SAAD DAHLEB BLIDA'S UNIVERSITY

Faculty of Technology

Department of Civil Engineering

MASTER DISSERTATION IN CIVIL ENGINEERING

Specialty: Steel and Composite Constructions

ANALYSIS OF TOURISTIC CAMP WITH MULTI-BLOCS AND A SEMI-OLYMPIC SUSPENDED SWIMMING POOL

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يهدف هذا المشروع إلى دراسة مركز سياحي يتألف من ثلاث اجزاء مختلفة ذات طابع استعمال متعدد

الجزء الاول يهدف إلى دراسة بناية ذو بنية حديدية متكون من طابق ارضي +3 طوابق عليا +3 طوابق سفلى التي سيتم انجازها بولاية البليدة المصنفة ضمن المنطقة الزلزالية رقم III حسب القواعد الجزائرية المضادة للزلازل (RPA99 version 2003) ..الدراسة الحركية تمت باستعمال برنامج ألي ROBOT STRUCUTRAL ANALYSIS2019

الجزء الثاني يهدف إلى دراسة حضيرة ذو بنية حديدية تكون ايضا من 3 طوابق سفلى خرسانة مسلحة حيث ستبنى هذه الحظيرة من اجل مسبح اولمبي..الدراسة الحركية تمت باستعمال برنامج ألي ROBOT

الجزء الثالث دراسة مسبح اولمبي ذو مساحة كبيرة 325 متر مربع حيث يجب دراسة التفاعل الذي يحدث بين الماء و المسبح اثناء و بعد الزلزال و تأثيره على الطوابق الاخرى..الدراسة الحركية تمت باستعمال برنامج ألي SAP2000V16

Abstract

The current final project composes of three principal parts:

Part 1: study a steel building multi-use with (G+3), located at Blida's province, high seismicity zone,) the analysis provided by ROBOT STRUCUTRAL ANALYSIS 2019 software.

Part 2: study a steel hangar with infrastructure made from reinforced concrete used for a suspended swimming pool, its analysis also provided by ROBOT.

Part 3: study a suspended swimming pool, with an area of 325m² it is an obligation to verify the behavior of the interaction fluid and pool in a seismic state to verify and calculate all sections needed in security state, its analysis provided by SAP2000 V 2016 software.

Resumé

Ce projet de fin d'études se compose en trois parties essentielles :

Partie 1 étude d'un bâtiment R+3 multi usage qu'il sera implanté à la wilaya de Blida zone III, sismicité élevée , la modélisation est donnée par logiciel ROBOT STRUCTURAL ANALYSIS.

Partie 2 : étude d'un hangar simple avec infrastructure en béton armé, fait pour une piscine semi olympique, la modélisation est donnée par logiciel ROBOT STRUCTURAL ANALYSIS.

Partie 3 : étude d'une piscine suspendue semi olympique sa superficie est 325m² en prenant en compte l'interaction fluide-piscine et son influence sur la structure sa modélisation fait par le logiciel SAP2000V16.

THANKS

Firstly we would thank ALLAH for all the sacrifices, patient and all the support that we've been surrounded specially under this pandemic situation.

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Thanks to all our parents, beloved ones, families and friends specially Mourad for all the love and the motivation that they offered us to go forward toward our dreams.

This project is the end of our chapter in university, from the bottom of our hearts we thank every person contribute in our success

Imene BELGACEM

Ihcen BELKHELFA

You will learn you will feel the happiness of your success

When you decide to go out from your comfort zone go forward towards your dream and put the negative traditional vibes in side

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LIST OF SYMBOLS

f_{c28}	Compressive strength
f_{t28}	Tensile strength
f_y	Yield stress
f_u	Ultimate stress
E	Modulus of elasticity
G	Shear modulus
V	Poison of linear thermal expansion
α	Coefficient of ration in elastic stage
G	Dead load
Q	Live load
M_{sd}	Bending moment
A	Cross section area
W_{pl}	Plastic section modulus of the number
M_{plrd}	Plastic moment of resistance of the cross section
V_{sd}	Shear force
Nsd	Axial force
f_{adm}	Admissible deflection
$\mathcal{X}$	Buckling reduction's factor
M_{cr}	Critical buckling moment
I_{eff}	Effective moment of inertia
I	Moment of inertia
Ø	Capacity reduction factor
α	Imperfection factor
k_c	Moment correction factor
λ	Slenderness ration
β	Reduction factor
$\mathcal{X}_{lt}$	Reduction factor for lateral tensional buckling
ε_y	Yield strain
A	Acceleration
Δx	Displacement
V_{ref}	Basic wind velocity
P_{ref}	Basic wind pressure
C_r	Roughness factor

I_V	Turbulence intensity
C_e	Exposure coefficient
L	Length
b	Width
d	Depth
μ	Snow load shape
M_{su}	Ultimate moment in span
M_{ss}	Service moment in span
A_{min}	Minimal cross section area
τ_u	shear stress
d	nominal bolt diameter
d0	hole diameter
n	number of friction surfaces
ɑ	Coefficient of ration in elastic stage
tp	Thickness of the plate
A_s	tensile strength area
F_{pcd}	Design preload force
F_{tEd}	Design tensile force per bolt
F_{tRd}	design tension resistance per bolt
F_{vrd}	design shear resistance per bolt
μ	slip factor
k	factor defined where it occurs
g	acceleration of gravity
Q	behavior factor
ε	viscous damping ratio (%)
s	spacing of transverse reinforcement
γ_c	partial factor for concrete
γ_s	partial factor for steel
γ_m	partial factor for material property
I_a	second moment of steel area in composite part
I_c	moment of steel area in composite part
I_s	second moment of area of the rebar composite part
M_0	Total mass of water in pool
M_i	Impulsive mass
M_c	Convective mass

CONTENTS

GENERAL INTRODUCTION

'Civil engineering is that field of engineering concerned with planning design and construction for environmental control development of natural resources buildings transportation facilities and other structures required for health welfare safety employment and pleasure of mankind : FEDRICK.S. Merit hand book for civil engineers'

From the previous inspirational quote of ***Mr FEDRICK***, we can see the aim of any civil engineer is planning, designing, estimating supervising construction, managing construction and execution of structures like buildings, bridges, roads…
One of the most concerned type and a pillar of this large field is carry out of building (planning and construction) which this last require any engineer a clear steps and a source of knowledge because it is all about an application of certain fundamental rules on the type of structure you have, we take an example of one of the structural analysis: (earthquake) its analysis is very important to see the seismic and the dynamic behavior to evaluate all the kind of internal forces and stress to avoid the collapse , any engineer in this field must have essential skills because it's about assuring the security of human's life , take in consideration each unfavorable approach , problem –solving and the decisions making.

Those last years , in Algeria ,a new type of structure has appeared and its progress is still going on the path till now : recently the steel building is being used for almost every type of structure including high-rise buildings, bridges … compared to the concrete it has a lot of advantages : rehabilitee : specially if properly maintained with painting , the properties of steel do not change appreciably with time , large elasticity and ductility: means little risk of failure , its members might be cut and prepared for assembly and design in factories because there is a great adaptation to prefabrication , lesser construction time , high strength and light weight nature which this last will influence on the load applied on structure because the dead load is considered as the bigger part of the total structure.

Our present work consists of large studying of a touristic camp with its deferent types of materials / uses and this composes from three parts:

Part 1: Steel building

The first part of this project is about to study a steel building with a composite slab , composed of 7 stages , firstly we introduce some basic information about the materials we've worked with , also the codes we've followed then we pass to the second stage that handle the pre-sizing of elements (beam and joist) in construction and finale phase and columns also, then we went to explore the climatic evaluation so through the third stage we discovered how to calculate the wind and snow pressure but it considered very small compared to the seismic pressure due to

many reasons ; region , type of structure (encountered with two blocs)… after this stage we have continued to evaluate the load and check the secondary elements here in our case we have only (accroterion and balanced staircase) .
all the information obtained from those four chapters are very essential for the fifth chapter which is the earthquake load design on structure where we've evaluated the seismic response by response spectrum function and verify all the points needed ,after we've assured the security in dynamic load ,then it was the time to pass to the instability states (internal force check) : it is the verification of elements stage where we've verified the buckling , resistance strength of our deferent elements (columns, beams, bracing) then we pass to the design between element that might be with bolts or weld .
So our first bloc is very ready to be excused on reality after achieving and arriving till the point of assembly design.

Part2: hangar

we have an hangar situated next to the steel building its use for a swimming pool we've evaluated its climatic load (wind in two directions) and we've verified all our elements safely and finally with the assembly design.

Part 3 : suspended swimming pool:

as we mentioned in the previous part there is a suspended semi Olympic swimming pool in the hangar situated above two basements ,we started this part with pre-sizing and presentation of the pool then we've passed to a large research about the interaction between fluid and pool to determine the sloshing phase and check the effect of this pool on the whole structure (displacement , shear reaction) , after it we've passed to reinforcement calculation for beam, column and wall then the last stage is the foundation calculation .

1.1 GENERALITIES

1.1.1 Introduction

The aim of this project is to study a touristic camp that composes of: steel building, reinforced concrete infrastructure and hangar with a semi-Olympic suspended swimming pool. This camp will be located in the city of Khezrouna in Blida's province.

1.1.2 Description

- **Composition**

- Two buildings unsymmetrical in shape, weight and type of materials used (steel, reinforced concrete) as well as in geometry.
- A seismic joint separated the buildings between them.
- In the two buildings, there are two basements, and an underground.

- **Geometry**

1. Steel building

Elevation dimensions

Total height with acroterion :……………….……..……. 20,21 m.

Total height without acroterion : …………………………..…. 19,21 m.

Ground floor's height : ………………………..….... 5,78 m.

1^{st}, 2^{nd} storey's height : …………………………...... 4.42 m.

3^{rd} storey's height : ………………………..……. 4.59 m.

Plan dimensions

Total length : ……………….…………….. 37.03m.

Total width : ………….……………………. 15,36m.

2. Elevated swimming pool

Elevation dimensions

Total height : ……………….…..……..……….. 2,5m.

Plan dimensions

Total length : ……..……………...…………. 25m.

Total width : ……..…………………………..…..13m.

3. Infrastructure (reinforced concrete)

Elevation dimensions

Underground's height : …………………….…….. 3,06m.

Basement's height : …………………………….. 3,4m.

4. Hangar

Plan dimensions

Total length : …………………...………. 39,1m.

Total width : ……………………………..19,68m.

Elevation dimensions

Total height : ……………..……………… 10,2m.

1.1.3 Codes / regulations

abbreviation	Title	use
RPA99 2003	règlement parasismique algérien pour l'étude sismique.	all
CCM97	règles de calcul des constructions en acier.	all
RNV 2013	Règlement de neige et vent	Steel building , hangar
BAEL91	béton aux états limite.	Swimming pool
EC3	Design of steel structure	Steel building , hangar
EC4	Design of composite steel and concrete structure	Steel building
EC8	Design of structures for earthquake resistance silos -tanks	Swimming pool
	Calcul des reservoirs	Swimming pool
DTR C2.2	Document technique (charges permanentes et d'exploitations).	all

Table 1.1.1 : Codes used.

1.1.4 Classification according to RPA99 version 2003

This camp is located in Blida which this last is classified as a zone of high seismicity: zone III, it is classified as being a common building with a medium importance because the total height does not exceed 48m, and this type of structure can accommodate more than 300 persons simultaneously so it's group 2 as it will be located on desk ground with stress 1,8bar.

1.1.5 Characteristics of the chosen materials

- **Reinforced concrete**

Concrete is defined by a high value of compressive strength at the age of 28 days, Fc28**.**

Concrete is defined by a low value of its tensile strength: $f_{t28} = 0.06f_{c28} + 0.6$.

A CPA concrete dosed at 350 KG / m³ was used in this project, the characteristics:

density……………………………………………………………$\rho = 25\text{kn/m}^3$

Compressive strength………………………………………………$f_{c28} = 25$ MPA

Tensile strength …………………………………………………$f_{t28} = 2{,}1$ Mpa

Modulus of elasticity …………………………………………E = 14000MPA.

Deferred deformation modulus:……………………………$E_{v28} = 10818.86$MPA.

Longitudinal deformation……………………………………$E_{i28} = 32164.2$ MPA.

Partial factors for materials for ultimate limit states must be used:

Design situation	for concrete γ_c	for reinforcing steelγ_s
Persistent & Transient	1,5	1,15
Accidental	1,2	1

Table 1.1.2: Partial factors for materials for ultimate limit states.

Service limit state SLS

The service stress of concrete is given by: $\sigma_{bc} = 0{,}6 \times f_{c28} = 15MPA$

Ultimate limit state ULS

The ultimate stress of concrete in compression is given by: $\sigma_{bc} = \frac{0{,}85 f_{c28}}{\gamma_b}$

- **Structural steel for the frame elements**

The grad of steel with its characteristics obtained: (CCM97 page 15)

Yield stress $f_y = 275$ MPA

ultimate stress $f_u = 430$MPA

Modulus of elasticity E = 2.1 10^5 MPA

Shear modulus G = 8.1 10^4 MPA

Poison ration in elastic stage V = 0.3

Coefficient of linear thermal expansion $\alpha = 12 . 10^{-6}$ per ^{0}C

- **Connecting device**

Bolting

In our project we used two types of bolts:

- Non-preloaded bolts class for secondary elements.
- Preloaded bolts for the principal main frames.

Welded connections

Welds are made for plates, which this operation is considered more rigid than the bolting connection and the deference is in creating a partial embedding.

1.2 PRE-SIZING OF ELEMENTS

1.2.1 Introduction

In this stage, we'll make an approximate estimate of the profiles cross sections dimensions of our current building and that going to be based on the principal of the descent of load that transmitting from floors to the main principal elements.

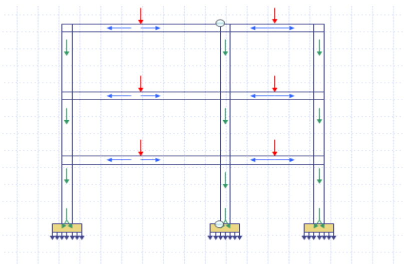


Figure 1.2.1: Load path.

1.2.2 Evaluation of Dead / live load (G, Q)

- **Terrace floor**

Element	Density$(\mathrm{kN/m^3})$	Thickness(cm)	Weight $(\mathrm{kN/m^2})$
Gravel	17	4	0,68
multi-layered waterproofing	6	2	0,12
Slope	22	10	2,2
Cork thermal insulation	4	4	0,16
Reinforced concrete slab	25	10	2,5
HB55	-	-	0,15
Cement plaster	18	2	0,360

Table 1.2.1 Dead load for terrace floor.

Global dead load $= 6.17\mathrm{kN/m^2}$; live load $\mathrm{Q} = 1\mathrm{kN/m^2}$

- **Current floor**

element	Density $(\mathrm{kN/m^3})$	Thickness (cm)	Weight $(\mathrm{kN/m^2})$
floor tile	22	2	0,44
mortar	20	2	0,40
sand	17	2	0,34
Reinforced concrete slab	25	10	2,5

Plaster coating	10	2	0,20
HB55	-	-	0,15
Partition wall	-	-	1

Table 1.2.2 Dead load for current floor.

Global dead load **G** $= 5{,}03\text{kN/m}^2$

- **Exterior walls**

  $G = 2{,}76\text{kN/m}^2$

- **Live load**

Floor	Nature	Q (KN/m²)
Terrace	inaccessible terrace	1
3rd floor	restaurant room	2,5
2nd floor	sports Hall	4
1st floor	sports Hall	4

Table 1.2.3 Live load for current floor.

1.2.3 Frame element's pre-sizing

- **Introduction**

A composite floor structure is presented for a steel-concrete model (work together) in the form of ribbed steel as it allows a good load's distribution. In order to avoid the sliding between the ribs and concrete used, the lateral walls of the steel trays must be stamped.
The calculation of the mixed floor is done in two steps: the construction and final phase.

- **Joist**

-Type 01: 8,02m span joist

It has a span of 8,02 m and spacing of 3m.

Here we use an approximate and simplified formula:

$\frac{L}{25} < h < \frac{L}{15}$ $320.8 \leq h \leq 534.66$

so we choose an **IPE300** with $W_{ply} = 628{,}4\text{cm}^3$.

Construction phase

Profile's self weight ………………..……..……..$g_p = 0.422\ ^{KN}/_{ml}$

Fresh concrete's weight ………..……....…..…$g = 2.5\ ^{KN}/_{m^2}$

HB55weight ……………...……………………$g_t = 0.15\ ^{KN}/_{m^2}$

live load of workers ………………………Q = 0.75 $^{KN}/_{m^2}$

- ***Load Combinations***

Ultimate limited state ULS

$q_u = 1.35\ g_p + 1.35(g + g_t) \times 3 + 1.5Q \times 3 = 14{,}67\ KN/ml$

Service limited state 'SLS'

$q_s = g_p + (g + g_t) \times 3 + Q \times 3 = 10{,}622\ KN/ml$

- ***Bending moment check***

Profile's class: Web $\frac{d}{tw} = \frac{248{,}6}{7{,}1} = 35 < 72 \ldots\ldots\ldots\ldots\ldots\ldots$ class1.

Flange $\frac{c}{tf} = \frac{60}{10{,}7} = 5{,}60 < 10 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots$ class1.

So the profile is class1: the bending moment in the cross section must satisfy this

condition: $M_{sd} \leq M_{plrd} = \frac{W_{pl} \times f_y}{\gamma_{mo}}$ $M_{sd} = \frac{q_u l^2}{8} = 118{,}24$ KN.m ; M_{plrd} =172,81 KN.m

M_{sd} =118,24 KN.m $\leq$ M_{plrd} = 172,81 KN.m………………checked.

- ***Shear force check***

It must be verified: $V_{Sd} \leq V_{plRd} = \frac{f_y A_v}{\sqrt{3}\ \gamma_{M_0}}$ $V_{sd} = \frac{q_u}{2} l$ =58,82KN

$A_{vz} = A - 2bt_f + (t_w + 2r)t_f$ = 1769,82mm² $V_{plrd} = \frac{275 . 1769{,}82}{1\sqrt{3}} 10^{-3}$ = 281KN

V_{sd} =58,82 KN $\leq$ V_{plrd} =281 KN …………………..……checked.

- ***Interaction of shear force***

V_{sd} =58,82KN $\leq$ $0.5V_{plrd}$ = 140,5 KN ……………..……checked.

No interaction between the shear force and the bending moment so there is any need to reduce the resistance in bending.

- ***Verification of deflection***

It must be checked: $\frac{5\times q_s \times L^4}{384\times E \times I_Y} \leq \overline{f}$ $\quad f_{adm} = \frac{L}{250}$

$f^{max} = \frac{5\,.10{,}62.8020^4}{384.2{,}1.10^5.8356.10^4} = 32{,}6\ mm$ $\quad f_{adm} = \frac{L}{250} = 32{,}08$ mm

$f^{max} = 32{,}6$ mm $\leq f_{adm} = 32{,}08$mm………….……….not checked.

Here in this case we should add props so:

$f^{max} = \frac{5\,.10{,}62.4010^4}{384.2{,}1.10^5.8356.10^4} = 2{,}03\ mm$

- ***Verification of lateral torsional buckling***

$$Msd \leq M_{bRd} = \chi_{LT} \frac{W_{ply} f_y}{\gamma_{M_1}} \beta_W$$

$$\chi_{LT} = \frac{1}{\left(\Phi_{LT} + \sqrt{(\Phi_{LT})^2 - (\overline{\lambda}_{LT})^2}\right)}$$

M_{brd} **:** The design resistance of not maintained element laterally at bucking.

The value of $\overline{\lambda}_{LT}$ was determined by the following formula:

$$\lambda_{LT} = \frac{L / iz}{\left[1 + \frac{1}{20}\left(\frac{L / iz}{h / t_f}\right)^2\right]^{0,25} \sqrt{C_1}}$$

= 69,73

$\overline{\lambda}_{LT} = \frac{69{,}73}{93{,}9\varepsilon} = 0{,}8 > 0{,}4$ … … there is a risk of lateral buckling .

$\beta_w = 1$ … … . . classe 1 $\quad$ Fy = 275MPA $\quad W_{ply} = 628{,}4\text{cm}^3$

$\phi_{LT} = 0{,}5\left[1 + \alpha_{LT}\left(\overline{\lambda}_{LT} - 0{,}2\right) + \overline{\lambda}^2{}_{LT}\right] = 0{,}88$ $\quad \chi_{LT} = 0{,}79$

We calculate now *Mbrd :* $M_{brd} = \frac{0.79\times 275\times 1\times 628{,}4.10^3}{1.1} = 125{,}22\ KN.m$

M_{sd} =118,24KN.m ≤ M_{brd} = 125,22 KN.m …………………checked.

All the verifications needed are checked successfully for the initial phase.

Final phase

The concrete now is hardened, consequently the profile and the slab will constitute a mixed section and work together, and the loads of this phase are:

Profile's weight ……………...……….$g_p = 0{,}422\ ^{KN}/_{ml}$

Permanent load terrace ……...…...………$g = 7.17\ ^{KN}/_{m^2}$

live load ………………..………………Q = 1 $^{KN}/_{m^2}$

- ***Combination of loads***

Ultimate limited state ULS $q_u = 1.35\ g_p + 1.35(g) \times 3 + 1.5Q \times 3 = 34{,}10\ KN/ml$

Service limited state SLS $q_s = g_p + (g) \times 3 + Q \times 3 = 24{,}93\ KN/ml$

- ***Width of the mixed slab***

In the calculations of composite beams and joists, we take into account on each side of the axis of the beam, a width of the slab equal to the smallest value

$\frac{l}{4} = 2m$ ….. l: length of a joist. , e=3m ….e: spacing between joists.

- ***Position of the plastic neutral axis***

$R_{concrete} = 0{,}57 \times f_{c28} \times b_{eff} \times h_c$ $= 0{,}57 \times 25 \times 45 \times 2000$ =1282,5KN.

$R_{steel} = 0{,}95 \times f_y \times A_a$ $= 0{,}95 \times 275 \times 5380 = 1405{,}525$ KN.

$R_{steel} > R_{concrete}$ $R_w = 0{,}95 \times A_{web} \times f_y = 0{,}95 \times 1765{,}06 \times 275 = 461{,}12KN.$

So the neutral axis is located in the top flange of the profile, which is mean:

$M_{plrd}=R_a \frac{ha}{2} + R_b\left(\frac{hc}{2} + hp\right)$ =310,22KN.m

- ***Bending moment check***

M_{sd} =274,16KN.m ≤ M_{plrd} = 310,22 KN.m …………………checked.

- ***Verification of shearing force***

V_{sd} =136,91KN ≤ V_{plrd} = 281 KN ……………..…………checked .

- ***Verification of the interaction of shear force***

V_{sd} =136,91KN ≤ 0.5V_{plrd} = 140,5KN …………..…...………checked.

- ***Deflection***

It Must be checked: $f^{\max} = \frac{5}{384}\frac{q_s.L^4}{E\ I_{el}} \leq \bar{f}$ $I_c = \frac{A_a \times (H_c+2H_p+H_a)^2}{4(1+mv)} + \frac{b_{eff} \times Hc^3}{12m} + I_a$

$v = \frac{A_a}{A_b} = \frac{5380}{45 \times 2000} = 0{,}059$ $m = \frac{E_a}{E_b} = 15$

$I_c = \frac{5380\times(45+2\times55+300)^2}{4(1+15\times0{,}059)} + \frac{2000\times45^3}{12\times15} + 8356\times10^4 = 2{,}32\ 10^8 mm^4$

$f1^{max} = \frac{5\ .24{,}93.8020^4}{384.2{,}1.10^5.2{,}32.10^8} = 27{,}53\text{mm}$ $\quad f_{adm} = \frac{L}{250} = 32{,}08$ mm

$f^{max} = f_c + f1_{max} = 2{,}03 + 27{,}53 = 29{,}56\text{mm} \leq f_{adm} = 32{,}08\text{mm}$…………checked.

- ***Verification of lateral buckling***

In this phase it is not necessary to verify the lateral buckling, because the top flange is held by the concrete slab.

- ***Calculation number of connectors***

Done by: $n = \frac{R_l}{P_{rd}}$

HB55…………………………..…h=45mm.

Diameter ………………………..d=19mm.

We determine $P_{rd} = K_t \times \min\left(P_{rd1}, P_{rd1}\right)$

$$P_{rd} = \min\begin{cases} \frac{\sqrt{f_{c28}\times E_c}}{\gamma_v} \times 0{,}29 \times \alpha \times d^2 \\ 0{,}8 \times f_u \times \frac{\pi\times d^2}{4\times\gamma_v} \end{cases}$$

f_{c28}=25MPA $\quad E_c$=30, 5 KN/mm² $\quad f_u$=400MPA $\quad \gamma_v$=1, 25

$\alpha = 0{,}2(\frac{h}{d} + 1) \leq 1$ ………………… α =0, 67

$P_{rd1} = 0.29 \times 0.67 \times 19^2 \times \frac{(25\times30.5.1000)^{0.5}}{1.25} = 49\text{KN}$

$P_{rd2} = \frac{0.8\times400\times\pi\times19^2}{4\times1.25} = 72{,}58\text{KN}$

K_t :reduction factor done by this expression : $\quad K_t = \frac{0{,}85}{\sqrt{nc}} \times \frac{b_0}{h_p} \times (\frac{h}{hp} - 1)$

nc =1 (number of connectors per rib) $\quad K_t = 0{,}99$ $\quad$ so $\quad P_{rd} = 48{,}51KN$

we determine R_L

$R_L = \min(R_{concrete}; R_{steel}) = \min(1282{,}5\text{KN}; 1405{,}525) = 1282{,}5\text{KN}$.

So $\quad n = \frac{R_l}{P_{rd}} = \frac{1282{,}5}{48{,}51} = 26{,}43$

so we choose 27 connectors for a half span.

The minimal value of spacing must be superior from the diameter 5times.

$e_{min} \geq 5.d = 5\times19 = 95\text{mm}$.

We determine now the spacing value

$e = \frac{L}{n-1} = \frac{8{,}02}{27-1} = 0{,}30m$ $\quad$ so we choose a spacing of e =30cm.

For the total length of joist we have 54 connectors.

- ***Type 02 : 6,54m span joist***

$\frac{L}{25} < h < \frac{L}{15}$ 261, 6 ≤ h ≤ 436 so we choose IPE270

Construction phase

$q_u = 14{,}59\ KN/ml$ $q_s = 10{,}561\ KN/ml$

	M_{sd}	M_{plrd}	checking
IPE270	78	133,1	Checked
	V_{sd}	V_{plrd}	**checking**
IPE270	47,7	232,23	Checked
	V_{sd}	$0.5V_{plrd}$	**checking**
IPE270	47,7	116,11	Checked
	f^{max}	f_{adm}	**checking**
IPE270	4,03	32,7	checked

Table 1.2.4: Joint (6,54m) check in construction phase.

