bmatrix}$$

SLS Verification:

$\boldsymbol{M_s}$ = 56.32 KN.m

$\boldsymbol{N_s}$ = -1001.44 KN

S= e. l + 15.Ace = 25.270+ 15(10.125) = 6901.87cm2

$\sigma = \frac{NS}{S} + \frac{Ma.v\prime}{I} = \frac{1001.44}{6901.87} + \frac{56.32.135.10^2}{4.1.10^7}$ = 12.65 kg /cm2 <σ = 150 kg /cm2

Reinforcement horizontal :

According to the RPA , the percentage of minimum reinforcement vertical and horizontal of the piers and given as a suite :

-globally, in the section of the sail 0,15%

As= 0.0015.100.25 = 3,75 cm2 it adopts 8HA8 / face of a spacing equal to

25 cm according To the DTR(B. C. 2.42)

Wv = A_min = 0,0015.25.270= 10.125 cm2

It has :A_**h**= 2/3(Wv) = 0,666.10.125 = 6,75 cm2 it adopts 16 T8 = 8.04 cm2/2 face 8T8/

ml

Spacing 12 cm

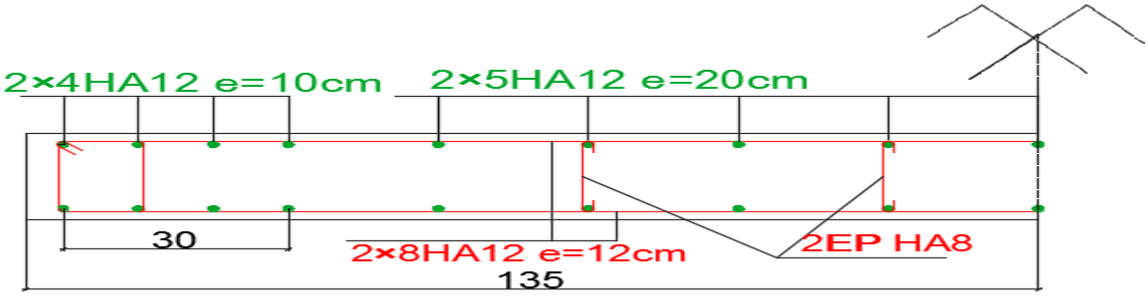


Figure V – 15 Reinforcement of the sail in the X-direction.

V-4-2-2- Sail ⇒ direction (y) :

L= 300 cm, e = 25 cm, h = 468 cm,28 = 25 MPA in The horizontal section of wing :

S = e. $\boldsymbol{L}$ = 25x300 = 7500 cm2

I = $\frac{e.L^3}{12} = \frac{25*.300^3}{12}$ = 5.62.10^7 $\boldsymbol{cm}^4$

V and V' : lever arm

V=V'= L/2 = 300 /2 = 150 cm

C'= C = $\frac{I}{S.V} = \frac{5.62*10^7}{7500*150}$ = 49.95 cm

Length of buckling l:

l_f = 0,85.L = 0,85.468 =397.8 cm
Elancement :

$\boldsymbol{\lambda} = \frac{lf.\sqrt{12}}{e} = \frac{397.8.\sqrt{12}}{of\ 25}$ = 55.12 > 50

$\boldsymbol{\alpha}$= 0,6.$\frac{50^2}{\lambda}$ = 0.49

The section reduced Br :

Br = d.(a-2)= 300(25-2) =6900 cm2

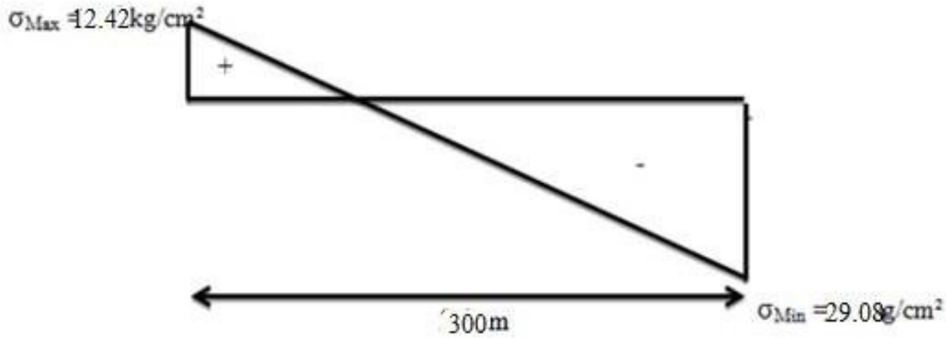
❖ **A SLA :**

$$\begin{Bmatrix} \boldsymbol{NA} = 624.65 \text{ KN} \\ \\ \boldsymbol{MY} = 777.65 \text{ KN}.m \end{Bmatrix}$$

T = 490.97 KN

$\boldsymbol{\sigma}_{\max} = \frac{NA}{S} + \frac{M.has\,.V}{I} = \frac{-624.655}{7500} + \frac{777.65*150*10^{2\,2}}{5.62.10^{7}}$ = 0.1242 KN / cm2 = 12.42 kg/cm2

$\boldsymbol{\sigma}_{\max} = \frac{NA}{S} + \frac{M.a\,.V}{I} = \frac{-624.655}{7500} + \frac{777.65*150*10^{2\,2}}{5.62.10^{7}}$ = -0.2908. KN/cm2 = -29.08 kg/ cm2



Lt = $\frac{L.\sigma_{min}}{\sigma_{min}+\sigma_{max}} = \frac{270\times27.31}{27.31+11.15}$ = 191.72 cm

Lc = 300 –lt =300 – 210.21 = 78.28 cm

Na $_{\text{lim}}$ = $\propto (\frac{b.cf28}{0.9yb}) = \frac{0,49.(6900.250)}{0.9*1.15}$ = 816666.66 kg = 8166.66 KN

$\sigma_{\text{bna}} = \frac{MA\,min}{e.l} = = \frac{8166.66}{25.300}$ = 108.88 Kg /cm2

