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Department of Civil Engineering and Hydraulics

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Graduation Project prepared for obtaining the Master's degree

Field: CIVIL ENGINEERING (Structural)

Seismic study of a reinforced concrete structure by Non-linear Temporal Analysis

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KOULOUGHLI AYA

CHERTIOUA RANYA

DEDICATION

I dedicate this modest work to:

To my very dear parents.

To my sister and my brothers.

To all my friends.

To the one who shared this work with me, my dear friend and partner "Ranya".

Aya.

DEDICATION

I dedicate this modest work to:

To my very dear parents.

To my dear nephew Adib

To my sister Nihad and my brother walla.

To my friends Zineb,Nourhane,Nahla,Fatima and Oumnia.

To the one who shared this work with me, my dear friend and partner "Aya".

Ranya

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Symbols	Notation
A_{ser}	Section of steels at SLS.
A_t	Section of a transversal reinforcement
A	Acceleration zone Coefficient
B	Area of a concrete section
B_r	Reduced section
b	Width of section
Cv, Ca	seismic coefficient
SLS	Serviceability limit state
ULS	Ultimate limit state
E	Modulus of longitudinal elasticity
Ei	Instantaneous modulus of elasticity
Es	Modulus of elasticity of steel
f_{c28}	Characteristic compressive strength (MPa)
f_{t28}	Characteristic tensile strength (MPa)
G	Permanent action
H	Height, the anchoring height of a foundation (m)
h_t	Total height of the floor
h_0	Thickness of the compression slab
h_e	Free floor height
I	Moment of inertia

Symbols list:

K_i	Elastic lateral stiffness of the building in the direction under consideration.
K_e	Effective lateral stiffness of the building in the direction under consideration
Q	Variable load
Q	Quality factor
q_u	Ultimate load
q_s	Service charge
L	Length or span
L_p	The length of plastic hinges.
Lmax	The greatest span between two successive bearing elements (m).
Lx	Distance between bare joists.
Ly	Distance between support axes of the main beams.
M_R	Moment of resistance
M	Moment in general
Ma	Moment on support
Mua	Ultimate moment of calculation.
Mser	Moment of service calculation
Mt	Moment in span
M0	Isostatic moment
Nser	Normal service effort
Nu	Ultimate Normal Effort
N	Normal force due to vertical loads

Symbols list:

n	the number of steps on the flight, Equivalence coefficient
P	Applied concentrated load (SLS or ULS).
PF1	The modal participation factor of the first mode of the structure
R	global behavior coefficient
S	Section, area
S_t	Reinforcement spacing.
T1, T2	Characteristic period, associated with the category of the site.
T_i	Elastic fundamental period (in seconds) in the direction under consideration calculated by elastic dynamic analysis
Te	Effective fundamental period of the building in the direction under consideration, in seconds
V	Shear force.
W	Dead weight of the structure.
WQi	Operating expenses
WGi	Weight due to permanent loads and those of any fixed equipment.
X,Y,Z	Coordinates in general.
y	Ordinate of the neutral fiber.
b 0	Gross thickness of the weapon of a section, width of the rib
d	Usable height.
e	Eccentricity, thickness
f	Arrow.
fbu	Concrete compressive stress at ULS.R
fe	Elastic limit.

Symbols list:

fcj	Characteristic resistance to compression at "j" days expressed in (MPa).
ftj	Characteristic tensile strength at "j" days expressed in (MPa).
$\mathbf{h_N}$	. Height measured in meters from the base of the structure to the top level.
$\boldsymbol{\sigma_{bc}}$	Concrete compressive stress.
$\boldsymbol{\sigma_s}$	Compressive stress in steel
$\boldsymbol{\upsilon}$	Poison coefficient
$\boldsymbol{\sigma}$	Normal stress.
$\boldsymbol{\gamma b}$	Safety factor.
$\boldsymbol{\gamma s}$	Safety factor.
$\boldsymbol{\varphi}$	Ground internal friction angle (degrees).
$\boldsymbol{\sigma_{adm}}$	Admissible stress at foundation level (bars).
$\boldsymbol{\tau}$	Limit shear value given by the BAEL (MPa)
$\boldsymbol{\tau_u}$	Shear stress (MPa).
	amplification factor of damping (damping correction factor)
$\boldsymbol{\eta}$	Weighting coefficient depending on the nature and duration of the operating load
$\boldsymbol{\mu_l}$	Reduced ultimate moment.
$\boldsymbol{\alpha_1}$	The modal mass coefficients
$\boldsymbol{\varphi_{1,roof}}$	Amplitude of the first mode of vibration at the top (Amplitude of the fundamental mode)
$\boldsymbol{\rho}$	Volume weight.
$\boldsymbol{\varepsilon_s}$	Relative elongation
h	Total height of a reinforced concrete section.

Symbols list:

$\boldsymbol{\lambda}$	Slenderness
$\boldsymbol{\alpha}$	Coefficient functions of the mechanical slenderness λ
ξ	Critical Damping Percentage
$\mathbf{C_T}$	Coefficient depending on the bracing system and the type of infill
$\boldsymbol{B_c}$	Area of the cross of the considered column
$\boldsymbol{N_d}$	Calculation compressive normal force under accidental combinations
$\boldsymbol{\delta_{ek}}$	Displacement due to the seismic forces Fi (including torsion effect)
$\boldsymbol{\delta_k}$	Horizontal displacement at each level « K » of the structure
$\boldsymbol{b_i}$	Centre of gravity of the structure at each level
$\emptyset$	Reinforcement diameter
$\boldsymbol{L_r}$	Overlap length
$\mathbf{l_f}$	Buckling length
$\mathbf{M_G}$	bending moment developed by permanent loads
$\mathbf{M_Q}$	bending moment developed by live loads
***C*1**	Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response.
***C*2**	Modification factor to represent the effect of pinched hysteresis shape, cyclic stiffness degradation, and strength deterioration on the maximum displacement response
D	Dynamic amplification factor, dimension of the building measured at its base in the direction of calculation considered.

Abstract

ملخص:

هذا المشروع هو عبارة عن دراسة مفصلة لبناية مكونة من طابق ارضي زائد اربع طوابق علوية. اشتملت الدراسة على جزئين رئيسين:

الجزء الاول: مفاهيم عامة حول التحليل الخطي و اللاخطي للبنايات و التصميم القائم على الاداء، تفصيل عام للمشروع يشتمل على تعريف بالبناية مع إعطاء الابعاد الاولية للعناصر و حمولة كل العناصر المكونة لها.

إشتمل هذا الجزء أيضا إجراء دراسة ديناميكية خطية للمبنى بواسطة برنامج ETABS ، الذي يزودنا بالنتائج النهائية التي تسمح بتسليح مختلف العناصر المقاومة المكونة للبناية مع الاخذ بعين الاعتبار كل توصيات القوانين الجزائرية في مجال البناء و اللوائح الزلزالية.

الجزء الثاني: تم تخصيص هذا الجزء للدراسة اللاخطية باستعمال الطريقتين الستاتيكية اللاخطية و الديناميكية اللاخطية

بالنسبة للطريقة الستاتسكية اللاخطية فقد تمت يدويا برسم "منحنى القدرة"الذي يقارن ب "منحنى الطلب الزلزالي " لتحديد "نقطة الجودة" للمنشأ تعتبر هذه الطريقة دقيقة نسبيا، اما الطريقة الديناميكية اللاخطية فهي افضل و ادق طريقة للتحليل (تحليل خطوة ب خطوة)، في هذا المشروع اخترنا برنامج Seismostruct و هو برنامج مخصص للتحليل اللاخطي.

هذا التحليل يمثل أساسا للتصميم المضاد للزلازل المعروف ب": التصميم القائم على الاداء "، بهدف الاقتراب قدر الامكان من السلوك الحقيقي للبناية.

الكلمات المفتاحية: البناية، الخرسانة المسلحة، تحليل ديناميكي خطي، تحليل ديناميكي لاخطي، **طريقة ستاتيكية خطية،** طريقية ديناميكية لاخطية ، التصميم القائم على الأداء

Abstract:

This project is a detailed study of a building consisting of a ground floor plus four upper floors. The study included two main parts:

The first part: general concepts about linear and nonlinear analysis of buildings and performance-based design. A general description of the project includes a definition of the building, giving the initial dimensions of the elements and the load of all its structural elements.

This part also included conducting a linear dynamic analysis of the building using the ETABS program, which provides us with the final results that allow the reinforcement of the various structural elements that make up the building, considering all the recommendations of Algerian rules in the field of construction and seismic regulations.

The second part: This part was devoted to the study of nonlinearity using the nonlinear static and nonlinear dynamic methods.

As for the nonlinear static method, it was done manually by drawing the "capacity curve" which is compared to the "seismic demand curve" to determine the "performance point" of the structure, in this project we chose the Seismostruct program, which is a program dedicated to nonlinear analysis.

This analysis forms the basis for the anti-seismic design known as “performance-based design”, with the aim of getting as close as possible to the real behavior of the building.

Keywords: structure; Reinforced concrete, linear dynamic analysis, linear static method, nonlinear dynamic method, performance-based design

Résumé :

Ce projet est une étude détaillée d'un bâtiment composé d'un rez-de-chaussée et de quatre étages supérieurs. L'étude comportait deux parties principales :

La première partie : concepts généraux sur l'analyse linéaire et non linéaire des bâtiments et la conception basée sur la performance. Une description générale du projet comprend une définition du bâtiment, donnant les dimensions initiales des éléments et la charge de tous ses éléments structuraux.

Cette partie comprenait également la réalisation d'une analyse dynamique linéaire du bâtiment à l'aide du programme ETABS, qui nous fournit les résultats finaux qui permettent le renforcement des différents éléments structurels qui composent le bâtiment, en tenant compte de toutes les recommandations des règles algériennes dans le domaine de la construction et de la réglementation parasismique.

La deuxième partie : Cette partie a été consacrée à l'étude du non linéarité en utilisant les méthodes statiques non linéaire et dynamique non linéaire.

Quant à la méthode statique non linéaire, elle a été réalisée manuellement en traçant la "courbe de capacité" qui est comparée à la "courbe de demande sismique" pour déterminer le "point de performance" de l'ouvrage, dans ce projet nous avons choisi le programme Seismostruct, qui est un programme dédié à l'analyse non linéaire.

Cette analyse est à la base de la conception antisismique dite « conception basée sur la performance », dans le but de se rapprocher le plus possible du comportement réel du bâtiment

Mots clés : structure ; Béton armé, analyse dynamique linéaire, méthode statique linéaire, méthode dynamique non linéaire, conception basée sur la performance.

General introduction

General introduction:

Civil engineers use different methods to build safe constructions; these methods have known various changes through the years. The most known and used method is the response spectrum which is an elastic linear analysis, though it’s an effective method it does not represent the real behavior of the structure. The nonlinear behavior is favorable in the seismic case, because it allows the structures to undergo inelastic displacements with limited damage and without collapse, no loss of stability, for this the structure must have a ductility and an adequate energy dissipation capacity. The most tow methods used in nonlinear analysis are:

- The static pushover analysis which consist of drawing the "capacity curve" and compare it to the "seismic demand curve" to determine the "performance point" of the structure. Push-over analysis is also relatively simple and computationally efficient, which makes it a useful tool for evaluating the seismic performance of large, complex structures. However, it should be noted that pushover analysis is a simplified method and should be used in conjunction with more advanced analysis methods such as dynamic analysis, to get a more accurate representation of the structure behavior under seismic loads.
- The dynamic nonlinear time history analysis, where one holds account of the nonlinearity of the structural elements.

In this type of analysis, the seismic action is represented by accelerograms, and the calculation is carried out by direct integration of the equations of the motion, using a numerical integration technique stepping over time. This method is more accurate than push-over analysis as it considers the nonlinear behavior of structures and soil-structure interaction. However, it is computationally more demanding, and it requires a detailed model of the structure and the soil.

In this work we will make a complete civil engineering study of design and verification of a building (ground +4 floors) located in the city of "Boumerdes" zone (III), high seismicity zone according to the latest Algerian Seismic code RPA 99 versions 2003.

This project is organized into five chapters, this introduction and a general conclusion. The **first chapter** is a description of the nonlinear dynamic behavior of the reinforced concrete structures, and we are going to speak about performance-based design, the performance levels and the methods used. **The second chapter** is devoted to the formulation of the dynamic equations of the structures under seismic solicitation as well as their methods of resolution, for the linear systems, and for the nonlinear systems.

The third chapter is devoted to the presentation of the structure, pre-dimensioning and load distribution. **The fourth chapter** is a linear elastic study, in order to dimension a building and to reinforce its structural elements following the Algerian seismic code (RPA 99 version 2003) and the limit state method BAEL 91.

In **last chapter**, both nonlinear time history analysis and pushover analysis were conducted to the structure, following the ATC-40 and ASCE-41.

Chapter I:

Nonlinear behavior and performance-based design

I. Non linear behavior:

I.1. Introduction:

While buildings are usually designed for seismic resistance using elastic analysis, most will experience significant inelastic deformations under large earthquakes. Modern performance-based design methods require ways to determine the realistic behavior of structures under such conditions. Enabled by advancements in computing technologies and available test data, nonlinear analyses provide the means for calculating structural response beyond the elastic range, including strength and stiffness deterioration associated with inelastic material behavior and large displacements. As such, nonlinear analysis can play an important role in the design of new and existing buildings.

Nonlinear analyses involve significantly more effort to perform and should be approached with specific objectives in mind. Typical instances where nonlinear analysis is applied in structural earthquake engineering practice are to:

1) Assess and design seismic retrofit solutions for existing buildings;
2) Design new buildings that employ structural materials, systems, or other features that do not conform to current building code requirements;
3) Assess the performance of buildings for specific owner/stakeholder requirements. **[1]**

Nonlinear analysis can be more complex and computationally intensive than linear analysis, but it can provide a more accurate prediction of a building's response to extreme loading events. Additionally, understanding and accounting for nonlinear behavior can help to ensure the safety and performance of a building during such events.

The purpose of this chapter is to briefly describe the nonlinear dynamic behavior of structures. To this end, we will present some of the most used models of behavior, and define certain characteristics and phenomena distinguishing nonlinear behavior from structures. This description will be preceded by the presentation of linear behavior.

I.2 Linear behavior:

To successfully conduct a nonlinear analysis of a structure, it is always lucid to precede it by a linear analysis and then introduce the sources of nonlinearity one by one.

In addition, the linear model is often used, which is the basis for the development of spectra regulatory. So, before starting the description of nonlinear behavior, we will recall what a linear behavior is by presenting the linear elastic model. **[2]**

Elastic strains are defined as strains that are proportional to the force which causes them and which disappear after the suppression of these forces in this case the deformation is said to be reversible.

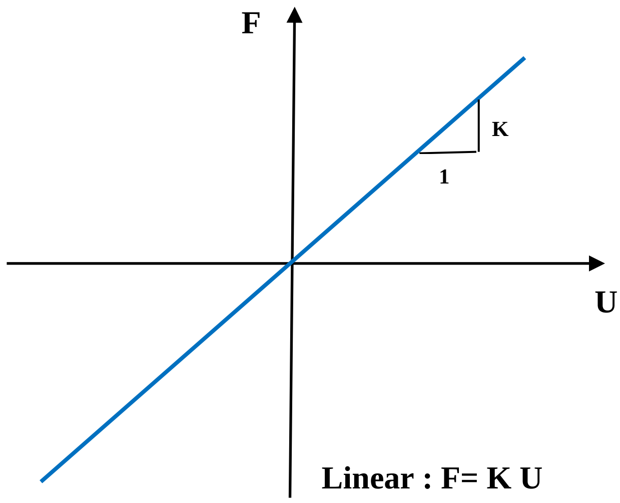


Figure I.1: linear elastic

I.3 Nonlinear behavior:

In the context of buildings, nonlinear behavior refers to the fact that the structural response of a building to external loads (such as wind or earthquakes) is not proportional to the magnitude of those loads. This means that the building's response may be significantly different at different levels of loading. To perform a nonlinear temporal analysis, one must first have a behavior model that translates the force-displacement relationship of the structural element considered according to the loading history. Such a model is named hysteretic model. [3]
There are mainly two types of non-linearity:

a. Nonlinear geometric behavior:

Nonlinear behavior occurs due to the geometry of a structure, such as the effects of large deflections or rotations on the behavior of a structure. These effects can occur when a structure is subjected to loads that cause significant deformations, such as in the case of cantilevers, arches, and other structures that are designed to deflect or rotate under load.

The nonlinear geometric behavior of a structure can has a significant impact on its overall performance and stability. For example, large deflections can cause a structure to become unstable, while large rotations can cause a structure to become rigid and less able to withstand loads. Examples of nonlinear geometric behavior in buildings include:

- **Large deflections:**

This occurs when a structure is subjected to loads that cause significant deformations, such as in the case of cantilevers, arches, and other structures that are designed to deflect under load. This can also occur in structures that are not designed to handle such large deformations, such as in the case of a beam that is too slender.

- **Nonlinear buckling:**

This occurs when a structure is subjected to compressive loads that cause it to buckle in a nonlinear manner. This can happen in columns or other slender structural elements that are not designed to handle compressive loads.

- **Large rotations:**

This occurs when a structure is designed to rotate or deflect under load, such as in the case of arches, domes, and other structures that are designed to rotate or deflect under load. It is important to consider these nonlinear geometric effects in the design process in order to ensure the safety and stability of a structure. Performance-based design approach allows for the consideration of these nonlinear effects in the design process and can lead to more efficient and cost-effective solutions **[3].**

b. Material nonlinearity:

It can come from the intrinsic law of behavior of the material (law behavior elastic-plastic for example) the cracking of the material, the behavior of the tensile concrete between two cracks, the mode of assembly between the columns and the beams of structures.

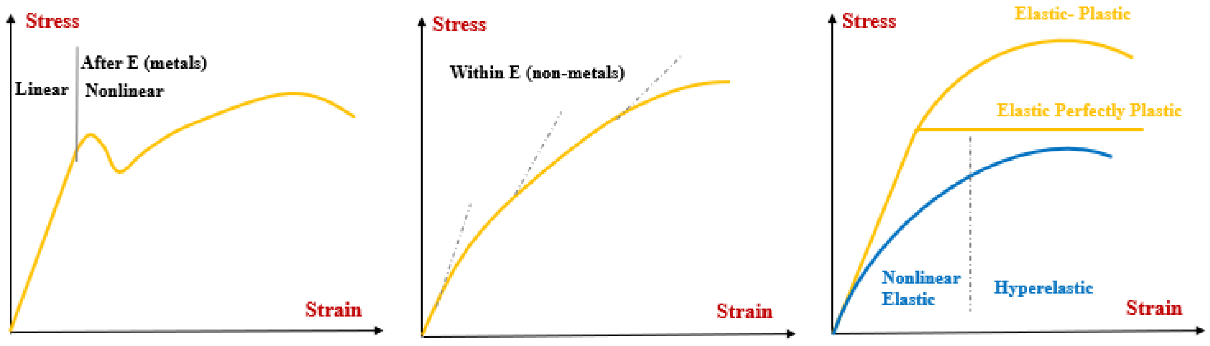


Figure I.2: Material-Nonlinearity

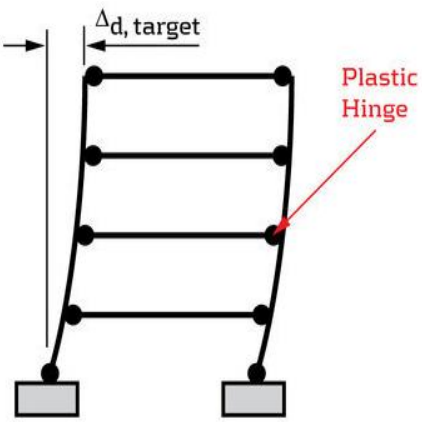
Figure 2 shows the stress strain curve for metal, non-metals & nonlinearity features. In case of metals, material moves to plastic zone after elastic limit, while non-metals have nonlinear stress strain curve (plastic, asbestos, fibres, etc.) from origin itself thus in both the cases curve shows the source of nonlinearity. Nonlinear curve helps to input exact stress vs strain after the yield point which gives the exact results. Furthermore, creep also considered as nonlinear phenomenon, at maximum temperature, even for small magnitude loads, kept applied for longer period of time, would cause failure.

Few Major nonlinear material (behavior) classifications:

1) Nonlinear elastic
2) Hyperplastic (example gasket (rubber) material)
3) Linear elastic perfectly plastic
4) Elastic perfectly plastic
5) Elastic time dependent plastic (creep)
6) Strain & temperature dependent elasticity and plasticity.

I.4. Plastic hinges:

A plastic hinge is a term used in structural engineering to describe a location in a structural member (such as a beam or column) where significant inelastic deformation occurs during a catastrophic event such as an earthquake or high wind. This deformation allows the structure to dissipate energy and can help prevent collapse. Plastic hinges are typically designed to form at specific locations in the structure and are an important aspect of seismic and wind design. They are usually located at the ends of structural members where the member is most likely to bend, such as at the base of a column or at the midspan of a beam. **[4]**



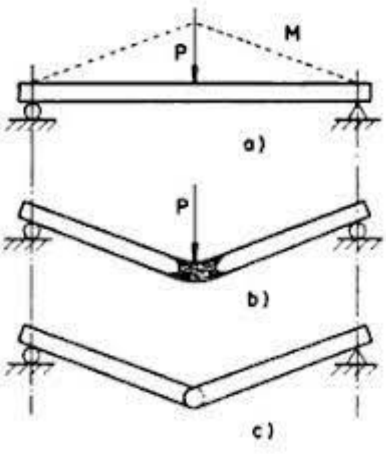


Figure I.3: Plastic hinge

It's worth mentioning that the plastic hinges are designed to form at specific locations, and the strength and ductility of the structure is dependent on the design of the plastic hinges and their locations. The engineer should take into consideration various factors such as the seismic hazard and the intended use of the structure when determining the location and design of the plastic hinges.

In reinforced concrete structures, a plastic hinge is typically created by designing the member with a reduced amount of reinforcement at the location where the hinge is intended to form. This allows the concrete to crush and the steel reinforcement to yield, creating a hinge that can rotate and absorb energy.

In steel structures, a plastic hinge is typically created by designing the member with a reduced amount of material at the location where the hinge is intended to form. This allows the steel to yield and absorb energy at the hinge location.

I.4.1. Position of plastic hinges:

In reinforced concrete structures, plastic hinges are typically located at the ends of structural members where the member is most likely to bend, such as at the base of a column or at the mid-span of a beam.

For columns, plastic hinges are usually located at the base where the column is most likely to experience bending moments due to gravity loads and seismic forces. The base of a column is also the location where the column is most likely to experience large inelastic deformations.

For beams, plastic hinges are usually located at the midspan where the beam is most likely to experience the largest bending moments due to gravity loads and seismic forces.

The mid-span of a beam is also the location where the beam is most likely to experience large inelastic deformations.

It's worth noting that the exact location of the plastic hinge depends on the specific design and loading conditions of the structure. The engineer should take into consideration various factors such as the seismic hazard and the intended use of the structure when determining the location and design of the plastic hinges.

I.4.2 Formation of hinges:

We investigate collapse of a simply supported beam under central point load. [4]

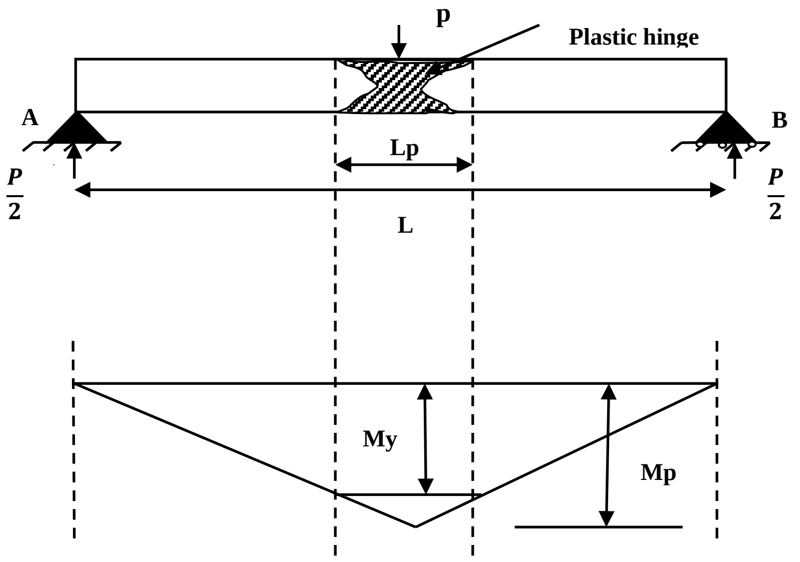


Figure I.4: Plastic hinges in simply supportive beam.

At mid-span;

Mmax = $\frac{P\,L}{4}$

Load P is increased until the bending at mind-span cross section reaches the fully plastic moment $\frac{P\,L}{4} = M_p$

- A plastic hinge formed at mid-section.
- Collapse will occur at further load increase

The value of load P given by;

$$P = \frac{4\,M_p}{L} \tag{I.1}$$

P = collapse load for beam.

L_p = The length of plastic hinges.

The load at which yield first occurs is given by:

$$M_y = \frac{p}{2}\left(\frac{l-l_p}{2}\right) \tag{I.2}$$

With substituting: $P = \frac{4M_p}{L}$

$$\gg L_p = L\left(1 - \frac{M_y}{M_p}\right) \tag{I.3}$$

I.5 Performance based seismic design:

I.5.1 Introduction:

Design for seismic resistance has been undergoing a critical reappraisal in recent years, with the emphasis changing from "strength" to "performance". For most of the past 70 years - the period over which specific design calculations for seismic resistance have been required by codes - strength and performance have been considered to be synonymous. However, over the past 25 years there has been a gradual shift from this position with the realization that increasing strength may not enhance safety, nor necessarily reduce damage. The development of capacity design principles in New Zealand in the 1970's was an expression of the realization that the distribution of strength through a building was more important than the absolute value of the design base shear. It was recognized that a frame building would perform better under seismic attack if it could be assured that plastic hinges would occur in beams rather than in columns (weak beam/strong column mechanism), and if the shear strength of members exceeded the shear corresponding to flexural strength. This can be identified as the true start to performance based seismic design, where the overall performance of the building is controlled as a function of the design process.

As an understanding developed in the 1960s and 1970s of the importance of inelastic structural response to large earthquakes, the research community became increasingly involved in attempts to quantify the inelastic deformation capacity of structural components.

Generally, this was expressed in terms of displacement ductility capacity, which was chosen as a useful indicator because of its apparent relationship to the force-reduction factor R, commonly used to reduce expected elastic levels of base shear strength to acceptable design levels. **[5]**

So, we will discuss the use of performance-based seismic design, approval of designs, and an overview of the performance-based design process. But first we’re going to speak briefly about the methods used before PBSD.

I.5.2 Force based design:

In force-based design method, forces are calculated corresponding to elastic response to a design acceleration response spectrum. In this process, elastic stiffness is used. These elastic forces are then divided by a response reduction factor (R) representing the assessed displacement ductility capacity. The structure is then designed for these reduced forces, and the displacement is checked with code specified limits, if the displacement does not meet the requirement the process is repeated. [6]

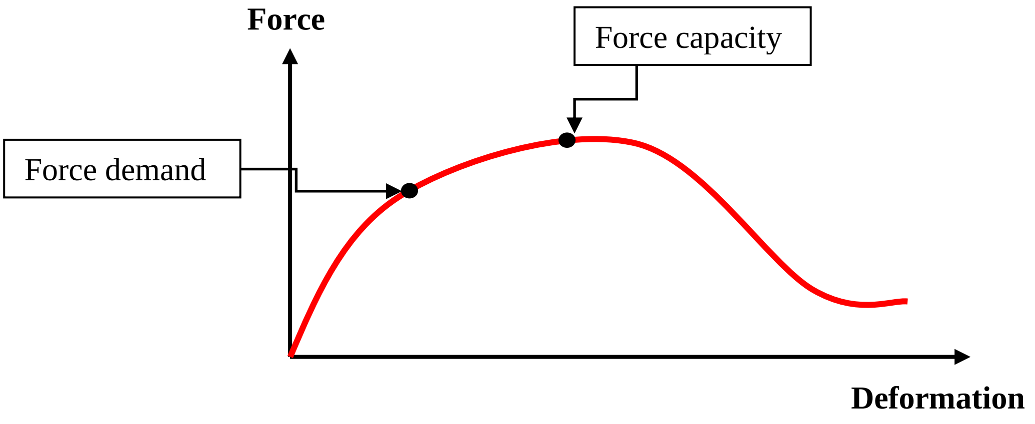


Figure I.5: capacity-demand curve.

- **Force capacity:** is a representation of the structure's ability to resist the seismic demand.
- **Force demand:** is a representation of the earthquake ground motion.

I.5.3 Displacement based design:

This approach uses the displacement response spectrum as basis for calculating the base shear force. It also depends on studying the building considering its inelastic phase. It is considered as one of the simplest design approaches for analysis of the multi-degree of freedom structures. In this method, the structure is characterized by the secant stiffness and equivalent damping of an equivalent single-degree of freedom structures. This design is bases on achieving specified displacement limit state defined either by material strain limits, or non-structural drift limits obtained from design codes under the design level seismic intensity. [7]

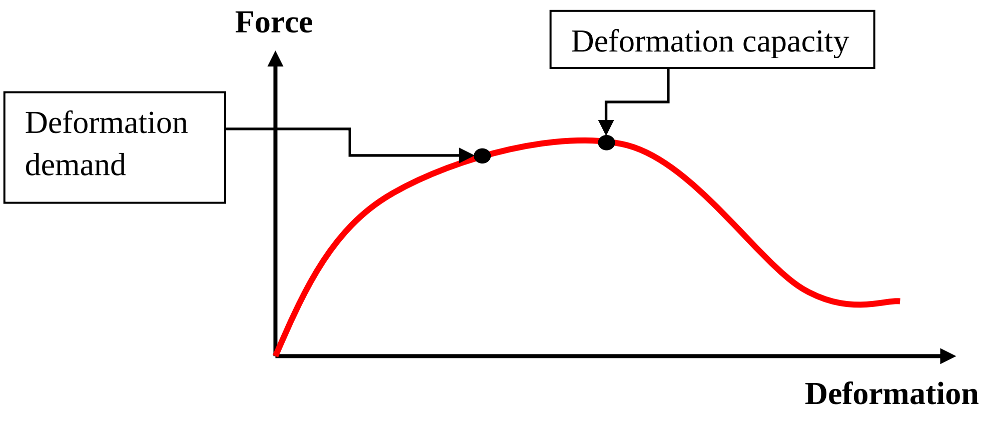


Figure I.6: deformation capacity – deformation demand curve

I.5.4 Capacity based design:

Capacity based design (CBD) is a design method used in structural engineering that focuses on the strength and behavior of a structure under various loading conditions. The goal of CBD is to design a structure that has sufficient strength and ductility to resist the loads it will experience during its service life, while also controlling the amount of damage that the structure will sustain during extreme events such as earthquakes and windstorms.