Final phase

$q_u = 34\ KN/ml$ $q_s = 24{,}87\ KN/ml$

Width of the composite slab is: $b_{eff} = 1635mm$

- ***Position of the plastic neutral axis***

$R_{concrete} = 0.57 \times f_{c28} \times b_{eff} \times h_c$ $= 0.57 \times 25 \times 45 \times 1635$ =1048KN

$R_{steel} = 0{,}95 \times f_y \times A_a$ $= 0{,}95 \times 275 \times 4590 = 1199{,}13\ KN$

$R_{steel} > R_{concrete}$ So: M_{plrd}=243,13KN.m

	M_{sd}	M_{plrd}	checking
IPE270	181,77	243,13	Checked
	V_{sd}	V_{plrd}	**checking**
IPE270	111,18	232,23	Checked
	V_{sd}	$0.5V_{plrd}$	**checking**
IPE270	111,18	116,11	Checked
	f^{max}	f_{adm}	**checking**
IPE270	21,02	32,7	checked

Table 1.2.5: Joint (6,54m) check in final phase.

- ***Calculation of connectors***

HB55 ……….……h=45mm. Diameter …………d=19mm.

Connectors number is 22 **connectors** for half span with e=38mm

so we take 44 **connectors** for the total joist's length.

- **Beam terrace floor**

Beams are principal elements, which support loads of the floors and transmit them to columns, they are stressed mainly by a bending moment $\frac{L}{25} < h < \frac{L}{15}$ $447.6 \leq h \leq 746$

so we choose IPE500

Construction phase

Profile's weight ……………………..$g_p = 0{,}907\ {}^{KN}/_{ml}$

Fresh concrete's weight …………..…$g = 2{,}5\ {}^{KN}/_{m^2}$

HB55weight ………………………$g_t = 0{,}15\ {}^{KN}/_{m^2}$

live load of workers ……………………$Q = 0{,}75\ {}^{KN}/_{m^2}$

- ***Joists reactions***

Joist of 8,02m $R_{joist} = \frac{q_u\,l}{2} = 58{,}82KN$ $R_{joist} = \frac{q_s\,l}{2} = 42{,}59KN$

Joist of 6,54m $R_{joist} = \frac{q_u\,l}{2} = 47{,}70KN$ $R_{joist} = \frac{q_s\,l}{2} = 34{,}53KN$

ULS: R_s =58,82+47,70= 106,52KN SLS: R_s = 42,59+34,53= 34,95KN

- ***Combination of load***

$$q_u = 1.35\,g_p + 1.35(g + g_t) \times 0{,}20 + 1.5Q \times 0{,}20 = 1{,}94\ KN/ml$$

$$q_s = g_p + (g + g_t) \times 0{,}20 + Q \times 0{,}20 = 1{,}42\ KN/ml$$

- ***Bending moment check***

profile's class

Web $\frac{d}{tw} = \frac{426}{10{,}2} = 41{,}76 < 72 \ldots\ldots\ldots\ldots\ldots\ldots$ class1.

Flange $\frac{c}{tf} = \frac{73{,}9}{16} = 4{,}61 < 10 \ldots\ldots\ldots\ldots\ldots\ldots..$ class1.

So our profile is class 1

	M_{sd}	M_{plrd}	**checking**
IPE500	227,46	603,35	Checked
	V_{sd}	V_{plrd}	**checking**
IPE500	174,89	693	Checked
	V_{sd}	$0.5V_{plrd}$	**checking**
IPE500	174,89	346,5	Checked
	$f^{max}\ (f_1 + f_2)$	f_{adm}	**checking**
IPE500	10,86		checked

Table 1.2.6: Beam check in initial phase.

- ***Lateral buckling check***

The verification of the lateral buckling in this phase is required by EC4, however the beam is held laterally by the joists on both sides, so the beam is not likely to be buckled.

-Final phase

Profile's weight ……………..……..$g_p = 0{,}907\ {}^{KN}/_{ml}$

Permanent load terrace ……………$g = 7{,}17\ \ {}^{KN}/_{m^2}$

Operating overload……………………Q = 1 ${}^{KN}/_{m^2}$

$q_u = 3{,}46\ KN/ml$ $\qquad$ $q_s = 2{,}54\ KN/ml$

- ***Joists reactions***

Joist of 8,02m $\qquad R_{u\ joist} = 136{,}7KN \qquad R_{s\ joist} = 99{,}9KN$

Joist of 6,54m $\qquad R_{u\ joist} = 111{,}18KN \qquad R_{s\ joist} = 81{,}32KN$

ULS: R_s =247,88KN $\qquad$ SLS: R_s =181,22KN

- ***Position of the plastic neutral axis***

Width of the mixed slab $\quad b_{eff} = 2790mm$

$R_{concrete} = 0.57 \times f_{c28} \times b_{eff} \times h_c \quad = 0.57 \times 25 \times 2790 \times 45$ =1789KN

$R_{steel} \quad = 0.95 \times f_y \times A_a \qquad = 0.95 \times 275 \times 11600$ =3030,5KN

$R_{steel} > R_{concrete} \qquad R_w = 0.95 \times A_{web} \times f_y = 0.95 \times 4345{,}2 \times 275 = 1135{,}18KN$

$M_{plrd} = R_a \frac{ha}{2} + R_b \left(\frac{hc}{2} + hp\right)$ =896,27KN.m

	M_{sd}	M_{plrd}	**checking**
IPE500	578,20	896,27	Checked
	V_{sd}	V_{plrd}	**checking**
IPE500	323,8	693	Checked
	V_{sd}	$0.5V_{plrd}$	**checking**
IPE500	323,8	346,5	Checked
	f^{max}	f_{adm}	**checking**
IPE500	32,3	37,3	checked

Table 1.2.7: Beam check in final phase.

- ***Calculation of connectors***

HB55 ………..……h=45mm Diameter …………d=19mm

Connectors number is 37 **connectors** for half span with e=31mm.

So we take 74 **connectors** for the total joist's length.

Note: the profiles found are also checked for the current floor.

Floor	joist 8,02 (m)	joist 6,54 (m)	beam 11,19 (m)
Terrace floor	IPE300	IPE270	IPE500
Current floor	IPE300	IPE270	IPE500

Table 1.2.8: Beams and joist final section

- **Column**

Columns are vertical and straight elements intended to support axial compression loads. They are used to support floors, roofs…and they allow transmitting the entire loads supported to the foundations.

The most requested column's area is: $S = (\frac{11.19+9.73}{2} \times \frac{8,02+6.54}{2})$=76,1m²

Terrace floor ……….…………….7.17$\times$ 75 = 537,75KN

Current floor ………….……….5.03$\times$ 75 = 377,25KN

Joist type 1……………………….0.422$\times$ 4.01 = 1,69KN

Joist type 2…………….……….0,361$\times$ 3.27 = 1,180KN

Principal beam………….……….0,907$\times$ 10.36 = 9,39KN

Height level	Surface	G	Q	1.35G+1.5Q
3^{rd}	76,1	550,01	76,1	856,66
2^{nd}	76,1	939,52	190,25	1553,72
1^{st}	76,1	1329,03	304,4	2250,79
Ground floor	76,1	1718,54	304,4	2776,629

Table 1.2.9: Load evaluation for column

N=856,66KN $\quad A \geq \frac{N}{f_y} = \frac{856{,}66\times 10^3}{275} = 3115{,}12mm^2$

A=31,36cm²……………we choose HEA200 avec A=53,83cm²

- ***Buckling***

It must be checked: $N_{b.rd} \leq \chi \frac{\beta_a \times A \times f_y}{\gamma_{M1}} \beta_A = 1 \ldots\ldots class\ 1{,}2{,}3 \qquad \gamma_{m1} = 1.1$

Note columns are restrained in both directions for four levels so the buckling length is: Lb= L0 / 2 , 4,59 / 2 =2,29m

HEA200, N= 862, 49 KN, A= 53.83cm²

Following Y following Z

$\lambda_y = \frac{229}{8.28} = 27,65 \quad \leq \quad \lambda_z = \frac{229}{4.98} = 45{,}98$

The value of damping in z is bigger than y which means risk of buckling in z

Choice of the buckling curve: (CCM97.Tab 5.5.3)

$\frac{h}{b} = \frac{190}{200} = 0.95 < 1.2$ *, tf = 10 ≤100 ……..z-z : c……a=0,49*

$\overline{\lambda_z} = \frac{44.37}{93.9} = 0.47 \qquad \varphi = 0{,}5\left[1 + \alpha\left(\bar{\lambda} - 0{,}2\right) + \bar{\lambda}^2\right] = 0.67$

$\chi_z = \frac{1}{\phi_z + \sqrt{{\phi_{zy}}^2 - {\bar{\lambda}_z}^2}} = 0.86 \leq 1$

$N_{brd} = \frac{0.86\times 1\times 275\times 5383}{1.1} \times 10^{-3} = 1157{,}345KN$

$N_{sd} = 860{,}79 \leq N_{brd} = 989KN \ldots\ldots\ldots$ verified.

We follow those same steps to verify buckling for the other floor levels.

Note! Because Any Cross section's profile is made with 12m of length, to avoid the falling of profiles and to respect the economic pillar we choose for columns those following sections:

Height level	Column's section	Effort (KN)
3rd	HEA360	856,66
2nd	HEA360	1553,72
1st	HEA500	2250,79
Ground floor	HEA500	2776,629

Table 1.2.10: Final columns cross sections.

1.2.4 Conclusion

The pre-sizing chapter helped us to discover that the load constitutes a bigger part of the total load on a structure (dead, live), as we noticed the term elements and support are defined relative to each other and that was discovered through the load path which is mean it is necessary to work with this point wisely with a reasonable results due to the project you work.

1.3 CLIMATIC LOAD STUDIES

1.3.1 Introduction

In Algeria, the climatic load (wind or snow) calculation is based on the perspective code RNV2013. It is generally depends on many pillars: the region you will build in and the shape of your building.

1.3.2 Wind load introduction

The wind load is a climatic natural load, in Algeria its calculation is based on the perspective code RNV2013. It is generally depends on the region you will build in, city, mountains, desert and the shape of your structure.
In case of building or hangar with (two walls exposed to wind interior and exterior) the wind load pressure depends to the peak velocity pressure and external and internal coefficients.
We set other type for example bridge or stadium case here the whole structure is subjected to wind so the calculation here is based on the peak velocity and the external coefficients.

Peak velocity pressure is calculated by determining some factors about the geometry of your building and the basic wind pressure, (related to the zone).

1.3.3 Wind coefficients

- ***Wind zone***

This camp is located in Blida's province, so according RNV code it's zone I.

Wind zone I: basic wind velocity Vref = 25 m/s Basic wind pressure pref = 375 N/m²

- ***Topography coefficient***

The topographic coefficient takes into account the increase of the wind's speed specially when it blows over obstacles such as hills , isolated subsidence and in our case it is a flat site with : Ct(z) = 1.

- ***Terrain category***

Terrain category I….. Lake or flat and horizontal area with negligible vegetation and without obstacles, therefore:

K_t	z_0	Z_{min}	Ɛ
0.170	0.01	1	0.44

Table 1.3.1: Terrain parameters.

- ***Roughness factor***

It accounts for the variability of the wind velocity at the site of the structure due to:

$C_r = K_t \times Ln\left(\frac{z}{z_0}\right)$ $\quad for \quad z_{min} \leq z \leq 200m.$

$C_r = K_t \times Ln\left(\frac{z_{min}}{z_0}\right)$ $\quad for \quad z \leq z_{min}.$

- ***Turbulence intensity***

Defined as the standard deviation of the turbulence divided by the mean wind velocity:

$I_V\ (z) = \frac{1}{C_t \times \ln\left(\frac{z}{z_0}\right)}$ $\quad for \quad z > z_{min}$

$I_V\ (z) = \frac{1}{C_t \times \ln\left(\frac{z_{min}}{z_0}\right)}$ $\quad forr \quad z \leq z_{min}$

- ***Exposure coefficient***

Done by the following formula: $C_e(z) = C_t{}^2 \times C_r{}^2 \times \{1 + 7I_v\}$

- ***Peak velocity pressure***

Done by the following formula: $q_p(z) = q_{ref} \times C_e(z)$

All the results are summarized in this table:

Z_e (m)	cr	Iv	ce	P ref $[N/m^2]$	qp $[N/m^2]$
19,21m	1,285	0,132	3,152	375	1182

Table 1.3.2: Wind load coefficients

1.3.4 Wind directions

1-**Vertical face W1 (b=28,47m; d=14,56m)**

- ***Reference height H (m)***

According to figure 2.1 page 51, in our case $h = 19{,}21m \leq b = 37{,}03m$ *so* The reference height in our case is: $z_e = h = 19{,}21m.$

- ***Dynamic factor***

The detailed procedures for calculating the structural factor Cd is given in the following expression: $\frac{1+2\times g\times I_v\times\sqrt{(Q^2+R^2)}}{1+7\times I_v}$

g : Peak factor defined as the ratio of the maximum value of the fluctuating part of the response to its standard deviation.

Q^2 : Background factor, allowing for the lack of full correlation of the pressure on the structure.

R^2 : Resonance response factor, allowing for turbulence in resonance with the vibration mode.

symbol	expression	results
z_{eq}	$0.6\times h$	11,526
L_i	$300\times(\frac{z}{200})^{\varepsilon}$	85,46
Q^2	$\frac{1}{1+0.9\times(\frac{b+h}{L_i})^2}$	0.71

Table 1.3.3: Dynamic coefficient parameters

F	H / 250	0,076
$\eta_{1.x}$	$\frac{0.5}{\sqrt{F}}$	1,80
δ	$\delta=\delta_s+\delta_a \qquad \delta_a=0$	0.08
V_m	$C_r\times C_t\times V_{ref}$	32,32
N_x	$\frac{\eta_{1.x}\times L_i}{V_m}$	4,73
R_n	$\frac{6.8\times N_x}{(1+10.2N_x)^{5/3}}$	0,048
η_h	$\frac{4.6\times h\times N_x}{L_i}$	5,03
η_b	$\frac{4.6\times b\times N_x}{L_i}$	9,21
R_h	$\frac{1}{\eta_h}-\frac{1}{2\times{\eta_h}^2}\times(1-e^{-2\eta_h})$	0.17
R_b	$\frac{1}{\eta_b}-\frac{1}{2\times{\eta_b}^2}\times(1-e^{-2\eta_b})$	0,10
R^2	$\frac{\pi^2}{2\times\delta}\times R_N\times R_h\times R_b$	0,053
υ	$\eta_{1.x}\times\sqrt{\frac{R^2}{Q^2+R^2}}\geq 0.08$	0.45

G	$\sqrt{\{2 \times \ln(600\upsilon)\}} + \dfrac{0.6}{\sqrt{\{2 \times \ln(600\upsilon)\}}} \geq 3$	3.52
Cd	$\dfrac{1 + 2 \times g \times I_v \times \sqrt{(Q^2 + R^2)}}{1 + 7 \times I_v}$	**0.94**

Table 1.3.4: Dynamic coefficient's parameters2.

- ***Pressure coefficients***

A. External pressure coefficient

1.Vertical wall:

For the calculation of Cpe values. The walls should be divided as shown in the figure of our code (page80). b= 37,03m d=14,56m 2h = 38,42m $e = Min(b; 2h)$ e = b = 37,03m

So: d < e.

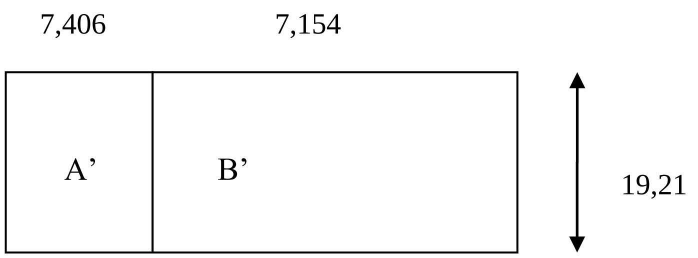


Figure 1.3.1: Vertical wall W1

The loaded surfaces of the walls are all considered bigger than 10m² .So: Cpe = Cpe10.

Zone A': Cpe10 = -1.0 zone D : Cpe = +0,8

Zone B' : Cpe10 = -0.8 zone E : Cpe10 = -0,3

2. Roof

In our case, we have a plate roof with acroterion of $h_p = 100cm$.

e =37,03m , b=37,03m , d=14,56m

The Loaded surfaces of the walls are all considered bigger than 10m² .So: Cpe = Cpe10

$\frac{h_p}{h} = 0,05$ So:

Zone F : Cpe10 = -1,4 zone H : Cpe = -0,7

Zone G : Cpe10 = -0.9 zone I : Cpe10 = -0,2

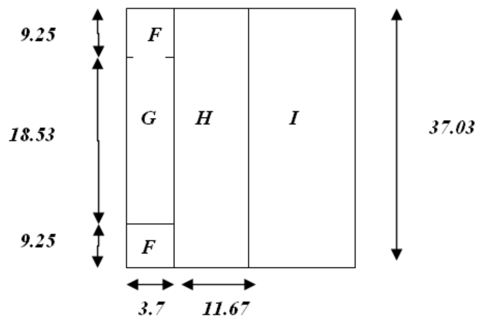


Figure 1.3.2: Roof W1

B. Internal pressure coefficient

The wind pressure acting on the internal surfaces of a structure should be obtained following those steps:

In case of multi storey building with interior walls the internal pressure coefficient values are:$C_{pi} = -0.6$ $C_{pi} = 0.4$ (both directions).

- ***Wind pressure***

The case if we have two sides of walls (exteriors and interiors): $W(z) = q_p \times (C_{pe} - C_{pi})$.

	Zone	q_p	C_{pe}	C_{pi}	W(z)	C_{pi}	W(z)
H=19.21m	A’	1182	-1.0	-0.6	-472,8	0.4	-1654,8
	B’		-0.8	-0.6	-236,4	0.4	-1418,4
	D		+0.8	-0.6	+1654,8	0.4	+472,8
	E		-0.3	-0.6	354,6	0.4	-827,4
Roof	F	1182	-1.4	-0.6	-962,7	0.4	-2166,07
	G		-0.9	-0.6	-361,01	0.4	-1564,38
	H		-0.7	-0.6	-120,33	0.4	-1323,71
	I		-0.2	-0.6	+481,35	0.4	-722,02

Table 1.3.5: Wind pressure W1.

- ***Friction***

Friction should be considered in case: $A_p \geq 4 \times A_l$

A_p =279,69m² ≤ 4 × $A_l = 2845{,}38m^2$….so the friction isn’t considered in this case.

- ***Global result W1***

	Zone	C_d	A_{ref}	W(z)	F_1	W(z)	F_2
Z=19,21m	*A’*	0,94	142.26	-472,8	-63224	-1654,8	-221287
	B’		137,42	-236,4	-30536,36	-1418,4	-183221,5
	D		711,34	+1654,8	1106507,70	+472,8	316142,25
	E		711,34	354,6	237106,69	-827,4	-553248
roof	*F*		34.22	-962,7	-30966	-2166,07	-69675
	G		68.541	-361,01	-23259	-1564,38	-100790
	H		432.14	-120,33	-48879	-1323,71	-537706
	I		0	+481,35	0	-722,02	0

Table 1.3.6: Wind force W1.

2-Vertical face W2 ***(b=14,56, d=37.03)***

- ***Reference height H (m)***

According to figure 2.1 page 51, in our case $b = 14{,}56m < h = 19{,}21m \leq 2b = 29{,}12m \quad so$:

The reference height in our case is: $z_e = h = 19{,}21$ and $z_e = b = 14{,}56$

- ***Peak velocity pressure***

Done by the following formula: $q_p(z) = q_{ref} \times C_e(z)$

All the results are summarized in this table:

Z_e (m)	cr	Iv	Ce	P ref $[N/m^2]$	qp $[N/m^2]$
19,21m	1,285	0,132	3,152	375	1182
14,56m	1,238	0,137	3	375	1127,11

Table 1.3.7: Wind load coefficients

- ***Dynamic coefficient***

We follow the same steps mentioned in the previous direction we found:

for h=19,21m…………………Cd=1,02

For h=14,56m…….…………..Cd=0,92

- ***Pressure coefficients***

A. External pressure coefficient

1.Vertical wall:

For the calculation of Cpe. The walls should be divided as shown in the figure of our code.

b= 14,56m d=37,03m 2h = 38,42m

$e = Min(b; 2h)$ e = b = 14,56m So: d ≥ e.

The loaded surfaces of the walls are all considered bigger than 10m².

So: Cpe = Cpe10.

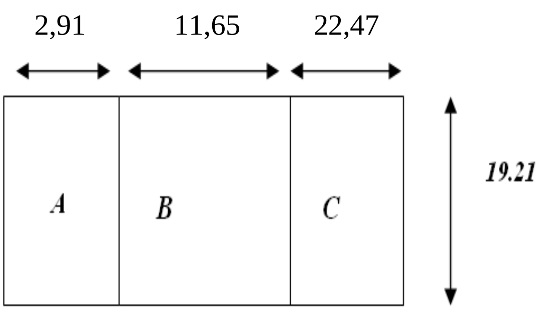


Figure 1.3.3: Vertical wall W2

A : Cpe10 = -1.0 D : Cpe = +0,8

B : Cpe10 = -0.8 E : Cpe10 = -0,3

C : Cpe10 = -0.5

2.**Roof**

e =14,56m , b=14,56m , d=37,03

The loaded areas of the wall : G , H and I are all considered bigger than 10m² .So: Cpe = Cpe10

The loaded area of the wall F is considered less than 10m, which is mean:

$1m^2 < A_{F,} = 9{,}68 < 10m^2 \rightarrow c_{pe\,F} = C_{pe1} + \left(C_{pe10-}C_{pe1}\right)log(A)$

Zone F : Cpe =-1.4 Zone H : Cpe = -0,7

Zone G : Cpe10 = -0.9 zone I : Cpe10 = -0,2

- ***Wind pressure***

	Zone	q_p	C_{pe}	C_{pi}	W(z)	C_{pi}	W(z)
Z : 14,56m	A	1127,11	-1.0	-0.6	-450,844	0.4	-1577,954
	B		-0.8	-0.6	-225,422	0.4	-1352,532
	C		-0.5	-0.6	112,711	0.4	-1014,399
	D		+0.8	-0.6	1577,954	0.4	450,844
	E		-0.3	-0.6	338,133	0.4	-788,977
Z : 19,21m	A	1182	-1.0	-0.6	-472.8	0.4	-1654.8
	B		-0.8	-0.6	-236.4	0.4	-1418.4
	C		-0.5	-0.6	118.2	0.4	-1063.8
	D		+0.8	-0.6	1654.8	0.4	472.8
	E		-0.3	-0.6	361.01	0.4	-842.36
roof	F	1203.375	-1.4	-0.6	-962.7	0.4	-2166.06
	G		-0.9	-0.6	-361.01	0.4	-1564.38
	H		-0.7	-0.6	-120.33	0.4	-1323.70
	I		-0.2	-0.6	481.35	0.4	-722.022

Table 1.3.8: Wind pressure W2.

- ***Friction***

A_l =711,73m² $\leq 4 \times A_p = 1118{,}76m^2$….so the friction isn't considered in this case.

- ***Result***

Z	Zone	C_d	A_{ref}	W1	F_1	W2	F_2
14,56	A	0,92	42,36	-450,844	-17569	-1577,954	-61492
	B		169,624	-225,422	-35178	-1352,532	-210985
	C		327,16	112,711	33924	-1014,399	-305321
	D		211,99	1577,954	307748	450,844	87928,46
	E		211,99	338,133	65946	-788,977	-153874

19,21	A	1.02	55,90	-472.8	-26958,11	-1654.8	-94353.3
	B		223,79	-236.4	-53962	-1418.4	-323772,21
	C		431,64	118.2	52041	-1063.8	-468362,20
	D		279,69	1654.8	4720876	472.8	134882
	E		279,69	361.01	2019.4	-842.36	-240311.66
roof	F	1.02	5.87	-962.7	-5764	-2166.06	-12969
	G		11.75	-361.01	-4326.7	-1564.38	-18749
	H		94.37	-120.33	-11582.6	-1323.70	-127415
	I		118.34	481.35	58102.2	-722.022	-87152

Table 1.3.9: Wind pressure W2.

What we learned

After reviewing the results obtained we found that if the load hits on supposed direction, this last will create a positive pressure on the wall we call it here windward wall, this positive pressure will help the wall to push up on the building meanwhile it affects the other wall to pull up (leeward wall) as it create suction flow over the roof to get out of the building which is mean a negative pressure.

Regarding the code we are used the external and internal wind pressure coefficients must be determined in both directions, when it hits the length us when it hits the width so those pillars are not fixed, the word of wind ward and leeward will change the moment you change the direction and this last is related directly to the sign of pressure.

1.3.5 Snow load

The characteristic load of snow S per unit of surface in horizontal projection of roofs or any other surface subjected to the accumulation of snow, it is obtained by the following expression S = μ × S_k.

S_k : Characteristic value of snow load on the ground.

μ : Snow load shape coefficient.

According to the snow annex, the area is B so: $S_K = \frac{0{,}04H+10}{100}$

the altitude above sea level H=260m so: $S_K = \frac{0{,}04\times 260+10}{100}$ = 0,204

$\mu = 0{,}8$ ………………….. Flat roof.

S = 0, 8. 0,204= 0, 1632 KN/m².

1.3.6 Conclusion

Through this chapter of climatic load studies, we discovered the influence of the wind directions on the obtained results, as we see the importance of such study that plays an important role in design of tall structure because it exerted loads on building.

1.4 SECONDARY ELEMENTS

1.4.1 Introduction

When designing a building, we take into account two types of elements: structural elements that play an important role in transmitting and withstanding loads and other type of elements called non structural elements which play a secondary role and they do not provide a high bearing resistance comparing to the principal elements.

In this part of project, we have two secondary elements (staircase and acroterion) and through this stage we'll verify, evaluate their loads and finally take up their final sections.

1.4.2 Staircase

There are so many types of steps: bull nose, round ended, filer splayed, balanced steps….

here in our project the type of steps used are balanced staircase.

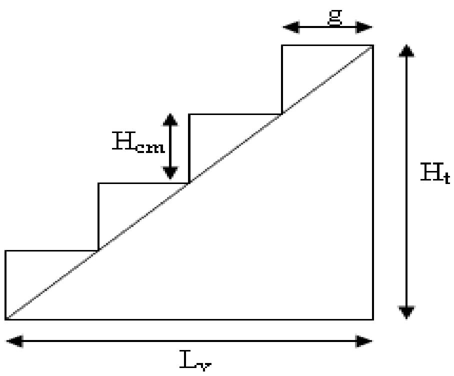


Figure 1.4.1: staircase

Practically we must follow the following conditions:

$22cm \ \leq g \ \leq 33cm \ldots\ldots\ldots\ldots\ldots we\ take\ g = 30cm$

$16cm \ \leq h \ \leq 20cm \ldots\ldots\ldots\ldots\ldots we\ take\ h = 17cm$

g :Stair tread. h: step's height.

To check this condition, we use the empirical formula of **BLONDEL**.