σ_{bna} = 108.88 kg /cm2 >σMax =12.42 KN/cm2

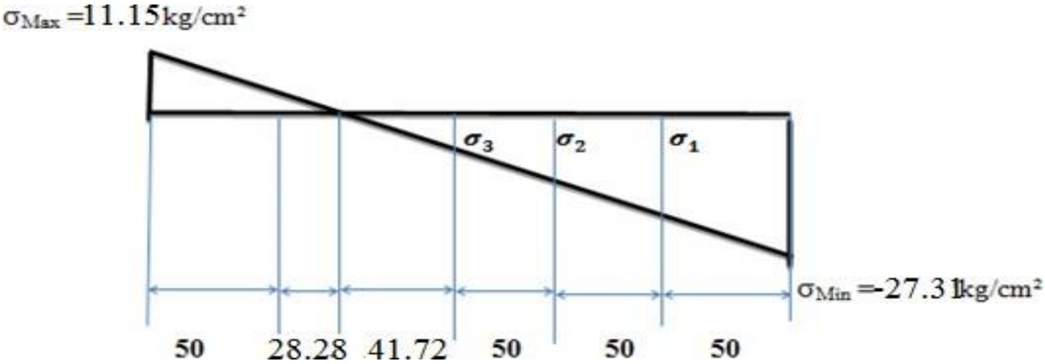
. Area compressed concrete is enough.

tense zone:

In this case, the calculation is made for a vertical strip of width **d:**

d ≤ min ($\frac{heb}{2}$; $\frac{2L,}{3}$) = min (230; 54.86)

So it takes a band of 55 cm



Band (1):

tan $\alpha = \frac{-29.08}{210.21} = \frac{\sigma1}{155.21} = \sigma1 =$ - 21.47 kg/cm2

$\sigma_{max} = \frac{-29.308 \times 21.47}{2}$ = - 25.27 kg/cm2
As = $\frac{f.\gamma s}{fe} = \frac{25.27.55.25}{4000}$ = 8.68 cm2

We adopt 8HA 12 /2face = 9,04 cm 2 ⇒ 4HA 12 /front Spacing = 10 cm

$Amin$ = 0,002.55.25= 2,75 cm2

As= 9,04 cm2 >$Amin$ CV

Band(2) :
tan $\alpha = \frac{25.27.55.25}{155.21} = \frac{\sigma 1}{100.21} = \sigma 1$ = - 13.86 kg/cm2

$\sigma_{max} = \frac{-13.86 \times 21.47}{2}$ = - 17.66 kg/cm2
As = $\frac{f.\gamma s}{fe} = \frac{17.66.50.25}{4000}$ = 6.07 cm2

It adopts 6HA 12 /2face = 6.78 cm2 ⇒ 3 HA 12 /side

Escapement = 15 cm

$Amin$ = 0,002.55.25= 2.75 cm2

As= 6.78 cm2 >$Amin$ CV

Band (3):
tan $\alpha = \frac{-13.86}{95.72} = \frac{\sigma 1}{45.21} = \sigma 1$ = - 6.25 kg/cm2

$\sigma_{max} = \frac{-13.86 \times 6.25}{2}$ = - 10.05 kg/cm2
As = $\frac{f.\gamma s}{fe} = \frac{10.05.55.25}{4000}$ = 3.45cm2
We adopt 4HA 12 /2fac =4,52 cm2 ⇒

2HA 12 /side Escapement = 15 cm

A_{min} = 0,002.55.25= 2,75 cm2

A_s= 4.52 cm2 >A_{min}.............................. CV

For the other bands, we adopt a reinforcement of how symmetrical the spacing of the bars is by the RPA:
S ≤ min(1.5.a ; 30 cm) ;

S≤min(35;30cm) , we adopt the 20 cm spacing end = S/2, So we choose :

St = 10 cm in area about St = 20 cm in area current

The length of the area of about = l /10 = 270/10 = 27cm one adopts

30 cm, **Check the shear stress** :

$\tau b \leq \tau b = \frac{0{,}07.fc28}{yb} = 15{,}22$ kg/ cm2

$\tau b = \frac{\nabla}{b0*d} = \frac{490.97}{25.0.9.300} = 0{,}0727$ KN/cm2 = 7.27 Kg / cm2............ CV

Limit state of structural form stability:

$$\begin{cases} \boldsymbol{MU} = 108.05 \text{ KN.m} \\ \boldsymbol{NU} = 1672.4 \text{ KN} \\ \boldsymbol{MS} = 78.71 \text{ KN.m} \end{cases}$$

e 0 = $\frac{MU,}{NU} = \frac{108.05.102}{1672.46}$ = 0,0646 = 6.46 cm

$\boldsymbol{ea}$= max {2cm ; $\frac{l}{250}$ } =2cm

$\boldsymbol{\alpha}$ = 10.[1-(M_U /1,5.M_S)] =10.[1-(108.05 /1,5.78.71)] = 0,85

.Φ= 2

$\boldsymbol{e_1}$ =(3.lf 2 /10000.(h)(2 +$\boldsymbol{\alpha.\Phi}$) = (3.(374.4)2 /10000.25).(2 + 0,85.2) = 6,23 cm

$\boldsymbol{e_y}$= $\boldsymbol{e_a}$+$\boldsymbol{e_1}$= 8,23cm

$\boldsymbol{e_y}$= $\boldsymbol{e_a}$+$\boldsymbol{e_1}$= 8,39 cm

$\sigma = \frac{NU}{S} + \frac{NU.e_{x.V}}{I} \pm \frac{NUDE.e_{y\,V}}{I}$

$$\begin{bmatrix} \sigma 1 = 64.66 \text{ kg/cm2} \\ \sigma 2 = 27.40 \text{ kg/cm2} \rightarrow < \sigma bna = 108.88 \text{ kg/cm2} \\ \sigma 3 = 17.19 \text{ kg/cm2} \\ \sigma 4 = 20.06 \text{ kg/cm2} \end{bmatrix}$$

SLS Verification:

$\boldsymbol{M_s}$ = 78.71 KN.m ,$\boldsymbol{N_s}$ = -1222.7 KN

As= $As_{,\,min}$= 0.0015×25×300 = 11.25 cm^2

S= e. l + 15.Ace = 25.300+ 15(11.25) = 7668.75 cm2

$\sigma = \frac{NS}{S} + \frac{Ma.v\prime}{I} = \frac{1222.7}{7668.75} + \frac{78.7150.10^2}{5.62.10^7}$ = 17.04 kg /cm2 <σ = 150 kg /cm2

horizontal Reinforcement:

According to the RPA , the percentage of minimum reinforcement vertical and horizontal of the piers and given as a suite :

-globally, in the section of the sail 0,15%

As= 0.0015.100.25 = 3,75 cm2 it adopts 8HA8 / face of a spacing equal to

25 cm according To the DTR(B. C. 2.42)

