SAAD DAHLEB BLIDA 01 UNIVERSITY

Faculty of civil engineering

Civil engineering department

MASTER’S DISSERTATION IN CIVIL ENGINEERING

Option: Steel and composite structures

DESIGN OF MULTI STOREY CAR PARKING OF 6FLOORS+2BASEMENTS

Taking into account the soil-structure interaction

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ملخـــــــص

في إطار مشروع نهاية الدراسة , اخترنا كمثال للدراسة مبنى غير منتظم الشكل يستخدم ك (موقف سيارات) تتكون البنية العلوية من طابق أرضي و 6 طوابق ذات هيكل معدني و تتكون البنية التحتية من طابقين تحت أرضيين من الخرسانة المسلحة و هذا مع الأخذ بعين الاعتبار التفاعل بين الأرض و البنية , و سيتم مقارنة نتائج النموذجين (مع و بدون) تفاعل بين التربة و المبنى لتحديد أهمية هذا الأخير و أخذه بعين الاعتبار.

يقع المبنى في منطقة زلزالية عالية وتقع على تربة ضعيفة. تم تحديد أبعاد العناصر المشكلة للهيكل وفقًا للوائح الجزائرية المعمول بها (CCM97، RPA99) وكذلك اللوائح الأوروبية.

Résumé

Dans le cadre de notre projet de fin d'études, on a choisi comme exemple d'étude, un bâtiment de forme irrégulière à usage "Parking" ; dont la superstructure (RDC+6 étages) faite en charpente métallique et mixte ; et l'infrastructure (2 sous-sols) en béton armé. Et cela en tenant en compte l'interaction sol-structure. Les résultats des deux modèles (avec et sans interaction) seront comparés pour déterminer l'importance de la prise en compte de cette dernière.

Le bâtiment est implanté dans une zone à forte sismicité et implantée sur un sol faible. Le dimensionnement des éléments structuraux a été effectué conformément à la règlementation algérienne en vigueur (**CCM97 ; RPA99..)** ainsi que les aux règlements européens.

Abstract

As part of our project, we chose as a study example, an irregularly shaped building for "Parking" use; such as: the superstructure (6 floors) is made of a steel and composite frame work; and the infrastructure (2 basements) in reinforced concrete. And this by taking into account the soil-structure interaction. The results of both of the models (with and without interaction) will be compared to determine the importance of taking the SSI into account.

The building is located in an area with high seismicity and implanted on weak soil. The design of the structural elements was carried out in accordance with the Algerian regulations **(CCM97; RPA99 ...)** as well as European regulations.

Dedications and thanks

Before thanking anybody, I send my most sincere feelings of gratitude and thanks to ALLAH the almighty, the merciful who has given me the strength and courage to carry on this modest work.

And then;

2020 is a year to be remembered, it has been hard for everyone; so I must say that it was very hard to find balance between maintaining a healthy mental condition and keeping up the progress of my final year's project. Seeing where I was and where I am now makes me really proud, but none of this would've been possible if I didn't have people to lead or support me:

Thank you to my family I dedicate this work to all of you; My mother Mekarzia Leila , the one who always believed in me , supported me even in my darkest days , and pushed me to go forward ; My father Abdelkader , my role model and the reason I've chosen this path in the first place ; My sister, Nada and My brother Salah eddine;

To Benaziba Myria, who has been more than a friend to me, who believed in me and pushed me forward;

To my guardian angel; no words will be enough to thank you for everything that you have done; through the best and worst of this journey; I dedicate this to you.

I express my deep gratitude to my mentor, Mr. Derbal Ismail for the time he gave me, the scientific assistance, as well as his valuable advice;

And lastly I would like to give a special THANK YOU to MYSELF and thank myself for keeping up the work through hard times.

And for all the future engineers reading this;

"You're not going to get very far in life based on what you already know. You're going to advance in life by what you're going to learn after you leave here."

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LIST OF SYMBOLS

E : modulus of elasticity / young’s modulus

G : shear modulus

ν : Poisson’s ratio.

M_{sd} : soliciting moment

$M_{pl,Rd}$: plastic resistant moment .

$M_{el,Rd}$: elastic resistant moment .

V_{sd} : shear force

$V_{pl,Rd}$: plastic transverse shear force

$V_{el,Rd}$: elastic shear force

I_y : moment of inertia according to axis y

I_z moment of inertia according to axis x

W_{el} : elastic resistant modulus .

W_{pl} : plastic resistant modulus

t : thickness .

t_w : web thickness

t_f: flange thickness

h : height of the section

b : width of the section

A : area of the section

d : Hauteur de la portion droite de l’âme.

G : weight per meter

A_v : shearing area

L : length in general

l_f : buckling length

N : normal force

f_y : steel’s elastic limit

f : beam sag (deflection).

γ : safety factor .

λ : slenderness

$\bar{\lambda}$: reduced slenderness

e :.steel’s Elastic reduction coefficent

χ : buckling reduction coefficent

σ : normal stress

h : height of the steel beam

t : thickness of the concreate slab

b : width of the slab

g: acceleration of gravity.

A: zone acceleration coefficient.

η: damping correction factor.

A: Coefficient of behavior of the structure.

T: Period.

Q: Quality factor.

Vt: base shear.

W: dynamic weight of the structure.

Δ_k^i Horizontal displacement of two adjacent floors in a building in direction i.

δ_{ek}^i : is the horizontal displacement due to seismic forces at level K in direction i.

GENERAL INTRODUCTION

This project is the result of 5 years of university study related to the field of civil engineering, specializing in metal and composite construction, it consists of the study of a building with 6 floors and 2 basements taking into account the soil-structure interaction which is a phenomenon very often neglected in the design of structures in Algeria, so this project will consist in underlining the importance of the consideration of the latter in and its influence on the response of the structure to dynamic stresses.

The building superstructure consists of a metal frame stabilized against lateral loads using X-shaped bracing; and composite slab. As for the infrastructure, it consists of composite columns, a peripheral RC wall and a strap footing

This work was organized in the following way which after the introduction we have:

In chapter 1; we gave general information about the building characteristics; the geographic situation; the used materials properties; and the chosen structural system detailing.

In chapter 2; we made an evaluation of the loads acting on our structure (permanent loads , live loads and climatic loads such as snow and wing); using the Algerian regulations (DTR-BC-2.2 and RNV2013);

In chapter 3; we preliminarily designed the structural elements so that they can be used for the initial evaluation of the dynamic response of the structure under seismic loading, the primary design was done using (CCM97, EUROCODE 3, 4, and empirical expressions);

In chapter 4; we used the software ROBOT STRUCTURAL ANALYSIS for the seismic design of our structure based on the Algerian seismic regulation; we then did all the necessary checks imposed by the RPA (base shear, inter-storey drift);

In chapter 5; we verified the structural elements and made sure they meet all the resistance and stability conditions imposed by the design regulations.

In chapter 6; the connections joining the structural elements were designed according to: CMM97; RPA99V2003; and EUROCODE3. Some connections where designed using Robot structural analysis;

In chapter 7; the infrastructure is designed (Basement and footing)

In chapter 8; we explained what soil-structure interaction is, as well as its importance and why it is necessary to take it into account, we then proceeded to the application of the research on our designed structure, using the software SAP2000 and compared results obtained by the model where we did not consider the soil-structure interaction, and the one where we took the latter into account.

Finally, we end with a conclusion that summarizes the essence of the work done.

CHAPTER 01 :

General informations.

1. GENERAL INFORMATIONS

1.1. Presentation of the study project

This project concerns the study of a steel-framed open deck parking composed of 6storys + 2 basements, taking into account the soil-structure interaction. The structure will be built in the city of Hadjout (Wilaya of TIPAZA) which is an area with high seismicity (ZONE III).

1.1.1. Geometric characteristics

- **Elevation dimensions :**
  - Total height …………………………………………… 22.4m
  - Basement height ……………………………………… 2.8m
  - Story height ……………………………………………2.8m
- **Plan dimensions :**
  - Total length …………………………………………………64.68m
  - Total width……………………………………………………..37m

1.1.2. Data concerning the site of implantation

- Altitude: 98m.
- Snow zone: Zone B.
- Wind zone: Zone I.
- Seismicity zone: Zone III (region of high seismicity)

1.2. Structural characteristics

1.2.1. Frame work of the structure

We opt for a framework where the vertical forces acting on the roof and the floors are transmitted to the foundations by bending of the beams and compression of the columns; and the horizontal forces are transmitted to the foundations by the vertical bracing system

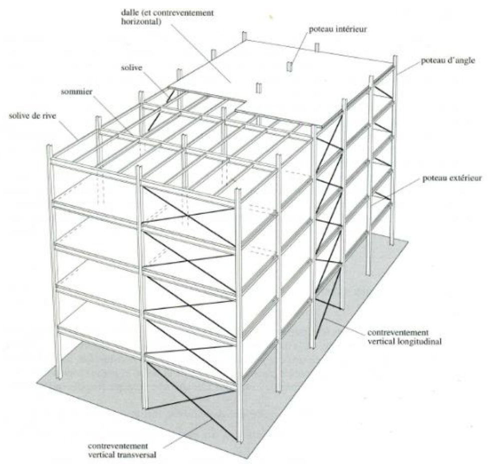


Figure 1. 1 : elements of a multi-storey building's frame work.

1.2.2. Floors

For the floor we chose a composite floor (concrete-steel) .It is constructed of slabs and beams acting compositely together. Composite floor are composed of a concrete topping cast onto metal decking.

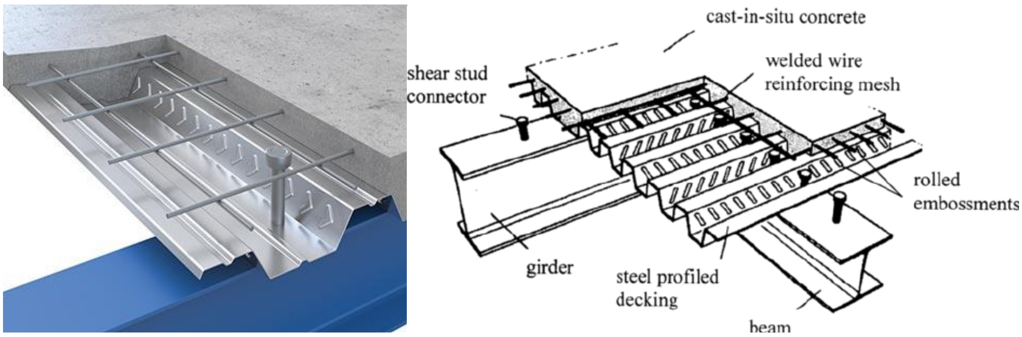


Figure 1. 2: composition of a composite slab.

The composite floor is a structural element which has many advantages:

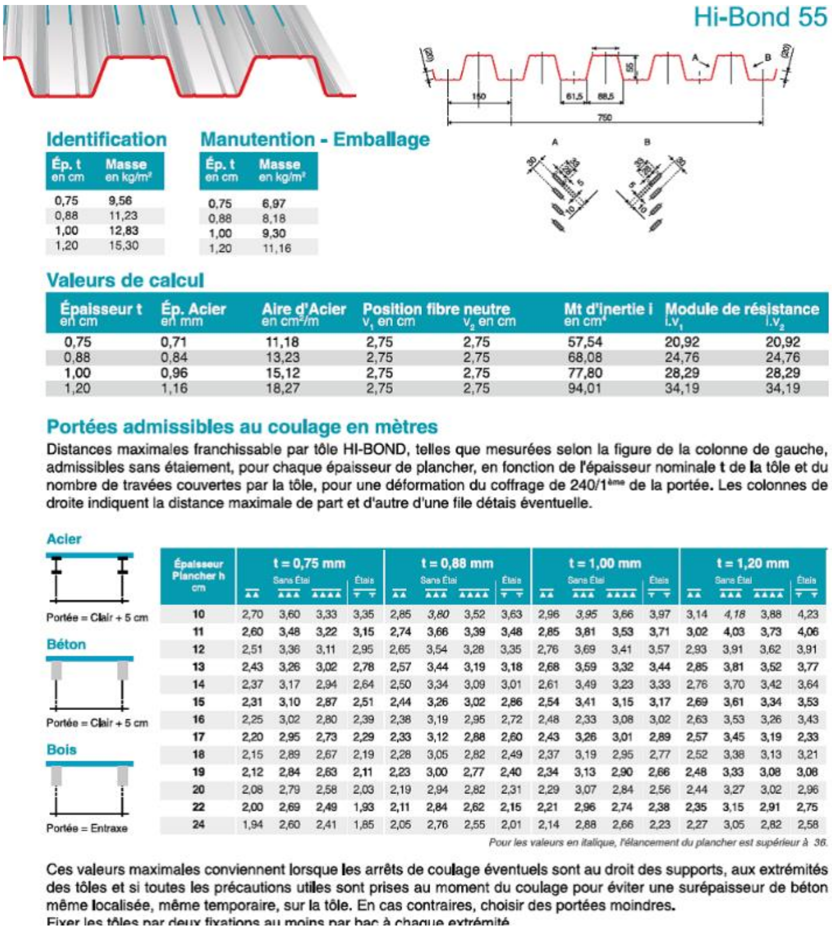
- A gain on the total weight of the structure coming from smaller dimensions.
- Greater flexural rigidity (Small deflection).
- A reduction in the height of the structural floor and, therefore, an increase in the useful height of each story.

Profiled steel decking, this element's function is:

- Ensuring efficient and watertight formwork by eliminating formwork removal operations.
- Constitute a work platform before the concrete is implemented.
- Avoiding often the installation of props.

The connection between the slab, the profiled steel decking and the supporting structure is ensured by **connecting studs.**

In our case, we use a Hi-Bond 55 with the following characteristics:



Identification

Ép. t en cm	Masse en kg/m²
0,75	9,56
0,88	11,23
1,00	12,83
1,20	15,30

Manutention - Emballage

Ép. t en cm	Masse en kg/m²
0,75	6,97
0,88	8,18
1,00	9,30
1,20	11,16

Valeurs de calcul

Épaisseur t en cm	Ép. Acier en mm	Aire d'Acier en cm²/m	Position fibre neutre v_1 en cm	v_2 en cm	Mt d'inertie i en cm⁴	Module de résistance $i.v_1$	$i.v_2$
0,75	0,71	11,18	2,75	2,75	57,54	20,92	20,92
0,88	0,84	13,23	2,75	2,75	68,08	24,76	24,76
1,00	0,96	15,12	2,75	2,75	77,80	28,29	28,29
1,20	1,16	18,27	2,75	2,75	94,01	34,19	34,19

Portées admissibles au coulage en mètres

Distances maximales franchissable par tôle HI-BOND, telles que mesurées selon la figure de la colonne de gauche, admissibles sans étaiement, pour chaque épaisseur de plancher, en fonction de l'épaisseur nominale **t** de **la** tôle et du nombre de travées couvertes par la tôle, pour une déformation du coffrage de 240/1ème de la portée. Les colonnes de droite indiquent la distance maximale de part et d'autre d'une file détais éventuelle.

Acier
Portée = Clair + 5 cm
Béton
Portée = Clair + 5 cm
Bois
Portée = Entraxe

Épaisseur Plancher h cm	t = 0,75 mm Sans Étai ▲▲	▲▲▲	▲▲▲▲	Étais	t = 0,88 mm Sans Étai ▲▲	▲▲▲	▲▲▲▲	Étais	t = 1,00 mm Sans Étai ▲▲	▲▲▲	▲▲▲▲	Étais	t = 1,20 mm Sans Étai ▲▲	▲▲▲	▲▲▲▲	Étais
10	2,70	3,60	3,33	3,35	2,85	*3,80*	3,52	3,63	2,96	*3,95*	3,66	3,97	3,14	*4,18*	3,88	4,23
11	2,60	3,48	3,22	3,15	2,74	3,66	3,39	3,48	2,85	3,81	3,53	3,71	3,02	4,03	3,73	4,06
12	2,51	3,36	3,11	2,95	2,65	3,54	3,28	3,35	2,76	3,69	3,41	3,57	2,93	3,91	3,62	3,91
13	2,43	3,26	3,02	2,78	2,57	3,44	3,19	3,18	2,68	3,59	3,32	3,44	2,85	3,81	3,52	3,77
14	2,37	3,17	2,94	2,64	2,50	3,34	3,09	3,01	2,61	3,49	3,23	3,33	2,76	3,70	3,42	3,64
15	2,31	3,10	2,87	2,51	2,44	3,26	3,02	2,86	2,54	3,41	3,15	3,17	2,69	3,61	3,34	3,53
16	2,25	3,02	2,80	2,39	2,38	3,19	2,95	2,72	2,48	2,33	3,08	3,02	2,63	3,53	3,26	3,43
17	2,20	2,95	2,73	2,29	2,33	3,12	2,88	2,60	2,43	3,26	3,01	2,89	2,57	3,45	3,19	2,33
18	2,15	2,89	2,67	2,19	2,28	3,05	2,82	2,49	2,37	3,19	2,95	2,77	2,52	3,38	3,13	3,21
19	2,12	2,84	2,63	2,11	2,23	3,00	2,77	2,40	2,34	3,13	2,90	2,66	2,48	3,33	3,08	3,08
20	2,08	2,79	2,58	2,03	2,19	2,94	2,82	2,31	2,29	3,07	2,84	2,56	2,44	3,27	3,02	2,96
22	2,00	2,69	2,49	1,93	2,11	2,84	2,62	2,15	2,21	2,96	2,74	2,38	2,35	3,15	2,91	2,75
24	1,94	2,60	2,41	1,85	2,05	2,76	2,55	2,01	2,14	2,88	2,66	2,23	2,27	3,05	2,82	2,58

Pour les valeurs en italique, l'élancement du plancher est supérieur à 36.

Ces valeurs maximales conviennent lorsque les arrêts de coulage éventuels sont au droit des supports, aux extrémités des tôles et si toutes les précautions utiles sont prises au moment du coulage pour éviter une surépaisseur de béton même localisée, même temporaire, sur la tôle. En cas contraires, choisir des portées moindres.
Fixer les tôles par deux fixations au moins par bac à chaque extrémité.

Figure 1. 3 : Data sheet for Hi-Bond55.

Shear connecters: We cannot speak of the collaborative effect between concrete and steel if there is no link between the two materials to ensure that they will work together as one. This connection which ensures this behavior is made through connectors.There are several types of connectors**,** in our case **headed studs** will be used**.**

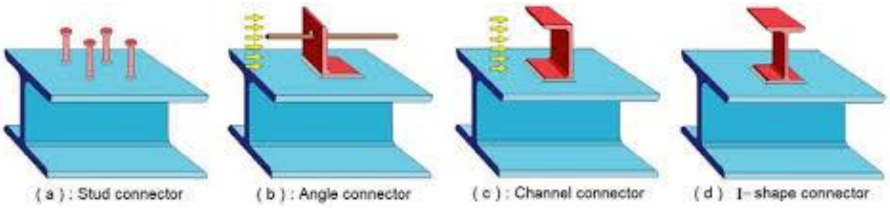


Figure 1. 4 : some types of shear connectors.

Studs of height h = 95mm and diameter d = 19mm are used, which are assembled by welding.

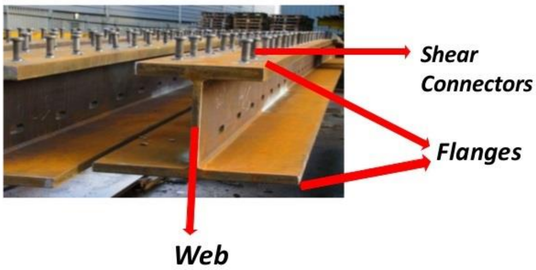


Figure 1. 5 : Stud connectors.

1.2.3. Access ramp

The structure also has two ramps that connect between each half-storey of the structure, inclined by 15%

1.2.4. Building facades

According to ***ARCELORMITTAL*** *Building & Construction Support:*[**1**]

The ventilation surfaces must be at least equal to 50% of the total surface of these facades and correspond to at least 5% of the floor surface of a level;

– The maximum distance between the opposite facades open to the open air is less than 75 m.

These characteristics correspond to the need to be able to easily evacuate the fumes at high temperature in favor of fresh. The two criteria are verified for our structure.

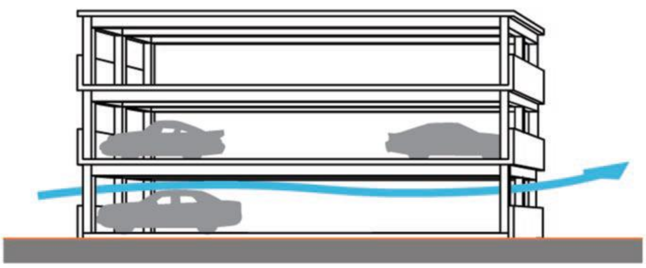


Figure 1. 6 : ventilation in a parking lot.

1.2.5. Bracing

Bracing is a system to ensure the stability of a structure confronted to horizontal effects .it is intended to transmit horizontal forces in the foundations.

1.3. Properties of the used materials

1.3.1. Steel

Steel is a material characterized by its good tensile strength. We use the following types of steel:

a) Construction steel :

The mechanical characteristics of the different grades of steel are as follows:

Steel grade	Thickness (mm)			
	t < 40 mm		40 mm < t < 100 mm	
	F_y (N/mm^2)	F_u (N/mm^2)	F_y (N/mm^2)	F_u (N/mm^2)
Fe 360	235	360	215	340
Fe 430	275	430	255	410
Fe 510	355	510	355	490

Table 1. 1 : Mechanical characteristics of steel grades according to nominal thickness.

We use Fe430 grade steel which has the following characteristics according to EC3:

- Yield strength: F_Y=275 MPa.
- Tensile strength: Fu= 430 MPa.
- Density: ρ = 7850 kg /m^3.
- Modulus of elasticity (Young's modulus): E =210 000 MPa.
- Shear modulus: $G = \left(\frac{E}{2(1+v)}\right)$ =84 000 MPa.
- Poisson's ratio : $\nu = 0{,}3$

a) Reinforcement steel (Rebar)

	Grade	F_y (Mpa)
Plain bars	Fe 220	215
	Fe 240	235
(HA)bars	Fe 400	400
	Fe 500	500

Table 1. 2 : Mechanical characteristics of reinforcement steel.

We use: -Deformed bars (high adhesion rebar): FeE500.
-Mesh reinforcement: TLE52, ∅ = 6 mm for the slabs.

1.3.2. Concrete

Concrete is a building material, composed of aggregates, sand, cement, water and possibly additive to modify its properties. It has excellent resistance to compression. Strength class for the concrete used in out project is:

- C25/30 for the slabs and vertical elements.
- C25/35 for the footing.

a) Concrete resistance

- **Concrete characteristics**

– Compressive strength at 28 days: $F_{c\,28}$ = 25 MPa
– Tensile strength at 28 days: $F_{t\,28}$ = 0,6+0.06 $F_{c\,28} \rightarrow F_{t28} = 2.1MPa$
– Density: ρ = 2500 dan/m^3.
– Shrinkage coefficient: $\varepsilon = 2 \times 10^{-4}$.

- **Poisson's ratio :**

The Poisson's coefficient is the ratio between the relative increase in the transverse dimension and the relative longitudinal shortening;

$$\vartheta = \frac{\text{Lateral strain}}{\text{Longitudianl strain}}$$

According to BAEL, the values are as follows: $\upsilon = 0$ at ULS $\upsilon = 0.2$ at SLS

- **Young's modulus**

This module is defined under the action of normal stress of long or short duration.

– **Instantaneous Young's modulus**

For less than 24h load application: $E_{ij} = 11000\sqrt[3]{f_{cj}}$ $D'o$ù : $E_{i28} = 32164.195MPa$

– **Long-term Young's modulus**

For long-term load application: $E_{ij} = 3700\sqrt[3]{f_{cj}}$ $where$: $E_{i28} = 10818.86\ MPa.$

b) Ultimate stress

By definition, "a limit state" is a particular state beyond which a structure, or a part of this structure ceases to fulfill its functions or no longer satisfies the conditions for which it was designed. We distinguish

- **Ultimate limit state**

Which corresponds to the maximum bearing capacity value:

– static equilibrium
– Strength of the structure or one of its elements.
– Structural stability.

The ultimate compressive stress at the ultimate limit state (ULS) is given by: $\sigma_{bu} = \dfrac{0.85\, fc28}{\gamma_b}$

γ_b = 1.5 in case of persistent and transient actions

γ_b = 1.15 in case of accidental actions

- **Serviceability limit state**

Which defines the state beyond which the operating and sustainability conditions of the construction or of one of its elements are no longer satisfied:

– Crack opening
– Excessive deformations of the bearing elements
– Uncomfortable vibrations for users, etc. …

The ultimate compressive stress at the serviceability limit state is given by: σ_{bc}= 0,6f_{c28} = 15 MPa.

- **Shear stress :**

The ultimate shear stress can take the following values:

– Little damaging cracking: $\overline{\tau}$= min (0.13 fc28, 4 MPa) = 3.25MPa
– Damaging or very damaging cracking: $\overline{\tau}$= min (0.10 fc28, 3MPa) = 2.5 MPa.

1.4. Used Regulations

- CCM97 D.T.R.-B.C.-2.44.
- RPA99 version 2003 D.T.R.-B.C.-2.48.
- RNV2013 D.T.R.-C2-4.7.
- Charges permanentes et surcharges d'exploitation (D.T.R.-B ; C-2.2).
- BAEL91 – CBA93.
- Eurocode 3.
- Eurocode 1.
- Eurocode 4.

CHAPTER 02:

Loads assessment.

2. LOADS ASSESMENT

2.1. Introduction

In order to design the structural elements to safely resist all actions that they are likely to face during service, an assessment of all the loads and overloads acting on the latter is essential.

Loads are assessed in accordance with the regulations (D.T.R-BC.2.2) [**2**] and Eurocode 1. [**3**]

2.2. Dead loads

2.2.1. Common floor

Component	Density (KN/m^3)	Thickness (m)	G (KN/m^2)
Poured asphalt and bituminous concrete	**25**	**0.05**	**1.25**
Reinforced concrete slab	**25**	**0.12**	**3**
Profiled steel decking Hi-bond 55	**--**	**--**	**0.13**
G=4.38kN/m²			

Table 2. 1 : Load assessment for the common floor.

2.2.2. Accessible roof

Component	Density (KN/m^3)	Thickness (m)	G (KN/m^2)
Poured asphalt and bituminous concrete	**25**	**0.05**	**1.25**
Reinforced concrete slab	**25**	**0.12**	**3**
Multilayer waterproofing	**--**	**0.2**	**0.12**
Thermal insulation (cork blocks)	**--**	**0.4**	**0.16**
Profiled steel decking Hi-bond 55	**--**	**--**	**0.13**
G=4.66kN/m²			

Table 2. 2 : Load assessment for the roof.

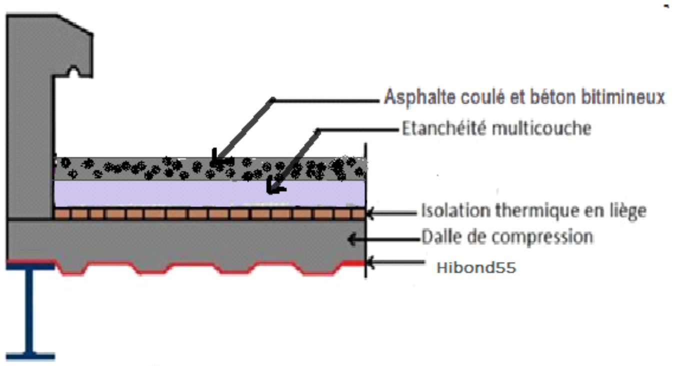


Figure 2. 1 : Composition of the accessible roof.

2.3. Live loads

The building is used for parking, so the operating load for the floors and the roof according to [**2**] is: $Q = 2.5kN/m^2$

2.4. Climatic loads

2.4.1. Snow loads

2.4.1.1. Introduction

The snow accumulated on the roof of the structure produces an overload which must be taken into account for the verification of the elements of this structure.
To assess this load, the RNV2013 regulation[4] was used, which is applicable to all constructions in Algeria located at an altitude below 2000 meters.
The project is at an altitude of 98m above sea level

2.4.1.2. Snow load on the roof S (kN/m^2)

According to the DTR, the characteristic snow load S per unit of area in horizontal roof projection is obtained by the following formula:

$$\boldsymbol{S = \mu . S_k}$$

Such as:

- S_k Characteristic value of snow on the ground for the given location ,in our case WILAYA of TIPAZA (42) therefore the snow zone is **zone B**

$$S_K = \frac{0.04H + 10}{100}$$

With H; the altitude

H=98m

After calculation we will have:

$$S_K = \frac{0.04 \times 98 + 10}{100}$$

$$\boldsymbol{S_k = 0.1392\ kN/m^2}$$

- μ ; is the snow load shape coefficient, given according to the shape of the roofing ; we have a flat roof where $\alpha = 0$ therefore $\mu = 0.8$

$$S = \mu . S_k = 0.8 \times 0.1392$$

$$\boldsymbol{S = 0.114 kN/m^2}$$

2.4.2. Wind action

2.4.2.1. Introduction

The effect of the wind on a construction has a great influence on the stability of the structure and it is predominant when it is steel construction. For this, an in-depth study must be carried out to determine the various actions due to the wind, using [**4**]
The regulations apply to constructions to witch the height is less than 200 m.
The structure studied has a height of 21m

For a rectangular construction, we will consider two wind directions. The calculation must be performed separately for the two directions.

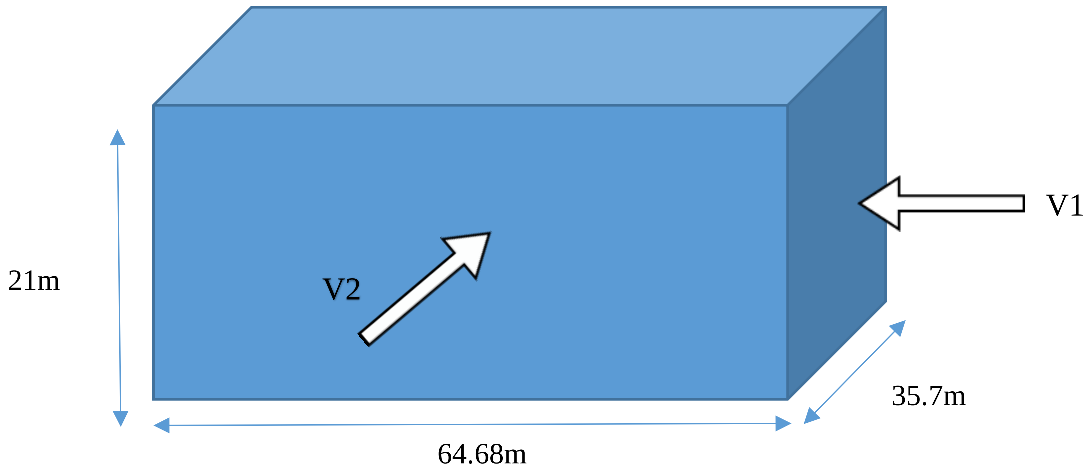


Figure 2. 2 : Considered Wind directions.

2.4.2.2. Basic values

a. basic velocity pressure (or dynamic reference pressure) $q_{réf}$

$q_{réf}$ Is the dynamic reference pressure given in table 2.2 (chapter 2 of[**4**]), it is dependent of the wind zone.

Our structure is located in the Wilaya of Tipaza; therefore ZONE I. the reference pressure is then: $q_{réf} = 375N/m^2$

b. site effects $(K_T\ ;\ z_0\ ;\ z_{min}\ ;\ \varepsilon)$

According to the regulations, the terrains are classified into 4 categories defined in Table 2.4 followed by photos illustrating the roughness of each terrain category in Appendix 4.

Our structure will be built on a suburban area so it belongs to category III, hence:

- The terrain factor $K_T = 0.215$
- The roughness length $z_0 = 0.3$
- The minimum height $z_{min} = 5$
- Coefficient $\varepsilon = 0.61$

c. Topographic coefficient $C_t(z)$

The coefficient $\boldsymbol{C_t(z)}$ takes into account the slope of the site, for our case, we have a flat site from where $\boldsymbol{C_t(z) = 1}$

d. dynamic coefficient C_d

In our case the dynamic coefficient can be taken at the simplified value $\boldsymbol{C_d}$= 1 (according to 3.2 Chapter 3[**4**])

The dynamic coefficient is calculated according to expression (3.1) of RNV2013

$$C_d = \frac{1 + 2 \times g \times I_v(z_{eq}) \times \sqrt{Q^2 + R^2}}{1 + 7 \times I_v(z_{eq})}$$

- z_{eq} is the equivalent height of the construction given by figure 3.1.