57cm ≤ g+2h ≤ 64cm

g+2h=30+2.17=64cm 57cm ≤ 64cm ≤ 64cm…………………..checked.

so we adopt ***h = 17 cm, g = 30cm***

Number of counter steps: $N_{c.s} = \frac{H}{2.h}$

Number of steps: $N_s = N_{c.s} - 1$

Length of flight of stairs: $L_v = N_m \times g$

Slop: $tg_{\alpha} = \frac{H/2}{L}$

Soffit's thickness: L / 30 ≤ e ≤ L / 20 ……..L = $(L_v{}^2 + H^2)^{1/2}$

	H(m)	$N_{c.m}$	N_m	L_v(m)	α°	e (cm)
ground floor	5,78	17	16	4,8	31°	20

Table 1.4.1: Staircase dimensions

- **Loads evaluation**

Components	e (cm)	Density Kn / m3	Weight Kn / m²
Soffit's weight	0,2	25	5
Step's weight	0.17	25	4.25
Laying mortar	0,02	20	0,40
Floor tile	0,02	22	0,44
Plaster	0,02	10	0,20
Baluster			
Dead load			10
live load			2,5

Table 1.4.2: load evaluation

- **Load combination**

	G	Q	ULS	SLS
Soffit	10	2.5	17,25	12,5

Table 1.4.3: load combination

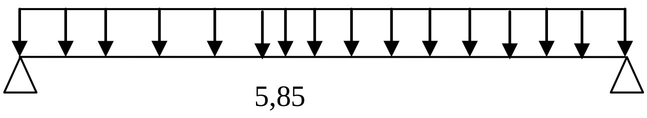


Figure 1.4.2: Staircase static load

- **ULS efforts**

$T_{max} = 50{,}45KN$ $M_{max} = 73{,}79KN.m$

We consider that we have a partial embedding therefore:

So: $\begin{cases} M_{su} = -0{,}4\ M_{max} = -29{,}516\ \mathrm{kN.m} \\ M_{spu} = 0{,}85\ M_{max} = 62{,}72\mathrm{Kn.m} \\ T_{max} = 50{,}45\ \mathrm{kN} \end{cases}$

- **SLS efforts**

$T_{max} = 36{,}56\mathrm{KN}$ $\qquad$ $M_{max} = 53{,}47\mathrm{KN.m}$

We consider that we have a partial embedding therefore:

So: $\begin{cases} M_{ss} = -0{,}4\ M_{max} = -21{,}38\ \mathrm{kN.m} \\ M_{sps} = 0{,}85\ M_{max} = 45{,}44\mathrm{Kn.m} \\ T_{max} = 35{,}56\ \mathrm{kN} \end{cases}$

	M (ULS) KN.m	M (SLS) KN.m
constraint	29,516	21,38
span	62,72	45,44

Table 1.4.4: Staircase efforts

- **Reinforcement of the staircase**

The reinforcement is done for width of 1m.

$\mu = \frac{Mu}{bd^2 f_{bc}} = \frac{62{,}72\times10^6}{1000\times162^2\times14{,}17}$ = 0,168 << $\mu_R = 0{,}391$

$\alpha = 1{,}25\left(1-\sqrt{1-2\mu}\right) = 1{,}25\ (1-\sqrt{1-2\times0{,}168})\ \ \alpha = 0{,}232$

$z = d(1-0{,}4\alpha) = 162(1-0{,}4\times0{,}232)\ \ z = 146{,}93mm$

$A_u = \frac{M_u}{z\times\ \sigma_s} = \frac{62{,}72\times10^6}{146{,}93\times347{,}82}$ =12,27cm²

$\mu < 0{,}03 \quad A_{min} = 0{,}23\left(\frac{f_{t28}}{f_e}\right)bd \quad = 0{,}23\ \times\left(\frac{2{,}1}{400}\right)\times1000\times16 \quad A_{min} = 1{,}95\ cm^2$

According to RPA: $A_{Smin} = 0{,}5\%b.h \quad A_{Smin} = 9cm^2$

So we choose 8HA14 ➔ A= 12,32cm²

We do the same steps to find the section of reinforcement with: M =29,516Kn.m

We find 5HA16 ➔ A= 10,05cm²

- ***Spacing***

	Spacing calculation	e (cm)
span	100 / 8 = 12,5	12
constraint	100 / 7 = 14,2	14

Table 1.4.5: Spacing values for staircase reinforcement

- **Distribution reinforcement**

Calculated by this expression $\frac{A_s}{4} \leq A_r \leq \frac{A_s}{2}$

	$\frac{A_s}{4}$	$\frac{A_s}{2}$	A_r cm²	Ø
span	3,08	6,16	4,62	3T14
constraint	2,51	5,02	4,62	3T14

Table 1.4.6: Distribution reinforcement results for staircase

- **Condition of non-fragility**

$$A_{min} = 0{,}23\left(\frac{f_{t28}}{fe}\right)bd = 0{,}23 \times \left(\frac{2{,}1}{400}\right) \times 1000 \times 162$$

Span: $A_{min} = 1{,}95$ cm² < A= 12,32cm² ... ... checked.

constraint : $A_{min} = 1{,}95$ cm² < A= 10,05cm² ... ... checked.

- **Shear force check**

it must be checked: $\tau_u = \frac{T_u}{bd} \leq \overline{\tau_u}$

$\overline{\tau_u}$ = min $\left(\frac{0{,}15.f_{c28}}{\gamma_b}, 4\right) = \min(2{,}5\ ;\ 4) = 2{,}5MPA$ $\quad T_u = 50{,}45$KN

$\tau_u = 0{,}311MPA \leq \overline{\tau_u} = 2{,}5MPA$...checked.

- **Bearing zone check**

If $T_u - \frac{M_u}{0{,}9d} < 0$ ... ...not necessary to verify the bottom steel.

If $T_u - \frac{M_u}{0{,}9d} > 0$............................verify the bottom steel:$A_s \geq \frac{\gamma_s}{fe}\left(Tu - \frac{M_u}{0{,}9d}\right)$

$T_u - \frac{M_u}{0{,}9d}$= 50, 45-$\frac{62{,}72}{0{,}9 \times 0{,}162}$ =-379,72KN < 0 so the reinforcements are not subjected to any tensile force.

- **Service limited state check**

The cracking is little harmful, so it is not necessary to check the tensile reinforcement.

We follow all steps according to BAEL and we summarized the results in this table:

	M_{ser}(KN.m)	A_s(cm²)	σ_{bc}	$\overline{\sigma_{bc}}$	$\sigma_{bc} \leq \overline{\sigma_{bc}}$
constraint	21,38	10,05	4,89	15	checked
span	45,44	12,32	9,69	15	checked

Table 1.4.7: SLS check

- **Deflection**

It is not necessary to calculate the deflection if two following conditions are satisfied:

$\frac{h}{L} \geq \frac{1}{16} \ldots\ldots\ldots\ldots\ldots \frac{18}{585} = 0{,}030 \leq \frac{1}{16} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots$ not checked.

$\frac{A_S}{b.d} \leq \frac{4{,}2}{f_e} \ldots\ldots\ldots\ldots \frac{1005}{1000.162} = 6{,}2.10^{-3} \leq 0{,}0105 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots$ checked.

$\frac{h}{L} \geq \frac{M_t}{10.M_0} \ldots\ldots\ldots\ldots 0{,}09 \geq 0{,}1 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots$ not checked.

So it is necessary to verify the deflection.

$$\Delta f_T = f_v - f_i \leq \bar{f} \begin{cases} f_i = \dfrac{M_{ser} L^2}{10 E_i I_{fi}} \\ f_v = \dfrac{M_{ser} L^2}{10 E_v I_{fv}} \\ \bar{f} = \dfrac{L}{1000} + 0{,}5 \end{cases} \qquad I_0 = \frac{bh^3}{12} + 15A_s\left(\frac{h}{2} - d\right)^2 + 15A'_s\left(\frac{h}{2} - d'\right)^2$$

$$I_{Fi} = 1{,}1.\frac{I_0}{1 + \lambda_i.\mu} \qquad I_{Fv} = 1{,}1.\frac{I_0}{1 + 0{,}4.\lambda_i.\mu}$$

$$\begin{cases} \lambda_i = \dfrac{0{,}05 f_{t28}}{\delta\left(2 + \dfrac{3b_0}{b}\right)} \\ \lambda_v = \dfrac{0{,}02 f_{t28}}{\delta\left(2 + \dfrac{3b_0}{b}\right)} \end{cases} ; \quad \begin{cases} \delta = \dfrac{A_s}{b_0 d} \\ \mu = 1 - \dfrac{1{,}75 f_{t28}}{4\delta\sigma_s + f_{t28}} \\ \sigma_s = \dfrac{M_{ser}}{A_s d} \end{cases}$$

We summarized all the obtained results in those following table

M_{ser} (KNm)	A_s (cm^2)	δ	σ_s (MPa)	λ_i	λ_v	μ	I_0 (cm^4)	I_{fi} (cm^4)	I_{fv} (cm^4)
45,44	12,32	0,0076	227,67	2,76	1	0,59	411430 ,49	171622,6	273489,6

Table 1.4.8: Deflection parameters for staircase.

f_i (mm)	f_v (mm)	Δf_T (mm)	$\bar{f}$ (mm)	check
3,10	9,32	6,2	11,7	checked

Table 1.4.9: Deflection check.

1.4.3 Acroterion

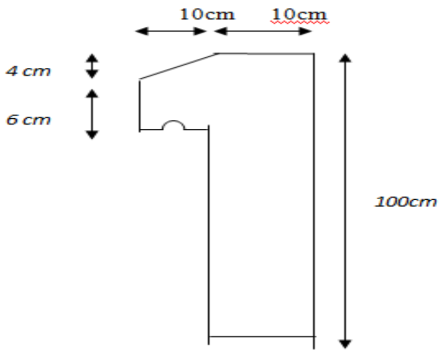


Figure 1.4.3 Acroterion's cross section

- **Load evaluation**

According to [RPA 99 version 2003] the acroterion is considered as a non-structural element which acts a horizontal force "FP" due to earthquake , its expression defined : $F_P=4\times A\times C_P\times W_P$

A = 0.25 (zone III, group 2) according to table 4-1 of the RPA99.

C_P = 0.3 according to RPA Table 6–1.

W_P = 2.70KN / ml

$F_P=4\times0.25\times0.8\times2,70$

F_P= 2,16 KN/ml so Q.h=2,16KN/ml.

The live load Q of the acroterion (due to a handrail) is less preponderant than the action of the force due to the earthquake.

G_{ser} = 2,70 KN/ml

Qh = 2,16 KN/ml.

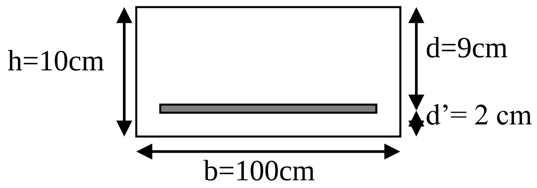
- **Efforts**

	N_u	M_u	T_u
ULS	3,6	3,24	2,16
SLS	2,23	2,16	2,16

Table 1.4.10: Acroterion's efforts

- **Reinforcement**

The Calculation is done on a rectangular section with: h=10cm b=100cm d=9cm



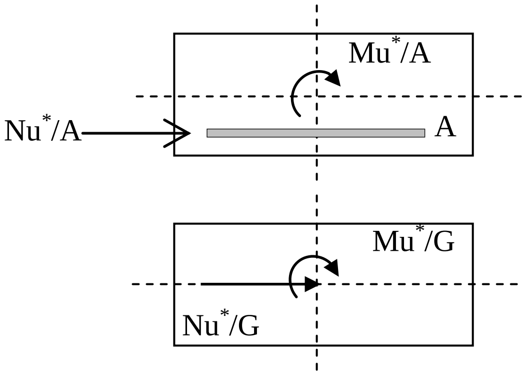


Figure 1.4.4 Acroterion's efforts

The reinforcements will be calculated at simple bending

M_f=M_u+N_u×($\frac{h}{2}$-c) = 3,34KN.M

α	Z (cm)	A_s (cm²)	A_{s1} (cm²)	A_{s2} (cm²)
0.045	8,834	1,08	0	0,97

Table 1.4.11: Calculated reinforcement section's parameters.

- **Condition of non-fragility**

Section (cm2)	A_s^{min} (cm2)	choice	$A^{chosen}{}_{min}$	St (cm)
80*50	1,09	4HA8	2,01	25

Table 1.4.12: Non fragility check for actorerion

- **Distribution reinforcements**

$A_r \geq \frac{As}{4}$ = 0,503 cm² so we choose 4HA8=2, 01 cm² with $St = \frac{h}{4}$=25cm

- **Eccentricity**

If e_0> $\frac{h}{2}$-c………………… it is a partially compressed section.

e_0 = M_u / N_u = 0, 9 m > $\frac{h}{2}$- c=0,03m.

SLS

$$\begin{cases} e_0 = \frac{M_{ser}}{N_{ser}} = \frac{2,16}{2,23} = 0,96m \\ \frac{h}{2} - c' = \frac{10}{2} - 2 = 0,03m \end{cases} \quad e_0 > \frac{h}{2} - c' \ldots\ldots$$ it is a partially compressed section.

Y1(cm)	I(cm⁴)	σbc(MPA)	σs(MPA)	$\overline{\sigma bc}$(MPA)	$\overline{\sigma s}$(MPA)	check
2,26	1,75	2,78	124,78	15	201,63	checked

Table 1.4.13: SLS check

Figure 1.4.5 Acroterion's design reinforcement

1.4.4 Conclusion

Through this chapter we have seen that the reinforcement for secondary elements isn't that much important like the principal ones, but we should verify their sections (shear force, deflection…) to avoid any risk.

1.5 EARTHQUAKE LOAD ON STRUCTURE

1.5.1 Introduction

The earthquake's phenomenon is considered as one of the most earth's destructive forces (seismic waves) that measured with a seismometer known as a seismograph which those waves create horizontal pressure on building ,causing them to a totally or partially collapse (depends to earthquake's magnitude). To withstand that type of collapse, the building with its elements need a valuable behavior (rigidity, resilience, ductility) to provide more strength and no failure risk during or after a seismic event).

Through this stage we'll evaluate the seismic and the dynamic response of our structure.

Ductility: aptitude of material to deform plastically before attending the risk of failure , other meaning it's the transition between the two pillars of any material strength; after reaching the ultimate stress here the material can deform only plastically , ductile material have high toughness that resists in large area before the sudden breaking as the case of brittle material.

Figure1.5.1 Damaged building (earthquake Boumerdes2003)

In order to face that type of natural disasters, here in Algeria our weapon is the Algerian code RPA99 V2003, this Algerian code has deferent stages where they handle all the steps and the procedures, details and remarks that you might face through your evaluation of the seismic response. According to the Algerian code there are three methods made for that kind of study:

-Equivalent static method.

-Seismic analysis by response spectrum

-Seismic analysis by time history function.

In our current project, we analyze our structure by the response spectrum method, because we have to solve all the deferential equations and evaluate all the nodes by the finites element method which this last requires us to analyze this building with Robot Analysis Structural software to evaluate the performance of each node.

Response spectrum

is a plot of the maximum response (displacement, velocity acceleration...) to a specified load function that subjected to a given ground motion.

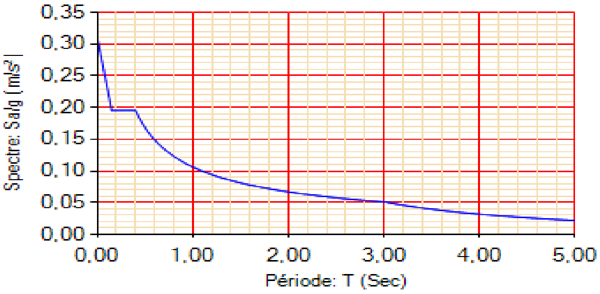


Figure 1.5.2: response spectrum.

1.5.2 Dynamic analysis

- **Initial model**

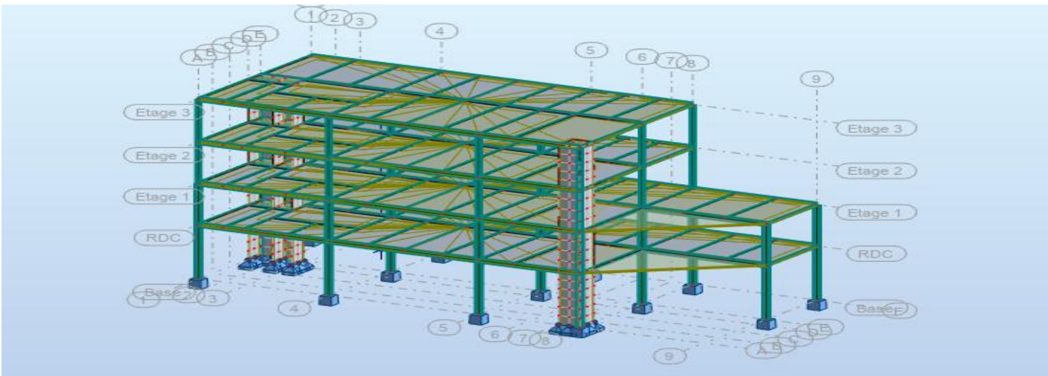


Figure 1.5.3: 3D model of the initial model.

Mode	period	The effective modal masses for the modes (%)					
		U_X	U_Y	U_Z	$\sum U_x$	$\sum U_y$	$\sum U_z$
1	1,12	0,01	69,73	0	0,0	68,77	0
2	0,71	0,33	2.50	0	0,45	71,77	0
3	0,6	74.67	0,07	0	71,46	72,04	0
4	0,26	0,01	0	0	71,47	72,04	0
5	0,22	0.01	20.14	0	73,51	72,04	0
6	0,16	0,01	0	0	73,52	72,04	0
7	0,14	0,05	0	0	73,56	72,04	0
8	0,10	0,02	0	0	73,58	72,04	0
9	0,09	0 ,61	0	0	74,19	72,04	0
10	0,09	0	17,23	0	74,19	89,27	0
11	0,08	0	2,48	0	74,2	91,76	0

Table 1.5.1: modal analysis results for initial model

Interpretation

after lunching the first dynamic analysis we find: fundamental period of T= 1,12 s.

The first mode is translation in y-y.

The second mode is torsion mode.

The third mode is translation in x-x.

1.5.3 Seismic analysis

- **Base shear force calculation**

The global base shear force V that applied at the base of structure, it determined by this expression: $V = \frac{A \times D \times Q}{R} \times W$

A: Zone acceleration coefficient.

In our case the building belongs to group 2: common structure or medium importance Seismic zone III (strong seismicity) Blida. So $\mathrm{A} = 0{,}25$

D: Average dynamic amplification factor, depending on the site category, the damping correction factor η and the fundamental period of the structure T.

$$D = \begin{bmatrix} 2{,}5\,\eta & 0 \leq \mathrm{T} \leq \mathrm{T}_2 \\ 2{,}5\,\eta\,(\mathrm{T}_2/\mathrm{T})^{\frac{2}{3}} & \mathrm{T}_2 \leq \mathrm{T} \leq 3\mathrm{s} \\ 2{,}5\eta(\mathrm{T}_2/3)^{\frac{2}{3}}(3/\mathrm{T})^{\frac{2}{3}} & \mathrm{T} \geq \mathrm{T}_2 \end{bmatrix}$$

T_2: Characteristic period associated to the category of the ground, in our project S2 so: $T_2 = 0.4s$

η: Damping correction factor ,given by : $\eta = \left(\frac{7}{2+\xi}\right)^{\frac{1}{2}} \geq 0.7$

ξ is the critical damping ratio (%) depending on constitutive material, structure

In our case $\xi = 5$ (tab.4.2) therefore: $\eta = 1{,}0$

Estimation of the empirical period

The value of the empirical period (T) of the structure is calculated by this expression:

$T = C_T\, h_N^{3/4}$ with :

$h_N = 19{,}21$ m (global height of the building).

C_T Depends to the bracing system used and the fill type

According to the table 4.6 from RPA2003, in our case:

a self supporting portal frame (reinforced concrete or steel) with a masonry infill.

So $C_T = 0.050$………… $\mathrm{T} = 0{,}050 \times 19{,}21^{3/4} = 0{,}45\mathrm{s}$.

In case of composite floor we can also calculate the fundamental period by this expression:

$T = 0{,}09 h_n / \sqrt{d}$

$d_x = 37{,}47$m……………………T = 0,28 s.

d_y= 14,56 m……………………T = 0,45 s.

According to the code, we must choose the smallest value of T in each direction.

X axis …….T= min (0,45s, T=0,28s)……… T=0,28 ≤ T_2.

Y axis …….T= min (0,45s , T=0,45s) ……..... T=0,45 ≥ T_2.

	T	$\mathbf{T_2}$	$\boldsymbol{\eta}$	**D**
$\boldsymbol{D_x}$	0,28	0,40	1.0	2,5
$\boldsymbol{D_y}$	0,45	0,40	1.0	2,3

Table 1.5.2: amplification factor.

R: Global behavior coefficient of the structure. In our case:

steel frame with a composite bracing system comprising a reinforced concrete newel and steel portal frame. R=4.

Q: Quality factor of the structure given by the following expression: Q = 1 + Σ P_a

Criteria	**observation (yes/no)**	**Penalty xx**	**observation (yes/no)**	**penalty yy**
1.Minimal conditions on bracing lines	Observed	0	Non observed	0,05
2. Redundancy in plan	Observed	0	Non observed	0,05
3. Regularity in plan	Observed	0	Non observed	0,05
4. Regularity in elevation	Observed	0	Observed	0
5. Control of material quality	Observed	0	Observed	0
6. Control of construction quality	Observed	0	Observed	0
Q	Qx =1		Qy = 1,15	

Table 1.5.3 : Qualité factor

W: Total weight of the structure. $W = \sum W_i$ With $W_i = W_{Gi} + \beta W_{Qi}$

in our initial mode W= 12709,28 KN.

Following those steps, the shear forces of the initial model done:

$V_x = \frac{0,25\times2,5\times1}{4} \times 12709,28$ =1985,825KN.

$V_y = \frac{0,25\times2,3\times1,15}{4} \times 12709,28$ =2101KN.

- **Base shear force check**

One of the first conditions recommended by the Algerian code RPA99 V2003 is about the seismic forces obtained by the response spectrum doesn't have to be less than 80% of the base shear force obtained by the equivalent static method.

	0,8 V (S.E.M)	V(R. S) (KN)	Checking
X	1588,66KN	2118,19	checked
Y	1680,8	4377,09	checked

Table 1.5.4: base shear reaction check

- **Inter-storey drift check**

Must be checked: $\Delta_k^x \leq \overline{\Delta}$ *and* $\Delta_k^y \leq \overline{\Delta}$

$\overline{\Delta} = 0.01h_e$ $\delta_k^x = R\delta_{ek}^x$; $\delta_k^y = R\delta_{ek}^y$, $\Delta_k^x = \delta_k^x - \delta_{k-1}^x$; $\Delta_k^x = \delta_k^y - \delta_{k-1}^y$

We summarized the deferent values of the storey drift in the following table:

Storey	Allowable storey drift (cm)	Inter storey drift x (cm)	Checking for x	Inter storey drift y (cm)	Checking for y
3rd	4,59	2	checked	5,7	unchecked
2nd	4,42	1,9	checked	5,3	unchecked
1st	4,42	1,6	checked	4	checked
GF	5,78	1,1	checked	2,5	checked

Table1.5.5: inter storey check

Interpretation

The inter-storey lateral displacements exceed the admissible values; therefore it is necessary to increase the lateral rigidity of the structure and this can be done:

-increasing the dimensions of the columns .

- adding bracing system in the structure.

We'll choose the two options: adding bracing system 2UPN100 that is all about its disposition and increasing the column's cross section to:

Previous column's cross section	New section supposed
HEA360	HEA360
HEA500	HEA550

Table1.5.6 column's new cross section.

- **Final model**

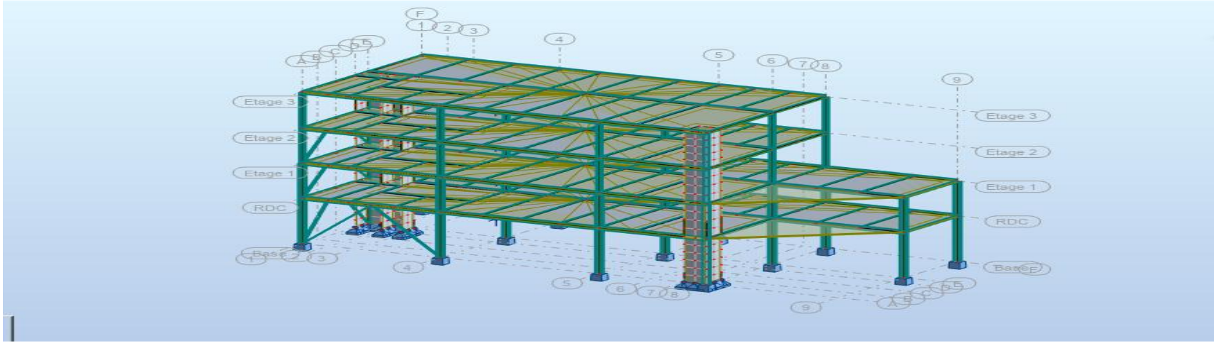
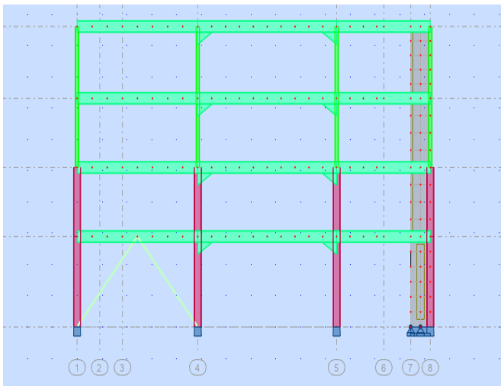


Figure 1.5.4: 3D model view (final model)



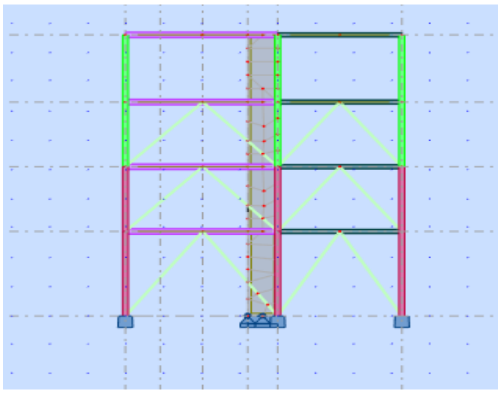


Figure 1.5.5: xz view

figure 1.5.6: yz view

Mode	period	The effective modal masses for the modes (%)					
		U_X	U_Y	U_Z	$\sum U_x$	$\sum U_y$	$\sum U_z$
1	1,03	59,25	1,88	0,000	59,25	1,88	0,000
2	0,97	3,24	51,64	0,000	62,49	53,52	0,000
3	0,68	0,29	2,41	0,000	62,78	55,93	0,000
-	-						
20	0,25	0,94	13,64	0,000	90,60	93,06	0,00

Table 1.5.7: modal analysis results for final model

After lunching the new analysis we found:

The effective mass participation exceeds 90% from the 20th mode.

The first mode is a translation mode x-x.

The second mode is a translation mode y-y.

The third mode is a torsion mode.

- **Base shear force check**

We follow the same steps mentioned in the initial model:

Vx (KN)	2842,12	$0.8Vx$	2273,69
Vy (KN)	3006,97	$0.8Vy$	2405,57

Table 1.5.8 base shear force.

$0.8Vx$	2273,69	**Fx**	3083,93	checked
$0.8Vy$	2405,57	**Fy**	4204,20	checked

Table 1.5.9: base shear check final model.

- **Inter-storey drift check**

Storey	Allowable storey drift (cm)	Inter storey drift x (cm)	checking	Inter storey drift y (cm)	checking
3rd	4,59	4,3	checked	4,5	checked
2nd	4,42	4,4	checked	3,2	checked
1st	4,42	3,8	checked	3,7	checked
GF	5,78	2,5	checked	2,5	checked

Table 1.5.10 inter storey drift check

1.5.4 Conclusion

In this chapter we discovered how to analyze and design a structure subjected to earthquake in order to avoid the risk of failure and collapse, it is very essentially to provide an adequate ductility to the structural components even if there are no chances for earthquakes.