Wv = A_min = 0,0015.25.275= 10.5 cm2

It has :h= 2/3(Wv) = 0,666.11.25 = 7.5 cm2 it adopts 16 T8 = 8.04

cm2/2 face 8T8/ ml

Spacing 12 cm

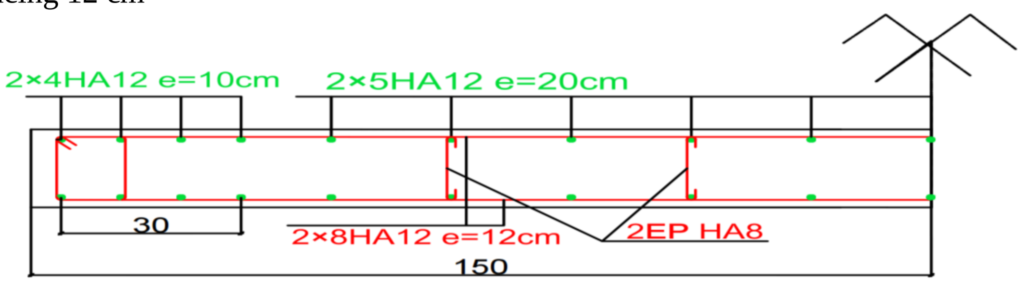


Figure V-16 Sail reinforcement in the Y direction sole footings

V.5.Conclusion :

This chapter presented the structural design results of the building's various components, based on safety standards and engineering requirements defined by the applicable codes. Modeling and structural analysis software are effectively utilized to obtain accurate and reliable results that ensure the stability and safety of the structure under various loads, particularly seismic loads. These results represent the practical outcome of the previously studied theoretical phases and constitute a key step toward the preparation of the project's execution documents.

Chapter VI :

Study of Foundations

VI.1 Introduction

The infrastructure is the part below the level 0.00 whose purpose is to convey to the ground the efforts made by the elements of the structure (columns, walls, walls...). This transmission can be direct (in the case of footings resting on the ground or in the event of riffle) or be provided through other elements (for example, the case of footings on piles).

The determination of the foundation work is done according to the conditions of resistance and settlement related to the characteristics of physical and mechanical soil.

The questions addressed in this chapter apply to the determination of the dimensions and the reinforcement elements, foundations as reinforced concrete elements.

In the most general case, an element of the structure may be passed to its foundation (assumed horizontal):

- A normal stress.
- A horizontal force, resulting, for example, form the action of an earthquake.
- A couple that can be of constant size and is exercised in different planes.

VI.2 Factor in choosing the type of foundation

The choice of the type of foundation depends mainly on the following factors:

- The bearing capacity of soil.
- The loads were transmitted to the ground.
- The distance between the axes of the columns.
- The nature of the soil.
- The depth of the soil is resistant.

VI.3 Choice of type of foundation

According to the characteristics of the soil (an average stress allowable =1.5 bars), on which is located the book ,and the proximity of the good soil in relation to the surface, we conducted first to consider the isolated footing as a solution primary. After RPA 2003 (A. 10.1.4.1), the foundations are dimensioned by the combinations of the following actions:

G + Q + E

0.8 G + E

- **Calculation of constrained**:

$$\sigma_{adm}(ELU) = \frac{q_u^{reel}}{\gamma_q}$$

$$\sigma_{adm}(ELS) = \frac{q_u^{reel}}{\gamma_q}$$

With :

γ_q : Coefficient of safety in the different states limits

γ_q =2.... (ELU)

γ_q = 3 .... (ELS)

Therefore :

$\sigma_{ELS} = \frac{q_u^{reel}}{3} \Rightarrow q_u^{real} = 3 \times \sigma_{ELS}$

$\sigma_{ELU} = \frac{q_u^{reel}}{2} \Rightarrow q_u^{real} = 2 \times \sigma_{ELS}$

$\sigma_{ELU} = \frac{3}{2} \times \sigma_{ELS}$

$\sigma_{ELU} = \frac{3}{2} \times 1.5 = 2.25\ bar$

Calculation of isolated footing

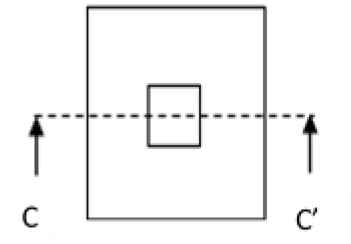
We propose in the first place isolated footings, for this, we will carry out the first inspection, such as:

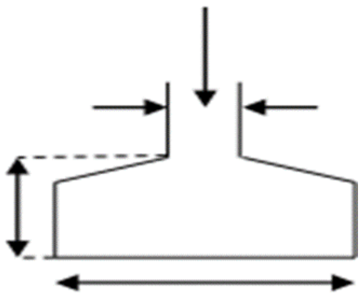
$\frac{N_S}{S} \leq \sigma_{floor}$

σ_{floor}: $permissible\ Stress\ of\ the\ soil.(\sigma_s = 2bar)$

The columns of our structure are square at the base of the section (A*B) where the soles are square (A*B)

Either:





Plan view cut cc' **Figure VI.1 :** Schema of the foundation isolated.

$\frac{N_S}{(A \times B} \leq \sigma_{floor} \Rightarrow (A \times B \geq \frac{N_S}{\sigma_{floor}}$..........................1

$\frac{A}{a} = \frac{B}{b} \Rightarrow A = \frac{a}{b} \times B$..................................2

a and b : dimensions of the column to the base, we replace 2 in 1

$\frac{a}{b} \times B^2 \geq \frac{N_s}{\sigma_{floor}} \Rightarrow B \geq \sqrt{\frac{N_S \times b}{\sigma_{soil} \times a}}$

ELU : $B \geq \sqrt{\frac{624.81\ x\ 0.4}{200\ x\ 0.4}} = 1.77\ m$

ELS : $B \geq \sqrt{\frac{452.52 \times 0.3}{200 \times 0.3}} = 1.5\ m$

According to the results:

the obtained limit states (SLS and SLE), the normal stresses and moments concentrated for

B = 1.77 m remains below the bearing capacity of soil (σ_1, σ_2 < σ_sol). It follows that the sole isolate perfectly meets the requirements of the study, without the risk of overlap or overload. Therefore, the isolated footings are considered to be consistent and committed to the work.