$z_{eq} = 0.6 \times h > z_{min}$ Hence $z_{eq} = 0.6 \times 21 = 12.6$m

- $I_v(z_{eq})$ is the intensity of the turbulence for $z = z_{eq}$; given 2.4.6

$$z_{eq} > z_{min}$$

$I_v(z_{eq}) = \frac{1}{C_t(z) \times \ln(\frac{z_{eq}}{z_0})}$ We got: $I_v(z_{eq}) = 0.268$

- Q^2 is the quasi-static part given in 3.3.1 by:

$$Q^2 = \frac{1}{1 + 0.9 \times \left(\frac{(b+h)}{L_t(z_{eq})}\right)^{0.63}}$$

$L_t(z_{eq})$ Is the turbulence length scale for $z = z_{eq}$; given by 3.3.a :

$z_{min} \le z_{eq} \le 200m$ $L_t(z_{eq}) = 300 \times \left(\frac{z}{200}\right)^{\varepsilon}$ We got t: $L_t(z_{eq}) = 55.55$

For V1: $Q^2 = \frac{1}{1+0.9\times\left(\frac{(35.7+21)}{55.55}\right)^{0.63}} = 0.523$

For V2: $Q^2 = \frac{1}{1+0.9\times\left(\frac{(64.68+21)}{55.55}\right)^{0.63}} = 0.458$

- R^2 is the resonant response factor given in 3.3.2 by the expression :

$$R^2 = \frac{\pi^2}{2 \times \delta} \times R_N \times R_h \times R_b$$

R_N : The non-dimensional function of the spectral density of the power given by (3.5) :

$R_N = \frac{6{,}8 \times N_x}{(1+10{,}2\times N_x)^{5/3}}$

- N_x is the non-dimensional frequency in the wind direction x given by:

$N_x = \frac{n_{1x} \times L_i(z_{eq})}{V_m(z_{eq})}$

- $V_m(z_{eq}) = C_r(z) \times C_t(z) \times V_{ref}$

Zone I ⇒ $V_{ref} = 25m/s$ $C_t(z) = 1$; Z=Z_{eq}= 12.6m; $C_r(z) = 0.8$

$V_m(z_{eq}) = 0.8 \times 1 \times 25 = 20.1\ m/s$

n_{1x}: Fundamental frequency given by: $n_{1x} = \frac{0.5}{\sqrt{f}} = 1.09\ [HZ]$

$N_x = 3.015$

$R_N = \frac{6,8 \times 3.015}{(1+10,2 \times 3.015)^{5/3}} = 0.064$

R_h et R_b Are aerodynamic admittance functions given by:

$R_h = \left(\frac{1}{\eta_h}\right) - \left(\frac{1}{2 \times \eta_h{}^2}\right) \times \left(1 - e^{-2 \times \eta_h{}^2}\right) \qquad pour\ \eta_h > 0$

$R_b = \left(\frac{1}{\eta_b}\right) - \left(\frac{1}{2 \times \eta_b{}^2}\right) \times \left(1 - e^{-2 \times \eta_b{}^2}\right) \qquad pour\ \eta_b > 0$

$\eta_h = \frac{4,6 \times N_x \times h}{L_i(z_{eq})} = 5,25 \qquad \eta_b = \frac{4,6 \times N_x \times b}{L_i(z_{eq})} = 8.91$

$R_h = 0.095$ (3.7.a)

$R_b = 0.056$

δ is the logarithmic decrement of vibration damping given by (3.9) $\delta = 0.05$

We got: $R^2 = 0.034$

- g is the peak factor given by 3.11

$$g = \sqrt{2 \times Ln(600 \times v)} + \frac{0.6}{\sqrt{2 \times Ln(600 \times v)}} \geq 3$$

We got: for V1: $g = 3.37$ and V2: $g = 3.40$

Therefore: $\boldsymbol{C_d = 0.81}$ ***for V1***

$\boldsymbol{C_d = 0.80}$ ***For V2***

2.4.2.3. Peak velocity pressure $q_p(ze)$

It is given by :$q_p(ze) = q_{réf} \times Ce(ze)[N/m^2]$ (2.1)

$Ce(ze)$ is the wind exposure coefficient which takes into account the effects of the roughness of the terrain, the site topography, the height above the ground and the turbulent nature of the wind, it is given by the following expression :

$$Ce(z) = Ct(z)^2 \times Cr(z)^2 \times [1 + 7I_{vz}]$$

Such as:

Ct topographic factor =1

Cr the roughness factor given by (2.3)

I_{vz} The turbulence intensity given by (2.5)

ze Is the reference height for the external pressure (chap.2 §2.3.2) for the windward walls of buildings with vertical walls, Ze is determined as shown in Figure II.1:

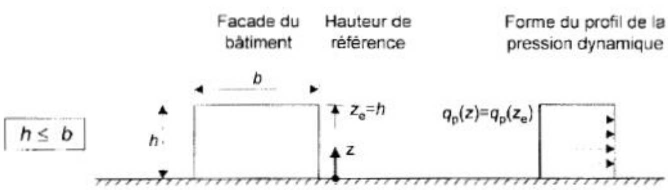


Figure 2. 3 : Reference height, ze, depending on h and b, and corresponding velocity pressure peak.

In our case $h = 35{,}7m$ and $b = 64.68m. \rightarrow h < b$ therefore the reference height is: $\boldsymbol{ze = 21m}$

The table below summarizes the calculation of the peak velocity pressure:

Z(m)	$C_r(z)$	I_{vz}	$Ce(z)$	$q_p(z)$ N/m^2
21	0.913	0.235	2.21	827

Table 2. 3 : Calculation of peak velocity pressure.

The distribution of dynamic pressure is shown in the figure below:

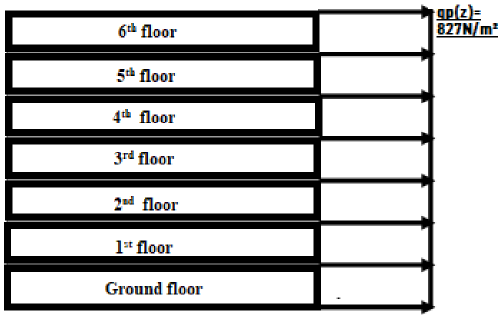


Figure 2. 4 : Distribution of the peak velocity pressure.

2.4.2.4. Wind pressure (aerodynamic pressure)

The aerodynamic pressure is obtained by expression (2.6) of [**4**]

$$W(zj) = q_p(ze) \times [Cpe - Cpi]$$

A. *Wind direction V1* We have: b=35.7; d=64.68; h=21

a) the pressure coefficient for the external pressure C_{pe}

Is the pressure coefficient for the external pressure depending on the size of the loaded area A., it is obtained from the expression (5.1) of [**4**].

$$C_{pe} = C_{pe.1} \; si \; S \leq 1 \, m^2 .$$

- $C_{pe} = C_{pe.10} + (C_{pe.10} - C_{pe.1}) \times log10(S)$ $\quad$ ***si*** $1 \, m^2 < S < 10 \, m^2$.
- $C_{pe} = C_{pe.10}$ $\quad$ ***si*** $\quad S \geq 10 \, m^2$.

- **vertical walls**

The walls are divided according to figure 5.1 such that:

$e = \min(b; 2h) = \min(35.7; 42) = 35.7$

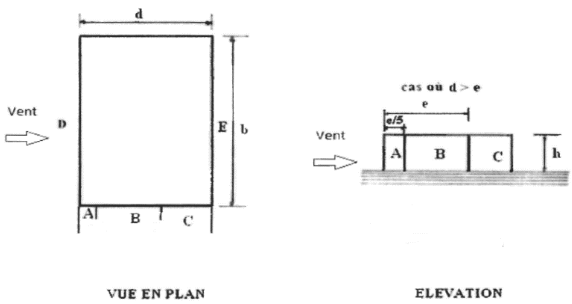


Figure 2. 5 : Key for vertical walls.

The areas of each zone are grouped in the table below:

Zones (m)	A	B	C	D	E
H(m)	21	21	21	21	21
L(m)	7.14	28.56	28.98	35.7	35.7
S(m^2)	149.94	599.76	608.58	749.7	749.7

Table 2. 4 : Vertical walls areas in direction V1.

Therefore:

A	B	C	D	E
-1	-0.8	-0.5	+0.8	-0.3

Table 2. 5 : The coefficients Cpe corresponding to each zone of vertical walls in the direction V1.

- **Roof**

The height of the parapet is h=1.2m
We have a flat roof which will be divided according to Figure 5.2 of [**4**].

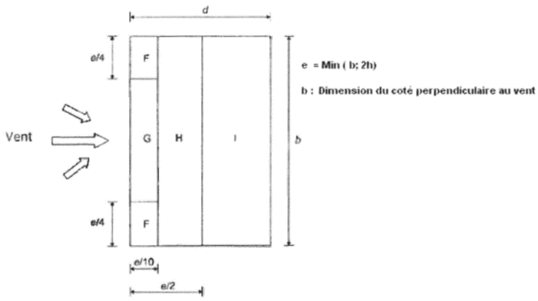


Figure 2. 6 : Key for flat roofs.

The areas of each zone are grouped in the table below:

Zones (m)	F	G	H	I
Length (m)	8.925	17.85	35.7	46.83
Width (m)	3.57	3.57	14.28	35.7
S(m^2)	31.86	63.72	509.796	1671.8

Table 2. 6: Roof areas in the direction V1.

$\boldsymbol{C_{pe} = C_{pe.10}}$ Because the loaded area A for the structure is larger than 10 m²

$\frac{h_p}{h} = \frac{1.2}{21} = 0.057$

So according to table 5.2 of RNV2013, the external pressure coefficients for the roof are:

Zone	F	G	H	I
$\boldsymbol{C_{pe}}$	-1.4	-0.9	-0.7	± 0.2

Table 2. 7: Cpe values for the roof in the direction V1.

b) Internal pressure coefficient C_{pi}

The pressure coefficient is determined according to 5.2 of the regulations. Our building does not have a dominant side, the C_{pi} is to be determined from figure 5.14

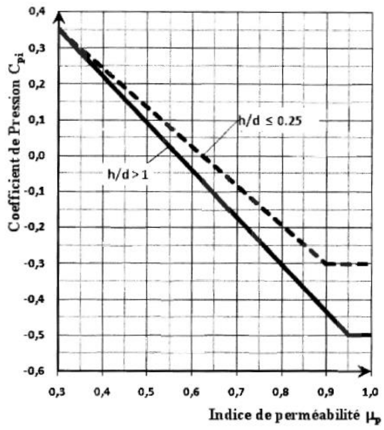


Figure 2. 7 : Internal pressure coefficients Cpi of buildings without a dominant side.

We first determine the permeability index μp as follows:

$$\mu p = \frac{\sum of\ the\ surfaces\ of\ the\ openings\ where\ \boldsymbol{C_{pe}} \leq 0}{\sum of\ the\ surfaces\ of\ the\ openings} = \frac{1386.5}{1686.384} = 0.82$$

This means that in the V1 direction, our structure [parking garage] is 82% permeable.

$\frac{h}{d} = \frac{21}{64.68} = 0.32$ Therefor; from figure 5.14 we find: $C_{pi} = -0.25$

The calculation of the aerodynamic pressure and will be done according to the previous expressions, the calculation is summarized in the following tables:

Zone	$q_p(z)$ N/m²	Area(m²)	C_{pe}	C_{pi}	W(ze) (N/m²)	Total force (KN)
A	827	149,94	-1	-0, 25	-620.25	-93
B	827	599,76	-0,8	-0, 25	-454.85	-272.80
C	827	608,58	-0,5	-0, 25	-206.75	-125.82
D	827	749,7	0,8	-0, 25	868.35	651.01
E	827	749,7	-0,3	-0, 25	-41.35	-31

Table 2. 8: Wind pressure W (ze) (N/m²) values for the vertical walls in the direction V1.

Zone	$q_p(z)$ N/m²	Area (m²)	C_{pe}	C_{pi}	W(ze) (N/m²)	Total force(kN)
F	827	31.68	-1 ,4	-0,25	-951.05	-30.1
G	827	63.72	-0.9	-0,25	-537.55	-34.25
H	827	509,796	-0,7	-0,25	-372.15	-189.72
I	827	1676,115	-0 ,2	-0,25	41.35	69.30

Table 2. 9 : Wind pressure W (ze) (N/m²) values for the roof in the direction V1.

B. *Wind direction V2* We have: b=64.68; d=35.7; h=21

a) the pressure coefficient for the external pressure C_{pe}

- **Vertical walls**

$C_{pe} = C_{pe.10}$

The walls are divided according to figure 5.1 such that: $e = \min(b; 2h) = \min(64.6; 42) = 42$

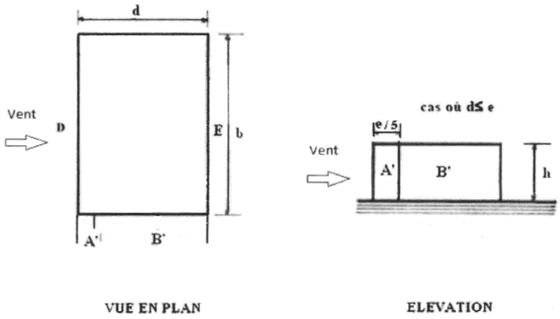


Figure 2. 8: Key for vertical walls.

The areas of each zone are grouped in the table below:

Zones (m)	A'	B'	D	E
H(m)	21	21	21	21
L(m)	8.4	27.3	64.68	64.68
S(m²)	176.4	573.3	1358.28	1358.28

Table 2. 10: Vertical walls areas in the direction V2.

For our structure, all surfaces are greater than 10m², hence $C_{pe} = C_{pe.10}$

A'	B'	D	E
-1	-0.8	+0.8	-0.3

Table 2. 11: Cpe values for the roof in direction V2.

- **Roof**

The height of the parapet is h=1.2m We have a flat roof which will be divided according to Figure 5.2 of [**4**]

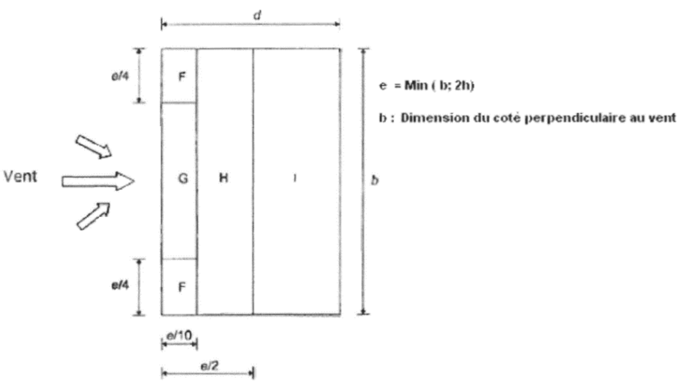


Figure 2. 9 : Key for flat roof.

The areas of each zone are grouped in the following table:

Zones (m)	F	G	H	I
Length (m)	10.5	43.68	64.68	64.68
Width (m)	4.2	4.2	16.8	14.7
S(m^2)	44.1	183.456	1086.624	950.796

Table 2. 12 : Roof areas in the direction V2.

$\boldsymbol{C_{pe} = C_{pe.10}}$ Because the loaded area A for the structure is larger than 10 m²

$\frac{h_p}{h} = \frac{1.2}{21} = 0.057$

So according to table 5.2 of RNV2013, the external pressure coefficients for the roof are:

Zone	F	G	H	I
$\boldsymbol{C_{pe}}$	-1.4	-0.9	-0.7	± 2

Table 2. 13: Cpe values for the roof in the direction V2.

b) Internal pressure coefficients $\boldsymbol{C_{pi}}$

The pressure coefficient is determined according to 5.2 of the regulations. Our building does not have a dominant side, the $\boldsymbol{C_{pi}}$ is to be determined from figure 5.14

We first determine the permeability index µp as follows:

$$\mu p = \frac{\sum of\ the\ surfaces\ of\ the\ openings\ where\ \boldsymbol{C_{pe}} \leq 0}{\sum of\ the\ surfaces\ of\ the\ openings} = \frac{1143.072}{1686.384} = 0.68$$

This means that in the V2 direction, our structure [parking garage] is 68% permeable.

$\frac{h}{d} = \frac{21}{35.7} = 0.6$

From figure 5.14 we find $C_{pi} = -0.175$

The calculation of the aerodynamic pressure and will be done according to the previous expressions, the calculation is summarized in the following tables:

Zone	$\boldsymbol{q_p(z)}$ N/m^2	Area (m^2)	$\boldsymbol{C_{pe}}$	$\boldsymbol{C_{pi}}$	W(ze) (N/m^2)	Total force (kN)
A'	827	176.4	-1	-0, 175	-682.275	-120.35
B'	827	573.3	-0,8	-0, 175	-516.875	-296.32
D	827	1358.25	0,8	-0, 175	806.325	1095.19
E	827	1358.28	-0,3	-0, 175	-103.375	-140.41

Table 2. 14 : Wind pressure W(ze) (N/m²) values for the vertical walls in the direction V2.

Zone	$\boldsymbol{q_p(z)}$ N/m^2	Area (m^2)	$\boldsymbol{C_{pe}}$	$\boldsymbol{C_{pi}}$	W(ze) (N/m^2)	Total force (kN)
F	827	44.1	-1 ,4	-0,175	-1013.075	-44.67
G	827	183.456	-0.9	-0,175	-599.575	-110
H	827	1086.624	-0,7	-0,175	-434.175	-471.78
I	827	950.796	-0 ,2	-0,175	-20.675	-19.65

Table 2. 15 : Wind pressure W (ze) (N/m²) values for the roof in the direction V2.

2.4.2.5. Wind force Fw calculated from pressure coefficient

The force exerted on construction or a construction element is obtained by the vectorial summation of the forces $F_{w,e}$; $F_{w,i}$; F_{fr} which, according to the regulation RNV2013 are determined by the expressions:

- external forces:: $F_{w,e} = C_d \times \Sigma W_e \times A_{réf}$
- internal forces:: $F_{w,i} = \Sigma W_i \times A_{réf}$
- friction forces: $F_{fr} = C_{fr} \times q_p(z) \times A_{fr}$

Such as:

W_e Is the external pressure on the individual surface at height ze, given by: $W_e = q_p(ze) \times Cpe$

W_i Is the internal pressure on the individual surface at height zi, given by $W_i = q_p(ze) \times Cpi$

$A_{réf}$ is the reference area of the individual surface.

$C_{fr} = 0.01$ Given by table -2.8- for a smooth surface.

$A_{fr} = d \times h$ is the area of external surface parallel to the wind.

The effects of wind friction on the surface can be neglected when the total area of all surfaces parallel to the wind (or slightly inclined with respect to the wind direction) is less than or equal to 4 times the total area of all exterior surfaces perpendicular to the wind (upwind and downwind). 2(d×h) ≤ 4(2b× h).

Direction V1

$2(35.7 \times 21) \leq 4(2(64.68) \times 21)$

$2716.56 \leq 5997.6$

Direction V2

$2(64.68 \times 21) \leq 4(2(35.7) \times 21)$

1499.4 ≤ 10866.24

The friction force is neglected according to 2.6.3 of the RNV2013 regulation.

The calculation of $F_{w,i}$ and $F_{w,e}$ will be summarized in the following tables:

a) Direction V1 of the Wind

Zone	$q_p(z)$ N/m^2	C_{pe}	C_{pi}	W_e	W_i	Area (m^2)	C_d	$F_{w,e}$ (KN)	$F_{w,i}$ (KN)
A	827	-1	-0, 25	-827	-206.75	149,94	0.81	-374.58	-435.8
B	827	-0,8	-0, 25	-661	-206.75	599,76			
C	827	-0,5	-0, 25	-413	-206.75	608,58			
D	827	0,8	-0, 25	661	-206.75	749,7			
E	827	-0,3	-0, 25	-248	-206.75	749,7			

Table 2. 16 : Calculation of Fwi and Fwe for vertical wall in the direction V1.

Zone	$q_p(z)$N/m^2	C_{pe}	C_{pi}	W_e	W_i	Area (m^2)	C_d	$F_{w,e}$ (KN)	$F_{w,i}$ (KN)
F	827	-1.4	-0,25	-1157.8	-206.75	31.68	0.81	-531.7	-471.6
G	827	-0.9	-0,25	-744.3	-206.75	63.72			
H	827	-0,7	-0,25	-578.9	-206.75	509,79			
I	827	-0 ,2	-0,25	-165.4	-206.75	1676,115			

Table 2. 17: Calculation of Fwi and Fwe for the roof in the direction V1.

b) Direction V2 of the wind

Zone	$q_p(z)$ N/m^2	C_{pe}	C_{pi}	W_e	W_i	Area (m^2)	C_d	$F_{w,e}$ (KN)	$F_{w,i}$ (KN)
A’	827	-1	-0,175	-827	-144.725	176.4	0.80	299.72	-501.65
B’	827	-0,8	-0,175	-661.6	-144.725	573.3			
D	827	0,8	-0,175	661.6	-144.725	1358.28			
E	827	-0,3	-0,175	-248.1	-144.725	1358.28			

Table 2. 18 : Calculation of Fwi and Fwe for vertical wall in the direction V2.

Zone	$q_p(z)$ N/m^2	C_{pe}	C_{pi}	W_e	W_i	Area (m^2)	C_d	$F_{w,e}$ (KN)	$F_{w,i}$ (KN)
F	827	-1.4	-0,175	-1157.8	-144.725	44.1	0.80	-779.1	-327.79
G	827	-0.9	-0,175	-744.3	-144.725	183.456			
H	827	-0,7	-0,175	-578.9	-144.725	1086.624			
I	827	-0 ,2	-0,175	-165.4	-144.725	950.796			

Table 2. 19 : calculation of Fwi and Fwe for the roof in the direction V2.

2.5. Conclusion

A proper load assessment is necessary to encore that the building meets the safety regulations and that these loads do not exceed the bearing limit of the structural elements.

In this chapter, we have provided the general principles and procedures for determining the loads acting on the studied structure (permanent loads, operating loads and climatic loads). The results found will be used in the next chapters which concern the sizing of the elements of the structure (joists, beams, columns, etc.).

CHAPTER 03:

Preliminary design of the structural elements.

3. PRELIMINARY SIZING OF THE STRUCTURAL ELEMENTS

3.1. Introduction

Before starting the dynamic analysis of the structure, a pre-sizing of the load-bearing elements of the structure is necessary (joists; beams and columns) and this, under static loads evaluated in the previous chapter. The sections obtained must be rechecked another time, under dynamic loads.

3.2. Joists

The joists are IPE beams that work in simple bending .We opt for a joist spacing of e = 2m (see the data sheet of hi-bond55 in chapter 1).

We suppose that the beam is hinged at both ends.

3.2.1. Pre-sizing

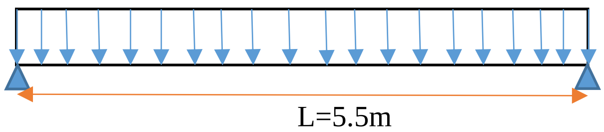


Figure 3. 1 : Static loads diagram for joists.

For pre-sizing we used the following expression: $\frac{L}{25} \leq h \leq \frac{L}{15}$ **[5]**

Such as:

h is the height of the joist and L=5.5m is the span of the joist.

$120m \leq h \leq 333.33m$ We consider an IPE180.

3.2.2. Local buckling check

• Web subjected to bending	• Flange subjected to compression
$\frac{d}{t_w} \leq 72\varepsilon \ \ with \ \ \varepsilon = \sqrt{\frac{275}{f_y}} = 0.92$	$\frac{C}{t_f} \leq 10\varepsilon \ \ avec \ \varepsilon = \sqrt{\frac{275}{f_y}} = 0.92$
$\frac{180-2\times8}{5.3} = 30.94 < 66.24$ Therefore the web is class 1	$\frac{45}{8} = 5.625 < 9.2$Therefore the flanges are class 1

This confirms that the IPE 180 section is not subject to local buckling.

3.2.3. Loads assessment for the joists

3.2.3.1. Construction phase

In the construction phase, concrete is considered fresh, which means that the steel-concrete connection is not yet established; in this case the profiled steel decking takes over the construction loads itself, and the concrete is considered as an overload.

The loads in the construction phase are:

- Self-weight of the steel profile (IPE180) $G_p = 0.188kN/ml$
- Dead weight of fresh concrete $G_{concrete} = 3kN/m^2$
- Self-weight of the profiled steel decking $G_{PSD} = 0.13kN/m^2$
- Construction overload (worker) $Q_{worker} = 0.75kN/m^2$

Loads combinations:

- In the Ultimate limit state

$$q_u = 1.35 \times ((G_{PSD} + G_{concrete}) \times e + G_p) + 1.5(Q \times e) = 1.35 \times ((0.13 + 3) \times 2 + 0.188) + 1.5 \times (0.75 \times 2) = 10.95kN/ml$$

- In the serviceability limit state

$$q_s = (G_{PSD} + G_{concrete} + Q) \times e + G_p = (0.13 + 3 + 0.75) \times 2 + 0.188 = 7.948kN/ml$$

3.2.3.2. Final phase

In the final phase the hardened concrete contributes to the overall resistance of the beam, and takes up part of the final loads (+ operating loads)

- **The loads in the final phase are:**
  - Self-weight of the slab $G_{slab} = 4.66kN/m^2$
  - Self-weight of the profile $G_{profile} = 0.188kN/ml$
  - Operating load of the parking $Q = 2.5kN/m^2$
- **Loads combinations:**
  - In the Ultimate limit state

$$q_u = 1.35\left((G \times e) + G_p\right) + 1.5(Q \times e) = 1.35\left((4.66 \times 2) + 0.188\right) + 1.5 \times (2.5 \times 2) = 20.33kN/ml$$

- In serviceability limit state

$$q_s = (G + Q +) \times e + G_p = (4.66 + 2.5) \times 2 + 0.188 = 14.508kN/ml$$

3.2.4. verifications

3.2.4.1. Construction phase

a) Bending strength check

According to the CCM97[**6**], the design value of the bending moment M_{sd} at each cross-section should satisfy: $M_{sd} \le M_{pl,rd}$

$$M_{sd} = \frac{q_u l^2}{8} \rightarrow M_{sd} = \frac{10.95 \times 5.5^2}{8} = 41.4kN.m$$

$$M_{pl,rd} = \frac{W_{pl} \times f_y}{\gamma_{m0}} = \frac{166.4 \times 275 \times 10^{-3}}{1} = 45.7kN.m$$

$$M_{sd} = 41.4kN.m \le M_{pl,rd} = 45.76kN.m$$

Section performance: $\frac{M_{sd}}{M_{pl,rd}} = 90.47\%$

The condition is verified.

b) Shear resistance check

According to [**6**], The design value of the shear force V_{sd} at each cross-section should satisfy: $V_{sd} \le V_{pl,rd}$

$V_{sd} = \frac{q_u \times l}{2} = \frac{10.95 \times 5.5}{2} = 30.11kN$

$V_{pl,rd} = \frac{A_{vz} \times f_y}{\gamma_{m0}\sqrt{3}} = \frac{11.25 \times 275 \times 10^{-1}}{1 \times \sqrt{3}} = 178.61kN$

$V_{sd} = 30.11kN \leq V_{pl,rd} = 178.61kN$ The condition is verified.

c) bending moment-shear force interaction check

According to[**6**], the interaction must be verified when $V_{sd} > 0.5V_{pl,rd}$

In our case $0.5V_{pl,rd} = 89.30 > 30.11kN$ there is therefore no need to verify the interaction.

d) Joist deflection check

According to [**6**], we must check that $f \leq f_{allowable}$;

During the construction phase, props must be put at mid-span

$f = \frac{5 \times q_s \times L^4}{384 \times E \times I_y} = \frac{5 \times 7.95 \times 2.25^4}{384 \times 2.1 \times 1317} \times 10^2 = 0.096cm$

$f_{allowable} = \frac{L}{250} = \frac{225}{250} = 0.9cm$

$f = 0.096cm \leq f_{allowable} = 0.9cm$ The condition is verified.

e) Verification of stability against lateral torsional buckling

According to [**6**], (article 5.5.2.) It must be verified that:
$M_{sd} \leq M_{b,rd}$

$$M_{b,rd} = \frac{\chi_{lt} \times \beta_w \times W_{pl,y} \times f_y}{\gamma_{m1}}$$

$\beta_w = 1$ (Class 1 cross – section)

χ_{lt} is the reduction factor for the lateral torsional buckling that must be calculated

$$\bar{\lambda}_{lt} = \sqrt{\frac{\beta_w \times W_{pl,y} \times f_y}{M_{cr}}} = \frac{\lambda_{lt}}{\lambda_1}\sqrt{\beta_w}$$

For the rolled profile we can use the simplified expression: $\lambda_{lt} = \frac{KL/i_z}{\sqrt{C_1} \times \left[\left[\frac{K}{K_W}\right]^2 + \frac{1}{20}\left[\frac{KL/i_z}{h/t_f}\right]^2\right]^{0.25}}$

With: $K = 1$; $K_w = 1$; $C_1 = 1.132$; $i_z = 2.05cm$

$$\lambda_{lt} = \frac{\frac{5500}{20.5}}{\sqrt{1.132} \times \left[1 + \frac{1}{20}\left[\frac{\frac{5500}{20.5}}{\frac{180}{8}}\right]^2\right]^{0.25}} = 149.43$$

$\bar{\lambda}_{lt} = \frac{149.43}{93.91 \times 0.92} = 1.73 > 0.4$ *there is a risk of lateral torsional buckling*

$\phi_{lt} = 0.5 \times \left[1 + \alpha_{lt} \times (\bar{\lambda}_{lt} - 0.2) + \bar{\lambda}_{lt}^{\,2}\right]$ $\alpha_{lt} = 0.21$ *for hot rolled profiles*

$\phi_{lt} = 0.5 \times [1 + 0.21 \times (1.73 - 0.2) + 1.73^2] = 2.15$

$$\chi_{lt} = \frac{1}{\phi_{lt}+\sqrt{\phi_{lt}^2-\bar{\lambda}_{lt}^2}} = \frac{1}{2.15+\sqrt{2.15^2-1.73^2}} = 0.29$$

$M_{b,rd} = \frac{0.29\times1\times166.4\times10^3\times275}{1.1}\times10^{-6} = 12.13kN.m$ $M_{b,rd} = 12.13kN.m < M_{sd} = 41.4kN.m$

The condition is not checked we need to add props to the joists at mid span to diminish the soliciting bending moment.

We'll have:

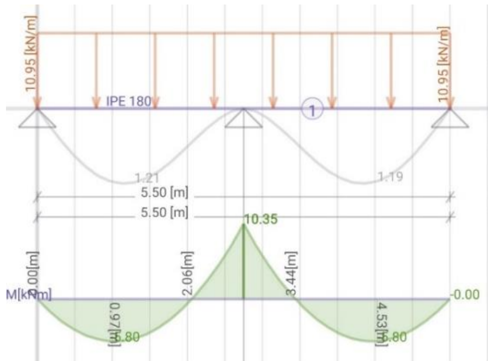


Figure 3. 2: Bending mment diagram for the joist after adding props.

$M_{sd(on\ supports)} = 10.35kN.m < M_{b,rd} = 12.13kN.m$ The condition is verified.

$M_{sd(at\ mid-span)} = 5.8kN.m < M_{b,rd} = 12.13kN.m$ The condition is verified.

3.2.4.2. Final phase

➢ **Width of the slab; effective width**

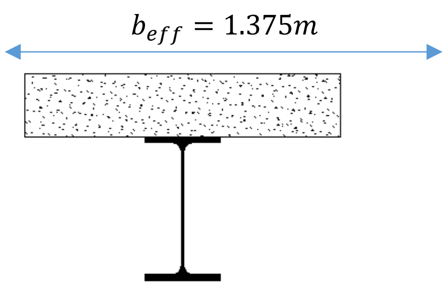


Figure 3. 3 : Effective width for joists.

➢ **Position of the plastic neutral axis** [7]

- $R_{concrete} = 0.57\times f_{ck}\times b_{eff}\times h_c$

$R_{concrete} = 0.57\times25\times1375\times65\times10^{-3} = 1273.59kN$

- $R_{steel} = 0.95\times f_y\times A_a$

$R_{steel} = 0.95\times275\times2395\times10^{-3} = 625.69kN$

$$\boldsymbol{R_{concrete} > R_{steel}\ neutral\ axis\ is\ in\ the\ concrete\ slab}$$

a) Bending strength check

We must check that $M_{sd} \le M_{pl,rd}$ $M_{sd} = \frac{q_u l^2}{8} \to M_{sd} = \frac{20.358\times5.5^2}{8} = 76.97kN.m$

$$M_{pl,rd} = R_{steel} \times \left[\frac{h_a}{2} + h_c + h_p - \left(\frac{R_{steel}}{R_{concrete}} \times \frac{h_c}{2}\right)\right] \qquad [\mathbf{7}]$$

$$M_{pl,rd} = 625.69 \times \left[\frac{180}{2} + 65 + 55 - \left(\frac{625.69}{1273.59} \times \frac{65}{2}\right)\right] \times 10^{-3} = 121.4kN$$

$M_{sd} = 76.97N.m \leq M_{pl,rd} = 121.4kN.m$ The condition is verified.

Section performance: $\frac{M_{sd}}{M_{pl,rd}} = 63.4\%$

b) Shear strength check

We must check that $V_{sd} \leq V_{pl,rd}$ $\quad V_{sd} = \frac{q_u \times l}{2} = \frac{20.33 \times 5.5}{2} = 55.98kN$

$$V_{pl,rd} = \frac{A_{vz} \times f_y}{\gamma_{m0}\sqrt{3}} = \frac{11.25 \times 275 \times 10^{-1}}{1 \times \sqrt{3}} = 178.61kN$$

$V_{sd} = 55.98kN \leq V_{pl,rd} = 178.61kN$ The condition is verified.

c) Bending moment-shear force interaction check

According to the [**6**] the interaction must be verified when $V_{sd} > 0.5V_{pl,rd}$

In our case $0.5V_{pl,rd} = 55.98kN > 89.305kN$

Therefore it is not necessary to check the interaction.

d) Joist deflection check

We must check: $f_{total} \leq f_{allowable}$

$$f = \frac{5 \times q_s \times L^4}{384 \times E \times I_c} \; with \; I_c = \frac{A_a \times (h_c + 2 \times h_p + h_a)^2}{4 \times (1 + m \times v)} + \frac{b_{eff} \times {h_c}^3}{12 \times m} + I_a \; ;$$

With: $v = \frac{A_a}{A_b} = \frac{2395}{1375 \times 55} = 0.031; m = \frac{E_a}{E_b} = 15$

$$I_c = \frac{2395 \times (65 + 2 \times 55 + 180)^2}{4 \times (1 + 15 \times 0.0431)} + \frac{1375 \times 65^3}{12 \times 15} + 1317 \times 10^4 = 6109.68 \times 10^4 mm^4$$

$$f_{finale} = \frac{5 \times 14.508 \times 5.5^4}{384 \times 2.1 \times 6190.68} \times 10^2 = 1.33cm$$

$f_{total} = f_{construction} + f_{final} = 0.096 + 1.33 = 1.42cm \quad ; f_{allowable} = \frac{L}{250} = \frac{550}{250} = 2.2cm$

$f = 1.42cm \leq f_{allowable} = 2.2cm$ The condition is verified.

e) Verification of stability against lateral torsional buckling

In the final phase, it is not necessary to check the lateral torsional buckling, because the upper flange is held by the concrete slab.

All the resistance and stability conditions imposed by[**6**], *and* [**7**], *are verified; we will then use the IPE180 profile for joists with the following characteristics:*

Profile	weight (kg/ml)	Section A (cm^2)	Dimensions				characteristics					
			h (mm)	*b (mm)*	*t_w (mm)*	*t_f (mm)*	*I_y (cm^4)*	*I_z (cm^4)*	*W_{ply} (cm)*	*W_{plz} (cm)*	*i_y (cm)*	*i_z (cm)*
IPE180	*18.8*	*23.95*	*180*	*91*	*5.3*	*8*	*1317*	*100.9*	*166.4*	*34.6*	*7.42*	*2.05*

Table 3. 1 : IPE180 profile characteristics.

We followed the same steps for the common floor we find that the profile IPE220 checks the conditions

3.2.5. Shear connectors [5]

There are several types of connectors, for our structure we will use connectors “headed studs” with the following characteristics:

$$\begin{cases} height\ \ h = 95mm \\ diameter\ d = 19mm \end{cases}$$

Figure 3. 4 : Headed stud connector

The calculation of the connectors will be done according to Eurocode 04.