1.6 FRAME-ELEMENTS VERIFICATION

1.6.1 Introduction

As well as we must verify any construction under any seismic effects, other verifications must be done to avoid the risk of failure: check the element's resistance and the instability state, those lead to a sudden failure includes material failure and structural instability that subjected to internal efforts**.**

The aim of this stage is to verify all the probable instability states and the resistance checks.

Figure 1.6.1 Column failures due to compression

1.6.2 Verification of columns

They are subjected to a compound bending: axial force (N) and bending moments M.

Here the stresses must be combined in the most unfavorable case:

-Case 1: maximum compressed effort Nsd + corresponding My.sd and Mz.sd moments.

-Case 2: maximum My.sd moment + corresponding Nsd and Mz.sd.

-Case 3: maximum Mz.sd moment + corresponding Nsd and My.sd.

$$\frac{N_{sd}}{\chi_{\min} \times A \times f_y / \gamma_{M1}} + \frac{K_y \times M_{y.sd}}{W_{pl.y} \times f_y / \gamma_{M1}} + \frac{K_z \times M_{z.sd}}{W_{pl.z} \times f_y / \gamma_{M1}} \leq 1$$

Condition 1: $N^{max}{}_{sd}$ with $M_{y.corr}$ and $M_{z.corr}$

- **According to Y axis**

$\eta_1 = \frac{k_c+k_{c1}}{k_c+k_{c1}+k_{b11}+k_{b12}}$ $\eta_2 = \frac{k_c+k_{c2}}{k_c+k_c+k_{b21}+k_{b22}}$

η … … … … … … … … … … ..Fixed base………………… $\eta = 0$

η … … … … … … … … … …hinged base………………… $\eta = 1$

$$\eta_1 = \frac{193598{,}62+253167{,}42}{193598{,}62+253167{,}42+49537{,}51+43074{,}17} = 0{,}78$$

$\eta_2 = 0$According to CCM97 we use the fixed nodes method determinate the buckling length $\frac{L_b}{L_0} = \frac{1+0.145\times(\eta_1+\eta_2)-0.265\times\eta_1\times\eta_2}{2-0.364\times(\eta_1+\eta_2)-0.247\times\eta_1\times\eta_2}$, $L_0 = 5{,}78m$, L_{by} =5,34m

$\lambda_y = \frac{l_{by}}{i_y}$ =23,26……….……$\lambda_y = \frac{\lambda_y}{93{,}9\varepsilon} = 0{,}26 > 0.2$ … … … … . . … ..risk of buckling.

$$\phi_y = 0.5 \times \left(1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y{}^2\right)$$

ϕ_y=0, 54

$$\chi_y = \frac{1}{\phi_y + \sqrt{\phi_y{}^2 - \bar{\lambda}_y{}^2}} = \frac{1}{0.52+\sqrt{0.52^2-0.32^2}} = 0{,}98$$

$K_y = 1 - \frac{\mu_y \times N_{sd}}{\chi_y \times A \times f_y} \leq 1{,}5$ $\mu_y = \bar{\lambda}_y(2.\beta_{M.y} - 4) \leq 0{,}90$

$\beta_{M.y} = 1.8 - 1.7\,\Psi \Rightarrow \Psi = \frac{M_{min}}{M_{max}} = \frac{-96{,}73}{206{,}68} = -0{,}46$ $\beta_{M.y} = 2{,}12$

K_y =0, 99 $\leq$ 1,5

- **According to Z axis**

We follow same steps mentioned in y axis; we summarized all the results obtained in this table:

η_1	η_2	L_{bz}(m)	λ_z	ϕ_z	χ_z	χ_{min}
0,91	0	5,55	0,89	1,01	0,66	0,66

Table 1.6.1 Buckling parameters

M_{min}KN.m	M_{max}KN.m	Ψ	$\beta_{M.z}$	μ_z	K_z
-0,28	-0,5	0,56	1,40	-1,05	1

Table 1.6.2 Buckling parameters2

Verification

$$\frac{3777\times10^3}{0.66\times\frac{21180\times275}{1.1}} + \frac{0.99\times(97{,}49\times10^6)}{\frac{4822000\times275}{1.1}} + \frac{1\times(0{,}28\times10^6)}{\frac{1107000\times275}{1.1}} = 1 \leq 1$$ ……………..well checked.

2) $M_{sd\,max} = 234{,}35\text{KN.m}$ $N_{sd\,corr} = 2564{,}89\text{KN}$ $M_{z\,corr} = 0{,}14\text{KN}$

$$\frac{2564{,}89\times10^3}{0.66\times\frac{21180\times275}{1.1}} + \frac{0.99\times(234{,}35\times10^6)}{\frac{4822000\times275}{1.1}} + \frac{1\times(0{,}14\times10^6)}{\frac{1107000\times275}{1.1}} = 0{,}72 \leq 1$$ …………..well checked.

3) $M_{sd\,max} = 28{,}67\text{KN.m}$ $N_{sd\,corr} = 1696\text{KN}$ $M_{y\,corr} = 55{,}57\text{KN}$

$$\frac{1696\times10^3}{0.66\times\frac{21180\times275}{1.1}} + \frac{0.99\times(28{,}67\times10^6)}{\frac{4822000\times275}{1.1}} + \frac{1\times(55{,}57\times10^6)}{\frac{1107000\times275}{1.1}} = 0{,}48 \leq 1$$ …………..well checked.

We continued the same steps to verify the other columns, and all of them resist and verify in buckling without increasing in their section.

1.6.3 Verification of beams

Figure 1.6.2 Principal beam 11,19m

- **Effort**

$M_{sd} = 1195{,}08 KN.m$ $V_{sd} = 548{,}75 KN$

- **Resistance check**

M_{plrd}=896,27KN.m……….. (Calculated in chapter 2)

$M_{sd} = 1195{,}08 KN.m \geq M_{plrd}$ =896,27 KN.m ………………………..……not verified.

- **Shear force check**

$V_{sd} = 548{,}75 KN \leq V_{plrd}$ = 693KN…………………………………well checked.

- **Bending moment-shear force interaction**

V_{sd} =*548,75KN* $\geq 0.5 V_{plrd}$ =*346,5 KN* **……….……….**there is interaction between M, V

There is an interaction between bending moment and the shear force as this beam's section doesn't resist under bending here we should increase the beam's section to **IPE600.**

We lunch again the analysis with this section, all the verifications mention in this table:

Profile	**M_{sd} (KN.m)**	**M_{plrd}(KN.m)**	**checking**
IPE600	1183 ,58	1361,29	checked
	V_{sd} (KN)	**V_{plrd}(KN)**	**checking**
IPE600	462	968,5	checked
	V_{sd}(KN)	**$0.5V_{plrd}$(KN)**	**Checking**
IPE600	462	484,25	No interaction

Table 1.6.3 Beams check

1.6.4Verification of bracing system

in our project we used bracing type V (two diagonals members forming a V shape meet at center point on the superior horizontal member, it can significantly reduce the buckling capacity of the compression brace .

- **RPA's condition**

All the bars of the triangulated braced piers must be calculated to withstand 1,25 times of the force determined by $V = \frac{ADQ}{R}W$..............(8.4.3.1page69).

So we create new combination: $1{,}25E_x + G + Q$

- **Resistance check in tension**

It must be checked N ≤ N_{trd} $N_{trd} = \frac{A \times f_y}{\gamma_{M0}}$ $N_{trd} = \frac{1350 \times 275}{1} = 371{,}25\ kN$

$N_{sd(2UPN100)} = 415{,}06kN$

$N_{sd(UPN100)} = 207{,}53KN < Nbrd = 371{,}25KN$ … … … … … well checked.

- **Resistance check in compression**

It must be checked N ≤ $Nbrd = \chi \times \beta \times A \times \frac{fy}{\gamma m1}$

$N_{sd(2UPN100)} = 11{,}51kN$ $N_{sd(UPN100)} = 5{,}75KN$

$Nbrd = 0{,}14 \times 1 \times 1350 \times \frac{275}{1} = 51{,}97KN > N_{sd(UPN100)} = 5{,}75KN$ … … checked.

1.6.5 Conclusion

Through this stage we discovered the importance of following all the codes you have and all the verifications needed into assure the security of each element, we take an example of bracing system where the RPA99 requires the type of bolts (pre-loaded bolts) and multiply the dynamic force 1,25 times (E_x, E_y) to avoid the risk of buckling and the element will be subjected mostly to tensile.

1.7 DESIGN OF JOINTS

1.7.1 Introduction

As we have the reinforcement in concrete buildings, the steel structure consists an operation called assembly of steel components into frame on site; generally this can be achieved through bolting and sometimes welding between elements to ensure the transmission and the distribution of various static and sometimes dynamic stress .

In our present study, the bolted connection is the most used method because this last is generally has the advantage of the demount ability in case of errors with good recovery of the initial components.

Figure1.7.1: Bolted connection

1.7.2 Connection column-beam

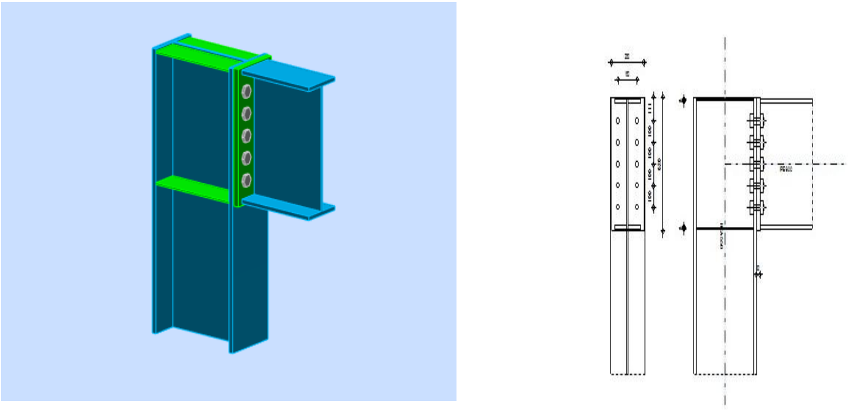


Figure 1.7.2 Beam-Column connections

- **Beam's efforts**

$Msd = 675{,}45 KN.m \qquad Vsd = 369{,}39 KN$

- **Weld**

L1 = 220mm

L2=104mm

L3=562mm

As $= \sum li \times ai = (2l1 \times a) + (4\,l2 \times a) + (2l3 \times a) = 23760mm^2$

The grad of steel used is S275 so: $\gamma mw = 1{,}3$; $\beta w = 0{,}85$; $fu = 430MPA$

The Weld's thickness on flange:

$$a_f \geq t_{fb} \times \frac{f_y}{\gamma_{mo}} \times \frac{\beta_w \times \gamma_{m2}}{f_{us} \times \sqrt{2}} \qquad a_f \geq 19 \times \frac{275}{1} \times \frac{0.85 \times 1.3}{430 \times \sqrt{2}} = 9{,}49\text{mm}$$

The Weld's thickness on web:

$$a_w \geq t_{wb} \times \frac{f_y}{\gamma_{mo}} \times \frac{\beta_w \times \gamma_{m2}}{f_{us} \times \sqrt{2}} \qquad a_w \geq 12 \times \frac{275}{1} \times \frac{0.85 \times 1.3}{430 \times \sqrt{2}} = 6\text{mm}$$

So we choose $as = 12mm$

- **Weld check**
  - ***Bending moment with the axial load check (M and N)***

$$\sqrt{2}\left[\frac{\text{Nsd}}{\sum \text{li} \times \text{ai}} + \left(\frac{\text{Msd}}{\text{Iys}} \times \frac{\text{h}}{2}\right)\right] \leq \frac{\text{fus}}{\beta\text{w} \times \gamma\text{m2}}$$

$Iys = (2l1 \times a \times d1^2) + (4l2 \times a \times d2^2) = 871918080\ mm^4$

$d1 = \frac{h}{2} + \frac{a}{2} = 306\text{mm} \qquad d2 = \frac{h}{2} - tf - \frac{a}{2} = 275mm$

applying the last condition we find: $328{,}66 \leq 389.14$ … … … … … … … … well checked.

  - ***Axial load with the shear force check (N and V)***

$$\sqrt{\mathbf{2} \times \left(\frac{Nsd}{\sum li \times ai}\right)^{\mathbf{2}} + \mathbf{3} \times \left(\frac{Vsd}{2\ l3 \times a}\right)^{\mathbf{2}}} \leq \frac{fus}{\beta w \times \gamma m2}$$

Applying the last condition we find: $47{,}43 \leq 389.14$ … … … … … … … … … well checked.

- **Pre-loaded bolts check**
  - ***Choice of bolt's diameter***

plate thickness $t = 30\ mm$

$\text{t} \leq 10\ \text{mm} \qquad \text{d} = (12\,;\ 14)\ \text{mm}$

$10 \leq \text{t} \leq 25\ \text{mm} \qquad \text{d} = (16\,;\ 20\,;\ 24)\ \text{mm}$

$\text{t} \geq 25\ \text{mm} \qquad \text{d} = (24\,;\ 27\,;\ 30)\ \text{mm3}$

we have t = 30 mm so due to those conditions we take: $\text{d} = 30\ \text{mm} \ldots\ldots \text{d}_0 = 33\text{mm}$

- ***Constructive layout***

t = 30 mm; so for the bolt‘s disposition we choose two rows within 5 HA bolts with diameter Ø = 30 mm, class 10.9 in each row.

- ***Distance between bolt’s centre lines***

$1.5d0 \leq e1 \leq \max(12t\,;150mm)$	$45 \leq e1 = 101 \leq 360mm$
$2.2d0 \leq p1 \leq \min(14t\,;200mm)$	$66 \leq p1 = 90 \leq 200mm$
$1.5d0 \leq e2 \leq \max(12t\,;150mm)$	$45 \leq e2 = 101 \leq 360mm$
$3d0 \leq p2 \leq \min(14t\,;200mm)$	$90 \leq p2 = 160 \leq 200mm$

f_{ub} : Ultimate tensile strength: 1000MPA for preloaded bolts class 10.9.

A_s : Bolt’s section area mm².

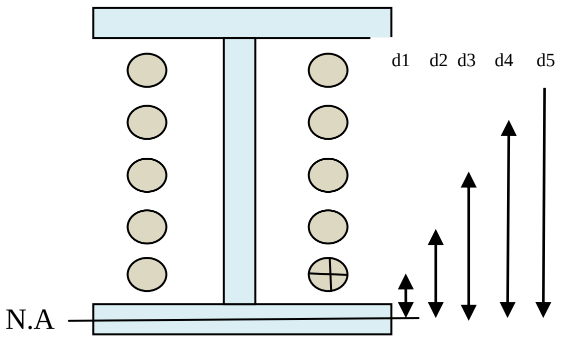


Figure 1.7.3 constructive layout

- ***Shear force resistance check***

$Fv,sd \leq Fs,rd$

$Fv,sd = \frac{Vsd}{nb} = \frac{369,39}{10} = 39,93kN \qquad Fsrd = \frac{ks.np.\mu.(Fp)}{\gamma M2}$

Ks =1 For holes with nominal tolerances.

μ = 0,3

n = 2 Number of friction interfaces.

γ_{ms} = 1,25

$Fp = 0.7 \times As \times fub = 392,7KN$

$FS.rd = \frac{1\times0.3\times2\times(392,7)}{1,25} = 188,49KN$

$Fv,sd = 39,93KN \leq Fs,rd = 188,49KN$ ……………………………………..$Well\ verified.$

- ***Tension resistance check***

$Ft,sd \leq Ft,rd$

$$Ft,rd = \frac{0,9 \times A_S \times f_{ub}}{\gamma_{mb}} = \frac{0,9 \times 561 \times 1000}{1,5} = 336,60KN$$

$$Ft,sd = F_{M1} = \frac{M_{sd} \times d5}{n \sum \mathrm{d}_\mathrm{i}^2}$$

d1 mm	d2 mm	d3 mm	d4 mm	d5 mm	$\sum \mathrm{d}_\mathrm{i}^2$ m²
110,5	200,5	290,5	380,5	470,5	0,50

Table 1.7.1 Distances between bolt's center lines

$$Ft,sd = \frac{675,45 \times 0,47}{2 \times 0,50} = 315,93KN \leq Ft,rd = 336,60KN \ldots\ldots..Checked$$

- ***Shear and tension check***

$$Fv,sd \leq Fsrd = \frac{ks.nf.\mu.(Fpc - 0.8.Ft.sd)}{\gamma Ms}$$

$$Fsrd = \frac{1 \times 0.3 \times 2 \times (392,7 - 0,8 \times 315,93)}{1,25} = 67,17KN \geq Fv,sd = 39,93KN$$

- ***Column's resistance in tension check***

It must be checked: $Ft \leq Ft.rd$

$$Ft.rd = fy \times twc \times \frac{beff}{\gamma m0}$$

twc : column's web thickness =13,5mm beff : bolts center line spacing =339mm

$$\mathrm{Ft.rd} = 275 \times 12,5 \times \frac{339}{1,1} = 1059,37\mathrm{kN}$$

$$\mathrm{Ft} = \frac{\mathrm{M}}{\mathrm{h - tf}}$$

$$Ft = \frac{675,45}{0,54 - 0,024} = 1309,01KN$$

$Ft \leq Ft.rd$ ……………………….Not verified (strain in column's web).

The condition isn't satisfied, we should choose a stiffener its thickness equal to the column's flange thickness e_p = 24 mm.

- ***Column's web resistance in compression check (not stiffened)***

It must be checked $F_C \leq F_{c.rd}$

$$F_{c.rd} = f_y \times t_{wc} \times (1,25 - 0,5\gamma_{m0}\frac{\sigma_n}{f_y})\frac{b_{eff}}{\gamma_{m0}}$$

σ_n: Axial compressive stress in column's web. $\sigma_n = \frac{Vsd}{A} + \frac{Msd}{wely}$

$$\sigma_n = \frac{369,39 \times 10^3}{21180} + \frac{675,45}{4146} \times 10^3 = 180,35\,\mathrm{N/mm^2}$$

$b_{eff} = t_{fb} + 2t_p + 5(t_{fc} + r_c)$ =339mm

$F_{c,rd} = 275 \times 12,5 \times (1,25 - 0,5 \times 1,1 \times \frac{180,35}{275}) \times \frac{339}{1,1}$ =942,10KN

$F_c \le F_{c\,rd}$ … … … … … … … … …. not verified .

We add a stiffener in order to increase the resistance strength.

- ***Column's web resistance in shear***

it must be checked $Fv \le Vr$

$Vr = \frac{0{,}58 \times fy \times hc \times twc}{\gamma_{M0}} = \frac{0{,}58 \times 275 \times 540 \times 12{,}5}{1{,}1} \times 10^{-3} = 978{,}75kN$

$Fv \le VR$ not verified.

We add a stiffener in order to increase the column's resistance strength.

1.7.3 Bracing system design

for the bracing system we have 2UPN100 which this last makes the bolts doubly sheared, connected by gusset.

- **RPA'S condition**

The design of joints in the bracing system must be calculated to allow the development of the maximal forces in the bars where it must be calculated on the basis with 1,5 times of the force determined by $V = \frac{ADQ}{R} W$……………(8.4.3.2page69).

So we create new combinations: $1{,}5E_x + G + Q$ $1{,}5E_y + G + Q$

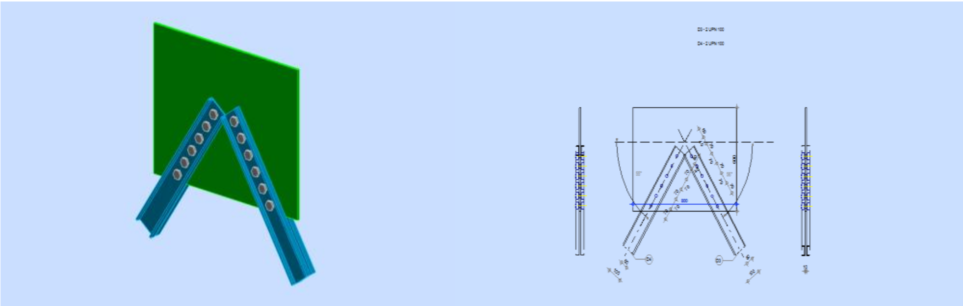


Figure1.7.4: Bracing system connection

- **Effort**

The effort taken up by 2UPN100 is:

$NSd\ 1 = 291{,}55KN$ $NSd\ 2 = 343{,}24KN$

- **Shear resistance per shear plane**

bolts Numbers and diameters

We suppose d0 = 22 mm so this corresponds bolts with diameter d = 20 mm and class 6.8

$Fv.rd = \frac{0.6 \times fub \times As}{\gamma mb}$ $Fub = 600N/mm^2$

As=245mm² tensile stress area

γmb=1.25 bolt's resistance coefficient

m=2 number of shear planes.

$$\mathrm{Fv.rd} = \frac{0.6\times600\times245}{1.25} = 70{,}56\mathrm{kN}$$

$$\mathrm{Fv,sd} = \frac{\mathrm{Vsd}}{\mathrm{n}} \ldots\ldots\ldots\ldots\ldots \quad \mathrm{n} = \frac{\mathrm{Vsd}}{\mathrm{Fv.sd}} = \frac{343{,}24}{70{,}56} = 4{,}84 \ldots\ldots\ldots\ldots\ldots\ldots\ldots \mathrm{n} = 6.$$

- **Constructive layout**

$1.5d0 \le e1 \le max\ (12t\ ; 150mm)$ e1=40mm

$2.2d0 \le p1 \le min\ (14t\ ; 200mm)$ $\mathrm{p1} = 70\mathrm{mm}$

$1.5d0 \le e2 \le max\ (12t\ ; 150mm)$ $e2 = 50mm$

$3d0 \le p2 \le \min(14t\ ; 200mm)$ $p2 = 70mm$

It must be checked: $Fv.sd < \min(\ Fb.rd\ , Fv.rd)$

$Fb.rd = \frac{2.5\times\alpha\times fu\times d\times t}{\gamma mb}$ $\boldsymbol{d} = 20mm\ ; d_0 = 22mm\ ;\ t = 10mm\ ;\ \gamma mb = 1.25$

α is the smallest of : $\min\left\{\frac{\mathrm{e1}}{3\,d_0}; \frac{\mathrm{p1}}{3\,d_0} - \frac{1}{4}; \frac{\mathrm{fub}}{\mathrm{fu}} \mathrm{ou}\ 1\right\} = \{0.60; 0{,}96; 1.86; 1\} = 0.60$

$$\mathrm{Fb.rd} = \frac{2.5\times0.6\times430\times20\times10}{1.25} = 103{,}20\ \mathrm{KN}$$

$Fv,sd = \frac{343{,}24}{6} = 57{,}20KN < Fv.rd = 70{,}56KN$ **……..**checked.

Gusset dimensions

It was verified in Robot, we obtained:

AB=800mm AC=660mm with a =10mm

1.7.4 Beam-Joist connection

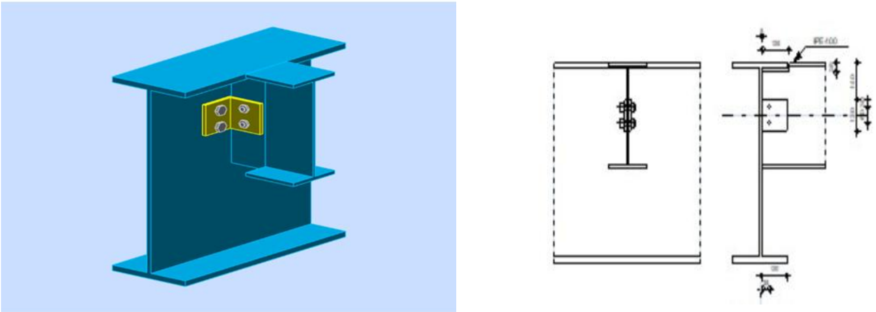


Figure 1.7.5: Beam-Joist connection

- **Bolts numbers and diameters**

We suppose d0 = 22 mm so this corresponds to bolts with diameter d = 20 mm and class 4.6

$Fv.rd = \frac{0.6 \times fub \times As}{\gamma mb}$ $\quad Fub = 400N/mm^2$

As=245mm² tensile stress area.

γmb=1.25 bolt's resistance coefficient

m=2 number of shear planes.

$F_{V,rd} = 0{,}6 \times 400 \times \frac{245}{1{,}25} = 47{,}04kN$

$\mathrm{Fv,sd} = \frac{\mathrm{Vsd}}{\mathrm{n}} \ldots\ldots\ldots\ldots.. \quad \mathrm{n} = \frac{\mathrm{Vsd}}{\mathrm{Fv.sd}} = \frac{160{,}22}{47{,}04} = 3{,}4 \ldots\ldots\ldots\ldots\ldots\ldots..\mathrm{n} = 4.$

- **Constructive layout**

$1.5d0 \leq e1 \leq max\,(12t\,;150mm)$ $\qquad 26{,}4 \leq e1 \leq 150mm$

$2.2d0 \leq p1 \leq min\,(14t\,;200mm)$ $\qquad 48{,}4 \leq p1 \leq 168mm$

$1.5d0 \leq e2 \leq max\,(12t\,;150mm)$ $\qquad 33 \leq e2 \leq 150mm$

$3d0 \leq p2 \leq \min(14t\,;200mm)$ $\qquad 66 \leq p2 \leq 168mm$

- **Verification of the bearing resistance**

We have an angle iron with those dimensions: $120 \times 120 \times 12$ mm

it must be verified : $\mathrm{F1} > F\mathrm{b}$

$F_B = 2{,}5.\alpha.f_u.d.\frac{t}{\gamma_{Mb}}$

$\alpha = \min\left(\frac{e_1}{3d_0}; \frac{P_1}{3d_0} - \frac{1}{4}; \frac{f_{ub}}{f_u}; 1\right)$ $\quad d = 20mm \quad d_0 = 22mm \quad t = 12mm$

$\gamma_{Mb} = 1{,}25 \quad f_u = \frac{360N}{mm^2} \quad e_1 = 35mm\ ; \quad P_1 = 50mm$

$\alpha = min(0{,}53; 0.0{,}50; 1{,}11; 1) \quad \alpha = 0{,}50$

$F_B = 2{,}5 \times 0{,}50 \times 360 \times 20 \times \frac{12}{1{,}25} \quad F_B = 86{,}4kN$

For one bolt: $Fv,sd = \frac{Vsd}{4} = \frac{160{,}22}{4} = 40{,}05KN < 86{,}4kN$ ……...............…..verified

1.7.5 Column-Column connection

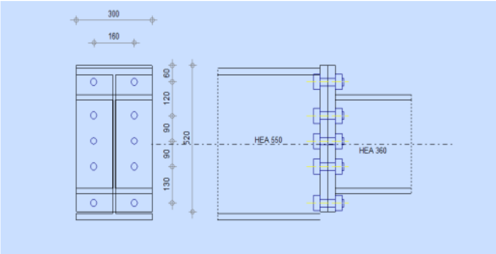


Figure 1.7.6 Column-Column connections

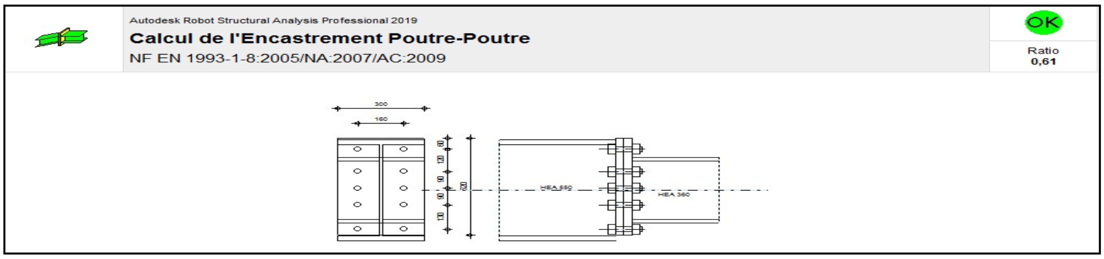


Figure 1.7.7 Column-Column connections 2

- **Stress**

$N_{sd\ max} = 436{,}35 KN$ $M_{ysd} = 128{,}28 KN.m$ V =62,06KN

- **Plate dimensions**

Length (mm)	Width (mm)	Thickness (mm)	Grade of steel (Mpa)
520	300	30	275

Table 1.7.2 Plate dimensions

- **Bolts**

bolt's diameter mm	Bolt's class	bolt's number / row's number	e1 mm	P mm
27	8.8	5 / 2	160	120.90.90.130

Table 1.7.3 Bolts parameters

1.7.6 Column base plate HEA500

Column base plates are used to connect a column with the foundation (concrete)where the loads are coming from top to the bottom , as steel known by strength ,the concrete section is going to bear the load coming from column so if it has less bearing strength and a high load the more we need larger concrete area .our column base plate is provided between the two materials(steel column and slab plate) this transmission made by an important elements which is a metallic plate its place at the base it's putted by anchors into the ground this help to distribute concentrated load of column to the slab base bellow. In our case we have a restrained design joint at the base.