The CBD method is based on the principle that the structure's capacity to resist loads must be greater than or equal to the demand imposed by the loads. The structure's capacity is determined by the strength and ductility of the materials used to construct the structure and the design of the structural elements such as beams, columns and connections. The demand is determined by the loads the structure will experience during its service life, including gravity loads, seismic loads, and wind loads. CBD is commonly used in seismic and wind design, as well as in the design of other critical structures such as bridges and tall buildings. This method is considered to be more realistic, reliable, and efficient than the traditional design methods, as it accounts for the potential inelastic behavior of the structure and its ability to dissipate energy through deformation.

I.5.5 Performance based design:

I.5.5.1 Definition:

Performance based design (PBD) is a design method used in structural engineering that focuses on the overall performance of a structure under various loading conditions, rather than just the strength and behavior of individual structural components. The goal of PBD is to design a structure that can perform its intended function even under extreme loading conditions such as earthquakes and windstorms.

PBD is based on the principle that the structure's ability to perform its intended function under extreme loading conditions is more important than the strength and behavior of individual structural components. The performance of the structure is evaluated using performance criteria such as structural integrity, serviceability, and functionality. The designer will use analysis tools such as non-linear static analysis and non-linear dynamic analysis to evaluate the behavior of the structure under different loading conditions. The designer will also use testing methods such as shake table tests and large-scale testing to validate the behavior of the structure.

PBD is commonly used in seismic and wind design, as well as in the design of other critical structures such as bridges and tall buildings. It is a more comprehensive approach than traditional design methods, as it considers the overall performance of the structure and its ability to continue to function after an extreme event. It also allows for more flexibility in the design of the structure and can lead to more efficient and cost-effective solutions. **[8]**

I.5.5.2 Building performance levels:

According to American codes FEMA-356, FEMA-273, ATC-40 the first step to do a performance-based design is to choose the performance level, generally chosen by the owner. The three commonly used performance levels are:

a) **Immediate occupancy (IO):** Immediate occupancy refers to the ability of a building to be occupied immediately after an event, with minimal or no damage. This means that the building should be able to withstand the event without significant damage to the structural or non-structural elements, and should be safe for occupants to return to the building as soon as possible. In order to achieve immediate occupancy, the building must be designed to resist the loads imposed by the event and to remain functional.

b) **Life safety (LS):** Life safety refers to the ability of a building to protect the lives of its occupants during an event. This means that the building should be able to withstand the event without collapsing or causing injury to the occupants. In order to achieve life safety, the building must be designed to resist the loads imposed by the event and to remain stable. The building should be able to withstand the event without causing injury to the occupants, and the occupants should be able to safely evacuate the building.

c) **Collapse prevention (CP):** Collapse prevention refers to the ability of a building to prevent collapse during an event. This means that the building should be able to withstand the event without collapsing or becoming unstable. In order to achieve collapse prevention, the building must be designed to resist the loads imposed by the event and to remain stable. The building should be able to withstand the event without collapsing, and should remain stable and functional after the event.

It's worth noting that, the specific requirements for each of these performance levels will depend on the location and the intended use of the building, as well as the level of hazard. Buildings that are located in areas with high seismic or wind hazard will typically be required to have higher performance levels than buildings located in areas with low hazard.

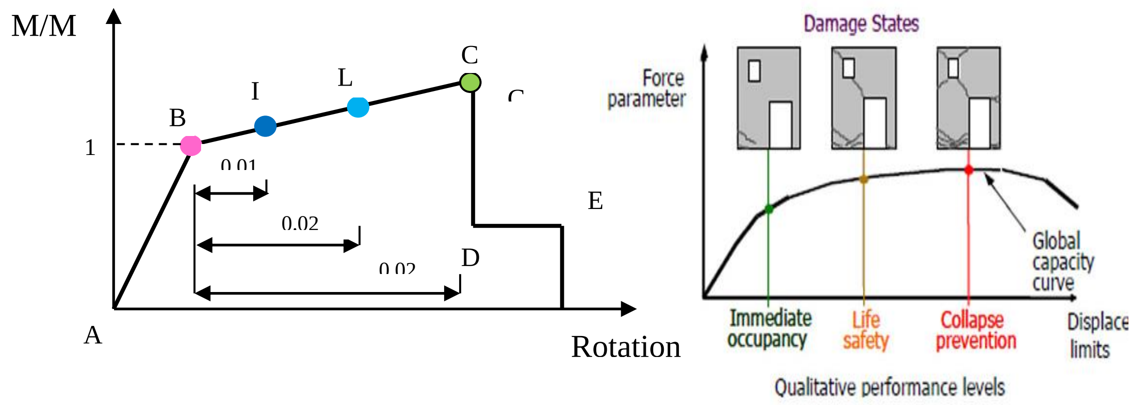


Figure I.7: performance levels.

Design earthquake	**Moderate earthquake**	**Major earthquake**	**Severe earthquake**
Damage	No damage	Repairable damage	No collapse
Column deformation	Elastic	rotations<0.01radians	rotations<0.02radians
Beam deformation	Elastic	rotations<0.02radians	rotations<0.025radians
Allow drift index	0.005	0.010	0.02
Performance level	Immediate occupancy	Life safety	Collapse prevention

Table 1.1.: seismic criteria

In summary, immediate occupancy, life safety, and collapse prevention are three important performance levels that are used in building codes and standards to describe the expected performance of a building during and after an extreme event. Immediate occupancy refers to the ability of a building to be occupied immediately after an event, with minimal or no damage. Life safety refers to the ability of a building to protect the lives of its occupants during an event, and collapse prevention refers to the ability of a building to prevent collapse during an event. The specific requirements for each of these performance levels will depend on the location and the intended use of the building, as well as the level of hazard.

I.5.6 Use of Performance-Based Seismic Design:

"Performance-based seismic design is a method that can be applied to both new and existing structures, with the aim of assessing and enhancing their seismic performance.

This approach can be used to:

1) Evaluate the probable seismic performance of a specific building,
2) design new structures that meet specific performance requirements,
3) Retrofit existing structures to improve their seismic resilience."

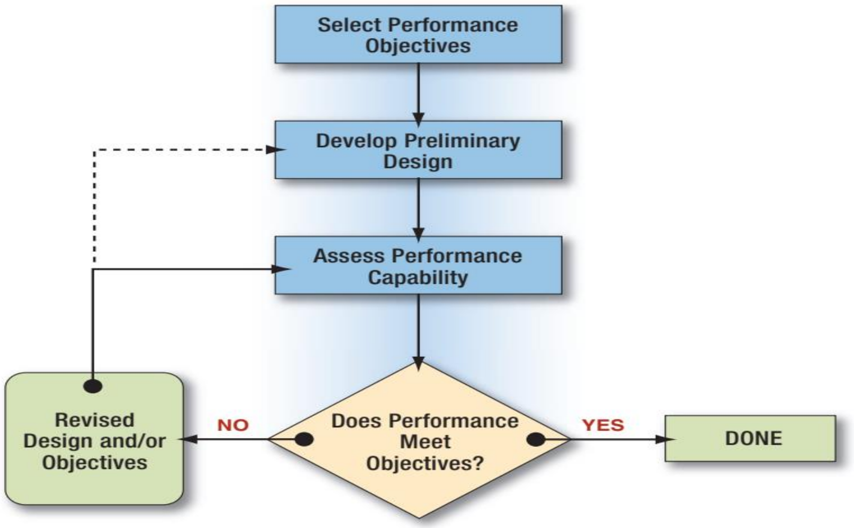


Figure I.8: Performance-based design flow diagram [9]

I.5.7 Performance estimation:

I.5.7.1. Non-linear static analysis:

Non-linear static analysis (also known as pushover analysis) is a method used to evaluate the seismic performance of a building by applying incremental loads to the structure until it reaches its maximum capacity.

Both the **ATC40** [10] and **FEMA356** [11] documents present similar performance-based engineering methods that rely on nonlinear static analysis (pushover) procedures for prediction of structural demands. They differ in the technique used to calculate the global inelastic displacement demand (performance point, target displacement) for a given ground motion.

- **The capacity-spectrum method (ATC40) [10] :**

Displacement demand (performance point) is determined from the intersection of a capacity curve, derived from the pushover curve, with a demand curve that consists of the smoothed response spectrum representing the design ground motion, modified to account for hysteretic damping effects.

- **The coefficient method (FEMA356) [11]:**

The coefficient method is based on guidelines provided by the Federal Emergency Management Agency (FEMA) in its publication "NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures" (FEMA 356) . The method uses a set of coefficients, such as importance factor, occupancy category, and seismic design category, to determine the seismic forces that a building must be able to withstand.

The importance factor is used to adjust the seismic forces based on the importance of the building. For example, a hospital would have a higher importance factor than a single-family residence. Occupancy category is used to adjust the seismic forces based on the type of occupancy of the building. For example, an assembly occupancy (such as a theater) would have different seismic forces than a residential occupancy.

Seismic design category is used to adjust the seismic forces based on the level of seismicity in the area where the building is located. For example, a building in an area with a high level of seismicity would have to be designed to withstand greater seismic forces than a building in an area with a low level of seismicity.

The coefficient method is considered as a simplified method for design of building structures, as it does not require detailed analysis of the building's structural system and instead relies on coefficients to estimate seismic forces. However, it is not applicable for buildings with irregular shapes or structural systems, or for buildings that are located in regions with extreme seismicity.

I.5.7.2. Non linear dynamic analysis:

Nonlinear dynamic analysis is a method used to evaluate the behavior of a structure, such as a building or bridge, under dynamic loads, such as those caused by earthquakes, winds, or explosions. It is a more advanced method than the linear dynamic analysis and the pushover analysis.

The analysis involves simulating the effects of a dynamic load on the structure using advanced mathematical models and computer software. These models consider nonlinear behavior of the structure, such as material yielding, geometric nonlinearity, and nonlinear boundary conditions. The nonlinear dynamic analysis also captures the time-dependent behavior of the structure, unlike pushover analysis.

One of the key advantages of nonlinear dynamic analysis is that it can accurately predict the behavior of a structure under realistic loading conditions. This allows engineers to identify potential failure points and design mitigation measures to increase the structure's resilience. Additionally, nonlinear dynamic analysis can also be used to evaluate the performance of existing structures and identify areas that may need to be strengthened or retrofitted to improve their performance. It is important to note that nonlinear dynamic analysis is a more accurate and reliable method than pushover analysis, but it is also more computationally intensive and requires more advanced computer software and specialized expertise.

- **Time history analysis:**

Time history analysis is a dynamic analysis that involves simulating the actual ground motion that a structure would experience during an earthquake. The load is applied as a time-varying function, and the response of the structure is evaluated at each point in time. The acceleration time history can be obtained from observations of past earthquakes or from simulations using ground motion prediction equations.

Once the acceleration time history is defined, it is applied to the structure as a load. The response of the structure is then evaluated at each point in time using numerical methods such as the finite element method or the boundary element method. The response of the structure includes the displacement, velocity, and acceleration of the structure at each point in time.

The results of a time history analysis can be used to determine the maximum response of the structure, including the maximum displacement, velocity, and acceleration. Additionally, a time history analysis can be used to evaluate the damage that the structure may experience under the specified seismic loads. This can include the determination of strength and ductility demand ratios, and damage assessment.

Overall, time history analysis is an important method for understanding the seismic behavior of structures and for designing and retrofitting structures for better seismic performance. For these raisons, using time history analysis in a graduation project can provide a practical understanding of the seismic behavior of structures. Additionally, time history analysis can be used to validate the results obtained from other analysis methods, such as pushover analysis. It can also be used to study the effect of different parameters on the seismic performance of structures, such as the effect of soil-structure interaction on the response of a structure.

I.6. Conclusion:

In summary, the chapter discusses nonlinear behavior in buildings and the concept of performance-based seismic design (PBD) and how it differs from traditional design methods such as force-based design and displacement-based design.

Furthermore, nonlinear behavior refers to the fact that the structural response of a building to external loads (such as wind or earthquakes) is not proportional to the magnitude of those loads. This means that the building's response may be significantly different at different levels of loading. Nonlinear analyses, which involve more effort and computation, can provide a more accurate prediction of a building's response to extreme loading events and ensure the safety and performance of a building during such events.

Chapter II:

Dynamic response of structures under seismic actions

II.1 Introduction:

One can conceive the dynamic study as an extension of the static calculation, by means of, simply, the introduction into the equilibrium equation of the additional dynamic forces Caused by the movement of the structure. Thus, the dynamic equilibrium relation is written:

$$M\ddot{U} + C\dot{U} + KU = P(t) \qquad \text{(II-1)}$$

This equation further includes the stiffness forces (KU) related to the displacements that Characterize the static problem, the forces of inertia (MU) related to the vector acceleration and the damping forces (CU) which are associated with the velocity vector. Considering these forces functions of time, transforms the static problem posed in the form of a system of algebraic equations into a dynamic problem described by a system of differential equations coupled. As a result, dynamic calculation is very different from static calculation, and requires much more of work. **[2]**

The resolution of equation (II-1) can be carried out using several methods (elastic or inelastic) that are available to predict behavior seismic of structures:

- ✓ Linear static analysis
- ✓ Linear dynamic analysis
- ✓ Nonlinear static analysis (pushover)
- ✓ Nonlinear dynamic analysis (nonlinear time history analysis)

II.2 Earthquakes:

Earthquakes are usually caused by incipient seismic waves during jerky movements of the earth's crust in a rupture zone (fault zone). Waves of various natures and speeds travel different paths before reach a site and subject the ground to various movements. The impact of earthquakes on structures is largely determined by the three parameters describing ground movement: acceleration, velocity, and displacement. Factors such as distance, direction, depth, epicenter, and local soil characteristics influence the movement of the ground during an earthquake. The seismic waves can be classified into body waves (P wave and S wave) and surface waves (Love wave and Raleigh wave). **[12]**

II.2.1 Seismic action:

The action of an earthquake on a structure results in a variable displacement of the ground in time$u_g(t)$, which involves alternating components of translation $U(t)$and rotation θ (t)

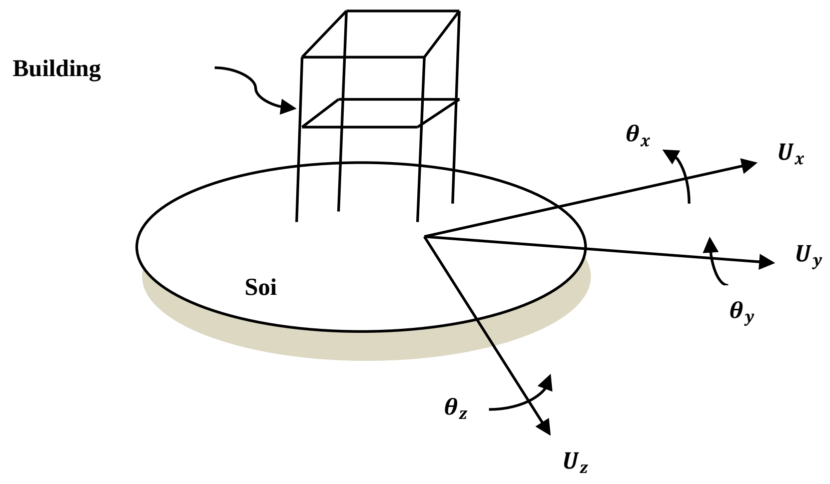


Figure II.1: Seismic action on a structure

In seismology, the characterization of earthquakes is carried out in various ways. The Magnitude (Richter scale) represents the energy released at the focus; Intensity makes it possible to characterize the damage observed according to a quantitative scale (Mercalli). For engineers, the most interesting parameters are the ground displacement laws $u_g(t)$ and accelerograms $\ddot{u}_g(t)$. **[12]**

The seismic action can also be determined using the ground acceleration (or accelerogram) $\ddot{u}_g(t)$ in translation and in rotation. Rotational components generally have negligible effects. The vertical translation component is weaker than the horizontal components. The main effect of the earthquake is therefore a horizontal movement of the ground. In the temporal dynamic calculation, the seismic action is represented by accelerograms, which can be generated artificially, simulated or even obtained from real recordings (figure 2.2). From the calculation point of view; three important seismic parameters contribute to cause damage to structures: amplitude of ground motion, the frequency content of the accelerograms and the duration of the earthquake.

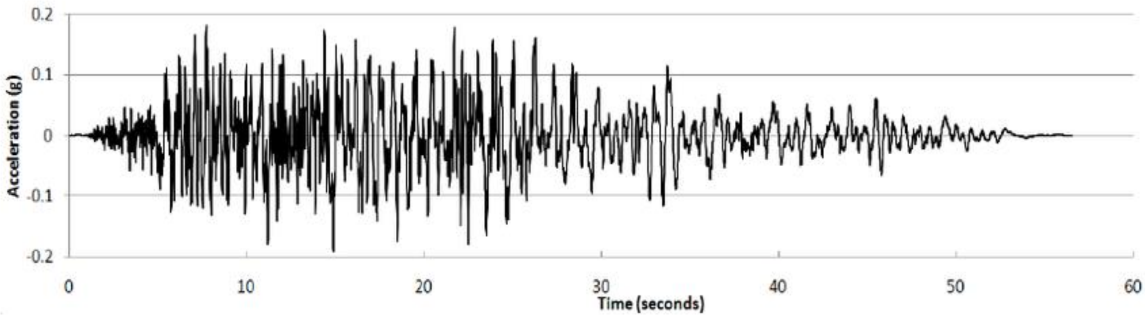


Figure II.2: Recorded accelerogram of the Imperial Valley Earthquake of 12 October 1979, Cerro Prieto Station, 237o Component

II.3 Dynamic response of structures:

The most widespread application of structural dynamics in civil engineering is in the study of the response of structures to earthquakes. The main reason is that earthquakes generate significant inertial forces for the large majority of buildings.

The dynamic response of a structure leads to the determination of the displacements of the structure over time. For a linear system, we add the static response maximum to the maximum dynamic response, in order to obtain the total response. In the case of a nonlinear system, it is first necessary to calculate the effects of the applied forces in a static way, and then add the effects of the dynamic forces to obtain the total response. **[12]**

II.4 Resolving the Equations of Motion for a Linear System:

II.4.1 Equations of motion:

The writing of the equations of the movement, for a simple system in shears, led to define the matrices of mass, damping and rigidity. We can write the dynamic equilibrium equation in the form of the matrix equation:

$$f_I + f_D + f_s = P(t) \tag{II-2}$$

Where:

f_I : is the vector of the forces of inertia which is written f_I =M$\ddot{U}$

f_D: The vector of the damping forces whose expression is f_D=C$\dot{U}$

f_s: The vector of the elastic forces which is expressed f_s=KU

The equation becomes:

$$M\ddot{U}(t) + C\dot{U}(t) + KU(t) = 0 \tag{II-3}$$

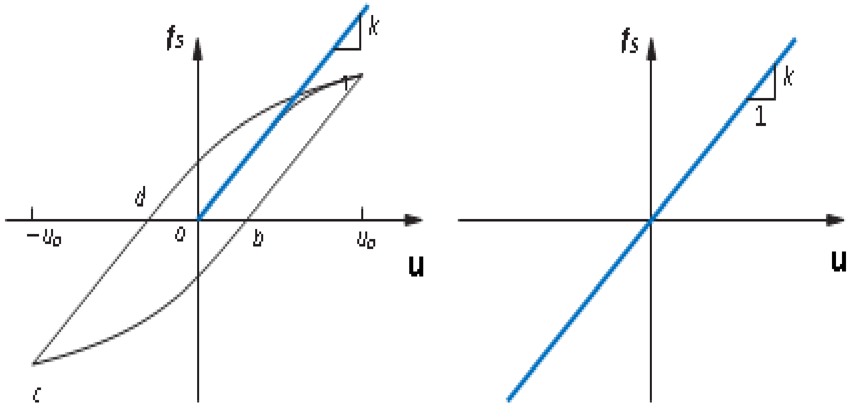


Figure II.3: Force–displacement relation

In addition, the displacement composition rule makes it possible to express the displacement Absolute according to the relative displacement by the following expression:

$\ddot{U}_t(t)$= u+r.u_g (II-4)

Where r is the dynamic coupling vector, which relates the direction of motion at the base with the direction of each degree of freedom.

$M\ddot{U}(t) + C\dot{U}(t) + KU(t)$ = -M.r.$\ddot{u}_g(t)$=p(t) (II-5)

As the system (II-5) is linear its resolution can be done either by the method of direct integration step by step in time, or by the method of modal superposition. It is the latter which is generally used, when it comes to linear systems.

II.4.2 Modal superposition:

Modal superposition is a technique used in structural dynamics to analyze the response of a system to external excitations, such as earthquakes, wind, or other types of loads. Modal superposition is a very efficient for calculating the response in the time, linear structures with several degrees of freedom subjected to a loading dynamic (or base acceleration).

The solution sought is a linear combination of the solutions of decoupled equations. Modal analysis consists in transforming the geometric coordinate equations into a new system of generalized coordinates.

For the calculation of the response in the geometric coordinate system, from the modal responses y_i, displacements of the modal component u_i are given by:

$u_i(t) = \Phi_i y_i(t)$ (II-6)

The travel vector is the sum of all modal components.

U(t) = Φ y(t)

U: geometric displacement vector.

Y: generalized displacement vector.

Φ: the modal matrix that allows the transformation of geometric coordinates to generalized coordinates.

The equation of motion is written as follows:

$[M]\{\ddot{u}\} + [C]\{\dot{u}\} + [K]\{U\} = P(t)$ (II-7)

By replacing equation (2-II) in equation (II-7):

$M\Phi\ddot{Y} + C\Phi\dot{Y}$+K$\Phi$Y= P (t) (II-8)

The division is made by multiplying the equation of the given motion by the equation (II.8) by transposing the vibration modeΦ_i^t:

$\Phi_i^t M\Phi\ddot{Y} + \Phi_i^t C\Phi\dot{Y} + \Phi_i^t k\Phi y = \Phi_i^t P(t)$ (II-9)

Using the orthogonality properties for both vibration modes (i and j):

$\Phi_i^t M \Phi_j = 0$ for i≠j (II-10)

$\Phi_i^t k \Phi_j = 0$ for i≠j (II-11)

And we also assume that the orthogonal property applies to the damping matrix C:

$\Phi_i^t C \Phi_j = 0$ for i ≠ j (II-12)

So the equation becomes:

$$M_i \ddot{y}_i + C_i \dot{y}_i + k_i y_i = P_i(t) \quad \text{(II-13)}$$

With:

$M_i = \Phi_i^t M \Phi_i$ General mass of mode i

$C_i = \Phi_i^t C \Phi_i$ Generalized depreciation of mode i

$K_i = \Phi_i^t K \Phi_i$ General rigidity of mode i

$P_i(t) = \Phi_i^t P(t)$ General loading of mode i

On divise l'équation (2.13) par M_i on obtient :

$$\ddot{y}_i + 2\zeta_i \dot{y}_i + w_i^2 y_i = \frac{P_{i(t)}}{M_i} \quad \text{(II-14)}$$

With:

$$w_i^2 = \frac{K_i}{M_i} \quad \text{(II-15)}$$

$$\zeta_i = \frac{C_i}{2 w_i M_i} \quad \text{(II-16)}$$

w_i : Proper pulsation of the i mode.

ζ_i : Mode i damping rate.

For each mode considered i, the equation (II.14) can be solved either by Step-by-step integration, either by using a decomposition of the load in series Fourier, or by applying the integral of Duhamel. [2]

The answer for i mode using the Duhamel integral of writes:

$$y_{i(t)=\frac{1}{M_i w_{Di}} \int_0^t P_{i(\mathcal{T})} e^{-\zeta_i w_i (t-\mathcal{T}}}) \operatorname{Sin} w_{Di}(t-\mathcal{T})\, d\mathcal{T} \quad \text{(II-17)}$$

Where:

$$w_{Di} = w_i \sqrt{1-\zeta_i^2} \quad \text{(II-18)}$$

w_{Di} : Pulsation of damped vibrations of mode i

After solving the equations for each of the generalized coordinates $y_i(t)$, we get the geometric displacements $u_i(t)$.

$u_i(t) = \sum_{j=1}^{n} \Phi_j^i \, y_j(t)$ (II-19)

Φ_j^i : Degree of freedom i of mode j

II.5 Resolving the Equations of Motion for a non-linear System:

The motion equations (II-5) described above is only valid for low intensity seismic stresses where the behavior of the structure remains linear during vibrations.

In the case of seismic stresses of major intensities, the mechanical properties of the structure change during vibration and therefore the behavior of the structure becomes non-linear. Non-linear analysis must be used to obtain a dynamic response realistic.

Two main examples requiring this non-linear calculation:

- If the elastic limit of the material is reached and causes plastic deformation of some members, the overall stiffness of the structure will be modified during the dynamic response.
- If the structure is slender, the axial forces in the columns can cause P-Δ effects significant causing, once again, a reduction in the apparent stiffness of the structure during the dynamic response.

In the case of nonlinear calculation, one integrates directly the coupled equations of the movement in using a step-by-step numerical integration technique. The full answer of system is divided into short increments Δt (or steps) of time.

We obtain the response of the system at each time step, assuming that the system is linear having the properties calculated at the beginning of the step. At the end of each time step, the properties of the system are modified according to their state of stresses and deformations.

II.5.1. Incremental equations of motion:

The equation of motion for degree of freedom i at time t is written as follows:

$m_i \ddot{u}_i(t) + F_{Di}(t) + F_{Si}(t) = -m_i r_i \ddot{u}_g(t)$ (II-20)

With:

$F_{Di}(t)$: Total nonlinear damping force acting on the DDL (i) at time t

$F_{Si}(t)$: Total nonlinear restoring force acting on the DDL (i) at time t

m_i : Mass corresponding to the DDL (i).

r_i : Component of the coupling vector corresponding to the DOF (i).

$\ddot{u}_i(t)$: Acceleration of the DOF(i) with respect to the base.

$\ddot{u}_g(t)$: Ground acceleration.

We also have the restoring and damping forces which represent the influence on the DOF (i) of all the elements of the system and are written:

$$F_{Di}(t) = \sum_{j=1}^{N} f_{Dij}\,(t)$$

$f_{Dij}(t)$: Nonlinear force applied on the DOF (i) at time t to cause a velocity $\dot{u}_g(t)$ at DOF (j), at time t.

The equation of motion at one time increment t + Δt is written:

$$m_i\ddot{u}_i(t+\Delta t) + F_{Di}(t+\Delta t) + F_{Si}(t+\Delta t) = -m_i r_i \ddot{u}_g(t+\Delta t) \quad \text{(II-21)}$$

We subtract equation (II-20) from equation (II-23):

$$m_i[\ddot{u}_i(t+\Delta t) - \ddot{u}_i(t)] + [F_{Di}(t+\Delta t) - F_{Di}(t)] + [F_{Si}(t+\Delta t) - F_{Si}(t)] = -m_i r_i[\ddot{u}_g(t+\Delta t) - \ddot{u}_g(t)] \quad \text{(II-22)}$$

The equation becomes:

$$m_i\Delta\ddot{u}_i(t) + \Delta F_{Di}(t) + \Delta F_{Si}(t) = -m_i r_i \Delta\ddot{u}_g(t) \quad \text{(II-23)}$$

The equations of the system become matrix equations, the dynamic balance of forces can be written:

$$M\Delta\ddot{U}(t) + \Delta F_D(t) + \Delta F_S(t) = -Mr\Delta\ddot{u}_g(t) \quad \text{(II-24)}$$

$\Delta F_D(t)$: C(t).$\Delta\dot{u}(t)$

$\Delta F_S(t)$: K(t).$\Delta u(t)$

$\Delta\ddot{u}_g(t)$:$\ddot{u}_g(t+\Delta t) - \ddot{u}_g(t)$

We must calculate the elements of the matrices $k(t)$ and $C(t)$ at each time step, such as:

$$C_{ij}(t) = \left(\frac{df_{Dij}}{d\dot{u}_j}\right)_t \quad \text{(II-25)}$$

$$k_{ij}(t) = \left(\frac{df_{Sij}}{du_j}\right)_t \quad \text{(II-26)}$$

We replace the equations and in the equation, we obtain the incremental equation of motion:

$$M\Delta\ddot{U}(t) + C(t)\Delta\dot{U}(t) + k(t)\Delta U(t) = -M\,r\,\Delta\ddot{U}(t) \quad \text{(II-27)}$$

II.5.2 Techniques resolution and procedures:

Analysis of a nonlinear dynamic problem, using the method of finite elements, requires the use of stepwise integration algorithms to solve the equation dynamic equilibrium, there are two types of temporal diagrams: The implicit diagram and explicit. The latter is easy to implement, because it allows calculating the result of the equation at time (t + Δt) as a function of the quantities at time (t), the major drawback of this method lies in the need to consider a relatively small time step to allow convergence of the scheme. Implicit methods are more difficult to implement as soon as it comes to dealing with highly non-linear problems; indeed, so that the equilibrium equation is validated at time (t + Δt), a convergence is performed on (Δt), so if the nonlinearities are important during (Δt), the convergence of the problem will be difficult, on the other hand, the advantage of such a method is the stability unconditional of the scheme thus allowing the use of a larger time step.

II.5.2.1 Newmark Algorithm Family:

The Newmark algorithm family is a set of mathematical algorithms that are used to solve the equations of motion in structural analysis and design. The algorithms are based on the Newmark method, which is a numerical integration technique that can be used to predict the dynamic response of structures subjected to earthquakes or other types of excitation.

A. Constant Average Acceleration Method (Newmark-beta method):

The Newmark method uses a constant average acceleration approximation to calculate the acceleration and velocity of the structure at each time step, which allows for accurate and efficient calculations of the response.