- **Headed stud strength $\boldsymbol{P_{rd}}$**

The ultimate shear strength of a headed stud is given by:

$$\boldsymbol{P_{rd}} = K_T \times \boldsymbol{min}[\boldsymbol{P_{rd1}}; \boldsymbol{P_{rd2}}]$$

Such as:

$\boldsymbol{P_{rd1}}$ Rod breaking strength $P_{rd1} = 0.8. f_u. \frac{\pi \times {d_{rd}}^2}{4 \times \gamma_v} \rightarrow P_{rd1} = 0.8 \times 430 \times \frac{\pi \times 19^2}{4 \times 1.25} = 72.58kN$

$\boldsymbol{P_{rd2}}$ Crush resistance of concrete $P_{rd2} = 0.29 \times \alpha \times {d_{rd}}^2 \times \sqrt{f_{ck} \times E_{cm}} \times \frac{1}{\gamma_v}$

$$\alpha = 1\ car \frac{h}{d} > 4$$

$P_{rd2} = 0.29 \times 1 \times 19^2 \times \sqrt{25 \times 30.5} \times \frac{1}{1.25} = 73.133$

- Profiled steel decking influence

The reducing coefficient K_T is given according to the direction of the ribs of the profile steel decking, for a profiled steel decking whose ribs are perpendicular to the joist, the reduction coefficient is:

$$K_T = \frac{0.85}{\sqrt{n_c}} \times \frac{b_0}{h_p} \times \left[\frac{h}{h_p} - 1\right]$$

With:

$-n_c$ Number of connectors per rib; we take $n_c = 1$ - $h = 95mm$ Connector height

-$h_p = 55mm$ Sheet height -$b_0 = 88.5$mm

$K_T = \frac{0.85}{\sqrt{1}} \times \frac{88.5}{55} \times \left[\frac{65}{55} - 1\right] = 0.99$ Therefore $P_{rd} = 72.2kN$

- Shearing force taken up by the connectors

$R_l = \boldsymbol{min}[\boldsymbol{R_{concrete}}; \boldsymbol{R_{steel}}] = \boldsymbol{min}[1273.59; 625.69] = 625.69kN$

- Number of connectors per half-span

$N^{bre}{}_{connectors} = \frac{R_l}{P_{rd}} = \frac{625.69}{72.2} = 10.31$

We take $N^{bre}{}_{connectors} = 11$ per half-span, or 22 all along the beam.

- Connector spacing:

$Esp = \frac{L}{N^{bre}-1} = \frac{5500}{22-1} = 261.9$ *We opt for 22 connectors 250mm apart.*

3.3. Main beams

We suppose that the beam is fixed at both ends.

3.3.1. Pre-sizing

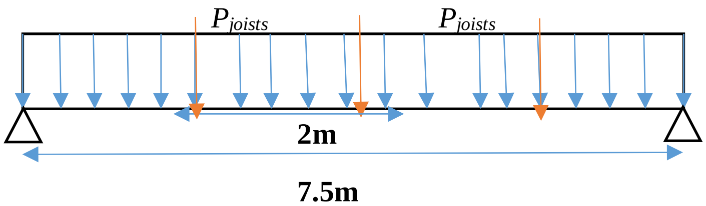


Figure 3. 5 : Static loads diagram for primary beams.

For pre-sizing we used the following expression: $\frac{L}{25} \leq h \leq \frac{L}{15}$

Such as:

H is the height of the beam; L=7.5m is the span of the beam

$300m \leq h \leq 500m$ We choose an IPE300.

3.3.2. Local buckling check

• Web subjected to bending	• Flange subjected to compression
$\frac{d}{t_w} \leq 72\varepsilon \ with\ \varepsilon = \sqrt{\frac{275}{f_y}} = 0.92$ $\frac{300-2\times10.7}{7.1} = 39.23 < 66.24$ Therefore the web is class 1	$\frac{C}{t_f} \leq 10\varepsilon \ avec\ \varepsilon = \sqrt{\frac{275}{f_y}} = 0.92$ $\frac{75}{10.7} = 7 < 9.2$ Therefore the flanges are class 1

This confirms that the IPE 300 section is not subject to local buckling.

3.3.3. Loads assessment for the main beam (intermediate beam)
3.3.3.1. Construction phase

The loads in the construction phase are:

Evenly distributed load

- Self-weight of the steel profile (IPE300) $G_p = 0.422kN/ml$
- Dead weight of fresh concrete $G_{concrete} = 3kN/m^2$
- Self-weight of the profiled steel decking $G_{PSD} = 0.13kN/m^2$
- Construction overload (worker) $Q_{worker} = 0.75kN/m^2$

Loads combinations

- In the Ultimate limit state

$q_u = 1.35((G_{PSD} + G_{concrete}) + G_p) \times b + 1.5(Q \times b)$

$= 1.35 \times (3.13 \times 0.15 + 0.422) + 1.5 \times (0.75 \times 0.15) = 1.37kN/ml$

- In the serviceability limit state

$q_s = \left(G_{PSD} + G_{concrete} + Q\right) \times b + G_p = (0.13 + 3 + 0.75) \times 0.15 + 0.422 = 1kN/ml$

The most stressed beam takes 2 concentrated loads, each one of them representing the reaction of the joists on each side; and a load evenly distributed over its width (the weight of the floor).

Support reactions of the joists

$P_{joists\ (ULS)} = \frac{q_u \times 5.5}{2} + \frac{q_u \times 4.55}{2} = \frac{10.95 \times 5.5}{2} + \frac{10.95 \times 4.55}{2} = 55.02kN$

$P_{Joists(SLS)} = \frac{q_S \times 5.5}{2} + \frac{q_S \times 4.55}{2} = \frac{7.95 \times 5.5}{2} + \frac{7.95 \times 4.55}{2} = 39.94kN$

3.3.3.2. Final phase

Evenly distributed load

➢ **The loads at final phase are**

- Self-weight of the slab $G_{slab} = 4.66kN/m^2$
- Self-weight of the profile $G_{profile} = 0.422kN/ml$
- The operating load of a parking $Q = 2.5kN/m^2$

➢ **Loads combinations :**

➢ In the Ultimate limit state

$q_{ELU} = 1.35((G \times b) + G_{profile}) + 1.5(Q \times b) = 1.35((4.66 \times 0.15) + 0.422) + 1.5 \times (2.5 \times 0.15) = 2.07kN/ml$

- In the serviceability limit state

$q_{ELS} = (G + Q +) \times e + G_{profile} = (4.66 + 2.5) \times 0.15 + 0.422 = 1.49kN/ml$

Support reactions of the joists

$P_{joists\ (ULS)} = \frac{q_u \times 5.5}{2} + \frac{q_u \times 4.55}{2} = \frac{20.33 \times 5.5}{2} + \frac{20.33 \times 4.55}{2} = 102.16kN$

$P_{Joists(SLS)} = \frac{q_S \times 5.5}{2} + \frac{q_S \times 4.55}{2} = \frac{14.508 \times 5.5}{2} + \frac{14.508 \times 4.55}{2} = 72.90kN$

3.3.4. Verifications

3.3.4.1. Construction phase

a) Bending strength check

$M_{sd} \leq M_{pl,rd}$

The calculation of Msd is done by an app called beam design

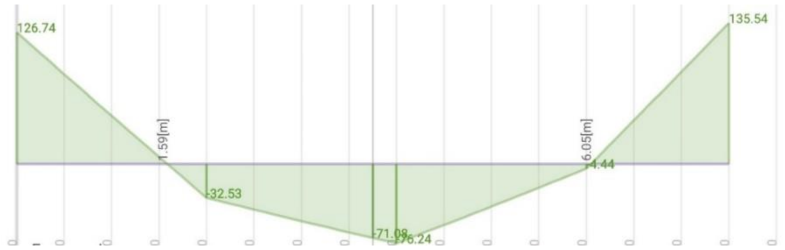


Figure 3. 6: Bending moment diagram for the main beam. (Construction phase)

- At mid-span

$M_{sd1} = 76.24kN.m$

- On the supports

$M_{sd2} = 135.54kN.m$

$$M_{pl,rd} = \frac{W_{pl} \times f_y}{\gamma_{mo}} = \frac{628.4 \times 275 \times 10^{-3}}{1} = 221.18kN.m$$

$maxM_{sd1.2} = 135.54kN.m \leq M_{pl,rd} = 221.18kN.m$ The condition is verified.

Section performance: $\frac{M_{sd}}{M_{pl,rd}} = 62\%$

b) Shear strength check

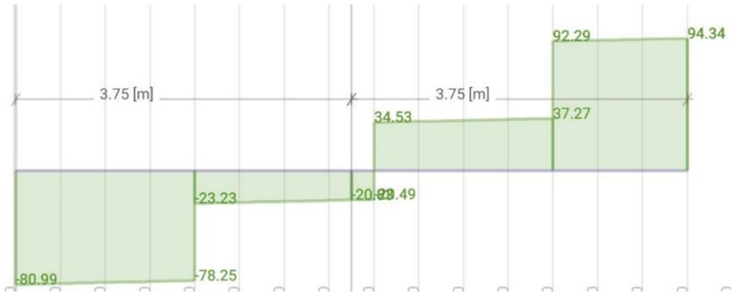


Figure 3. 7: Shear force diagram for main beam. (Construction phase)

$V_{sd} \leq V_{pl,rd}$

$V_{sd} = 94.34kNkN$

$$V_{pl,rd} = \frac{A_{vz} \times f_y}{\gamma_{mo}\sqrt{3}} = \frac{25.68 \times 275 \times 10^{-1}}{1 \times \sqrt{3}} = 407.72kN$$

$V_{sd} = 94.34kN \leq V_{pl,rd} = 407.72kN$ The condition is verified.

c) bending moment-shear force interaction check

According to the CCM97 the interaction must be verified when $V_{sd} > 0.5V_{pl,rd}$

In our case $0.5V_{pl,rd} = 203.86kN > 94.34kN$

There is therefore no need to verify the interaction.

d) Beam deflection check

We must check that $f \leq f_{allowable}$

$$f_{concentrated\ load} = \frac{19P_S \times L^3}{384EI} = \frac{19 \times 39.94 \times 7.5^3}{384 \times 2.1 \times 11770} \times 10^2 = 0.449cm$$

$$f_{distributed\ load} = \frac{q_S \times L^4}{384 \times E \times I_y} = \frac{1. \times 7.5^4}{384 \times 2.1 \times 11770} \times 10^2 = 0.037cm$$

$$f = 0.487cm \ ; \ f_{allowable} = \frac{L}{250} = \frac{750}{250} = 3cm$$

Therefore $f = 0.487cm \leq f_{allowable} = 3cm$ The condition is verified.

3.3.4.2. Final phase:

➢ **Width of the slab; effective width**

$$b_{eff} = inf \begin{cases} \frac{2l_0}{8} = \frac{2 \times 7.5}{8} = 1.875m \\ b = 4.75m \end{cases} \rightarrow b_{eff} = 1.875m$$

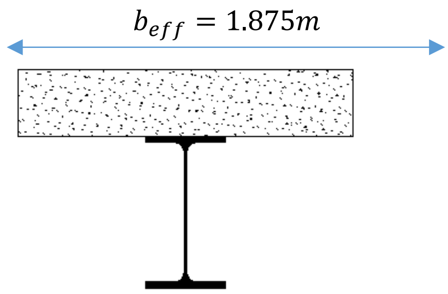


Figure 3. 8 : Effective width for the beam.

➢ **Position of the plastic neutral axis**

- $R_{concrete} = 0.57 \times f_{ck} \times b_{eff} \times h_c$

$$R_{concrete} = 0.57 \times 25 \times 1875 \times 65 \times 10^{-3} = 1736.71kN$$

- $R_{steel} = 0.95 \times f_y \times A_a$

$$R_{steel} = 0.95 \times 275 \times 5381 \times 10^{-3} = 1407.35kN$$

$$\boldsymbol{R_{concrete} > R_{steel}\ neutral\ axis\ is\ in\ the\ concrete\ slab}$$

a) Bending strength check

$M_{sd} \leq M_{pl,rd}$

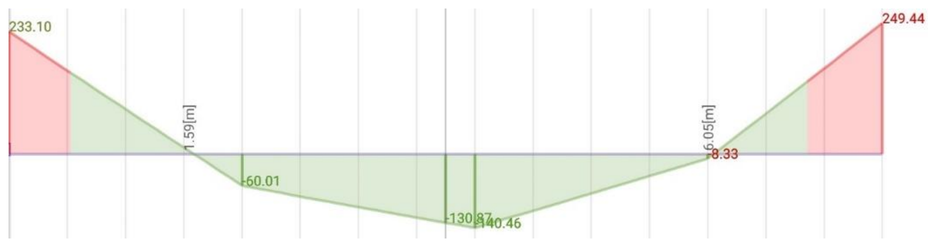


Figure 3. 9: Bending moment diagram for the main beam. (Final phase)

- At mid-span : $M_{sd1} = -140.46kN.m$

- On the supports :$M_{sd2} = 249.44kN.m$

$M_{pl,rd} = R_{steel} \times \left[\frac{h_a}{2} + h_c + h_p - \left(\frac{R_{steel}}{R_{concrete}} \times \frac{h_c}{2}\right)\right]$

$M_{pl,rd} = 1407.35 \times \left[\frac{300}{2} + 65 + 55 - \left(\frac{1407.35}{1736.71} \times \frac{65}{2}\right)\right] \times 10^{-3} = 342.91kN.m$

$M_{sd} = -249.44kN.m \leq M_{pl,rd} = 342.91kN.m$ The condition verified.

Section performance $\frac{M_{sd}}{M_{pl,rd}} = 73\%$

b) Shear strength check

We must check that: $V_{sd} \leq V_{pl,rd}$

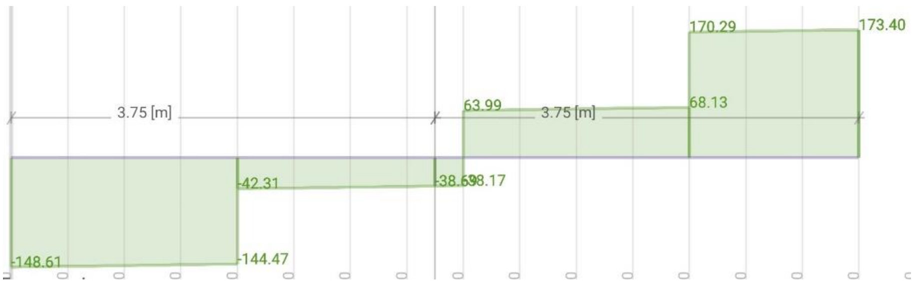


Figure 3. 10: shear force diagram for the main beam. (Final phase)

$V_{sd} = 173.4kN$

$V_{pl,rd} = 407.72kN$

$V_{sd} = 173.4kN \leq V_{pl,rd} = 407.72kN$ The condition is verified.

c) Bending moment-shear force interaction check

$0.5V_{pl,rd} = 173.4kN > 203.86kN$ Therefore it is not necessary to check the interaction.

d) Beam deflection check

We must check that: $f_{total} \leq f_{allowable}$

$I_c = \frac{A_a \times (h_c + 2\times h_p + h_a)^2}{4\times(1+m\times v)} + \frac{b_{eff} \times {h_c}^3}{12\times m} + I_a$

With:$v = \frac{A_a}{A_b} = \frac{6261}{1875\times55} = 0.06$; $m = \frac{E_a}{E_b} = 15$

$I_c = \frac{6261\times(65+2\times55+330)^2}{4\times(1+15\times0.06)} + \frac{1875\times65^3}{12\times15} + 11770 \times 10^4 = 33065.42 \times 10^4 mm^4$

$f_{final} = f_{concentrated\ load} + f_{distributed\ load}$

$f_{concentrated\ load} = \frac{19P_S \times L^3}{384EI} = \frac{19\times72.9\times7.5^3}{384\times2.1\times33065.42} \times 10^2 = 0.292cm$

$f_{distributed\ load} = \frac{q_S \times L^4}{384\times E\times I_y} = \frac{1.49\times7.5^4}{384\times2.1\times33065.42} \times 10^2 = 1.76 \times 10^{-4} cm$

$f_{final} = 0.292cm$

$f_{total} = 0.487 + 0.292$

$f_{total} = 0.779cm \leq f_{allowable} = 3cm$ The condition is verified.

e) Verification of stability against lateral torsional buckling

It is not necessary to check the stability against lateral torsional buckling for the main beam because the beam is held laterally by the joists. It is considered that the beam does not risk buckling

All the resistance and stability conditions imposed by CCM97 and eurocode04 are verified; we will then use the IPE360 profile for the box springs with the following characteristics:

Profile	Weight (kg/ml)	Section A (cm²)	Dimensions				characteristics					
			h (mm)	b (mm)	t_w (mm)	t_f (mm)	I_y (cm^4)	I_z (cm^4)	W_{ply} (cm)	W_{plz} (cm)	i_y (cm)	i_z (cm)
IPE300	42.2	53.81	300	150	7.1	10.7	8355	603.8	628.4	125.2	12.46	3.35

Table 3. 2 : IPE 300 Profile characteristics.

We followed the same steps for the common floor we find that the profile IPE300 checks the conditions

3.3.5. Shear connectors

- **Headed stud strength** $\boldsymbol{P_{rd}}$

$P_{rd} = 72.2kN$

- Shearing force taken up by the connectors

$R_l = \boldsymbol{min}[\boldsymbol{R_{concrete}}; \boldsymbol{R_{steel}}] = \boldsymbol{min}[1736.71kN; 1201.3kN] = 1201.39kN$

- Number of connectors per half-span

$N^{bre}{}_{connectors} = \frac{R_l}{P_{rd}} = \frac{1201.3}{72.2} = 16.63 \cong 17$ Per half-span, that is 34 all along the beam.

- Connector spacing

$Esp = \frac{L}{N^{bre}-1} = \frac{7500}{34-1} = 227.27mm$

We opt for 34 connectors 220mm apart.

3.4. Pre-sizing of beams of the ramp

The structure also includes two ramps which connect between each half-storey of the structure, the ramp consists of a composite slab and a steel profile on each side.

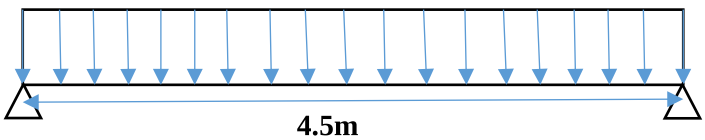


Figure 3. 11 : Static loads diagram for the beams of the ramps.

The center distance $e = 3.75m$

For pre-sizing we use the expression that we used for the other beams: $\frac{L}{25} \leq h \leq \frac{L}{15}$

L=4.5m $180m \leq h \leq 300m$

We consider an IPE180

The Profile was checked according to the previous method.

All the resistance and stability conditions imposed by the CCM97 and the EUROCODE04 are verified; we will then use the IPE180profile for the beams of the ramp.

3.4.1. Shear connectors

We opt for 22 connectors spaced 200mm apart.

3.5. Pre-sizing of the columns

3.5.1. Calculation of vertical loads [2]

The most stressed column is 3-E; 3-N; 5-E; 5-N; that carries and area of:

$$S = 6.25 \times 5.025 = 31.4m^2$$

- Dead loads :

Weight of the roof slab ....4.66 × 31.4 = 146.324kN
Weight of the common floor slab ....4.38× 31.4 = 137.532kN
Weight of the beam (IPE 300)...0,422×6.25= 2.63kN
Weight of the joist (IPE 180)...0,188 ×7.275= 1.36kN

- Live loads

Floor level	*overloads*	$\sum overloads$	$\sum overloads$	$\sum Overloads\ (\frac{kN}{ml}$
Roof	Q_0 =2.5	$\sum = Q_0$	2.5	78.5
6th	Q_1 =2.5	$\sum = Q_0 + Q_1$	5	157
5th	Q_2 =2.5	$\sum = Q_0 + 0.95(Q_1+Q_2)$	7.25	227.65
4th	Q_3 =2.5	$\sum = Q_0 + 0.9(Q_1 + Q_2 + Q_3)$	9.25	290.45
3rd	Q_4 =2.5	$\sum = Q_0 + 0.85(Q_1+Q_2+Q_3+Q_4)$	11	345.4
2nd	Q_5 =2.5	$\sum = Q_0+0.8(Q_1+Q_2+Q_3+Q_4+Q_5)$	12.5	392.5
1st	Q_6 =2.5	$\sum = Q_0 + 0.75(Q_1 + Q_2 + Q_3+Q_4+Q_5+Q_6)$	13.75	431.75
-1st	Q_7 =2.5	$\sum = Q_0 + 0.7(Q_1 + Q_2 + Q_3+Q_4+Q_5+Q_6+Q_7)$	14.75	463.15
-2nd	Q_8 =2.5	$\sum = Q_0 + 0.65(Q_1 + Q_2 + Q_3+Q_4+Q_5+Q_6+Q_7+Q_8)$	15.5	486.7

Table 3. 3 : Live loads regression.

- The calculation of vertical loads is represented in the following table:

Column	G (KN)	Q (KN)	N=1,35G+1,5Q (KN)
6th	151.794	78.5	**322.67**
5th	294.8	157	**633.48**
4th	437.8	227.65	**932.50**
3rd	580.81	290.45	**1219.80**
2nd	723.814	345.4	**1495.24**
1st	866.819	392.5	**1758.95**
Ground level	1009.824	431.75	**2010.88**
1st basement	1152.829	463.15	**2251.04**
2nd basement	1295.834	486.7	**2479.42**

Table 3. 4: Calculation of vertical loads.

3.5.2. Pre-sizing

Columns are elements stressed in axial compression; the design value Nsd of the compression force in each cross section must satisfy the following condition:

$N_{sd} \leq N_{c,rd} = \frac{A \times f_y}{\gamma_{m0}}$ →We have: $A \geq \frac{N \times \gamma_{m0}}{f_y}$ [**6**]

Nsd : compression force

f_y = 235 N/mm².

γ_{Mo} = 1.1

We chose to change the column's section every 3 levels, the ground level column will be continued as a composite column to the basement

Level	**Calculated A(cm²)**	*Choice*	*class*	*Selected column section (cm²)*
6th 5th 4th	43.64	***HEA 220***	**I**	64.34
3rd 2nd 1st	91.07	***HEA 300***	**I**	112.5
Ground level 1st basement 2nd basement	104.33	***HEA 320***	**I**	124.4

Table 3. 5: Pre-sizing of the columns.

3.5.3. Verification of the stability against flexural buckling

According to CCM97, the columns being compressed must be checked with the following expression:$N_{sd} \leq N_{b,rd}$

$N_{b,rd} = \frac{\chi \times \beta_A \times A \times f_y}{\gamma_{m1}}$ $with\ \beta_A = 1\ \ and\ \gamma_{m1} = 1.1\ \ for\ class 1$ [**6**]

We check the buckling along the axis which correspond to the lowest inertia of the profile therefore along the axis z-z

Verification for the ground level columns HEA320:

- Buckling length

$$L_f = 0.7L = 0.7 \times 2.8 = 1.96m$$

- Reduced slenderness

$$\bar{\lambda} = \frac{\lambda}{\lambda_1}\begin{cases} \lambda_1 = 93.91\varepsilon\ \rightarrow \varepsilon = 1 \\ \lambda = \frac{L_f}{i_z} = \frac{196}{7.49} = 26.17 \end{cases} \rightarrow \bar{\lambda} = \frac{26.17}{93.91} = 0.28 > 0.2\ so\ there\ is\ a\ risk\ of\ buckling$$

- Reduction factor for the relevant buckling mode

  $\chi = \frac{1}{\left(\phi + \sqrt{\phi^2 - \bar{\lambda}^2}\right)} \leq 1\quad such\ as : \phi = 0.5 \times \left(1 + \alpha\left(\ \bar{\lambda} - 0.2\ \right) + \bar{\lambda}^2\right)$

The imperfection factor α corresponding to the appropriate buckling curve, it is determined in the (CCM 97 Table 55.1 and Table 55.3)$\alpha = 0.49$

$$\phi = 0.5 \times (1 + 0.49 \times (\ 0.28 - 0.2\ ) + 0.28^2) = 0.56 \rightarrow \chi = \frac{1}{(0.56 + \sqrt{0.56^2 - 0.28^2})} = 0.95 \leq 1$$

Therefore: $N_{b,rd} = \frac{0.95\times1\times12440\times235}{1.1}\times10^{-3} = 2524.75kN$

$N_{sd} = 2244.47 \leq N_{b,rd} = 2524.75kN$

The **HEA 360** profile meets all CCM97 resistance and stability requirements.

In this table, the summary of the checks for stability against buckling for each column

Floor	Profile	$N_{sd}(+selfweight\ of\ profile)$	$\bar{\lambda}$	ϕ	χ	$N_{b,rd}$	OBS
6th, 5th, 4th	*HEA 220*	936.742	0.38	0.62	0.9	1237.08	V
3rd, 2nd, 1st	*HEA300*	1770.61	0.28	0.56	0.95	2283.23	V
GL	*HEA320*	2026.024	0.28	0.56	0.95	2524.75	V

Table 3. 6 : Verification of the stability against flexural buckling.

The chosen **HEA profiles for columns** meet all the condition of resistance and stability imposed by CCM97

The table below sums up the main characteristics of the chosen columns:

Profile	Weight (kg/ml)	Section A (cm²)	Dimensions				Characteristics					
			h (mm)	*b (mm)*	*t_w (mm)*	*t_f (mm)*	*I_y (cm^4)*	*I_z (cm^4)*	*W_{ply} (cm)*	*W_{plz} (cm)*	*i_y (cm)*	*i_z (cm)*
HEA 220	*50.5*	*64.34*	*210*	*220*	*7*	*11*	*5410*	*1955*	*568.5*	*270.6*	*20.67*	*5.51*
HEA 300	*88.3*	*112.5*	*290*	*300*	*8.5*	*14*	*13670*	*4736*	*1112*	*518.1*	*11.86*	*7.49*
HEA 320	*97.6*	*124.4*	*310*	*300*	*11.5*	*20.5*	*22930*	*6985*	*1628*	*465.7*	*13.58*	*7.49*

Table 3. 7 : Characteristics of the profiles used for columns.

3.6. Conclusion

This chapter allowed us to initially determine the sections of the elements of the structure, using firstly imperial expressions to approach the section value, and then conditions imposed by design regulations in order to be able to model the building with profiles close to the resistant sections

But checking the frame elements with these loads alone is not sufficient to make the final sizing that is why we will start the dynamic analysis of the structure.

CHAPTER 04:

Dynamic analysis

4. DYNAMIC ANALYSIS

4.1. Introduction

A dynamic analysis is linked to the inertial forces developed by a structure when it is excited by means of dynamic loads applied suddenly (for example, wind action, an explosion, an earthquake).In our case, the earthquake action is preponderant, so we are talking about a **seismic analysis.**

One of the major threats to a structure are earthquakes. The latter can occur at any time and with unpredictable intensity causing material and human damage.

Figure 4. 1 : Tohoku Japan earthquake 2011.

Figure 4. 2 : Tohoku Japan earthquake 2011.

The engineer's role is to adequately design structures to withstand these earthquakes.

The purpose of a seismic analysis is:

- Estimating the possible levels and modes of deformation of the structure on a given soil.
- Knowing the areas of the structure most exposed to rupture in case of strong tremors.

According to RPA99 / VERSION 2003[**9**] there are 3 methods for the evaluation or the calculation of the seismic forces acting on the building:

- Lateral force analysis method.
- Modal response spectrum analysis method.
- Seismic analysis using accelerograms.

In our case the ***Modal response spectrum analysis*** method will be used for the evaluation of the seismic forces acting on our structure.

4.2. Modal response spectrum analysis :

This method is undoubtedly the most frequently used method for the seismic analysis of structures in Algeria. Response spectrum analysis is a linear-dynamic statistical analysis method , the principle of this method resides in the determination of the natural modes of vibration of an essentially elastic structure and the maximum of the effects generated by the seismic action, this one being presented by a response spectrum.

Eigen modes depend on the mass of the structure, damping and inertial forces.

This method is based on the following hypothesis:

- Concentration of masses at floor level.

- Only the horizontal displacements of the nodes are taken into account.

- The number of modes to be taken into account is such that the sum of the coefficients of these modes is at least equal to 90%.

- Or that all the modes having an effective modal mass higher than 5% of the total mass of the structure are retained for the determination of the total response of the structure.

- The response spectrum is automatically generated by ROBOT STRUCUTRAL ANALYSIS based on the RPA2003 regulations and data about the structure and the soil.

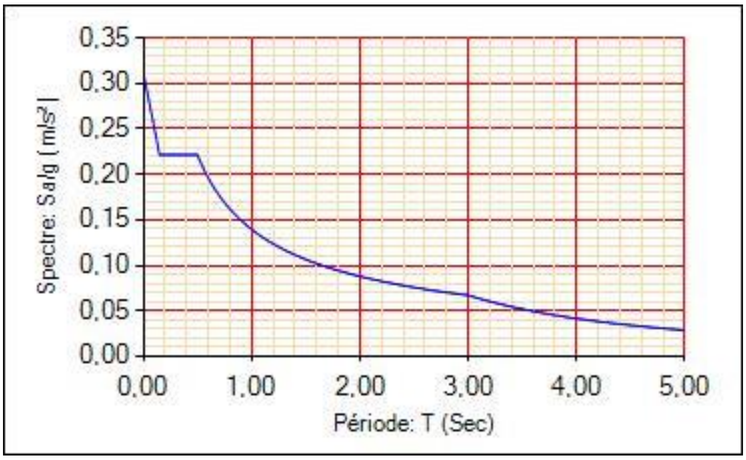


Figure 4. 3 : Graphic representation of the response spectrum.

With:

- g: acceleration of gravity, (g = 10N / s2)
- A: zone acceleration coefficient.
- η: damping correction factor.
- A: Coefficient of behavior of the structure. Depends on the bracing system.
- T1, T2: Characteristic periods associated with the site category.
- Q: Quality factor. (Q)

4.3. Base shear calculation using the equivalent static lateral force method

In the seismic code (RPA99V2003[**9**]), the lateral force $\boldsymbol{V_t}$ is used as a reference value of the total seismic design base shear. The base shear calculated using the modal response spectrum analysis should not be less than 80 % of the lateral base shear force.
It is calculated using the following equation:

$$\boldsymbol{V_t} = \frac{A \times D \times Q}{R} \times W$$

- $\boldsymbol{A}$: Zone acceleration coefficient according to the seismic zone and the using group of the building. In our case the building belongs to group 2: *common structure or medium importance (parking)* , and the Seismic zone is *zone III (strong seismicity) TIPAZA.*
  Therefore: $\boldsymbol{A = 0.25}$
- $\boldsymbol{D}$: Average dynamic amplification factor, depending on the site category, the damping correction factor η and the fundamental period of the structure T

$$D = \begin{bmatrix} 2{,}5\,\eta & 0 \le T \le T_2 \\ 2{,}5\,\eta\,(T_2/T)^{\frac{2}{3}} & T_2 \le T \le 3s \\ 2{,}5\eta(T_2/3)^{\frac{2}{3}}(3/T)^{\frac{2}{3}} & T \ge T_2 \end{bmatrix}$$

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

Where ξ (%) is the critical damping ratio (%) depending on constitutive material, structure type and importance of infil. In our case $\xi = 4$ (tab.4.2) therefore: η =1, 08

T_2: Characteristic period associated to the category of the site, in our case it's a soft site (s3) so

$T_2 = 0.5s$

- The value of the fundamental period (T) of the structure can be estimated from empirical expression or calculated by analytical or numerical methods.

According to *Dr.Taleb Rafik" :* [**11**]

$$T = \begin{cases} T_{analytical}\ if\ T_{analytical} \le T_{empirical} \\ T_{empirical}\ if\ T_{empirical} < T_{analytical} < 1.3T_{empirical} \\ 1.3T_{empirical}\ if\ T_{analytical} \ge 1.3T_{empirical} \end{cases}$$

The empirical expression to be used depending on the case is as follows:

$T_{emperical} = C_T \times {h_N}^{3/4}$

Such as:

- h_N Is the height of the building in meters from the foundation or from the top rigid basement. $h_N = 21m$
- $C_T = 0.085$ According to the table 4.6 from RPA2003

$$T_{emperical} = 0.085 \times 21^{3/4} = 0.83$$

$T_{analytical}$ Is determined by the dynamic analysis of the structure.

- ***Q***: Quality factor of the structure given by the following formula:

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

Pq: Penalty to be applied depending on whether the criteria of quality q "is satisfied or not"

Criteria Q	*Penalty*	
	P_x	P_y
1. Minimal conditions on bracing lines	0	0
2. Redundancy in plan	0	0
3. Regularity in plan	0.05	0.05
4. Regularity in elevation	0	0
5. Control of material quality	0	0
6. Control of construction quality	0	0
Q=1+	0.05	0.05

Table 4. 1 : Penalty to be applied to the quality factor.

According to the table 4.4 of the RPA2003 $Q_x = Q_y = \mathbf{1.05}$

- ***W***: Total weight of the structure that is equal to the sum of the weights $\boldsymbol{W_i}$ calculated at every floor

$W = \sum W_i$ with $W_i = W_{Gi} + \beta W_{Qi}$

β: Weighting coefficient depending on the nature and the duration of the live load $\beta = 0.6$
The weight of the structure is given by the software used (ROBOT STRUCTURAL ANALYSIS)

- ***R:*** Global behavior coefficient of the structure according to the lateral force resisting system in our case, the case of and X-braced system (9.a) **R=4.**

4.4. Initial model

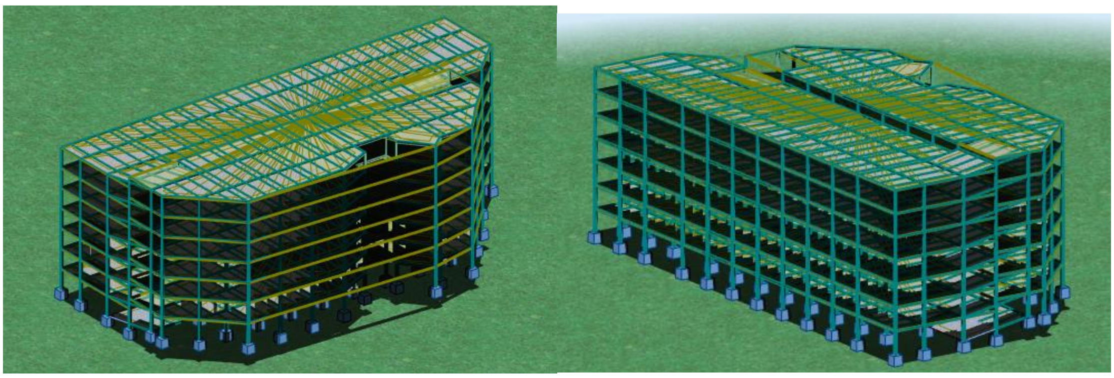


Figure 4. 4:3D Initial model of the structure on ROBOT STRUCTURAL ANALYSIS

4.4.1. Modal analysis

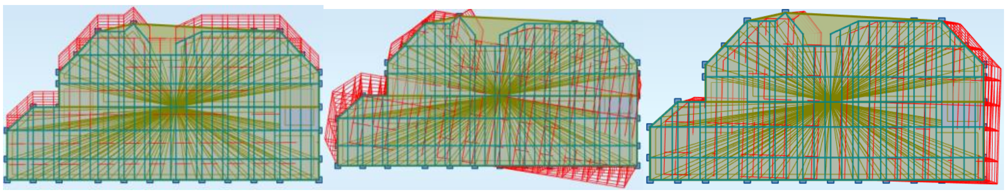
	Frequency	Period	effective mass%x	effective mass% y	sum x	sum y	Direction
1	0,31	3,22	0,01	72,81	0,01	72,81	**Disp according to y**
2	0,53	1,90	2,16	0,03	2,18	72,84	**Torsion around z**
3	0,67	1,50	74,56	0,03	76,74	72,87	**Disp according to x**
4	0,89	1,13	0,00	12,99	76,74	85,86	
5	1,48	0,67	0,11	0,09	76,85	85,95	
6	1,49	0,67	0,00	0,44	76,85	86,39	
7	1,61	0,62	0,00	4,74	76,85	91,14	
8	1,92	0,52	11,34	0,00	88,19	91,14	
9	2,37	0,42	0,00	2,55	88,20	93,68	
10	2,61	0,38	0,00	0,02	88,20	93,71	
11	3,07	0,33	0,05	1,47	88,24	95,17	
12	3,40	0,29	3,71	0,05	91,96	95,23	

Table 4. 2: Dynamic results of the initial model.