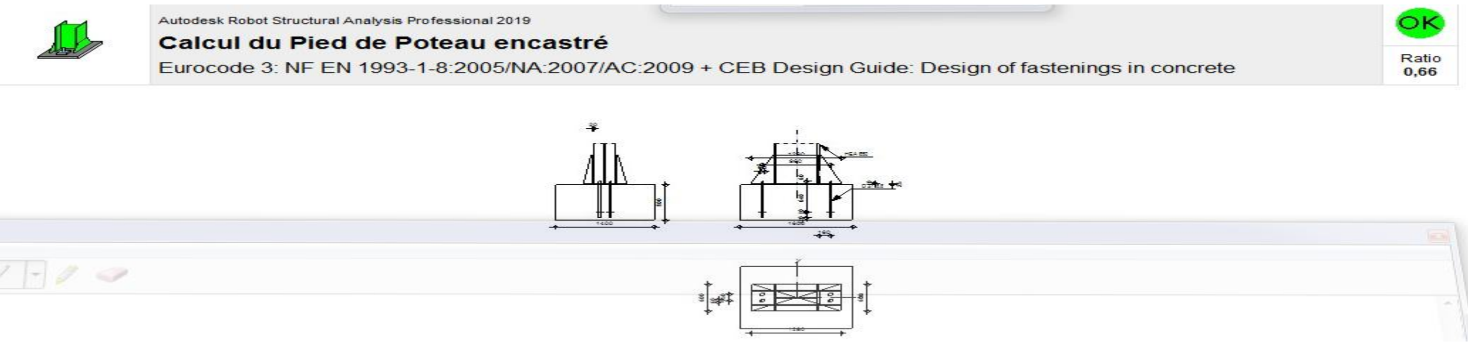


Figure 1.7.8 : Base plate for column

- **Plate dimensions**

Length (mm)	Width (mm)	Thickness (mm)	Grade of steel (Mpa)
1260	600	25	275

Table 1.7.4: Plate dimensions for base plate

- **Anchors**

class	Diameter(mm)	Effective area	Bolt's n / row	Vertical spacing
8,8	27	4,59	2/2	150

Table 1.7.5: Anchor dimensions

Anchor dimensions $l_1 = 60mm$ $l_2 = 640mm$ $l_3 = 120mm$

- **Anchor plate**

Length (mm)	Width (mm)	Thickness (mm)	Grade of steel (Mpa)
60	60	10	275

Table 1.7.6 Anchor plate dimensions

- ***Stiffeners***

Length (mm)	Width (mm)	Thickness (mm)	Height (mm)
1280	600	20	640

Table 1.7.7: Stiffeners dimensions

- **Footing (concrete)**

Length (mm)	Width (mm)	Height (mm)
1400	1400	800

Table 1.7.8: Footing's dimensions

- **Weld**

$a_p = 10\text{mm}$……………………..…base plate.

$a_s = 7\text{mm}$ … … … … … … … … … . . … …. Stiffeners**.**

1.7.7 Conclusion

In this chapter we noticed that in steel buildings, connections including the details are to be designed for expected forces into assure the right transmission of efforts between elements.

2.1 PRESENTATION

2.1.1Introduction

This part consists to study a steel hangar that located at this touristic camp next to two blocs. It composes of several frames and a duopitch roof stabilized by bracing system.

2.1.2 Geometry

In our case the dimensions of this hangar are:

- Global length……………………..39,43m.
- Global width ……………………..19,68m.
- Eave height ………………………10,2m.
- Slope percent ………..……………15%.

Wall 1: long side

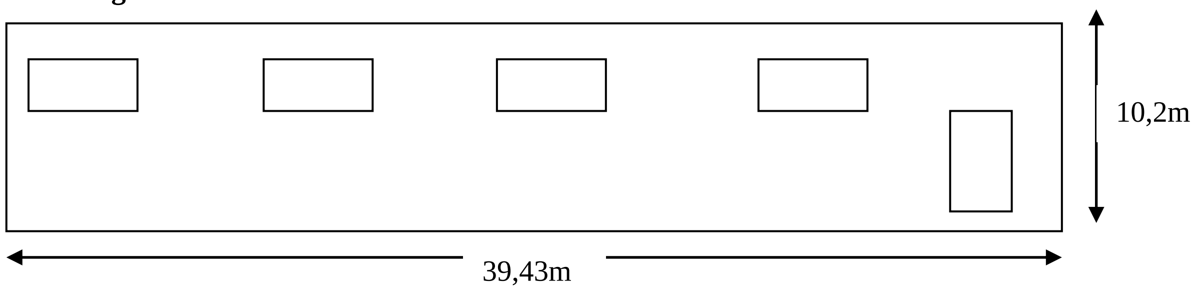


Figure 2.1.1: Elevation view of the long side(x-z).

Wall 2: Pinion

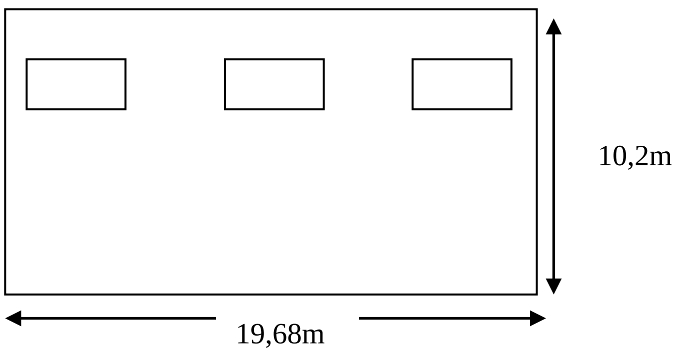


Figure 2.1.2: Elevation view of the pinion (y-z).

2.2 CLIMATIC LOAD STUDIES

2.2.1 Wind load studies

- **Introduction**

In case of building or hangar with (two walls exposed to wind interior and exterior) the wind load pressure depends to the peak velocity pressure, external and the internal coefficients.

- Altitude: 15%=$\frac{x}{\left(\frac{19,68}{2}\right)}$ $=== x = 1,47m$ ….. Ht=10,2+1,47=11,67m

- **Wind coefficients**
- ***Wind zone***

This camp is located in Blida's province, so according to RNV code it's zone I.

Basic wind velocity Vref = 25 m/s Basic wind pressure pref = 375 N/m² .

- ***Topography coefficient*** *Ct* (z) = 1.
- ***Terrain category***

K_t	Z_{min}	Ɛ	z_0
0.170	1	0.44	0.01

Table 2.2.1: Coefficients due to the ground type.

- ***Reference height***

According to RNV , in the two directions we have :

direction 1 : b=39,43m > h=10,2m…………$h_{eq} = 10,2m$

direction 2 : b= 19,68m > h=10,2m…………$h_{eq} = 10,2m$

Following all the steps to calculate the rest of coefficients, all the results are summarized in this table:

Z_e (m)	cr	Iv	ce	P ref $[N/m^2]$	qp $[N/m^2]$
10,2	1,17	0,14	2,71	375	1016,25

Table 2.2.2: Wind's coefficients.

- **Wind directions**

Vertical face W1 (b=39,43m; d=19,68m)

- ***Dynamic factor***

In our case we consider **Cd = 1** because: the global height is less than 15m.

- ***Pressure coefficients***

1. External pressure coefficient

a) Vertical wall:

For the calculation of Cpe values, the walls should be divided as shown in the figure of the Algerian code (snow and wind load calculation) *page80*.

b= 39,43mm ; d=19,68m ; 2h = 20,4m ; $e = Min(b; 2h)$; e = 2h = 20,4m

The loaded surfaces of the walls are all considered bigger than 10m² .So: Cpe = Cpe10.

	A’	B’	D	E
Area (A)	41,616	159,12	402,186	402,186
Cpe	-1	-0,8	0,8	-0,3

Table 2.2.3: External pressure coefficients of wall W1

b) Roof

In our case, we have a duopitch roof with two sides. The roof, including its protruding parts, should be divided in zones.

The recommended zones are given in Figure 5.4 page 86.

The reference height ze should be taken as h.

For a perpendicular wind’s direction to the generators, it must be defined by θ = 0 ̊ and the slope 15% → α = 8.49 °. b=39,43m d=19,68m, e=20,4m.

The external pressure coefficients Cpe.10 1 for zone F, G, H, I, J and E are summarized in the following table:

	F	G	H	I	J
Area (A)	10,404	59,62	307,56	307,56	80,43
Cpe	-0,9	-0,8	-0,3	-0,4	-1

Table 2.2.4: External pressure coefficients of roof W1.

2.Internal pressure coefficient

The wind pressure acting on the internal surfaces of a structure should be obtained following those steps:

The internal pressure coefficient Cpi , depends on the size and the distribution of the openings in the building envelope, when in at least two sides of the buildings(facade or roof), the total area of openings in each side is more than 30% of the area of that side , the action on the structure should not be calculated from the rules given in this section in this case.

$A_o = (0{,}6 \times 2{,}5) \times 4 + (1{,}2 * 3) = 9{,}6m^2$ $\mathrm{A_o}$: Area of the openings in this building.

$A_{f1} = (39{,}43 \times 10{,}2) = 402{,}186m^2$ $\mathrm{A_{f1}}$: Area of a facade .

$\frac{9{,}6}{402{,}186} \times 100 = 2{,}38\% < 30\% \ldots \ldots$ verified (no esolated roof).

For buildings without a dominant face, the internal pressure coefficient Cpi should be determined from this Figure, and is a function of the ratio of the height and the depth of the building, h / d, and the opening ratio jJ for each wind direction, which should be determined.

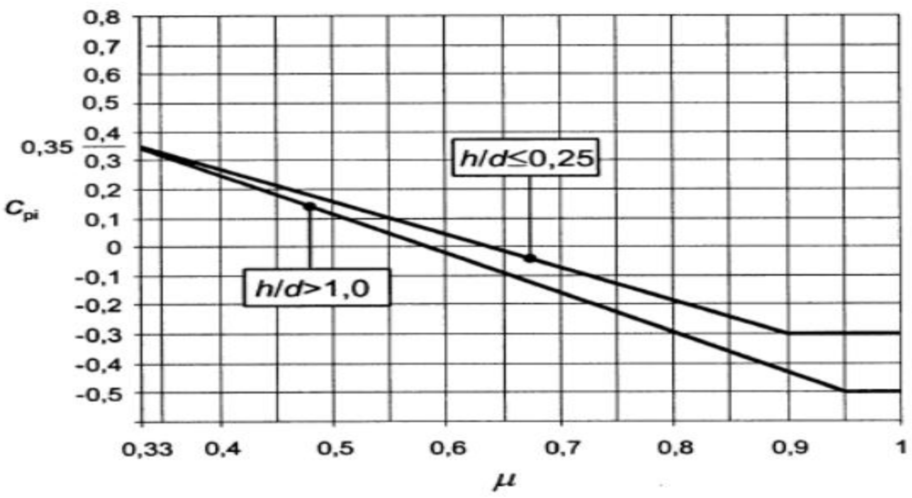


Figure 2.2.1: Permeability factor

For values between h/d = 0, 25 and h/d = 1, a linear interpolation may be used.

$$\mu p = \frac{\sum erea\ of\ opening\ where\ cpe\ is\ negative}{\sum erea\ of\ all\ opening}$$

$$\mu p = \frac{(0{,}6\times2{,}5\times4)+(1{,}2\times3\times1)}{(0{,}6\times2{,}5\times8)+(1{,}2\times3\times2)} = \frac{9{,}6}{19{,}2} = 0{,}5 \qquad \frac{h}{d}=0,\ 51$$

Applying a linear interpolation $y = y_1 + \left[\left(\frac{x-x_1}{x_2-x_1}\right) \times (y_2 - y_1)\right]$ *we find* Cpi=0,11

- ***Wind pressure on surfaces: (aerodynamic pressure)***

The case if we have two sides of walls (exteriors and interiors), we calculate the pressure with this formula: $W(z) = p_p \times (C_{pe} - C_{pi})$

	Zone	$p_p\ [N/m^2]$	C_{pe}	C_{pi}	$W(z)\ [N/m^2]$
Wall	A'	1016,25	-1.0	0,11	-1128,03
	B'		-0.8		-924,7875
	D		+0.8		701,2125
	E		-0.3		-416,6625
roof	F	1016,25	-0,9		-1026,4125
	G		-0.8		-924,785
	H		-0.3		-416,6625
	I		-0.4		-518,2875
	j		-1		-1128,0375

Table 2.2.5: Wind pressure W1.

Vertical face W2 (b=19,68m; d=39,43m)

- ***Pressure coefficients***

1. External pressure coefficient

***a)* Vertical wall:**

The loaded surfaces of the walls are all considered bigger than 10m² .So: Cpe = Cpe10.

	A'	**B'**	**C**	**D**	**E**
Area (A)	45,93	183,73	230,48	229,66	229,66
Cpe	-1	-0,8	-0,5	+0,8	-0,3

Table 2.2.6: External pressure coefficients of wall W2

b) Roof

In our case we have two zones its area is less than 10m², which is mean:

	F	**G**	**H**	**I**
Area (A)	9,68	9,68	77,46	291,16
Cpe	-1,3	-1,3	-0,6	-0,5

Table 2.2.7: External pressure coefficients of roof W2.

2. Internal pressure coefficient

$A_o = (0,6 \times 2,5) \times 3 = 4,5m^2$ A_o : Area of the openings in this building.

$A_{f1} = (19,68 \times 10,2) = 200,73m^2$ A_{f1}: Area of a facade .

$\frac{4,5}{200,73} \times 100 = 2,24\% < 30\% \ldots\ldots\ldots\ldots\ldots$ verified (no esolated roof).

For values between h/d = 0, 25 and h/d = 1, 0 linear interpolation may be used.

$\mu p = \frac{(0,6\times2,5\times3)}{(0,6\times2,5\times3)} = 1$ $\frac{h}{d}$=0,29

Applying a linear interpolation $y = y_1 + \left[\left(\frac{x-x_1}{x_2-x_1}\right) \times (y_2 - y_1)\right]$ $we\ find$ Cpi= -0,31

- ***Wind pressure on surfaces: (aerodynamic pressure)***

	Zone	$\mathbf{p_p}$ **[N/m²]**	$\mathbf{C_{pe}}$	$\mathbf{C_{pi}}$	**W(z)N/m²**
Wall	A	1016,25	-1.0	-0,31	-701.212
	B		-0.8		-497,962
	C		-0.5		-193,087
	D		+0.8		497,962
	E		-0.3		10,162
roof	F	1016,25	-1.3		-1006.08
	G		-1.3		-1006.08
	H		-0.6		-294,7125
	I		-0.5		-193,0875

Table 2.2.8: Wind pressure W2

2.2.2 Snow load studies

S = μ × S_k

The altitude above sea level H=260m so: $S_K = \frac{0,04\times260+10}{100}$ = 0,204

$\mu = 0,8$ ………………….. Duopitch roof. So: S = 0,8×0,204= 0, 1632 KN/m².

2.2.3 Conclusion

Through this stage, we noticed that the wind calculation of an hangar is mainly similar to the building and that due to what is in common between the two structures; both of them are subjected to external and internal pressures.

We've noticed also that when the wind hits a big area of a façade, the more it creates big values of pressure, and that was seen through the wind pressure values in the two directions, so our verification and the elements final section will be based on the unfavorable case (big values of wind pressure). And in our case it is W1

2.3 FRAME ELEMENTS VERIFICATION

2.3.1. Introduction

This hangar is composed from deferent frames and elements that play huge role in transmitting the efforts and loads to the foundation (Column , rafters , purlins…).in order to verify and study the behavior of this elements under climatic load and verify all the instability states supposed, we will analysis this type of structure in ROBOT STRUCUTRAL ANALYSIS.

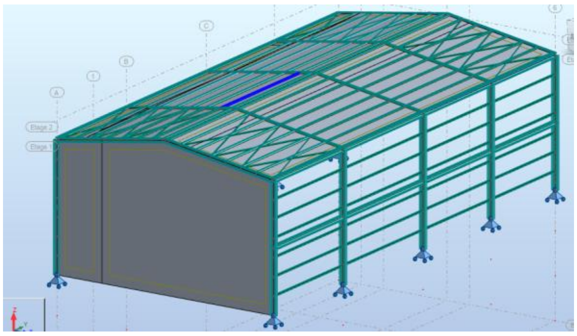


Figure 2.3.1: 3D view of hangar

2.3.2. Design calculation of purlins

- **Introduction**

At any steel construction the term purlin refers to the roof of structure, it is the most commonly used in metal building (hangar), they play an important role in transmitting the loads to the rafters meanwhile they can act as a compressive elements as a part of the bracing system also they participate in assuring the stabilization of the rafters under lateral buckling.

- **Conditions of dimensioning and calculating purlins**

In order to calculate the design and make all the important verifications of purlins those three conditions must be checked:

-Conditions of the resistance (ULS).

-Conditions of the shape stability (ULS).

-Conditions of the deflection (SLS).

- ***Resistance (ULS)***

It must be verified: $(\frac{M_y}{M_{ply}})^{\alpha} + (\frac{M_z}{M_{plz}})^{\beta} \leq 1$; $V_y \leq V_{ply}$; $V_z \leq V_{plz}$

Stress: $M_y = 38{,}96\text{KN.m}$ $M_z = 5{,}82\ \text{KN.m}$ $V_z = 13{,}60\text{KN}$

$\alpha = 2$; $\beta = 5n \geq 1$ $n = \frac{Nsd}{Npl}$ (in our case the axial effort Nsd = 0) ….β = 1.

$V_{plz} = \frac{A_{vz} \times f_y}{\sqrt{3} \times \gamma_{mo}} = \frac{11{,}25 \times 10^2 \times 275}{\sqrt{3}} \times 10^{-3} = 178{,}61\text{KN} \geq V_z = 13{,}60\text{KN}$ … ….checked.

$M_{ply} = \frac{166{,}4 \times 10^3 \times 275}{1} \times 10^{-6} = 45{,}76\ \text{KN.m} \geq M_y = 38{,}96\text{KN.m}$ … …. ….checked.

$M_{plz} = \frac{36{,}4 \times 10^3 \times 275}{1} \times 10^{-6} = 10{,}01\ \text{KN.m} \geq M_z = 5{,}82\text{KN.m}$ … … … …...checked.

$(\frac{38{,}96}{45{,}76})^2 + (\frac{5{,}82}{10{,}01})^1 = 1{,}30 \leq 1$ … … … … … … … … … … … … … … … … .. …..not verified.

- ***Shape stability (ULS)***

$(\frac{M_y}{M_{dev}}) + (\frac{M_z}{M_{plz}}) \leq 1$; $M_{dev} = \mathcal{X}_{LT} \times \beta_w \times \frac{W_{ply} \times f_y}{\gamma_{mo}}$

$\mathcal{X}_{LT}$: Lateral buckling's reduction coefficient.

Remark: all the procedures to determine this factor are mentioning in page 21.

In our case: $\mathcal{X}_{LT} = 0{,}51$ so: $M_{dev} = 45{,}22\text{KN.m}$

$\left(\frac{38{,}96}{45{,}22}\right) + \left(\frac{5{,}82}{10{,}01}\right) = 1{,}44 \leq 1$……………..not verified.

- ***Deflection (SLS)***

It must be checked: $f^{max} \leq f_{adm}$

$f_{adm} = \frac{11{,}19}{200} = 5{,}595\text{cm}$ $\geq$ $f^{max} = 24{,}3\text{cm}$. Not Checked

We have to increase the profile's stiffness.

Following same steps, the final purlin's profile section **IPE300.**

2.3.3 Vertical Bracing design calculation

In our case we use a diagonal bracing's system to provide a lateral stability preventing collapse of roof affected under wind's loads.

- **Verification of the tensile stress UPN140**

$N_{SD} = 78{,}83\text{kN} < Ntrd = \frac{20{,}4 \times 10^2 \times 275}{1} = 561\text{KN}$ … … checked.

- **Verification of the compressive effort**

$N_{sd} = 72{,}60KN < Nbrd = \chi \times \beta \times A \times \frac{fy}{\gamma m1}$.

Following the same steps to get the buckling reduction factor χ as it mentions in page 27.

In our case $\chi = 0{,}60$ which is mean $\quad \mathrm{Nbrd} = 0{,}14 \times 1 \times 20{,}4 \times 10^2 \times \frac{275}{1{,}1} = 81{,}96\mathrm{KN}$

$N_{sd} = 72{,}60\ KN < Nbrd = 81{,}96KN \ldots\ldots..well$ Checked.

2.3.4 Design calculation of Girts UPN160

Girts are secondary framing members that run horizontally between main frame columns and between endwall columns. Girts provides lateral support to the wall panel to resist wind loads.

- **Conditions of the resistance (ULS)**

Stress: $\mathrm{M_y} = 27{,}09\mathrm{KN.m}$ $\quad \mathrm{M_z} = 2{,}71\ \mathrm{KN.m}$ $\quad \mathrm{V_z} = 13{,}48\mathrm{KN}$

$\mathrm{V_{plz}} = 181{,}86 \geq \mathrm{V_z} = 13{,}48\mathrm{KN}$

$\mathrm{M_{ply}} = 34{,}5\ \mathrm{KN.m} \geq \mathrm{M_y} = 27{,}09\mathrm{KN.m} \ldots.\mathrm{checked}$

$\mathrm{M_{plz}} = 8{,}8\ \mathrm{KN.m} \geq \mathrm{M_z} = 2{,}71\mathrm{KN.m} \ldots\ldots\ldots...\mathrm{checked.}$

$(\frac{27{,}09}{34{,}5})^2 + (\frac{2{,}71}{8{,}8})^1 = 0{,}92 \geq 1 \ldots\ldots\ldots\ldots\ldots\ldots.\ldots..\mathrm{verified.}$

- **Conditions of shape stability (ULS)**

In our case: $\mathcal{X}_{\mathrm{LT}} = 0{,}92$ so: $\mathrm{M_{dev}} = 32{,}07$KN.m→ $\mathrm{M_y} = 27{,}09 \leq \mathrm{M_{dev}} = 32{,}07\mathrm{KN.m}$

- **Conditions of the deflection (SLS)**

$\mathrm{f_{adm}} = \frac{11{,}19}{200} = 4{,}865\mathrm{cm}$

Y-Y $\quad \mathrm{f^{max}} = 4{,}2\mathrm{cm} < 4{,}865cm\ldots.$ verified.

Z-Z $\quad \mathrm{f^{max}} = 0{,}3\mathrm{cm} < 4{,}865cm\ldots..$ verified.

2.3.5 Verification of eave strut HEA360

- **Compound bending with the risk of lateral buckling**

$$\frac{N_{sd}}{f_y \times A \times \mathcal{X}_{min}/\gamma_{m1}} + \frac{K_{LT} \times M_{sd}}{\mathcal{X}_{LT} \times W_{plz} \times f_y/\gamma_{m1}} \leq 1$$

following the same steps as we did in the previous chapters to get all the coefficients of this formula: $\frac{100{,}96 \times 10^3}{275 \times 14280 \times \frac{0{,}27}{1.1}} + \frac{1{,}11 \times 69{,}97 \times 10^6}{0{,}43 \times 802{,}3 \times 10^3 \times \frac{275}{1{,}1}} = 1 \leq 1 \ldots..\mathrm{checked}$**.**

- **Deflection**

$f_{adm} = \frac{11,19}{200} = 5,595cm \quad \geq \quad f^{max} = 3,6cm.$ Well Checked.

2.3.6 Verification of Rafters HEA320

- **Compound bending with the risk of lateral buckling**

$\frac{N_{sd}}{f_y \times A \times X_{min}/\gamma_{m1}} + \frac{K_{LT} \times M_{sd}}{\chi_{LT} \times W_{plz} \times f_y/\gamma_{m1}} \leq 0,83 \leq 1$…….well checked.

- **Deflection**

$f_{adm} = \frac{9.95}{200} = 4,97cm \quad \geq \quad f^{max} = 3,0cm.$ Well Checked.

2.3.7 Design calculation of columns HEA500

It must be verified $\frac{N_{sd}}{\chi_{\min} \times A \times f_y / \gamma_{M1}} + \frac{K_y \times M_{y.sd}}{W_{pl.y} \times f_y / \gamma_{M1}} + \frac{K_z \times M_{z.sd}}{W_{pl.z} \times f_y / \gamma_{M1}} \leq 1$

Following same steps mentioned in chapter 6 parts 1 we resume all buckling coefficients in this following table:

λ_y	ϕ_y	χ_y	$\beta_{M.y}$	μ_y	K_y
0,56	0,74	0,59	1,3	0,84	0,99

Table 2.3.1 buckling parameters

λ_z	ϕ_z	χ_z	$\beta_{M.z}$	μ_z	K_z
1,63	2,18	0,20	1,3	-2,28	1

Table 2.3.2 buckling parameters 2

$N_{sdcorr} = 158,07KN. \quad M_{ymax} = 641,3KN.m \quad M_{zcorr} = 102,44\ KN.m$

$\frac{158,07 \times 10^3}{0.21 \times \frac{197500 \times 275}{1.1}} + \frac{1 \times (641,3 \times 10^6)}{\frac{9319000 \times 275}{1.1}} + \frac{1 \times (102,44 \times 10^6)}{\frac{1059000 \times 275}{1.1}} = 0,16 \leq 1$……………..well checked.

2.3.8 Seismic separation joint dimension

Seismic joint creates a separation between two buildings where its width must be determined by this expression: $d_{min} = 15 + (\delta_1 + \delta_2) \geq 40mm.$

δ_1: Displacement due to seismic forces (E_x) in steel building for H=10,2m.

δ_2: Displacement due to wind forces (E_y) in hangar for H=10,2m.

$\delta_1 = 23mm$, $\delta_2 = 21mm \quad d_{min} = 15 + (23 + 21) = 59mm \geq 40mm$ … … … … checked.