The basic assumption of this method holds that the relative acceleration of each DOF is constant during a time step and that the properties of the system do not change during this time step. [2]

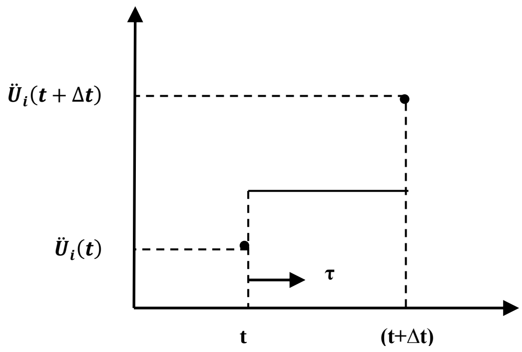


Figure II.4: assumption of constant average acceleration

The relative acceleration of DOF i during a time step is written as follows:

$$\ddot{u}_i(\tau) = \frac{1}{2}[\ddot{u}_i(t) + \ddot{u}_i(t+\Delta t)]; t \leq \tau \leq t + \Delta t \quad \text{(II-28)}$$

By integrating equation (2-28), we obtain the relative speed of DOF i during a time step.

$$\dot{u}_i(\tau) = \dot{u}_i(t) + \frac{1}{2}(\tau - t)[\ddot{u}_i(t) + \ddot{u}_i(t + \Delta t)] \quad \text{(II-29)}$$

The velocity at the end of time is written:

$$\dot{u}_i(t + \Delta t) = \dot{u}_i(t) + \frac{1}{2}\Delta t[2\ddot{u}_i(t) + \Delta\ddot{u}_i(t)] \quad \text{(II-30)}$$

The velocity increment during the time step is:

$$\Delta\dot{u}_i(t) = \dot{u}_i(t + \Delta t) - \dot{u}_i(t) = \frac{1}{2}\Delta t[2\ddot{u}_i(t) + \Delta\ddot{u}_i(t)] \quad \text{(II-31)}$$

By combining all the degrees of freedom, we can write an expression for the vector of velocity increments.

$$\Delta\dot{U}_i(t) = \frac{1}{2}\Delta t\left[2\ddot{U}(t) + \Delta\ddot{U}(t)\right] \quad \text{(II-32)}$$

We proceed in the same way for the displacement for a DOF i during a time. By combining all the DOF, the expression for the vector of the increments of displacements, can be written as follows:

$$\Delta U(t) = \Delta t\dot{U}(t) + (\Delta t^2)\left[\ \frac{1}{2}\ \ddot{U}(t) + \frac{1}{4}\ \Delta\ddot{U}(t)\right] \quad \text{(II-34)}$$

B. Linear acceleration method:

The basic assumption of this method is that the relative acceleration of each DOF varies linearly during a time step while the properties of system remain constant during this time step. The relative acceleration of DOF i during a time step is written as follows:

$$\ddot{u}_i(t) = \ddot{u}_i(t) + \left[\frac{(\tau - t)}{\Delta t}\right][\ddot{u}_i(t + \Delta t) - \ddot{u}_i(t)];\, t \le \tau \le \Delta t \quad \text{(II-35)}$$

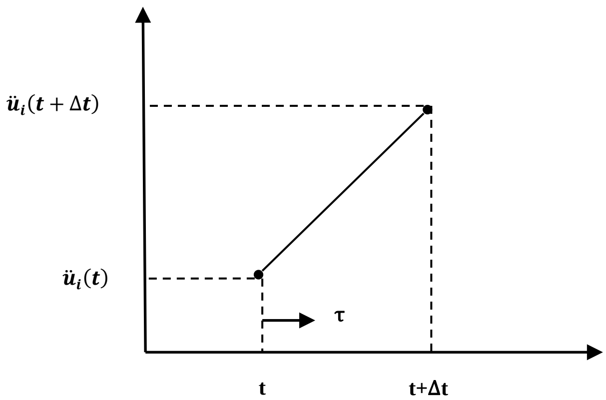


Figure II.5: Linear acceleration assumption

By integrating equation (II.35), we obtain the relative velocity of DOF i during a time step.

$$\dot{u}_i(t) = \dot{u}_i(t) + (\tau - t)\ddot{u}_i(t)\left[\frac{(\tau - t)^2}{2\Delta t}\right][\ddot{u}_i(t + \Delta t) - \ddot{u}_i(t)];\, t \le \tau \le t + \Delta t\ . \quad \text{(II-36)}$$

The velocity at the end of time is written:

$$\dot{u}_i(t + \Delta t) = \dot{u}_i(t) + \Delta t\ddot{u}_i(t) + \frac{1}{2}\Delta t\Delta\ddot{u}_i(t). \quad \text{(II-37)}$$

The velocity increment during the time step is written:

$$\Delta\dot{u}_i(t) = \Delta\ddot{u}_i(t) + \frac{1}{2}\Delta t\Delta\ddot{u}_i(t)\ . \qquad \text{(II-38)}$$

By combining all the DOF, the expression of the vector of the increments of velocity is written:

$$\Delta\dot{u}_i(t) = \Delta t\ddot{u}_i(t) + \frac{1}{2}\Delta t\Delta\ddot{u}_i(t). \qquad \text{(II-39)}$$

We proceed in the same way for the relative displacement for a DOF i during a time.

By combining all the DOF, the expression for the vector of the increments of displacements, can be written as follows:

$$\Delta u(t) = \Delta t\dot{u}_i(t) + \Delta t^2\left[\ \frac{1}{2}\ \ddot{u}_i(t) + \frac{1}{6}\Delta\ddot{u}_i(t)\right]. \qquad \text{(II-40)}$$

C. Newmark's algorithm:

Newmark's implicit algorithm is very useful in programming; it combines the constant average acceleration method with the linear acceleration method. This algorithm depends on two parameters γ and β. The vector of the increments of speed and displacements is written in a general way.

$$\Delta\dot{u}(t) = \Delta t\ddot{u}(t) + \gamma\Delta t\Delta\ddot{u}(t). \qquad \text{(II-41)}$$

$$\Delta u(t) = \Delta t\dot{u}(t) + \Delta t^2\left[\frac{1}{2}\ \ddot{u}(t) + \beta\Delta\ddot{u}(t)\right]. \qquad \text{(II-42)}$$

Particular choices of γ and β make it possible to find the preceding diagrams of integration:

- Constant average acceleration $\gamma = \frac{1}{2}\ ,\ \beta = \frac{1}{4}$
- Linear acceleration $\gamma = \frac{1}{2}\ , \beta = \frac{1}{6}$

D. Integration of equations of motion:

To solve the vectors of displacements, velocities and accelerations at each time step, we consider the victor of displacements, u (t) as the basic variable. We write equation (II.42) as a function of the vector of acceleration increments.

$$\Delta\ddot{u}(t) = \left[\frac{1}{\beta(\Delta t)^2}\right]\Delta u(t) - \left[\frac{1}{\beta\Delta t}\right]\dot{u}(t) - \left[\frac{1}{2\beta}\right]\ddot{u}(t) \qquad \text{(II-43)}$$

We replace the equation (II.43) in the equation (II.41) we obtain:

$$\Delta\dot{u}(t) = \Delta t\ddot{u}(t) + \left[\frac{\gamma}{\beta\Delta t}\right]\Delta u(t) - \left[\frac{\gamma}{\beta}\right]\dot{u}(t) - \left[\frac{\gamma\Delta t}{2\beta}\right]\ddot{u}(t) \qquad \text{(II-44)}$$

To keep a single variable in the problem, we replace equations (II.43) and (II.44) in the incremental equation of motion (II.27).

$$M\left(\left[\frac{1}{\beta(\Delta t)^2}\right]\Delta u(t)-\left[\frac{1}{\beta\Delta t}\right]\dot{u}(t)-\left[\frac{1}{2\beta}\right]\ddot{u}(t)\right)+C(t)\left(\left[\frac{\gamma}{\beta\Delta t}\right]\Delta u(t)-\left[\frac{\gamma}{\beta}\right]\dot{u}(t)-\left[\frac{\gamma}{2\beta}\right]\Delta\ddot{u}(t)\right)+K(t)\Delta u(t)=-Mr\Delta\ddot{u}_g(t) \quad (\mathrm{II}-45)$$

The equation can be put in the form:

$$\widetilde{K}(t)\Delta u(t)=\Delta\tilde{P}(t) \quad \text{(II-46)}$$

With:

$$\widetilde{K}(t)=k(t)+\left[\frac{1}{\beta(\Delta t)^2}\right]M+\left[\frac{\gamma}{\beta\Delta t}\right]C(t) \quad \text{(II-47)}$$

$$\Delta\tilde{P}(t)=-Mr\Delta\ddot{u}_g(t)+M\left(\left[\frac{1}{\beta\Delta t}\right]\dot{u}(t)-\left[\frac{1}{2\beta}\right]\ddot{u}(t)\right)+C(t)\left(\left[\frac{\gamma}{\beta}\right]\dot{u}(t)-\left[\frac{\gamma}{2\beta}-1\right]\Delta\ddot{u}(t)\right) \quad \text{(II-48)}$$

Equation (II-45) is a linear system of equations that can be solved to find the victor of increments of displacements. Afterwards, we obtain the vector of the speed increments with the equation (II-44). The vectors of displacements and velocities at the beginning of the next time step are calculated by:

$$U(t+\Delta t)=U(t)+\Delta U(t) \quad \text{(II-49)}$$

$$\dot{U}(t+\Delta t)=\dot{U}(t)+\Delta\dot{U}(t) \quad \text{(II-50)}$$

We could solve the vector of increments of accelerations using equation (II-43); another way to proceed is to calculate the vector of accelerations for the next integration step using the equation of motion.

$$\ddot{u}(t+\Delta t)=M^{-1}\left[-Mr\ddot{u}_g(t+\Delta t)-F_D(t+\Delta t)-F_S(t+\Delta t)\right] \quad \text{(II-51)}$$

From where:

$$F_D(t+\Delta t)=F_D(t)+C(t)\Delta\dot{u}(t) \quad \text{(II-52)}$$

$$F_S(t+\Delta t)=F_S(t)+K(t)\Delta u(t) \quad \text{(II-53)}$$

II.6 Calculation methods:

The determination of the seismic response of the structure and its dimensioning can be done by several calculation methods, the choice of which depend both on the type of the structure and nature of the dynamic excitation, it is therefore a question of moving towards one of the following analysis methods:

- Equivalent Lateral Force Method (linear).
- Response Spectrum Method (linear).
- Non linear static analysis (push-over)
- Non linear Time history analysis.

II.6.1 Equivalent Static Method:

This method of finding design lateral forces is also known as the static method or the equivalent static method or the seismic coefficient method. This procedure does not require dynamic analysis; however, it accounts for the dynamics of building in an approximate manner. The static method is the most straightforward and requires less computational effort, relying on formula provided by the code of practice. To start, the design base shear is calculated for the entire building and then distributed among the various floor levels. The resulting lateral forces at each floor are then allocated to specific lateral load-resisting components. **[15]**

Equivalent static method can be used:

1. If the building is regular in plan and elevation with a height at most or equal 65 m in zone I and II and 30 m in zone III, the building or block studied must satisfied the conditions of regularity in plan and elevation. **[17]**
2. The studied building or block presents an irregular configuration while respecting, the height conditions set out in the first condition with the condition's additional information**.** **[17]**

II.6.2 Response Spectrum Method:

Dynamic analysis may be performed either by response spectrum method or by the time-history method. In the response spectrum method, the peak response of a structure during an earthquake is obtained directly from the earthquake response (or design) spectrum. This procedure gives an approximate peak response, which is quite accurate for structural design purposes. In this approach, the multiple modes of response of a building to an earthquake are considered. For each mode, a response is read from the design spectrum, based on the modal frequency and the modal mass. The responses of different modes are combined to provide an estimate of total response of the structure using modal combination methods. **[16]**

Response spectrum method can be used:

For structures represented by planar models, in two directions orthogonal, the minimum vibration modes to retain is three (03) in each direction of excitation considered, and the sum of the effective modal masses for the selected modes is equal to at least 90% of the total mass of the structure. **[17]**

II.6.3 Nonlinear static analysis (push-over):

The method is based on the assimilation of the vibratory response of the structure to that of a simple system with an equivalent degree of freedom. This response is presented through a curve force displacement which is obtained by a nonlinear static calculation of the model overall structure subjected to the horizontal action of the earthquake. The seismic action present in the form of acceleration fields or forces applied to the height of the building according to a given profile. The intensity of the seismic action is then increased by incrementally to build the nonlinear response curve. **[18]**

II.6.4 Non linear Time history analysis:

Although the spectrum method is a useful technique for the elastic analysis of structures, it is not directly transferable to inelastic analysis because the principle of superposition is no longer applicable. Also, the analysis is subject to uncertainties inherent in the modal superimposition method. The actual process of combining the different modal contributions is a probabilistic technique and, in certain cases, it may lead to results not entirely representative of the actual behavior of the structure. The THA technique represents the most sophisticated method of dynamic analysis for buildings. In this method, the mathematical model of the building is subjected to accelerations from earthquake records that represent the expected earthquake at the base of the structure. The method consists of a step-by-step direct integration over a time interval; the equations of motion are solved with the displacements, velocities, and accelerations of the previous step serving as initial functions.

In the time history analysis calculation, the seismic action is represented by accelerograms, which can be artificial, recorded or simulated.

- **Artificial accelerograms:**

Artificial accelerograms should be set up to match the spectrum elastic response, with at least three accelerograms. The duration of accelerograms must be compatible with magnitude and other characteristics specific to the seismic event used to define the acceleration, and when one does not have no specific data, the minimum duration of the game should be, steady state of the accelerograms is equal to 10s.

- **Recorded or simulated accelerograms:**

Recorded accelerograms or accelerograms produced from a physical simulation of source mechanisms and wave propagation can be used, provided that the samples used are recognized as representative of the characteristics of the seismic sources and ground conditions of the site. **[19]**

II.7. Conclusion:

The dynamic response of structures under seismic actions is a crucial aspect of seismic design and analysis. A structure's response to seismic loads depends on its mass, stiffness, and damping characteristics, as well as the characteristics of the ground motion.

To assess the seismic performance of a structure, various analysis methods can be used, such as linear static analysis, dynamic analysis, probabilistic seismic hazard analysis, and performance-based seismic design. The choice of method depends on the building's location, use, and complexity, as well as the goals of the analysis. The results of these analyses provide valuable information that can be used to improve the seismic performance of structures and ensure their safety during earthquakes.

Chapter III:

Building presentation

III.1 Introduction:

Our goal in this work is to examine a reinforced concrete building (R+4) intended for residential use, which requires a fundamental understanding on which a civil engineer depends in order to achieve a structure that is both secure and cost-effective. In this chapter, we will outline the various components and materials composing the building and outline the calculation methods with a preliminary overview of the project.

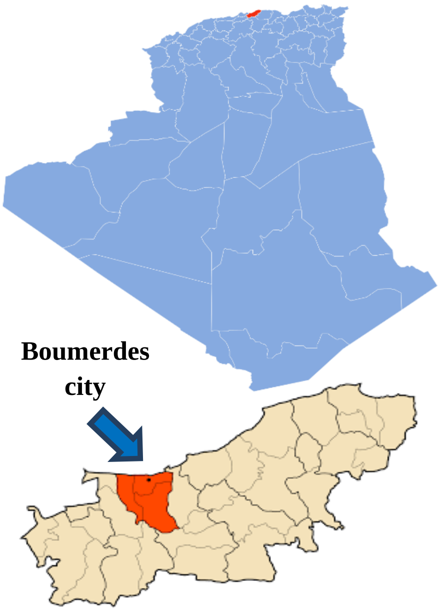
III.2 Presentation of the building:

The project entrusted to us is a building for residential use, comprising a ground floor level and 4 floors with an inaccessible terrace. It will be located in the city of Boumerdes which is classified as a high seismicity zone (Zone III) according to the latest Algerian seismic code (RPA 99 v 2003). All calculations are carried out in accordance with the regulations in force, namely:

- BAEL 91 **[20]** modified in 99 and CBA93 **[21]**.
- RPA99 version 2003 **[17]**.
- DTR BC.2.2 **[22]**.

III.2.1 Localization of the project:

The site of our project is located in the city of Boumerdes the north of Algeria



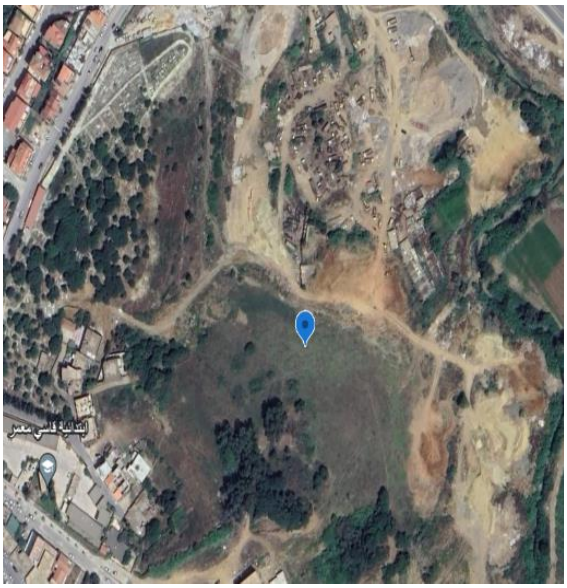


Figure III.1: Location plan and ground plan.

According to the Algerian seismic code (RPA 99 version 2003) our Collective residential buildings whose height does not exceed 48 m. is classified as Routine or medium-sized works (Group 2).

- The building consists on each floor of2 F4 apartments:
- The ground floor to the 4th floor for residential use.

III.2.2 Geometric characteristics:

a. Dimensions in elevation:

In elevation, the building has a total height of 15.30 m with a floor height of 3.06 m (Figure.III.2)

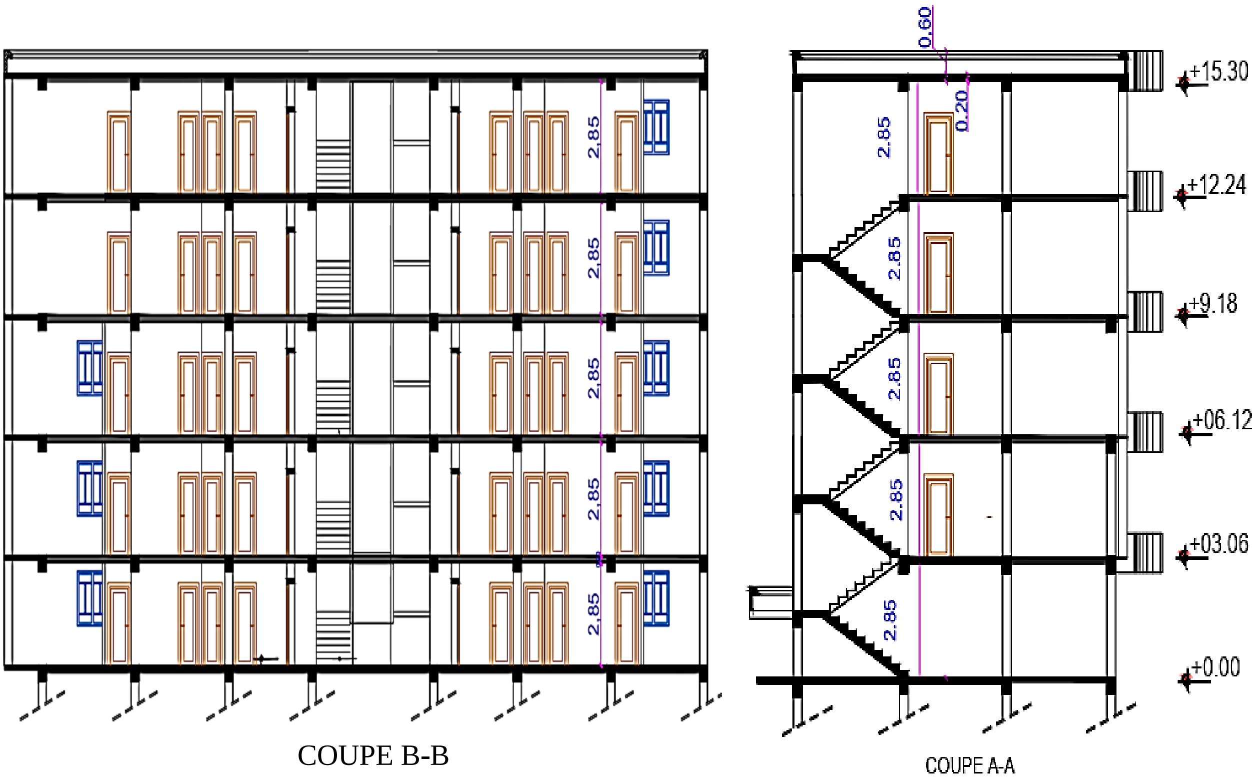


Figure.III.2 Elevation dimensions

b. Drawing dimensions (Dimensions en plan) :

The building has rectangular shape (Figure III.3) with:

- Overall width in plan … … … … … … … … … … 29.35 m
- Overall length in plan… … … … … … … … … … 13.20 m

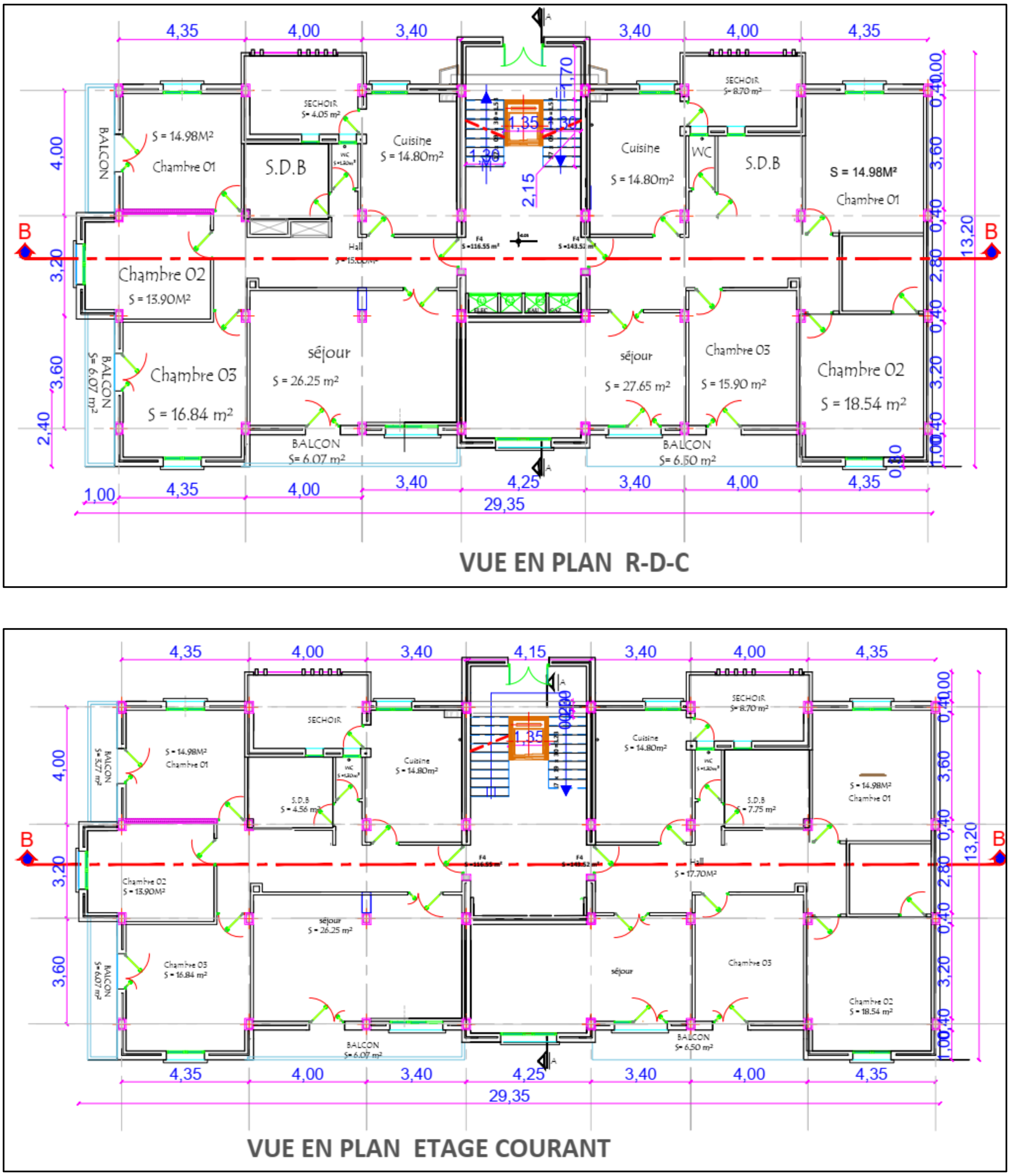
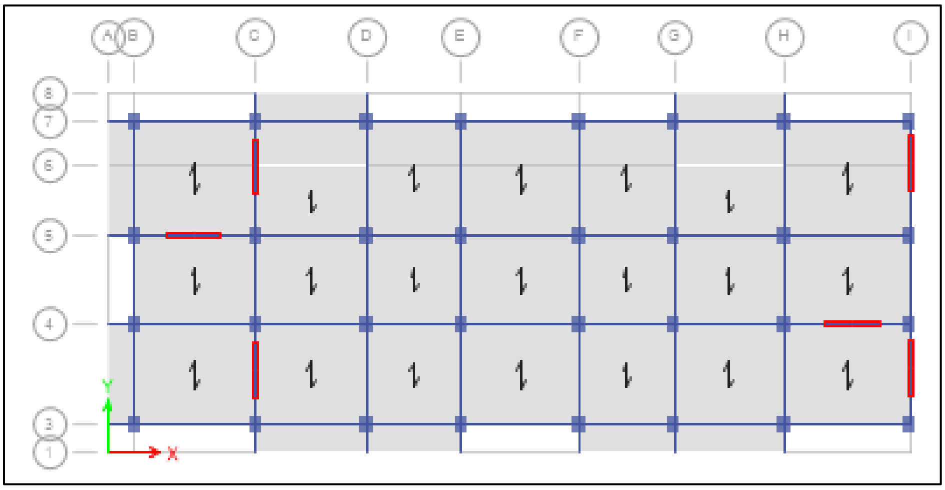


Figure III.3: Ground and stories plan view

III.2.3 Structural system:

The building under study has a total height of 15.9m, which requires, according to RPA99V2003, the use of a bracing system other than freestanding gantries. The system assumed beforehand is the mixed bracing system provided by walls and frames with justification of the interaction between frames and walls.



FigureIII.4: Shear walls arrangements

In this building we have two types of floors:

- Hollow block floors with a compression slab reinforced with a welded mesh, making the whole monolithic.
- The overhangs will be made of solid slabs.

III.2.4 classification of the building according to RPA99 version 2003:

The building is a work classified in (use group 2), because it is for residential use, the height of which does not exceed 48 m. According to the conditions of RPA 2003, a check of regularity in plan and elevation must be carried out **[17]:**

III.2.4.1Regularity in plane:

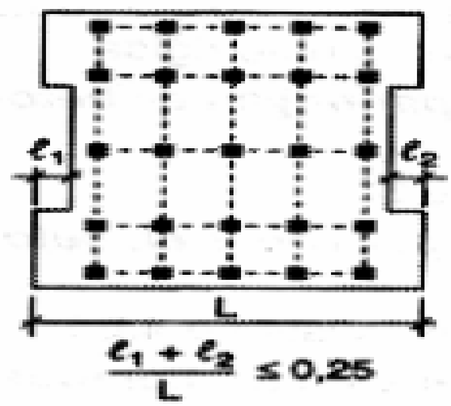
1. **Geometric regularity:**

❖ Condition 1 :

$0.25 \leq \frac{L_x}{L_y} \leq 4$

$0.25 \leq \frac{29.35}{13.2} \leq 4 \leftrightarrow 0.25 < 2.22 < 4 \ldots\ldots\ldots V.C$

❖ Condition 2 :



$\frac{l_1+l_2}{L} \leq 0.25$

$x_1 = 4.30\ m\ ;\ y_1 = 1\ m$

$x_2 = 10.5\ m\ ;\ y_2 = 1\ m$

$x_3 = 4.30\ m\ ;\ y_3 = 1\ m$

$x_4 = 4.10\ m\ ;\ y_4 = 1\ m$

According to X-X: $\frac{4.30+10.5+4.30+4.10}{29.35} = 0.79 > 0.25 \dots\dots\dots\dots$ unverified condition.

According to Y-Y: $\frac{1+1+1+1}{13.20} = 0.30 > 0.25 \dots\dots\dots\dots\dots$ unverified condition.

So, the structure is **irregular in plan.**

2. **Structural regularity in plan:**

- According to article Ar 3.5.1.a2 of the RPA2003 at each level and for each direction, the distance between the center of mass and the center of rigidity does not exceed 15% of the dimension of the building measured perpendicular to the direction of the seismic action considered

Center of rigidity*:*

X_{CR}: 16.08 m

Y_{CR}: 7.11m

Center of mass:

X_{CM}: 14.65 m

Y_{CM}: 6.33 m

With:

Lx=29.35 m

Ly=13.20 m

We must have: $e_x \leq 0.15L_X$ and $e_y \leq 0.15L_y$

e_x = XCR-XCM $\longrightarrow$ e_X= 16.08 -14.65 =1.43 m < 0.15Lx = 4.40 m ……. V

e_y = YCR-YCM $\longrightarrow$ e_y =7.11 -6.33= 0.78 m < 0.15Ly= 1.98 m ……… V

- According to article Ar 3.5.1.a4 of the RPA2003 the floors must have sufficient rigidity with respect to that of the vertical braces to be considered non-deformable in their plane. In this context, the total area of floor openings must remain less than 15% of that of the latter.

We must have: $S_{opening}/S_{floor} \leq 15\%$

$S_{opening} = 4.25 \times 4 = 17\ m^2$

$S_{floor} = 387.42\ m^2$

$S_{opening}/S_{floor} = 0.04 < 15\% \dots\dots\dots\dots verified\ condition$

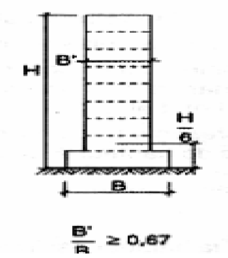
So this floor is considered non-deformable (rigid diaphragm).

The other floors are automatically checked for this condition (identical floor).

III.2.4.2 Regularity in elevation:

1. **Geometric regularity:**

According to RPA 2003 paragraph§3.5.1.b4: In the case of setbacks in elevation, the variation in the plan dimensions of the building between 2 successive levels does not exceed 20% in the two directions of calculation (all levels are identical).



- Condition1: $\frac{B'}{B} \geq 0.67 \rightarrow \frac{29.35}{29.35} = 1 > 0.67 \ldots\ldots\ldots..verified\ condition$

The construction is considered regular in elevation

2. **Structural regularity:**

Vertical structural irregularities that can adversely affect the seismic resistance of a building.

- The bracing system of our structure is continuous throughout the height.
- Masses and rigidity are distributed on a regular way.

So, our structure is regular in elevation.

According to the conditions of RPA99/version 2003 our structure is considered **irregular**, so the equivalent static method is not applicable.

III.3 Characteristic of the site of Assisi:

According to the geotechnical report of the geotechnical soil laboratory **(Annex)**, the foundation soil is a succession of compact crusts and limestone encrustations (from an anchoring depth of 0.2m becoming gravelly from a depth of 3m).

The examination, of all the physical, mechanical and chemical characteristics makes is possible to carry out the following elements of appreciation:

III.3.1. Physical characteristics:

- The water content values found on the limestone crusts are between 5% and 8%.
- The degree of saturation oscillates between 23 and 34%. Such a range denotes a natural water state of a dry soil.
- The values of the dry density vary for limestone crusts between 1.56 and 1.6 t/m3. For the apparent wet densities are of the order of 1.66t/m3 to 1.74 t/m3.

According to the geotechnical standard, this formation belongs to the families of semi-dense soils.

- According to the particle size analysis carried out on the limestone crusts showed that the percentage of passers-by at 0.08mm is less than 40%, it is a grainy formation.

III.3.2. Mechanical characteristics:

The mechanical parameters provide direct access to the bearing capacity of soils, compatible with acceptable deformation (settlement) (shear tests).

- The cohesion values for the limestone crusts are low; they are of the order of 0.13to 0.24 bars.
- The angle of friction is higher; it is between 19° and 24°.

After tests carried out in laboratories and on site (dynamic penetrometer).The allowable soil stress varies between 2.3 and 3 bars, so the stress to be taken into consideration is equal: $\overline{\boldsymbol{\sigma_s}} = \mathbf{2.3}\ \boldsymbol{bars}$

III.3.3. Chemical characteristics:

With regard to chemical and mineralogical analyzes of the soil; they indicate zero aggressiveness (no presence of sulphate), normal cement may be suitable for making foundation concrete.