- Observations :

The modal analysis of the structure led to:

- The effective mass participation exceeds 90% from the 12th mode.
- The first mode is a translation mode parallel to the Y axis of 72.81% of mass participation.
- The second mode is a rotation (torsion) mode.
  The third mode is a mode of translation parallel to the X axis of 74.56%.



Mode 1 Mode 2 Mode 3

Figure 4. 5: The three first modes of vibration of the initial model.

4.4.2. Seismic analysis

4.4.2.1. Base shear check

The weight of the initial model is given by the software: $W = 65890.62kN$

$$Q_{x,y} = 1.05\ ; A = 0.25\ ;\ R_{x,y} = 4\ ; D_{x,y} = 1.65$$

The lateral force will be calculated using the previous expression (5.3)

Direction	V(response-spectrum method)	V_t(lateral force method)	$0.8 \times V_t$	$\frac{0.8V}{V_t}$	Observation
x	5470,35kN	7134.71kN	5707.77kN	$1.04 < 1$	Increase the seismic action
y	3830,11kN	7134.71kN	5707.77kN	$1.47 < 1$	Increase the seismic action

Table 4. 3 : Base shear verification in the initial model.

4.4.2.2. Inter-storey drift check

In case of a parking we have 2 different levels in each storey. For that, we're going to assume that we have two independent blocs with a floor height of 2.8m (bloc A and B)

We must check that: $\Delta_k^x \leq \overline{\Delta} \;\; et \;\; \Delta_k^y \leq \overline{\Delta}$

Where : $\overline{\Delta} = 0.01 h_e = 2.8cm$

With : $\delta_k^x = R\delta_{ek}^x \;\; ; \;\; \delta_k^y = R\delta_{ek}^y$ and $\Delta_k^x = \delta_k^x - \delta_{k-1}^x \;\; ; \;\; \Delta_k^x = \delta_k^y - \delta_{k-1}^y$

$\boldsymbol{\Delta_k^x}$ **:** the relative horizontal displacement of two adjacent floors in a building it Corresponds to the relative displacement of the level K compared to the level K-1 in the direction x-x (same thing for the y-y direction).

$\boldsymbol{\delta_{ek}^x}$ **:** is the horizontal displacement due to seismic forces at level K in the direction x-x.

If the lateral displacements between floors exceed the allowable values, it is necessary to increase the lateral rigidity of the structure. For this we can:

- Increase the dimensions of the existing posts.
- Add sails in the structure.

The inter-storey drift is calculated automatically by the software.

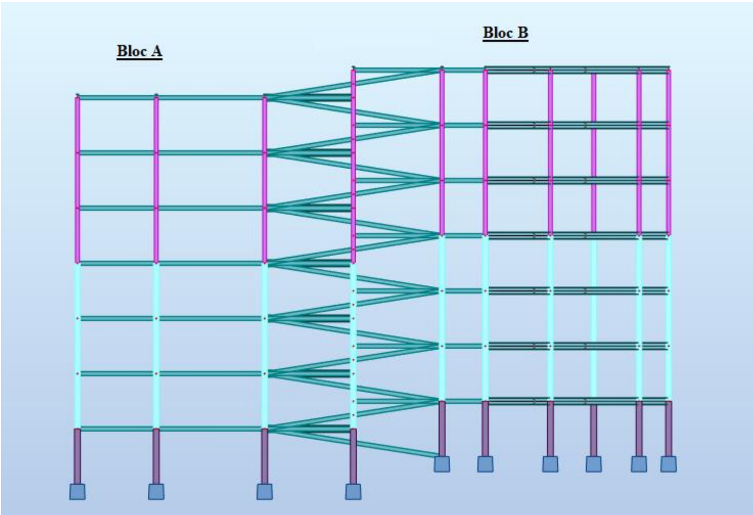


Figure 4. 6 : Bloc A and B representation.

Bloc A

Floor level	Allowable inter-storey drift (cm)	Inter-storey drift (x) (cm)	Observation	Inter-storey drift (y) Δ_k^y (cm)	Observation
1 (+2.8)	2.8	2,3	Verified	4,3	Not verified
2(+5.6)	2.8	4,7	Not verified	10,6	Not verified
3(+8.4)	2.8	5,3	Not verified	13,4	Not verified
4(+11.2)	2.8	4,7	Not verified	13,7	Not verified
5(+14.0)	2.8	5,3	Not verified	15,0	Not verified
6(+16.8)	2.8	4,1	Not verified	14,0	Not verified
7(+19.6)	2.8	2,4	Verified	10,7	Not verified

Table 4. 4: Inter-storey drift verification for Bloc A.

Bloc B

Floor level	Allowable inter-storey drift (cm)	Inter-storey drift (x) (cm)	Observation	Inter-storey drift (y) Δ_k^y (cm)	Observation
1(+4.2)	2.8	4,0	Not verified	9,0	Not verified
2(+7.0)	2.8	6,0	Not verified	13,1	Not verified
3(+9.8)	2.8	5,8	Not verified	13,5	Not verified
4(+12.6)	2.8	5,5	Not verified	14,3	Not verified
5(+15.4)	2.8	5,9	Not verified	15,2	Not verified
6(+18.2)	2.8	4,1	Not verified	12,4	Not verified
7(+21.0)	2.8	2,2	Verified	8,7	Not verified

Table 4. 5:Inter-storey drift verification for Bloc B.

4.4.2.3. Observations

- The inter-storey drift is not verified, the drift value is way higher than the allowable storey-drift value, and it means that our structure is too flexible.
- Second mode of vibration is a torsion.
- The structure is more flexible on the Y direction and less flexible in the X direction.

X-shaped bracing is added to the initial model, after many tries we used:

2UPN220 in the Y direction and 2UPN160 in the X direction.

The section of the columns is changed to increase the stiffness of the building; we used:

- HEB 450 for the ground floor;
- HEB 400 for the1st ,2nd and 3rd;
- HEB 360 4th, 5th and 6th floor.

4.5. Final model

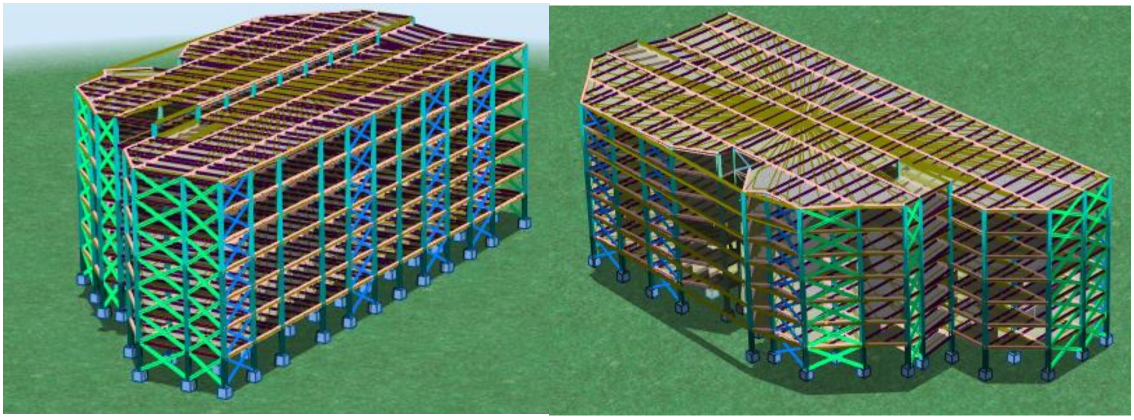


Figure 4.7:3D final model of the structure on ROBOT STRUCTURAL ANALYSIS.

4.5.1. Modal analysis

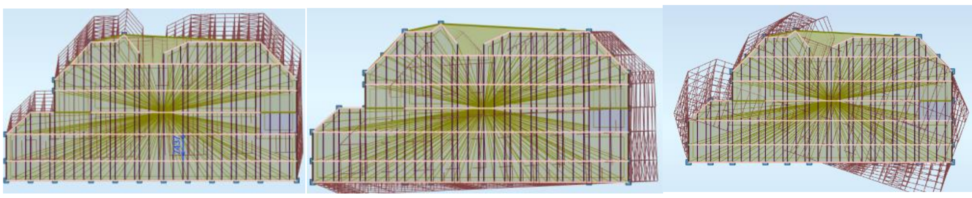
Mode	Frequency	Period	effective mass%x	effective mass% y	Sum x	Sum y	Direction
1	1,25	0,80	0,93	73,26	0,93	73,26	**Disp according to y**
2	1,48	0,68	71,61	1,21	72,54	74,47	**Disp according to x**
3	1,52	0,66	0,00	0,43	72,54	74,90	**Torsion around z**
4	2,41	0,42	2,29	0,30	74,83	75,21	
5	4,09	0,24	0,00	0,00	74,83	75,21	
6	4,20	0,24	0,01	16,44	74,84	91,65	
7	4,77	0,21	14,29	0,00	89,13	91,65	
8	6,83	0,15	0,00	0,05	89,13	91,70	
9	7,53	0,13	0,06	3,69	89,19	95,40	
10	7,78	0,13	0,20	0,19	89,39	95,59	
11	8,51	0,12	4,00	0,33	93,40	95,91	

Table 4. 6: Dynamic results of the final model

- Observations :

The modal analysis of the structure led to:

- The effective mass participation exceeds 90% from the 11th mode.
- The first mode is a translation mode parallel to the Y axis of 73.26% of mass participation.
- The second mode is a mode of translation parallel to the X axis of 71.1%.
- The third mode is a rotation (torsion) mode.
- $T_{analytical} < T_{empirial}$



Mode 1 Mode 2 Mode 3

Figure 4.8: the three first modes of vibration of the final model.

4.5.2. Seismic analysis

4.5.2.1. Base shear check

The weight of the final model is given by the software: $W = 67616{,}07kN$

$$Q_{x,y} = 1.05\,; A = 0.25\,;\ R_{x,y} = 4\,; D_{x,y} = 1.97$$

The lateral force will be calculated using the previous expression (5.3)

Direction	V(response-spectrum method)	V_t(lateral force method)	$0.8 \times V_t$	$\frac{0.8V_t}{V}$	Observation
x	8986,86	8757.96	7006.39	$0.77 < 1$	**No need to increase the seismic action**
y	8449,57	8757.96	7006.39	$0.82 < 1$	

Table 4. 7: Base shear verification.

4.5.2.2. Inter-storey drift check

Bloc A

Floor level	Allowable inter-storey drift (cm)	Inter-storey drift (x) (cm)	Observation	Inter-storey drift (y) Δ_k^y (cm)	Observation
1(+2.8)	2.8	0,8	Verified	1,2	Verified
2(+5.6)	2.8	1,7	Verified	1,8	Verified
3(+8.4)	2.8	2,0	Verified	2,2	Verified
4(+11.2)	2.8	2,0	Verified	2,4	Verified
5(+14)	2.8	2,0	Verified	2,4	Verified
6(+16.8)	2.8	1,8	Verified	2,2	Verified
7(+19.6)	2.8	1,5	Verified	2,0	Verified

Table 4. 8:Inter-storey drift verification for Bloc A.

Bloc B

Floor level	Allowable inter-storey drift (cm)	Inter-storey drift (x) (cm)	Observation	Inter-storey drift (y) Δ_k^y (cm)	Observation
1(+4.2)	2.8	1,2	Verified	1,8	Verified
2(+7)	2.8	1,7	Verified	2,3	Verified
3(+9.8)	2.8	1,8	Verified	2,4	Verified
4(+12.6)	2.8	1,7	Verified	2,4	Verified
5(+15.4)	2.8	1,6	Verified	2,4	Verified
6(+18.2)	2.8	1,4	Verified	2,2	Verified
7(+21)	2.8	1,1	Verified	2,0	Verified

Table 4. 9:Inter-storey drift verification for Bloc B.

4.5.2.3. Observations

- The inter-storey drift is verified for the Bloc A and B
- All the conditions imposed by the regulation RPA99V2003 are verified.

4.6. Conclusion:

After subjecting the structure to earthquake loads in both direction X and Y it can be clearly seen that the preliminary sizing results of the columns were unsafe , inadequate and did not satisfy the rigidity condition of the building until we increased their size and added X shaped bracing in both directions of the building.

Therefore we can say that a dynamic analysis is indispensable for the design of any building.

CHAPTER 05:

Verification of the structural elements.

5. VERFICATION OF THE STRUCTURAL ELEMENTS

5.1. Introduction

After subjecting the structure to dynamic loads, or more exactly seismic loads, additional stress will act on the structural elements that has been pre-sized in chapter 3 .Therefor, a check or a verification must be done in order to ensure that our structural elements will not fail under seismic loading.

We need to do a resistance check (to internal forces) and a stability check (buckling).

5.2. Loads combinations

We talk about loads combination when more than one load type is acting on the structure. Building codes usually specify a variety of load combinations together with load factors (weightings) for each load type in order to ensure the safety of the structure under different maximum expected loading scenarios.

The combinations used in our case are which are recommended by the RPA2003:

- ULS : $\mathrm{G+Q}$
- SLS : $1.35\mathrm{G}+1.5\mathrm{Q}$
- SEISMIC LOADS : $\begin{cases} \mathrm{G+Q\pm E_{x,y}} \\ 0.8\mathrm{G}\pm \mathrm{E_{x,y}} \end{cases}$

5.3. Verification of the columns

The columns are subjected to compound bending where each column is subjected to a axial force "N" and two bending moments My and Mz. Verification is done for all load combinations listed in the regulations under the most unfavorable stresses in both directions.

The different forces acting on the columns must be combined in the most unfavorable cases, which are:

- Case 1: Maximum compression Nsd and a corresponding My.sd and Mz.sd moments.
- Case 2: A maximum My.sd moment and a corresponding Nsd and Mz.sd compression.
- Case 3: A maximum Mz.sd moment and a corresponding Nsd and My.sd compression.

5.3.1. Stability check

Members which are subjected to combined bending and axial compression should satisfy:

$$\frac{N_{sd}}{\chi_{min}\times\frac{A\times f_y}{\gamma_{m1}}}+\frac{K_y\times M_{sd,y}}{W_{pl,y}\times\frac{fy}{\gamma_{m1}}}+\frac{K_z\times M_{sd,z}}{W_{pl,z}\frac{fy}{\gamma_{m1}}}\leq 1$$

Where:

- $N_{sd}; M_{sd,y}; M_{sd,z}$ are the design values of the compression force and the maximum moments about the y-y and z-z axis along the member, respectively
- $K_{y,z}=1-\frac{\mu_{y,z}\times N_{sd}}{\chi_{y,z}\times A\times f_y}\leq 1.5$
- $\mu_{y,z}=\bar{\lambda}_{z,y}\times\left(2\times\beta_{my,z}-4\right)+\left(\frac{W_{pl,yz}-W_{el,yz}}{W_{el,yz}}\right)\leq 0.9$
- $\beta_{my,z}$ Are equivalent uniform moment factors for flexural buckling
- $\chi_{min}=min\left[\chi_y;\chi_z\right]$

NB: According to CCM "article 5.2.5.3" a metal frame can be classified as braced if the bracing system reduces its horizontal displacement by at least 80%. In this case the buckling length calculation is done by the method of fixed nodes.

As an example we checked the 6th floor column:

profile	A (cm^2)	I_y (cm^4)	I_z (cm^4)	$W_{pl,y}$ (cm^3)	$W_{pl,z}$ (cm^3)	i_y (cm)	i_z (cm)
HEB360	180.6	43190	10140	2683	1032	15.46	7.49

Table 5. 1 : Characteristics of the HEB360 profile.

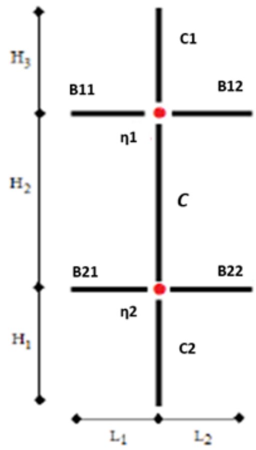
For slenderness$\bar{\lambda}_{max} > 0.2$ and $\frac{N_{sd}}{N_{b,rd}} > 0.1$the buckling effects may be ignored and only cross sectional checks apply.

y-y: $\bar{\lambda}_y = \frac{\lambda_y}{93.9\times\varepsilon} = \frac{L_{fy}/i_y}{93.9\times\varepsilon}$ with : $\varepsilon = \sqrt{\frac{235}{\varepsilon}} = 1$

z-z: $\bar{\lambda}_z = \frac{\lambda_z}{93.9\times\varepsilon} = \frac{L_{fz}/i_z}{93.9\times\varepsilon}$

We shall determinate the buckling length L_{fy} ; L_{fz} using the fixed nodes method such as:

$$\frac{L_f}{L_0} = \frac{1+0.145\times(\eta_1+\eta_2)-0.265\times\eta_1\times\eta_2}{2-0.364\times(\eta_1+\eta_2)-0.247\times\eta_1\times\eta_2}$$



With:

- $\eta_1 = \frac{k_c+k_{c1}}{k_c+k_{c1}+k_{b11}+k_{b12}}$
- $\eta_2 = \frac{k_c+k_{c2}}{k_c+k_c+k_{b21}+k_{b22}}$
- k_c :the stiffness of the concerned column.
- k_{c1} ; k_{c2}:the stiffness of the adjacent columns.
- k_{b11}; k_{b12} ; k_{b21}; k_{b22} : The rigidity of the beams associated with the node considered.

NB: $\eta = 0$ if the base is fixed

$\eta = 1$ if the base is hinged

y-y:

$k_c = k_{c2} = \frac{I_c}{H} = \frac{43190}{280} = 154.25cm^3$

$k_{c1} = 0$

$k_{b11} = k_{b21} = \frac{I_{b(IPE300)}}{L} = \frac{8356}{750} = 11.15cm^3$

$k_{b12} = k_{b22} = 0$

$\begin{cases}\eta_1 = 0.93\\ \eta_2 = 0.96\end{cases} \Rightarrow \frac{L_{fy}}{L_0} = 0.95 \Rightarrow L_{fy} = 2.661m$

z-z:

$k_c = k_{c2} = \frac{I_c}{H} = \frac{10140}{280} = 36.21cm^3$

$k_{b11} = k_{b21} = \frac{I_{b(IPE180)}}{L} = \frac{1317}{550} = 2.39$

$k_{b12} = k_{b22} = \frac{I_{b(IPE180)}}{L} = \frac{1317}{455}$

$= 2.89cm^3$

$\begin{cases}\eta_1 = 0.87\\ \eta_2 = 0.93\end{cases} \Rightarrow \frac{L_{fz}}{L_0} = 0.91 \Rightarrow L_{fz} = 2.559m$

$$\begin{cases}\dfrac{L_{fy}/i_y}{93.91\times\varepsilon} = \dfrac{266.1/15.46}{93.91\times 0.92} = 0.2 = 0.2\ there\ is\ a\ risk\ of\ buckling\ according\ to\ axis\ y-y\\ \dfrac{L_{fz}/i_z}{93.91\times\varepsilon} = \dfrac{255.9/7.49}{93.91\times 0.92} = 0.39 > 0.2\ there\ is\ a\ risk\ of\ buckling\ according\ to\ axis\ z-z\end{cases}$$

Buckling curve choice:

$$\frac{h_{(HEB280)}}{b_{(HEB280)}} = \frac{360}{300} = 1.2 = 1.2 \ \ with\ t_f = 19mm < 100mm$$

$$\alpha_z = 0.49\ ;\ \alpha_y = 0.35$$

$$\begin{cases} y-y \Rightarrow \phi_y = 0.5\left(1+\alpha_y \times \left(\bar{\lambda}_y - 0.2\right) + \bar{\lambda}_y^{\ 2}\right) = 0.5(1+0.34\times(0.2-0.2)+0.2^2) = 0.52 \\ z-z \Rightarrow \phi_z = 0.5\left(1+\alpha_z \times \left(\bar{\lambda}_z - 0.2\right) + \bar{\lambda}_z^{\ 2}\right) = 0.5(1+0.49\times(0.39-0.2)+0.39^2) = 0.623 \end{cases}$$

$$\begin{cases} y-y \Rightarrow \chi_y = \frac{1}{\phi_y+\sqrt{\phi_y^{\ 2}-\bar{\lambda}_y^{\ 2}}} = \frac{1}{0.52+\sqrt{0.52^2-0.2^2}} = 1 \\ z-z \Rightarrow \chi_z = \frac{1}{\phi_z+\sqrt{\phi_z^{\ 2}-\bar{\lambda}_z^{\ 2}}} = \frac{1}{0.623+\sqrt{0.623^2-0.39^2}} = 0.9 \end{cases}$$

$$\chi_{min} = \mathbf{0.9}$$

Case 1

$$\frac{N_{sd}}{\frac{\chi_{min}\times A\times f_y}{\gamma_{m0}}} = \frac{525,31\times10^3}{\frac{0.9\times180.6\times10^2\times275}{1.1}} = 0.13 > 0.1$$

$$\beta_{M,\varphi} = 1.8 - 0.7\varphi \qquad \varphi = \frac{M_1}{M_2}$$

- For $M_{sd,y}$

$$\varphi = \frac{97.35}{-97.72} = -0.98 \rightarrow \ \beta_{M,\varphi} = 1.8 - 0.7\times(-0.98) = 2.489$$

$$\mu_y = 0.2\times(2\times2.489-4) + \left(\frac{2683-2400}{2400}\right) = 0.32 \leq 0.9$$

$$K_y = 1 - \frac{0.32\times525,31\times10^3}{1\times180.6\times10^2\times275} = \mathbf{0.97} \leq 1.5$$

- For $M_{sd,z}$

$$\varphi = \frac{-53.71}{56.75} = -0.91 \rightarrow \ \beta_{M,\varphi} = 1.8 - 0.7\times(-0.91) = 2.435$$

$$\mu_z = 0.39\times(2\times2.435-4) + \left(\frac{1032-676.1}{676.1}\right) = 0.86 \leq 0.9$$

$$K_z = 1 - \frac{0.86\times525,31\times10^3}{0.9\times180.6\times10^2\times275} = \mathbf{0.89} \leq 1.5$$

Therefor:

$$\frac{525,31\times10^3}{0.9\times\frac{180.6\times10^2\times275}{1.1}} + \frac{0.97\times97.35\times10^6}{2683\times10^3\times\frac{275}{1.1}} + \frac{0.89\times63.04\times10^6}{1032\times10^3\times\frac{275}{1.1}} = 0.48 \leq 1$$ The condition is verified.

Case 2

$$\frac{N_{sd}}{\frac{\chi_{min}\times A\times f_y}{\gamma_{m0}}} = \frac{240.66\times10^3}{\frac{0.9\times180.6\times10^2\times275}{1.1}} = 0.06 < 0.1$$

- For $M_{sd,y}$

$$\varphi = \frac{-141.26}{225.69} = -0.63 \rightarrow \ \beta_{M,\varphi} = 1.8 - 0.7\times(-0.63) = 2.241$$

$$\mu_y = 0.2 \times (2 \times 2.241 - 4) + \left(\frac{2683 - 2400}{2400}\right) = 0.21 \leq 0.9$$

$$K_y = 1 - \frac{0.21 \times 240.66 \times 10^3}{1 \times 180.6 \times 10^2 \times 275} = \mathbf{0.98} \leq 1.5$$

- For $M_{sd,z}$

$$\varphi = \frac{2.52}{-2.76} = -0.91 \rightarrow \ \beta_{M,\varphi} = 1.8 - 0.7 \times (-0.89) = 2.439$$

$$\mu_z = 0.39 \times (2 \times 2.439 - 4) + \left(\frac{1032 - 676.1}{676.1}\right) = 0.86 \leq 0.9$$

$$K_z = 1 - \frac{0.86 \times 240.66 \times 10^3}{0.9 \times 180.6 \times 10^2 \times 275} = \mathbf{0.95} \leq 1.5$$

Therefor:

$$\frac{240.66 \times 10^3}{0.9 \times \frac{180.6 \times 10^2 \times 275}{1.1}} + \frac{0.98 \times 225.69 \times 10^6}{2683 \times 10^3 \times \frac{275}{1.1}} + \frac{0.95 \times 2.52 \times 10^6}{1032 \times 10^3 \times \frac{275}{1.1}} = 0.40 \leq 1$$

The condition is verified.

Case 3

$$\frac{N_{sd}}{\frac{\chi_{min} \times A \times f_y}{\gamma_{m0}}} = \frac{351.5 \times 10^3}{\frac{0.9 \times 180.6 \times 10^2 \times 275}{1.1}} = 0.08 < 0.1$$

- For $M_{sd,y}$

$$\varphi = \frac{101.96}{-102.12} = -0.99 \rightarrow \ \beta_{M,\varphi} = 1.8 - 0.7 \times (-0.99) = 2.499$$

$$\mu_y = 0.2 \times (2 \times 2.499 - 4) + \left(\frac{2683 - 2400}{2400}\right) = 0.3 \leq 0.9$$

$$K_y = 1 - \frac{0.3 \times 351.57 \times 10^3}{1 \times 180.6 \times 10^2 \times 275} = \mathbf{0.97} \leq 1.5$$

- For $M_{sd,z}$

$$\varphi = \frac{-85.65}{91.1} = -0.95 \rightarrow \ \beta_{M,\varphi} = 1.8 - 0.7 \times (-0.95) = 2.455$$

$$\mu_z = 0.39 \times (2 \times 2.455 - 4) + \left(\frac{1032 - 676.1}{676.1}\right) = 0.72 \leq 0.9$$

$$K_z = 1 - \frac{0.72 \times 351.5 \times 10^3}{0.9 \times 180.6 \times 10^2 \times 275} = \mathbf{0.93} \leq 1.5$$

Therefor

$$\frac{351.57 \times 10^3}{0.9 \times \frac{180.6 \times 10^2 \times 275}{1.1}} + \frac{0.97 \times 101.96 \times 10^6}{2683 \times 10^3 \times \frac{275}{1.1}} + \frac{0.93 \times 91.1 \times 10^6}{1032 \times 10^3 \times \frac{275}{1.1}} = 0.56 \leq 1$$

The condition is verified.

The table below sums up the calculation of the flexural buckling for the columns of the building

level	Case	profile	N_{sd} (kN)	$M_{sd,y}$ $(kN.m)$	$M_{sd,z}$ $(kN.m)$	$\bar{\lambda}_y$	$\bar{\lambda}_z$	χ_{min}	$\frac{N_{sd}}{N_{b,rd}}$	K_y	K_z	r	
6th storey	1	HEB 360	525,31	97.35	63.04				0.15	0.97	0.89	**0.48**	V
	2		240.66	225.96	2.52	0.2	0.39	0.89	0.07	**0.98**	**0.95**	**0.4**	V
	3		351.57	101.96	91.1				0.11	0.97	0.93	**0.56**	V
5th storey	1	HEB 360	811.08	97.69	63.04				0.23	0.93	0.84	**0.67**	V
	2		285.26	108,41	2.13	0.19	0.39	0.9	0.08	0.97	0.95	**0.23**	V
	3		525.12	102.10	91,10				0.15	0.95	0.89	**0.59**	V
4th storey	1	HEB 360	1261,06	115.16	1.82				0.38	0.91	0.74	**0.47**	V
	2		1294,68	115,16	1.82	0.19	0.39	0.9	0.18	0.91	0.74	**0.47**	V
	3		698.94	101.24	91,62				0.21	0.94	0.86	**0.61**	V
3rd storey	1	HEB 400	1923,86	117,05	2.27				0.44	0.88	0.7	**0.56**	V
	2		1923,86	117,05	2.27	0.18	0.41	0.92	0.44	0.88	0.7	**0.56**	V
	3		873.49	102.04	91,62				0.19	0.94	0.87	**0.60**	V
2nd storey	1	HEB 400	2646,00	107,99	4.97				0.60	0.83	0.59	**0.70**	V
	2		2646,00	107,99	4.97	0.18	0.41	0.92	0.60	0.83	0.59	**0.70**	V
	3		1048.36	102.11	88,44				0.23	0.92	0.81	**0.60**	V
1st storey	1	HEB 400	3376,36	110,92	4.97				0.77	0.8	0.47	**0.86**	V
	2		3376,36	110,92	4.97	0.18	0.41	0.92	0.77	0.8	0.47	**0.86**	V
	3		1223.69	114.05	86,38				0.27	0.92	0.82	**0.65**	V
Ground floor	1	HEB 450	4114,59	130.78	33.18				0.84	0.78	0.49	**0.93**	V
	2		2259.33	249,96	47.53	0.11	0.3	0.96	0.45	0.9	0.73	**0.77**	V
	3		576.92	182.43	55,95				0.12	0.96	0.93	**0.46**	V

Table 5. 2 : Verification of the columns for flexural buckling.

Lateral torsional buckling is not considered because $\bar{\lambda}_{LT} < 0.4$

For L=2.8m according to **ANNEX B of the CCM97**

$$\lambda_{LT} = \frac{KL/i_z}{\sqrt{C_1}\left[\left[\frac{K}{K_W}\right]^2 + \frac{1}{20}\left[\frac{KL/i_z}{h/t_f}\right]^2\right]^{0.25}} \quad \text{and} \quad \bar{\lambda}_{LT} = \frac{\bar{\lambda}_{LT}}{\lambda_1} \times \sqrt{\beta_W}$$

Column	iz	h	tf	K	Kw	C1	λ_{LT}	β_W	$\bar{\lambda}_{LT}$
HEB360	7.49	360	22.5	1	1	2.927	20.32	1	0.23
HEB400	7.4	400	24	1	1	2.927	20.88	1	0.24
HEB450	7.33	450	26	0.7	1	1.879	30.11	1	0.34

Table 5. 3: Calculation of $\bar{\lambda}_{LT}$

5.4. Verification of the beams

5.4.1. Main beams

The main beams are IPE300 with a steel grade S275. Internal forces such as shear force and the moment stressing the beams are obtained using calculation software ROBOT SA.

5.4.1.1. Bending strength check

The design value of the plastic resistance moment of the composite section $M_{pl,rd}$ was calculated in the chapter 3 such as $M_{pl,rd} = 342.91 kN.m$

The maximum design value of the bending moment was obtained at ULS 1.35G+1.5Q:

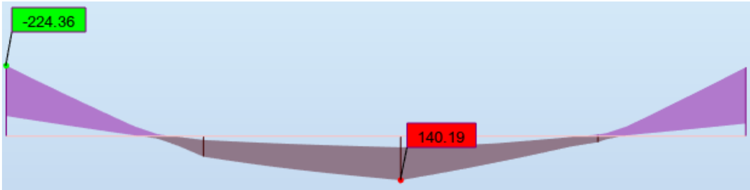


Figure 5. 1 : Bending moment diagram for the main beam.

$M_{sd} = 224.36kN.m < \mathrm{M_{pl,rd}} = 342.91kN.m$ The condition is satisfied.

5.4.1.2. Shear strength check

Design value of the plastic resistance of the composite section to vertical shear $\mathrm{V_{pl,rd}}$ was calculated in the chapter 3 such as $\mathrm{V_{pl,rd}} = 361.54kN.m$

The maximum shear force value acting on the section was obtained at ULS 1.35G+1.5Q:

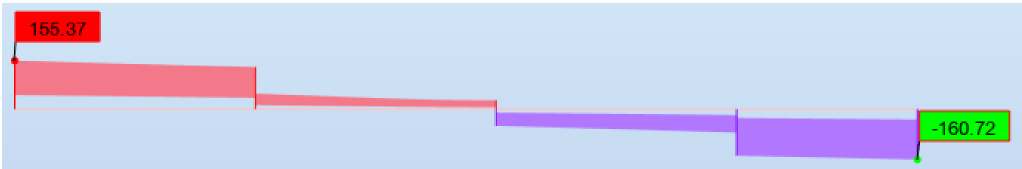


Figure 5. 2: Shear force diagram for the main beam.

$V_{sd} = 160.72 < V_{pl,rd} = 407.72kN$ The condition is satisfied.

5.4.1.3. Bending moment-shear force interaction

$$0.5V_{pl,rd} = 203.86kN > 160.72kN$$

There is no interaction between the bending moment and the shear force.

5.4.2. Joists

Joists are IPE 180 with a steel grade S275. Internal forces such as shear force and the moment stressing the beams are obtained using calculation software ROBOT SA.

5.4.2.1. Bending strength check

The design value of the plastic resistance moment of the composite section $\mathrm{M_{pl,rd}}$ was calculated in the chapter 3 such as $\mathrm{M_{pl,rd}} = 121.4kN$.

The maximum bending moment value was obtained at ULS 1.35G+1.5Q

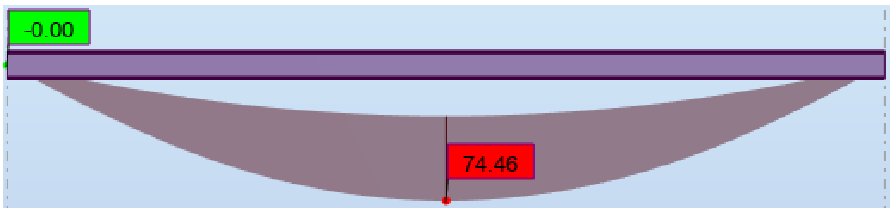


Figure 5. 3: bending moment diagram for the joists.

$M_{sd} = 74.46 < \mathrm{M_{pl,rd}} = 121.4kN$ The condition is satisfied.

5.4.2.2. Shear strength check

The maximum shear force value was obtained at ULS 1.35G+1.5Q

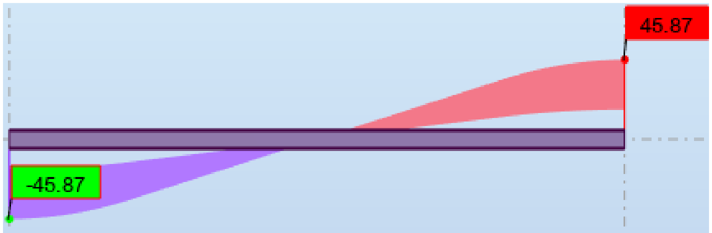


Figure 5. 4: shear force diagram for the joists.