Between the steel building and the hangar we have to keep a spacing of 60cm for the seismic joint.

2.4 DESIGN OF JOINTS

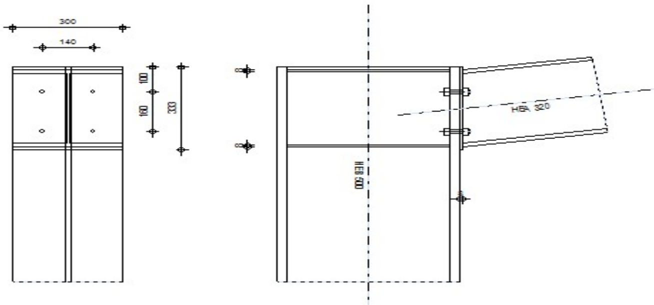
2.4.1 Introduction

In the previous structure (steel building) we have calculated the assembly and all the design frame joints manually, so in order to explore and learn more the two methods, hangar's design joint will be calculated and verified automatically by Robot Structural Analysis.

2.4.2 Design joint Column –rafter HEA500-HEA320

Autodesk Robot Structural Analysis Professional 2019
Calcul de l'Encastrement Traverse-Poteau
NF EN 1993-1-8:2005/NA:2007/AC:2009

OK
Ratio
0,61



Assemblage satisfaisant vis à vis de la Norme Ratio 0,61

- **Bolts**

bolt's diameter	Bolt's class	bolt's number / row's number	e1 mm	F_{trd}
14	4.8	2 / 2	140	33,12

Table 2.4.1: Bolt's characteristics.

- **Plate**

Height mm	Width mm	Thickness mm
333	300	8

Table 2.4.2: Plate's characteristics.

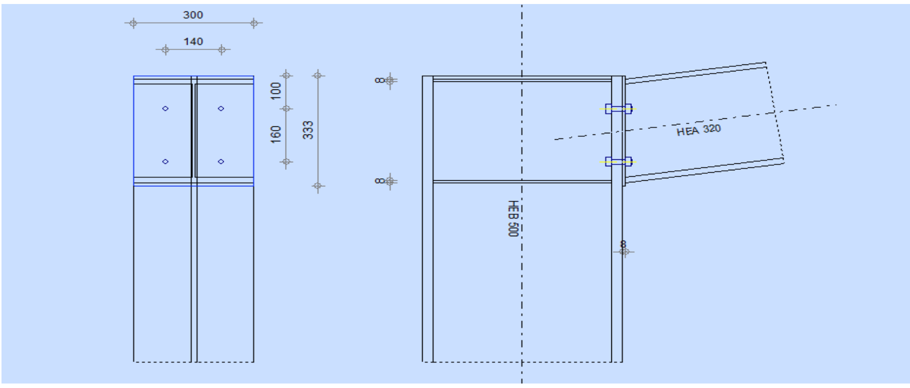
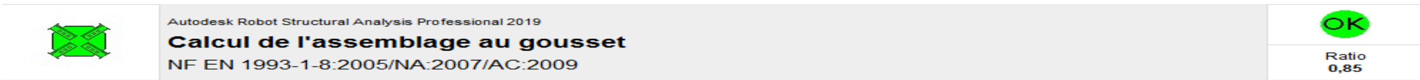
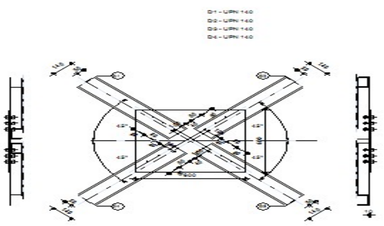


Figure 2.4.1: Design assembly column-rafter

2.4.3 Design Bracing system





- **Bolts**

bolt's diameter	Bolt's class	bolt's number	d	fub
14	4.8	3	15	400

Table 2.4.3: Bolt's characteristics

- **Gusset**

Height mm	Length mm	Thickness mm
600	600	10

Table 2.4.4: Gusset characteristics

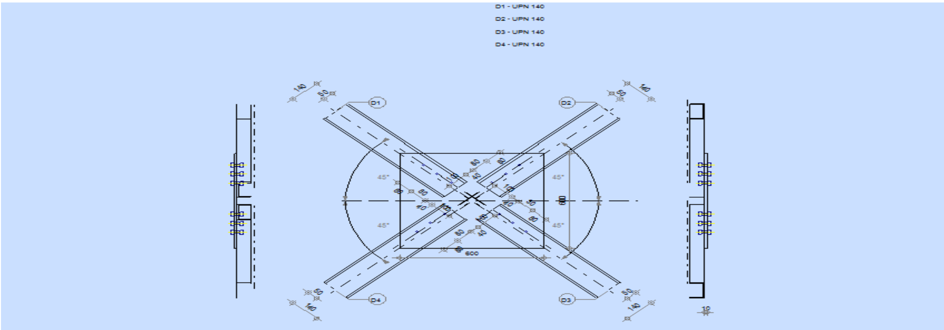


Figure 2.4.2: Design assembly of bracing.

2.4.4 Base plate column HEA500

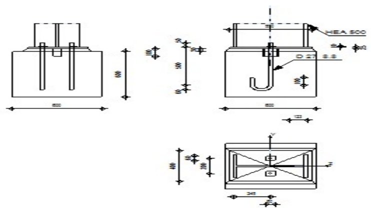
Autodesk Robot Structural Analysis Professional 2019

Calcul du Pied de Poteau articulé

Eurocode 3: NF EN 1993-1-8:2005/NA:2007/AC:2009 + CEB Design Guide: Design of fastenings in concrete

OK

Ratio 0,31



Assemblage satisfaisant vis à vis de la Norme Ratio 0,62

- **Plate**

length mm	Width mm	Thickness mm
550	400	25

Table 2.4.5: Plate’s characteristics.

- **Anchor**

diameter	class	number	ev	fu
27	8.8	2	200	800

Table 2.4.6: Bolt’s characteristics

- **Footing**

Length mm	Width mm	Height mmm
600	600	600

Table 2.4.7: footing characteristics

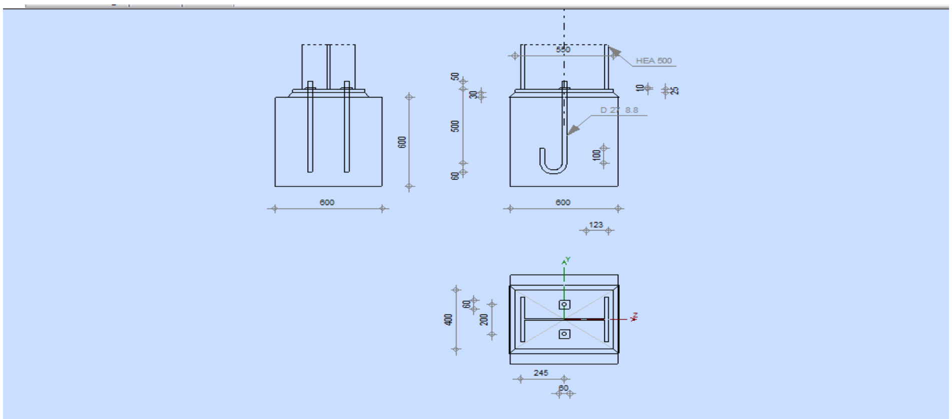


Figure 2.4.3: Design assembly of base column

2.4.5 Conclusion

Through this stage we were able to study the bolt connection by ROBOT software, the design joint of all the components of this hangar was verified successfully (bolt, plate, gusset …) and that was seen through the rate mentioned above (green: refers to good assembly).

3.1 PRESENTATION

3.1.1 Description

This suspended swimming pool is considered as a part of this current touristic camp, where this last is situated at Blida's province. The current pool is made by reinforced concrete located exactly at the hangar in its underground, above two basements with an area of 325m².

Length ……………………..L=25m.

Width……………….…….W =13m.

3.1.2 Components

The nature of the structural components are made by a reinforced concrete because they received a huge load especially the one of the pool with its height dimensions (semi Olympic), that's why here we will use a solid slab instead of a coffer slab to transmit the load to the two directions.

1) Beam

BAEL91 Conditions $\frac{L}{15} \leq h \leq \frac{L}{10}$ and $0{,}3.h \leq b \leq 0{,}7.h$

	h (cm)	b(cm)
beam L= 6,5m	60	35

Table 3.1.1: Beam's dimensions

RPA99 Conditions

it must respect the conditions below

			beam	checking
Seismic zone III	Condition 1	$b \geq 20cm$	35	Checked
	Condition 2	$h \geq 30cm$	60	Checked
	Condition 3	$\frac{h}{b} \leq 4$	1,71	Checked

Table 3.1.2: Beam's verification

2) Column

The area of the most requested column is $S = \frac{5+5}{2} \times \frac{6{,}5+6{,}5}{2}$= 32,5m².

2nd basement	floor	6,38×32,5=207,35
	Beam I	0,6×0,35×6,5×25=34,125
	Beam II	0,6×0,35×5×25=26,25
	column	0,4×0,4× (3,96-0,6) ×25=13,44
	Total	281,165
1st basement	floor	6,38×32,5=207,35
	Beam I	0,6×0,35×6,5×25=34,125
	Beam II	0,6×0,35×5×25=26,25
	column	0,4×0,4× (3,4-0,6) ×25=11,2
	Total	278,925

Table 3.1.3: Column's dead load

	N_G(KN)	N_Q(KN)	N_u(KN)	B_r (cm^2)	a(cm)	b(cm)	section
2nd basement	281,165	81,25	501,44	266,43	31,68	11	40*40
1st basement	278,925	81,25	498,423	264,83	34	10,27	40*40

Table 3.1.4: Column's dimensions

3) Slab

Fire resistance

e = 7 cm for one hour of shot.

e = 11cm for two hours of shot.

e = 17.5cm for four hours of shot. We choose e = 15cm.

Acoustic insulation

According to the CBA rules in Algeria, the thickness of the slab must be greater or equal than 13cm to obtain a good acoustic insulation. We choose e = 15cm.

Bending strength

$\alpha = \frac{l_x}{l_y}$ $\alpha = \frac{5}{6,5} = 0,76 \leq 1$

$0,4 \leq 0,76 \leq 1$ So the slab is bearing in two directions …*BAEL99*

Slab resting on three or four constraints……… $\frac{L_x}{50} \leq e \leq \frac{L_x}{40}$

$\frac{650}{50} \leq e \leq \frac{650}{40}$ ………$13 \leq e \leq 16,25$ So we choose e=16cm.

The choice of the thickness of the solid slabs, e ≥ max (15; 15; 16) cm⇒ e = 20 cm

4) Wall

According to RPA99 the minimum value of the thickness is 15 cm,

H=2,5m ….height of the pool's walls $e \geq \frac{2,5}{20}$ … …. we choose e=20cm

3.2 INTERACTION FLUID-POOL

3.2.1 Introduction

There are a lot of varieties of swimming pools existed at this époque. In general we define the word swimming pool as a tank containing water with different types of shape and multitude of forms; also it can be found buried or above-ground and inside or outside a building.

There are three types of swimming pools:

Aboveground pool: it might be in kit or freestanding pool.

In ground pool: we distinguish, shell, masonry pools, as it might be in kit.

Suspended pool: founds on upper level of a building, made by reinforced concrete posed on beams.

3.2.2 The relation between Suspended Pool and tank

The calculation of swimming pool is assimilated to a tank due to the interaction between fluid and structure (swimming pool), the seismic behavior of a suspended pool represents a complex phenomenon ,so the dimensioning of such structure requires a knowledge of its seismic behavior which we take in consideration the water movement with the height of swimming pool (suspended) in seismic state, they present very important pillar that our calculation must be based in. we distinguish the object of doing such study is to ensure the stability and the security of the seismic behavior of an elevated swimming pool with taking in consideration the effects of the stored water of the interaction (water-pool) and the effect of the sloshing.

Sloshing: refers to small movements of a liquid contained in a tank or a pool, it describes a free surface oscillation of a fluid in a partially filled pool, for slosh the liquid must have a free surface.

Keywords*: interaction (fluid-pool), tank, sloshing, seismic behavior.*

3.2.3 Interaction (fluid-pool)

the fluid-pool coupling is probably the most encountered interaction faced after the ground-structure coupling. A seismic analysis of such structures (tank, swimming pool…) is very needed because they aren't independent of each other and both they behave like a coupled dynamic system; these effects introduce substantial changes in the modal characteristics of the structure such as frequencies and natural modes of vibrations. Practically a big number of situations involve this type of interaction here are some cases:

- Fluttering airplane wings.

- Suspension bridges, skyscrapers and vibrating cables under the effect of wind.
- Tanks partially filled with liquid undergoing the sloshing effect surface of the liquid.

This interaction is a part of multiphysical problems where two materials haven't the same behavior but the two interact in a strongly coupled system.

The deformations of the structure under the effect of the forces imposed by the flow of the fluid, modify the configuration of the interface fluid structure, the conditions of flow of the fluid are affected which induces a modification of the force exerted on the structure at the interface level, completing the cycle of interaction.

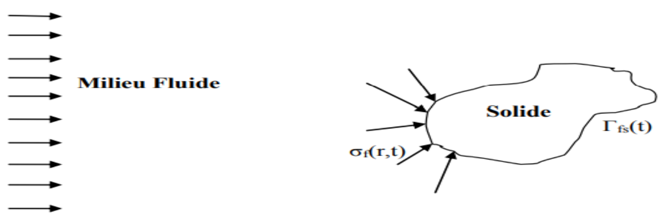


Figure 3.2.1: Interaction fluid-structure

The interaction between fluid-pool can be mathematically modeled based on the movement equations of the structure (rigid or deformable) and of the fluid (compressible or incompressible) ,In order to study the model of this type of interaction it's necessary to :

- Identify and know the characterizations of the physical phenomenon studied.
-Choice of a method.

3.2.4 Methods of calculation

Through those methods we can calculate the pressure exerted on the walls of this suspended swimming pool, from a dynamic point of view there are two cases:

-If a tank is completely full of water or all empty it is essentially make a one mass structure.(the value of the relative displacement is zero).

-The tank partially filled of water so the water has free surface to move there will be a *sloshing* during an earthquake and this make essentially two mass structure:

impulsive mass*:* when the walls of the tank or pool accelerate back and forth a certain fraction of the water is forced to participate in this motion, which exerts a reactive force the same as would be exerted by a mass that is attached rigidly to the tank at the proper height.

Convective mass*:* the motion of the tank walls excites the water also into oscillation which in turn exert an oscillating force on the tank, this oscillating force is the same as would be exerted by a mass can oscillate horizontally against a restraining spring, this mass corresponds to the fundamental mode of oscillation of the water which this last describe the important mode for most earthquake problems.

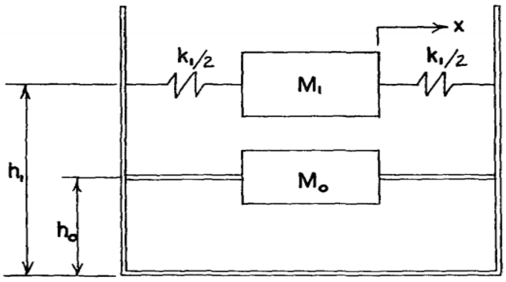


Figure 3.2.2: Convective mass

M0 and M1 produce dynamic forces equivalent to those produced by the global water. Therefore, ***an understanding the earthquake damage of suspended water pool requires a deep understanding of the dynamic forces associated with sloshing water.***

After understanding the phenomenon of the mass and sloshing we set above some methods for the verification of the fluid pool:

1-The JACOBSEN and AYRE method

In which the stresses produced by water waves are neglected *(no convective forces)* to take into account only the impulsive forces. Its conditions are:

- Walls of the tank or pool must be rigid and not deformable.
- Fluid must be considered: incompressible and non-viscous.
- Only the horizontal acceleration direction must be considered.
- We suppose the values of the displacement are small.
- This method can be used for cylindrical tanks or pools only.

2-The HUNT and PREISTLEY method

This method differs from the previous one by taking into account, the convective effort, the pressure that made by the formation of a wave and it can be used for rectangular pools also, same conditions as the first one but its formulas applied for two types of tanks:

- The filling rate H / L <1.5.
- The filling rate H/L >1,5.

3-HOUZNER method

Because of the complexity of the previous methods, *Houzner* established an approximate method based on decomposition of the action of the liquid: a passive action and an active action.

4-Recommendations of EC8

It handles the deferent types of tanks (rigid or deformable) and also the shape (rectangular or cylindrical) and like the other methods the fluid movements determined by two components (convective and impulse mass) .

The choice of a method depends to the complicity of the structure studied, which is mean: if the filling rate H / L <1.5 it is preferable to work with *HOUZNER* or *EC8* method because HUNT and PREISTLEY method leads to a complicate force expressions but in case we have a big altitude of the tank the filling rate H/L >1, 5, HOUZNER or EC8 methods give approximate results of 10% this is why in this case it's important to use *HUNT* and *PREISTLEY* method to give the exact values .

5-Finite elements method

Objective:

It's a numerical method for solving partial deferential equations with two or three variables by the subdivision of a large system into smaller and simpler that called finite elements interconnected at points called *nodes*.

The modeling of fluid-structure developed by the finite element method is a recent discipline which aims to analyze and simulate deferent problems of this interaction which this last leads to differential equations with boundary conditions.

This model is used for the determination of the seismic affects on this current pool which must reproduce with ideal stiffness, mass and the geometric properties and also take in account the hydrodynamic response of the contained liquid by representing them with a simplified mechanical model (spring-mass) with two types of options :

One degree of freedom model: To deal with this concept, the tank is considered completely filled or completely empty.

Two degree of freedom model: about this model, the tank is considered partially filled which the action of the fluid is composed of two:

- Passive action causing inertial forces generated by impulsive mass.
- Active action causing sloshing forces generated by convective mass.

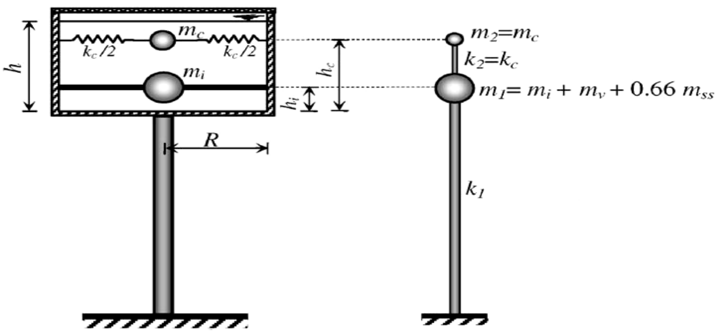


Figure 3.2.3: Two degree of freedom model

3.3 APPLICATION

3.3.1 Objective

Here the study is carried out for the behaviour of B+2 stories building ,through it the elevated swimming pool's model will have deferent water depths which will be analiyized then a comparative results will be done in terms of displacment storey , storey drift and shear force by the time history function. (all the details of this part are illsutred in appendix A)

3.3.2 Choice of the method

The filling rate is: $\frac{H}{L} = \frac{2,5}{25} = 0,1 < 1,5$ …….. Means we can work with the both methods HOUZNER / EC8 methods.

- **EC8 recommendations**

According the EC8, the impulsive, convective mass, hc and hi are all represented in this table:

h/L	*Mi/Me*	*Mc/Me*	*Hi/h*	*Hc/h*
0.3	*0.176*	*0.824*	*0.400*	*0.521*
0.5	*0.300*	*0.700*	*0.400*	*0.543*
0.7	*0.414*	*0.586*	*0.401*	*0.571*
1	*0.548*	*0.452*	*0.419*	*0.616*
1.5	*0.686*	*0.314*	*0.439*	*0.690*
2	*0.763*	*0.237*	*0.448*	*0.751*
2.5	*0.810*	*0.190*	*0.452*	*0.794*
3	*0.842*	*0.158*	*0.453*	*0.825*

Table 3.3.1: EC8 recommendations

The equivalent system is specified by the following quantities for rectangular pool of h=2,5m

$\frac{\mathrm{h}}{\mathrm{L}} = \frac{2,5}{12,5} = 0,2$ We choose 0, 3. Means:

0, 3…………..….$\frac{Mi}{8125} = 0,176 \dots \dots \dots \dots \dots . . \mathrm{Mi} = 1430\mathrm{KN}$

0, 3…………..….$\frac{Mc}{8125} = 0,824 \dots \dots \dots \dots \dots . . \mathrm{Mc} = 6695\mathrm{KN}$

0, 3……………..….$\frac{hi}{h} = 0,400 \dots \dots \dots \dots . . \dots \dots hi = 1m$

$0,3 \ldots\ldots\ldots\ldots\ldots \frac{hc}{h} = 0{,}521 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 1{,}3m$

Kc=259,76 KN/m

- **Houzner Method**

The equivalent system is specified by the following quantities for rectangular pool of length 2L and water depth h=2,5m

$\mathrm{M}_0 = H \times b \times d \times \rho = 2{,}5 \times 25 \times 13 \times 10^3 = 8215KN$

M_0 : Total mass of water in pool.

$$M_i = M_0 \times \frac{TANH(1{,}7L/h)}{(1{,}7L/h)} = 8215 \times \frac{\mathrm{TANH}(1{,}7 \times \frac{12{,}5}{2{,}5})}{1{,}7 \times \frac{12{,}5}{2{,}5}} = 955{,}88\mathrm{KN}.$$

M_i : Impulsive mass.

$$M_c = M_0 \times \frac{0{,}83 \times TANH(1{,}6h/L)}{(1{,}6h/L)} = 8215 \times \frac{0{,}83 \times \mathrm{TANH}(1{,}6 \times \frac{2{,}5}{12{,}5})}{1{,}6 \times \frac{2{,}5}{12{,}5}} = 6729{,}4\mathrm{KN}$$

M_c : Convective mass.

$$k_c = \frac{3 \times {M_c}^2}{\mathrm{M}_0} \times \frac{g \times h}{L^2} = 262{,}44\mathrm{KN} / \mathrm{m}$$

$$h_i = \frac{3}{8} h \times \{1 + \alpha \times [\frac{\mathrm{M}_0}{M_c} \times (\frac{L}{h})^2 - 1]\}.$$

If the heights hi and hc are to be determined on the basis of the dynamic fluid forces exerted on the walls of the tank only (not on the floor), the following values should be used for both cylindrical and rectangular tanks: $\alpha = 0 \;\; so \quad h_i = \frac{3}{8} h = 0{,}93m$

$$h_c = h\left[1 - \frac{ch\left(1.58 \times \frac{h}{L}\right) - 1}{\sqrt{\frac{27}{8}} \cdot \frac{h}{L}\, sh\left(\sqrt{\frac{27}{8}} \frac{h}{L}\right)}\right]$$

h_c= 1, 25 m

	Housner	[3]
Mi (KN)	955,88	1430
Mc(KN)	6729,40	6695
hi(m)	0,93	1
hc(m)	1,25	1,3
Kc (KN/m)	262,44	259,76

Table 3.3.2: Comparison between Housner and EC3

Those results show that the stiffness and the convective mass obtained by Housner are higher than Eurocode values, by calculating the error rate of stiffness and mass we find:

$\Delta M_c = \frac{6729{,}40-6695}{6729{,}40} \times 100 = 0{,}50\%$ $\Delta K_c = \frac{262{,}44-259{,}76}{262{,}44} \times 100 = 1\%$

It is obvious that the error rate between the mass and stiffness is very close with a very small percent which this last will bring us to very close results.

With *Houzner* method, we follow the same steps to calculate those values for various water depths, we find:

h (m)	**0,5h (m)**	**0,6h (m)**	**0,65h (m)**	**0,7h (m)**	**0,75h (m)**	**0,95h (m)**
M_i (KN)	238,97	344,11	403,86	468,38	537,68	862,68
M_c (KN)	3370,07	4043,14	4379,49	4715,69	5051,75	6394,25
h_i (m)	0,46	0,56	0,60	0,65	0,70	0,89
h_c (m)	0,62	0,75	0,81	0,87	0,93	1,18
k_c (KN/m)	65,82	94,73	111,15	128,87	147,90	236,95

Table 3.3.3: Housner's results with deferent water depths

- ***Remarks***

The impulsive mass is very small compared to the convective mass.

The impulsive and the convective mass increase with increasing the water depth.

3.3.3 Pool's model

In order to ensure and verify the siesmic response of this pool and check the intraction of fluid structre , we will use this software SAP2000.

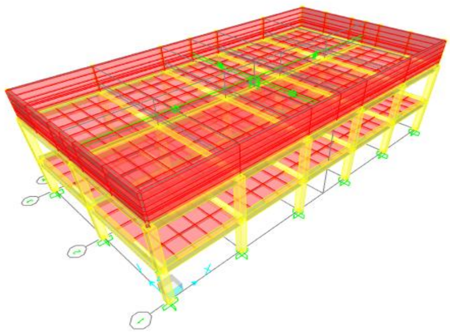


Figure 3.3.1: Pool's 3D View

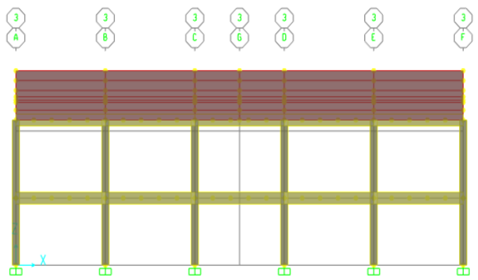


Figure 3.3.2: Pool's X-Z View

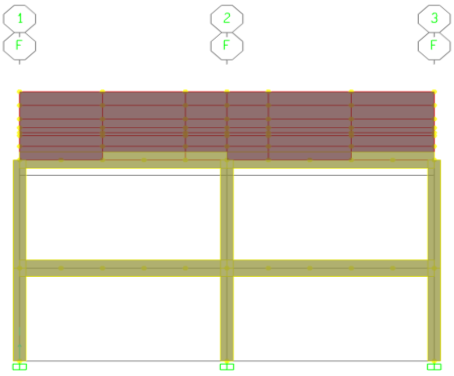


Figure 3.3.3: Pool's Y-Z View

Steps to introduce the mass values:

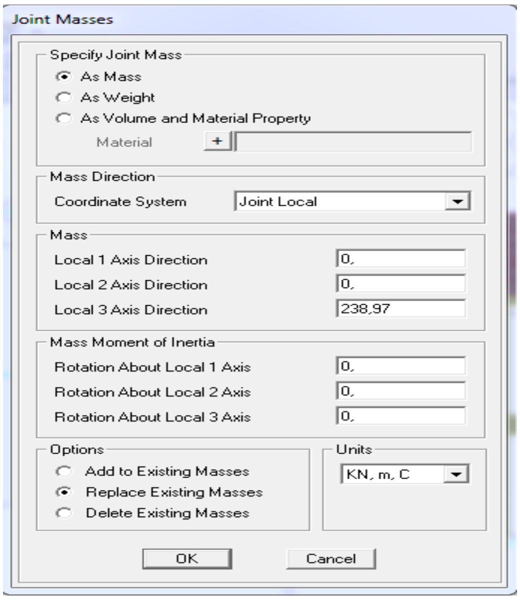


Figure 3.3.4: Impulsive mass

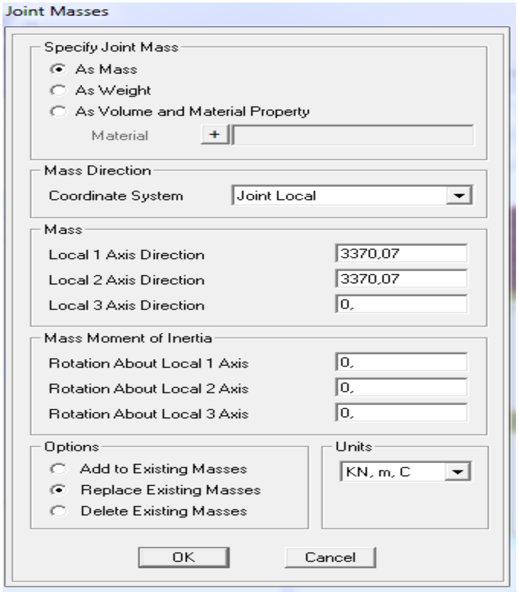


Figure 3.3.5: Convective mass

After analyzing the model, the natural periods of this partially filled pool are summarized in this following table:

	period	**Participating Mass x**	**Participating Mass y**	**Type of deform**
Mode 1	0,54	0	0,86073	Translation y
Mode 2	0,51	0,88068	0	Translation x
Mode 3	0,47	1,362E-20	0,01452	torsion

Table 3.3.4: Housner's results with deferent water depths

➢ **Time history**

Time history gives the response of a structure over time during and after the application of a load, which allows us to define this function in our studies, to verify the effect of sloshing during and after an earthquake.