VI.4.Isolated footings:

$\mathbf{G+Q+E}\begin{cases}M=203.39\ KM\\ N=-2.77\ KM\end{cases}$ with N_1 ; M_1 l'effort and the normal time to the'ELU

$\mathbf{1.35\ G+1.5\ Q}\begin{cases}M=283.08\ KM\\ N=-3.8\ KM\end{cases}$ with N_2 ; M_2 l'effort and the normal time to the'ELU

c=c'=5cm

a) Preliminary sizing

$$e_0=\frac{M_{ser}}{No_{ser}}$$

$$e_0=\frac{2.77}{203.39}=0.014$$

We have : $e_0<\frac{b}{6}$

$$e_0<\frac{0.36}{6}=0.06\ m$$

$$\frac{A}{B}=\frac{a}{b}=\frac{40}{40}\rightarrow A=B$$

$$AB\geq\left(1+3\frac{e_0}{b}\right)\frac{N_{SER}}{\sigma_{ground}}\rightarrow B^2\geq\left(1+3\frac{e_0}{b}\right)\frac{N_{SER}}{\sigma_{floor}}$$

$$B\geq\sqrt{\left(1+3\frac{e_0}{b}\right)\frac{N_{SER}}{\sigma_{floor}}}$$

$$B\geq\sqrt{\left(1+3\frac{0.014}{0.4}\right)\frac{203.39}{200}}=1.1\ m$$

We choose : A=B=1.5 m

Calculating the height ht :

$$\boldsymbol{d\geq Max}\left(\frac{(B-b}{4};\frac{A-A}{4}\right)=\left(\frac{1.5-0.4}{4};\frac{1.5-0.4}{4}\right)$$

d ≥ Max (0. 27, 0.27) m

d ≥ 0. 27 m ⇒ d = 30 cm

ht = d+c = 30+5 ⇒ ht = 35 cm

-Headroom :

ht/4 ≤ hl ≤ ht/2 ⇒ 8.75 cm ≤ hl ≤17.5 cm it takes
hl = 15 cm

b) **constraints Check:**

- Weight of the foundation base:

$$P_s = 25B\left(A \times h_t + \frac{h_t - h_l}{2}(A + A)\right)$$

$$P_s = 25 \times 1.5\left(1.5 \times 0.15 + \frac{0.35 - 0.15}{2}(1.5 + 0.4)\right)$$

$$P_s = 15.56\ KN$$

- Weight of fill : γ = 18 KN

$$P_r = \delta_r\left(\frac{((A + a)}{2}[B(h_t - h_l) + 2B(D - h_t) + 2a((D - h_t)]\right)$$

$$P_r = 18\left(\frac{(1.5 + 0.4)}{2}[1.5(0.35 - 0.15) + 2 \times 1.5(1.5 - 0.35) + 2 \times 0.3(1.5 - 0.35)]\right)$$

$$P_r = 46.23\ KN$$

Weight above the column:

$$P_p = \left[has \times b\left(D - h_t\right)\right] \times 25$$

$$P_p = \left[0.40 \times 0.40\left(1.5 - 0.35\right)\right] \times 25$$

P_p = 4.6 KN

- Weight total :

$\boldsymbol{P_T = P_s + P_r + P_p}$ **=15.56+46.23+4.6**
P_T = 66.39 KN

N'ser = Nser + P_T

No ser = 203.39+ 66.39 =269.78 KN

$$e_0 = \frac{M_{ser}}{No_{ser}}$$

$$e_0 = \frac{2.77}{203.39} = 0.014\ m$$

We have :

$$e_0 < \frac{B}{6}$$

$e_0 < \frac{1.5}{6} = 0.25\ m$

Therefore, the diagram of the constraints is trapezoidal

$$\sigma_2 = \left(1 - \frac{6 \times e'_0}{B}\right)\frac{N'_{ser}}{AB}$$

$$\sigma_2 = \left(1 - \frac{6 \times 0.014}{1.5}\right)\frac{269.78}{1.5^2}$$

$$\sigma_2 = 113.19\ {}^{KN}/_{m^2}$$

$$\sigma_1 = \left(1 + \frac{6 \times e'_0}{B}\right)\frac{N'_{ser}}{AB}$$

$$\sigma_1 = \left(1 + \frac{6 \times 0.014}{1.5}\right)\frac{269.78}{1.5^2}$$

$$\sigma_1 = 126.62\ {}^{KN}/_{m^2}$$

$$\sigma_{((A/4)} = \frac{\sigma_2 + 3 \times \sigma_1}{4} = \left(1 + 3\frac{e'_0}{B}\right)\frac{N'_{ser}}{AB}$$

$$\sigma_{((A/4)} = \left(1 + 3\frac{0.014}{1.5}\right)\frac{269.78}{1.5^2}$$

$\sigma_{((A/4)} = 123.26 < \sigma_{ground}$=200KN /m^2

c) **Verification of stability to the overthrow:**

$e'_0 = \frac{M_{ser}}{No_{ser}} = 0.01\ m\ ; \frac{B}{4} = 0.375\ m$

$e'_0 < \frac{B}{4} \rightarrow verified$

d) Reinforcement The SLU :

$\boldsymbol{N'_u = N_u + 1.35P_T}$

$\boldsymbol{N'_u = 283.08 + 1.35 \times 66.39 = 372.71\ KN}$

$\boldsymbol{e_0 = \frac{M_u}{N'_u} = \frac{3.8}{372.71}}$

$\boldsymbol{e_0 = 0.010}$

$\boldsymbol{\frac{B}{1.5} = \frac{1.5}{24} = 0.0625m \Rightarrow e_0 < \frac{B}{24}}$

$$\boldsymbol{A_X = A_Y = \frac{N'_u\left(1 + 3\frac{e_0}{B}\right)}{8 \times d \times \sigma_s}(A - A)}$$

$$A_X = A_Y = \frac{372.71\left(1+3\frac{0.014}{1.5}\right)}{8\times 0.3\times 347.82}(1.5-0.4)\times 10 = 5.01m^2$$

It takes :

A_x=A_y= 6 HA14 = 9.24 cm2

The height of bank is :

With spacing is equal to 20 cm $e \geq \{6\phi + 6\,cm\} = \{6\times 1.6 + 6\,cm\}$ $e \geq \{15.60\ cm\}$

It takes : e = 20 cm.

Length of sealing :

Is = 40. φ = 40x1.6 = 64 cm > A/4 = 23.1 cm → verified

- so all the bars must be extended until the end and with the anchors curved.