$V_{sd} = 45.87 < V_{pl,rd} = 178.61kN$ The condition is satisfied.

5.4.2.3. Bending moment-shear force interaction

$V_{sd} = 45.87kN.m < 0.5V_{pl,rd} = 89.305kN.m$

There is no interaction between the bending moment and the shear force

5.4.3. Verification of the bracing system

5.4.3.1. X-X Direction

In the direction x-x we have double back to back UPN160 profiles as bracing, the profiles must be checked for resistance in compression and tension

According to RPAv2003(8.4.3.1) the seismic force used for the design of the bracing bars must be increased by 1.25

$N_{sd(compression)} = 675.25kN$

$N_{sd(tension)} = 654.68kN$

$L = 5.73m$ Because we are using a connection at the middle, the buckling length will be

$L_f = \frac{L}{2} = 2.865m$

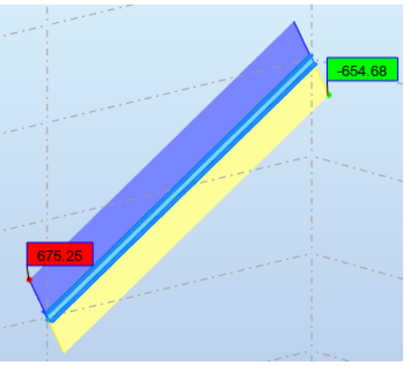


Figure 5. 5 : Axial force diagram for the bracing in the x-x direction.

5.4.3.1.1. Bar in tension

The design value of the tension force N_{sd} at each cross section shall satisfy:

$$N_{sd} \leq 2N_{t,Rd} = 2 \times min\{N_{pl,Rd}\,; N_{u,Rd}\}$$

The design plastic resistance of the gross cross-section: $N_{pl,Rd} = \frac{Af_y}{\gamma_{mo}} = 2\frac{24.10^2 \times 275}{1.1} = 1200kN$

$N_{sd} = 654.68kN < N_{pl,Rd} = 1200kN$

5.4.3.1.2. Bar in compression

a) Resistance check

The design value of the compression force N_{sd} at each cross-section shall satisfy:

$N_{sd} \leq 2 \times N_{c,Rd}$

$N_{c,Rd} = 1200kN > N_{sd} = 675.25kN$

b) Buckling stability check (flexural buckling)

The verification is done according to the axis the axis of low inertia (z)

$N_{sd} \leq N_{pl,Rd} = \frac{\chi_z \times A \times \beta_A \times f_y}{\gamma_{mo}}$ With $\beta_A = 1$

$\bar{\lambda}_z = \frac{L_{fz}/2 \times i_z}{93.91 \times \varepsilon} = \frac{287/2 \times 1.89}{93.91 \times \varepsilon} = 0.87 > 0.2\ there\ is\ a\ risk\ of\ flexural\ buckling$

$\phi_z = 0.5 \times \left(1 + \alpha_z \times (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^{\ 2}\right) = 0.5 \times (1 + 0.49 \times (0.87 - 0.2) + 0.87^2) = 1.04$

$\chi_z = \frac{1}{\phi_z + \sqrt{\phi_z^{\ 2} - \bar{\lambda}_z^{\ 2}}} = \frac{1}{1.04 + \sqrt{1.04^2 - 0.87^2}} = 0.62$

$N_{pl,Rd} = \frac{2 \times 0.62 \times 24 \times 10^2 \times 1 \times 275}{1.1} \times 10^{-3} = 744KN$

$N_{sd} = 675.25kN < N_{pl,Rd} = 744kN$

5.4.3.2. Y-Y direction

In the direction y-y we have double back to back UPN220 profiles as bracing, the profiles must be checked for resistance in compression and tension

Same thing will be done in this direction, the results are summed up bellow.

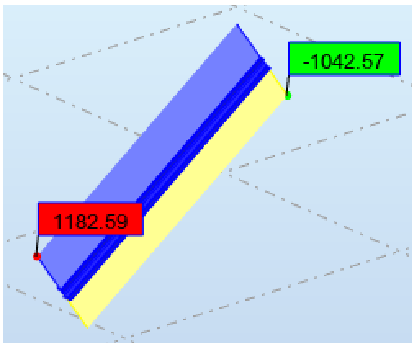


Figure 5. 6: Axial force diagram for the bracing in the y-y direction.

$N_{sd(compression)} = 1182.59kN$

$N_{sd(tension)} = 1088.64kN$

5.4.3.2.1. Bar in tension

$N_{sd} = 1088.64kN < N_{pl,Rd} = 1870kN$

5.4.3.2.2. Bar in compression

a) Resistance check

$N_{sd} = 1182.59kN < N_{pl,Rd} = 1870kN$

b) Stability check (Flexural buckling)

L_{fz}	$\bar{\lambda}_z$	ϕ_z	χ_z	$N_{pl,Rd}$	N_{sd}	r	observation
2.44	0.61	0.78	0.78	1458.6	1182.59	0.77	Verified

Table 5. 4: Verification of flexural buckling for the bracing system according to y-y direction.

5.5. Conclusion

As observed; the values of the bending moments and shear forces on the beams have little to no change compared to the preliminary sizing and they meet with all the safety conditions imposed by the CCM97 and EUROCODE4

The verification of the columns and the bracing system led to an increase in their section (that has not been mentioned); the current size of the columns and bars verifies all the conditions imposed by the CCM97.

CHAPTER 06:

Design of joints.

6. DESIGN OF JOINTS

6.1. Introduction

Properly designing beams, columns and bracing system is not enough to say that the building's security is ensured, the most important part of steel construction is the design of "JOINTS"

Joints are structural elements used for joining different members of a structural steel frame work. Their job is to ensure the proper transmission of the internal forces between the different structural elements for example: column-column connection; beam-beam connection; beam-column connection etc...

The most used type of connection nowadays are:

- Bolted connections
- Welded connections
- Bolted-welded connections

In our present study, bolted connection is the widely used mode. Because it has the advantage of easy demountability, with full recovery of the initial components.

6.2. Main Beam-Joist connection

The connection is done by out using two angles which connects the end of the IPE180 joist with the web of the IPE300 beam. It's a shear (simple) connection that transmits only shear force from the joist, which is$V_{sd} = 45.87kN$.

As a preliminary choice we used M16 bolts class 6.8 and equal legs angles 100×100×10

Bolts	Number	Class	d (mm)	d_0 (mm)	A_s (mm^2)	F_{ub} (N/mm^2)
M16	4	6.8	16	18	157	600

Table 6. 1: Characteristics of the bolts for the main beam-joist connection.

6.2.1. Positioning of holes for bolts

According to CCM97; the positioning of the holes for bolts shall be determined as follows:

t is the thickness of the thinner outer connected part

$$t = min\{t_{w(beam)}; t_{w(joist)}\} = min\{7.1; 5.3\} = 5.3mm$$

$$\begin{cases} 1.2 \times d_0 \leq e_1 \leq max\{12t; 150mm\} \rightarrow 21.6 \leq e_1 \leq 150 \rightarrow e_1 = 25mm \\ 2.2 \times d_0 \leq p_1 \leq min\{14t; 200mm\} \rightarrow 39.6 \leq p_1 \leq 74.2 \rightarrow p_1 = 50mm \\ 1.5 \times d_0 \leq e_2 \leq max\{12t; 150mm\} \rightarrow 27 \leq e_2 \leq 150 \rightarrow e_2 = 50mm \\ 3 \times d_0 \leq p_2 \leq min\{14t; 200mm\} \rightarrow 54 \leq p_2 \leq 74.2 \rightarrow \\ p_2 = 0 \; because \; we \; only \; have \; one \; row \; of \; bolts \end{cases}$$

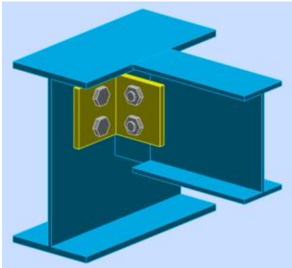


Figure 6. 1: Beam-joist connection.

6.2.2. Verification of the bolts

The connection is type "A" bearing type, therefore the bolts must satisfy the three following conditions:

$$\begin{cases} F_{v,sd} \leq min\{F_{v,rd}; F_{b,rd}\} \\ F_{v,rd}(is\ the\ design\ shear\ resistance\ per\ shear\ plane) \\ F_{b,rd}(is\ the\ bearing\ resistance\ per\ bolt) \\ F_{t,sd} \leq F_{t,rd}(The\ design\ tensile\ force) \end{cases}$$

$F_{t,rd} = \frac{0.9\times f_{ub}\times A}{\gamma_{m2}}$; $F_{v,rd} = \frac{0.5\times f_{ub}\times A}{\gamma_{m2}}$; $F_{b,rd} = \frac{2.5\times\alpha\times d\times t\times f_u}{\gamma_{m2}}$ with : $\alpha = min\left\{\frac{e_1}{3d_0}; \frac{p_1}{3d_0} - \frac{1}{4}; \frac{f_{ub}}{f_u}; 1\right\}$

The eccentricity of the bolts from the point of application of the shear force (beam's web) induces a flexural moment which shears the bolts, therefore the overall shear force applied to the bolt will be a combination of both the shear force from the beam and the shear force created by the bending moment.

a) Forces acting on bolts in the main beam - angle connection

- **Shear check**

- $F_{v,rd} = \frac{0.5\times600\times157}{1.25} \times 10^{-3} = 37.68kN$
- $F_{b,rd} = \frac{2.5\times0.46\times16\times10\times430}{1.25} \times 10^{-3} = 63.29kN$ $\alpha = min\{0.46; 0.67; 1.16; 1\} = 0.46$

$$F_{v,sd} = \sqrt{\left(\frac{V_{sd}}{n}\right)^2 + \left(\frac{\left(\frac{V_{sd}}{2}\times e_2\right)\times z}{\sum {z_i}^2}\right)^2}$$

$$F_{v,sd} = \sqrt{\left(\frac{45.87}{4}\right)^2 + \left(\frac{\left(\frac{45.87}{2}\times0.05\right)\times0.05}{0.05^2}\right)^2} = 25.64kN < min\{F_{v,rd}; F_{b,rd}\} = 37.68kN$$

The condition is verified.

- **Tension check**

$$F_{t,rd} = \frac{0.9\times600\times157}{1.5} = 56.52kN$$

$$F_{t,sd} = \left(\frac{\left(\frac{V_{sd}}{2}\times e_2\right)\times z}{\sum {z_i}^2}\right) = \frac{\left(\frac{45.87}{2}\times0.05\right)\times0.05}{0.05^2} = 24.76kN < 56.52kN$$

The condition is verified.

- **Simultaneous action of tensile and shear force in the bolt**

$$\frac{F_{v,sd}}{F_{v,rd}} + \frac{F_{t,sd}}{1.4F_{t,rd}} \leq 1 \Rightarrow \frac{25.64}{37.68} + \frac{24.76}{1.4\times56.52} = 0.993 \leq 1$$

The condition is verified.

b) Forces acting on bolts in the joist - angle connection

- **Shear check**

- $F_{v,rd} = 75.63kN$
- $F_{b,rd} = \frac{2.5\times0.67\times16\times10\times430}{1.25} \times 2 \times 10^{-3} = 126.58kN$

$$F_{v,sd} = \sqrt{\left(\frac{V_{sd}}{n}\right)^2 + \left(2\times\frac{\left(\frac{V_{sd}}{2}\times e_2\right)\times z}{\sum {z_i}^2}\right)^2}$$

$$F_{v,sd} = \sqrt{\left(\frac{45.87}{2}\right)^2 + \left(\frac{\left(2\frac{45.87}{2}\times 0.05\right)0.05}{0.05^2}\right)^2} = 51.28kN < 75.63kN$$ The condition is verified.

6.2.3. Shear verification for angle

To design the angle the following condition must be verified: $V_{sd} \leq V_{pl,rd} = \frac{\times\frac{f_y}{\sqrt{3}}}{\gamma_{m2}}$

According to (5.21) of CCM97 It is not necessary to take into account the holes if:

$$A_{v.net} \geq \left(\frac{f_y}{f_u}\right) \times \left(\frac{\gamma_{m2}}{\gamma_{m0}}\right) \times A_v$$

$A_{v.net} = 6.4cm^2 > \left(\frac{400}{500}\right) \times \left(\frac{1.25}{1.1}\right) \times 10 = 9.09cm^2$ Therefore we must calculate $A_{v.eff}$

$$A_{v.eff} = A_{v.net}\left(\frac{f_u}{f_y}\right) \times \left(\frac{\gamma_{m0}}{\gamma_{m2}}\right) = 6.4 \times \left(\frac{500}{400}\right) \times \left(\frac{1.1}{1.25}\right) = 7.04cm^2$$

$$V_{pl,rd} = \frac{705\times\frac{275}{\sqrt{3}}}{1.25} \times 10^{-3} = 95.26kN > 45.87kN$$ The condition is verified.

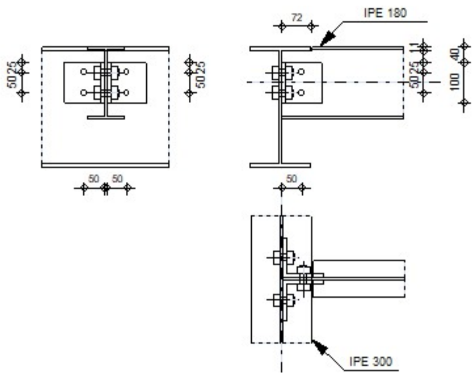


Figure 6. 2: Beam-joist connection details.

6.3. Beam-column connection

6.3.1. Model 1 (main beam-column)

For this connection a plate is **welded** with to the flanges and the web of the beam, it is drilled symmetrically on either side of the beam. The same holes are made on the flange of the column that allow the two elements to be joined together using **high strength bolts**. This is a rigid connection which transmits: $V_{sd} = 155{,}42kN$ and $M_{sd} = -224{,}46kN.m$

As a preliminary choice we used two rows of 4 M27 high resistance bolts class 10.9

bolts	number	Class	d (mm)	d_0 (mm)	A_s (mm^2)	F_{ub} (N/mm^2)
M27	8	10.9	27	30	459	1000

Table 6. 2: Characteristics of the bolts for the main beam-column connection.

We just estimated the dimensions of the steel plate, the later will then be verified:

$$\begin{cases} h_p = 480mm \\ b_p = 200mm \\ t_p = 20mm \end{cases}$$

6.3.1.1. Positioning of holes for bolts

$$\begin{cases} 1.2 \times d_0 \leq e_1 \leq max\{12t; 150mm\} \rightarrow 36 \leq e_1 \leq 240 \rightarrow e_1 = 65mm \\ 2.2 \times d_0 \leq p_1 \leq min\{14t; 200mm\} \rightarrow 66 \leq p_1 \leq 200 \rightarrow p_1 = 100mm \\ 1.5 \times d_0 \leq e_2 \leq max\{12t; 150mm\} \rightarrow 45 \leq e_2 \leq 240 \rightarrow e_2 = 50mm \\ 3 \times d_0 \leq p_2 \leq min\{14t; 200mm\} \rightarrow 90 \leq p_2 \leq 200 \rightarrow p_2 = 100mm \end{cases}$$

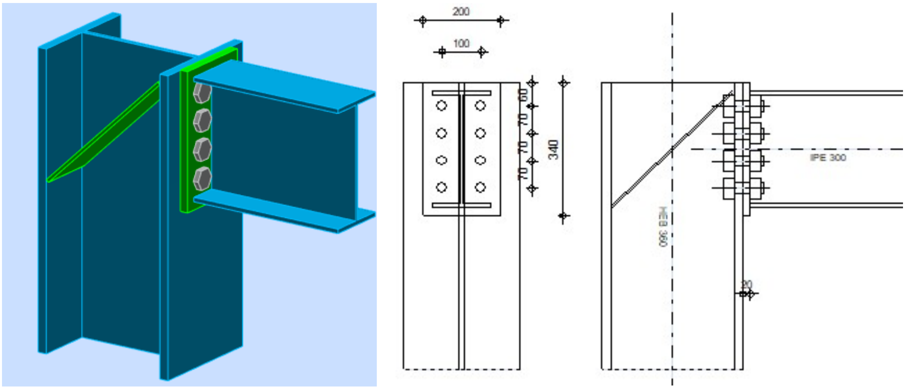


Figure 6. 3: Main beam-column connection.

6.3.1.2. Loads assessment

- The bolts are verified to shear and tension, therefore the bending moment must be transformed into a tensile force

Steel plate (340 × 200 × 20)

$F_{t,sd} = F_{M1}$

$$F_{M1} = \frac{M_{sd} \times d_4}{n_f \sum {d_i}^2}$$

i	d₁	d₂	d₃	d₄	$\sum d_i^2$
d_i(mm)	64.65	134.65	204.65	274.65	139625.49

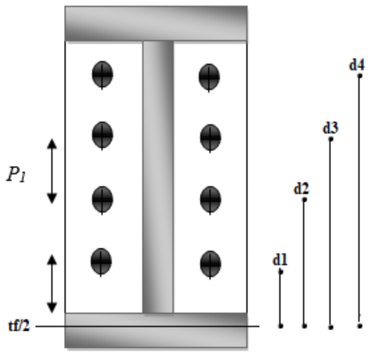


Figure 6. 4: Lever arm for bolts

$$F_{M1} = \frac{224,46 \times 10^6 \times 274.65}{2 \times 139625.49} \times 10^{-3} = 220.76kN$$

Shear force per bolt

$$F_{vsd} = \frac{V_{sd}}{n_p \times n_b} = \frac{155,42}{1 \times 8} = 19.42kN$$

6.3.1.3. Verification of weld resistance

In order for the weld to resist the applied forces; the following conditions must be verified:

$$\begin{cases} [{\sigma_\perp}^2 + 3 \times ({\tau_\perp}^2 + {\tau_\parallel}^2)]^{0.5} \leq \dfrac{f_u}{\beta_w \times \gamma_{mw}} \\ \sigma_\perp \leq \dfrac{f_u}{\beta_w \times \gamma_{mw}} \end{cases}$$

Such as:
-$\sigma_\perp$: Normal stress perpendicular to the throat;
-$\tau_\perp$Normal stress parallel to the weld axis;
-$\tau_\parallel$ Tangent stress (in the plane of the groove) perpendicular to the axis of the weld;

β_w Correlation factor which takes the following values:

Steel grade	Fe 360	Fe 430	Fe 510
β_W	0,80	0,85	0,90

Table 6. 3: βw Correlation factor

$$S275 \rightarrow \begin{cases} \beta_W = 0.85 \\ \gamma_{mw} = 1.3 \end{cases}$$

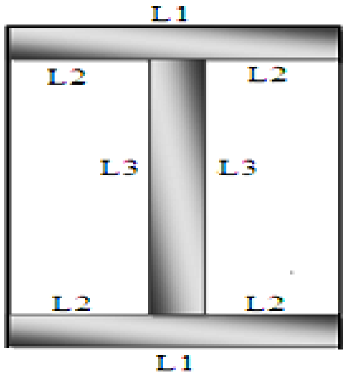


Figure 6. 5: length of the weld bead.

- It is assumed that the moment M is taken up only by the weld beads 1 and 2;
- It is assumed that the force V is taken up by the weld bead 3;
- We assume that the thickness of the weld of the flange is $a_f = 11mm$
- We assume that the thickness of the web is $a_w = 10mm$ (to be verified).

$$\begin{cases} L_1 = b_{beam} = 150mm \\ L_2 = \frac{b - t_w}{2} = \frac{150 - 7.1}{2} = 71.45mm \\ L_3 = h - 2 \times t_f = 300 - 2 \times 10.7 = 278.6mm \end{cases}$$

a) N_{sd} *and* V_{sd} *effect*

$$\sqrt{2 \times \left(\frac{N_{sd}}{\sum l_i \times a_i}\right)^2 + 3 \times \left(\frac{V_{sd}}{2l_3 \times a_i}\right)^2} \leq \frac{f_u}{\gamma_{mw} \times \beta_w} ..$$

$N_{sd} = 0 \,;\, V_{sd} = 155.42kN$

$$\sqrt{3 \times \left(\frac{155.42 \times 10^3}{2 \times 278.6 \times 10}\right)^2} = 48.31kN < \frac{430}{1.3 \times 0.85} = 389.14kN$$ The condition is verified.

b) N_{sd} *and* M_{sd} *effect*

$$\sqrt{2} \times \left(\frac{N_{sd}}{\sum l_i \times a_i} + \frac{M_{sd}}{I_{g/s}} \times v_{max}\right) \leq \frac{f_u}{\gamma_{mw} \times \beta_w}$$ $N_{sd} = 0 \,;\, M_{sd} = 224.46kN.m$

$$I_{g/s} = 2 \times [a.l_1.{d_1}^2 + 2.a.l_2.{d_2}^2] \; with \begin{cases} d_1 = \frac{h}{2} + \frac{a}{2} \\ d_2 = \frac{h}{2} - t_f - \frac{a}{2} \end{cases} ; v_{max} = \frac{h}{2}$$

$$with \begin{cases} d_1 = \frac{300}{2} + \frac{10}{2} = 155mm \\ d_2 = \frac{300}{2} - 10.7 - \frac{10}{2} = 134.3mm \end{cases}$$

$$I_{g/s} = 2 \times [11 \times 150 \times 155^2 + 2 \times 11 \times 71.45 \times 134.3^2] = 1.359856173 \times 10^8 mm^4$$

$$\sqrt{2} \times \left(\frac{224.46 \times 10^6}{1.359856173 \times 10^8} \times 150\right) = 350.14N/mm^2 < 389.14N/mm^2$$ The condition is verified.

6.3.1.4. Verification of the bolts

a) **Shear check**

For high strength bolts we must check that: $F_{v,sd} \leq F_{s,rd}$

$F_p = 0.7 \times 459 \times 1000 \times 10^{-3} = 321.3kN$

$$F_{s,rd} = \frac{K_s \times n_p \times \mu \times F_p}{\gamma_{m2}} \rightarrow \begin{cases} F_p = 0.7 \times A_s \times F_{ub} \ (\,the\ preloading\ force) \\ Ks = 1\ normal\ holes\ (\boldsymbol{CCM97\ 6.5.6.1}) \\ \mu \text{ (slip factor)} = 0.3(\boldsymbol{CCM97\ 6.5.6.3}) \\ n_p = 1\ number\ of\ the\ friction\ planes \end{cases}$$

$= \frac{1 \times 1 \times 0.3 \times 321.3}{1.25} = 77.112kN > F_{vsd} = 19.42kN$ The condition is verified.

b) Tension check

$F_{t,sd} \leq F_{t,rd}$

$F_{t,rd} = \frac{0.9 \times A_s \times F_{ub}}{\gamma_{mb}} = \frac{0.9 \times 459 \times 1000 \times 10^{-3}}{1.5} = 275.4 > 220.76kN$ The condition is verified.

c) Combined shear and tension check

$F_{s,rd} = \frac{K_s \times n_p \times \mu \times (F_p - 0.8F_{t,sd})}{\gamma_{m2}}$

$F_{s,rd} = \frac{1 \times 1 \times 0.3 \times (321.3 - 0.8 \times 220.76)}{\gamma_{m2}} = 34.72kN > F_{vsd} = 19.42kN$ The condition is verified.

6.3.1.5. Verification of the overall connection

$M_r \geq M_{s,rd} \rightarrow M_r = \frac{N_1 \times \sum {d_i}^2}{d_4};\ N_1 = n \times f_p$

n is the number of bolts per row n=2 → $N_1 = n \times f_p = 2 \times 321.3 = 642.6kN$

$M_r = \frac{642.6 \times 139625.49}{274.65} = 326.68kN.m > 224{,}46kN.m$ The condition is verified.

6.3.1.6. Verification of the column

$N_t = N_c = N_r = \frac{M}{d} = \frac{224{,}46}{0.3 - 0.0107} = 775.87kN$

a) Zone in tension

$N_t \leq F_t = \frac{f_y \times t_{wc} \times b_{eff}}{\gamma_m} \rightarrow b_{eff} = t_{fb} + 2t_p + 5(t_{fc} + r_c)$

$b_{eff} = 10.7 + 2 \times 20 + 5(22.5 + 27) = 298.2mm$

$F_t = \frac{275 \times 11 \times 298.2}{1.1} \times 10^{-3} = 820.05kN > N_t = 775.87kN$ The condition is verified.

b) Zone in Compression

$$N_c \leq F_c = f_y \times t_{wc}(1.25 - 0.5 \times \gamma_{m0} \times \frac{\sigma_c}{f_y}) \times \frac{b_{eff}}{\gamma_{m0}}$$

$b_{eff} = 305.7mm$

$\sigma_c = \frac{V}{A} + \frac{M}{W_{el,c}} = \frac{155{,}42 \times 10^3}{180.6 \times 10^2} + \frac{224.46 \times 10^6}{2400 \times 10^3} = 102.15N/mm^2$

$F_c = 275 \times 12.5 \left(1.25 - 0.5 \times 1.1 \times \frac{102.15}{275}\right) \times \frac{298.2}{1.1} \times 10^3 = 974.46\text{kN} > N_c = 775.87kN$

The condition is verified.

c) Zone in shear

$N_r \leq F_r = 0.58 \times f_y \times h_c \times t_{wc}$

$$F_r = \frac{0.58 \times f_y \times h_c \times t_{wc}}{\gamma_{m0}} = \frac{0.58 \times 275 \times 360 \times 12.5 \times 10^{-3}}{1.1} = 717.75kN > N_t = 775.87kN$$

The condition is not verified.

We need to a stiffener to the column's web (either a diagonal stiffener or a steel plate welded to web)

In our case we used a diagonal web stiffeners such as t (thickness) =8mm

6.3.2. Model 2 (joist-column)

The calculation is done following the exact same way that we calculated the joist-beam connection because it's a simple connection.

The connection is done by out using two angles which connects the end of the IPE180 joist with the web of the HEB360 column.

We used M16 bolts class 5.8 and equal legs angles 100×100×10

Bolts	Number	Class	d (mm)	d_0 (mm)	A_s (mm^2)	F_{ub} (N/mm^2)
M16	4	5.8	16	18	157	500

Table 6. 4: Characteristics of the bolts for the joist-column connection.

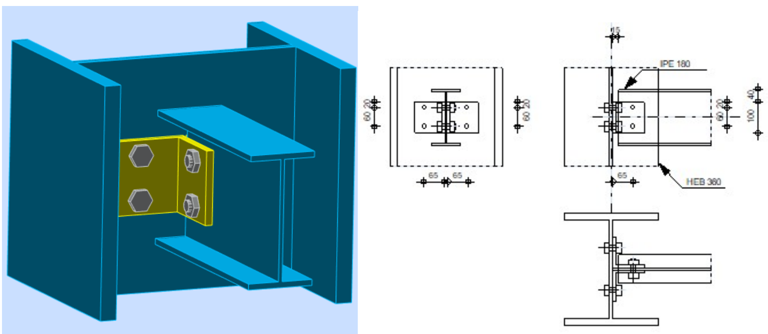


Figure 6. 6: secondary beam-column connection.

6.4. Bracing system

6.4.1. In the X-X direction

In our case for the bracing system we have 2UPN160 which makes the bolts of the connection between diagonal - gusset doubly sheared.

According to **RPA99V2003** (8.4.3) "*the joints must be designed to allow the maximum forces to be developed in the bars or must be calculated on the basis of 1.5 times the force determined by the seismic study*". Therefore: $N_{sd} = -787{,}04kN$

6.4.1.1. Column-2UPN160 connection

a) Gusset-corner connection

Connection for X shaped bracing are located at column-beam joint; the gusset is assembled to the flange of the two elements by welding we assume:

- "a" the thickness of the weld bead for this connection, such as a=5mm;
- Gusset $(550 \times 450 \times 20)$;

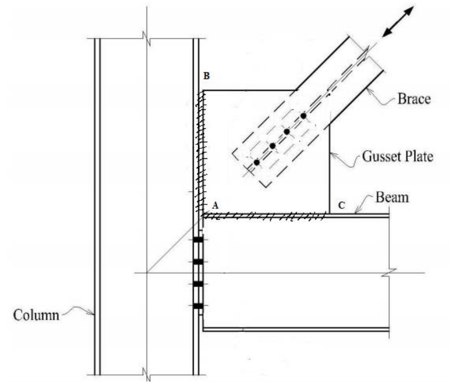


Figure 6. 7: gusset-corner connection.

Verification of gusset weld:

$AB = 550mm\,;\; AC = 450mm\,; tp = 20\,mm$

The weld beads are verified according to the following expression

$$N_{sd,i} \le F_{w,rd} = \frac{a \times \sum L \times f_u}{\beta_w \times \gamma_{mw} \times \sqrt{3 - \sin^2 \alpha}} \quad , N_{sd,i} = \alpha \times$$

$$N_{sd} \begin{cases} N_{sd,AB} = 733.05 \times \cos 29.25 = 639.58kN \\ N_{sd,AC} = 733.05 \times \cos 60.75 = 312.51kN \end{cases}$$

- **Verification of AB**

$$F_{w,rd} = \frac{5 \times 550 \times 430}{0.85 \times 1.3 \times \sqrt{3 - \sin^2 29.25}} = 644kN > N_{sd,AB} = 639.58kN$$

The condition is verified.

- **Verification of AC**

$$F_{w,rd} = \frac{5 \times 450 \times 430}{0.85 \times 1.3 \times \sqrt{3 - \sin^2 60.75}} = 585.175kN > N_{sd,AC} = 312.51kN$$

The condition is verified.

b) Gusset-bar(2UPN140) connection

Bolts	Number	Class	d (mm)	d_0 (mm)	A_s (mm^2)	F_{ub} (N/mm^2)
M24	4	8.8	24	26	303	800

Table 6. 5: Characteristics of the bolts for the gusset-2UPN140 connection.

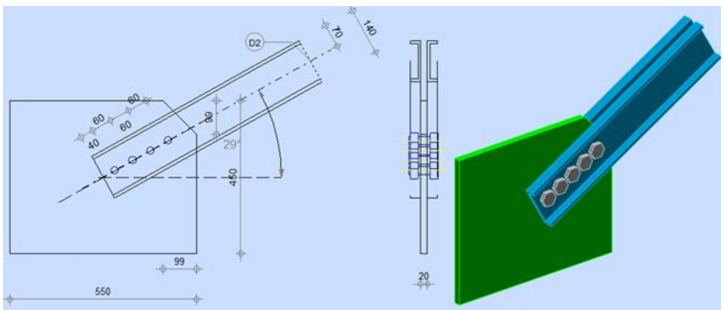


Figure 6. 8: Gusset-2UPN140 connection.

- **Positioning of holes for bolts**

$$\begin{cases} 1.2 \times d_0 \leq e_1 \leq max\{12t; 150mm\} \rightarrow 31 \leq e_1 \leq 240 \ \rightarrow e_1 = 40mm \\ 2.2 \times d_0 \leq p_1 \leq min\{14t; 200mm\} \rightarrow 57.2 \leq p_1 \leq 200 \rightarrow p_1 = 60mm \\ 1.5 \times d_0 \leq e_2 \leq max\{12t; 150mm\} \rightarrow 39 \leq e_2 \leq 240 \rightarrow e_2 = 80mm \end{cases}$$

- **Bolts verification**

$$\begin{cases} F_{v,sd} \leq min\{F_{v,rd}; F_{b,rd}\} \\ F_{v,rd} (is\ the\ design\ shear\ resistance\ per\ shear\ plane) \\ F_{b,rd} (is\ the\ bearing\ resistance\ per\ bolt) \\ F_{t,sd} \leq F_{t,rd} (The\ design\ tensile\ force) \end{cases}$$

$$F_{v,sd} = \frac{N_{sd}}{n_b} = \frac{-787{,}04}{4} = 196.76kN$$

$$F_{v,rd} = \frac{0.6 \times 800 \times 353 \times 2}{1.25} \times 10^{-3} = 271.11kN \ ;$$

$F_{b,rd} = \frac{2.5 \times 0.55 \times 24 \times 20 \times 800}{1.25} = 422.4kN$ With: $a = min\{0.55; 0.58; 1.39; 1\} = 0.55$

$F_{v,sd} = 196.76kN < min\{F_{v,rd}; F_{b,rd}\} = 271.11kN$ The condition is verified.

6.4.1.2. Mid connection

We choose the same type of bolts as the previous connection with the same class.

Bolts	Number	Class	d (mm)	d_0 (mm)	A_s (mm^2)	F_{ub} (N/mm^2)
M22	3	8.8	22	24	303	800

Table 6. 6: Characteristics of the bolts for the mid connection.

Gusset ($600 \times 500 \times 20$)

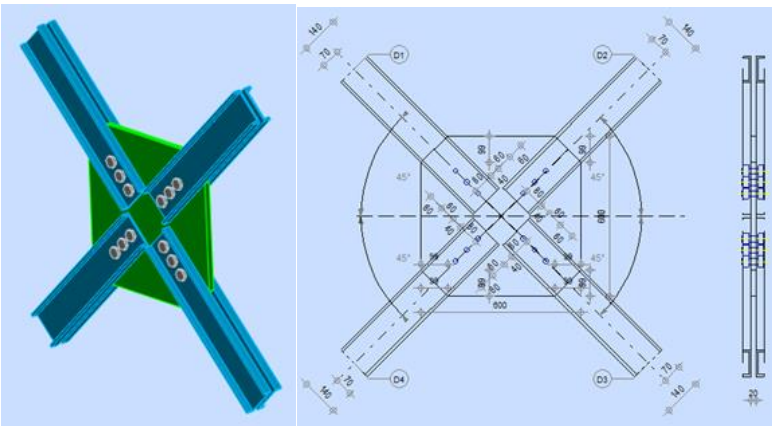


Figure 6. 9: Bracing mid connection.

Due to symmetry of the connection we can check just one bar of the joint.

- **Bolts verification**

$$F_{v,sd} = \frac{N_{sd}}{n_b} = \frac{733.05}{3} = 244.35kN$$

$F_{v,sd} = 244.35kN \leq min\{F_{v,rd}; F_{b,rd}\} = 271.11kN$ The condition is verified.

6.4.2. In the Y-Y direction

Same steps as the x-x direction had been followed to calculate the connection in y-y direction using 2UPN220 bars. We chose

- **For the corner connection**
  - a=8mm;
  - 5 bolts M24 8.8;
  - Gusset ($550 \times 450 \times 20$);
  - $N_{sd} = -1209{,}69$;
  - $F_{v,sd} = 234.89kN$;

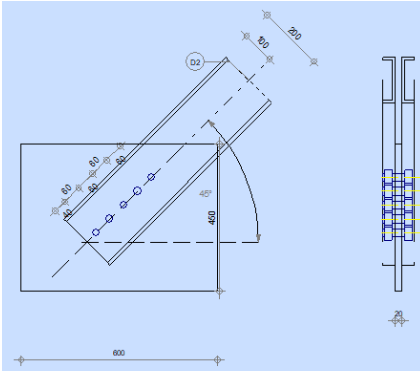


Figure 6. 10: connection details for bracing in Y-Y direction.