So, the geological nature of the terrain consists of soils with average geotechnical characteristics.

According to the seismic regulations (RPA99/version 2003) in force all the geotechnical data make it possible to classify the site in **category S3(Soft site).**

The building is located in an area classified by RPA99/version 2003 as a high seismic zone (zone III); therefore, seismic rules should be designed in the project design.

III.4 Characteristics of materials:

The characteristics of the materials used in the construction will comply with the technical rules for the design and calculation of reinforced concrete structures (BAEL 91 modified 99) **[20]** and with any regulations in force in Algeria (RPA 99 version 2003) **[17]** and (CBA93) **[21]**

The materials used in the composition of the structure undoubtedly play an important role in the resistance of buildings to earthquakes.

Their choice is often the result of a compromise between various criteria such as; the cost, the availability on site and the ease of implementation of the material generally prevail over the criterion of mechanical resistance. The latter, on the other hand, is decisive for large constructions.

III.4.1 Introduction:

Reinforced concrete is a composite material, made up of two essential components: concrete and steel. The structural quality of reinforced concrete is imperatively linked to the quality of its constituents. In what follows we will study the main properties of concrete and steel.

In our study, we used the limit state concrete regulations, namely BAEL91, CBA 93 and the Algerian seismic code RPA 99, 2003 version. The BAEL 91 regulations are based on the limit states defined below.

III.4.2 Definition:

A limit state is that for which a condition required of a construction or one of its elements (such as stability and durability) is strictly satisfied and would cease to be so in the event of an unfavorable modification of an action.(Increase or reduction depending on the case). There are two limit states:

- **Ultimate limit states: (ULS)**

Match limit:

- Either the static equilibrium of the construction (no reversal).
- Either the resistance of one of the materials (no rupture).
- Or form stability (buckling).

- **Service limit states: (SLS)**

Which define the conditions that the work must meet so that its normal use and durability are ensured.

- Compressive limit state of concrete.
- Crack opening limit state.
- Strain limits state.

III.4.3 Concrete:

III.4.3.1 Definition:

Concrete is a heterogeneous material made up of a mixture of hydraulic binder (cement), inert materials called aggregates (sand, gravel, etc.), water and additives if necessary. The concrete used in the structure of the work must comply with the technical rules for the study and design of reinforced concrete works.

Concrete should have the following advantages:

- Good compressive strength.
- Flexibility of use.
- Good fire resistance
- A possibility of obtaining prefabricated elements of different shapes.

III.4.3.2 Concrete batching:

The choice of the proportions of each of the constituents of the concrete in order to obtain the desired mechanical and implementation properties is called the formulation. It is a very important operation. The constituents per 1m3 of concrete are as follows:

- 350 kg of cement (CPA-325, CRS).
- 400 liters of sand « $D \leq 5mm$ ».
- 800 liters of gravel « $10mm \leq D \leq 30$ mm ».
- 180 liters of mixing water.

The concrete obtained will have a density which varies between 2200 Kg/m3 and 2500 Kg/m3 this formulation leads to a reinforced concrete with a density of (ρ = 25 KN/m3).

III.4.3.3 Physical and mechanical characteristics of concrete:

III.4.3.3.a Physical characteristic:

1) The density:

The density of the concretes is between (2200 and 2500) Kg/m3. This density can increase with the modality of implementation, in particular with vibration.

2) Shrinkage:

It is the decrease in length of a concrete element, it can be likened to the effect of a lowering of temperature which leads to a shortening.

3) Dilation (CBA Art A.3.1.3.3):

Since the coefficient of thermal expansion of concrete is evaluated at 10-5, for a variation of ±20 we obtain: $\Delta l = \pm$ (2/1000) x lengths.

4) Creep:

It is the phenomenon of deformation caused over time under a constantly applied fixed load.

III.4.3.3.b Mechanical characteristic:

a) **Compressive strength (BAEL 91 Art A.2.1.11) [20]:**

In common cases, a concrete is defined by the value of its compressive strength for a period of 28 days, which is called the required (or specified) characteristic value. This, denoted "f_{C28}", is measured by axial compression of a right cylinder of revolution of 200 cm2 of section and of a height double to its diameter.

The compressive strength is given (at j days) by:

- For resistors $f_{c28} \leq 40\ MPA$
  - $f_{cj} = \frac{j}{(4.76+0.83j)} \times f_{c28}$ Si $j \leq 28j$
  - $f_{cj} = f_{c28}$ Si $j > 28j$
- For resistors $f_{c28} > 40\ MPA$
  - $f_{cj} = \frac{j}{1.4+0.95j} \times f_{c28}$ Si $j \leq 28j$
  - $F_{cj} = f_{c28}$ Si $j > 28j$

In our project we take: $\boldsymbol{f_{c28} = 25\ MPA}$ **.**

b) Tensile strength (CBA.93 Art A. 2.1.1.2) [21] :

The characteristic tensile strength of concrete at "j" day, notedf_{tj} , is conventionally defined by the relationship:

$f_{tj} = 0.6 + 0.06\ f_{cj}$ $\quad\quad$ $f_{tj}\ et\ f_{cj}\ en\ MPA$

$f_{c28} = 25\ MPA$; We then find $\boldsymbol{f_{t28} = 2.1\ MPA}$

❖ **Modulus of longitudinal deformation:**

➢ **Instantaneous longitudinal deformation modulus: (BAEL91 Art A-2.1. 2.1) [20]**

For a loading lasting less than 24 hours, the instantaneous longitudinal deformation modulus of the concrete for j days is equal:

$E_{ij} = 11000\sqrt[3]{f_{cj}}$ $\quad (MPA)$ $\quad\quad i: instantaneous$ $\quad j: days$

For our case: $f_{c28} = 25\ MPA\ \ so\ \ \boldsymbol{E_{ij} = 32164.195\ MPA}$

➢ **Modulus of deferred longitudinal deformation: (BAEL91 Art A-2.1. 2.1) [20]**

The modulus of deferred longitudinal deformation of concrete "E_{vj}" at "j" days due to creep and shrinkage is given by the formula:

$E_{vj} = 3700\sqrt[3]{f_{cj}} \quad MPA$

For our case: $f_{c28} = 25\,MPA \quad so \quad \boldsymbol{E_{vj} = 10818.865\,MPA}$

❖ **Poisson coefficient: (BAEL91 Art A-2.1.3) [20]:**

The Poisson's ratio represents the relative variation in transverse dimension of a part subjected to a relative variation in longitudinal dimension.

$$\upsilon = \frac{\frac{\Delta d}{d}}{\frac{\Delta l}{l}} \begin{cases} \frac{\Delta d}{d} : \text{Relative elongation of cross} - \text{section} \\ \frac{\Delta l}{l} : \text{Relative elongation of the longitudinal section} \end{cases}$$

In the calculation, the Poisson's ratio is taken equal to

- $\upsilon = 0$ at ULS (cracked concrete)
- $\upsilon = 0.2$ at SLS (uncracked concrete)

❖ **Volume weight:**

We adopt the value $\rho = 25 \ KN/m^3$

❖ **Restraint calculation limits:**

- **Restraint limits at ultimate limit state (ULS): (Art A-4. 3.4 BAEL91) [20]:**

The ultimate limit state is generally defined by the limit of mechanical resistance beyond which there is collapse of the structure. $f_{bu} = \frac{0.85\times f_{c28}}{\theta\times\gamma_b}$ With:

γ_b: Safety coefficient.

- $\gamma_b = 1.15$ Accidental combinations.
- $\gamma_b = 1.5$ Common combinations.

θ : is a coefficient which considers the duration of application of the loads.

- $\theta = 1$ If the duration of application of the loads is greater than 24 hours.
- $\theta = 0.9$ If the duration of application of the loads is between 1h and 24h.
- $\theta = 0.85$ If the duration of application of the loads is less than 1 hour.

For $\gamma_b = 1.5\ et\ \theta = 1$, we'll have $\boldsymbol{f_{bu} = 14.16\,MPA.}$

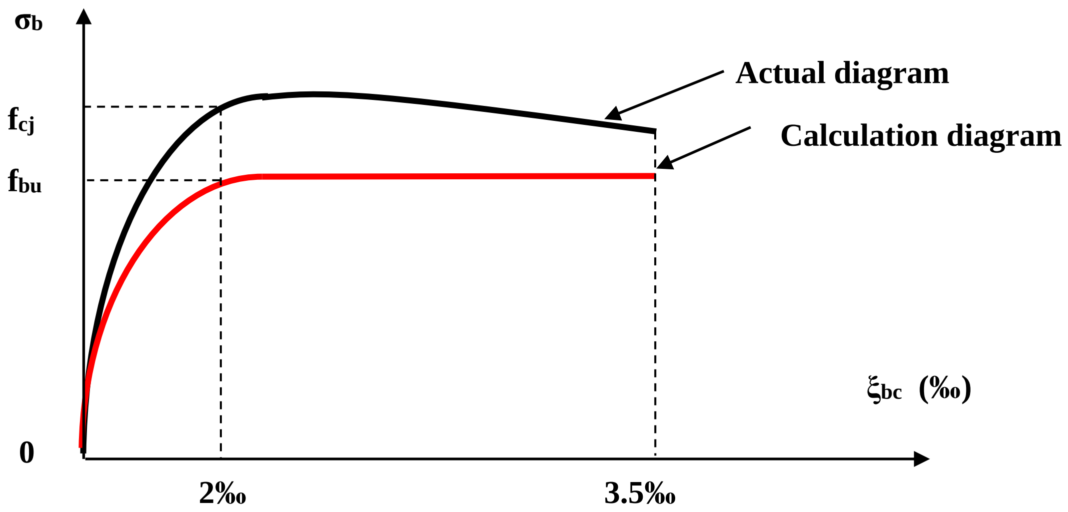


Figure III.5: Stress-strain diagram of concrete.

- **Restraint Limits at Service Limit State (SLS) :(BAEL91.Art A-4.5.2) [20]**

  The service limit state is a loading state beyond which the construction can no longer ensure the comfort and durability for which it was designed:

  - The serviceability limit state with respect to concrete compression
  - Crack opening serviceability limit state.
  - Deformation serviceability limit state.

La contraint limite de service est donnée par : $\overline{\sigma_{bc}} = 0.6 f_{c28} = 15\ MPA \quad with\ \overline{\sigma_{bc}} \leq \sigma_{bc}$

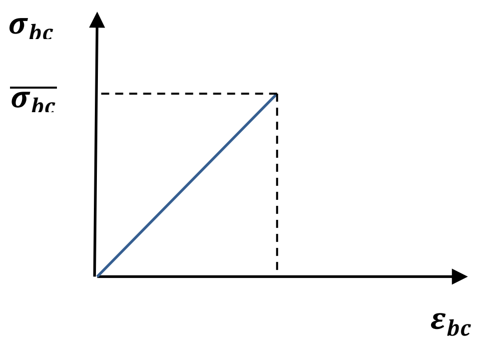


Figure III.6: Stress-strain diagram at ULS

- **Ultimate concrete shear stress: (BAEL91.A.5.1.2) [20]**

The ultimate shear stress for straight reinforcement (a=90°) is given by the following expressions:

$$\tau_u = \frac{V_u}{bd} \le \bar{\tau}_u$$

$$\bar{\tau}_u = \text{Min}\left(\frac{0.20 \text{ x} f_{c28}}{\gamma_b} \;;\; 5 \text{ MPA}\right) \text{If: cracking is non} - \text{detrimental.}$$

$$\bar{\tau}_u = \text{Min}\left(\frac{0.15 \text{ x} f_{c28}}{\gamma_b} \;;\; 4 \text{ MPA}\right) \text{If: cracking is detrimental or very detrimental.}$$

III.4.4 Steel:

The steel material is an alloy (Iron + Carbon) in low percentage, steel is a material characterized by a good resistance both in traction and in compression. So the solution to the problem of non-tensile strength of concrete is to integrate steel reinforcements into the concrete pieces to take up the tensile forces.

The steels used to make the reinforced concrete parts are:

- High adhesion and high elastic limit (HA) steels of grade FeE400:

According to (Art.7.2.2 of RPA99) the longitudinal reinforcement of the main elements must be of high adhesion with fe ≤ 500MPa and the relative elongation under specific maximum loads must be greater than or equal to 5%. We take for the longitudinal reinforcement Bars with high adhesions (HA): **FeE400.**

- Welded mesh (TS) of FeE500 grade and elongation at break of 8%: They are used as reinforcements in compression slabs.

a) Modulus of elasticity of longitudinal steels: (Art A-2.2.1 BAEL91) [20]

Steels are also characterized by the modulus of longitudinal elasticity. Experiments have shown that its value is fixed whatever the grade of steel. Es =2.105 MPA.

b) Calculation limit stresses:

❖ **Limit stresses at the ultimate limit state (ULS):**

We adopt the following stress-strain diagram:

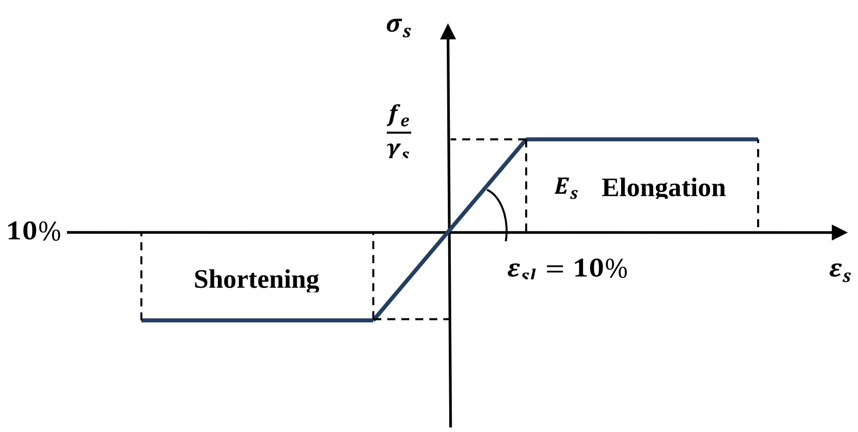


Figure III.7: Stress-strain diagram of steel

$\overline{\sigma}_s = \frac{f_e}{\gamma_s}$ For $\varepsilon_{se} \leq \varepsilon_s \leq 10‰$

$\overline{\sigma}_s = E_s \times \varepsilon_s$ For $\varepsilon_s \leq \varepsilon_{se}$ $\rightarrow$ with $\varepsilon_s = \frac{f_e/\gamma_s}{E_s}$

Such as: $\begin{cases} \gamma_s = 1.15 & \text{for a sustainable situation} \\ \gamma_s = 1 & \text{for an accidental situation} \\ \varepsilon_s = & \text{Relative elongation} \\ E_s = 2 \times 10^5 \text{ MPA} \end{cases}$

For our case: $\overline{\sigma}_s = 348MPa$ $\rightarrow$ for a permanent situation.
$\overline{\sigma}_s = 400\text{MPa}$ $\rightarrow$ accidental situation.

- **Limit stresses at service limit state (SLS):**

The limitation of crack openings is a function of the stresses in the reinforcements; therefore, the value of σ_s is given according to the type of cracking.

- **Minimally harmful cracking: (BAEL91Art A-4.5.32) [20]**

The element located in a closed and covered place therefore no limitation of the stress

"σs = fe"

- **Detrimental cracking: (BAEL91Art A-4.5.33) [20]**

If the element is exposed to bad weather, the stress is limited to:

$$\sigma_{st} \leq \overline{\sigma_{st}} = min(\frac{2}{3}f_e, 110\sqrt{\eta f_{tj}})$$

- **Very detrimental cracking: (BAEL91Art A-4.5.34) [20]**

If the element is exposed to an aggressive environment, the constraint is limited to:

$$\sigma_{st} \leq \overline{\sigma_{st}} = \min(\frac{1}{2}f_e, 90\sqrt{\eta f_{tj}})$$

With:

η: Coefficient of cracking which depends on the type of steel.

- $\eta = 1$ for smooth round steels.
- $\eta = 1.6$ for high adhesion steels.

c) Sealing factor: (BAEL91.Art A-6.1.21) [20]

- $\Psi s = 1$ For smooth round steels.
- $\Psi s = 1.5$ For high adhesion steels.

d) The equivalence coefficient: (C.B.A.93.art.A.4.5.1) [21]

The equivalence coefficient noted "n" with:

$$n = \frac{E_s}{E_b} = 15 \begin{cases} n: \text{Equivalence Coefficient} \\ E_s: \text{Modulus of deformation of steel} \\ E_b: \text{Modulus of deformation of concrete} \end{cases}$$

III.5 Actions and solicitations:

III.5.1 Actions: (BAEL.91. Art. A.3.1) [20]

The actions are all the loads (forces, torque, etc.) applied to the structure, as well as the consequences of static or state modifications (shrinkage, temperature variation, settlement of the supports) which lead to deformations of the structure. There are three categories of actions:

a. Permanent Actions (G): (BAEL.91. Art. A.3.1, 2) [20]

They have a constant or very little variable intensity over time; they include:

- The self-weight of the structure.
- Forces due to earth or liquids whose levels very little.
- Fixed equipment charges.
- The force due to the permanent deformations imposed on the structure (shrinkage, creep, etc.).

b. Variable actions (Q): (BAEL.91. Art. A.3.1, 3) [20]

Their intensity varies frequently and significantly over time. We distinguish:

- Operating expenses.
- Climatic actions (defined by the Snow and Wind rules).
- Actions due to temperature.

c. Accidental actions (Fa):

These are actions from phenomena occur and with a short duration of application such as earthquakes, shocks, explosions…..

III.5.2 Solicitations:

The stresses are the efforts caused, at each point and on each section of the structure, by the actions which are exerted on it, the stresses are expressed in the form of forces, efforts (normal and cutting forces) moments (bending or twisting).

- **Calculation stresses with respect to the ultimate limit states of strength and shape stability:**

a. Fundamental Combination: (C.B.A.93.art.A.3.3.2.1) [21]

In a lasting or transient situation (as opposed to so-called accidental situations), there is no need to consider the fundamental combination:

$$1.35G_{max} + G_{min} + \gamma_{QI}Q_1 + \sum 1.3\Psi_{0i}Q_i$$

With:

- G_{max} : unfavorable permanent actions.
- G_{min}: favorable permanent actions.
- Q_1 : Basic variable share.
- Q_i : Other so-called accompanying actions (with i >1).
- Ψ_0 : Combination value coefficient.

b. Accidental Combination: (C.B.A.93.art. A.3.3.2.2) [21]

If it is not defined by specific texts, the combination of actions to consider is:

$$G_{max} + G_{min} + Fa + \Psi_{1i}Q_i + \sum \Psi_{2i}Q_i$$

With:

- Fa: Nominal value of accidental action.
- $\Psi_{1i}Q_i$: Frequency of a variable action.
- $\Psi_{2i}Q_i$: Quasi-permanent value of another variable action.

c. Calculation stress with respect to serviceability limit states: (C.B.A.93.Art.A.3.3.3) [21]

They result from the following action combination called a rare combination:

$$G_{max} + G_{min} + Q1 + \sum \Psi_{0i}Q_i$$

III.6 Calculation assumptions:

III.6.1 ultimate limit state of strength (ULS): (B.A.E.L.Art. A4.5, 1) [20]

- Straight sections remain flat after deformations.
- There is no relative slip between the reinforcement and the concrete.
- Stress-strain diagrams have become for:

- Concrete in compression.
- Steel in tension and compression.

- Tensile strength of concrete is neglected.
- The positions that the deformation diagram of a cross section can take pass through at least one of the three pivots.
- We can assume that the section of a group of several stretched or compressed bars is concentrated in its center of gravity.

III.6.2 Ultimate serviceability limit state (SLS): (B.A.E.L Art. A.4.5, 2) [20]

- Straight sections remain flat.
- There is no relative slip between the reinforcement and the concrete.
- Steel and concrete are considered linear elastic materials. By convention, the ratio between the coefficients of longitudinal elasticity of steel and or coefficient of equivalence, is equal to: $n = \frac{E_s}{E_b}$
- It is assumed that the steel section is concentrated in its center of gravity.

III.7 Pre-dimensioning of floors:

The floor is a generally flat platform, which serves to separate between two levels that transmit the loads and overloads, directly applied to it, to the load-bearing elements while ensuring comfort functions such as sound, heat and air isolation. Tightness of extreme levels. For our building, two types of floors will be used:

- Hollow block slab
- concrete cantilever slab

Hollow block slab consists of:

1- Hollow block (slabs): whose role is filling, it has no resistance function.
2- Joists: resistant floor elements.
3- Compression slab: it is a reinforced concrete slab; its height varies from 4 to 6 cm.
4- Welded mesh resting on reinforced concrete joists placed in the direction of the smallest dimensions.

The thickness of the floors is chosen in such a way as to satisfy the conditions of use more than the conditions of resistance. To find this thickness, we will check the following conditions:

III.7.1 Hollow block slab:

III.7.1.1. Fire Resistance Condition: [24]

The fire resistance requirements according to [24] are:

$e = 07cm$: For one (01) hour of fire break.

$e = 11cm$: For two (02) hour of fire break.

$e = 17.5cm$: For four (04) hour of fire break.

For our structure we take e = 15 cm.

III.7.1.2. acoustic Conditions:

According to the OMS In order to allow a conversation in comfortable conditions indoors during the day, the level of interfering noise should not exceed 35 dBA The isolations required are generally of the order of 25 to 40 dB(A) following the cases [23]. Such values would be easy to obtain if the facade is continuous, i.e. without glazing (case of our floor). In this case, it would suffice to use a surface mass of the order of 200 kg/m2 according to DTR C3.1.1. [25]. to ensure minimum sound isolation, the thickness of our floor (hollow body) must be greater than or equal to 15 cm (the surface mass rule) DTRC.3.1.1 Annex III [25];

We adopt a thickness of: e=16cm.

1. **Deflection resistance condition: CBA93 (article B.6.8.4.2.4) [21] .**

The height of the floor is determined from the stiffness condition given by the CBA93 as follows:

$$h_t \geq \frac{L_{max}}{22.5}$$

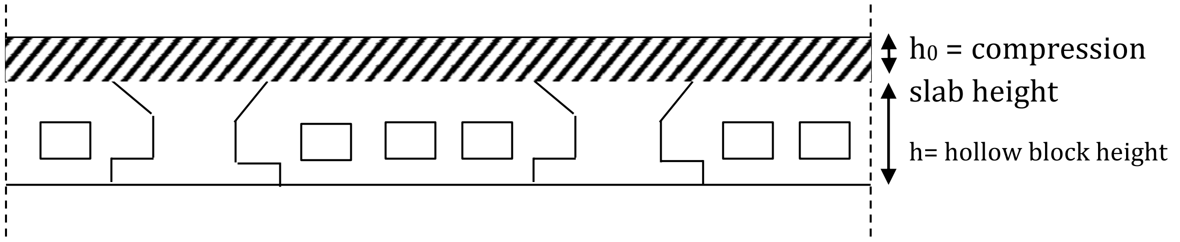


Figure III.8: Vertical section of a hollow block floor.

With:

L_{max}:.Maximum length between columns supports according to the disposition of the beams adopted.

h_t: Total floor height.

b: distance between rib axes.

- **Ribs :**

These are small reinforced concrete beams forming the framework of a floor; they are calculated for simple bending (united with the compression slab).

- **Position of ribs:**

The disposition of the joists is done according to two criteria:

- Criteria of the small range.
- Criteria of continuity (the sense that there are more supports).

For our project, the disposition is carried out according to the first criterion and this for all the floors as shown in the figure below

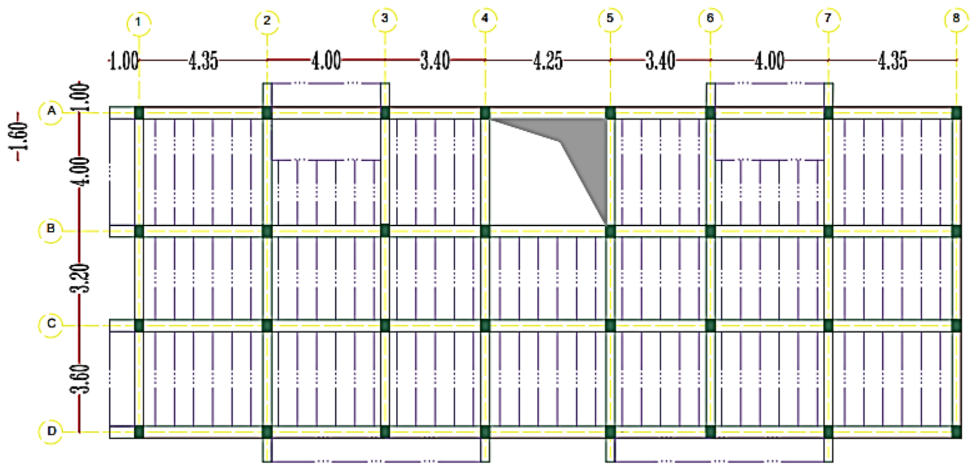


Figure III.9: Disposition of joists

According to the arrangement of the beams chosen: $L_{max} = 360\ cm \Rightarrow \mathrm{h_t} \geq \frac{360}{22.5} = 16cm$

So, we adopt for a floor (16+4) $\begin{cases} h_{hollow\ block} = 16\ cm \\ h_{compression\ slab} = 4cm \\ h_t = 20\ cm \end{cases}$

- **Dimensioning of joists:**

The joists are reinforced concrete elements, which ensure the transmission of the loads to the beams. They are characterized by their small section and are calculated as sections in T.

Joist Width Calculation [24]:

$0.3h_t \leq b_0 \leq 0.6h_t$ With $h_t = 20cm$

$6cm \leq b_0 \leq 12$ cm

For construction reasons, we take $b_0 = 10cm$

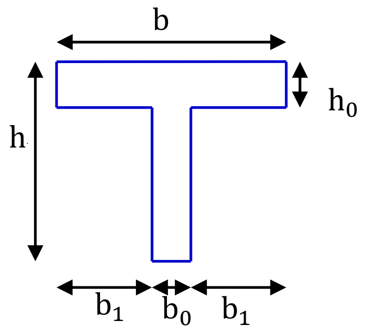


Figure III.10: schema of section in T.

Calculation of the width of a leg of the T-section:

According to CBA 93 (Art A. 4.1.3), the effective width b is determined as follows:

$$\left(\frac{b-b_0}{2}\right) \leq \min\left(\frac{l_x}{2}, \frac{L_{ymin}}{10}\right)$$

b: Effective width.

b_0: joist width (b_0 = 10cm).

L_x: The distance between the neighboring faces of two consecutive joists (Distance between bare of two beams):L_x = 65 – b_0 = 55 cm.

L_y: Distance between bare supports of the minimum span of the joists:$L_y = 280cm$

With: $\frac{b-10}{2} \leq \min\left(\frac{l_x}{2}, \frac{l_y}{10}\right) \Rightarrow \frac{b-10}{2} = \min(27.5{,}28) \Rightarrow b = 65$

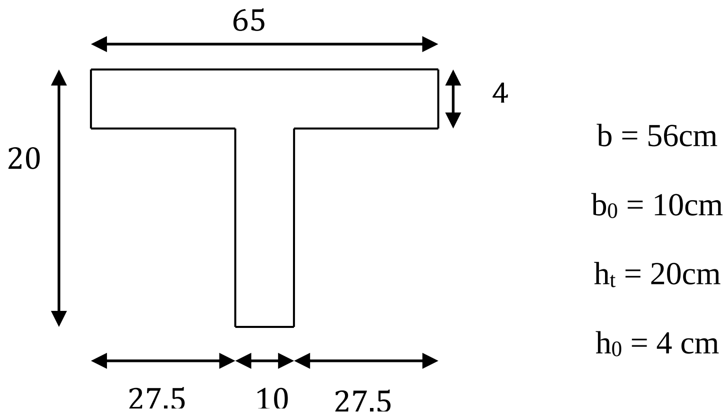


Figure.III.11: T-section dimensions.

III.7.2.Pre-dimensioning of solid slabs:

The slabs are horizontal elements of thin thickness in reinforced concrete cast in place. They are characterized by their small range Lx and Ly the large range. The dimensioning of the thickness "e" of this type of floor depends on the following criteria:

III.7.2.1. Fire resistance criteria [24]:

$\boldsymbol{e \geq 7cm}$ For one hour of fire break.

$\boldsymbol{e \geq 11cm}$ For two hours of fire break.

$\boldsymbol{e \geq 17.5cm}$ For four hours of fire break.

We take a thickness equivalent to more than 2 hours of firebreak so we take: $\boldsymbol{e = 15cm}$

III.7.2.2. Resistance to bending criteria [24]:

The conditions that must be satisfied according to the number of supports are the following:

- Concrete cantilever slab bearing in two directions ($\frac{Lx}{Ly} > 0.4$):

Isostatic span: $\frac{L_x}{40} \leq e \leq \frac{L_x}{25}$

Hyper static span: $\frac{L_x}{50} \leq e \leq \frac{L_x}{35}$

- Concrete cantilever slab bearing in one direction ($\frac{Lx}{Ly} < 0.4$):

Isostatic span: $\frac{l_x}{30} \leq e \leq \frac{l_x}{20}$

Hyper static span: $\frac{L_x}{35} \leq e \leq \frac{L_x}{25}$

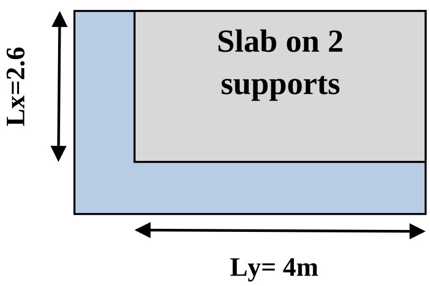
L_x Being the smallest span of the most stressed panel (worst case)

Case°1: slabs resting on 2 supports with the dimensions $\begin{cases} Lx = 2.6\ m \\ Ly = 4\ m \end{cases}$

So $\alpha = \frac{Lx}{Ly} = 0.65 < 0.4$

⇒ The isostatic panel bears in two direction

$$\frac{L_x}{40} \leq e \leq \frac{L_x}{25} = \frac{260}{40} \leq e \leq \frac{260}{25} = 6.5 \leq e \leq 17.33$$



FigureIII.12: Slab on 2 supports.

Finally, and according to the above conditions we take:

e = 15 cm.

III.7.3. Pre-dimensioning of balconies:

The balcony consists of a concrete solid slab recessed at one end and free at the other.

- **Calculation of thickness e:**

The thickness is conditioned by:

$$e > \frac{l}{20}$$

l: Balcony width.

We take the unfavorable value of “l”.