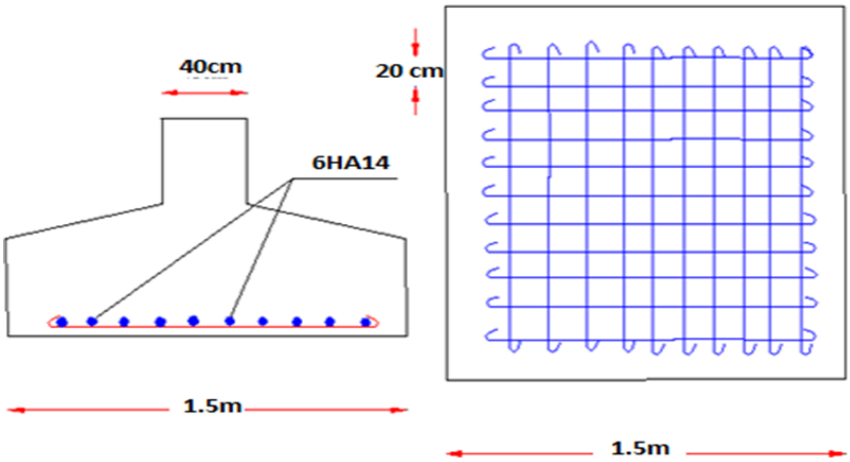


Figure VI.2: Schematic of reinforcement of the foundation isolated in column

VI.5.Foundation under sails

foundation under sails:

These soles are also in the columns (see figure), so it was determined – from ROBOT software -the loads applied to the center of gravity of each set (the sail - column), it has led us to size as isolated footings.

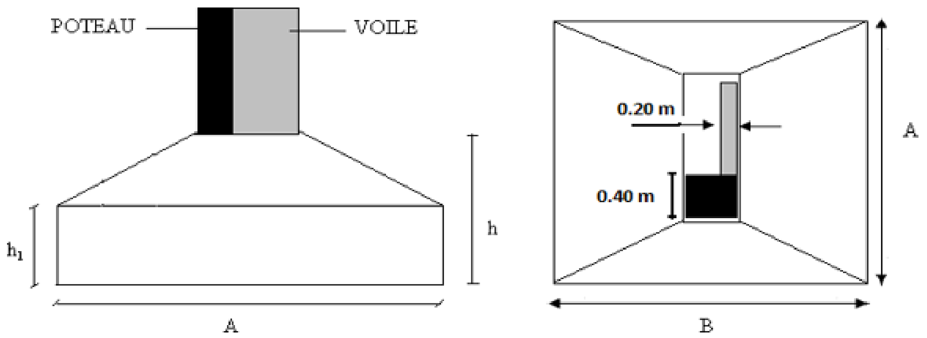


Figure VI.3: Diagram of the footing under the sail.

a) foundation under sail: ($\mathbf{L = 2.00\ m}$):

RPA2003 (section 10.1.4.1)
Cracking very detrimental (BAEL-91)
Pre-sizing the effort and the normal time to SlE.
Verification by the effort and the normal time to the SlU.

$G + Q + E \begin{cases} N_1 = 1385{,}56 \text{ KN} \\ M_1 = 2{,}83 \text{ KN.m} \end{cases}$ with N_1, M_1 l' effort and the normal time to the' SLS

$1.35\ G + 1.5\ Q \begin{cases} N_1 = 1924{,}00 \text{ KN} \\ M_1 = 3{,}95 \text{ KN.m} \end{cases}$ with N_1, M_1 the effort and the normal time to the SLU

$c = c' = 5.0 \text{ cm}, \sigma_s = 347.82 \text{ MPa}$

The preliminary sizing:

$$e_0 = \frac{M_{ser}}{No_{ser}} = 0.002 \text{ m}$$

, We have : $e_0 < \frac{b}{6}$

$$AB \geq \left(1 + 3\frac{e_0}{b}\right)\frac{N_{ser}}{\sigma_{ground}} \rightarrow B^2 \geq \left(1 + 3\frac{e_0}{b}\right)\frac{N_{ser}}{\sigma_{floor}}$$

$$B \geq \sqrt{\left(1 + 3\frac{e_0}{b}\right)\frac{N_{ser}}{\sigma_{floor}}} \geq 3.08 \text{ m}$$

We choose : $\mathbf{B = 3.5\ m}$

Height useful :

$$\frac{B - b}{4} \leq d$$

$\frac{3.5 - 0.20}{4} \leq \mathrm{d} \rightarrow 0.82\ \mathrm{m} \leq \mathrm{d}$

(d = 85 cm

It takes : $\mathrm{h} = \mathrm{d} + \mathrm{c}$

$\mathrm{h} = 85 + 5 = 90\ \mathrm{cm}$

$\mathrm{h}/4 \leq \mathrm{h}_1 \leq \mathrm{h}/2 \Rightarrow \ \mathrm{h}_1 = 35\ \mathrm{cm}$

Check constraints :

Self-weight of the sole : $\mathrm{P_S} = 25\ (\mathrm{B}\left(\ \times \mathrm{h}_1 + \frac{\mathrm{h}-\mathrm{h}_1}{2} (\ \mathrm{A} + \mathrm{A})\right)$

$\mathrm{P_S} = 196.22\ \mathrm{KN}$

Weight of fill : $\mathrm{P_r} = \delta_\mathrm{r} \left(\frac{(\mathrm{A}-\mathrm{A})}{2}[\ \mathrm{B}(\ \mathrm{h_t} - \mathrm{h}_1) + 2\ \mathrm{B}(\mathrm{D} - \mathrm{h_t}) + 2\mathrm{a}((\mathrm{D} - \mathrm{h_t})]\right.$

$\mathrm{P_r} = 18\left(\frac{(3.5 - 0.20)}{2}[3.5(0.90 - 0.35) + 2 \times 3.5(1.50 - 0.90) + 2 \times 0.20(1.50 - 0.90)]\right.$

$\mathrm{P_r} = 189.04\ \mathrm{KN}$

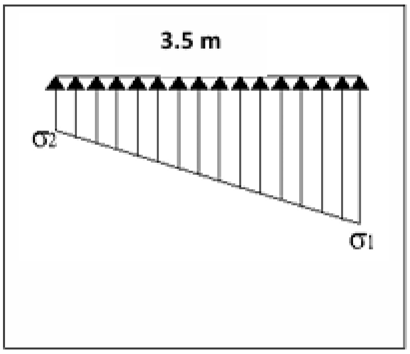
Weight of wall : $P_m = 25 \times 0.20 \times 1 \Rightarrow P_m = 4\mathrm{KN}$

Weight of the front of column : $\mathrm{P_a} = (0.40)^2 \times (1.5 - 0.90) \times 25 = 2.4\ \mathrm{KN}$

total Weight : $\mathrm{P_T} = \mathrm{P_a} + \mathrm{P_m} + \mathrm{P_S} + \mathrm{P_r} = 391.66\ \mathrm{KN}.$