6.5. Column-column connection (using cover plates)

Column to column connection is called "column splice" and they are very common in steel structures. One of the most important steps for connecting two columns is the center line of the top has to match the center line of the bottom column. This will assure that the load from the top column is transferred properly to the bottom and down to the footing or foundations.

A column to column connection allows the transmission of bending moment, axial and shear forces.

As an example we are going to assemble an HEB360 to an HEB400. ue to the difference in section we are going to need to add a steel plate on each side of the HEB360 column to insure the connection, the steel plate will be 20mm thick.

For the calculation of this connection we referred to $[\mathbf{12}]$

6.5.1. Flange-cover plate

6.5.1.1. Connection details

- M24 class 8.8 normal bolts

Bolts	Number	Class	d (mm)	d_0 (mm)	A_s (mm^2)	F_{ub} (N/mm^2)
M24	8	8.8	24	26	353	8000

Table 6. 7: Characteristics of the bolts of the flange-cover plate connection.

- Steel plate ($360 \times 280 \times 15$)
- Force : $\begin{cases} N_{sd} = 1261{,}06kN \\ V_{sd} = 141{,}44kN \\ M_{sd} = 115{,}16kN.m \end{cases}$

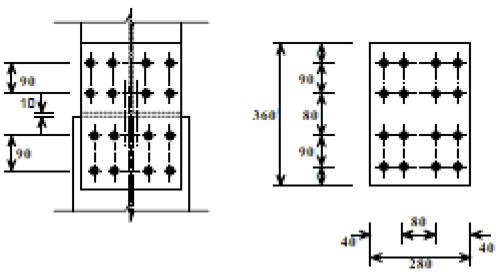


Figure 6. 11: Flange cover plate connection details.

6.5.1.2. Positioning of holes for bolts

$$\begin{cases} 31.2 \le e_1 \le 180 \ \rightarrow e_1 = 50mm \\ 57.2 \le p_1 \le 200 \rightarrow p_1 = 90mm \\ 39 \le e_2 \le 180 \rightarrow e_2 = 40mm \\ 78 \le p_2 \le 200 \rightarrow p_2 = 80mm \end{cases}$$

6.5.1.3. Loads assessment

The loads acting on the flange cover plate are both tension and compression forces:

- **Compression :**

$N_{sd;fc} = \frac{M_{sd}}{h_c} + N_{sd} \times \frac{A_f}{A_t} = \left(\frac{115{,}16\times10^6}{400} + 1261{,}06 \times 10^3 \frac{7200}{19780}\right) 10^{-3} = 746.93kN$

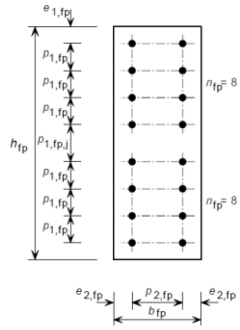
- **Tension**

$N_{sd;ft} = \frac{M_{sd}}{h_c} + N_{sd} \times \frac{A_f}{A_t} = \left(\frac{115{,}16\times10^6}{400} - 1261{,}06 \times 10^3 \frac{7200}{19780}\right) 10^{-3} = -171.13kN$

No need to verify tension strength

6.5.1.4. Verification of the plate

We need to check compression strength, for that:



$$N_{sd;fc} \le N_{rd;fc} = \begin{cases} \frac{A_{f,p} \times f_{y,p}}{\gamma_{m0}} \; if \; \frac{P_{fp,j}}{t_{f,p}} \le 9\varepsilon \\ \frac{\chi \times A_{f,p} \times f_{y,p}}{\gamma_{m1}} \; if \; \frac{P_{fp,j}}{t_{f,p}} > 9\varepsilon \end{cases}$$

$\frac{80}{15} = 5.33 < 9 \times 0.92 = 8.28 \rightarrow N_{rd;fc} = \frac{A_{f,p}\times f_{y,p}}{\gamma_{m0}}$

$N_{rd;fc} = \frac{280\times15\times275}{1.1} \times 10^{-3} = 1050kN > N_{sd;fc} = 746.93kN$ The condition is verified.

6.5.1.5. Verification of the bolts

We must verify that: $N_{sd;fc} \le n_b \times min\{F_{v,rd}; F_{b,rd}\}$

$F_{v,rd} = \frac{0.6\times800\times353}{1.25} \times 10^{-3} = 135.552kN$;

$F_{b,rd} = \frac{2.5\times0.64\times24\times15\times800}{1.25} = 290.4kN$ With: $a = min\{0.64; 0.71; 1.86; 1\} = 0.64$

$F_{v,sd} = 746.93kN < 8 \times min\{F_{v,rd}; F_{b,rd}\} = 1084.416kN$ The condition is verified.

6.5.2. Web-cover plate

6.5.2.1. Connection details

- M24 class 8.8 normal bolts

Bolts	Number	Class	d (mm)	d_0 (mm)	A_s (mm^2)	F_{ub} (N/mm^2)
M24	6	8.8	24	26	353	8000

Table 6. 8: Characteristics of the bolts of the web- cover plate connection.

- Steel plate $(500 \times 300 \times 15)$
- Force : $\begin{cases} N_{sd} = 1261{,}06kN \\ V_{sd} = 141{,}44kN \\ M_{sd} = 115{,}16kN.m \end{cases}$

6.5.2.2. Positioning of holes for bolts

$$\begin{cases} 31.2 \le e_1 \le 180 \rightarrow e_1 = 50mm \\ 57.2 \le p_1 \le 200 \rightarrow p_1 = 75mm \\ 39 \le e_2 \le 180 \rightarrow e_2 = 75mm \\ 78 \le p_2 \le 200 \rightarrow p_2 = 150mm \end{cases}$$

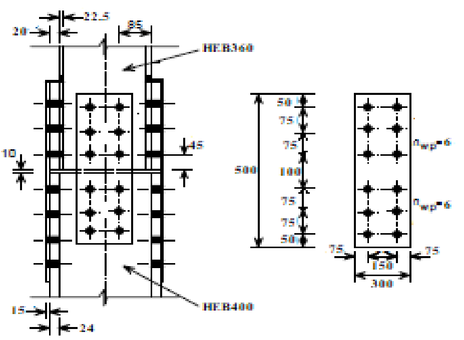
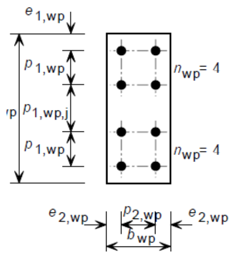


Figure 6. 12: Web cover plate connection details.

6.5.2.3. Loads assessment

The compression forced on the web cover plate is calculated using the following expression:

$$N_{sd,wp} = \frac{N_{sd} \times A_{wc}}{2 \times A_c} = \frac{1261{,}06 \times 10^3 \times 352 \times 13.5}{2 \times 197.8 \times 10^2} \times 10^{-3} = 151.48kN$$



6.5.2.4. Verification of the plate

We need to check compression strength, for that:

$$N_{sd;wp} \leq N_{rd;wp,c} = \begin{cases} \frac{A_{w,p} \times f_{y,p}}{\gamma_{m0}} \; if \; \frac{P_{wp,j}}{t_{w,p}} \leq 9\varepsilon \\ \frac{\chi \times A_{w,p} \times f_{y,p}}{\gamma_{m1}} \; if \; \frac{P_{wp,j}}{t_{w,p}} > 9\varepsilon \end{cases}$$

$$\frac{100}{15} = 6.66 < 9 \times 0.92 = 8.28 \rightarrow N_{rd;fc} = \frac{A_{f,w} \times f_{y,w}}{\gamma_{m0}}$$

$$N_{rd;wc} = \frac{300 \times 15 \times 275}{1.1} \times 10^{-3} = 1125kN > N_{sd;wc} = 151.48kN$$ The condition is verified.

6.5.2.5. Verification of the bolts

We must verify that: $N_{sd;fc} \leq n_b \times min\{F_{v,rd}; F_{b,rd}\}$

$F_{v,rd} = 135.552kN$;

$F_{b,rd} = 290.4kN$ With: $a = min\{0.64; 0.64; 1.86; 1\} = 0.64$

$F_{v,sd} = 151.48kN < 8 \times min\{F_{v,rd}; F_{b,rd}\} = 542.208kN$ The condition is verified.

6.6. Column base

The calculation of the column base is done in “chapter 7”.

6.7. Conclusion

- Connections are a very sensitive part of the framework (structural elements); it requires specific and detailed verification of every component of the joint, including the connected parts.
- The connected parts must be checked for added stress due to an eccentricity of a load or the interaction between elements.
- Reinforcement must be added to the weaker parts of the connected elements to prevent failure.

(a) Failed connection (b) Bolt thread stripping

Figure 6. 13: Failure of a connection.

CHAPTER 07:

Infrastructure.

7. INFRASTRUCTURE

The infrastructure of a building is the part that it underground level, in our case it consists of the basement and the foundation, in this chapter we will be designing both of these component.

7.1. BASEMENT

7.1.1. Design of the composite columns

Given that the basement is underground, surrounded by the soil, we consider that it will undergo a rigid body displacement, it means that during a seismic event the columns will not undergo any displacement (this is just a hypothesis). Therefore we are going to design the columns under ULS combination.

7.1.1.1. Preliminary design

We designed the composite column using the simplified method (**6.7.3.EC04)** [**8**]

Data		
The height of the column	L (mm)	2800
Buckling length	L_f (mm)	2800
Yield strength S275	f_y(MPa)	275
Compressive strength of concrete	f_{ck}(MPa)	25
Yield strength of the reinforcement	f_{sk}(MPa)	500
concrete C25/30	E_{cm}(MPa)	30.5
Modulus of elasticity of reinforcements and steel	$E_{s,a}$(MPa)	$2{,}1\times 10^5$
covering of the reinforcements	c (mm)	40
Reducing coefficient	α	0,85
Partial safety factor of steel	γ_a	1,1
Partial safety factor of concrete	γ_c	1,5
Partial safety factor of reinforcement	γ_s	1,15

Table 7. 1: General data for composite columns.

a) Profile section

We choose an HEB450 profile for the basement with the following characteristics:

Symbol	Dimensions					
G (Kg/m)	**h** (mm)	**b** (mm)	**t**wa (mm)	**t**f (mm)	**r**(mm)	**A**(mm^2)
171	450	300	14	26	27	21800

y-y					z-z			
Iy mm^4 10^4	**W**el.y mm^3 10^3	**W**pl.y mm^3 10^3	**i**y mm 10	**A**vz mm^2 10^2	**I**z mm^4 10^4	**W**el.z mm^3 10^3	**W**pl.z mm^3 10^3	**i**z mm *10
79890	3551	3982	19.14	79.66	11720	781.4	1198	7.33

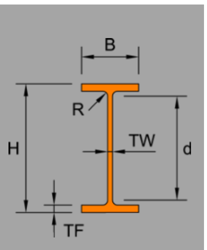


Table 7. 2: HEB450 profile characteristics.

b) Concrete section

- according to y-y: $40\ mm\ \leq c_y \leq\ 0{,}4.b\ \rightarrow 40\ mm\ \leq c_y \leq\ 0{,}4\times 300$

  $\rightarrow 40\ mm \leq\ cy\ \leq 120mm$ We choose $\boldsymbol{cy = 40mm}$
- according to z-z: $40mm \leq c_z \leq 0{,}3.h \rightarrow 40mm \leq c_z \leq 0{,}3\times 450$

$\rightarrow\ 40mm \leq c_z \leq 135\ mm$ We choose $\boldsymbol{c_z = 40mm}$

Therefore:

- b_c = 300 + (2 x 40)=400mm $\rightarrow b_c$ = 450mm
- h_c = 450 + (2 x 40)= 550 mm $\rightarrow h_c$ = 550mm
- Le rapport: $\frac{h}{b} = 1.22 \Rightarrow 0.2\ \leq h/b \leq\ 5 \Rightarrow 0.2\ \leq\ 1.22 \leq\ 5\quad ok$ **$(55cm \times 45cm)$**

c) Reinforcement section

0.3%$\leq A_r \leq$ 0.6% $A_C \Rightarrow A_r$= 0.50 % A_C; $As\ =\ 0.5$ % $[(550 \times 450) - 21800]$;

$\boldsymbol{As = 1128mm^2}$ we choose **4T20** ⟶ **A_r= 12.57 cm^2**

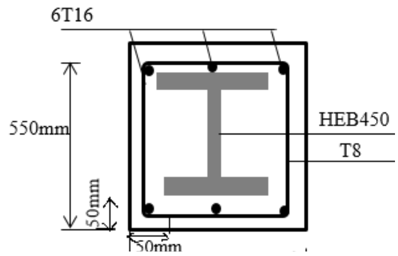


Figure 7. 1 : Composite column details.

d) Net concrete section

$A_{c\ net} = A_c - A_s - A_r \rightarrow A_{c\ nette} = 247500 - 21800 - 1257$

$\boldsymbol{A_{c\ net} = 224443\ mm^2}$

7.1.1.2. Loads assessment

$N_{sd}(basement)$ Is calculated in the following table:

$G_{composite\ column} = [(A_c \times \rho_c) + (A_r \times \rho_s) + (A_s \times \rho_s)] \times h$

$G_c = [(224443.10^{-6} \times 25) + (1257.10^{-6} \times 75) + (21800.10^{-6} \times 75)] \times 2.8 = 20.55kN$

Level	G	Q	1.35G+1.5Q
RDC	1049,16	431.75	2233.37
-1	1211.232	463.15	2329.88
-2	1391.92	486.7	2609.14

Table 7. 3 : Loads assessment for basement.

7.1.1.3. Verification of the composite column [**8**]

a) plastic resistance in axial compression

$$N_{pl,rd} = A_a.\frac{f_y}{\gamma_{ma}} + A_c.0{,}85\frac{f_{ck}}{\gamma_c} + A_s\frac{f_{sk}}{\gamma_s} = \left[\frac{275}{1{,}1}.21800\right] + \left[224443.0{,}85\frac{25}{1{,}5}\right] + \left[1257.\frac{500}{1{,}15}\right]$$

$$N_{pl,rd} = 8765.09kN$$

$$\delta = \left(\frac{A_a f_y}{\gamma_a}\right)/N_{pl,rd} = \frac{21800 \times 275}{1{,}1}/= 8765.09 \Rightarrow \delta = 0{,}62\ ;\ 0{,}2 < \delta < 0{,}9$$

b) Verification of the stability of composite column in axial compression

We need to check that $\begin{cases} N_{sd} \leq N_{by,rd}\ with\ N_{by,rd} = \chi_y.N_{pl,rd} \\ N_{sd} \leq N_{bz,rd}\ with\ N_{bz,rd} = \chi_z.N_{pl,rd} \end{cases}$

Expression	Numerical application	Results
$E_{cd} = \frac{E_{cm}}{\gamma_c^*}$	$E_{cd} = \frac{30500}{1,35}$	$E_{cd} = 24400$ MPa
$h^2 = \left(\frac{h_c}{2} - c - \frac{\varphi}{2}\right)^2$	$h^2 = \left(\frac{550}{2} - 50 - \frac{20}{2}\right)^2$	$h^2 = 46225\ mm^2$
$I_{sy} = A_s.h^2$	$I_{sy} = 1257 \times 46225$	$I_{sy} = 58104825\ mm^4$
$I_{cy} = \frac{b_c h_c^3}{12} - (I_{ay} + I_{sy})$	$I_{cy} = \frac{450\times550^3}{12} - (79890 \times 10^4 + 5810.48 \times 10^4)$	$I_{cy} = 53.8205.10^8 mm^4$
$(EI)_{ey} = E_a.I_{ay} + 0,8E_{cd}.I_{cy} + E_s.I_{sy}$ $(EI)_{ey}$= $(2,1\times 10^5 \times 79890 \times 10^4) + (0,8 \times 24400 \times 53.82 \times 10^8) + (2,1 \times 10^5 \times 5810.482 \times 10^4)$= $2.85.10^{14}$MPa		
$N_{cry} = \frac{\pi^2 (EI)_{ey}}{L_{fl}^2}$	$N_{cry} = \frac{\pi^2\times2.85\times10^{14}}{2800^2}$	N_{cry} =358815.07kN
$\bar{\lambda} = \sqrt{\frac{N_{pl,R}}{N_{cr}}}$	$\bar{\lambda}_y = \sqrt{\frac{8765.09}{358815.07}}$	$\bar{\lambda}_y = 0,15 < 0,2$
There is no risk of buckling according to axis y-y $\chi_y = 1 \rightarrow N_{sd} = 2609.14 < N_{pl,rd} = 8765.09kN$ The condition is verified.		

Table 7. 4: Verification of the stability of composite column in axial compression (Flexural buckling) y-y.

Expression	Numerical application	Results
$E_{cd} = \frac{E_{cm}}{\gamma_c^*}$	$E_{cd} = \frac{30500}{1,35}$	$E_{cd} = 24400$ MPa
$h^2 = \left(\frac{b_c}{2} - c - \frac{\varphi}{2}\right)^2$	$h^2 = \left(\frac{450}{2} - 50 - \frac{20}{2}\right)^2$	$h^2 = 27225mm^2$
$I_{sz} = A_s.h^2$	$I_{sz} = 1257 \times 27225$	$I_{sz} = 34221825\ mm^4$
$I_{cz} = \frac{h_c b_c^3}{12} - (I_{az} + I_{sz})$	$I_{cz} = \frac{550\times450^3}{12} - (11720 \times 10^4 + 3422.18 \times 10^4)$	$I_{cz} = 40.25 \times 10^8 mm^4$
$(EI)_{ez} = E_a.I_{az} + 0,8E_{cd}.I_{cz} + E_s.I_{sz}$ EI_{ez}= $(2,1\times 10^5 \times 11720 \times 10^4) + (0,8 \times 24400 \times 40.25\times 10^8) + (2,1 \times 10^5 \times 3422.18 \times 10^4)$= $1.038.10^{14}MPa$		
$N_{crz} = \frac{\pi^2 (EI)_{ez}}{L_{fl}^2}$	$N_{crz} = \frac{\pi^2\times1.038.10^{14}}{4000^2}$	N_{crz} =130795. kN
$\bar{\lambda}_z = \sqrt{\frac{N_{pl,R}}{N_{cr}}}$	$\bar{\lambda}_z = \sqrt{\frac{8765.09}{130795.64}}$	$\bar{\lambda}_z = 0,258 > 0,2$
We have $\frac{h}{b} = \frac{450}{300} = 1.5$ and $t_f < 40 \Rightarrow curve\ b \Rightarrow \alpha_z = 0,34$		
$\phi_z = 0,5[1 + \alpha_z(\bar{\lambda}_z - 0,2) + \bar{\lambda}_z^2]$	$\phi_z = 0,5 \times [1 + 0,34(0,258 - 0,2) + 0,258^2]$	$\phi_z = 0,54$
$\chi_y = \frac{1}{\phi_z + \sqrt{\phi_z^2 - \bar{\lambda}_z^2}}$	$\chi_z = \frac{1}{0,54+\sqrt{0,54^2-0,258^2}}$	χ_z =0.98
$\chi_z.N_{pl,rd}$	$0.98 \times 3950,7$	$\chi_z.N_{pl,rd} = 8640.83kN$
$N_{sd} = 2609.14 < \chi_z.N_{pl,rd} = 8640.83kN$ the condition is verified		

Table 7. 5: Verification of the stability of composite columns in axial compression (Flexural buckling) z-z.

The stability and resistance of composite columns in axial compression is verified.

7.1.1.4. Column base [13]

The base of the columns serves to transmit loads to the ground. These are metal plates fixed to the base of the columns by anchors on the concrete support. In our case, the column HEB450 is fixed at its base. The calculation of the base plate is done using robot structural analysis under the following loads: $N_{sd}max = -2945{,}60kN$; $M_{sd} = 77.9kN.m$; $V_{sd} = 26{,}92kN$

Autodesk Robot Structural Analysis Professional 2021

Fixed column base design

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

OK

Ratio 0,91

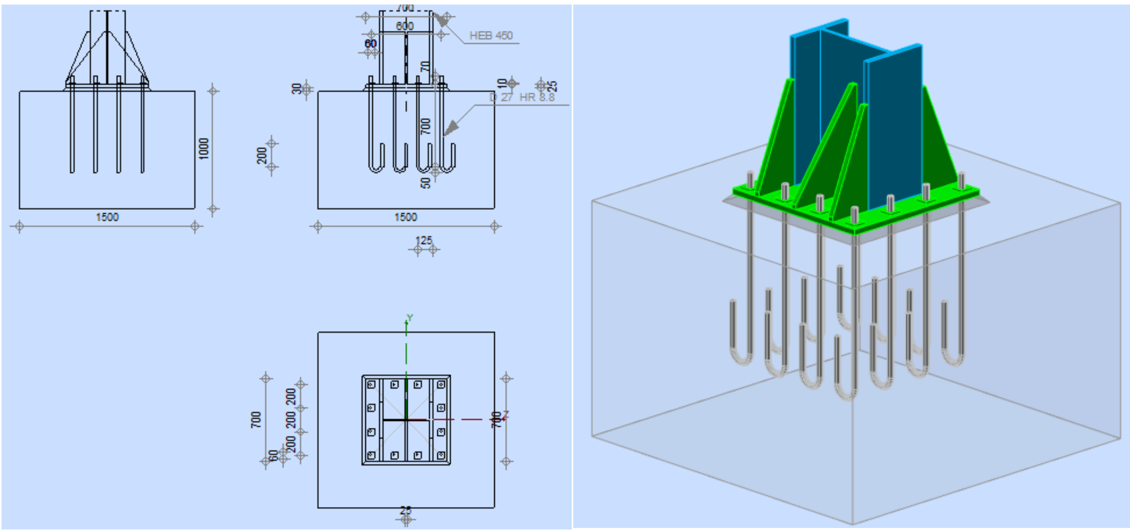


Figure 7. 2 : Base plate connection details.

a) Connection details :

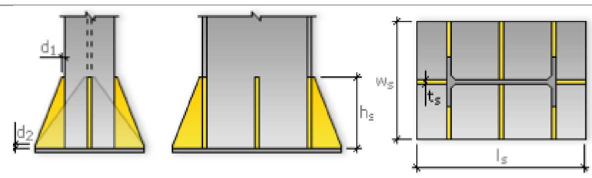
Anchor's dimensions						
$l_1 = 70mm$		$l_2 = 700mm$	$l_3 = 100mm$		$l_4 = 200mm$	
Anchor's plate						
Length (mm)		Width (mm)	Thickness (mm)		Grade of steel (Mpa)	
60		60	10		275	
Anchor's characteristics						
HR8.8	d=27mm	Fu=800Mpa	Nbh=4	Nbv= 4	Eh=200	Ev=200
Stiffeners						
I_s**(mm)**	W_s(mm)	h_s(mm)	t_s(mm)			
650	700	450	25			
spread footing (concrete)						
L=1500mm		B=1500mm	H=1000mm		C25/30	
Weld						
Footing plate of the column base			Stiffeners			
10mm			10mm			

Table 7. 6 : Column base connection details.

7.1.2. Design of the basement RC wall

A continuous peripheral wall will be provided between the level of the foundations and the basement floor level to ensure proper chaining of the building.

According to [9] **(10.1.2)** the peripheral basement RC wall must have the following characteristics:

- $t_{(PRCW)}$(thickness) $\geq 15cm$.
- The reinforcements are made up of two layers.
- The minimum percentage of reinforcement is 0.1% in both directions (horizontal and vertical).

Peripheral RC wall characteristics:

- We choose a e=20cm as thickness of the wall;
- specific weight of the soil($\gamma = 20kN/m^3$);
- angle of the ground friction ($\varphi = 35°$);
- The height is $h_0 = 2.8m$

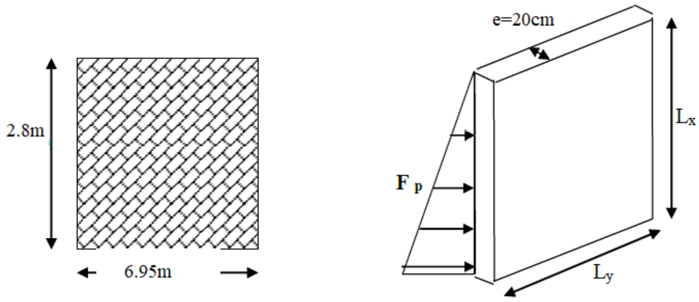


Figure 7. 3: Basement RC wall dimensions.

7.1.2.1. Loads assessment

The calculation of the peripheral wall comes down to the calculation of a panel of the slab fixed on its four sides, subjected to earth pressure.

a) Calculate the force of the earth push

$$F_p = \frac{1}{2} \times (K_a \times \gamma \times {h_0}^2) = \frac{1}{2} \times (0.27 \times 20 \times 5.6^2) = 84.672kN/ml$$

b) Loads combination and calculation

ULS: $\mathrm{Q_U} = 1.35 \times \mathrm{F_p} = 114.3kN/ml$ **SLS:** $\mathrm{Q_S} = \mathrm{F_p} = 84.672kN/ml$

c) Calculation of loads

For the reinforcement we take the largest panel whose characteristics are the following:

$L_x = 2.8$m and L_y= $6.95m$

According to [14] $\alpha = \frac{L_x}{L_y} = \frac{2.8}{6.95} = 0.41 > 0.4 \rightarrow$ the panel carries in both ways L_x and L_y.

Let "p" be the load applied by 1m² of the slab. For a strip of width 1 m, the bending moments at the center of the slab in both directions are given by the following expressions:

$$\begin{cases} M_x = \mu_x . p . {l_x}^2 \\ M_y = \mu_y . M_x \end{cases} with \begin{cases} \mu_x = \dfrac{1}{8(1 + 2.4 . \alpha^3)} \, with \, \mu_y \geq \mu_x \\ \mu_y = \alpha^3(1.9 - 0.9\alpha) \end{cases}$$

μ_x And μ_y are determined using the following table, the Poisson's ratio will be taken as $\upsilon = 0$ because we are calculating the loads operating on the element. (BAEL):

$\alpha = \frac{l_x}{l_y}$		0,40	0,50	0,60	0,70	0,80	0,90	1,00
$\upsilon = 0$	μ_x	0,1094	0,0946	0,0812	0,0683	0,0565	0,0458	0,0368
	μ_y	0,250	0,250	0,305	0,436	0,595	0,778	1,000
$\upsilon = 0{,}2$	μ_x	0,1115	0,0981	0,0861	0,0743	0,0632	0,0529	0,0442
	μ_y	0,293	0,373	0,476	0,585	0,710	0,846	1,000

Table 7. 7 : The values of μ_x and $\mu_y$according to α.

Calculation of the static moment:

ULS	SLS
$\begin{cases} \boldsymbol{M_{xu} = 0.1094 \times 114.3 \times 2.8^2 = 98.04 kN.m} \\ \boldsymbol{M_{yu} = 0.25 \times 98.04 = 24.51 kN.m} \end{cases}$	$\begin{cases} M_{xs} = 0.1094 \times 84.672 \times 2.8^2 = 72.62 kN.m \\ M_{ys} = 0.25 \times 72.62 = 18.155 kN.m \end{cases}$

Table 7. 8: Calculation of the static moment at ULS and SLS in both directions.

Shear force $T_{xu} = 177.23kN$ and $T_{yu} = 106.68kN$

7.1.2.2. Calculation of steel reinforcement ULS

The reinforcement will be calculated in both directions for a strip of length lx and ly and dimension (1m x h).

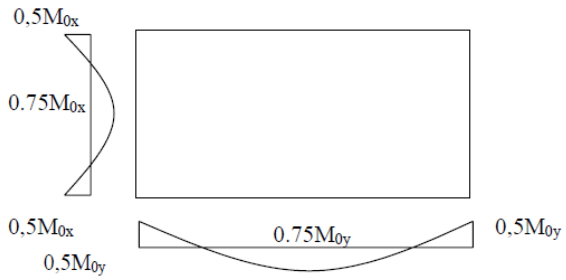


Figure 7. 4 : Static diagram for the basement RC wall.

a) Minimum section of reinforcement

Non-fragility condition d=17cm

$A_{NFC} = 0.23 \times b \times d \times \frac{f_{t28}}{f_e} \rightarrow A_{NFC} = 0.23 \times 100 \times 17 \times \frac{2.1}{500} \rightarrow \boldsymbol{A_{NFC} = 1.69cm^2}$

According RPA99v2003:$A_{RPA} = 0.1\%(b \times e) \rightarrow A_{RPA} = 0.1\%(100 \times 20) \rightarrow \boldsymbol{A_{RPA} = 2cm^2}$

b) Calculated section

The calculation of the reinforcement is done according to **ANNEX 1**, the results are demonstrated in the following table:

Direction	x-x		y-y	
	At support	On span	At support	On span
M_u (kN.m)	49.02	73.53	12.255	18.38
μ_U	$0.11 < \mu_R = 0.358$	$0.16 < \mu_R = 0.358$	$0.02 < \mu_R = 0.358$	$0.04 < \mu_R = 0.358$
α	0.179	0.232	0.035	0.053
Z(mm)	157.82	154.224	167.62	166.39
$A_s(cm^2)$	7.14	10.96	1.68	2.54
$A_{NFC}(cm^2)$	1.69	1.69	1.69	1.69
$A_{RPA}(cm^2)$	2	2	2	2
Choice	6T16	6T16	6T10	6T10
Spacing(cm)	15	15	15	15
$A_s(cm^2)$	12.06	12.06	4.71	4.71

Table 7. 9 : calculation of the reinforcement for basement RC wall.

7.1.2.3. Verification at SLS

The verification consists in limiting the stresses in the concrete and in the tensioned steels. The stress value in the serviceability limit state must not exceed the following limits:

-For concrete: $\overline{\sigma_{bc}} = 0.6 f_{c28} = 15MPa$

-For steel: $\overline{\sigma_{st}} = min\left\{\frac{2f_e}{3}; max\left[0.5f_e; 110\sqrt{\eta f_{tj}}\right]\right\} = min\{333.33; max[250; 201.63]\} = 250Mpa$

With: $\sigma_{bc} = \frac{M_s}{I} y$ and $\sigma_{st} = n\frac{M_s}{I}(d-y)$ (little damaging cracking).

According to BAEL y"" is obtained by resolving the following equation: $y^3 + py + q = 0$

Therefore: $I = \frac{1}{3} \times b \times y^3 + n \times A_{st(u)} \times (d-y)^2$ With n=15

-The table below sums up the verification results:

	axis	M_{ser}	$A_s(cm^2)$	Y(cm)	I(mm^4)	σ_{bc}	$\overline{\sigma_{bc}}$	check	σ_{st}	$\overline{\sigma_{st}}$	check
On span	x-x	54.46	12.07	6.353	31043,5.10^4	11.1	15	V	293	250	NV
	y-y	13.61	4.71	4,316	14960,1.10^4	3.93	15	V	179	250	V
At support	x-x	36.31	12.07	6.353	31043,5.10^4	7.43	15	V	195	250	V
	y-y	9.07	4.71	4,316	14960,1.10^4	2.62	15	V	119	250	V

Table 7. 10: Verification at SLS for the basement RC wall.

The condition is **NOT VERIFIED** for the x-x direction, therefore we need to increase the section of the reinforcement, and we choose 6T20 on span and at support we'll get:

	M_{ser}(kNm)	$A_s(cm^2)$	Y(cm)	I(cm^4)	σ_{bc}(MPa)	$\overline{\sigma}_{bc}$(MPa)	Vérification	σ_s(MPa)	$\overline{\sigma}_s$(MPa)	Vérification
Travée	54,460	18,850	7,515	42337,282	9,67	15	ok	192,67	201,6	ok
Appuis	36,310	18,850	7,515	42337,282	6,44	15	ok	128,46	201,6	ok

Table 7. 11: Second Verification at SLS for the basement RC wall in the x-x direction.

7.1.2.4. Shear force check

We must check that: $\tau_u = \frac{T_u^{max}}{bd} \leq \overline{\tau}_u = 0{,}05 f_{c28} = 1{,}25MPa$

$\tau_u = \frac{177.23\times10^3}{1000\times175} = 1.01MPa < 1.25MPa$ The condition is verified.

7.1.2.5. Concrete reinforcement layout drawing

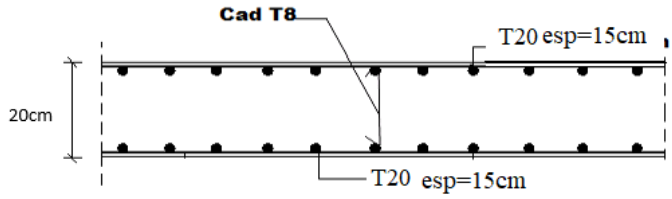


Figure 7. 5: Concrete reinforcement layout on span and at support x-x for basement wall.

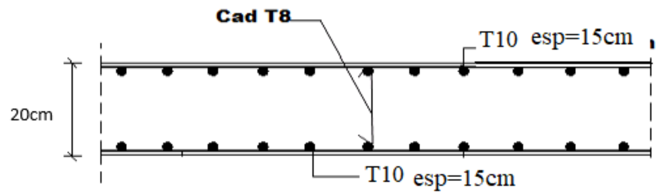


Figure 7. 6: Concrete reinforcement layout on span and at support y-y for basement wall.

7.2. FOUNDATION

Foundation is the lowest part of the building that is in direct contact with the soil which transfers loads from the structure to the soil safely. Generally, the foundation can be classified into two, namely shallow foundation and deep foundation.

A shallow foundation transfers the load to a stratum present in a shallow depth. The deep foundation transfers the load to a deeper depth below the ground surface**.**

The shallow foundation consists of:

- **Strap footing:** columns which are so closely spaced that their spread footings overlap or nearly touch each other.
- **Spread or isolated footing:** individual footing is provided to support an individual column.
- **Mat or raft foundation:** is a large slab supporting a number of columns and walls under the entire structure or a large part of the structure.

There are several factors that come into play when designing a footing (foundation), we can mention:

- σ soil: Stress of the soil ; admissible soil pressure equal to 2 bars.

- The classification of the soil.

- The efforts transmitted to the base...

7.2.1. Loads combination for designing footings

According to B.A.E.L, two loads combinations must be taken into account:

- **ULS : 1.35G+1.5Q**
- **SLS: G+Q**

According to RPA99V2003 (10.1.4.2) accidental load combinations must be taken into account:

- **G+Q+E**
- **0.8G∓E**

7.2.2. Footing choice

The choice of the foundation must satisfy the following criteria:

-Stability of the structure (rigidity).

- Ease of execution (formwork).

- Economy (reinforcement).

The surface of the foundation must be sufficient to distribute on the ground, the loads brought by the vertical carriers.

We start the choice of the foundation with the isolated footing; strap footing and then raft Foundation, each step will be checked.