So: $e > \frac{l}{20} = \frac{100}{20} = 5cm$ $e > 5cm$ we adopt: e = 15cm

III.8 Pre-dimensioning of stairs:

The staircase is the part of the structure that serves to ensure the connection between the different levels of a construction; it is determined by its rise, its

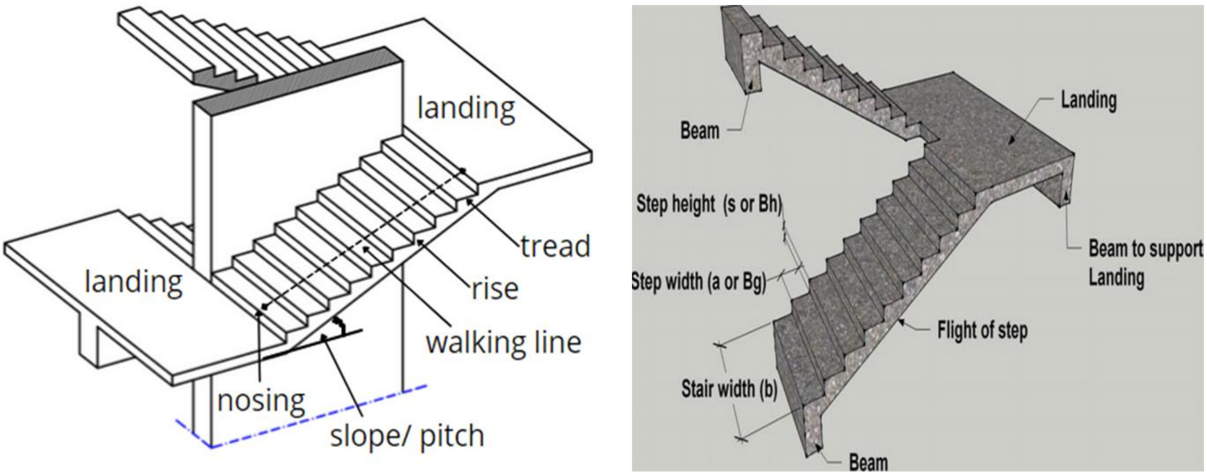


Figure III.13: Represents the schema of a stairs.

The choice of dimensions depends on the conditions of use and the intended destination of the staircase:

To ensure a pleasant staircase or maximum accessibility for the public, the module is adjusted according to the available setback and can be between 59 and 66 cm. The relationship between g and h is given by Blondel's relationship:

$$59 < 2\,\mathrm{h} + g < 66$$

Where:

h: step height(riser)

g: step width (tread).

For residential buildings, the ideal rise height would be 17 cm for a tread between 28 and 30cm.

We set the height of the riser h to 17 cm and the tread g = 30cm.

$$59 \leq 2\mathrm{h} + \mathrm{g} \leq 66 \quad \Rightarrow \quad 2 \times 17 + 30 = 64\ \mathrm{cm} \ldots\ldots\ldots\ldots\ (\mathrm{CV})$$

For the dimensioning of a staircase:

- **Number of risers**:

$$n = \frac{H}{h} = \frac{306}{170} = 18\ risers$$

- **Stringer**:
- $n = \frac{145}{17} = 9\ risers$

Determination of stringer thickness:

$$\tan\alpha = \frac{h}{g} = \frac{17}{30} = 0.57 \Longrightarrow \alpha = 29.68°$$

Riser length:

$$l = \frac{2.40}{\cos\alpha} = 2.79\ m$$

- **Evaluation of the thickness of the stringer and the bearing:**

Stringer thickness and landing is determined as follows: $\frac{\mathrm{L}}{30} \leq \mathrm{e} \leq \frac{\mathrm{L}}{20}$

$L = 2.79\ m \Longrightarrow 9.3\mathrm{cm} \leq \mathrm{e} \leq 13.95\ \mathrm{cm}$ We take: e = 13cm.

- **Landing thickness:**

For the landing, we adopt a thickness of 15cm.

- **Stairs quick release condition:**

$1.2\ \mathrm{m} \leq \mathrm{curtail} \leq 1.5$ For a collective building

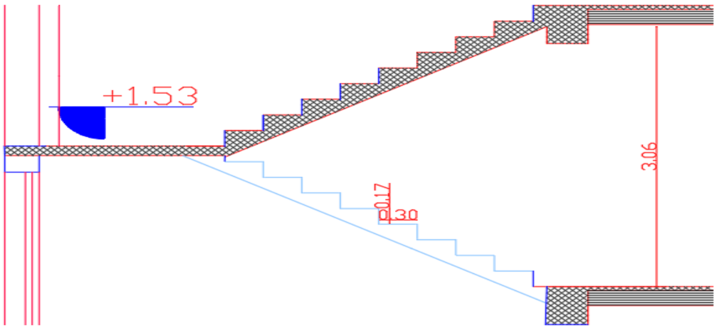


Figure III.14: Represents of stair dimensions.

III.9 Pre-dimensioning of the beams:

The beams are reinforced concrete elements of section (b×h) which ensure the transmission of loads and overloads from the floors to the vertical elements (columns, shear walls). Per-dimensioning will be done according to BAEL91 modified 99 and verifications according to RPA99/V2003.

III.9.1 Primary beams:

They are arranged perpendicular to the joists; their height is given according to the condition of the deflection which is:

$$\begin{cases} \frac{L_{max}}{15} \leq h \leq \frac{L_{max}}{10} \\ 0.3h \leq b \leq 0.7h \end{cases}$$

L_{max} : The maximum length between columns. **[20]**

h: beam height

b: beam width

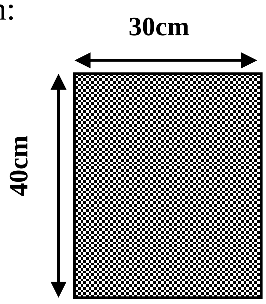
In our case, the most stressed beam is of length between columns: $L_{max} = 4.05m$

So: $\frac{405}{15} \leq h \leq \frac{405}{10} \rightarrow 27 \leq h \leq 40.5 \rightarrow h = 40cm$

So: $0.3 \times 40 \leq b \leq 0.7 \times 40 \rightarrow 12 \leq b \leq 28 \rightarrow b = 30cm$

Verification against RPA99 version 2003 (RPA Article 7.5.1) **[25]** is then carried out to satisfy the minimum values required for the dimensions of the beam:

$$\begin{cases} b \geq 20cm \rightarrow b = 30 \;.... cv \\ h \geq 30cm \rightarrow h = 40 \;... cv \\ \frac{h}{b} \leq 4 \rightarrow \frac{h}{b} = \frac{40}{30} = 1.33 < 4 \;... cv \end{cases}$$

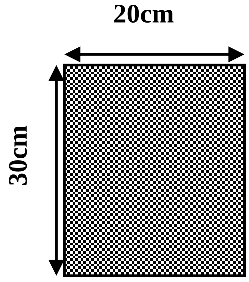


The condition is checked; therefore, we adopt

for the principal beams a section of:

$$\boldsymbol{b \times h} = (\mathbf{30\ cm} \times \mathbf{40cm})$$

III.9.2: Secondary beam:



They are arranged parallel to the joists; their height is given by:

$$\frac{L_{max}}{15} \leq h \leq \frac{L_{max}}{10}$$

In our case, the most stressed secondary beam is of length: L max = 360cm.

$$\begin{Bmatrix} \frac{L_{max}}{15} \leq h \leq \frac{L_{max}}{10} \\ 0.3h \leq b \leq 0.7h \end{Bmatrix} \to \begin{Bmatrix} \frac{360}{15} \leq h \leq \frac{360}{10} \to 24cm \leq h \leq 36cm \\ 0.3 \times 30 \leq b \leq 0.7 \times 30 \ \to 9 \leq b \leq 21 \end{Bmatrix} \to \begin{Bmatrix} h = 30cm \\ b = 20cm \end{Bmatrix}$$

Verification against RPA99 version 2003 (RPA Article 7.5.1) **[17]** is then carried out to satisfy the minimum values required for the dimensions of the beam:

$$\begin{Bmatrix} b \geq 20cm \to b = 20cm \ \dots . cv \\ h \geq 30\ cm \ \to h = 30\ cm \ \dots . cv \\ \frac{h}{b} \leq 4 \ \to \frac{h}{b} = 1.5 \leq 4 \ \dots . cv \end{Bmatrix}$$

The condition is verified, so we adopt for the beams Secondaries a section of:

$$\boldsymbol{b \times h} = (\mathbf{20cm} \times \mathbf{30\ cm})$$

III.10 Pre-dimensioning of the columns:

The pre-dimensioning of the columns will be done according to the calculation stresses in centered compression according to the rules of BAEL91, by applying the following three criteria:

- Resistance criteria.
- Stability form criteria.
- RPA99 rules (2003 version).

RPA requirements in zone III **[17]**

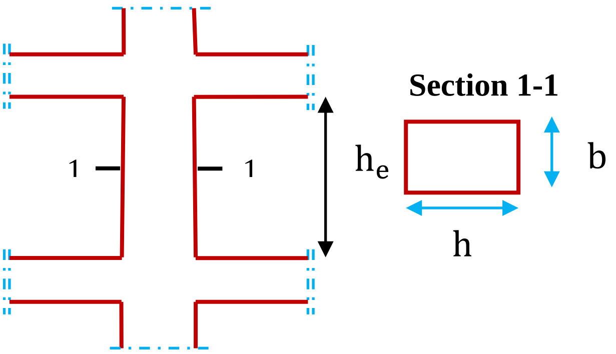


Figure III.15: Clear height of storey column

$$\begin{cases} min(b,h) \geq 30\ cm \\ \min(b,h) \geq \frac{h_e}{20} \\ \frac{1}{4} \leq \frac{b}{h} \leq 4 \end{cases}$$

$h_e = 3.06 - 0.4 = 2.66\ m$

The dimensions of the columns will be fixed after having carried out the descent of the load, while checking the recommendations of the RPA99 (2003 version) cited above. We first adopt the section of the columns as follows:

Floors	**Ground floor**	**1st and 2nd floor**	**3th and 4th floor**
Section (b*h) cm²	30×40	30×40	30×40

Table III.1: preliminary column section.

III.11 Pre-dimensioning of shear walls:

The shear walls are used on the one hand to brace the building by taking up the horizontal loads (earthquake, winds, etc.) and on the other hand to take up the vertical forces that they transmit to the foundations. The elements satisfying the condition (L≥ 4e) are considered as walls According to RPA99/V2003 (article 7.7.1) **[17]**. The dimensions of the sails must meet the following conditions:

$e \geq \max\left\{15cm\ ; \frac{h_e}{20}\right\}$

e = 11cm. For two (02) hours of fire break.

$e \geq \frac{h_e}{20} \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots (2).$

$L > 4\,e \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots (3).$

he = storey height -hp = storey height- 40cm

h_p : Total height of the beam.

h: Clearance floor height.

e : shear wall thickness.

L : shear wall length.

$e \geq \max\left(\frac{266}{20}, 15\right) \geq 15\ cm$

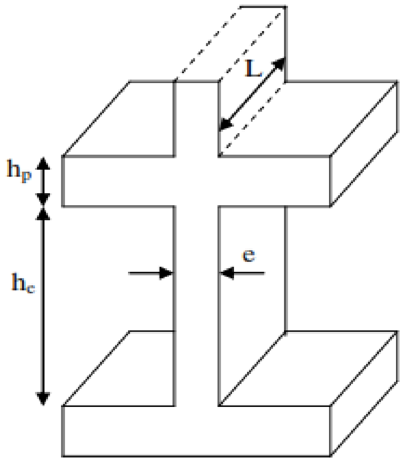


Figure III.16: shear wall section in elevation.

So: e = 20 cm. For the ground floor and common floors

For shear wall thick (e=20 cm) the length must be greater than or equal to 80 cm (L > 4th)

III.12Evaluation of loads and overloads:

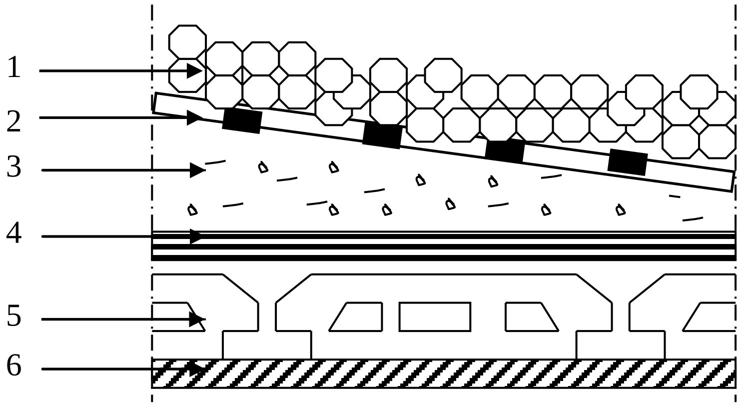
Evaluation of loads and overloads aims to determine the loads and overloads due to each load-bearing element at the level of each floor.

III.12.1 Floors:

III.12.1.1. Hollow block slab terrace inaccessible:

Protective gravel.

1. Multi-layer waterproofing.
2. Slope concrete.
3. Thermal insulation.
4. Hollow block slab.
5. Plaster coating.



FigureIII.17: Hollow block slab de terrace.

- **Permanent and operating loads (DTRB.C2.2)[22]:**

N°	Designation	e(m)	$\gamma(N/m^3)$	Load(N/m^2)
1	Protective gravel	0.05	17000	850
2	Multi-layer waterproofing	0.02	6000	120
3	Slope concrete	0.1	22000	2200
4	Thermal insulation	0.04	4000	160
5	Hollow block slab	0.2	14000	2800
6	Plaster coating	0.02	10000	200
Total permanent load				G_t =6330
Operating overload				Q_t =1000

Table III.2: Permanent loads from a floor inaccessible terrace story

III.12.1.2. Hollow block floor common story:

1. Floor tile
2. Bedding mortar
3. Sand bed
4. Hollow block floor
5. Plaster coating

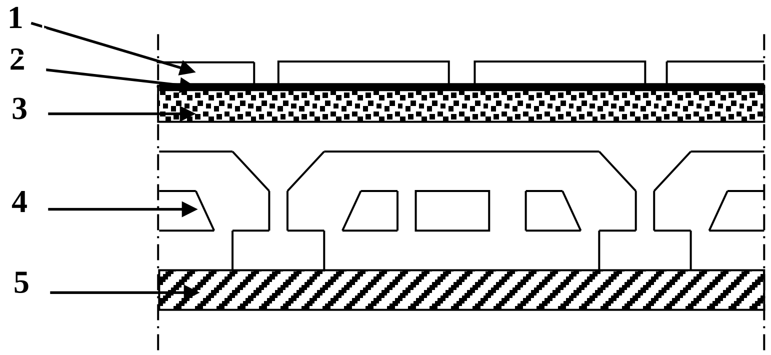


Figure III.18: Hollow block floor common story

- **Permanent and operating loads (DTRB.C2.2) [22]:**

N°	Designation	e (m)	$\gamma\ (N/m^3)$	Load (N/m^2)
1	Brick partition	0.1	9000	900
2	Floor tile	0.02	20000	400
3	Bedding mortar	0.02	20000	400
4	Sand bed	0.03	18000	540
5	Hollow block floor	0.2	14000	2800
6	Plaster coating	0.02	10000	200
Total permanent load				G_E =5240
Operative overload				Q_E =1500

Table III.3: Permanent and operating loads of hollow body common story

a. Balcony:

1. Floor tile.
2. Bedding mortar.
3. Sand bed.
4. Solid slab.
5. Plaster coating.

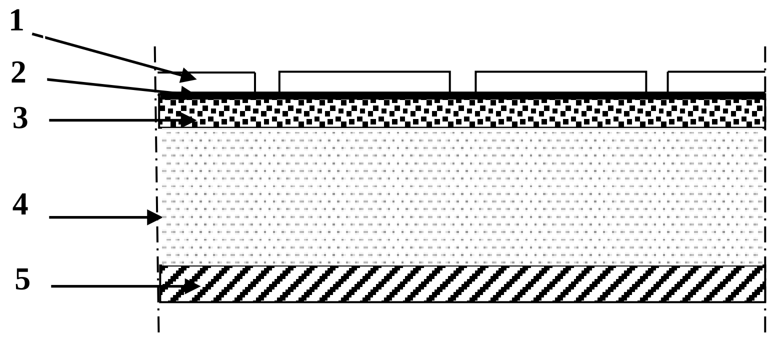


Figure III.19: Schema of a balcony

N°	Designation	e (m)	γ (N/m^3)	Load (N/m^3)
1	Floor tile	0.02	20000	400
2	Bedding mortar	0.02	20000	400
3	Sand bed	0.03	18000	540
4	Solid slab	0.15	25000	3750
5	Plaster coating	0.02	10000	200
Total permanent load				G_b =5290
Operative overload				Q_b =3500

Table III.4: The permanent and operating loads of a balcony

III.12.2 exterior wall:

N°	Designation	e (m)	γ (N/m^3)	Load
1	Cement rendering	0.02	18000	360
2	hollow brick of 15	0.15	9000	1350
3	Air gap	0.05	-	-
4	hollow brick of 10	0.1	9000	900
5	Plaster coating	0.02	10000	200
Total permanent load				G_E =2810

Table III.5: Permanent loads of exterior partitions

III.12.3 Stairs:

III.12.3.1. Masonry slab:

N°	Designation		e (m)	γ (N/m^3)	Load (N/m^2)
1	Floor tile	Horizontal	0,02	22000	440
		Vertical	0,02h/g	22000	249.33
2	Bedding mortar	Horizontal	0,02	20000	400
		Vertical	0,02h/g	20000	226.67
3	Weight of steps		0,17/2	25000	2125
4	Weight of masonry slab		0,13/cosα	25000	3735.21
5	Plaster coating		0,02/cosα	10000	200
6	Guard or safety rail		-	-	600
Total permanent load					G_p=7976.21
Operative overload					Q_p = 2500

Table III.6: Permanent loads of a stairs

III.12.3.2. Intermediate landing:

N°	Designation	e (m)	$\gamma\ (N/m^3)$	Load (N/m^2)
1	floor tile	0.02	22000	440
2	bedding mortar	0.02	20000	400
3	sand bed	0.02	18000	360
4	solid slab	0.15	25000	3750
5	plaster coating	0.02	10000	200
Total permanent load				$G_{pr} = 5150$
Operative overload				$Q_{pr} = 2500$

Table III.7: Permanent and operating loads of a landing

III.12.4 Parapet:

- **Permanent load:**

Parapet is subject to a permanent load due to its own weight:

$S = s_1 + s_2 + s_3$

$S = (0.6 \times 0.1) + (0.1 \times 0.2) + \left(\frac{0.1\times0.2}{2}\right) = 0.09\ m^2$

$G_{acro} = \rho \times S = 25 \times 0.09 = 2.25\ KN/ml$

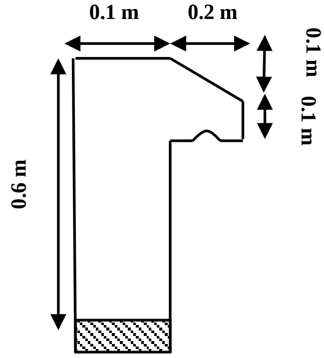


Figure III.20 : Schema of the parapet

With:

G: Own weight of the parapet in linear meter.

ρ: Density of concrete.

Designation	h(m)	Surface (m²)	Total own weight (KN/ml)	Operating overload (KN/ml)
Parapet	0.6	0.09	2.25	1

Table III.8: Permanent and operating loads of a parapet.

III.13 LOWERING LOADS:

The objective is to know the distribution and paths of the loads on all the supporting elements of the structure from the top to the foundations. In order to ensure the stability of the structure, it will be necessary to consider:

- The self-weight of the element.

- The floor load it supports.
- Secondary elements (staircase, parapet, etc.).
- The share of the distributed partition of the structure.

The lowering of the load is done from the highest level (framework or roof terrace) to the lower level and that to the lowest level (the foundations). We apply the digression laws only for residential floors.

III.13.1 Digression low (DTRB.C 2.2) [22]:

Let q0 be the operating load on the roof or the terrace covering the building Q1, Q2, Q3 or the respective operating loads of the floors of stories 1, 2, 3…n numbered from the top of the building. The following operating loads will be adopted for the calculation of the support points:

Under roof or terrace … … … … … … … … … … … Q_0

Under top floor (floor 1)… … … … … … … … … … $Q_0 + Q_1$

Under floor immediately below

(Floor 2) $Q_0 + 0.95\,(Q_1 + Q_2)$

(Floor 3) $Q_0 + 0.90\,(Q_1 + Q_2 + Q_3)$

(Floor 4) $Q_0 + 0.85\,(Q_1 + Q_2 + Q_3 + Q_4)$

(Floor n) $Q_0 + \frac{3+n}{2n}\,(Q_1 + Q_2 + Q_3 + Q_4 + \cdots Q_n)$

Taking into account the degression of overloads as follows:

$\sum Q = Q_0 = 1000\ \text{N}$

$\sum Q = Q_0 + Q_1 = 2500\ \text{N}$

$\sum Q = Q_0 + 0.95\,(Q_1 + Q_2) = 3850\text{N}$

$\sum Q = Q_0 + 0.90\,(Q_1 + Q_2 + Q_3) = 5050\ \text{N}$

$\sum Q = Q_0 + 0.85\,(Q_1 + Q_2 + Q_3 + Q_4) = 6100$

For the verification of their section in this case we take the following type of column:

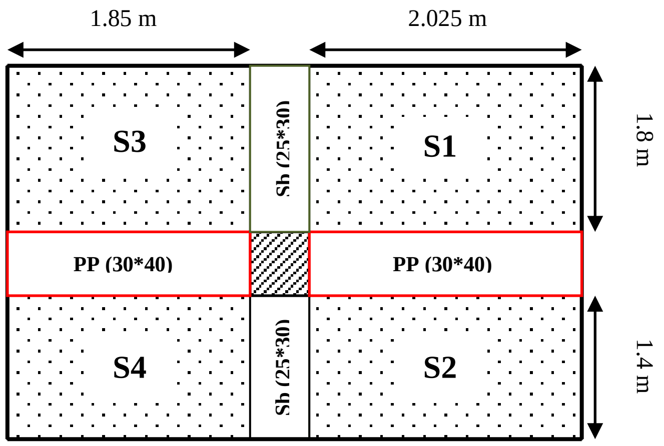


Figure III.21: Most stressed column diagram (adherent surface)

- Intermediate column : (the afferent surface)

$$A_1 = (2.025 \times 1.8) = 3.64 m^2$$

$$A_2 = (2.025 \times 1.4) = 2.83 m^2$$

$$A_3 = (1.85 \times 1.8) = 3.33 m^2$$

$$A_4 = (1.85 \times 1.4) = 2.59 m^2$$

So: $A_t = 12.39\ m^2$

- Beams :

$$\begin{cases} G_{mb} = 25 \times 2.025 \times 0.30 \times 0.40 = 6.075 KN \\ G_{mb} = 25 \times 1.85 \times 0.30 \times 0.40 = 5.55\ KN \\ G_{sb} = 25 \times 1.8 \times 0.20 \times 0.30 = 2.7\ KN \\ G_{sb} = 25 \times 1.4 \times 0.20 \times 0.30 = 2.1\ KN \end{cases}$$

So: $G_{tot.b} = 16.425\ KN$

Levels	Column Section (cm²)	The column area (m²)	Afferent surface
Ground $1^{st}2^{nd}3^{rd}4^{th}$	30×40	0.12	12.39

Table III.9: The different column sections

Floors	Levels	Elements	Self weight G(KN)	Operating load Q(KN)
4^{th}floor	N0	Terrace floor	$6.33 \times 12.3 = 78.42$	$1 \times 12.39 = 12.39$
		Beams	16.425	
	sum		94.845	12.39
3^{rd}floor	N1	Coming N0	94.845	$1.5 \times 12.39 + 12.39$
		Column	$0.4 \times 0.3 \times 3.06 \times 25 = 9.18$	
		Floor stories	$5.24 \times 12.39 = 64.92$	
		Beams	16.425	
	Sum		185.37	30.975
2^{nd}floor	N2	Coming N1	185.37	49.56
		Column	9.18	
		Floor stories	64.92	
		Beams	16.425	
	Sum		275.895	49.56
1^{st}floor	N3	Coming N3	275.895	68.145
		Column	9.18	
		Floor stories	64.92	
		Beams	16.425	
	Sum		366.42	68.145
RDC	N4	Coming N4	366.42	86.73
		Column	9.18	
		Floor stories	64.92	
		Beams	16.425	
	Sum		456.945	86.73
in ultimate limit states			1.35 G = 616.875	1.5Q=130.095
Normal force at the base of the column			$N_u = 746.97$ KN	

Table III.10: Load descent of column

❖ **Verification:**

According to the CBA93 (article B.8.11) [21], for the central columns in the case of a building with two spans, the ultimate compressive force Nu must be increased by 10% such that:

$$N_u = 1.10\,(1.35G + 1.5Q)$$

- The maximum normal effort: $N_u = 1.10 \text{ x } 746.97 = 821.667$ KN

So: $N_u = 821.667\ KN$ for the column(30×40).

The ultimate acting normal force N_u Bare of a column must be at most equal to the following value:

$$Nu \leq \overline{N} = \alpha\left[\frac{B_r \times f_{c28}}{0.9 \times \gamma_b} + A\,\frac{f_e}{\gamma_s}\right]$$

α : is a coefficient function of the mechanical slenderness λ.

$\lambda = \max(\lambda_x;\ \lambda_y)$; With:

$$\lambda_x = \sqrt{12} \times \frac{lf}{b} \quad ; \ \lambda_y = \sqrt{12} \times \frac{lf}{h}$$

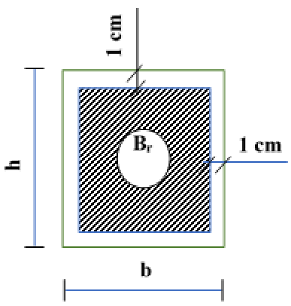
$$l_f = 0.7 \times l_0 \quad (\mathbf{BAEL91Art\ B.8.3.31}) \quad \Rightarrow l_f = 0.7 \times 3.06 = 2.14\text{m}$$

$$\lambda_x = \sqrt{12} \times \frac{2.14}{0.30} = 24.71 \quad ; \quad \lambda_y = \sqrt{12} \times \frac{2.14}{0.40} = 18.53$$

$$\lambda = 24.71 < 50 \ \Rightarrow selon\ (\mathbf{BAEL91Art\ B.8.4.1}).$$

$$\alpha = \frac{0.85}{1+0.2\left(\frac{\lambda}{35}\right)^2} = \frac{0.85}{1+0.2\left(\frac{24.71}{35}\right)^2} = 0.77$$

B_r: is the reduced cross-section of the column obtained by deducting from its real cross-section 1 cm thick over its entire peripheral.



$B_r = (h-2)\,(b-2)$

$\gamma_b = 1.50 \quad ; \quad \gamma_s = 1.15$

A: is the section of compressed steel considered in the calculation.

$A = \max(A_{BAEL}\,;A_{RPA})$

$$A_{min} = \max\left(\frac{4\text{cm}^2}{\text{m}}\text{perimeter};\ 0{,}2\%B\right) = \max(5.6\text{cm}^2\,;2.4\text{cm}^2) = 5.6\ \text{cm}^2$$

$$A_{min}^{RPA} = 0.9\%B = 10.8cm^2$$

column	N_u (KN)	α	A_{min}^{BAEL} (mm²)	A_{min}^{RPA} (mm²)	A (mm²)	B_r (mm²)	$\bar{N}$ (KN)	Condition
30×40	821.667	0.77	560	1080	1080	106400	1086.43	C.V

Table III.11: Results of buckling checks

III.14 conclusion:

In this 3rd chapter, we presented pre-registration of the project with all its characteristics; we gave the characteristics of the materials used as well as the codes and regulations in force.

The following Chapter will be the subject of the pre-dimensioning of all the structural and non-structural elements of our work.

The tables below sum up the different characteristics of the materials used for our calculations:

Resistance	At compression		25 MPa
	At traction		2.1 MPa
Allowable stress	ULS	Lasting situation	14.2 MPa
		Accidental situation	18.48 MPa
	SLS		15 MPa
Modulus of deformation	instantaneous		32164.195 MPa
	Deferred		10818.87 MPa

Table III.12: Mechanical properties of concrete.

Allowable stress	ULS	Lasting situation	348MPa
		Accidental situation	400 MPa
	SLS	Little harmful cracking	400 MPa
		Harmful cracking	201.63 MPa
		Very harmful cracking	164.97 MPa

Table III.13: Mechanical characteristics of steels.

Layers N°	**Depth (m)**	**Φ_{uu}(°)**	**C_{uu}(bars)**	**Density (t/m^3)**	**Sr (%)**	**W(%)**	**$\bar{\sigma}_s$ (bars)**
1	0.0-10.0	22	0.17	1.65	24%	6	2.3

Table III.14: Mechanical characteristics of the soil.

To close this chapter on the pre-dimensioning of the elements, we present below the dimensions adopted for the various elements making up the structure of our building:

a) Floor :

- hollow block floor 16+4 cm
- Solid slab floor 15 cm

b) Shear wall :

- shear wall thick e=20 cm

c) Beams :

- Main beams : b = 30 cm ; h = 40 cm
- Secondary beams : b = 20cm ; h = 30 cm

d) Columns :

- Columns RDC , 1^{st} ,2^{nd},3^{rd} and 4^{th} floor : $(30 \times 40)\ cm$

e) Stairs :

- e = 15 cm for Masonry slab.
- e = 15 cm for Intermediate landing.

Chapter IV:

Dynamic study

PART I: STRUCTURAL MODELLING AND SEISMIC STUDY.

IV.1. Introduction:

An earthquake is a sudden, violent shaking of the ground caused by the movement of tectonic plates beneath the Earth's surface. It is one of the most powerful and destructive natural disasters that can occur and cause significant damage to buildings, infrastructure and human life. A dynamic study of a building is conducted to assess the ability of the building to withstand seismic forces during an earthquake...A dynamic study of a building is crucial to ensure the safety of building occupants and the integrity of the structure during an earthquake. It helps to identify weak areas, design renovation measures and improve the seismic design of new buildings.

IV.2. Objectives and requirements:

According to the RPA99-2003 version section (3.4.A.1.a), any structure exceeding three levels of eleven meters (11 m) in height in zone IIb and III must contain shear walls.

With regard to the structure that is the subject of this present study, it is a building of ground+4 floors (15.30m) which exceeds 11m in zone III; it will therefore be braced by a bracing system proposed by the Algerian seismic code section (3.4.A) [17].

A seismic design must not only respect the regulations in force but also satisfy the architectural and economical aspect.

As part of our work, we attempted to analyze and study a ground+4 floors braced by a mixed shear wall-frame system using ETABS software version 20.1

IV.3. Structure modeling:

IV.3.1. used software:

The modeling of the structure is done in finite elements using the software ETABS 2022, which allows both static analysis and dynamic analysis (analysis of free vibrations, spectral modal analysis, temporal analysis).

Columns, beams and joists are modeled using finite elements “frame” type linear lines available in the software library (Column for columns and Beam for beams). Surface finite elements of type "membrane" are used to represent the compression slabs of the hollow body floors cavity whose role is only to distribute the gravity loads on the beams and the joists.

Floors are considered rigid in their plan (rigid horizontal diaphragms) by application of kinematic constraints using the "diaphragm" option available in the software. This consideration makes it possible to significantly reduce the number of degrees of freedom. The vertical loading is carried out using gravity loads (G and Q) in the form of surface loads (Shell load - Uniform load sets), and the seismic loading is obtained by applying a response spectrum in both directions (X and Y) to obtain respectively ($V_{xdyn}$and V_{ydyn}). The dynamic masses are evaluated using the relationship below prescribed in the Algerian seismic code (RPA 99 / Version 2003):

$W = G + \beta Q$

G: Permanent loads

Q: Operating loads.