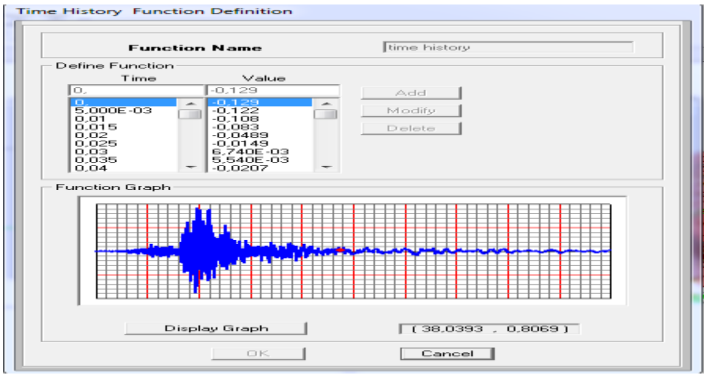


Figure 3.3.6: Time history function

3.3.4 Storey displacement

Water depth (h)	Pool (mm)	Basement 2(mm)	Basement 1(mm)
0,95h	284,805	19,245	142,215
0,75h	284,85	19,305	142,245
0,7h	284,855	19,315	142,255
0,65h	284,86	19,325	142,26
0,6h	284,87	19,34	142,27
0,5h	284,895	19,36	142,27

Table 3.3.5: Storey displacement

- ***Interpretation***

According to the results and our observations that based on this table of the joint displacement of each level, we notice that this last doesn’t change its values with increasing the filling rate of this pool.

3.3.5 Inter-Storey drift

storey	Uy (mm)	U.R	ΔU(mm)	Δ(mm)	checking
Pool	56,979	284,985	19,36	25	verified
Basement 2	53,107	265,535	142,27	39,6	Not verified
Basement 1	24,653	123,265	123,265	34	Not verified

Table 3.3.6: Inter-storey drift

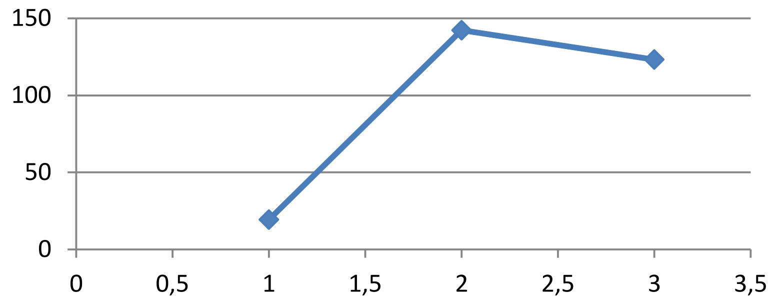


Figure 3.3.7: Curve storey drift –water depth y-y axis

- ***Interpretation***

The inter storey drift of the basements does not verify because of the high dimensions of the pool that allow the infrastructure to displace and the absence of the walls (car park state), that's why we should increase the concrete section of the components:

Column ……………………… (75*95) cm².

Beam ………………………..…….(80*50)cm² .

Slab……………………..………e=25cm.

Wall ……………………………….e=30cm.

After analyzing with those new properties sections, we summarize:

storey	U (mm)	U.R	ΔU(mm)	Δ(mm)	checking
Pool	11,47	57,35	13,45	25	verified
Basement 2	8,78	43,9	27,3	39,6	verified
Basement 1	3,32	16,6	16,6	34	verified

Table 3.3.7: Storey drift with new section

3.3.6 Verification of base shear force

The global base shear force V that applied at the base of structure, it determined by this expression $V = \frac{A.D.Q}{R}.w$ following the same steps as it mentions in page 45 (Dynamic design) and RPA's procedures we find:

A	0,25
D	2,13
Q	1,1
R	5
W (KN)	16393,97
V (KN)	1920,55

Table 3.3.8: Base shear force's calculation

Once the analysis is done, the shear force values are:

F (KN)	0,5h	0,6h	0,65h	0,7h	0,75h
FX (KN)	10469,43	10781,47	10488,376	10491,526	10491,658
FY (KN)	10519,751	10491,378	10483,237	10485,558	10487,603

Table 3.3.9: Base reactions

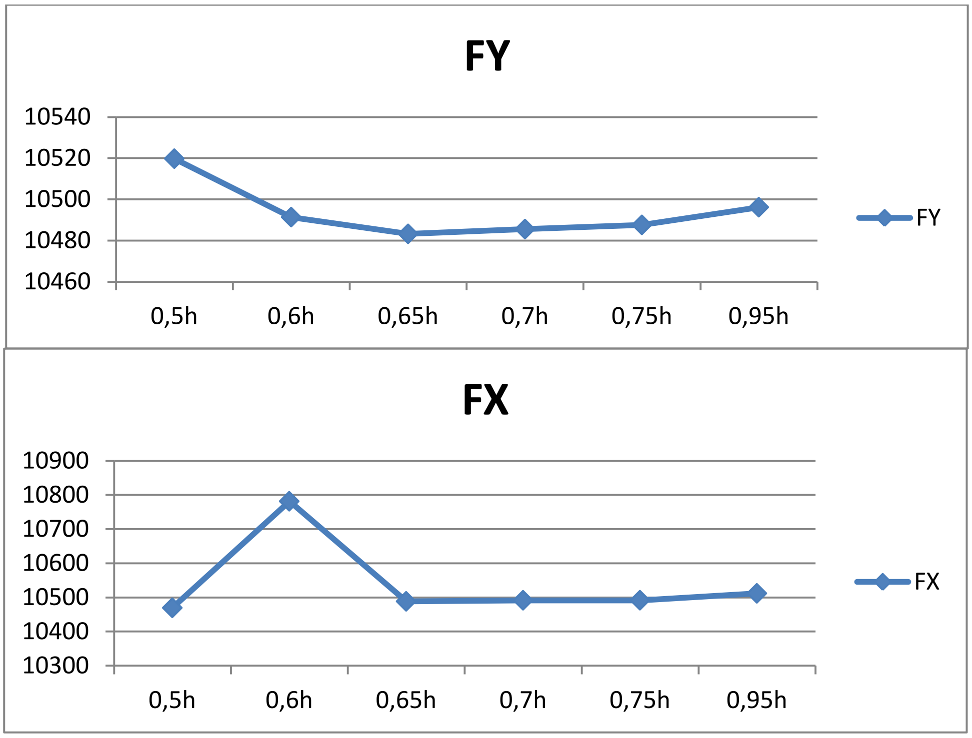


Figure 3.3.8: Base reaction evaluation

- ***Interpretation***

According to X axis we notice that the seismic forces increases in the interval 0, 5h—0,6h then it begins to decrease from 0,6h till 0,65h.

According to Y axis we notice the decrease of the seismic forces from 0,5h-0,65h then it begins to increase with small values till 0,95h.

F	0,5h	0,6h	0,65h	0,7h	0,75h	0,8 V	checking
FX	10469,43	10781,47	10488,376	10491,526	10491,658	1536, 44	checked
FY	10519,751	10491,378	10483,237	10485,558	10487,603		checked

Table 3.3.10: Base reactions verification

3.3.7 Joint displacement of the convective mass due to its water depth

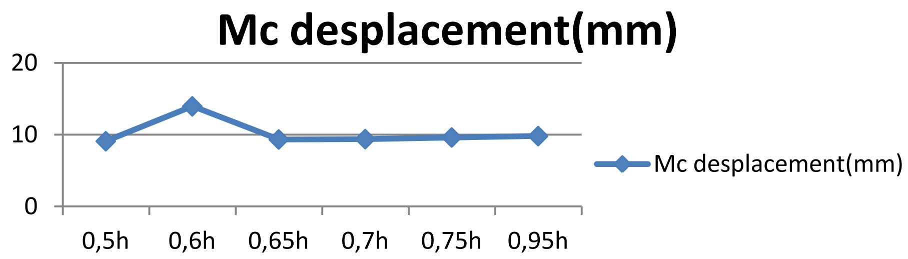


Figure 3.3.9: Convective mass displacement evaluation

- ***Interpretation***

We notice that the displacement increase rapidly in the interval 0,5h-0,6h and it reaches the maximum value 13,95mm in 0,6h then decrease directly until it reaches 0,95h.

3.3.8 Elements forces-frames

	0,5h	**0,6h**	**0,65h**	**0,7h**	**0,75h**
T(KN)	1081,92	1348,73	877,57	877,4	877,233
M (KN.m)	787,14	881,72	805,54	805,27	805,022

Table 3.3.11: Efforts-beam

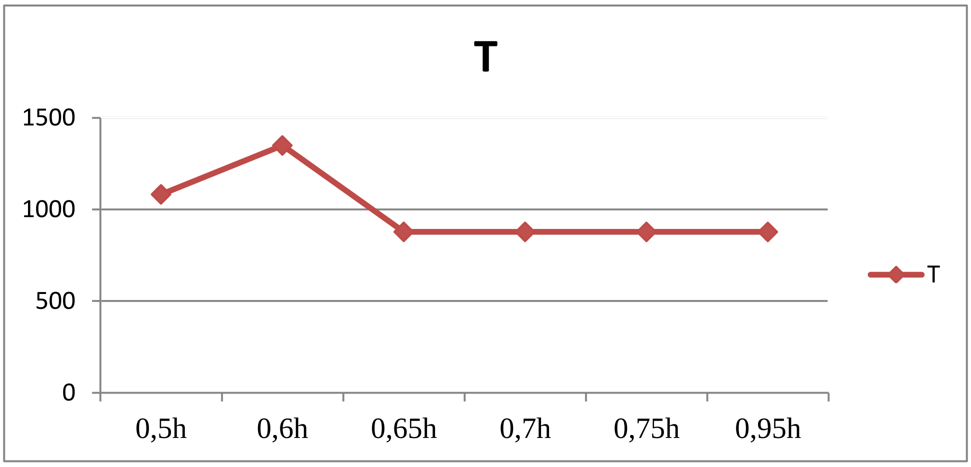


Figure 3.3.10: Shear force evaluation

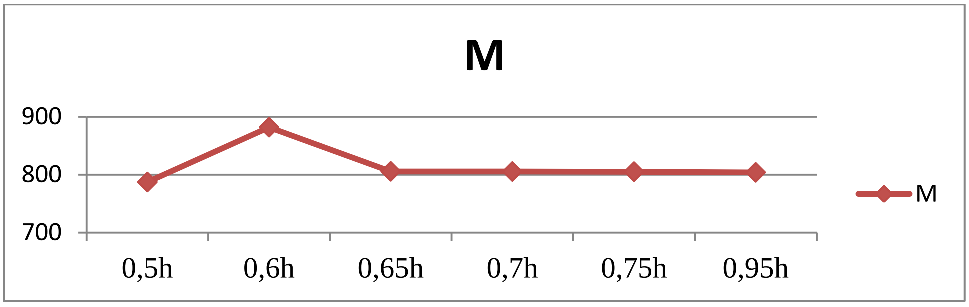


Figure 3.3.11: Bending moment evaluation

- ***Remark***

We notice the change of the values (T and M) in the interval 0,5-0,65h.

3.3.9 General Conclusion

After analyzing those results, the following conclusions are made of our study:

1. According to the finite elements method, working with time history, the joint displacement of the convective mass, storey, the bending moment and shear force vary depending on changing the water depth.

2. After the results we get, we see that the sloshing occurs in the interval (0.5h; 0.65h).

3. The displacement of the levels of stages increases slightly with few mm due to its water depth; this explained that the height of the swimming pool is small compared to a raised tank.

4. The Sloshing on the free surface has influence on the movement of the pool and this is because of the high dimensions of the pool +water which this last allow the pool to displace and this led us to increase the stiffness of the elements to avoid the storey drift.

5. Taking into account the fluid structure interaction plays important role in terms shear force and the bending moment.

6. The 2DDL model studied using Housner's expressions and the recommendations of [3] are very close.

3.4 REINFORCEMENT DESIGN

3.4.1 Introduction

After having made the evaluation of the efforts for various water depths, we found that the sloshing interval which explains the unfavorable interval that our calculation must be based in, according to the last curves of the efforts the highest value is when the pool is filled with 1,5m of water (0,6h).

3.4.2 Beam

The reinforcement of the beams is given by the simple banding's procedures.(no thermal study) so we do the calculation for the following situations:

1, 35G+1,5Q…………….………ULS99

G + Q ± E…………………….RPA99V2003

0.8 G ± E……………………...RPA99V2003

The reinforcement sections of the following beams are calculated based on **SOCOTEC** software for accidental and persistent situations; the following tables summarized all the results

- **1, 35G+1,5Q**

Storey	Section (cm^2)	Position	M^{max} (kNm)	A_s (cm^2)	A_s' (cm^2)
Pool	80x50	Span	222,68	7,26	0
		constraint	115,83	0	3,72
Basement2	80x50	Span	227,401	7,41	0
		constraint	118,05	0	3,8

Table 3.4.1: Reinforcement section 1,35G+1,5Q

- **G+Q± E**

Storey	Section (cm^2)	Position	M^{max} (kNm)	A_s (cm^2)	A_s' (cm^2)
Pool	80x50	Span	881,72	26,58	0
		constraint	831,87	0	24,94
Basement2	80x50	Span	650,22	19,1	0
		constraint	502,85	0	14,55

Table 3.4.2: Reinforcement section G+Q± E

- **0.8 G ± E**

Storey	**Section (cm^2)**	**Position**	**M^{max} (kNm)**	**A_s (cm^2)**	**A_s’ (cm^2)**
Pool	80x50	Span	875,38	26,37	0
		constraint	838,21	0	25,15
Basement2	80x50	Span	617,48	18,08	0
		constraint	520,95	0	15,1

Table 3.4.3: Reinforcement section1 0.8 G ± E

1) Longitudinal bars reinforcement

The minimum total percent of the longitudinal bars over the entire length of the beam is 0,5% at any section.

The maximal total percent of the longitudinal bars is:

- 4% in the current zone.

- 6% in the lapped zone.

The adopted final reinforcement is given by this following table:

Storey	Section (cm^2)	Position	M^{max} (KNm)	A_{sRPA}^{min} (cm^2)	A_s^{cal} (cm^2)	Choice of reinfor+ cements	A_s^{adp} (cm^2)
Pool	80*50	Span	881,72	20	26,58	6HA25	29,45
		constraint	838,21		25,15	6HA25	29,45
Basement2	80*50	Span	650,22	20	19,1	8HA20	25,13
		constraint	520,95		15,1	8HA20	25,13

Table 3.4.4: Final choice of reinforcement

➢ **Verification**

- ***Condition of non-fragility*** $A_s \geq A_s^{min} = 0,23bd\,\frac{f_{t28}}{f_e}$

f_{t28}=2,4MPA f_e=500MPA

Section (cm2)	A_s^{adp} **(cm2)**	A_s^{min} **(cm2)**	**Verification**
80*50	25,13	3,97	checked

Table 3.4.5: Verification of non fragility

- ***Service limited state SLS***

All the stress (concrete and steel) must be also calculated at the service limited state (M_{ser}) then comparative results for the both materials must be done with the allowable stress values.

It must be checked: $\sigma_{bc} \leq \overline{\sigma}_{bc}$ and $\sigma_s \leq \overline{\sigma}_s$

$\overline{\sigma}_{bc}$: Allowable concrete stress.

$\overline{\sigma}_s$: Allowable steel stress.

In our case it is ***damaged cracking***: because the area or the use of the whole structure isn't exposed in aggressive environment (behind sea water, gas, corrosive..).

All the details and the procedures to verify at service limited state are mentioned in appendix B.

All the results are summarized in the following table:

Storey	**Position**	**M_{ser} (KN.m)**	**σ_{bc} (MPa)**	**$\overline{\sigma}_{bc}$ (MPa)**	**σ_s (MPa)**	**$\overline{\sigma}_s$ (MPa)**	**Verification**
Pool	Span	162,42	3,71	18	88	250	checked
	constraint	84,45	1,93		45,7		
Basement2	Span	165,85	3,99	18	104,3	250	checked
	constraint	86,07	2,29		71,1		

Table 3.4.6: SLS verification

- ***Shear force***

We must verify the shear stress $\tau_u \leq \overline{\tau}_u$

$\overline{\tau}_u$: allowable shear stress done by:

in our case damaged cracking: $\overline{\tau_u} = min\begin{cases} 0{,}15\frac{f_{cj}}{\gamma_c} \\ 4MPA \end{cases}$ $\tau_u = \frac{T}{b.d}$

(all details mentioned in appendix B)

We summarized all results in this table:

Storey	**Position**	**T_u^{max} (kN)**	**τ_u(MPa)**	**$\overline{\tau}_u$ (MPa)**	**Verification**
Pool	Span	368,47	1,02	3	checked
	constraint	368,626	1,02	3	checked
Basement2	Span	368,47	1,02	3	checked
	constraint	368,626	1,02	3	checked

Table 3.4.7: Shear force verification

- ***Deflection***

Total deflection: $\bar{f} \geq \Delta f_t = f_v - f_i$

when L> $5m \ldots\ldots\ldots\ldots \bar{f} = 0{,}5 + \frac{L(cm)}{100}$ in our case L=6,5m............ $\bar{f} = 11{,}5mm$

f_i: deflection due to instantaneous load.

f_v: Deflection due to deferred load.

All the expressions and the details to calculate the deflection are mentioned in appendix B so we summarized:

storey	Ms (KN.m)	As(cm²)	Y(mm)	σ_s (MPA)	δ	λ_i
Pool	162,42	29,45	45,5	76,59	0,008	2,93
	84,45	29,45	54,61	39,82	0,008	2,93
Basement2	165,85	25,13	41,3	91,66	0,006	3,43
	86,07	25,13	34,7	63,41	0,005	4,58

Table 3.4.8: Deflection's results

μ	I(cm⁴)	I_{fi}(cm⁴)	I_{fv}(cm⁴)	f_i (mm)	f_v(mm)	Δf_t(mm)	f(mm)	checking
0,14	702703,8	543416,5	661241,7	3,5	10,52	7,01	11,5	Checked
0,13	702703,8	1274600	917386,7	0,77	2,33	1,55	11,5	Checked
0,15	648792,4	467532,3	589525,8	4,16	12,4	8,33	11,5	Checked
0,12	567478,5	1486200	812789,7	0,68	2,04	1,36	11,5	checked

Table 3.4.9: Deflection's results

2) **Transverse reinforcement**

The grad of steel chosen of the transverse reinforcement is high adhesion with a grade of FeE500.
The transverse reinforcements are determined from the formulas of the BAEL91 99 and those of the RPA99 version 2003:

According to B.A.E.L99 $S_t = \min(0{,}9d\,;40cm)$ .....so we choose $S_t = 40cm$.

According to RPA99

$$\begin{cases} A_t = 0{,}003 S_t b \\ S_t \leq Min\left(\frac{h}{4};12\phi_l\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{ Nodal zone} \\ S_t \leq \frac{h}{2} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{ Curent zone} \end{cases}$$

With S_t: maximal value of spacing between the transverse reinforcement.

So, we summarized here:

storey	T_u(kN)	τ_u(MPa)	BAEL91	RPA99		S_t^{adp} (cm)		A_t (cm^2)	Choice
			S_t(cm)	S_t(cm)CZ	S_t(cm)NZ	NZ	CZ		
Pool	368,47 *368,626*	1,02	40	40	20	10	20	1,5	2T10
basement	368,47 *368,626*	1,02	40	40	20	10	20	1,5	2T10

Table 3.4.10: Transverse reinforcement's results

- ***Lapped length***

Lr = 50Ø……………..…....seismic zone III.
For: T25 ………………….……Lr = 125cm.
For: T20.......................................Lr = 100cm.

- ***Design beam section***

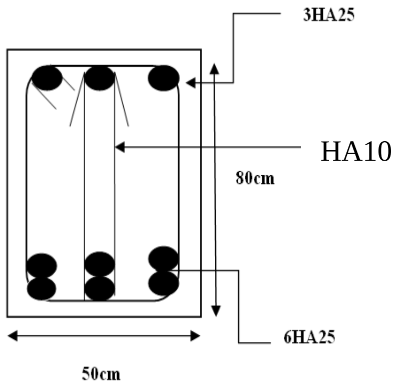


Figure 3.4.1: beam reinforcement (span)

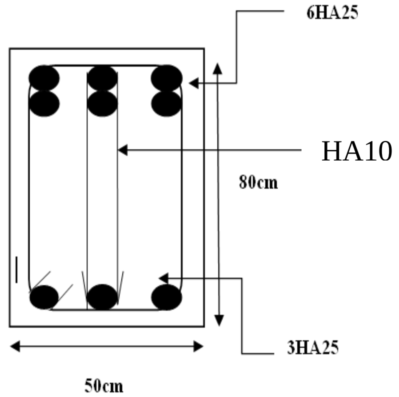


Figure3.4.2: beam reinforcement (constraint)

3.4.3 Column

Columns are subjected to compound bending (M, N), compression “N“, and bending moment “M , a section subjected to compound bending can be one of the following three cases:

Section entirely tensioned SET.

Section entirely compressed SEC.

Section partially compressed SPC.

The reinforcement sections of the following columns are calculated based on **SOCOTEC** software for accidental and persistent situations; the following tables summarized all the results

- **1, 35G+1,5Q**

side	N^{max}	M^{corr}	As	Asmin
75cm	1868,866	1,83	0	0
	N^{corre}	M^{max}	As	Asmin
75cm	129,31	152,43	0	3,3
95cm	486,26	303,25	0	4,2

Table 3.4.11: Reinforcement section ULS

- **G+Q± E**

side	N^{max}	M^{corr}	As	Asmin
75cm	1410,40	1679,71	0	57,64
	N^{corre}	M^{max}	As	Asmin
75cm	1325,179	1680,4648	0	58,62
95cm	129,31	152,43	0	2,96

Table 3.4.12: Reinforcement section G+Q± E

- **0.8 G ± E**

	N^{max}	M^{corr}	As	Asmin
75cm	1044,539	1421	0	48,81
95cm		41	0	0
	N^{corre}	M^{max}	As	Asmin
75cm	919,93	1679	0	63,18
95cm	350,415	2121,12	0	69,38

Table 3.4.13: Reinforcement section 0.8 G ± E

1) Longitudinal bars reinforcement

According to RPA's conditions: in zone III, the longitudinal bars must be in high adhesion respecting those conditions:

$0,9\% < \frac{A_s}{B} < 4\% \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \, curent\ zone$

$0,9\% < \frac{A_s}{B} < 6\% \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \, nodal\ zone$

B: concrete area section for column.

section	A_s^{cal} (cm^2)	A_s^{max} (CZ)(cm^2)	A_s^{max} (NZ)(cm^2)	Reinforcement choice	A_s^{chosen} (cm^2)
Side 95	69,38	285	427,5	8T25+8T25	78,54
Side 75	63,18			8T25+6T25	68,72

Table 3.4.14: Choice of reinforcement for column.

- ***Shear force***

We must verify the shear stress $\tau_u \leq \bar{\tau}_u$

$T_u(KN) = 711,45KN$ $\tau_u = \frac{T}{b.d}$ $\tau_u = 1,10MPA$

damaged cracking: $\overline{\tau_u} = min\begin{cases} 0,15\frac{f_{cj}}{\gamma_c} \\ 4MPA \end{cases}$ $so\ \ \tau_u = 3\ MPA \geq \overline{\tau_u} = 1,10MPA$**…….**checked.

- ***Service limited state***

$\sigma_{bc} \leq \bar{\sigma}_{bc}$ and $\sigma_s \leq \bar{\sigma}_s$

$\bar{\sigma}_{bc}$: Allowable concrete stress.

$\bar{\sigma}_s$: Allowable steel stress.

All the details are summarized in appendix B

Side	M_{ser} (KN.m)	σ_{bc} (MPa)	$\bar{\sigma}_{bc}$ (MPa)	σ_s (MPa)	$\bar{\sigma}_s$ (MPa)	Verification
75cm	111,15	1,15	18	22,4	250	checked
95cm	221,05	1,87		39,7		

Table 3.4.15: Verification SLS for column

2) **Transverse reinforcement of columns**

The transverse reinforcements are determined from the formulas of the BAEL91 99 and those of the RPA99 version 2003

according to BAEL91:

$$\begin{cases} S_t \leq min(0{,}9d; 40cm) \\ \varphi_t \leq min(^h/_{35}; ^b/_{10}; \varphi_l) \\ \dfrac{At \times f_e}{bS_t} \geq max(^{\tau_u}/_2; 0{,}4MPA) \end{cases}$$

According to RPA99:

$$\frac{A_t}{S_t} = \frac{p_a \times T_u}{h \times f_e}$$

Symbol	Name	values
p_a	Corrective coefficient	ρ_a=2,5………………if $\lambda_g \geq 5$ ρ_a=3,75……………...if $\lambda_g < 5$
λ_g	Geometric slenderness	$\lambda_g = L_b / a$
S_t	Spacing between transverse bars	$S_t = 10cm$ … … … … … ….. . nodal zone III $S_t = \min\left(\frac{b}{2}; \frac{h}{2}; 10\varphi_l\right)$……curent zone III

Table 3.4.16: transverse reinforcement parameters

φ_l: Longitudinal diameter of bars.

L_b: Length of buckling.

a: column's side**.**

section	Section (cm^2)	Bars	$Ø_l$ (mm)	S_t (cm)	
				Nodal zone	Current zone
95*75	78,54	8T25+8T25	25	10	25
95*75	68,72	8T25+6T25	25	10	25

Table 3.4.17: Calculation of St

Side (cm)	Section (cm^2)	L_f (m)	λ_g (%)	ρ_a	T_u^{max} (kN)	Zone	S_t (cm)	A_t^{cal} (cm^2)	Choice	A_s^{adp} (cm^2)
95	78,54	1,98	2,08	3.75	740,20	N	10	1,40	8T10	5,03
						C	15	2,1	8T10	5,03
75	68,72	1,98	2,08	3.75	711,4	N	10	1,34	8T10	5,03
						C	15	2,02	8T10	5,03

Table 3.4.18: Choice of transversal reinforcement for column

- ***Lapped length***

Lr = 50Ø……………..seismic zone III.