$\mathrm{N'}_{\mathrm{ser}} = \mathrm{N_{ser}} + \mathrm{P_T} = 1385.56 + 391.66$

$\mathrm{N'}_{\mathrm{ser}} = 1777.22\ \mathrm{KN}$



$\mathrm{e'}_0 = \frac{\mathrm{M_{ser}}}{\mathrm{No'_{ser}}}$

$\mathrm{e'_0} = \frac{2.83}{1777.22} = 0.0016\ \mathrm{m}$

We have : $e_0 < \frac{B}{6}$

$\mathrm{e_0} < \frac{3.5}{6} = 0.58\ \mathrm{m}$

, Therefore, the diagram of the constraints is trapezoidal

$\sigma_2 = \left(1 - 6\frac{\mathrm{e'_0}}{\mathrm{B}}\right)\frac{\mathrm{N'_{ser}}}{\mathrm{AB}}$

$\sigma_2 = \left(1 - 6\frac{0.0016}{3.5}\right)\frac{1777.22}{12.25} = 144.79\ \mathrm{KN/m^2}$

$\sigma_1 = \left(1 + 6\frac{\mathrm{e'_0}}{\mathrm{B}}\right)\frac{\mathrm{N'_{ser}}}{\mathrm{AB}}$

$\sigma_1 = \left(1 + 6\frac{0.0016}{3.5}\right)\frac{1777.22}{12.25} = 145.59\ \mathrm{KN/m^2}$

$$\sigma_{(A/4)} = \frac{\sigma_2 + 3\sigma_1}{4} = \left(1 + 3\frac{e'_0}{B}\right)\frac{N'_{ser}}{AB}$$

$$\sigma_{(A/4)} = \left(1 + 3\frac{0.0016}{3.5}\right)\frac{1777.22}{12.25}$$

$$\sigma_{(A/4)} = 145.39\ \mathrm{KN/m^2} < \sigma_{ground} = 150\mathrm{KN/m^2}$$

Verification of stability to the overthrow :

$$e_0 = \frac{M_{ser}}{No'_{ser}} = 0.0016\ \mathrm{m}; \frac{B}{4} = 0.875\ \mathrm{m}$$

$e_0 < \frac{B}{4} \rightarrow$ Verified

Reinforcement to The ELU :

$$N_u{}' = N_u + 1.35\ xP_T$$

$$N_u{}' = 1924 + 1.35 \times 391.66$$

$$N_u{}' = 2454.63\ \mathrm{KN}$$

$$e'{}_o = M_u/N_u{}' = 3.95/2454.63 = 0.0016\ \mathrm{m}$$

$$A_x = A_y = \frac{Nu'x(1 + 3e'{}_o/B) \times (A - a)}{8 \times h \times \sigma_s}$$

$$A_x = A_y = \frac{2454.63 \times 10 \times \left(1 + 3\frac{0.0016}{3.5}\right)(3.5 - 0.20)}{8 \times 0.90 \times 347.82} = 32.39\ \mathrm{cm^2}$$

It takes:

$$A_x = A_y = 18\ \mathrm{T16} = 36.19\ \mathrm{cm^2}$$

With spacing is equal to **20 cm**

-height free : $e \geq 6\varphi + 6 = 6 \times 1.6 + 6 = 15.6\ \mathrm{cm} \Rightarrow e = 20\ \mathrm{cm}$

-length of sealing : $\mathrm{ls} = 40.\varphi = 64\ \mathrm{cm}$

It takes $\mathrm{ls} = 80\ \mathrm{cm} > A/4 = 9.05\ \mathrm{cm} \rightarrow$ Verified

, So all the bars should be extended until the end and with anchors curved.

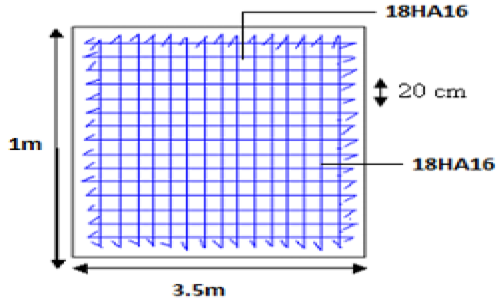


Figure VI.4: Diagram of reinforcement for the footing under the wall.

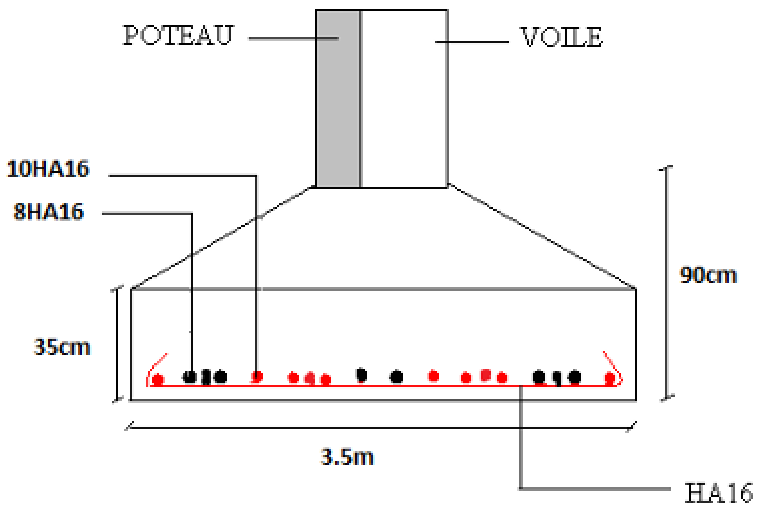


Figure VI.5: Diagram of reinforcement for the footing under the wall.