β: weighting coefficient, depending on the nature and duration of the operating loads, equal to 0.2 in our case (table 4.5)

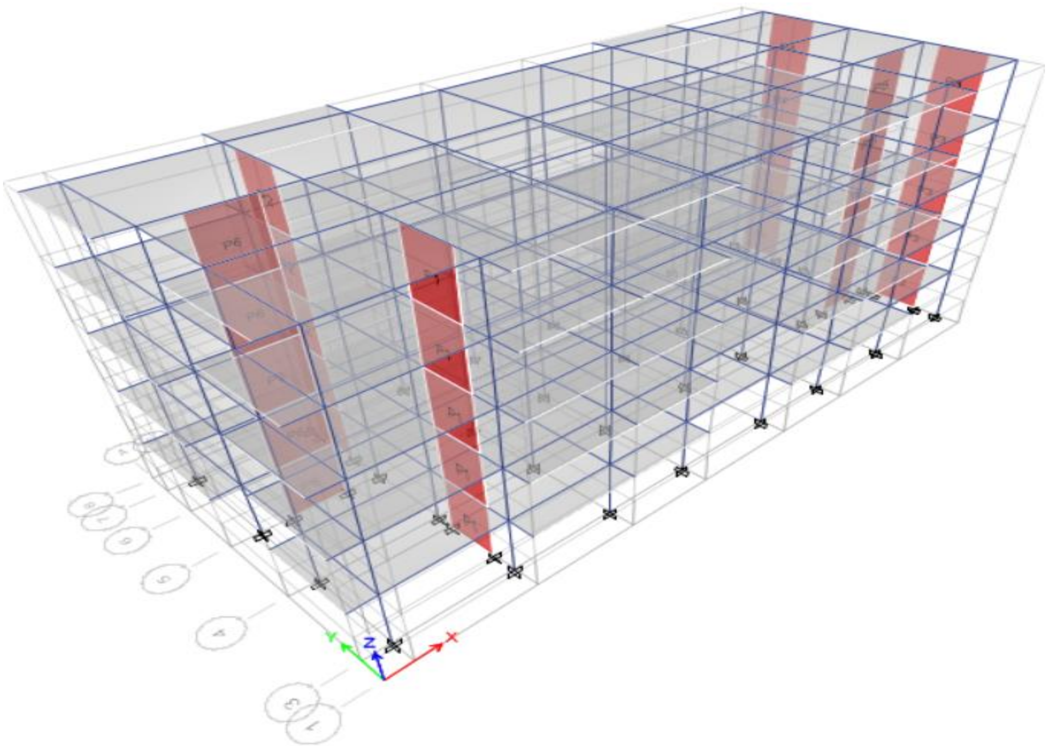


Figure IV.1: 3D view of the structure modelling with ETABS 2022 v20.1 software.

IV.3.2. Methods of calculation for seismic forces:

According to the Algerian seismic code (RPA99V2003), the calculation of global seismic forces can be done using two main methods:

- The equivalent static method.
- The dynamic method:
- The response spectrum analysis method.
- Time history analysis

For this project, we choose to work with the response spectrum method.

According to RPA99V2003 in this method; the maximum effects generated in the structure by seismic forces represented by a response spectrum are sought for each mode of vibration. These effects are then combined to obtain the response of the structure. (Article 4.3.3 P45)

The response spectrum is being given by the following expression:

$$\frac{S_a}{g} = \begin{cases} 1.25A\left(1+\frac{T}{T_1}\left(2.5\eta\frac{Q}{R}-1\right)\right) & 0 \leq T \leq T_1 \\ 2.5\eta(1.25A)\left(\frac{Q}{R}\right) & T_1 \leq T \leq T_2 \\ 2.5\eta(1.25A)\left(\frac{Q}{R}\right)\left(\frac{T_2}{T}\right)^{2/3} & T_2 \leq T \leq 3.0\ s \\ 2.5\eta(1.25A)\left(\frac{T_2}{T}\right)^{2/3}\left(\frac{3}{T}\right)^{5/3}\left(\frac{Q}{R}\right) & T \geq 3.0\ s \end{cases} \quad \text{RPA99V2003 (4.3.3)}$$

With:

A: zone acceleration coefficient (table 4.1)

η: Amplification factor of damping (damping correction factor) : when damping is different from 5 %): $\eta = \sqrt{\frac{7}{2+\xi}} \geq 0.7$

ξ: Critical Damping Percentage (RPA table 4.2)

R: global behavior coefficient (table 4.3).

T1, T2: characteristic periods associated with the category of site (table 4.7)

Q: quality factor (table 4.4)

IV.3.3. Determination of spectrum parameters:

Determination of the acceleration coefficient of the zone (A) (table 4.1 p37):

The zone acceleration coefficient depends on the seismic zone and the use group of the building.

In our case: $\begin{cases} \textbf{use groupe 2} \\ \textbf{sismic zone III (Boumerdes)} \end{cases} \rightarrow \mathbf{A = 0.25}$

Determination of the global behavior coefficient of the structure (R). (RPA .2003 Table 4.3 p38-39):

Assuming that our bracing system is a mixed shear wall-frame system with interaction

So $\rightarrow \mathbf{R = 5}$

Determination of the quality factor (Q):

The value of Q is determined by the formula: $\mathbf{Q = 1 + \sum_1^5 P_q}$ (formula 4.4 P 39)

With:

Pq: is the penalty to retain depending on whether the quality criteria Q is satisfied or not (table 4.4 P41)

The values to be retained are in the following table:

N°	Criterion P_q	Observation		Penalties	
		P_{qx}	P_{qy}	P_{qx}	P_{qy}
01	Minimum requirements for bracing members	Yes	Yes	0.00	0.00
02	Redundancy in plan	Yes	Yes	0.00	0.00
03	Regularity in plan	No	No	0.05	0.05
04	Regularity in elevation	Yes	Yes	0.00	0.00
05	Material quality control	Yes	Yes	0.00	0.00
06	Execution quality control	Yes	Yes	0.00	0.00

Table IV.1: values of the penalty factors P_q

So:

$$\begin{cases} Q_x = 1.05 \\ Q_y = 1.05 \end{cases}$$

Determination of periods T1 and T2 :

T1, T2 represent the characteristic periods associated with the category of site and given by the RPA99. (Table 4.7 P45)

site	S_1	S_2	$\boldsymbol{S_3}$	S_4
$T_{1(sec)}$	0.15	0.15	0.15	0.15
$T_{2(sec)}$	0.30	0.40	0.50	0.70

Table IV.2: Values of T_1 and T_2

In our case the soil is loose soil (site S3): $T_1 = 0.15s$ & $T_2 = 0.50s$

Determination of the damping correction factor η:

$\eta = \sqrt{\frac{7}{2+\xi}} \geq 0.7$ with: $\xi = 8.5$

So: $\eta = \sqrt{\frac{7}{2+8.5}} = 0.816 > 0.7$

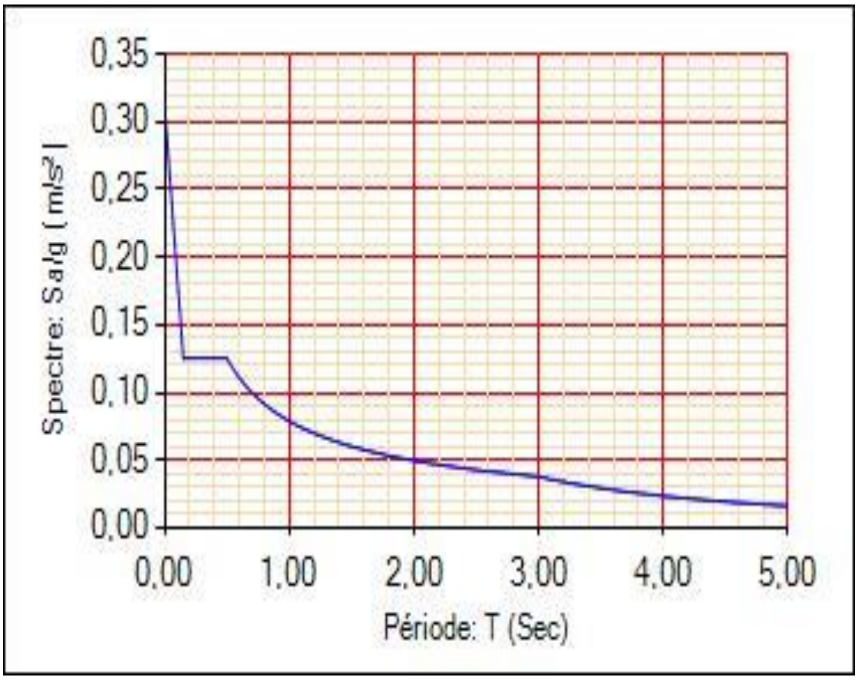


Figure IV.2: Response spectrum of the structure.

IV.3.3. Structure modelling:

The response spectrum being integrated in the ETABS20 software with which we modeled the structure, the data corresponding to our project are:

Zone III.

Use group 2

Quality factor (Qx =Qy = 1.05)

Global behavior factor of structure R = 5

soft soil S3

IV.3.4 shear walls positions:

Several positions have been tested in order to obtain a good behavior of structure, while trying to respect the architectural aspect which is an important point in our work and also to satisfy the conditions of the RPA99V2003 (Article 3.4.4 a P27).

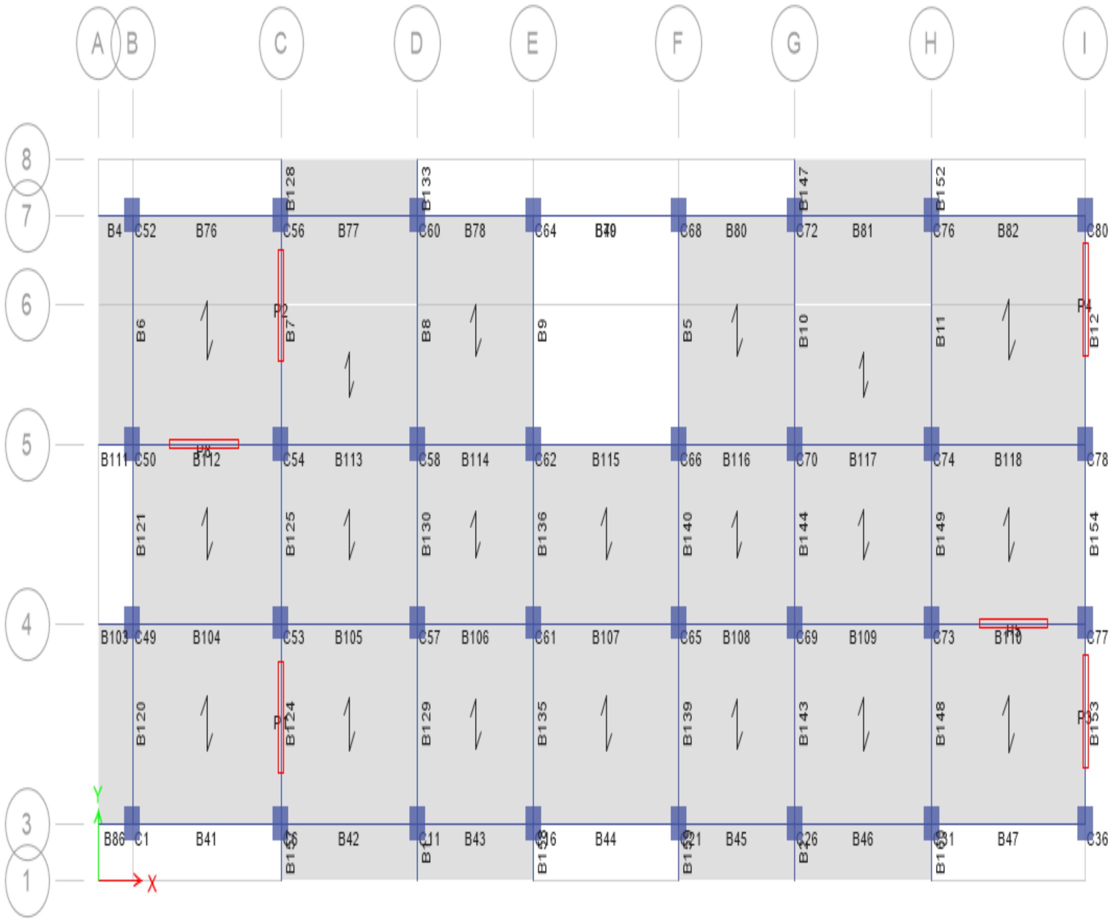


Figure IV.3: shear walls position

IV.3.5 justification of requirements of the RPA99 version 2003:

The RPA99v2003 requires six main verification to be met, which concern the dynamic behavior of the studied model.

- The first concerns the modal mass participation factors and their modes of vibration.
- The second involves the justification of the seismic force's resultant at the base.
- The third verification is related to the interaction under horizontal and vertical loads linked to the type of bracing, the fourth verification concerns the reduced normal force in the columns, and finally.
- The last concerns the justification with regard to deformations.

Vibration modes and rate of participation of masses:

For the structures represented by plane models in two orthogonal directions, the number of vibration modes to be retained in each of the two directions of excitation must be such that:

The sum of the effective modal masses for the modes selected is equal to at least 90% of the total mass of the structure.

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 minimum number of modes to be retrained is three (3) in each direction considered (Article 4.3.4 P45-46).[17]

NOTES:

This model period is T = 0.468s

The 1st and 2nd are translation modes

The 3rd mode is a rotation mode

One must retain the first 6 modes, so that the model mass reaches the 90% (according to the RPA99v2033)

The following table illustrates the results of the first fifteen vibration modes obtained:

Levels	Periods(sec)	(%) Of the model mass		Cumulative (%) of the modal mass		Rz(%)	Sum Rz (%)
		Ux	Uy	Ux	Uy		
1	0,468	0%	70%	0%	70%	5%	5%
2	0,424	78%	0%	78%	70%	2%	7%
3	0,359	2%	5%	80%	75%	68%	76%
4	0,127	13%	0%	92%	75%	0%	76%
5	0,123	0%	14%	92%	89%	1%	77%
6	0,096	0%	2%	92%	91%	14%	91%
7	0,065	5%	0%	97%	91%	0%	91%
8	0,056	0%	5%	97%	96%	1%	92%
9	0,045	0%	1%	97%	97%	5%	97%
10	0,042	2%	0%	99%	97%	0%	97%
11	0,034	0%	2%	99%	99%	0%	97%
12	0,032	1%	0%	100%	99%	0%	97%
13	0,028	0%	0%	100%	99%	2%	99%
14	0,025	0%	1%	100%	100%	0%	99%
15	0,021	0%	0%	100%	100%	1%	100%

Table IV.3: Period and mass participation rate of the structure.

The vibration modes are shown in the following figures:

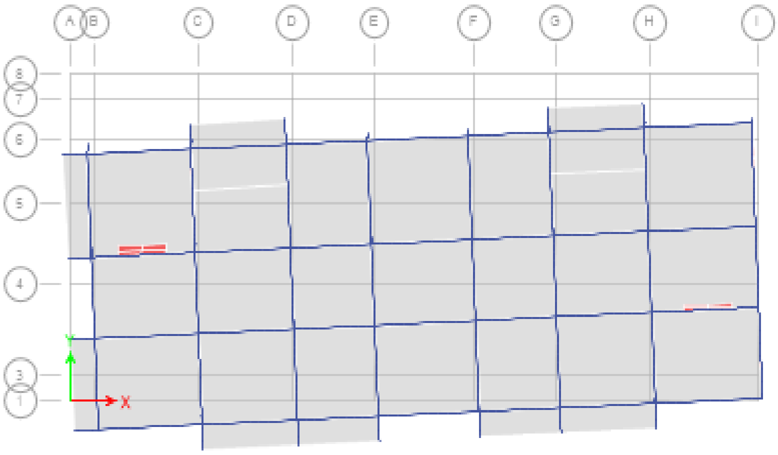


Figure IV.4: 1st vibration mode (translation along the y axis) T1 = 0.468s

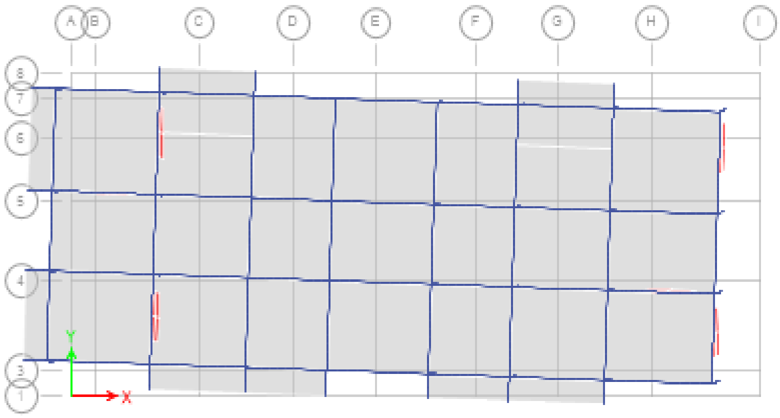


Figure IV.5: 2nd vibration mode (translation along the x axis) T2 =0.424s

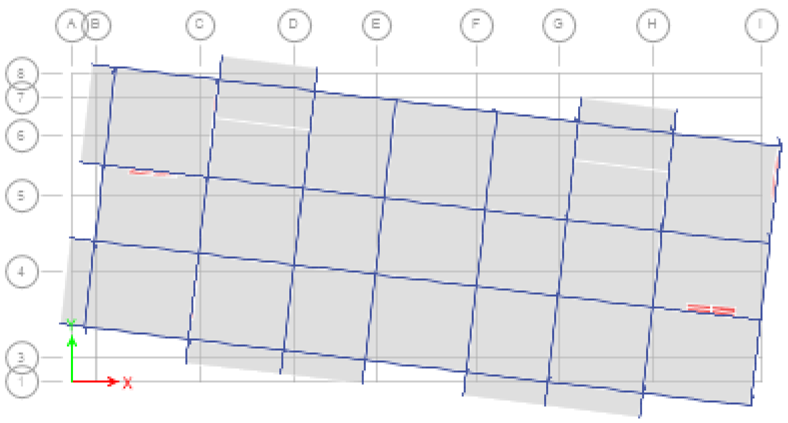


Figure IV.6: 3rd vibration mode (rotation around the z axis) T3 = 0.354s

1. Verification of the seismic resultant:

Referring to Article 4.3.6 of RPA99v2003, the seismic force resultant at the base V dynamic, obtained by combining the modal values, must not be less than 80% of the seismic force resultant determined by the equivalent static method Vst for a value of the fundamental period given by the appropriate empirical formula. If $V_{dyn} < 0.8V_{st}$, all response parameters (forces, displacement, moments, etc.) must be increased by the ratio $0.8V_{st}/V_{dyn}$.

We must therefore evaluate the shear force at the base of the structure by the equivalent static method given by the formula bellow:

$$\mathrm{Vst} = \frac{\mathrm{A} \times \mathrm{D} \times \mathrm{Q}}{\mathrm{R}} \times \mathrm{W} \quad (\ \text{RPA Art 4.2.3})$$

With:

$A = 0.25$; $Q = 1.05$; $R = 5$

W: total weight of the structure.

W is the sum of the weight Wi, calculated at each level (i).

$$W = \sum_{i=1}^{n} W_i \qquad with\ \ W_i = W_{Gi} + \beta W_{Qi} \qquad\qquad [17]$$

W_{Gi} : Weight due to permanent loads and those of any fixed equipment attached to the structure.

W_{Qi} : operating load.

β : weighting coefficient, depending on the nature and duration of the operating load and given by the table 4.5 of the RPA. In the case of a building for residential use, the value of this coefficient is set at 0.2.

From the results of modeling by ETABS20.01 we find:

$$W = 15496.40\text{KN}$$

So, all the parameters are calculated except the mean dynamic amplification factor D

The average dynamic amplification factor Dx and Dy, respectively for the longitudinal and transverse directions as follows:

$$D = \begin{cases} 2.5\eta & 0 \le T \le T_2 \\ 2.5\eta\ (\frac{T_2}{T})^{\frac{2}{3}} & T_2 \le T \le 3.0\ s \\ 2.5\eta\ (\frac{T_2}{3})^{\frac{2}{3}}\ (\frac{3}{T})^{\frac{5}{3}} & T \ge 3.0\ s \end{cases}$$

So to calculate the value of D we must calculate the period T:

2. Determination of the fundamental period T:

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

Empirical period

$$T = C_T h_N^{3/4} \qquad \text{(Article 4.2.4 P42)}$$

h_N : Height measured in meters from the base of the structure to the top level (N). For our case: $\mathbf{h_N = 15.30\ m}$

C_T : Coefficient depending on the bracing system and the type of infill. For our case

$\mathbf{C_T = 0.05}$ (table 4.6 P42)

So $T = 0.05 \times 15.3^{3/4} = 0.39s$

Remark: According to (Article 4.2.4 P42), the period can be calculated by two other formulas, Ryleigh formula or the formula:

$$T = {0.09h_N}/{\sqrt{D}}$$

According to RPA99v2003 applicable possibly for the case n°3 and 4 which is ours, quoted in RPA on (table 4.6 P42).

Where D is the dimension of the building measured at its base in the direction of calculation considered.

Dx = 29.35 m;

Dy = 13.20 m

Tx = 0.25s; Ty = 0.38 s

$T_x = \min(0.25; 0.39) \Longrightarrow T_x = 0.25s$

$T_y = \min(0.38; 0.39) \Longrightarrow T_y = 0.38s$

In the article (4.2.4 P42) RPA99v2003 requires that the period calculated from the numerical methods must not exceed those of the empirical formulas by more than 30%.

$T_{x\,analy} < 1.3T_{emp} \Leftrightarrow 0.46 > 0.33s \Longrightarrow T_x = 0.33s$

$T_{y\,analy} < 1.3T_{emp} \Leftrightarrow 0.46 < 0.49s \Longrightarrow T_x = 0.46s$

We must calculate (D) with 1.3 Temp

Direction X-X :

We have:

$0 < T = 0.33s < T_2 = 0.5s$

So: $D = 2.5\eta \Longrightarrow D = 2.5 \times 0.816 = 2.04$

Direction Y-Y:

$0 < T = 0.46s < T_2 = 0.5s$

So: $D = 2.5\eta \Longrightarrow D = 2.5 \times 0.816 = 2.04$

From the results of modelling by ETABS we find:

$W = 15496.40\text{KN}$

$V_{dyn} = 1538.2675\ KN$; $V_{dyn} = 1427.1484\ KN$

$V_{st} = 1508.96\ KN$; $V_{st} = 1508.96\ KN$

The verification of the seismic resultant referred to in Article 4.3.6 of RPA99v2003 is summarized in the following table:

Direction	V_{dyn} (KN)	V_{st} (kN)	V_{dyn}/V_{st}	Observation
X-X	1538.26	1508.96	1.019	Verified condition
Y-Y	1427.14	1508.96	0.945	Verified condition

Table IV.4: Verification of the shear force at the base.

3. Verification of the wall-frame interaction:

According to Article 3.4.4 of RPA99v2003, it is imperative for structure with mixed moment resistant frames/shear walls with interaction to meet the following conditions:

Shear walls must not take more than 20% of the load due to vertical loads.

Horizontal loads are resisted jointly by the shear walls and frames, as well as the loads resulting from their interactions at all levels.

In addition to the loads due to vertical load, the frames must take at least 25% of the floor's shear force.

Under vertical loads:

The shear walls must not take more than 20% of the loads:

$$\frac{\sum F_{\text{shear walls}}}{\sum F_{\text{frames}} + \sum F_{\text{shear walls}}} \leq 20\%$$

The frames must take at least 80% of loads:

$$\frac{\sum F_{\text{frames}}}{\sum F_{\text{frames}} + \sum F_{\text{shear walls}}} \geq 80\%$$

The results of the interaction under vertical loads are summarized in the table below:

Levels	Vertical loads (KN)		Vertical loads (%)		Observation
	Frames	Shear walls	Frames	Shear walls	
G floor	13628,636	1867,7714	87,9470683	12,0529317	Verified
1st floor	10933,7244	1525,7566	87,7542524	12,2457476	Verified
2nd floor	8265,678	1161,5102	87,6791449	12,3208551	Verified
3rd floor	5609,0909	785,8046	87,7120025	12,2879975	Verified
4th floor	2964,7114	397,8913	88,1671629	11,8328371	Verified

Table IV.5: Justification of the interaction under vertical loads.

Under horizontal loads:

The shear walls must not take more than 75% of the loads:

$$\frac{\sum F_{\text{shear walls}}}{\sum F_{\text{frames}} + \sum F_{\text{shear walls}}} \leq 75\%$$

The frames must take at least 25% of loads:

$$\frac{\sum F_{\text{frames}}}{\sum F_{\text{frames}} + \sum F_{\text{shear walls}}} \geq 25\%$$

The results of the interaction under horizontal loads are summarized in the table below:

Levels	Horizontal loads (KN)				Horizontal loads (%)				Observation
	Direction X-X		Direction Y-Y		Direction X-X		Direction Y-Y		
	frames	Shear walls	frames	Shear walls	frames	Shear walls	frames	Shear walls	
G floor	857,13	681,58	639,36	804,36	55,70	44,29	44,285	55,71	Verified
1st floor	869,34	563,90	522,43	819,11	60,65	39,34	38,942	61,057	Verified
2nd floor	799,42	432,16	482,26	682,65	64,90	35,09	41,399	58,60	Verified
3rd floor	626,02	303,34	386,76	511,27	67,36	32,639	43,067	56,93	Verified
4th floor	461,67	105,20	280,27	266,94	81,44	18,55	51,217	48,78	Verified

Table IV.6: Justification of the interaction under horizontal loads.

Comment:

Table IV.6 shows the horizontal loads converted into percentages taken up by the frames and walls at each storey. We note that the 25% limit required by RPA99/version 2003 has been generally respected, and so is the 75% limit. After satisfaction of the wall-frame interaction under horizontal and vertical loads, we can say that the appropriate system is a mixed system with wall-frame interaction.

4. **Verification of reduced normal force:**

In order to avoid or limit the risk of brittle fracture under overall stress due to earthquake, (Article 7.4.3.1) requires checking the normal design compressive force which is limited by the following condition:

$$v = \frac{N_d}{B_c f_{c28}} \leq 0.3$$

With:

B_c : Area of the cross of the considered column

N_d : Calculation compressive normal force under accidental combinations (seismic)

According to the CBA, in chapter B.8.2, (combinations of actions to be considered) in Article B.8.2.2 (columns subjected of loads due to gravity and to earthquake), there is this which follows: “the combinations of action to be considered are those given by DTR.BC 2.48 to which reference should be made. Therefore, for a structure with walls (Mixed)”

G+Q+Ex,y ; G+Q-Ex,y / 0.8G+Ex,y ; 0.8G-Ex,y

From experience, the most unfavorable combination was the combination:

G+Q+Ex,y ; G+Q-Ex,y

The results obtained are illustrated in the following table:

Levels	$\boldsymbol{B_c}$ (m²)	$\boldsymbol{f_{c28}}$ (Mpa)	$\boldsymbol{N_d}$ (KN)	v	Observation
Ground floor	0.27	25	1336.47	0.19	Verified
First floor	0.27	25	1021.23	0.15	Verified
Second floor	0.27	25	683.55	0.10	Verified
Third floor	0.27	25	404.70	0.05	Verified
Fourth floor	0.27	25	204.58	0.03	Verified

Table IV.7: Verification of the reduced normal force of the columns.

5. **Displacement verification:**

According to article 4.4.3, the horizontal displacement at each level K of the structure is calculated by:

$$\delta_k = R \times \delta_{ek}$$

With:

δ_{ek} : Displacement due to the seismic forces Fi (including torsion effect)

δ_k : Horizontal displacement at each level « K » of the structure

R: Global behavior coefficient, our case R =5

According to (Article 5.10), the relative lateral displacements of a floor in relation to the floor under it (K-1) must not exceed 1% of the story height:

$\Delta_k < 1\%\ h_k$

With:

h_k : The height of the floor

The relative displacement at level “K” with respect to level (K-1) is equal to:

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

The results obtained are summarized in the following table:

Levels	h_k (cm)	Direction X-X					Direction Y-Y					observation
		δ_{ek} (cm)	δ_k (cm)	δ_{k-1} (cm)	Δ_k (cm)	Δ_k/h_k	δ_{ek} (cm)	δ_k (cm)	δ_{k-1} (cm)	Δ_k (cm)	Δ_k/h_k	
G floor	306	0,116	0,58	0	0,58	0,18	0,145	0,725	0	0,725	0,236	Verified
1st floor	306	0,313	1,565	0,58	0,985	0,32	0,433	2,165	0,725	1,44	0,47	Verified
2nd floor	306	0,51	2,55	1,565	0,985	0,32	0,757	3,785	2,165	1,62	0,529	Verified
3rd floor	306	0,678	3,39	2,55	0,84	0,27	1,059	5,295	3,785	1,51	0,493	Verified
4th floor	306	0,801	4,005	3,39	0,615	0,2	1,316	6,58	5,295	1,28	0,419	Verified

Table IV.8: Verification against deformations.

From the table above, we see that the relative displacements of the levels are less than 1% of the floor height, which means that the condition is verified.

Justification with respect to the P-Δ effect:

The second order effects (or P-Δ effect) are the effects due to vertical loads after displacement. They can be neglected in the case of buildings if the following condition is satisfied at all levels (Article 5.9)

$$\theta = \frac{P_K \times \Delta_K}{h_K \times V_K} < 0.1 \qquad RPA99v2003\ (Article\ 5.9)$$

With:

P_K : Total weight of the structure and associated operating loads above level K

v_K : Floor sheer force of level K

Δ_K : Relative displacement of level K with respect to level K-1

h_K : Floor height

If $0.1 \le \theta_k \le 0.2$:The effect of (P-Δ) can be approximately taken into account by amplifying the effects of seismic action calculated using a first-order elastic analysis by the factor : $\frac{1}{1-\theta}$

If $\theta_k > 0.2$: The structure is partially unstable and must be resized.

The results of the verification are summarized in the table below:

Levels	h_k (cm)	P_k (KN)	Direction X-X			Direction Y-Y			Observation
			Δ_k (cm)	V_k (KN)	θ_k	Δ_k (cm)	V_k (KN)	θ_k	
G floor	306	15496,40	0,58	1538,26	0,0017	0,725	1427,14	0,002	Verified
1st floor	306	12459,48	0,985	1431,35	0,0026	1,44	1329,27	0,004	Verified
2nd floor	306	9427,18	0,985	1228,51	0,0023	1,62	1155,15	0,004	Verified
3rd floor	306	6394,89	0,84	925,38	0,0017	1,51	890,58	0,003	Verified
4th floor	306	3362,60	0,615	539,74	0,0011	1,28	541,22	0,002	Verified

Table IV.9: Verification of P-Δ effects.

Remark: note that the values of θ_k are less than 0.1 for all levels and this in both directions of calculation.