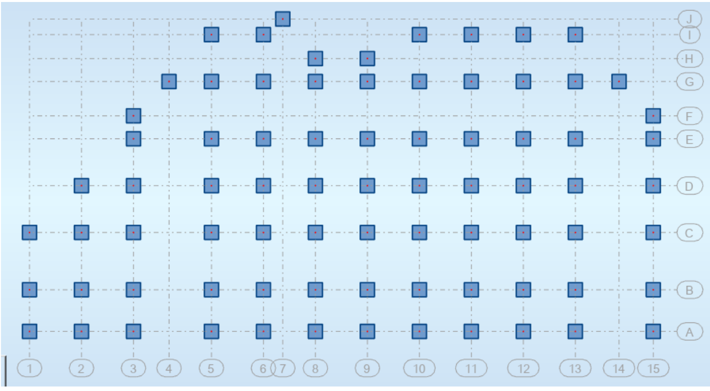


Figure 7. 7: arrangement of the columns.

7.2.2.1. Isolated footing

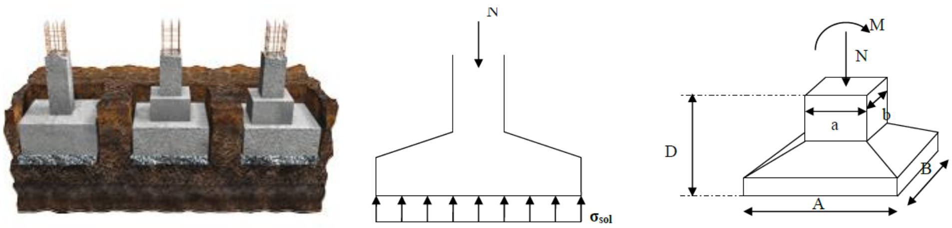


Figure 7. 8: Isolated footing.

Load combination (ELS)

We will adopt a homothetic sole, that is to say: The ratio of “A” to “B” is equal to the ratio “a” to “b”:

$\frac{A}{B} = \frac{a}{b}\, for\ a\ square\ column\ a = b\ , therefore\ A = B\ \rightarrow S = A^2$

A is determined by $A = \sqrt{S}\ such\ as\ S \geq \frac{N_{ser}}{\sigma_{soil}}$ we consider $S = \frac{N_{ser}}{\sigma_{soil}}$

To check the interference between two soles, check that:” L_{min}> 1.5xA” Such as:

L_{min} is the minimum center distance between two columns.

The results of the sections of the isolated footings are summarized in the following tables:

Line		column	N (KN)	S(m²)	A(m)	A(m)chosen	1.5xA	Lmin	CHECK
1		1A	234.26	1.17	1.08	1.5	2.25	2.5	V
		1B	240.64	1.2	1.09	1.5	2.25	2.5	V
		1C	252.51	1.26	1.12	1.5	2.25	2.5	V
2		2A	242.38	1.21	1.1	1.5	2.25	2.5	V
		2B	1532.44	7.66	2.76	3	4.5	2.5	NV
		2C	1503.53	7.51	2.7	3	4.5	2.5	NV
		2D	284.85	1.42	1.19	1.5	2.25	2.5	V
3		3A	276.35	1.38	1.17	1.5	2.25	2.5	V
		3B	1943.48	9.74	3.12	3.5	5.25	2.5	NV
		3C	1978.21	9.86	3.14	3.5	5.25	2.5	NV
		3D	336.81	1.68	1.29	1.5	2.25	2.5	V
		3E	209.56	1.04	1.01	1.5	2.25	2.5	V
		3F	228.4	1.142	1.06	1.5	2.25	2.5	V
4		4G	252.67	1.26	1.12	1.5	2.25	2.5	V
5		5A	277.7	1.38	1.17	1.5	2.25	2.5	V
		5B	1914.9	9.57	3.09	3.5	5.25	2.5	NV
		5C	2007.44	10.03	3.16	3.5	5.25	2.5	NV
		5D	1833.51	9.16	3.02	3.5	5.25	2.5	NV
		5E	1952.65	9.76	3.12	3.5	5.25	2.5	NV
		5G	1350.02	6.75	2.59	3	4.5	2.5	NV
		5I	263.66	1.31	1.14	1.5	2.25	2.5	V
6		6A	249.49	1.24	1.11	1.5	2.25	2.5	V
		6B	1557.61	7.78	2.78	3	4.5	2.5	NV
		6C	1633.91	8.16	2.85	3	4.5	2.5	NV
		6D	1515.14	7.57	2.75	3	4.5	2.5	NV
		6E	1636.13	8.18	2.86	3	4.5	2.5	NV
		6G	1626.63	8.13	2.85	3	4.5	2.5	NV
		6I	197.28	0.98	0.98	1.5	2.25	2.5	V
7		7J	130.23	0.65	0.8	1.5	2.25	2.5	V
8		8A	239.16	1.19	1.09	1.5	2.25	2.5	V
		8B	1554.92	7.77	2.78	3	4.5	2.5	NV
		8C	1633.91	8.16	2.85	3	4.5	2.5	NV
		8D	1497.55	7.48	2.73	3	4.5	2.5	NV
		8E	1633.73	8.16	2.85	3	4.5	2.5	NV
		8G	1188.29	5.94	2.43	3	4.5	2.5	NV
		8H	157.14	0.78	0.88	1.5	2.25	2.5	V
		9A	236.95	1.18	1.08	1.5	2.25	2.5	V
		9B	1551.92	7.75	2.78	3	4.5	2.5	NV
		9C	1633.91	8.16	2.85	3	4.5	2.5	NV

9		**9D**	1493.75	7.46	2.73	3	4.5	2.5	**NV**
		9E	1630.91	8.15	2.85	3	4.5	2.5	**NV**
		9F	1160.02	5.8	2.4	3	4.5	2.5	**NV**
		9H	151.66	0.75	0.88	1.5	2.25	2.5	**V**
10		**10A**	239.34	1.19	1.09	1.5	2.25	2.5	**V**
		10A	1553.41	7.76	2.78	3	4.5	2.5	**NV**
		10C	1632.4	8.16	2.85	3	4.5	2.5	**NV**
		10D	1495.21	7.47	2.73	3	4.5	2.5	**NV**
		10E	1632.41	8.16	2.85	3	4.5	2.5	**NV**
		10G	1565.77	7.82	2.79	3	4.5	2.5	**NV**
		10I	254.54	1.27	1.12	1.5	2.25	2.5	**V**
11		**11A**	241.83	1.2	1.09	1.5	2.25	2.5	**V**
		11B	1553.45	7.76	2.78	3	4.5	2.5	**NV**
		11C	1637.5	8.18	2.86	3	4.5	2.5	**NV**
		11D	1496.65	7.48	2.73	3	4.5	2.5	**NV**
		11E	1632.78	8.16	2.85	3	4.5	2.5	**NV**
		11G	1635.85	8.18	2.86	3	4.5	2.5	**NV**
		11I	247.16	1.23	1.1	1.5	2.25	2.5	**V**
12		**12A**	254.31	1.27	1.12	1.5	2.25	2.5	**V**
		12B	1558.07	7.79	2.79	3	4.5	2.5	**NV**
		12C	1637.5	8.18	2.86	3	4.5	2.5	**NV**
		12D	1521.36	7.6	2.75	3	4.5	2.5	**NV**
		12E	1637.19	8.18	2.86	3	4.5	2.5	**NV**
		12G	1627.78	8.13	2.85	3	4.5	2.5	**NV**
		12I	254.24	1.27	1.12	1.5	2.25	2.5	**V**
13		**13A**	278.69	1.39	1.17	1.5	2.25	2.5	**V**
		13B	1928.35	9.64	3.1	3.5	5.25	2.5	**NV**
		13C	2039.45	10.19	3.19	3.5	5.25	2.5	**NV**
		13D	1963.2	9.8	3.13	3.5	5.25	2.5	**NV**
		13E	2022.97	10.11	3.17	3.5	5.25	2.5	**NV**
		13G	1385.85	6.92	2.63	3	4.5	2.5	**NV**
		13I	268.89	1.34	1.15	1.5	2.25	2.5	**V**
14		**14G**	250.55	1.25	1.11	1.5	2.25	2.5	**V**
15		**A15**	305.75	1.52	1.23	1.5	2.25	2.5	**V**
		B15	316.6	1.5	1.22	1.5	2.25	2.5	**V**
		C15	278.12	1.39	1.18	1.5	2.25	2.5	**V**
		D15	251.72	1.25	1.11	1.5	2.25	2.5	**V**
		E15	207.94	1.03	1.01	1.5	2.25	2.5	**V**
		F15	219.78	1.09	1.04	1.5	2.25	2.5	**V**

Table 7. 12: Isolated footing verification.

Example:
N = 1943.48kN, S = N/σ =1943.48/200=9.71 m² , A=3.11m we choose: A= 3m

We must check: $Lmin > 1.5 \times A \rightarrow 2.5 < 5.25.$ The condition is not verified.

After these results, we notice that there is an overlap of the footings we then proceeds to the study of strap footing.

7.2.2.2. Strap footing

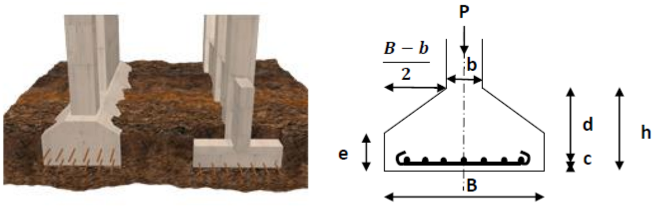


Figure 7. 9: Strap (continuous) footing.

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: $S \geq \frac{N_{ser}}{\sigma_{soil}}$ $with$ S= B.L
L = total length of the considered line. B = width of the footing.

Therefore, we calculate the width of the footing using: $\bar{\sigma}_{soil} \geq \frac{N}{S} = \frac{N}{B \times L} \rightarrow B \geq \frac{N}{\bar{\sigma}_{soil} \times L}$

All the results are summarized in this following table:

line	N (KN)	L(m)	B(m)	B(m)chosen	2.5B’	Lmin	CHECK
1	727.41	9.5	0.38	**1**	2.5	5	V
2	3563.2	14	1.27	**1.5**	3.75	5	V
3	4972.81	20.7	1.24	**1.5**	3.75	5	V
4	252.67	**One column in this line (will be joint to the previous line)**					
5	9599.88	28.53	1.68	**2**	5	5	V
6	8416.19	28.53	1.47	**1.5**	3.75	5	V
7	130.23	**One column in this line (will be joint to the previous line)**					
8	7904.7	26.24	1.5	**2**	5	5	V
9	7859.11	26.24	1.49	**1.5**	3.75	5	V
10	8372.6	28.53	1.49	**1.5**	3.75	5	V
11	6975.22	28.53	1.22	**1.5**	3.75	5	V
12	8490.45	28.53	1.48	**1.5**	3.75	5	V
13	9887.4	28.53	1.73	**2**	5	5	V
14	250.55	**One column in this line (will be joint to the previous line)**					
15	1579.91	20.7	0.38	**1.5**	3.75	5	V

Table 7. 13: Strap footing verification.

Example for line 13

$B \geq \frac{N}{\bar{\sigma}_{soil} \times L} = \frac{9887.4}{200 \times 28.53} = 1.73m\ B_{chosen} = 2m$ $L_{min} = 5m$(Min distance between lines).

$We\ must\ check$ $L > 2{,}5B'$ Such as B’ is the distance between 2 straps

$0 \rightarrow 5m < \frac{(2+1{,}5) \times 2{,}5}{2} = 4.375m$ The condition is verified.

We must also check the surface that the strap footing occupies does not exceed 50% of the total surface of the building $\rightarrow \frac{S_s}{S_t} \leq 50\% \rightarrow \frac{479.24}{1507.05} = 0.32 = 32\% < 50\%$ the condition is verified.

All the condition of use of the strap foundation are satisfied.

7.2.3. Preliminary design of a strap footing [15]

According to B.A.E.L the preliminary design must be done using the SLS combination.

Verification of the load-bearing condition taking into account the self-weight of the footing

$\bar{\sigma}_{soil} \geq \frac{N}{S} \rightarrow \frac{9887.4}{2\times 28.53} = 1.73bar < 2bar$ The condition is verified

a) rigidity condition $h \geq \frac{B-b}{4} + 0.05\ (m)$ (See figure 7.9)

- h (total height of the footing) must be greater than or equal to 15 cm.
- The edge height e of the trapezoidal flanges is 10 to 15−20 cm;

$h \geq \frac{2-0.55}{4} + 0.05 = 0.41m$ $we\ choose$ h = 0.50m

For construction reasons, the height of the shoes e is given as function of the diameter $\emptyset$ of the tension reinforcements. $e \geq max \begin{cases} 15cm \\ 6\emptyset + 6\text{cm for bars without a hook} \\ 12\emptyset + 6\text{cm for bars witht a hook} \end{cases}$

b) Punching verification

We must verify that: $q_{u(most\ stressed\ column)} = 2858.71kN \leq 0.045 \times u_c \times h \times \frac{f_{c28}}{\gamma_b}$

$u_c = (a + b + 2\times h) \times 2 = (0.55 + 0.45 + 2 \times 0.5) \times 2 = 4m\ ;\ f_{c28} = 30Mpa$

$q_{u(most\ stressed\ column)} = 2858.71kN \leq 1800kN$ Not verified.

Increase the height of the footing. $h = 70cm \rightarrow 2858.71kN \leq 3024kN$ Verified.

c) Stress check N=9887.4 kN; M=144.32kN.m

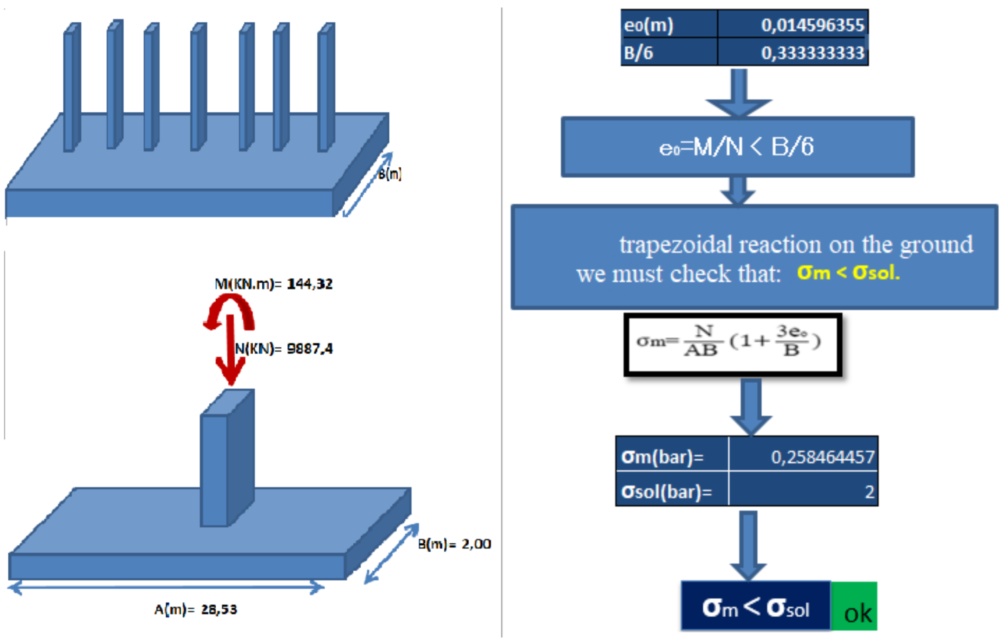


Figure 7. 10: Stress verification for the strap footing (line13)

7.2.4. Reinforcement [15]

The reinforcement is calculated method for 1 linear meter

a) **Main reinforcement** $A_s = \frac{N_U(B-b)}{8.d.\sigma_{st}} = \frac{2858.71\times10^3\times(2000-550)}{8\times650\times\frac{500}{1.15}} = 18.33cm^2$

b) **Secondary reinforcement** $A_r = \frac{A_s}{3} = \frac{18.85}{3} = 6.28cm^2$

We choose $\begin{cases}\text{main reinforcement } 6\text{HA20 per linear meter with } A_s = 18.85\text{cm}^2 \\ \text{secondary reinforcement } 6\text{HA12 } A_r = 6.79\text{cm}^2\end{cases}$

Spacing:

The reinforcement will be distributed along the strap foundation:

For the main reinforcement we used HA20 every 17cm and for the secondary reinforcement we used 6HA12 distant by 25cm.

For the height of the shoe $e \geq max\begin{cases}15cm \\ 12\times1.6+6\text{cm}=30\text{cm}\end{cases} \rightarrow e = 30cm$

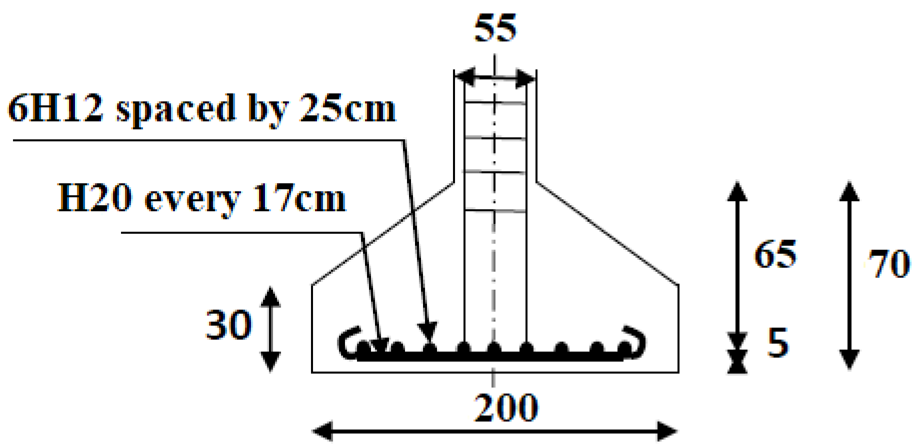


Figure 7. 11: Reinforcement drawing for the strap foundation.

7.3. Conclusion

The foundation technique therefore simultaneously concerns two problems: the evaluation of the loads taken up by the soil and the soil bearing capacity (usually given by the geotechnical report), and then the design of the intermediate element which transfers these loads.

The use of strip footings in this case is allowed because of the lightness of the structure. In this case is more economical, by avoiding having a raft, we saved in concrete.

CHAPTER 08:

Soil-Structure interaction.

8. SOIL-STRUCTURE INTERACTION

8.1. What is soil-structure interaction?

Most of the civil engineering structures involve some type of structural element with direct contact with ground. The ground is treated as if it is infinitely stiff, and the structure sitting on it is just going to shake back and forth following the ground motion as if the structure perfectly fixed in the ground; this hypothesis is valid as long as the soil is not very deformable, for example in the case of a rigid building on rocky ground. Up to this point this is how we've been assuming that the structure behaves, **but in reality is this assumption valid?**

The response is simply no.

When the external forces, such as earthquakes, act on these systems, neither the structural displacements nor the ground displacements, are independent of each other. The process in which the response of the soil influences the motion of the structure and the motion of the structure influences the response of the soil is termed as soil-structure interaction (SSI).The movement of the ground-structure system depends on the mechanical characteristics of each one of them, therefore various combinations of soil and structure can either amplify or diminish movement and subsequent damage.

The response of a structure to earthquake shaking is affected by interactions between three linked systems: the structure, the foundation, and the soil underlying and surrounding the foundation [16] **.**Soil-structure interaction analysis evaluates the collective response of these systems to a specified ground motion. The terms Soil-Structure Interaction (SSI) and Soil-Foundation-Structure Interaction (SFSI) are both used to describe this effect in the literature.

SSI effects are categorized as inertial interaction effects, kinematic interaction effects and and soil-foundation flexibility effects. Robert Whitman (Kausel, 2010) introduced the terms kinematic and inertial interaction in 1975. In the context of engineering analysis and design, these effects are related to:

- **Foundation stiffness and damping.**

Inertia developed in a vibrating structure gives rise to base shear, moment, and torsion. These forces generate displacements and rotations at the soil-foundation interface. These displacements give rise to energy dissipation via radiation damping and hysteretic soil damping, which can significantly affect overall system damping. Since these effects are rooted in structural inertia, they are referred to as **inertial interaction** effects.

- **Variations between foundation input motions and free-field ground motions.**

Foundation input motions and free-field motions can differ because of:

-**Kinematic interaction**, in which stiff foundation elements placed at or below the ground surface cause foundation motions to deviate from free-field motions due to base slab averaging, wave scattering, and embedment effects in the absence of structure and foundation inertia;

- Relative displacements and rotations between the foundation and the free-field associated with structure and foundation inertia.

(The term free-field refers to motions that are not affected by structural vibrations or the scattering of waves at, and around, the foundation.)

- **Foundation Deformations**

Forces and displacements applied by the superstructure and the soil induce flexural, axial, and shear deformations of structural foundation, they represent seismic demands for which foundation components should be designed, and they could be really important, especially for flexible foundations such as rafts and piles.

But why is it so important to take into consideration the soil structure interaction?

8.2. Why is it important to take into account the soil-structure interaction?

(Importance of inclusion of the soil-structure interaction)

The effects of soil-structure interaction (ISS) on the seismic response have not been seriously considered that after the 1971 earthquake in San Fernando and nuclear construction begins in California. The catastrophic consequences of many recent earthquakes in different parts of the world, such as the 1995 Kobe earthquake, have highlighted that the seismic behavior of a structure is highly influenced not only by the response of the superstructure, but also by the response of the foundation and the ground as well. Hence, the modern seismic design codes, such as Standard Specifications for Concrete Structures: Seismic Performance Verification JSCE 2005 stipulate that the response analysis should be conducted by taking into consideration a whole structural system including superstructure, foundation and ground.

During an earthquake, the waves generated from the hypocenter, the point of origin of the earthquake along a fault, propagate radially in the ground .The waves propagating in this way are modified according to the characteristics of the site's soil and are reflected until they reach the foundations of the structures (Figure 8.1). The excitement of the foundation in turn causes the excitement of the superstructure.

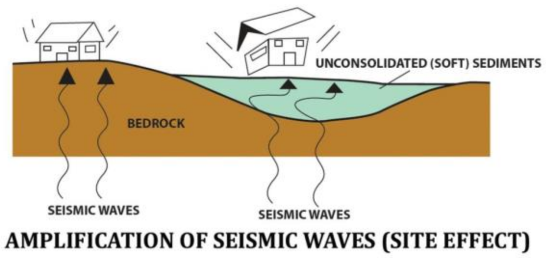


Figure 8. 1: Amplification of seismic waves (site effect).

Therefore, it is particularly important to consider the ISS in seismic areas where the dynamic response of soils can change the response of structures subjected to seismic excitation. Moreover, soft sites (soft ground) receiving rigid and massive structures can change the dynamic characteristics of the latter significantly.

We can list different scenarios of what could actually or more likely happen due to soil deformability:

- An increase of the vibration period of the first, which can cause a variation in addition or in less of the value of the acceleration according to the zone where the system is located on the elastic spectrum;

- A non-negligible damping (radiative damping + damping specific to the soil material)

- A rotation of the foundation which can significantly modify the calculation of the modal deformation and therefore the distribution of accelerations along the height of the building;

- The movement of the ground at the base of the building is assumed to be identical to that of the free field. In current cases, however, we see that this approximation is acceptable,

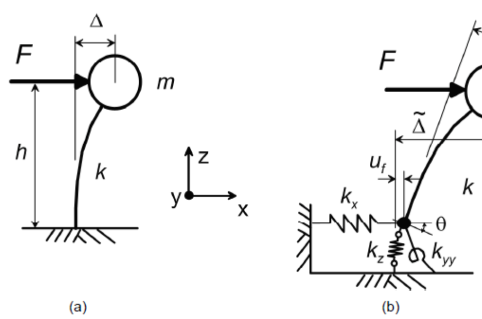


Figure 8. 2: Schematic illustration of deflections caused by force applied to: (a) fixed-base structure; and (b) structure with vertical, horizontal, and rotational flexibility at its base.

The figure bellow represent three different combinations of soil/structure:

a) The building is represented as flexible and perfectly fixed at the base.
b) A rigid building on a rigid soil.
c) A rigid building on a soft soil.

It is clear that the behavior of the soil-structure system is dependent on the mechanical characteristics of each one of the components.

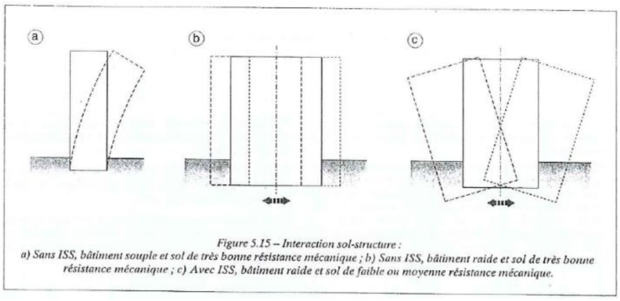


Figure 8. 3: soil structure interaction under various combination of soil/structure types.

8.3. When is it important to take into account the soil structure interaction?

(Cases where the soil-structure interaction must be taken into account)

Conventional structural design methods neglect the SSI effects. It is reasonable for light structures in relatively stiff soil such as low rise buildings and simple rigid retaining walls. The effect of SSI, however, becomes prominent for heavy structures resting on relatively soft soils for example nuclear power plants, high-rise buildings and elevated-highways on soft soil.

Eurocode 8 advises to study ISS when:

- Structures are unstable.
- The foundations are massive and deep.
- The structures are slender.
- The ground is not very rigid.

This is especially applicable to areas of high seismic activity.

The soil structure interaction is function of the following;

- The height or the slenderness of the structure relative to footing width;
- The stiffness of the structure relative to the stiffness of the soil;
- Mass of the structure relative to the mass of the soil supporting the footing;

8.4. Method of analysis using soil-structure interaction

The SSI problem is not easy to solve; especially when the system is geometrically complex or contains significant nonlinearities in the soil or structural materials, therefore it is rarely used in practice. There are two general ways to solve for SSI

a) Direct SSI analysis(Global analysis) :

In direct analysis the soil and structure are included within the same model and analyzed as a complete system ;it consist in directly solving the equation of dynamics controlling the behavior of the system {soil + foundation + structure}:

$$\underline{\underline{\mathbf{M}}}.\underline{\ddot{\mathbf{u}}} + \underline{\underline{\mathbf{C}}}.\underline{\dot{\mathbf{u}}} + \underline{\underline{\mathbf{K}}}.\underline{\mathbf{u}} = -\underline{\underline{\mathbf{M}}}.\underline{\mathbf{I}}a$$

- u: represents the displacement vector of the system according to the reference frame subjected to the acceleration **a** according to a Galilean coordinate system,

- I: a vector containing 1s for the directions subjected to the acceleration a,

- M, C, and K: the mass matrixes, damping and stiffness of the system.

- The symbol ·: represents the derivatives with respect to time.

The direct resolution of this system of equation uses the classical algorithms of the finite element method.

In this method, the ground is often discretized by solid elements and the superstructure by beam elements. Although this method provides a complete response from the soil-structure interaction, it however requires several difficult data to obtain. For example, this method requires the specification of a seismic signal on the surface between rock and ground, which is not currently available since the recordings are made at a single point. Moreover, the direct method requires a model of the ground on a sufficiently large space to adequately represent the modification of the seismic waves which implies a rather heavy numerical model.

Evaluation of site response using wave propagation analysis through the soil is important to this approach. Such analyses are most often performed using an equivalent linear representation of soil properties in finite element, finite difference, or boundary element numerical formulations [17]. direct analyses can address all of the SSI effects described above, but incorporation of kinematic interaction is challenging because it requires specification of spatially variable input motions in three dimensions.

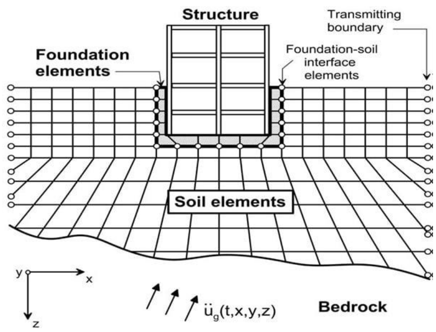


Figure 8. 4: Finite elements method for SSI modelling.

b) Indirect SSI analysis (substructure method) :

The substructure method, introduced by Kausel and Roesset (1974), aims to incorporate the soil-structure interaction into numerical analyzes by partitioning the problem into distinct parts which are then recombined to form the complete solution. Since the method requires the assumption of superposition, it assumes a linear response from the ground and the superstructure. On the other hand, this assumption is generally only respected in a linear-equivalent sense[**18**]and the numerically modeled elements of the ground and the superstructure deform in a non-linear way.

Basically, we use simplified linear solutions to solve the kinematic and the inertial responses from the SSI separately, then just add the two together to get the total response, valid for linear systems only.

8.4.1. Hybrid method

The hybrid methods consist in using a combination of the two other methods seen above while taking advantage of both of them. Thus hybrid methods aim to break down the system into two subdomains. The first subdomain: the “far field” is far enough from the foundation to be considered elastic. Its behavior can then be governed by dynamic impedances. The second subdomain: “the near field” is considered as having a non-linear behavior.

The modelling is achieved by partitioning the total soil-structure system into a near-field and a far-field with hemispherical interface. The near-field, which consists of the structure to be analyzed and a finite region of soil around it, is modelled by the finite element method. For the semi-infinite far-field, impedance matrix corresponding to the interface degrees of freedom is developed which accounts for the loss of energy due to waves travelling away from the foundation. For torsional vibrations, the far-field impedance matrix can be determined analytically. For general loading conditions a semi-analytical approach is adopted in which the far-field is modelled through continuous impedance functions placed in the three coordinate directions at the interface. These frequency dependent impedance functions are determined by using system identification methods such that the resulting hybrid model reproduces the known compliances of a rigid circular plate on an elastic half-space. Numerical results obtained using these far-field impedances indicate that the proposed model presents a realistic and economic method for the analysis of three-dimensional soil-structure interaction in surface or embedded structures.

8.5. Method of modelling of the soil-structure interaction

The methods of numerical simulations are classified into three types, substructure method, the finite element method and the hybrid method[**3**].

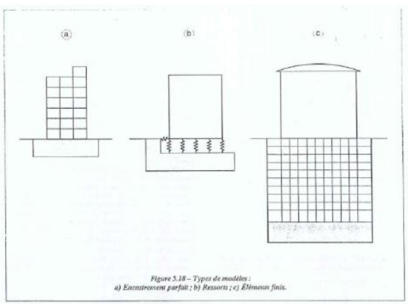


Figure 8. 5: Types of modelling of the structure a) fixed base method; b) springs method; c)finite element method.

8.5.1. Springs method (NEWMARK-ROSENBLUETH).

The ground is represented by springs connecting one or more nodes to a rigid base, see figure (8.5) our study is done on a type of foundation, which is a strap footing, so the ground will be modeled by horizontal springs, vertical springs and rotations. The stiffness of these springs is calculated by the formulas of Newmark & Rosenblueth [19] and their stiffness is equal to the stiffness of the soil. The ground spring method is based on an elastic reaction and does not take into account the mass of soil participating in the movement.

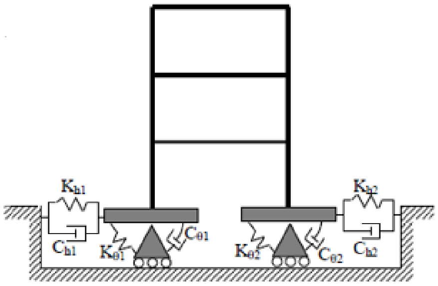


Figure 8. 6: Representation of the NEWMARK-ROSENBLUETH SSI modelling method.

8.5.2. Finite elements method

The ground is modeled as an assembly of rectangular elements in plane strain having two degrees of freedom in translation at each node, while the building structure is modeled as an assembly of beam elements. Because soil elements do not allow a rotational degree of freedom, the nodes of the soil-structure interface require special consideration, as shown in figure 8.6

In our case we are going to apply the soil-structure interaction on our structure using the springs approach.

8.6. Application of the soil-structure interaction on our structure:

8.6.1. Introduction

The purpose of this study is the determination of the effect of the soil-structure interaction on our structure, mainly and over view on the dynamic response of the structure. The study is carried out using Sap2000 where the structure is modeled by bar elements and the ground by springs with the same soil mechanical characteristics. The latter are evaluated using **Newmark-Rosenblueth** expression**s** and the characteristics of site categories by the Algerian code (RPA99).

Finally we will make a comparison between the first model were we did not considered the soil structure interaction (fixed base) and a second model where we added the basement floors ,the footing and modeled the soil structure (springs).

8.6.2. Reference model (fixed base model)

In order to make a good comparison we had to RE-model the structure on SAP2000 to get accurate results.

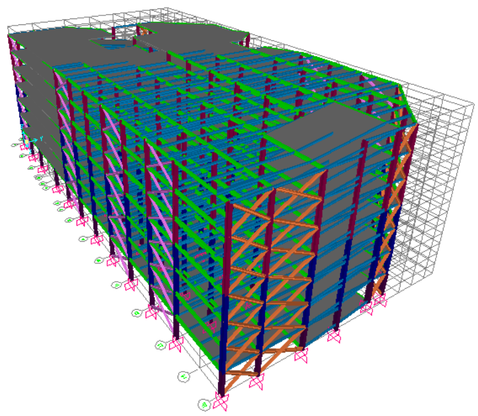


Figure 8. 7:3D fixed base Model on SAP2000.

8.6.2.1. Dynamic analysis

Mode	Period	effective mass%x	effective mass%y
1	0.799	1.01	73.18
2	0.7	72.07	1.24
3	0.679	0	0.43

Table 8. 1: Modal analysis results on SAP2000. (Fixed base model)

- $T_{analytical} = 0.79s \; < T_{emp} = 0.83$
- There was no need to increase the base shear force (see chapter 4) $\begin{cases} V_x = 8901{,}6kN \\ V_y = 8389{,}1kN \end{cases}$
- The mass participation reached 90% at the 30th mode for x-x axis and the 89th mode for y-y axis.

8.6.2.2. Inter-story drift check

sUx (storey drift) ; isUx (inter-storey drift)

Bloc A

Floor level	Allowable isU(cm)	sUx (cm)	isUx (cm)	Note	sUy (cm)	isUy (cm)	Note
1(+2.8)	2.8	0.9	0,9	Verified	1.3	1,3	Verified
2(+5.6)	2.8	2.7	1,8	Verified	3.3	2	Verified
3(+8.4)	2.8	4.8	2,1	Verified	5.7	2,4	Verified
4(+11.2)	2.8	7	2,2	Verified	8.3	2,6	Verified
5(+14)	2.8	9.1	2,1	Verified	10.9	2,6	Verified
6(+16.8)	2.8	11	1,9	Verified	13.3	2,4	Verified
7(+19.6)	2.8	12.6	1,6	Verified	15.4	2,1	Verified

Table 8. 2: Inter storey-drift check for fixed base model (block A).