For: T25 …………………Lr = 125cm.

- ***Design column section***

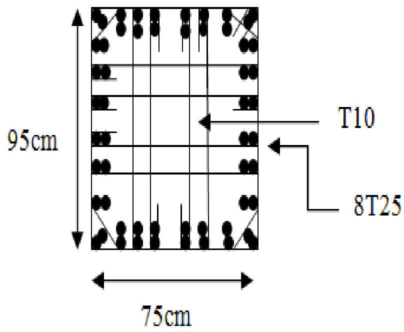


Figure 3.4.3 column reinforcement design

3.4.4 Wall

The calculation of the walls can be done by several methods where each one has advantages and characteristics, among these methods the ACI 318 (American regulation), which considers the wall-column as an element its shape (I, U…) subjected to axial load (P) , a shearing force (V) and a bending moment (M).

According to the regulations it is appropriate that:

- The horizontal and vertical reinforcements must provide resistance to the shearing force.

- The vertical reinforcements in the edge elements (confinement zone) at the two ends of the cross section of the wall must provide the resistance of the compound bending.

The reinforced concrete wall must be subject to:

- Resistance in buckling.

- Resistance of axial load.

- Resistance of shearing force.

- Compound bending.

All the steps and reference of this method are mentioned in Appendix C

- ***Resistance in buckling***

It must be checked: $Pu \leq \Phi Pn$ with Φ=0, 70

$$P_n = 0{,}55 \cdot f_{bc} \cdot A_g \cdot \left[1 - \left(\frac{k \cdot h_s}{32a}\right)^2\right]$$

$\mathrm{P} = 266{,}45\mathrm{KN}$ $A_g = (0{,}3 \times 4{,}4) + 0{,}3^2 = 1{,}41m^2$ k=1.00 a=30cm

$\mathrm{P} = 266{,}45\mathrm{KN} \leq 0{,}7 \times 13012{,}34 = 9108{,}64\mathrm{KN}$ ……………………………..……..checked.

- ***Resistance of shearing force***

It must be checked: $V_u < 0{,}664 \cdot A_{cv} \cdot \sqrt{f_{bc}}$

Vu = 124, 09 KN $A_{cv} = 4{,}4 \times 0{,}3 = 1{,}32m^2$ so Vu$< 3718{,}58KN$……checked.

Minimum percent of the horizontal and vertical reinforcement of the web wall

$\rho_v = \rho_h = \rho_n \geq 0{,}0025$

Maximal spacing: $S_{max} \leq$ min (3a; 45cm) = min (90cm, 45cm) =45cm

$A_{s.min} = 0{,}0025 \times 30 \times 100 = 7{,}5cm^2$.

If we choose section of HA12, in the two faces we will have: $2\times 1{,}13 = 2{,}26cm^2$

$\frac{2{,}26}{7{,}5} \times 100 = 30{,}13cm < 45cm \ldots\ldots\ldots\ldots..\ldots checked.$

- ***The necessary Web reinforcements for shearing force***

We opt for two table of reinforcement in HA12 with a spacing s = 25 cm

It must be checked: $\Phi V_n > V_u$

$\Phi = 0{,}75$ $V_n = A_{cv}\left(\alpha_c \cdot 0{,}083 \cdot \sqrt{f_{bc}} + \rho_n f_y\right)$

$\frac{h_w}{L_w} \leq \frac{2{,}5}{4{,}4} = 0{,}56 \leq 2 \ldots\ldots\ldots\ldots\ldots\ldots..\alpha_c = 3$

For a distance of 100 cm we have 2 × 5HA12= 11, 3 cm^2

$\frac{11{,}3}{100\times 30} = 0{,}00452$ $\emptyset V_n = (1{,}32 \times 10^6)(3 \times 0{,}083 \times \sqrt{18} + 0{,}00452 \times 500)$

$= 3227{,}11KN > \mathrm{Vu} = 124{,}09\ \mathrm{KN}$ …………………checked.

The shearing force is checked by 02 vertical and horizontal HA12 tables with spacing s = 25cm (lattice of HA12 and 20 × 20 cm²), distributed on each face of the web wall connected by pins.

- ***Reinforcement in compound bending***

M_u = 74, 34 KN.m P_u = 266, 45 KN

$\frac{P_u}{P_0} \leq 0{,}35$ $\quad P_0 = 0.85 f_{bc}(A_g - A_s) + A_s f_e$ $\quad A_s = 0{,}0025 \times 30 \times 440 = 33cm^2$

$P_0 = 0{,}85 \times 18[(1{,}32 \times 10^6) - 3300] + (3300 \times 500) = 561247{,}5KN$

$\frac{266{,}45}{561247{,}5} = 0{,}0004 < 0{,}35$………………..checked

with **SOCOTEC** program we calculate the necessary section of reinforcement for a rectangular section in compound bending : A_s=3,01cm².

- ***Dimensioning the edge elements***

We build the edge elements when $\frac{P_u}{A_g} \geq 0{,}2 \times f_{bc}$

$\frac{P_u}{A_g} = \frac{266{,}45 \times 10^3}{1{,}32 \times 10^6} = 0{,}2MPA \leq 0{,}2 \times 18 = 3{,}6MPA$

so it's not necessary to add the edge elements.

- ***RPA's conditions check***

$A_{min} \geq 0{,}15\%b.h = 0{,}15\% \times 5 \times 0{,}3 = 22{,}5cm^2$ = 4,5 cm² /ml. So

$A_{min} = 4{,}52\ cm^2$ …..4HA12.

$s \leq \min(1{,}5a\,;30cm) = \min(45cm\,;30cm) = 30cm.$

- ***Design***

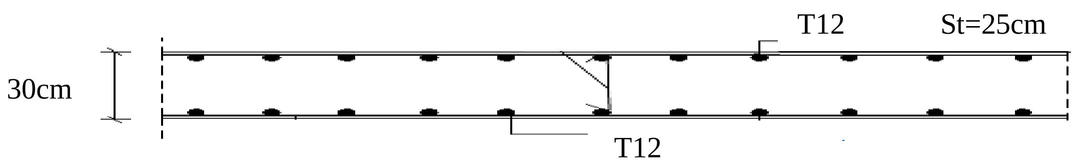


Figure 3.4.4 wall design reinforcement

3.4.5 Conclusion

Through this stage we've seen how to deal with the reinforcement in concrete (longitudinal and transverse bars) as we have seen the importance of following the RPA; every section has its maximum and minimum reinforcement and spacing values to take in consideration while doing the design.

3.5 FOUNDATION DESIGN

3.5.1. Introduction

Foundation are classified as shallow and deep foundation:

1.shallow foundation it might be:

-Individual or isolated footing.

-Combined footing.

-Strip foundation.

-Raft or mat foundation.

2. Deep foundation it might be:

-Pile foundation

-Drilled shaft or caissons

3.5.2 Isolated footing

most common type of foundation used for building construction. It constructed for a single column and also called a pad foundation.

$\frac{a}{b} = \frac{A}{B}$ We have a rectangular column $S \geq \frac{N}{\sigma_{sol}}$ $\sigma_{ground} = 1{,}8bar$

(a, b): columns dimensions.
(A, B): isolated footing dimensions.
S: pad footing area.
N: axial force (KN).
σ_{ground} : Allowable ground stress.

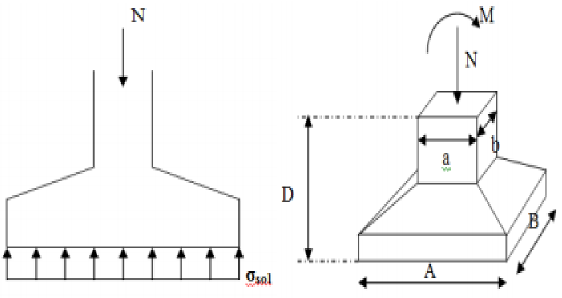


Figure 3.5.1: Isolated footing

We calculate the pad footing dimensions based on the previous expressions for all the columns we have in our infrastructure, we summarize:

line	N (KN)	S(m²)	A(m)	B(m)	A(m)chosen	B(m) chosen
1	452,19	2,51	1,58	1,58	2	2
	503,63	2,79	1,67	1,67	2	2
	521,15	2,89	1,7	1,7	2	2
	505,88	2,81	1,67	1,67	2	2
	485,94	2,69	1,64	1,64	2	2
2	1191,172	6,61	2,91	2,29	3	2,5
	1229,82	6,83	2,95	2,33	3	2,5
	1002,46	5,56	2,67	2,1	3	2,5
	991,01	5,50	2,65	2,09	3	2,5
	1173,59	6,51	2,89	2,28	3	2,5
	1134,16	6,30	2,84	2,24	3	2,5
3	1030,4	5,72	2,7	2,13	3	2,5
	1868,68	10,38	3,64	2,88	4	3
	1864,93	***10,36***	***3,64***	***2,87***	***4***	***3***
	1865,32	10,36	3,64	2,87	4	3
	1865,72	10,36	3,64	2,87	4	3
	929,552	5,16	2,57	2,03	3	2,5
4	1191,13	6,61	2,91	2,29	3	2,5
	1229,82	6,83	2,95	2,33	3	2,5
	1002,4	5,56	2,67	2,10	3	2,5
	991,01	5,50	2,65	2,09	3	2,5
	1173,05	6,51	2,89	2,28	3	2,5
	1134,15	6,30	2,84	2,24	3	2,5
5	469,23	2,6	1,61	1,61	2	2
	547,22	3,04	1,74	1,74	2	2
	572,29	3,17	1,78	1,78	2	2
	555,26	3,08	1,75	1,75	2	2
	511,72	2,84	1,68	1,68	2	2

Table 3.5.1: Axial loads applied on isolated footings

Example:

N = 1864, 93KN , S = $\frac{N}{\sigma_{ground}} = \frac{1864{,}93}{180} = 10{,}36\ m^2$, A.B= 10,36m² , $\frac{B}{A} = \frac{0{,}75}{0{,}95} = 0{,}78$

A ≥ 3,64 we choose: A= 4m and B ≥ 2,87 we choose: B = 3m.

We must check L > 1,5B so $(2+3)\times\frac{1{,}5}{2}=3{,}75m\ > L_{min}=3{,}34m$ ……..not checked.

After these results, we notice that there is an overlap in footing; we pass directly to another type of foundation (strip footing).

3.5.3 Strip footing

The axial force supported by the strip footing is the sum of the axial forces of all the columns which are located the same line.

We must check that: $S \geq \frac{N}{\sigma_{ground}} \dots\dots\dots\dots\dots\dots\dots S = B \times L$

L = total length of the considered line. B = width of the footing.

All the results are summarized in this following table:

line	N (KN)	L(m)	S(m²)	B(m)	B (m) chosen
1	2468,79	40,03	13,71	0,34	1
2	6722,212	25,75	37,34	1,45	1,5
3	9424,606	25,75	52,35	2,03	2,5
4	6722,01	25,75	37,34	1,45	1,5
5	2655,52	40,03	14,75	0,36	1

Table 3.5.2: strip footing's section

Example:

N = 9424, 606KN, S = $\frac{N}{\sigma_{ground}}=\frac{9424{,}606}{180}=52{,}35\ m^2$, $B \geq \frac{S}{L}=\frac{52{,}35}{25{,}75}=2{,}03m$

B chosen=2,5m l=6,5m

we must check L > 2,5B so $(2{,}5+1{,}5)\times\frac{2{,}5}{2}=5m\ < L=6{,}5m$ …….. Checked.

We must check $\frac{A_S}{A_t} \leq 50\%$ $\frac{176{,}93}{775{,}98}\times 100=22{,}80\% \leq 50\% \dots\dots\dots\dots$ Checked.

- ***Condition of rigidity***

$d \geq \frac{B-b}{4}$ $h_t \geq \frac{B-b}{4}+5$ (cm) $\frac{L_{max}}{9} \leq h \leq \frac{L_{max}}{6}$.

h_t : footing's height

h : release beam's height $55{,}55 \leq h \leq 83{,}33cm$ we choose $h=70cm$

B (cm)	b	d (cm)	d chosen	h_t (cm)
100	60	10	10	15
150	95	13,75	15	20
250	95	38,75	40	45

Table 3.5.3: footing's height

- ***Service limited state***

We calculate the eccentricity of the third line $e = \frac{M_S}{N_S} = \frac{1063{,}04}{6275{,}149}$ =0,16m=16cm

Bearing capacity condition

It must be verified: $\sigma_{ground} \geq \sigma = \frac{N_s}{S}$

Ns (KN)	S (m²)	σ (bar)	σ_{ground}	verification
6275,149	64,375	0,97	1,8	checked

Table 3.5.4: bearing capacity verification

- ***ULS (Reinforcement)***

The main reinforcement's Ab, arranged parallel to the width B, will be calculated by the (BIELLE) method, neglecting the self-weight of the footing.

$$A_b = \frac{N_U\left(1+\frac{3e_u}{L}\right)(B-b)}{8.d.\sigma_{st}} \qquad e_u = \frac{M_u}{N_u} = \frac{1{,}35\times1063{,}04}{1{,}35\times6275{,}149} = 0{,}16m = 16cm$$

$$A_b = \frac{8471{,}45\left(1+\frac{3\times16}{2575}\right)(250-95)}{8\times40\times43{,}47} = 96{,}15cm^2$$ We suppose 5T10 /ml.

The Distribution reinforcement Ar are arranged along the length.

$$A_r = \frac{A_b}{4} \times B = \frac{3{,}93}{4} \times 2{,}5 = 2{,}45cm^2.$$

These frames will distribute over the wings of the footing.

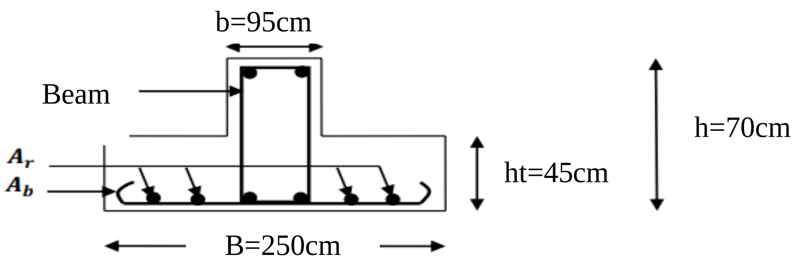


Figure 3.5.2: Strip footing's design

CONCLUSION

'Engineering is the art of directing the great sources of power in nature for the use and convenience of man' THOMAS TREDGOLD

Through this project we discovered that being in this field is like an adventure, each and every project has deferent details and problems from other ones, it challenges every things and this project helped us to see just a little deference.

it started in December with some basic knowledge then it continued till August and the result is analysis of touristic camp with multi-blocs and suspended swimming pool , what is good in our project in other meaning what we really feel proud of that we have explored the both sides of technology (research and application) through this project we have been able to analyze and study three deferent types of structure (steel building , hangar and reinforced concrete infrastructure) we also did a detailed research about one of the most dangerous and important interaction that can make a huge damages (interaction fluid-pool) for a semi Olympic suspended swimming pool with an area of 325m² it really helped us to learn how to make research, how to read technical articles ,how to evaluate the behavior of such a thing and how to think and give probabilities of the problem posed and finally how to solve it .

Also through our project we worked with (ROBOT and SAP2000) it was a very challenging to learn the both but we didn't give up and everything was done successfully , about the seismic analysis we're feeling more proud because we've worked with the two functions (response spectrum and time history function) now we know that response spectrum is derived from the time history also for the dynamic of structure we learned how to analyze our structure in seismic state (steel building , pool) as we tried the analyze in climatic state (hangar) and finally our biggest challenge was to do this simple new born project in English that pushed us to explore the Eurocodes files in its English version.

Our project isn't make yet in reality till now they have just gave the idea with simple architectural plan with so many mistakes and so many difficulties we've faced, during the earthquake analysis (we found many openings in areas where we must put our bracing system) but that doesn't mean we change the idea we must find a solution for it and that what we did for example instead of adding bracing type X we have respected the architectural plan and we have added bracing type V where it can be allowable . In general this project is just only our first step to the professional life where we will discover the civil engineering field in real.

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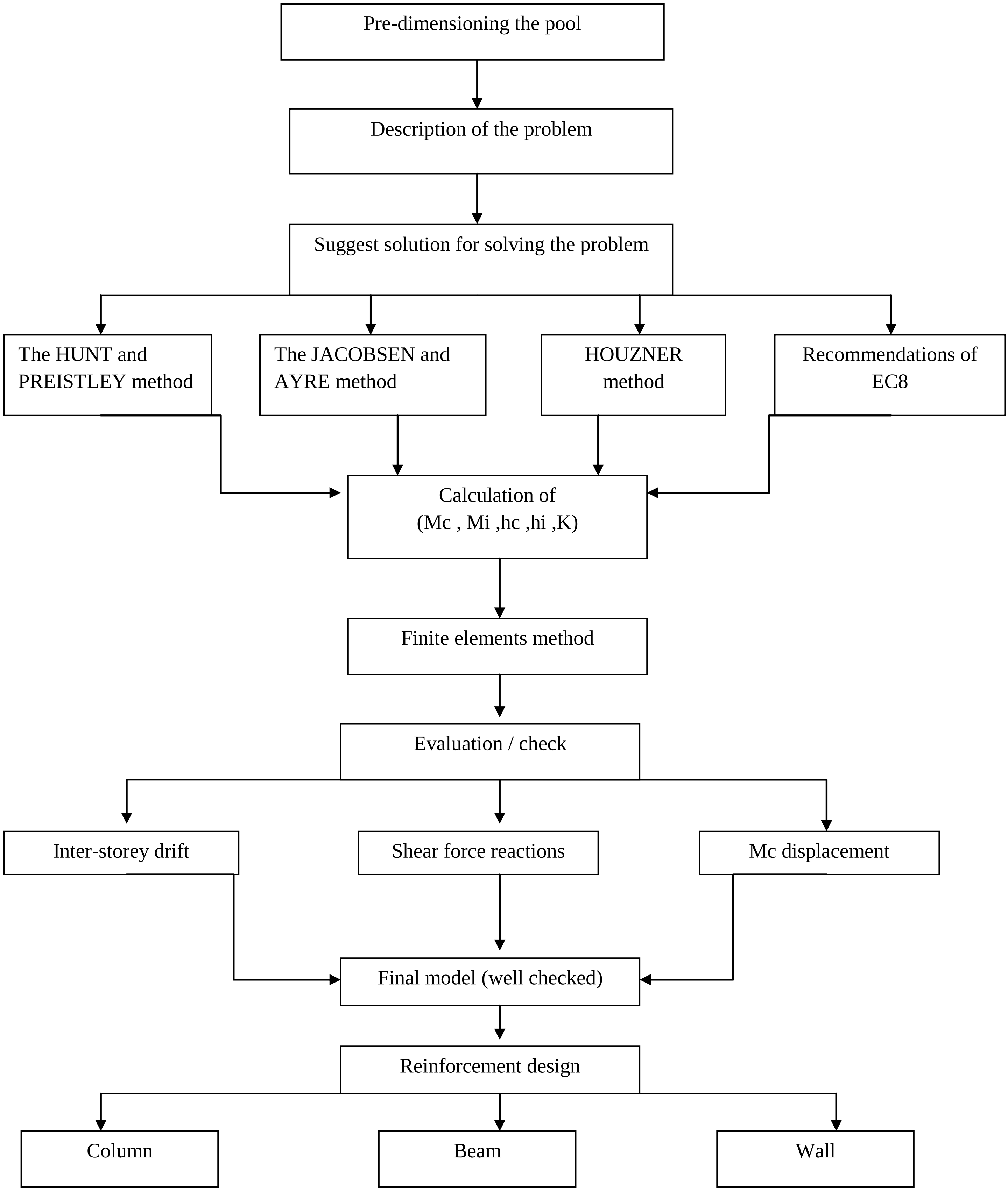
APPENDIX A


Pre-dimensioning the pool
Description of the problem
Suggest solution for solving the problem
The HUNT and PREISTLEY method
The JACOBSEN and AYRE method
HOUZNER method
Recommendations of EC8
Calculation of (Mc , Mi ,hc ,hi ,K)
Finite elements method
Evaluation / check
Inter-storey drift
Shear force reactions
Mc displacement
Final model (well checked)
Reinforcement design
Column
Beam
Wall

APPENDIX B

1) Shear force

q_u ; V_u ; d ; $\gamma_b = 1.5$; $\gamma_s = 1.15$; f_e ; f_{cj} ; $f_{tj} = 0.6 + 0.06 f_{cj}$; α ;

$$\bar{b} = \begin{cases} b & \text{section rectangulaire} \\ b_0 & \text{section en } T \end{cases}$$

$$\overline{\tau_u} = \begin{cases} min\left\{0.20\dfrac{f_{cj}}{\gamma_b}; 5\,MPa\right\} & pour\ FPP \\ min\left\{0.15\dfrac{f_{cj}}{\gamma_b}; 4\,MPa\right\} & pour\ FP\ ou\ FTP \end{cases}$$

$$\tau_u = \frac{V_u}{\bar{b}d}$$

$\tau_u < \overline{\tau_u}$

non: Redimensionner la section

oui:

$$\emptyset_t = \frac{\emptyset_{lmax}}{3} \quad ; \quad \emptyset_t \le min\left\{ \emptyset_{lmin} \ , \ \frac{h}{35}, \ \frac{\bar{b}}{10}\right\}$$

$$A_t = n \times \frac{\pi {\emptyset_t}^2}{4} \ avec$$ Avec n : Nombre de Brins

2) Service limited state

$$\overline{\sigma_{st}} = \begin{cases} f_e & pour\ \ FPP \\ min\left\{\frac{2}{3}fe\ ,\ \ max(\frac{fe}{2}, 110\sqrt{1.6 f_{t28}}\ )\right\} & pour\ \ FP \\ \min\left\{\frac{1}{2}\text{fe}\ ,\ \ 90\sqrt{\eta f_{t28}}\ )\right\} & pour\ \ FTP \end{cases}$$

y Solution positive de :

$$b\,y^2 + 30(A_s + A_s')y - 30(A_s\,d + A_s'\,d') = 0$$

$$I = \frac{1}{3}by^3 + 15As'(y - d')^2 + 15As(d - y)^2$$

$$k = \frac{Mser}{I} \quad ; \quad \sigma_{bc} = k\,y \quad ; \quad \sigma_{st} = 15\,k(d - y) \quad ; \quad \sigma_{sc} = 15K(y - d')$$

3) Recommendations for column

For columns located in zone III:
minimal diameter 12mm.

$$* \ 0{,}9 \langle \frac{A_s}{B} \langle 4\%$$ ....current zone

$$* \ 0{,}9 \langle \frac{A_s}{B} \langle 6\%$$ ....nodal zone

4) Deflection

$$f_v = \frac{M_{ser} l^2}{10 E_v I_{fv}} \qquad \Delta f_T = f_v - f_i \leq \bar{f} \qquad f_i = \frac{M_{ser} l^2}{10 E_i I_{fi}}$$

Position of the neutral axis determined by:

$$b y_1^2 + 30.(A_s + A_s').y_1 - 30.(d.A_s + d' A_s') = 0$$

$$\begin{cases} I_{fi} = \dfrac{1{,}1 I_0}{1 + \lambda_i \mu} \\ I_{fv} = \dfrac{1{,}1 I_0}{1 + 0{,}4 \lambda_i \mu} \end{cases} \quad I_0 = \frac{b y_1^3}{12} + 15\left[A_s (d - y_1)^2 + A_s' (y_1 - d')^2\right]$$

APPENDIX C

Design of Reinforced concrete wall by AC318

1. Resistance in buckling

$k = \frac{l_f}{l}$ K: buckling coefficient.

The values of $k = \frac{l_f}{l}$ are done in the following table:

Wall binding		Vertically reinforced Wall	Vertically non Reinforced wall
		k	
Restrained wall (top , bottom)	Floor of both sides	0.80	0.85
	Floor on one side	0.85	0.90
Hinged wall (top , bottom)		1.00	1.00

2. Minimum percent of the horizontal and vertical reinforcement

	Horizontal reinforcement	Vertical reinforcement
Maximal spacing between the axis	≤min (*lw*/5 ;3a ; 45cm)	≤min (*lw*/ 3 ;3a ; 45cm)
Minimal reinforcement Minimal percent	Ash ≥ $\rho_h.100.a$ $\rho_h \geq 0.0025$	Asv ≥ $\rho_v.lw.a$ $\rho_v = 0.0025 + 0.5\left(2.5 - \frac{h_w}{l_w}\right)(\rho_h - 0.0025) \geq 0.0025$

3. Resistance of shearing force

Shear force values	Horizontal and vertical reinforcement percent	checking
$Vu > 0.166 \times A_{cv} \times \sqrt{f_{bc}}$ Ou a≥25 cm	$\rho_v = \rho_h = \rho_n \geq 0.0025$ 2 tables disposed at each face of the wall ,by (pins) spacing S≤ min (3a ; 45cm)	$V_u < 0{,}664.A_{cv}.\sqrt{f_{bc}}$ $\Phi V_n > V_u$ $\Phi = 0{,}75$ $V_u = A_{cv}.\left(a_c.0{,}083.\sqrt{f_{bc}} + \rho_n.f_y\right)$ $a_c = 3 \quad pour : \frac{h_w}{l_w} \leq 1{,}5$ $a_c = 2 \quad pour : \frac{h_w}{l_w} \leq 2$

$Vu > 0{,}083 \times A_{cv} \times \sqrt{f_{bc}}$ a<25 cm	Vertical reinforcement - $\rho_v \geq 0{,}0012$ with HA16 - $\rho_v \geq 0{,}0015$ other HA Horizontal reinforcement - $\rho_h \geq 0{,}002$ with HA16 - $\rho_h \geq 0{,}0025$ with HA spacing S≤ min (3a ; 45cm)	$\Phi V_n > V_u$ $V_n = A_{cv}\left(a_c .0{,}083.\sqrt{f_{bc}} + \rho_n . f_y\right)$

Pu: axial force obtained in compression from the unfavorable combination.

Pn : nominal resisted axial force of the transversal section of the wall.

Φ : reduction's factor

A_g: brute section of the wall.

e : wall's thickness.

ρ : Percent of the reinforcement (vertical or horizontal).

Vu: shear force.

Vn : nominal resisted shear force of the transversal section.

l_w : Length in plan of the wall.

DICTIONARY ENGLISH-FRENSH

Shear force : effort tranchant
Bending moment : moment fléchissant
Load : charge
ULS : ELU
SLS : ELS
Displacement : déplacement
Bolts : boulons
Weld : soudure
Gusset : gousset
Footing : semelle
Grad of steel : nuance d’acier
Length of buckling : longueur de flambement
Lateral buckling : déversement
Deflection : la flèche
Damping : amortissement
Slenderness : élancement
Ground floor : Rez-de-chaussée
Storey : étage
Basement : sous sol
Pre-loaded bolts : boulons pre contraints
Bracing system : contreventement
Staircase : escalier
Live load : charge d’exploitation
Constraint : appuis
Span : travée
Inter-storey drift : déplacement inter étage
Column : Poteau
Beam : pouter
Wall : paroi , voile
Slab : dalle
Floor : plancher
Compound bending : flexion compose
Tensile : traction
Stiffeners : raidisseurs
Web : âme
Flange : semelle
Joist : solive
Base column : base Poteau
Duopitch roof : toiture à deux versants
Ground : sol
Roof : toiture
Purlins : pannes
Rafters : traverses
Slope : pente
Plate : platine