6. Verification for overturning:

To ensure stability against overturning, the building must satisfy the following equation:

$\frac{\sum W_k . b_i}{\sum F_k . h_k} \ge 1.5$ With:

W_k : The calculated weight at each level

b_i : Centre of gravity of the structure at each level

F_k : The sum of seismic forces at each floor level K

h_k : The height of the floor K

Direction X-X:

Levels	W_k	b_i	$W_k \times x_{cm}$	F_{kx}	h_k	$F_{kx} \times h_k$
Story5	3362.60	14.65	49262.09	539.74	3.06	1651.60
Story4	6394.89	14.65	93685.14	925.38	6.12	5662.32
Story3	9427.18	14.65	138108.187	1228.51	9.18	11277.72
Story2	12459.48	14.65	182524.35	1431.35	12.24	17519.72
Story1	15406.40	14.65	225703.76	1538.26	15.3	23535.37
			Sum=689283.52			Sum=59646.73

Table IV.10: Verification of overturning direction X-X

$$\frac{\sum W_k . b_i}{\sum F_k . h_k} = 11.41 > 1.5 \quad verified\ condition$$

Direction Y-Y:

Levels	W_k	b_i	$W_k \times x_{cm}$	F_{kx}	h_k	$F_{kx} \times h_k$
Story5	3362.60	6.33	21285.25	541.22	3.06	1656.13
Story4	6394.89	6.33	40479.65	890.58	6.12	5450.34
Story3	9427,18	6.33	59674.04	1155.15	9.18	10604.27
Story2	12459.48	6.33	78868.50	1329.27	12.24	16270.26
Story1	15496.40	6.33	98092.21	1427.14	15.3	21835.24
			Sum=2983399.65			Sum=55816.24

Table IV.11: Verification of overturning direction Y-Y

$$\frac{\sum W_k . b_i}{\sum F_k . h_k} = 5.29 > 1.5 \quad verified\ condition$$

IV.4. CONCLUSION:

After several tests on the layout of the bracing walls, which were modeled to balance between the strength criteria and the economic criteria, we were able to meet all the requirements of the RPA99v2003 while respecting the architectural aspect of the building, which posed a major problem to the layout of the walls. Finally, we arrived at a layout of the walls ensuring a good dynamic behavior of the building, and this was achieved by increasing the section of the columns. The final dimensions of the structural elements are shown in the following table:

Levels	Ground floor	First floor	Second floor	Third floor	Fourth floor
Columns (cm^2)	(45x60)				
Shear walls (cm)	Thickness: 20cm				
Principal beams (cm^2)	(30x40)				
Secondary beams (cm^2)	(20x30)				

PRT II: STUDY OF STRUCTURAL ELEMENTS.

VI.5 Introduction:

Once the different forces and stresses associated with each of the elements are known, their reinforcement becomes possible. The principle consists of applying calculation methods based on the limit stat philosophy to define total steel sections in the sections of the various concrete elements, which are essential for bearing the imposed loads. However, these steel sections must comply with standers defined by the various building codes. These standards also prescribe minimum and maximum reinforcement requirements as well as some construction arrangements, which will be explained in the following sections.

IV.6 study of beams:

Beams are horizontal reinforced concrete elements designed to resist bending loads. The loads are obtained by the following combinations:

$$ULS\begin{cases}1.35G+1.5Q\\G+Q\pm Ex,y\\0.8G\pm Ex,y\end{cases}\qquad SLS: G+Q$$

The reinforcement of the beams is carried out in accordance with CBA93 as well as the provisions of the RPA99v2003.

The beams are designed for simple bending and are subjected to bending moments and shear force, in our case we have two types of beams to study, namely:

Secondary beams (25×30)

Main beams (30×40)

Recommendations of RPA99v2003:

Temporary formwork:

$$\begin{cases}b\geq 20cm\\h\geq 30cm\\b_{max}\leq 1.5h+b_1\end{cases}\quad and\ \frac{h}{b}\leq 4\qquad (Art.7.5.1)$$

Reinforcement:

Longitudinal reinforcements (Art.7.5.2.1):

The minimum total percentage of longitudinal steel over the entire length of the beam is 0.5% $(b\times h)$ in any section.

$$either\ \begin{cases}secondary\ beams(25\times 30)\Longrightarrow A_{min}=0.5\%\ (25\times 30)=3.75\ cm^2\\main\ beams\ 30\times 40\Longrightarrow A_{min}=0.5\%\ 30\times 40=6\ cm^2\end{cases}$$

The maximum total percentage of longitudinal steel is:

$$A_{max}\begin{cases} 4\%\ in\ the\ typical\ zone \\ 6\% in\ the\ overlapping\ zone \end{cases} \qquad (\ zone\ III)$$

Either:

$$beams\ (25\times30)\begin{cases} A_{max}^{ty.z} = 4\% 20\times30 = 30\ cm^2 \\ A_{max}^{ov.z} = 6\%\ 20\times30 = 45\ cm^2 \end{cases}$$

$$beams\ (30\times40)\begin{cases} A_{max}^{ty.z} = 4\%(30\times40) = 48\ cm^2 \\ A_{max}^{ov.z} = 6\%\ (30\times40) = 72\ cm^2 \end{cases}$$

The minimum overlapping length is 50ϕ in zone III

The stirrups at the node arranged as transverse reinforcement of the columns can be made up of two superimposed U-bars, and the overlapping direction of these U-bars must be alternated.

The beams supporting low vertical loads and primarily subjected to seismic lateral forces must have symmetrical reinforcements with a mid-span section at least equal to half the section at support.

Transverse reinforcements:

The minimum quantity of transverse reinforcement is given by: $A_t^{min} = 0.3\% S_t \times b$

With:S_t: maximum spacing between transverse reinforcement given by:

$$\begin{cases} S_t \le \min\left(\frac{h}{4}; 12\emptyset_l\right) \Longrightarrow in\ nodal\ zone \\ S_t \le \frac{h}{2} \Longrightarrow outside\ the\ nodal\ zone \end{cases}$$

Such that:

$\emptyset_l$: The smallest diameter used; H: Height of the beam

The first transverse reinforcement must be located at a distance of 5 cm at most from the support or from the fixed end.

Calculation of reinforcement:

The reinforcement of the beams is calculated from the maximum stresses, taken from the ETABS 2022 software, and are summarized in the table below:

Beams		M_{span} **(KN.m)**		**combinaison**	$M_{support}$ **(KN.m)**		**combinaison**	V_{max} **(KN)**	**combinaison**
Main beam	Associated with walls	46.4	-41.9	G+Q+Ex	124.65	-121.86	G+Q+Ex	234.9	G+Q+Ex
	Not associated with the walls	26.6	-27.4	G+Q+Ex 0.8G+Ex	32.33	-63.44	G+Q+Ex 0.8G+Ex	-83	1.35G+1.5Q
Secondary beam	Associated with walls	29.3	-46.0	0.8G+Ey G+Q+Ey	86.07	-97.56	0.8G+Ey G+Q+Ey	-307.7	G+Q+Ey
	Not associated with the walls	14.6	-12.1	G+Q+Ey 0.8G+Ey	23.08	-29.99	G+Q+Ey	245.6	G+Q+Ex

Table IV.12: Maximum stresses in the beams.

The calculation of the reinforcement of the beams is done with simple bending, according to the ROBOT EXPERT 2010 software the results are summarized in the table below:

Beams type	**location**	$A_{calculated}$ A_1	$A_{calculated}$ A_2	$A_{\min BAEL}$	$A_{\min RPA}$	**Choice of reinforcements** A_1	**Choice of reinforcements** A_2	$A_{adopted}$
Main beams with	**span**	3.2	2.9	1.30	6	3HA14	3HA14	9.23
	support	9	8.8			6HA14	6HA14	18.46
Main beam without walls	**span**	3.1	1.9	1.30	6	3HA14	3HA14	9.23
	support	2.2	4.5			3HA14	3HA14	9.23
Secondary beams	**span**	2.9	4.7	0.65	3	4HA14	4HA14	12.30
	support	8.8	10.5			8HA14	8HA14	24.61
without the walls	**span**	2.8	1.2	0.65	3	3HA12	3HA12	6.78
	support	2.2	3			3HA12	3HA12	6.78

Table IV.13: Reinforcement of main and secondary beams.

Calculation of transverse reinforcement:

Main beams:

The transverse reinforcement of the beams is calculated using the formula (BAEL91)

$\emptyset t \leq$ min (h /35; b/10; $\emptyset_l$)

With:

$\emptyset_l$: Minimal diameter of beams longitudinal reinforcement.

$\emptyset t \leq$ min (40 /35; 30 /10; 1.4)

$\emptyset t$= 1.14cm so we take ϕt = 8 mm

Spacing calculation:

Spacing is calculated with RPA99v2003:

$$\begin{cases} S_t = \min\left(\frac{h}{4}\,; 12 \times \emptyset_l\right) = \min\left(\frac{40}{4}; 12 \times 1.4\right) = 10\text{cm. nodal zone} \\ S_t \le \frac{h}{2} = \frac{40}{2} = 20\text{ cm} \qquad \text{outside the nodal zone} \end{cases}$$

$$\begin{cases} S_t = 10\text{ cm nodal zone} \\ S_t = 15\text{ cm outside nodal zone} \end{cases}$$

The minimum section of transverse reinforcement:

$At_{min} = 0.003 \times S_t \times b = 0.003 \times 15 \times 30 = 1.35\ cm^2$

$\Longrightarrow\ A_t = 4T8 = 2.01\ cm^2$

The minimum lap length:

$L_r = 50 \times \emptyset_l = 50 \times 1.4 = 70\ cm$

Secondary beams:

$\emptyset_t \le \min\left(\frac{30}{35}; \frac{20}{10}; 1.2\right)$

$\emptyset_t = 0.85\ cm\ so\ we\ take\ \emptyset_t = 8\ mm \Longrightarrow\ A_t = 4T8 = 2.01\ cm^2$

Spacing calculation:

$$S_t = \min\left(\frac{h}{4}\,; 12 \times \emptyset_l\right) = \min\left(\frac{30}{4}; 12 \times 1.2\right) = 7.5\text{cm.} \qquad \text{nodal zone}$$

$$S_t \le \frac{h}{2} = \frac{30}{2} = 15\text{ cm} \qquad \text{outside the nodal zone}$$

$S_t = 10$ cm nodal zone

$S_t = 15$ cm current zone

Transverse reinforcement minimum section:

$At_{min} = 0.003 \times S_t \times b = 0.003 \times 15 \times 20 = 0.9\ cm^2$

$\Longrightarrow\ A_t = 4T8 = 2.01\ cm^2$

The minimum lap length:

$L_r = 50 \times \emptyset_l = 50 \times 1.2 = 60\ cm$

IV.7 Verification:

IV.7 .1 At the Ultimate limited state (ULS):

Condition of non-fragility:

$$A_{min} = 0.23 \times b \times d \times \frac{f_{t28}}{f_e} \leq A_{cal} \Rightarrow \begin{cases} main\ beams: A_{min} = 1.30\ cm^2 \\ secondary\ beams: A_{min} = 0.65 cm^2 \end{cases}$$

$$A_{min} < A_{cal}\ \ so\ the\ non-fragility\ condition\ is\ satisfied$$

Verification of shear stress:

The check to be made with respect to the maximum tangential stress is that relating to cracking which is not very harmful, such as:

$$\tau u = \frac{Tu}{b\ .\ d} \leq \bar{\tau}_u = \min\left(\frac{0.2\ fc28}{\gamma_b}\ ;\ 5MPa\right)$$

The verification concerns only the most unfavorable beams, the results are summarized in the table below:

Beams	V_{max}	τ_u	$\bar{\tau}_u$	Observation
Main beams with walls	234.97	2. 17	3.33	V.C
Without walls	83	0.76	3.33	V.C
Secondary beams with walls	307.75	5.69	3.33	N.V.C
Without walls	245.68	4.54	3.33	N.V.C

Table IV.14: Verification of shear stresses.

In the secondary beams there is a risk of shearing, for this the reinforcements are tilted 45° and the section is increased with 15 cm for the beams associated with the wall and (5cm / h) + (5cm / b) for those not associated with the wall:

$$\bar{\tau}_u = \min\left(\frac{0.27\ fc28}{\gamma_b}\ ;\ 7MPa\right) \rightarrow \bar{\tau}_u = 4.5\ Mpa$$

$\tau_u = 3.79\ Mpa < \bar{\tau}_u = 4.5\ Mpa$ ... ... VC (With walls / (30×35))

$\tau_u = 3.63\ Mpa < \bar{\tau}_u = 4.5\ Mpa$ ... ... VC (Without walls (25*35))

Verification of longitudinal reinforcement in shear:

Intermediate supports: $A_l \geq \left(V_u + \frac{M_a}{0.9\times d}\right) \times \frac{\gamma_s}{f_e}$

Edge supports: $A_l \geq \ V_u \times \frac{\gamma_s}{f_e}$

The results are summarized in the table below:

Beams	V_{max}	M_a	A_L	observation
Main with walls	234.97	121.86	27.69	verified
Without walls	83	32.33	18.46	verified
Secondary with walls	307.75	86.07	30.77	verified
Without walls	245.68	23.08	13.56	verified

Table IV.15: Verification of longitudinal reinforcement in shear.

IV.7 .2 At the service limited state (SLS):

The verifications to be made during a cracking that is not very harmful concerning: the limit state of compression of the concrete as well as the limit state of deformation (assessment of the deflection).

Concrete compression limit state:

We must check that:

$\sigma_{bc} = \frac{M_{ser}}{I} \times y \leq \bar{\sigma}_{bc} = 0.6 \times f_{c28} = 15Mpa$

Calculation of: $y = \frac{b\times y^2}{2} + nA'_s(y - c') - nA_s(d - y)$=0 with n = 15

Calculation of: $I = \frac{b\times y^3}{3} + nA'_s(d - c')^2 + nA_s(d - y)^2$

The calculation results are summarized in the following table:

<table>
<tr><th rowspan="2">Beams</th><th rowspan="2">zone</th><th rowspan="2">M_{ser}</th><th colspan="2">A_S</th><th rowspan="2">Y</th><th rowspan="2">I</th><th colspan="2">Strain</th><th rowspan="2">observation</th></tr>
<tr><th>A_S</th><th>A'_S</th><th>σ_{bc}</th><th>$\bar{\sigma}_{bc}$</th></tr>
<tr><td rowspan="2">Main with walls</td><td>span</td><td>7.04</td><td>3.2</td><td>2.9</td><td>11.49</td><td>45570.63</td><td>1,775038</td><td>15</td><td rowspan="2">V</td></tr>
<tr><td>support</td><td>12.17</td><td>9</td><td>8.8</td><td>16.93</td><td>102372.35</td><td>2,01263427</td><td>15</td></tr>
<tr><td rowspan="2">Without walls</td><td>span</td><td>27.72</td><td>3.1</td><td>1.9</td><td>10.72</td><td>43062.40</td><td>6,9006465</td><td>15</td><td rowspan="2">V</td></tr>
<tr><td>support</td><td>34.34</td><td>2.2</td><td>4.5</td><td>11.66</td><td>37832.80</td><td>10,5835254</td><td>15</td></tr>
<tr><td rowspan="2">Secondary with walls</td><td>span</td><td>7.5</td><td>2.9</td><td>4.7</td><td>11.48</td><td>26241.90</td><td>3,28101243</td><td>15</td><td rowspan="2">V</td></tr>
<tr><td>support</td><td>12.46</td><td>8.8</td><td>10.5</td><td>15.76</td><td>57238.37</td><td>3,43073361</td><td>15</td></tr>
<tr><td rowspan="2">Without walls</td><td>span</td><td>17.51</td><td>2.8</td><td>1.2</td><td>9.41</td><td>20100.80</td><td>8,19714141</td><td>15</td><td rowspan="2">V</td></tr>
<tr><td>support</td><td>16.53</td><td>2.2</td><td>3</td><td>10.62</td><td>19240.46</td><td>9,12392947</td><td>15</td></tr>
</table>

Table IV.16: Verification of the compressive limit state of concrete.

Deformation limit state:

Verification of the deflection is necessary if one of the following conditions is not verified according to the CBA 93(A 6.5.1):

$$\frac{h}{L} \geq \frac{M_t}{10 \times M_0}$$
$$\frac{A}{b_0 \times d} \leq \frac{4.2}{f_e} \qquad with\ \ b_0: the\ width\ of\ rib$$
$$L \leq 8\ m$$

A: tensioned reinforcement section.

Calculation example:

Main beam with walls:

$\frac{h}{L} = \frac{0.4}{4.35} = 0.091$

$\frac{M_t}{10 \times M_0} \rightarrow M_t = 0.85 M_t \rightarrow \frac{0.85}{10} = 0.085$

$\frac{h}{L} = 0.091 > \frac{M_t}{10 \times M_0} = 0.085 \ldots \ldots VC$

$\frac{A}{b_0 \times d} = \frac{3.2}{10 \times 36} = 0.008 < 0.0105 \ldots \ldots VC$

$L = 3.95\ m\ < 8\ m \ldots \ldots VC$

The table below summarizes the verification results of the 3 conditions for the types of beam:

Beams	h	b_0	L	A	$\frac{h}{L} \geq \frac{1}{16}$	$\frac{h}{L} \geq \frac{M_t}{10 \times M_0}$	$\frac{A}{b_0 \times d} \leq \frac{4.2}{f_e}$	Observation
Main with walls	40	30	3.9	3.2	$0.102 > 0.06$	$0.102 > 0.085$	$0.002 < 0.0105$	V
Without walls	40	30	3.9	3.1	$0.102 > 0.06$	$0.102 > 0.085$	$0.002 < 0.0105$	V
Secondary with walls	35	30	3.5	2.9	$0.1 > 0.06$	$0.1 > 0.085$	$0.003 < 0.0105$	V
Without walls	35	25	3.5	2.8	$0.1 > 0.06$	$0.1 > 0.085$	$0.003 < 0.0105$	V

Table IV.17: Verification of deflection conditions of beams.

The three conditions are satisfied, so the deflection check is not necessary.

IV.8 Beam reinforcement diagrams:

The reinforcement diagrams of the beams are shone in the figures below:

Main beams:

With walls:

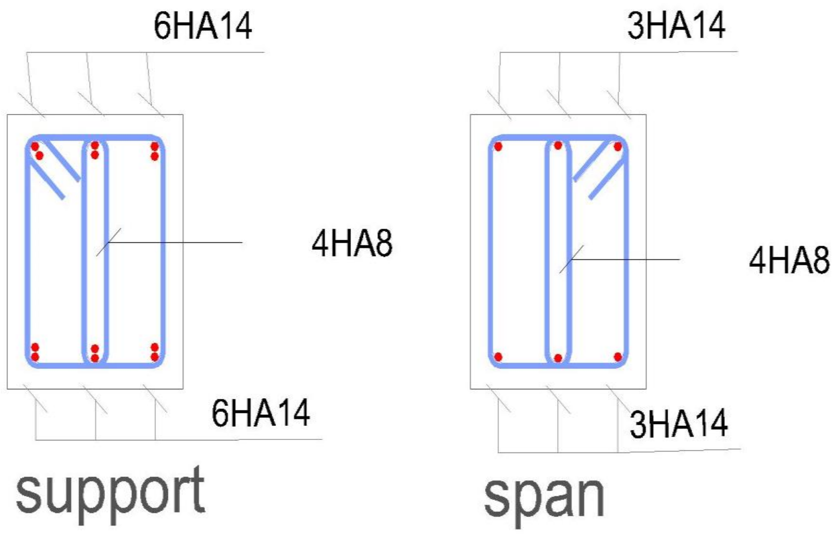


Figure IV.7: Reinforcement diagrams of the main beams with walls.

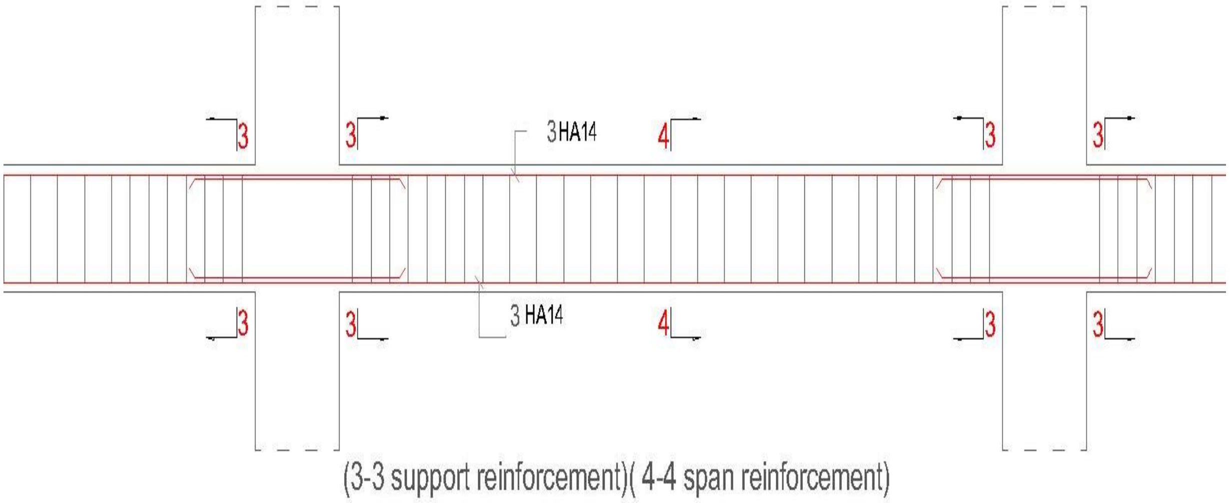


Figure IV.8: Schematic example of constructional provisions of the main beams with walls.

Without walls:

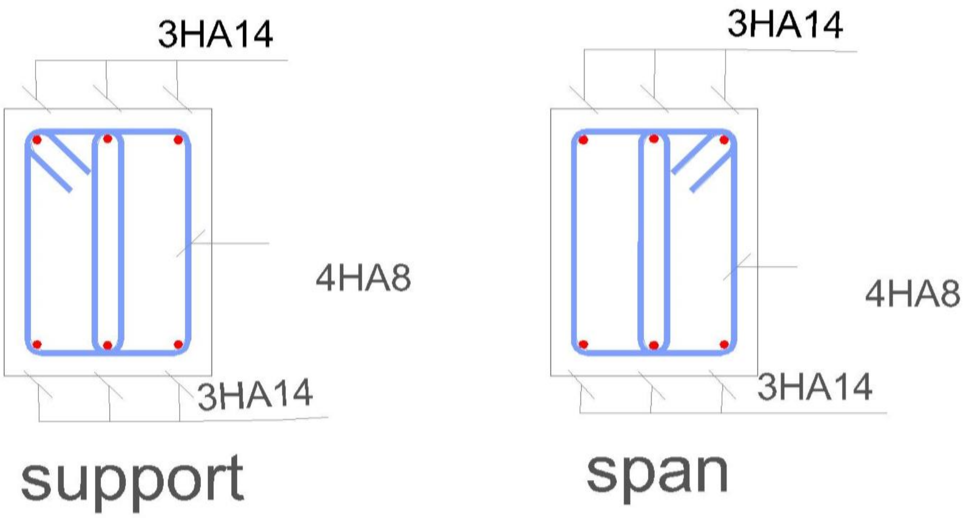


Figure IV.9: Reinforcement diagrams of the main beams without walls.

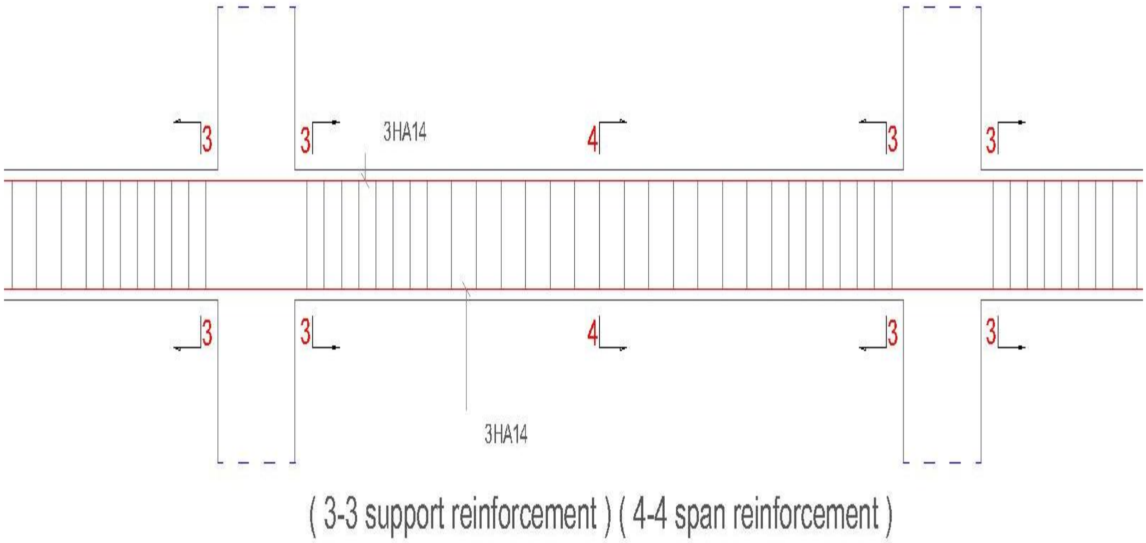


Figure IV.10: Schematic example of constructional provisions of the main beams without walls.

Secondary beams:

Without walls:

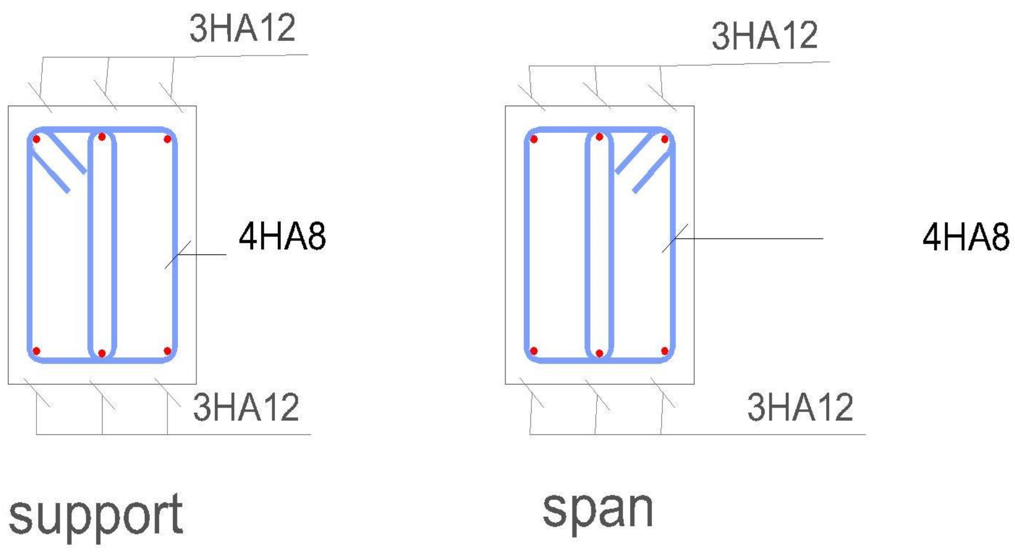


Figure IV.11: Reinforcement diagrams of the secondary beams without walls

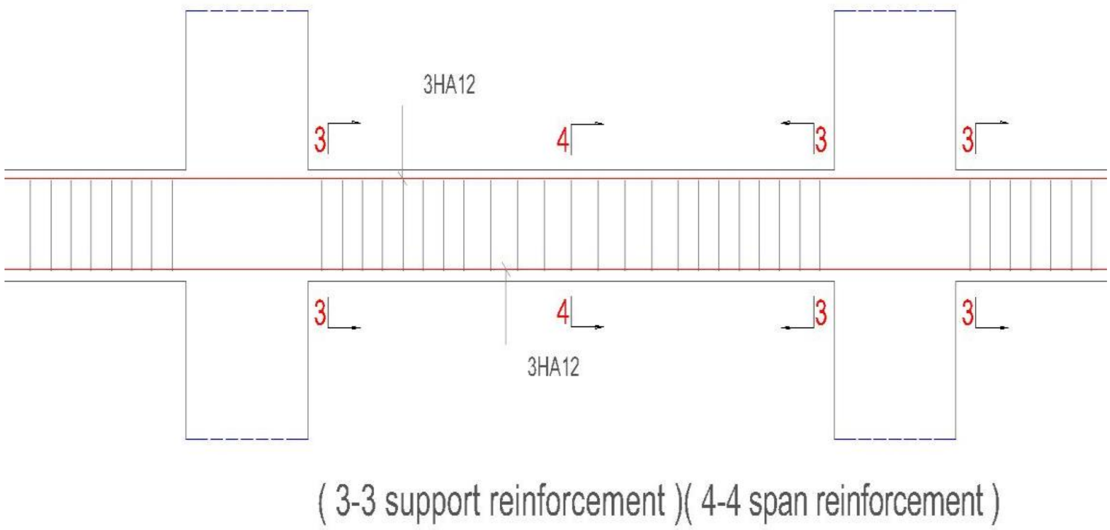


Figure IV.12: Schematic example of constructional provisions of the secondary beams without walls.

With walls:

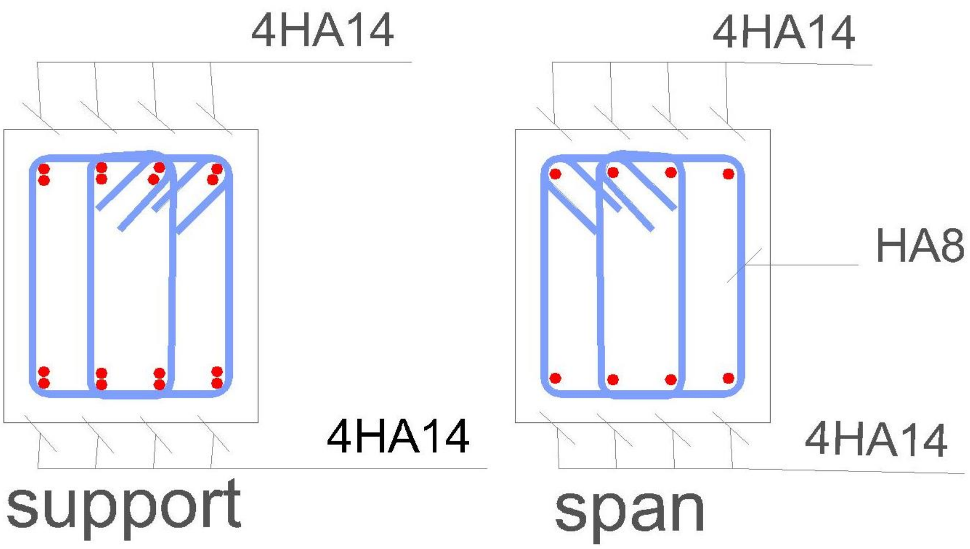


Figure IV.13: Reinforcement diagrams of the secondary beams with walls

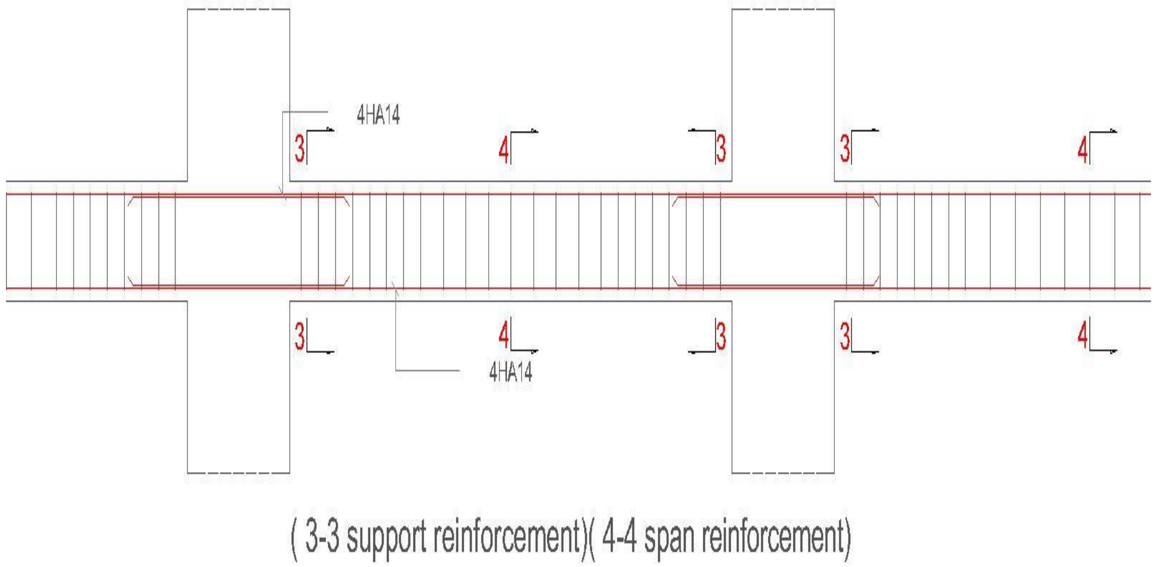


Figure IV.14: Schematic example of constructional provisions of the secondary beams with walls.