Bloc B

Floor level	Allowable isU(cm)	sUx (cm)	isUx (cm)	Note	sUy	isUy (cm)	Note
1(+4.2)	2.8	1.4	1,4	Verified	2	2,0	Verified
2(+7)	2.8	3.2	1,8	Verified	4.1	2,1	Verified
3(+9.8)	2.8	5.1	1,9	Verified	6.3	2,2	Verified
4(+12.6)	2.8	7	1,9	Verified	8.8	2,5	Verified
5(+15.4)	2.8	8.8	1,8	Verified	11.3	2,5	Verified
6(+18.2)	2.8	10.4	1,6	Verified	13.6	2,3	Verified
7(+21)	2.8	11.8	1,4	Verified	15.6	2,0	Verified

Table 8. 3: Inter storey-drift check for fixed base model (block B).

8.6.2.3. Internal efforts

columns		
$M_{max} = 236{,}90kN.m$	$V_{max} = 114{,}66kN$	$N_{max} = 4191{,}46kN$
Main beams		
$M_{max} = 225{,}71kN.m$	$V_{max} = 154{,}33kN$	
Bracing		
x-x		
N_{max} (tension)$= -522{,}33kN$	N_{max} (compresion)$= 542{,}90kN$	
y-y		
N_{max} (tension) $= -867{,}58kN$	N_{max}(compression) $= 964{,}42kN$	

Table 8. 4: Internal forces for the fixed base model.

Note: The dynamic analysis of the structure on SAP2000 gave us similar results as the ones we found on ROBOT STRUCTURAL ANALYSIS.

8.6.3. Springs method model

The soil is represented by a system of discrete elastic translational springs (vertical, horizontal translation and rotation) such as:

- vertical translation stiffness K_v
- horizontal translation stiffness K_h
- rotation stiffness K_θ

The stiffness is calculated for each seismic direction (horizontal, vertical and rotation) and they are calculated according to the shape of the footing (foundation). In our case we assume that the foundation is a rectangular raft (for practical reasons). The static stiffnesses of a rectangular surface foundation on a homogeneous ground are given by Newmark-Rosenblueth expression as shown in the following table:

Degrees of freedom	Static stiffness
Vertical	$K_v = \frac{G}{1-v}\beta_z \times \sqrt{A}$
Horizontal	$K_h = 2(1+v)\ G\beta_x \times \sqrt{A}$
Sway	$K_\theta = \frac{1+v}{4}G\beta_x(a^2+b^2)\sqrt{A}$
Torsion	$K_\phi = \frac{G}{1-v}\beta_\phi a^2 b$
G is the shear modulus of the soil; $\boldsymbol{\nu}$ is the Poisson's modulus of the soil; $\boldsymbol{\beta_x}$; $\boldsymbol{\beta_z}$; $\boldsymbol{\beta_\phi}$are parameters depending on the ratio $\frac{\boldsymbol{a}}{\boldsymbol{b}}$ and they are given by (figure8.8) abacus , such as: a: dimension parallel to the direction of the earthquake b: dimension perpendicular to the direction of the earthquake	

Table 8. 5 : Newmark-Rosenblueth expressions for the static stiffnesses of a rectangular surface foundation.

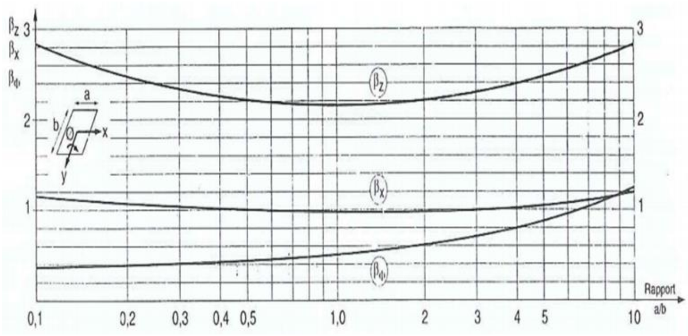


Figure 8. 8: Abacus giving the Newmark coefficients. (Buchi, 2013).

8.6.3.1. Springs stiffness calculation

Calculation parameters		
$\begin{cases} x-x \rightarrow a = 60m \text{ and } b = 30m \rightarrow \beta_x = 1;\ \beta_z = 2.2;\ \beta_\phi = 0.5 \\ y-y \rightarrow a = 30m \text{ and } b = 60m \rightarrow \beta_x = 1;\ \beta_z = 2.2;\ \beta_\phi = 0.4 \end{cases}$; $A = 1507.05m^2$ G=239.5Mpa; $\nu = 0.44$;		
RESULTS		
Degrees of freedom	**Static stiffness of the springs**	**Static stiffness of one spring** $\frac{K}{n_{nodes\ under\ raft=2710}}$
Shift about z-z axis	$K_z = 36526158.74kN/m$	13478.28kN/m
Shift about y-y axis	$K_y = 26776994.92kN/m$	9880.81kN/m
Shift about x-x axis	$K_x = 26776994.92kN/m$	9880.81kN/m
Torsion about z-z axis	$K_\phi = 1.01616 \times 10^{10}kN/m$	3749667.879kN/m
Sway about y-y axis	$K_{\theta y} = 1.506206 \times 10^{10}kN/m$	5557955.72kN/m
Sway about x-x axis	$K_{\theta x} = 1.506206 \times 10^{10}kN/m$	5557955.72kN/m

Table 8. 6: Springs stiffness calculation.

8.6.3.2. Model

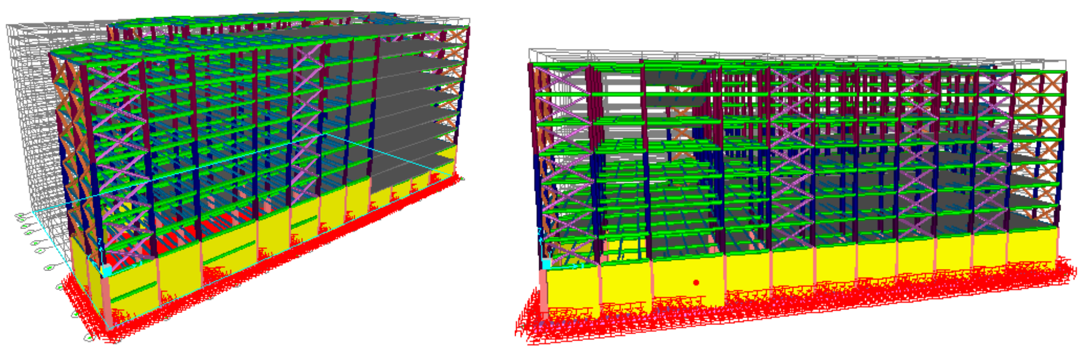


Figure 8. 9: 3D spring model.

8.6.3.3. Dynamic analysis

The dynamic analysis of the structure on SAP2000 gave the following results:

Mode	Period	effective mass%x	effective mass%y	displacement
1	1.34	0.068	62.56	Shift about y-y
2	1.1	64.68	0.051	Shift about x-x
3	0.66	0.001	0.008	Torsion

Table 8. 7: Modal analysis results on SAP2000. (Springs model)

- $T_{analytical} = 1{,}34s > T_{emp} = 0.83$
- There was no need to increase the base shear force (see chapter 4)
- The mass participation reached 90% at the 23td mode for x-x axis and the 20th mode for y-y axis.

8.6.3.4. Base shear check

$$Q_{x,y} = 1.05\ ;\ A = 0.25\ ;\ R_{x,y} = 4\ ;\ D_{x,y} = 1.66\ ;\ W = 111938{,}057kN$$

The lateral force will be calculated using the previous expression (chapter4)

Direction	V(response-spectrum method)	V_t(lateral force method)	$0.8 \times V_t$	$\frac{0.8V}{V_t}$	Observation
x	$10270.35kN$	10974.82kN	8779.86kN	$0.85 < 1$	No need to Increase the seismic action
y	$9336.34kN$	10974.82kN	8779.86kN	$0.94 < 1$	

Table 8. 8: Base shear verification.

8.6.3.5. Inter-story drift check **sUx(storey drift) ; isUx(inter-storey drift)**

Bloc A

Floor level	Allowable isU(cm)	sUx (cm)	isUx (cm)	Note	sUy (cm)	isUy (cm)	Note
0		6,39			7,3		
1(+2.8)	2.8	8,22	1,083	Verified	9,7	2,4	verified
2(+5.6)	2.8	10,6	2,38	Verified	12,77	3.07	Not verified
3(+8.4)	2.8	13,17	2,57	Verified	16,13	3.36	Not verified
4(+11.2)	2.8	15,73	2,57	Verified	19,6	3,47	Not verified
5(+14)	2.8	18,25	2,5	Verified	21,87	2,27	Verified
6(+16.8)	2.8	20,6	2,35	Verified	26,4	4,53	Not verified
7(+19.6)	2.8	22,6	2	Verified	29,6	3,2	Not verified

Table 8. 9: Inter storey-drift check for spring model (block A).

Bloc B

Floor level	Allowable isU(cm)	sUx (cm)	isUx (cm)	Note	sUy	isUy (cm)	Note
1,4		6,96			7,95		
1(+4.2)	2.8	8,8	1,84	Verified	10,9	2,95	Not verified
2(+7)	2.8	11,2	2,4	Verified	14,2	3,3	Not verified
3(+9.8)	2.8	13,5	2,3	Verified	17,5	3,3	Not verified
4(+12.6)	2.8	15,8	2,3	Verified	20,7	3,2	Not verified
5(+15.4)	2.8	18,1	2,3	Verified	24,2	3,5	Not verified
6(+18.2)	2.8	20,1	2	Verified	27,4	3,2	Not verified
7(+21)	2.8	21,9	1,8	Verified	30,5	3,1	Not verified

Table 8. 10: Inter storey-drift check for the springs model (block B).

8.6.3.6. Internal forces

Most stressed column		
$M_{max} = 244{,}26kN.m$	$V_{max} = 464.84kN$	$N_{max} = 3533{,}904kN$
Main beam		
$M_{max} = 236{,}8kN.m$	$V_{max} = 89.39kN$	
Bracing		
x-x		
N_{max} (tension) $= 465.34kN$	N_{max} (compresion)= $517{,}482kN$	
y-y		
N_{max} (tension)=$704.59kN$	N_{max}(compression) $= 938.392kN$	

Table 8. 11: Internal forces for springs model.

8.7. Comparison

After previously evaluating the two models (fixed base and spring model), we observed the following:

a) Increase in the period value

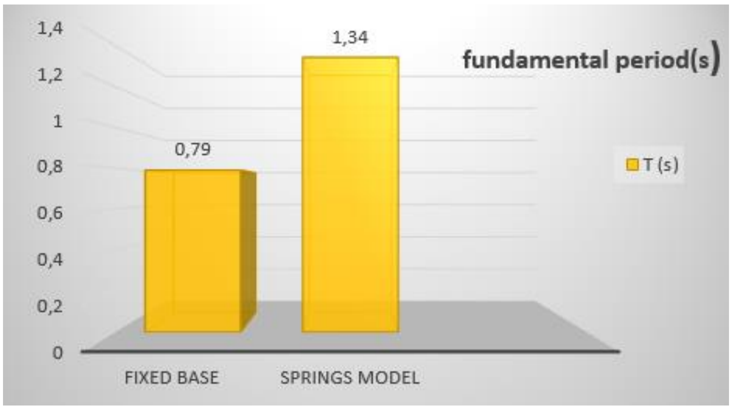


Figure 8. 10: Fundamental period comparison.

As it is clear in figure 8.10 the fundamental period of the building incrased significantly between the fixed base modal, that was our refrenc model and the springs model , the difference is 67.5%.

b) Increase of the base shear force

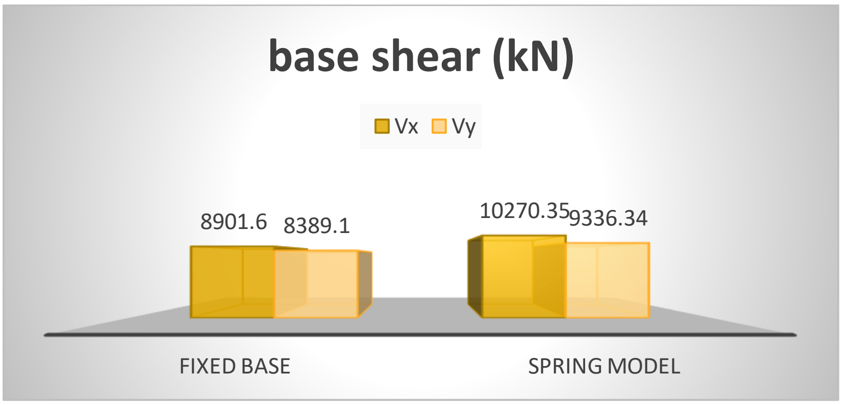


Figure 8. 11: Base shear comparison.

The seismic forces in both directions increased for the springs method compared to the model where the base was considered infinitely rigid.

The seismic force increased about 15% in the x-x direction and 12% in the y-y direction

c) **Increase in storey drift**

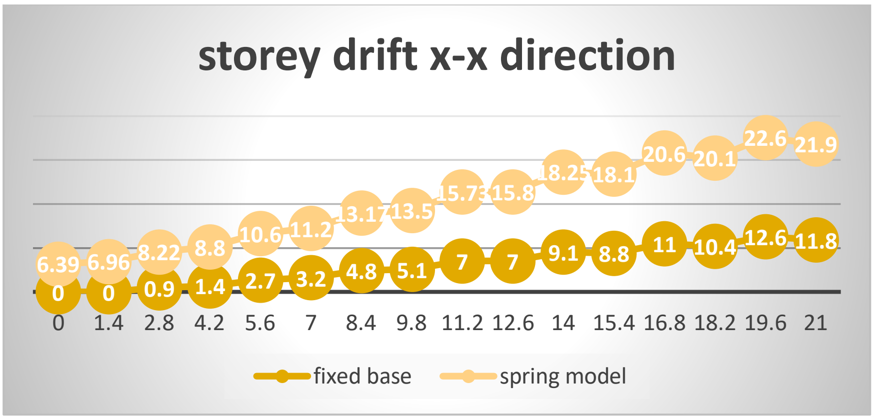


Figure 8. 12: storey drift comparison (x-x).

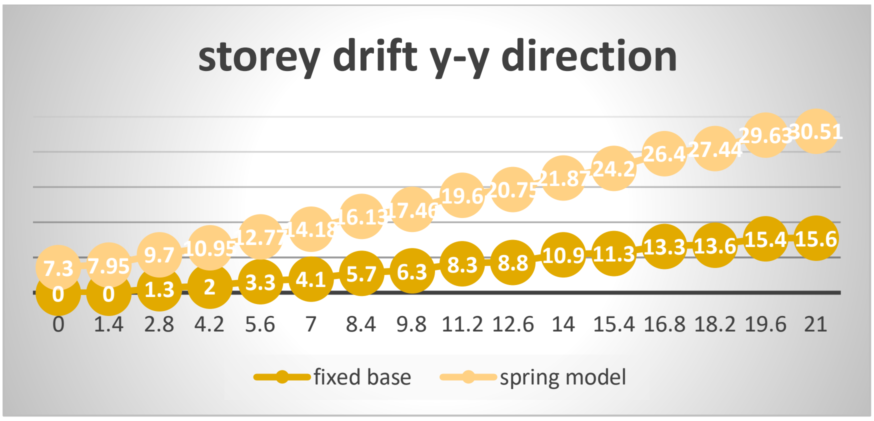


Figure 8. 13 :storey drift comparison (y-y).

We observed a noticeable increase of the storey drift at the top of the building 85% in the x-x direction and 95% in the y-y.

Although the inter-storey drift is still under the allowable limit in the x-x direction, it has exceeded the limit in the y-y direction.

d) **Increase in the internal efforts**

- **Central column**

- We observe that the axial effort taken up by the column has considerably decreased. we note a difference of 18% between the spring model and the reference model(figure 8.16).
- The bending moment increased with a negligible value.
- Shear force has increased with 305.4% for the spring model compared to the fixed base model. This value could influence the previous design of the structure (columns).

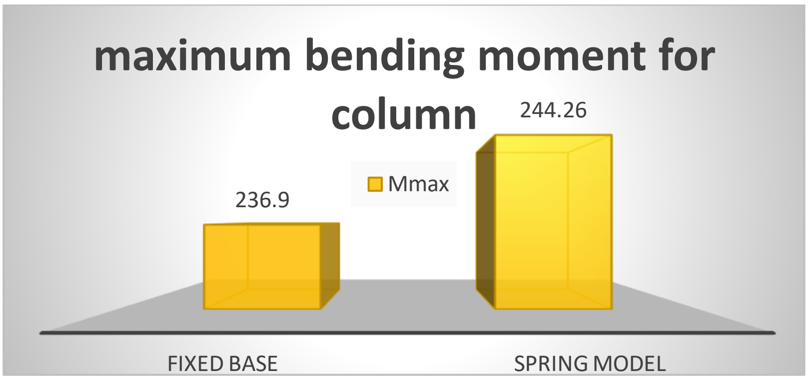


Figure 8. 14: Bending moment comparison for the column.

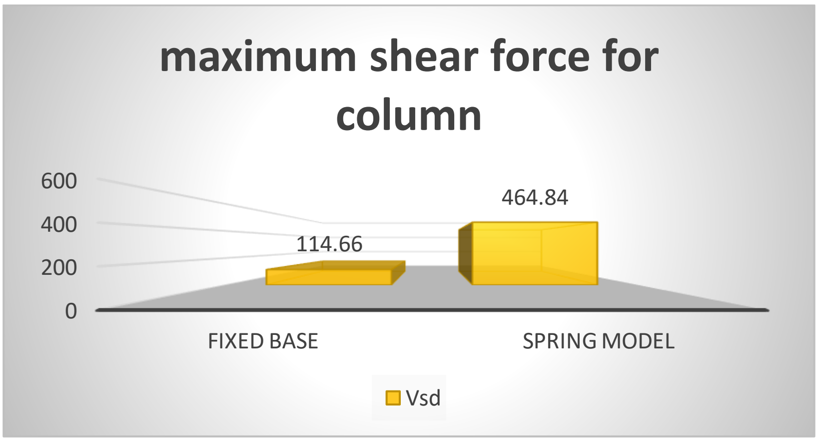


Figure 8. 15: Shear force comparison for the column.

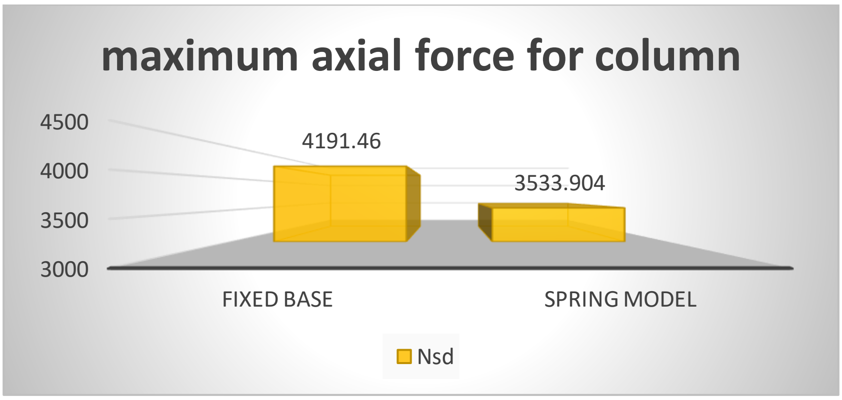


Figure 8. 16: Axial force comparison for the column.

- **Bracing and main beam**

For the bracing and the beam, we did not notice a considerable difference in the internal forces it is so minor that it can be ignored.

8.8. Conclusion

Reviewing the results, we note a significant difference between the fixed base model and the springs model. The more flexible the structure, the more significant the difference in results. Generally, there has been an increase in the forces and displacements in the SSI model caused by the increase in the seismic force because the mass of the structure is increased taking into account the mass of the 2 basements.

CHAPTER 08 : SOIL-STRUCTURE INTERACTION

After the seismic analysis, we conclude that the seismic response of the structure is influenced by taking into account the ISS, for which we have found that the more flexible the structure, the greater the displacements and the seismic forces. in addition , we can see a significant increase of ***the internal efforts, base shear force or seismic force, and period of the building.***

Moreover, I think the main deduction is that engineers should take into account the soil-structure interaction in the design of their structure, because although this phenomenon can have a positive effect on the structure, the hypothesis of the fixed base is not always accurate, and that is where we see the negative side of SSI.

GENERAL CONCLUSION.

GENERAL CONCLUSION

The purpose of our end of study project was to design and check a metal frame building of car parking use. As a future civil engineer, we had to think about the economical and resistance aspects. Based on the knowledge already acquired during our training cycle as a master 2 in civil engineering. We conclude the following:

A proper load assessment is necessary to encore that the building meets the safety regulations and that these loads do not exceed the bearing limit of the structural elements.

We have provided the general principles and procedures for determining the loads acting on the structure (permanent loads, operating loads and climatic loads). We concluded that the wind action was not preponderant comparing to the seismic action for the design of our structure and this due to the small height of our structure and the fact that it is located in a high seismicity zone,

Using imperial expressions to approach the section value, we adopted a preliminary size for the structural elements. However, checking the frame elements with these loads alone was not sufficient to make the final sizing that is why we performed a dynamic analysis wish was mainly a seismic analysis.

After subjecting the structure to earthquake loads in both direction X and Y we concluded that the preliminary sizing results of the columns were unsafe, inadequate and did not satisfy the rigidity condition of the building. We increased their size, added X shaped bracing in both directions of the building, and then verified the sections using the proper regulation (CCM97, EUROCODE04). We can say that a dynamic analysis is indispensable for the design of any building.

The joints design required specific and detailed verification of every component of the joint, including the connected parts. The connected parts were checked for added stress due to an eccentricity of a load or the interaction between elements.

For the foundation, the use of strip footings in this case is allowed because of the lightness of the structure. In this case, it is more economical, by avoiding having a raft, we saved in concrete.

Concerning chapter 8, the results obtained clearly showed that the soil-structure is mechanical phenomenon that should not be taking lightly due to its effect on the global dynamic response of the structure.

Considering that all civil engineering structures and construction are somehow linked to the ground, when designing a structure we should look at the overall SSFS(soil-structure-foundation system) because these three component work together in an exchange of stress, the behavior of one depends on the behavior of the other , with is depended on their mechanical characteristics.

Reviewing the results, we observed a significant difference between the fixed base model and the springs model in the forces and displacements. The more flexible the structure, the more significant the difference in results. Generally, there has been an increase in the SSI model caused by the increase in the seismic force because the mass of the structure is increased taking into account the mass of the 2 basements.

Even if we often consider the SSI as a positive effect, most cases it is not , therefore I strongly believe that the SSI methods should be introduced in the Algerian seismic regulation RPA, considering that Algeria is a country experiencing frequent and considerable seismic events.

REFRENCES

[**1**]**:** *ARCELORMITTAL Building & Construction Support*.

[**2**]**: (D.T.R-BC.2.2)** *« charges et surcharges d'exploitation »*.

[**3**]**:** Eurocode 01 *« actions on structure »*

[**4**]**:** RNV2013 « Wind and snow » Algerian regulation.

[**5**]**:** Mr SETHI (5th year).

[**6**]**:** Design and calculation rules for steel structures regulation CCM97. **(D.T.R-B.C.2.44)**

[**7**]**:** Eurocode 03: Calculation of steel structures, part 1-1, general rules and rules for buildings.

[**8**]**:** Eurocode 04: Design and verification of composite steel-concrete structures part 1-1, general rules and rules for buildings

[**9**]**:** Algerian seismic rules RPA99 version 2003; (Regulatory technical document D.T.R-B.C.2.48).

[**11**]**:**According to "*JOURNAL OF MATERIALS AND ENGINEERING STRUCTURES 4 (2017) 139–154Algerian Seismic Regulations RPA 99 - Version 2003 for Reinforced Concrete Building Structures: Interpretations and Proposals by Dr.Taleb Rafik" :*

[**12**] **:** "*Exemple : Assemblage de poteaux par couvre-joints by :access steel ; Eurocode made easy"*

[**13**]**:** *Metal frame course 1, Mr MENNADI. (4th year).*

[**14**]**:** BAEL99, D.T.U, Reinforced concrete to limit states 91, modified 99.**ANNEX F3**

[**15**]: *CALCUL DES SEMELLES DE FONDATIONS EN BÉTON ARMÉ(Cours et applications) CHERIFI FATIHA*

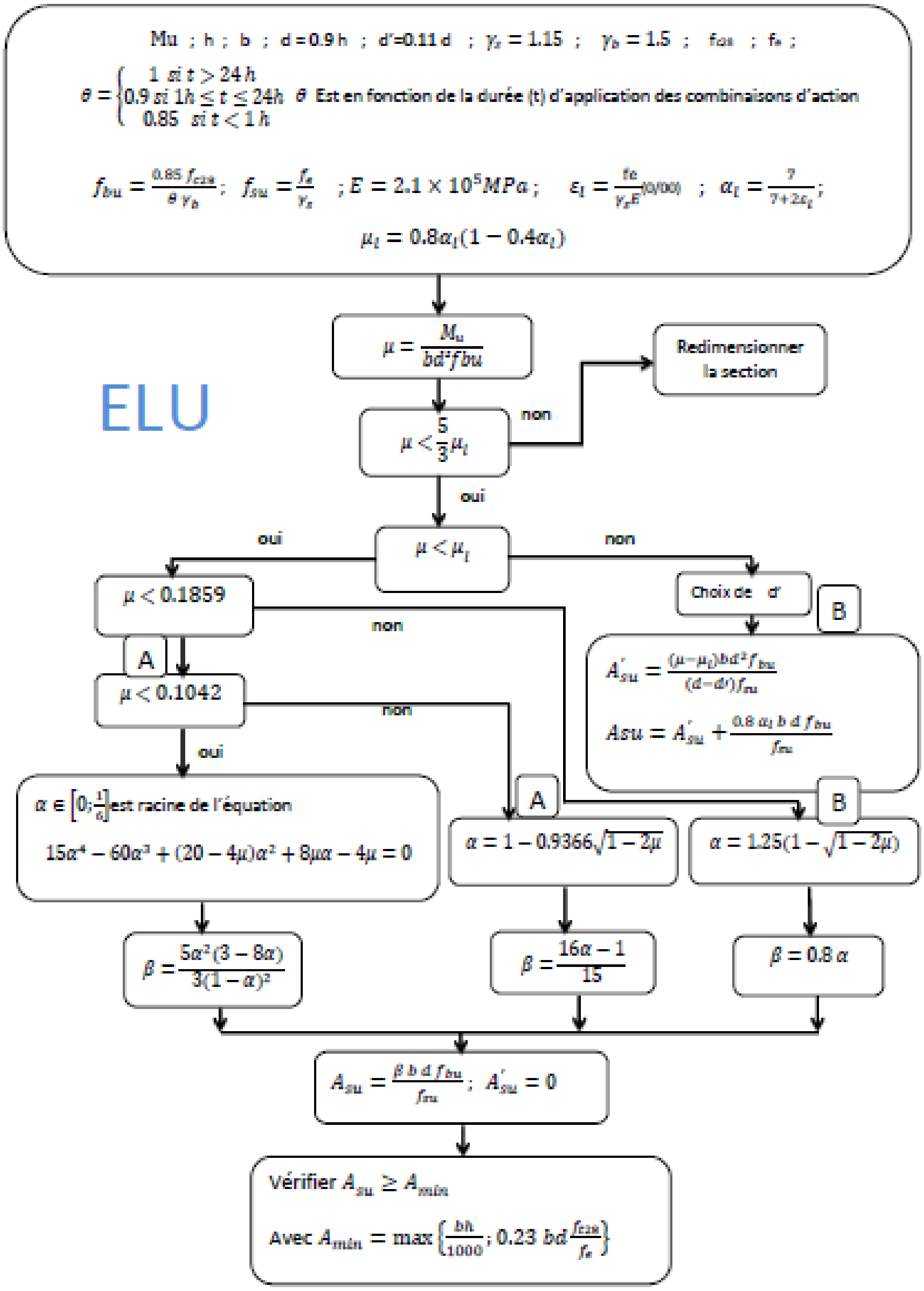
[**16**] : (FEMA, 2009).

[**17**] **:**(Wolf, 1985 ; Lysmer et al. 1999).

[**18**]**:** (NEHRP, 2012)

[**19**]**:** Newmark N.M., Rosenblueth E., (1971), Fundamentals of earthquake engineering. PrenticeHall, Englewood Cliffs, 1971.

ANNEX 1



Mu ; h ; b ; d = 0.9 h ; d'=0.11 d ; $\gamma_s = 1.15$; $\gamma_b = 1.5$; f_{c28} ; f_e ;
$\theta = \begin{cases} 1 \ si\ t > 24\,h \\ 0.9\ si\ 1h \leq t \leq 24h \\ 0.85\ \ si\ t < 1\,h \end{cases}$ θ Est en fonction de la durée (t) d'application des combinaisons d'action
$f_{bu} = \frac{0.85\, f_{c28}}{\theta\, \gamma_b}$; $f_{su} = \frac{f_e}{\gamma_s}$; $E = 2.1 \times 10^5 MPa$; $\varepsilon_l = \frac{fe}{\gamma_s E}$(0/00) ; $\alpha_l = \frac{7}{7+2\varepsilon_l}$;
$\mu_l = 0.8\alpha_l(1 - 0.4\alpha_l)$
ELU
$\mu = \frac{M_u}{bd^2 fbu}$
Redimensionner la section
non
$\mu < \frac{5}{3}\mu_l$
oui
$\mu < \mu_l$
oui
non
$\mu < 0.1859$
non
Choix de d'
B
A
$\mu < 0.1042$
non
$A'_{su} = \frac{(\mu - \mu_l) b d^2 f_{bu}}{(d - d') f_{su}}$
$Asu = A'_{su} + \frac{0.8\, \alpha_l\, b\, d\, f_{bu}}{f_{su}}$
oui
$\alpha \in \left[0; \frac{1}{6}\right]$ est racine de l'équation
$15\alpha^4 - 60\alpha^3 + (20 - 4\mu)\alpha^2 + 8\mu\alpha - 4\mu = 0$
A
$\alpha = 1 - 0.9366\sqrt{1 - 2\mu}$
B
$\alpha = 1.25(1 - \sqrt{1 - 2\mu})$
$\beta = \frac{5\alpha^2(3 - 8\alpha)}{3(1 - \alpha)^2}$
$\beta = \frac{16\alpha - 1}{15}$
$\beta = 0.8\,\alpha$
$A_{su} = \frac{\beta\, b\, d\, f_{bu}}{f_{su}}$; $A'_{su} = 0$
Vérifier $A_{su} \geq A_{min}$
Avec $A_{min} = \max\left\{\frac{bh}{1000}; 0.23\ bd\frac{f_{t28}}{f_e}\right\}$

ELS

M_{ser} ; h ; b ; d=0.9 h ; d'= 0.11 h ; $\overline{\sigma_{bc}} = 0.6\, f_{c28}$; As ; As'

$$\overline{\sigma_{st}} = \begin{cases} f_e & pour\ \ FPP \\ min\left\{\frac{2}{3} fe\ ,\ \ max(\frac{fe}{2}, 110\sqrt{1.6 f_{t28}})\right\} & \text{pour FP} \\ \min\left\{\frac{1}{2}\mathrm{fe}\ ,\ \ 90\sqrt{\eta f_{t28}}\right\} & pour\ \ FTP \end{cases}$$

y Solution positive de :

$$b\, y^2 + 30(A_s + A_s')y - 30(A_s\, d + A_s'\, d') = 0$$

$$I = \frac{1}{3} by^3 + 15As'(y-d')^2 + 15As(d-y)^2$$

$k = \frac{Mser}{I}$; $\sigma_{bc} = k\, y$; $\sigma_{st} = 15\, k(d-y)$; $\sigma_{sc} = 15K(y-d')$

$\sigma_{bc} \leq \overline{\sigma_{bc}}$

$\sigma_{st} \leq \overline{\sigma_{st}}$

oui

E.L.S vérifiée

non

Dimensionnement à l'E.L.S

« BIELLES METHOD »

méthode des bielles

semelle continue

$q_u = \sigma_{sol}$; $q_{ser} = \frac{2}{3} q_u$; p_u; p_{ser}; $\gamma_s = 1.15$

$f_{su} = \frac{f_e}{\gamma_s}$; a ; b ; $\rho = 25\ KN/m^3$

$$\frac{p}{q} = \sup\left(\frac{p_u}{q_u}, \frac{p_{ser}}{q_{ser}}\right)$$

$B \geq \frac{p}{q}$ *et* $A = 1$ m

$$d \geq \frac{B-d}{4}$$

$$h = d + 0.05\ (m)$$

ajouté $PP = \rho \times B \times 1 \times h$

$\Longrightarrow p_u$ *et* p_{ser}

$\Longrightarrow B \geq \frac{p}{q}$ *et* $A = 1$ m

$$A_s = \frac{p_u(B-b)}{8\ d\ f_{su}}$$

$$A_{réparation} = \frac{A_s}{4} B$$

semelle isolée

$q_u = \sigma_{sol}$; $q_{ser} = \frac{2}{3} q_u$; p_u ; p_{ser}; $\gamma_s = 1.15$

$f_{su} = \frac{f_e}{\gamma_s}$; a ; b ; $\rho = 25\ KN/m^3$

$$\frac{p}{q} = \sup\left(\frac{p_u}{q_u} ; \frac{p_{ser}}{q_{ser}}\right)$$

$AB \geq \frac{p}{q}$ *et* $\frac{A}{B} = \frac{a}{b} \Longrightarrow$ B $\geq \sqrt{\frac{p\,b}{q\,a}}$ *et* $A \geq \sqrt{\frac{p\,a}{q\,b}}$

$$\max\begin{pmatrix}\frac{A-a}{4}\\ \frac{B-b}{4}\end{pmatrix} \leq d_a ;\ d_b \leq \min\begin{pmatrix}A-a\\ B-b\end{pmatrix}$$

$$h = d + 0.05\ (m)$$

ajouté $PP = \rho \times B \times A \times h$

$\Longrightarrow p_u$ *et* p_{ser}

$\Longrightarrow$ *B et A*

$$A_b = \frac{p_u(B-b)}{8\ d\ f_{su}}$$

$$A_a = \frac{P_u\,(A-a)}{8\ d\ f_{su}}$$

$FPP \Longrightarrow A_s = A_s$ *calculée*

$FP \Longrightarrow A_s = 1.1\ A_s$ *calculée*

$FTP \Longrightarrow A_s = 1.5\ A_s$ *calculée*

$$\tau_{su} = 0.6\ \psi^2 f_{tj}\ ;\ Ls_{A,B} = \frac{\phi}{4}\frac{f_e}{\tau_{su}}$$

- $Ls_A \geq \frac{A}{4}$ *ou* $Ls_B \geq \frac{B}{4} \Longrightarrow$ *crochets*
- $\frac{A}{8} \leq Ls_A \leq \frac{A}{4}$ *ou* $\frac{B}{8} \leq Ls_B \leq \frac{B}{4} \Longrightarrow$ *pas de crochets*
- $0 \leq Ls_A \leq \frac{A}{8}$ *ou* $0 \leq Ls_B \leq \frac{B}{8} \Longrightarrow$ *pas de crochets mais :*