(b) foundation under sail:($L = 3.1$ m)

RPA2003 (section 10.1.4.1)

Cracking very detrimental (BAEL-91)

Pre-sizing the effort and the normal time to SlS

Verification by the effort and the normal time

$G + Q + E \begin{cases} N_1 = 1878{,}26 \text{ KN} \\ M_1 = 2{,}44 \text{ KN.m} \end{cases}$ with N_1, M_1 l' effort and the normal time to the' SLS

$1.35\text{ G} + 1.5\text{ Q} \begin{cases} \text{N}_1 = 2595{,}88 \text{ KN} \\ \text{M}_1 = 3{,}43 \text{ KN.m} \end{cases}$ with N_1, M_1 l' effort and the normal time to the SLU

$\text{c} = \text{c}' = 5.0 \text{ cm}, \sigma_s = 347.82 \text{ MPa}$

preliminary sizing :

$$e_0 = \frac{M_{ser}}{No_{ser}} = 0.0013 \text{ m}$$

We have : $e_0 < \frac{b}{6}$

$$AB \geq \left(1 + 3\frac{e_0}{b}\right)\frac{N_{ser}}{\sigma_{ground}} \rightarrow B^2 \geq \left(1 + 3\frac{e_0}{b}\right)\frac{N_{ser}}{\sigma_{floor}}$$

$$B \geq \sqrt{\left(1 + 3\frac{e_0}{b}\right)\frac{N_{ser}}{\sigma_{floor}}} \geq 3.57 \text{ m}$$

We choose : $\mathbf{B = 4.2\ m}$

Height useful :

$$\frac{B - b}{4} \leq d$$

$$\frac{4.2 - 0.20}{4} \leq \mathrm{d} \rightarrow 1 \text{ m} \leq \mathrm{d}$$

$(\mathrm{d} = 1.5 \text{ cm}$

It takes : $\mathrm{h = d + c}$

$\mathrm{h} = 100 + 5 = 110 \text{ cm}$

$\mathrm{h}/4 \leq \mathrm{h_1} \leq \mathrm{h}/2 \Rightarrow \mathrm{h_1} = 40 \text{ cm}$

Check constraints :

self-Weight of the sole : $\mathrm{P_S} = 25\ (\mathrm{B}\left(\ \times \mathrm{h_1} + \frac{\mathrm{h-h_1}}{2}(\mathrm{A + A})\right)$

$\mathrm{P_S} = 338.1 \text{ KN}$

Weight of fill : $\mathrm{P_r} = \delta_r\left(\frac{(\mathrm{A-A})}{2}[\mathrm{B(h_t - h_1) + 2\,B(D - h_t) + 2a((D - h_t)}]\right.$

$$\mathrm{P_r} = 18\left(\frac{(4.2 - 0.20)}{2}[4.2(1.10 - 0.40) + 2 \times 4.2(1.50 - 1.10) + 2 \times 0.20(1.50 - 1.10)]\right.$$

$\mathrm{P_r} = 116.28 \text{ KN}$

Weight of wall : $\mathrm{P_m} = 25 \times 0.20 \times 1 \Rightarrow \mathrm{P_m} = 4\text{KN}$

Weight of the front of column : $\mathrm{P_a} = (0.40)^2 \times (1.5 - 1.10) \times 25 = 1.6 \text{ KN}$

total Weight $\mathrm{P_T = P_a + P_m + P_S + P_r} = 459.98 \text{ KN}.$

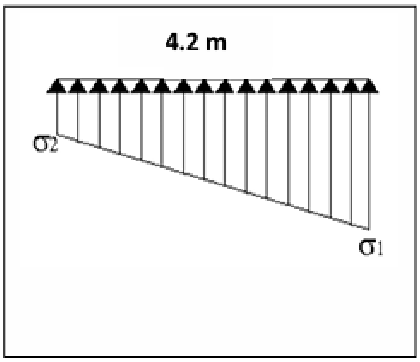
$\mathrm{N'_{ser} = N_{ser} + P_T} = 1878.26 + 459.98$

$\mathrm{N'_{ser}} = 2338.24 \text{ KN}$

$$\mathrm{e'_0} = \frac{\mathrm{M_{ser}}}{\mathrm{No'_{ser}}}$$

$$\mathrm{e'_0} = \frac{2.44}{2398.24} = 0.001 \text{ m}$$

, We have : $e_0 < \frac{B}{6}$



$e_0 < \frac{4}{6} = 0.66\ \text{m}$

, Therefore, the diagram of the constraints is trapezoidal

$$\sigma_2 = \left(1 - 6\frac{e_0'}{B}\right)\frac{N'_{ser}}{AB}$$

$$\sigma_2 = \left(1 - 6\frac{0.001}{4.2}\right)\frac{2398.24}{17.64} = 135.76\ \text{KN/m}^2$$

$$\sigma_1 = \left(1 + 6\frac{e_0'}{B}\right)\frac{N'_{ser}}{AB}$$

$$\sigma_1 = \left(1 + 6\frac{0.001}{4.2}\right)\frac{2398.24}{17.64} = 136.15\ \text{KN/m}^2$$

$$\sigma_{(A/4)} = \frac{\sigma_2 + 3\sigma_1}{4} = \left(1 + 3\frac{e_0'}{B}\right)\frac{N'_{ser}}{AB}$$

$$\sigma_{(A/4)} = \left(1 + 3\frac{0.001}{4.2}\right)\frac{2398.24}{17.64}$$

$$\sigma_{(A/4)} = 136.05\ \text{KN/m}^2 < \sigma_{ground} = 150\text{KN/m}^2$$

Verification of stability to the overthrow:

$$e_0 = \frac{M_{ser}}{No'_{ser}} = 0.0013\ \text{m}\ ; \frac{B}{4} = 1.05\ \text{m}$$

$e_0 < \frac{B}{4} \rightarrow$ Verified

Reinforcement The SLU :

$N_u' = N_u + 1.35\ xP_T$

$N_u' = 2595.88 + 1.35 \times 459.98$

$N_u' = 3216.85\ \text{KN}$

$e_o' = M_u/N_u' = 3.43/3216.85 = 0.0011\ \text{m}$

$$A_x = A_y = \frac{Nu'x(1 + 3e'o/B) \times (A - a)}{8 \times h \times \sigma_s}$$

$$A_x = A_y = \frac{3216.85 \times 10 \times \left(1 + 3\frac{0.0011}{4.2}\right)(4.2 - 0.20)}{8 \times 1.10 \times 347.82} = 42.07\ \text{cm}^2$$

It takes:

$\mathbf{A_x} = \mathbf{A_y} = 16\ \text{T20} = \mathbf{50.27}\text{cm}^2$

With spacing is equal to $\mathbf{20\ cm}$

-height free : $e \geq 6\varphi + 6 = 6 \times 1.6 + 6 = 15.6\ cm \Rightarrow e = 20\ cm$

-length of sealing : $ls = 40.\varphi = 64\ cm$

It takes $1s = 80\ cm > A/4 = 12.57\ cm \rightarrow$ Verified

, So all the bars should be extended until the end and with anchors, curved

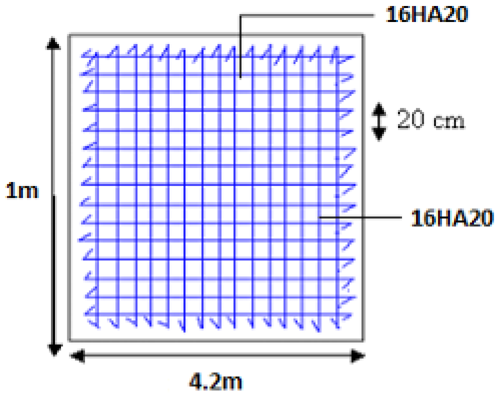


Figure VI.6: Reinforcement diagram of the footing under the sail

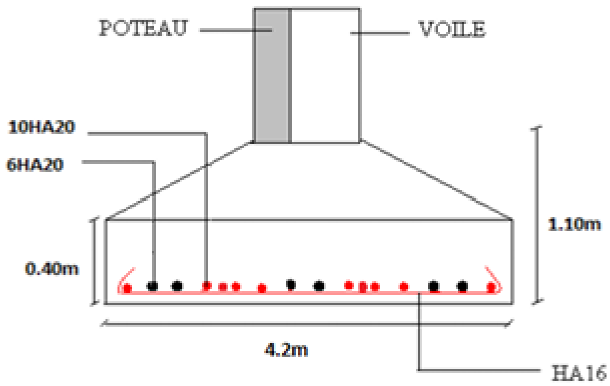


Figure VI.7: Diagram of reinforcement for the footing under the wall

VI. 6 Ground beam

VI.6.1 Definition

The beams are beams that connect the columns at the level of the infrastructure, their calculation is done as a part subjected to a moment from the base of the column, and a tractive effort $< F >$

VI.6.2. Sizing ground beam:

According to the RPA 99 (art. 10-1-1), the minimum dimensions of the cross section of the origins are:

- (25×30) cm^2....................category S2, S3
- (30x30) cm2 website of category S4

For our case (site closes S2) take a section of (30×25) cm^2

Reinforcement ground beam:

Longitudinal reinforcement:

The beams must be calculated to resist the pull under the action of a force is equal to
$F = Nu(max)/\alpha > 20KN$ [RPA 99 V 2003]
α : Coefficient function of the seismic zone, and the category of the site considered.
According to [RPA99/2003] :
$\alpha = 12$ (S3 site, IIa),
$\mathbf{Nu_{(max)}}$: normal stress ultimate column the most sought-after.
$Nu_u(max): 624806{,}14\ N = 624.80614\ KN.$
$F = 624.80614/12 = 52.07\ KN > 20KN$ Verified
$As = F/\sigma s = 52.07 * 10^3/348 = 149.62\ mm^2 = 1.50\ cm^2.$

State limit service :

If the cracking is considered to be harmful :

σa = min (2/3 fe ; 150η) η = 1.6

σa = min (2/3∗400 ; 150*1.6) = 240MPa ; Nser = 452.53 KN.

σres = (Nser/α)/Ai ≤ σa

=> As = (Nser/α)/σa = (452.53*10^3 /12) / 240 =157.13 mm^2 = 2 cm^2

Check the condition of non-fragility :

As ≥ 0.23 in. b. d. ft28 / fe

$f_{228} = 2.1$ MPa; fe $= 400$MPa.

d $= 0.9$ h $= 36$ cm. As $\geq 0{,}23 \times 30 \times 36 \times 2{,}1/400$

Ace $= 1.30$ cm^2

Recommendation of [RPA 99 v2003]

The reinforcement minimum must be 0.6 per cent of the section of concrete including reinforcement and longitudinal reinforcement transversals, such as the spacing of the frames should not exceed the least of:

St $\leq \min(20$ cm$, 15\Phi$ L$)$

So As $= 0.006(25 \times 30) = 4.5$ cm2

Is adopted : 6T12

With Ace $= 6.79$ cm^2

Transverse reinforcement :

It takes : $4\varphi 6$(A $= 1.13$ cm^2)

With. Spacing :

e $= \min(20$ cm$, 15 m_l)$

e $= \min(20$ cm$, 15 \times 1.2)$

e $= \min(20$ cm$, 18$ cm$)$

e $= 18$ cm

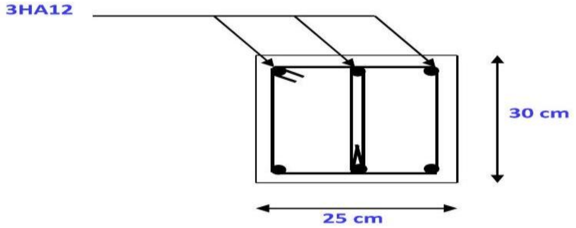


Figure VI.8: Diagram of Reinforcement ground beam

VI. 7 Conclusion:

The study of the foundations is a crucial step in the calculation of a book. The choice of foundation type is based on various parameters, including the characteristics of the soil and the geometrical requirements of the structure. For our project, we opted for the **isolated footings** under the columns and the **continuous footings** under the walls of sailing. These solutions have been carefully designed and verified in accordance with the standards in force, thus ensuring their compliance and their relevance to the specific needs of the work.

Conclusion general

This project of end of study allowed us to highlight the theoretical knowledge acquired during the duration of studies to analyze and study a project of building real. We understood how important it is to analyze a structure prior to the calculation. The analysis of the structure of a book is a very important step that can make a good earthquake-resistant design at the lowest cost.

The primary goal is, of course, the protection of human lives during a major earthquake.

The overabundance of sails in a structure does not automatically mean good resistance in relation to the earthquakes, but can be detrimental to the stability of the structure when the latter are badly placed.

Finally, this study has allowed us to give the theoretical learning of the training cycle of the engineer and, especially, to learn the different techniques of computation, the concepts and regulations governing the field of study.

Note that finally, this project is for us a first experience and was very beneficial and that the use of the computer tool has allowed us to save a lot of time, but the mastery of the software remains a very important step which requires the knowledge of some basic concepts of engineering sciences.

REGULATION :

- Rules Seismic Algerian RPA99/version2003
- Rules of Calculation of Concrete Reinforced The States Limits BAEL91

Course :

- *Court concrete army master 1 (Rm. DJIREB).*
- *Court concrete army master 2(Rm. MOKHTARI).*
- *Court concrete army master 1 (Rm. HARATHE).*
- *Memory years previous (2020/2021).*

Tools of computer

- *Software AUTOCAD 2021 (drawing).*
- *Software analysis of structures*

Autodeskrio Robot 2021

(modeling).

- *Word 2013(treatment of the texts).*
- *Excel 2010 (calculation).*