IV.10 Study of the columns:

Columns are structural elements that primarily resist axial compressive loads. Columns are a vertical member which transfers the loads from the beam or slab above to the foundation below. Their reinforcement will be done for combined bending according to the most unfavorable calculation combinations, introduced during the modelling with ETABS2022 software, as follow:

$$ULS\begin{cases}1.35G + 1.5Q \\ G + Q \pm Ex, y \\ 0.8G \pm Ex, y\end{cases} \qquad and\ SLS: G + Q$$

The adopted reinforcement will be the maximum obtained according to the following solicitations:

Maximum axial force and corresponding moment: $(N_{max} \rightarrow M_{corr})$

Maximum moment and corresponding normal force: $(M_{max} \rightarrow N_{corr})$

Minimal normal force and corresponding moment: $(N_{min} \rightarrow M_{corr})$

IV.10.1 recommendation of RPA99v2003:

Formwork of columns:

$$\begin{cases}Min(b_1, h_1) \geq 30cm \qquad in\ zoneIII \\ Min(b_1, h_1) \geq h_e/20 \\ 1/4 < b_1/h_1 < 4\end{cases}$$

Reinforcements:

The longitudinal reinforcements:

The longitudinal reinforcement must be of high adherence, straight and without hooks:

Their minimum percentage will be: $A_{min} = 0.9\%\ in\ zoneIII$

Their maximum percentage will be: $A_{max}\begin{cases}4\% \quad in\ regular\ zone \\ 6\% \quad in\ overlap\ zone\end{cases}$

$$\emptyset_{min} = 12\ mm\ (Minimum\ diameter\ used\ for\ longitudinal\ bars)$$

The minimum overlap length is: $50\ \emptyset\ in\ zone\ III$

The spacing between the vertical bars on one face of the column should not exceed: $20\ cm\ in\ zone\ III$

Overlap joints should be made, if possible, outside of nodal zones (critical zones).

The nodal zone is composed of the actual beam-column joint and the ends of the bars that converge there. The lengths to be considered for each bar are given in figure (IV.15):

$h' = Max(\frac{h_e}{6}; b_1; h_1; 60\ cm)$

$l' = 2h$

h_e : is the height of the floor.

$b_1; h_1$: Dimensions of the cross-sectional area of the column.

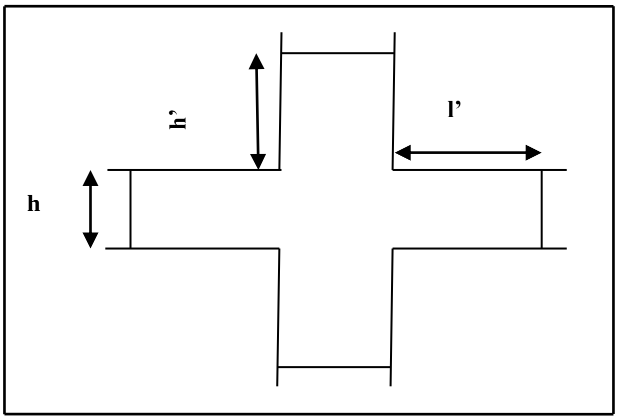


Figure IV.15: nodal zone

The numerical values related to the RPA99v2003 specifications are provided in the following table:

Levels	Section of the column	$A_{min}RPA\ (cm^2)$	$A_{max}RPA(cm^2)$	
			Regular zone	Overlap zone
Ground floor to 4th floor	45 ×60	24.3	108	162

Table IV.18: Minimum and maximum longitudinal reinforcement in columns.

Transverse reinforcements:

The transverse reinforcements of the columns are calculated using the formula:

$$\frac{A_t}{t} = \frac{\rho_a \times V_u}{h_1 \times f_e} \qquad (\boldsymbol{Art.7.4.2.2})$$

With:

V_u: The calculated shear forces.

h_1: Total height of the gross section.

ρ_a : corrective coefficient that considers the brittle shear failure mode, it is taken equal to 2.5 if the geometric slenderness λg in the considered direction is greater than or equal to 5 and 3.75 otherwise.

t: the spacing of transverse reinforcement, the value of which is determined by the previous formula; moreover, the maximum value of this spacing is fixed as follows:

In nodal zone:

$t \leq 10\ cm \qquad in\ zone\ III$

In regular zone:

$t' \leq Min(b_1/2\,, h_1/2\,, 10\ \emptyset_1)$

Where: $\emptyset_1$ is the minimum diameter of the longitudinal reinforcement bars of the column Minimum quantity of transverse reinforcement:

$$\frac{A_t}{t \times b_1}\ \ in\ \%\ is\ given\ as\ follows:$$

$$\begin{cases} A_t^{min} = 0.3\%\ (t \times b_1)\ if\ \lambda_g \geq 5 \\ A_t^{min} = 0.8\%\ (t \times b_1)\ if\ \lambda_g \leq 3 \end{cases}$$

$if: 3 < \lambda_g < 5\ \ interpolate\ between\ the\ previous\ limit\ values$

λ_g : Geometric slenderness of the column l

$\lambda_g = (\frac{l_f}{a}\ or\ \frac{l_f}{b})$

With “a” and “b”: the dimensions of the cross-section of the column in the direction of the considered deformation, and if is the buckling length of the column

The frames and stirrups must be closed by 135° hooks with a minimum straight length of 10 $\emptyset_t$ minimum

IV.10.2 solicitations:

The table below summarized the maximum loads on the columns for each level:

Columns: C6,C53,C56,C74,C73,C54

Levels	$N_{max} \rightarrow M_{cor}$		Nature	$N_{min} \rightarrow M_{cor}$		Nature	$M_{max} \rightarrow N_{cor}$		Nature
	KN	KN.m		KN	KN.m		KN	KN.m	
G floor	-1336.47	-107.16	G+Q+Ey	582.51	18.46	0.8G+Ey	-481.95	108.92	G+Q+Ey
1st floor	-1021.23	-88.49	G+Q+Ey	410.81	26.31	0.8G+Ey	-29.05	93.59	G+Q+Ex
2nd floor	-683.56	-70.41	G+Q+Ey	229.18	33.24	0.8G+Ey	-72.10	82.79	G+Q+Ex
3rd floor	-404.71	-22.83	G+Q+Ey	115.57	33.11	0.8G+Ey	-98.70	65.39	G+Q+Ey
4th floor	-204.59	-13.19	ULS	41.18	25.90	0.8G+Ey	-101.08	62.97	G+Q+Ey

Table IV.19: Maximum solicitations in the columns (C6, C53, C56, C74, C73, C54).

The other columns:

Levels	$N_{max} \rightarrow M_{cor}$		Nature	$N_{min} \rightarrow M_{cor}$		Nature	$M_{max} \rightarrow N_{cor}$		Nature
	KN	KN.m		KN	KN.m		KN	KN.m	
G floor	-969.79	-4.9	G+Q+Ey	582.51	18.46	0.8G+Ey	-481.95	108.92	G+Q+Ey
1st floor	-752.18	7.73	G+Q+Ey	410.81	26.31	0.8G+Ey	-29.05	93.59	G+Q+Ex
2nd floor	-584.29	-10.36	G+Q+Ey	229.18	33.24	0.8G+Ey	-72.10	82.79	G+Q+Ex
3rd floor	-393.51	-10.07	G+Q+Ey	115.57	33.11	0.8G+Ey	-98.70	65.39	G+Q+Ey
4th floor	204.59	-13.19	ULS	41.18	25.90	0.8G+Ey	-101.08	62.97	G+Q+Ey

Table IV.20: Maximum solicitations in the other columns

IV.10.3. Reinforcement:

Longitudinal reinforcement:

Calculation example:

Case of the columns: (C6, C53, C56, C74, C73, C54)

Let:

$b = 45\ cm\ ;\ h = 60\ cm\ ;\ d = 57\ cm\ ;\ f_e = 400Mpa$

Serviceability condition: $\gamma_s = 1.15\ ;\ \gamma_b = 1.5$

Accidental situation: $\gamma_s = 1.15\ ;\ \gamma_b = 1.5$

1st case: $\mathbf{N_{max} \Longrightarrow M_{corr}}$

$$\begin{cases} N_{max} = -1336.47\ \text{kN} \\ M_{corr} = -107.16\ \text{KN.m} \end{cases}$$

Calculation of the eccentricity:

$$e = e_0 + e_2 + e_a$$

With:

e_0 : Resultant eccentricities

e_2 : Second-order effects due to structural deformations that cause eccentricities

e_a : Additional eccentricities reflecting the initial geometric imperfections.

Calculate the eccentricity of the resultant:

$$e_G = \frac{M_{corr}}{N_{max}} = \frac{107.16}{1336.47} = 8.01\ \text{cm}$$

Calculating the additional eccentricity:

$$e_a = \max\left[2\ \text{cm}, \frac{L}{250}\right],\quad \text{L : length of the piece (BAEL A4.3.5)}$$

$$e_a = \max\ [2\text{cm}, 1.22\ \text{cm}]$$

$$e_a = 2\ \text{cm} = 0.02\ \text{m}$$

$e_1 = e_0 + e_a = 0.0801 + 0.02 = 0.1001 \ \text{m}$

Calculating the eccentricity due to second-order:

According to the article (A.4.3.5) of the CBA93, the second-order eccentricity e2 is related to the deformation of the structure. To determine the second-order eccentricity:

$\frac{l_f}{h} \leq \text{Max} \left[15 , 20 \ \frac{e_1}{h}\right]$

$3.56 \leq 15$ cm The eccentricity e2 is determined in a lump-sum manner

$2 = \frac{3\, l^2{}_f}{10^4\, h}(2 + \alpha\phi) = \frac{3 * 2.14^2}{10^4 \,.\, 0.60} \times (2 + 0.83 \times 2\,) = 0.00838\text{m}$

$\phi = 2$

$\alpha = \frac{M_G}{M_Q + M_G} = \frac{30.67}{6.1858 + 30.67} = 0.83$

so :

$e_T = e_1 + e_2 = 0.1001 + 0.00838 = 0.108 \ \text{m} = 10.8\text{cm}$

$e_t \leq \frac{h}{2} = \frac{60}{2} = 30 \ \text{cm.}$

The center of pressure is located inside the section. The following condition must be verified:

$N_u(d - \acute{d}) - M_{uA} \leq \left(0.337 - 0.81\frac{\acute{d}}{h}\right) \times b \times h^2 \times f_{bu}$

ou : $M_{uA} = N_u\left(d - \frac{h}{2} + e_t\right) = 1336.47 \times (0.57 - 0.30 + 0.108) = 505.19 \ \text{KM.m}$

$1336.47 \times 10^3 \times (570 - 30) - 505.19 \times 10^6 \leq (0.337 - 0.81 \times \frac{30}{600}) \times 450 \times 600^2 \times$

14.2

$216.50 KN.m \leq 682.068\text{KN.m}$

Therefore, the section is partially compressed. The calculation method is carried out by assimilation to simple bending.

$\mu_{bu} = \frac{M_{ua}}{bd^2 f_{bc}} = \frac{505.19 \times 10^6}{450 \times (570)^2 \times 14.2} = 0.243 > 0.186 \, Pivot\ B$

$\mu_{bu} < \mu_l = 0.392 \dots\dots\dots\dots\dots A' = 0$

$\alpha = 0.35$

$z = 490.2 mm$

$\rightarrow Z = d\,(1 - 0.4\alpha)$

$Z = 0.57(1 - 0.4 \times 0.35) = 0.490m$

$$A_{u1} = \frac{M_{ua}}{Z\sigma_S}$$

$$A_{u1} = \frac{505.19 \times 10^6}{490.2 \times 348} = 29.6143\ cm^2$$

We return to the combined bending:

$$A_2 = Au_1 - \frac{N_u}{\sigma_{st}} = 2961.43 - \frac{1336.47 \times 10^3}{348} = -879.001mm^2$$

The non-reinforced section resists the applied forces, so a minimum reinforcement is applied

$A_{min}(BAEL) = \max(4\,P, 0.2\%B) = (8.4\,; 5.4)$

$A_{min} = 8.4cm^2$

2nd case: $\mathbf{N_{min}} \Longrightarrow \mathbf{M_{corr}}$

$$\begin{cases} N_{min} = 582.514kN \\ M_{corr} = 18.46\ \ m \end{cases}$$

Calculate the eccentricity of the resultant:

$$e_G = \frac{M_{corr}}{N_{min}} = \frac{18.46}{582.514} = 3.16\ \ cm$$

$e_G \leq \frac{h}{2} = \frac{60}{2} = 25$ cm.

The center of pressure is located inside the section. The following condition must be verified:

$$N_u(d - \acute{d}) - M_{uA} \leq \left(0.337 - 0.81\frac{\acute{d}}{h}\right) \times b \times h^2 \times f_{bu}$$

Où : $M_{uA} = M_G + N_u\left(d - \frac{h}{2}\right) = 30.67 + 582.514(0.57 - 0.30\,) = 187.94KN.m$

$582.514 \times 10^3 \times (570 - 30\,) - 187.94 \times 10^6 \leq (0.337 - 0.81\frac{30}{600}) \times 450 \times 600^2 \times 14.2$

$126.61KN.m \leq$682.068KN.m

So, the section is entirely under tension N and e is the distance between the reinforcements.

$$A_1 = \frac{N_u \times e_2}{(d-d\prime)\sigma_{st}}$$

$$A_2 = \frac{N_u \times e_1}{(d-d\prime)\sigma_{st}}$$

$e_1 = \frac{h}{2} - d \acute{+} e \quad ; \quad e_2 = (d - d') - e_1$

$e_1 = (30 - 3) + 3.16 = 30.16\ cm$

$e_2 = (57 - 3) - 30.16 = 23.84cm$

$A_1 = \frac{582.514 \times 10^3 \times 238.4}{348 \times (570-30)} = 738.99\ mm^2$

$A_2 = \frac{582.514 \times 10^3 \times 301.6}{348 \times (570-30)} = 934.90mm^2$

$\mathrm{As} = \mathrm{A}_1 + \mathrm{A}_2 = \mathbf{16.74}\mathrm{cm}^2$

$A_{min} = \frac{B \times f_{t28}}{f_e} = \frac{600 \times 450 \times 2.1}{400} = 14.17cm^2$

$\mathrm{As} > \mathrm{Amin}$ we take As ; $\mathrm{As} = 16.74\ \mathrm{cm}^2$

3rd case: $\mathbf{M_{max}} \Longrightarrow \mathbf{N_{corr}}$

$\begin{cases} \mathrm{N_{corr}} = -481.95\ \mathrm{kN} \ \ (\text{compression}) \\ \mathrm{M_{max}} = 108.92\ \mathrm{kN.m} \end{cases}$

Calculation of the eccentricity:

$\mathrm{e} = \mathrm{e}_0 + \mathrm{e}_2 + \mathrm{e}_a$

Avec:

e_0 : Resultant eccentricities

e_2 : Second-order effects due to structural deformations that cause eccentricities

e_a : Additional eccentricities reflecting the initial geometric imperfections.

Calculate the eccentricity of the resultant

$\mathrm{e}_0 = \frac{\mathrm{M_u}}{\mathrm{N_u}} = \frac{108.92}{481.95} = 0.225\ \mathrm{m}$

Calculating the additional eccentricity:

$\mathrm{e}_a = \max\left[2\ \mathrm{cm}, \frac{\mathrm{L}}{250}\right]$, L : length of the piece (BAEL A4.3.5)

$e_a = \max[2cm , 1.22\ cm]$

$e_a = 2\ cm = 0.02\ m$

$e_1 = e_0 + e_a = 0.225 + 0.02 = 0.245\ m$

Calculating the eccentricity due to second-order:

According to the article (A.4.3.5) of the CBA93, the second-order eccentricity e2 is related to the deformation of the structure. To determine the second-order eccentricity:

$$\frac{l_f}{h} \leq \mathrm{Max}\left[15 , 20\ \frac{e_1}{h}\right]$$

$3.56 \leq 15$ cm The eccentricity e2 is determined in a lump-sum manner

$2 = \frac{3\ l^2{}_f}{10^4\ h}(2 + \alpha\phi) = \frac{3 * 2.14^2}{10^4 . 0.60} \times (2 + 0.83 \times 2) = 0.00838m$

$\phi = 2$

$\alpha = \frac{M_G}{M_Q + M_G} = \frac{30.67}{6.1858 + 30.67} = 0.83$

So:

$e_T = e_1 + e_2 = 0.245 + 0.00838 = 0.253\ m = 25.3cm.$

$e_t \leq \frac{h}{2} = \frac{60}{2} = 30\ cm.$

The centre of pressure is located inside the section. The following condition must be verified:

$N_u(d - \acute{d}) - M_{uA} \leq \left(0.337 - 0.81\frac{\acute{d}}{h}\right) \times b \times h^2 \times f_{bu}$

$M_{uA} = N_u\left(d - \frac{h}{2} + e_t\right) = 481.95 \times (0.57 - 0.3 + 0.253) = 252.05KN.m$

$481.95 \times 10^3 \times (570 - 30) - 252.05 \times 10^6 \leq (0.337 - 0.81\frac{30}{500}) \times 450 \times 500^2 \times 14.2$

$8.203KN.m \leq 682.068KN.m$

Therefore, the section is partially compressed. The calculation method is carried out by assimilation to simple bending

$$\mu_{bu} = \frac{M_{ua}}{bd^2 f_{bc}} = \frac{252.05 \times 10^6}{450 \times (570)^2 \times 14.2} = 0.121 < 0.186 \; Pivot\ A$$

$$\mu_{bu} < \mu_l = 0.392 \ldots\ldots\ldots\ldots\ldots A' = 0$$

$$\alpha = 0.162$$

$$\rightarrow Z = d\,(1 - 0.4\alpha)$$

$$Z = 0.57(1 - 0.4 \text{ x } 0.162) = 0.533\text{m}$$

$$A_{u1} = \frac{M_{ua}}{Z\sigma_S}$$

$$A_{u1} = \frac{252.05 \times 10^6}{533 \times 348} = 13.58\text{cm}^2$$

We return to the combined bending:

$$A_2 = Au_1 - \frac{N_u}{\sigma_{st}} = 1358 - \frac{481.95}{348} = 13.53\text{cm}^2$$

The non-reinforced section resists the applied forces, so a minimum reinforcement is applied

Amin (BAEL)= 14.17 cm²

Case of the other columns:

1st case: $\boldsymbol{N_{max}} \rightarrow \boldsymbol{M_{cor}}$

$$N_{max} = -969.79 \text{ kN}$$
$$M_{corr} = -4.9\text{KN.m}$$

Calculation of the eccentricity:

$$e = e_0 + e_2 + e_a$$

Avec:

e_0 : Resultant eccentricities

e_2 : Second-order effects due to structural deformations that cause eccentricities

e_a : Additional eccentricities reflecting the initial geometric imperfections.

Calculate the eccentricity of the resultant

$$e_G = \frac{M_{corr}}{N_{max}} = \frac{4.9}{969.79} = 0.505 \text{ cm}$$

Calculating the additional eccentricity:

$$e_a = \max\left[2 \text{ cm}, \frac{L}{250}\right], \quad L: \text{length of the piece (BAEL A4.3.5)}$$

$$e_a = \max[2\text{cm}, 1.22 \text{ cm}]$$

$$e_a = 2 \text{ cm} = 0.02 \text{ m}$$

$$e_1 = e_0 + e_a = 0.00505 + 0.02 = 0.025\text{m}$$

Calculating the eccentricity due to second-order:

According to the article (A.4.3.5) of the CBA93, the second-order eccentricity e2 is related to the deformation of the structure. To determine the second-order eccentricity:

$$\frac{l_f}{h} \leq \text{Max}\left[15, 20\,\frac{e_1}{h}\right]$$

$3.56 \leq 15$ cm The eccentricity e2 is determined in a lump-sum manner

$$2 = \frac{3\, l^2{}_f}{10^4\, h}(2 + \alpha\phi) = \frac{3 * 2.14^2}{10^4 \,.\, 0.60} \times (2 + 0.83 \times 2) = 0.00838\text{m}$$

$$\phi = 2$$

$$\alpha = \frac{M_G}{M_Q + M_G} = \frac{30.67}{6.1858 + 30.67} = 0.83$$

So:

$$e_T = e_1 + e_2 = 0.025 + 0.00838 = 0.0333 \text{ m} = 3.33 \text{ cm}.$$

$$e_t \leq \frac{h}{2} = \frac{60}{2} = 30 \text{ cm}$$

The center of pressure is located inside the section. The following condition must be verified:

$$N_u(d - \acute{d}) - M_{uA} \leq \left(0.337 - 0.81\frac{\acute{d}}{h}\right) \times b \times h^2 \times f_{bu}$$

while : $M_{uA} = N_u\left(d - \frac{h}{2} + e_t\right) = 969.79(0.57 - 0.3 + 0.0333) = 294.14\ \text{KN.m}$

$969.79 \times 10^3 \times (570 - 30) - 294.14 \times 10^6 \leq (0.337 - 0.81 \times \frac{30}{600}) \times 450 \times 600^2 \times 14.2$

$229.54\ KN.m \leq 682.068\ KN.m$

Therefore, the section is partially compressed. The calculation method is carried out by assimilation to simple bending

$\mu_{bu} = \frac{M_{ua}}{bd^2 f_{bc}} = \frac{294.14\times 10^6}{450\times(570)^2\times 14.2} = 0.141 < 0.186\ Pivot\ A$

$\mu_{bu} < \mu_l = 0.392 \ldots\ldots\ldots\ldots\ldots A' = 0$

$\alpha = 1.25(1 - \sqrt{1 - 2 \times 0.141} = 0.19$

$z = d(1 - 0.4\alpha) = 570 \times (1 - 0.4 \times 0.19) = 526.68\ mm$

$A_{u1} = \frac{M_{ua}}{Z\sigma_S}$

$A_{u1} = \frac{294.14\times 10^6}{526.68\times 348} = 1604.83\text{mm}^2 = 16.0483\text{cm}^2$

$$A_2 = Au_1 - \frac{N_u}{\sigma_{st}} = 1604.83 - \frac{969.79 \times 10^3}{348} = -1181.92\text{mm}^2$$

$A = 0\ cm^2$: It is not necessary to add reinforcement, the concrete alone is sufficient

Levels	columns	Section(cm²)	A_{cal}	$A_{min}RPA$	$A_{min}BAEL$	A_{ado}	A_{ado}
Ground-4th floor	C6+C53+C56+C77+C80	2700	9.349	24.3	14.17	10HA20	31.42
Ground-4th floor	The other columns	2700	16.04	24.3	14.17	10HA20	31.42

Table IV.21: Longitudinal reinforcements of the columns.

Transverse reinforcements:

$$\frac{A_t}{t} = \frac{\rho V_u}{h_1 \times f_e}$$

V_u =42.65KN

h_1 =60cm

f_e = 400 MPa

ρ : Corrective coefficient that considers the shear force failure mode such as:

$\rho = 2.5 \;\; if \; \lambda_g \geq 5 \; ; \; \rho = 3.75 \;\; if \; \lambda_g$

λ_g : is the geometric slenderness of the column

$$\lambda_g = \left(\frac{l_f}{a} \text{ ou } \frac{l_f}{b}\right)$$

$$\lambda_g = (3.56 \text{ ou } 4.75\,) = 3.56 < 5$$

$\rho = 3.75$

t : spacing between transverse reinforcement bars such as :

Nodal zone :

t ≤ 10 cm. In zone III

Current zone :

$t \leq Min\;(b_1/2\,, h_1/2\,, 10\;\emptyset_l)\;\; in\; zoneIII$

$t \leq Min\;(22.5{,}30{,}12) \;\; \Longrightarrow t = 10\; cm$

$$A_t = 100 \times \frac{3.75 \times\; 59.08 \times 10^3}{600 \times 450} = 0.821\ \text{cm}^2$$

The minimum quantity of transverse reinforcement

$3 < \lambda_g = 3.56 < 5$

$\Longrightarrow 0.8\%(t \times b) > A_{t\,min} > 0.3\%(t \times b) \Longrightarrow 3.6cm^2 > A_{t\,min} > 1.35\; cm^2$

After linear interpolation we have:

$A_{t\,min} = 2.47\; cm^2$

$A_{t\,min} = \max(A_t\; ;\; A_{t\,min})$

$= 2.47\; cm^2 \;\;\; so\; we\; take\; 6T8 = 3.018\; cm^2$

Levels	Ground floor to fourth floor
Section (cm²)	45×60
$\emptyset_{min}$ **(cm)**	1.4
l_f **(cm)**	214.20
λ_g	3.56
V_u **(KN)**	59.08
l_r **(cm)**	50
$t_{nodal\ zone}$ **(cm)**	10
$t_{current\ zone}$ **(cm)**	10
ρ	3.75
A_t **(cm²)**	0.821
$A_{t\ min}$ **(cm²)**	2.47
$A_{t\ adop}$ **(cm²)**	6HA8 = 3.02

Table IV.22: Transverse reinforcements of the columns.

We adopt for all floors:

According to the RPA99v2003 and BAEL91 rules, the diameter of transverse reinforcement must be greater than one-third of the maximum diameter of the longitudinal reinforcement.

$\phi t \geq \frac{\phi lmax}{3}$ ➔ 8 > 20 /3 =6.67…. Cv.

IV.10.3 verifications:

Verification of ultimate limit state of shape stability:

The columns are subjected to compound bending, for which, according to (Article B.8.4.1.P156) we are required to justify them with respect to the ultimate limit state of stability of shape. The relation to be verified is as follows: the columns have the same slenderness, with a length of $l_0 = 3.06\ m$ and a normal force equal to 1336.47 KN.

We must verify:

$$Nu \leq \overline{N} = \alpha \left[\frac{Br \times fc28}{0.9 \times \gamma b} + A \frac{fe}{\gamma s}\right] \quad \text{CBA93 (Art B.8.4.1)}$$

With:

α : is a coefficient depending on the mechanical slenderness λ

$\lambda = \max(\lambda x, \lambda y)$

$\lambda = \sqrt{12} \times \frac{l_f}{b}$

$$l_f = 0.7 l_0 \qquad \Longrightarrow \quad 0.7 \times 306 = 214.20 \text{ cm}$$

$\lambda = \sqrt{12} \times \frac{214.20}{45b} = 16.48$

$$\lambda = 16.48 < 50 \quad \Longrightarrow \alpha = \frac{0.85}{1 + 0.2\left(\frac{\lambda}{35}\right)^2} = \frac{0.85}{1 + 0.2\left(\frac{16.48}{35}\right)^2} = 0.814$$

Br : is the reduced section of the column obtained by deducting 1cm thickness from its actual section on its entire perimeter.

$B_r = (h-2)(b-2) = 0.2494\ m^2$

$\gamma b = 1.50 \quad ; \quad \gamma s = 1.15$

$A_s = 31.42\ cm^2$

$$\overline{N} = 0.814 \times \left[\frac{0.2494 \times 10^6 \times 25}{0.9 \times 1.5} + 3142 \times \frac{400}{1.15}\right] = 4649.069 KN$$

$$N_u = 1336.47 \text{ KN} < \overline{N}$$

So, there is no risk of buckling.

column	N_u (KN)	l_f (cm)	α	λ	A (mm²)	B_r (mm²)	$\overline{N}$ (KN)	observation
45×60	1336.47	214.20	0.814	16.48	3142	249400	4649.069	Verified

Table IV.23: Verification of ultimate limit state of shape stability

Verification for tangential solicitations:

The conventional design shear stress in the concrete τ_{bu} under seismic combination must be less than or equal to the following limit value:

$$\overline{\tau_{bu}} = \rho_d \times f_{c28}$$

Such that:

$$\rho_d = \begin{cases} 0.075 & \text{si } \lambda_g \geq 5 \\ 0.04 & \text{si } \lambda_g < 5 \end{cases} \qquad \text{RPA99 (Article 7.4.3.2)}$$

$$\tau_{bu} = \frac{V_u}{b_0 \times d}$$

The results are summarized in the following table:

Levels	Section(cm²)	l_f (m)	λ_g	ρ_d	d (cm)	V_u (KN	τ (Mpa)	τ_{bu} (Mpa)	observation
Ground-4th floor	45×60	2.14	3.56	0.04	57	59.08	0.23	1.00	Verified

Table IV.24: Verification for tangential solicitations

Verification of nodal areas:

In order to allow the formation of plastic hinges in the beams and not in the columns, the RPA99v2003 (Art.7.6.2) requires verifying that:

$$|\ M_n\ | + \ |M_S\ | \geq 1.25\ |M_W| \ + |\ M_e\ |$$

However, this verification is optional for the last two levels (building above R+2).

Determination of the moment of resistance:

The moment of resistance (M_R) of a concrete section essentially depends on:

- Dimensions of the concrete section
- The amount of reinforcement in the concrete section
- The elastic limit stress of steels

$M_R = Z \times A_s \times \sigma_s$ With: Z = 0.9 h (h: The total height of the concrete section)

And: $\sigma_s = \frac{f_e}{\gamma_s}$

Beams :

type	h(m)	Z(m)	A_S	A_S	σ_s	M_R	M_E

Main beam with walls	0.40	0.36	18.46	9.23	348	231.2	115.6
Main beam without walls	0.40	0.36	9.23	9.23	348	115.6	115.6
Secondary beam with walls	0.35	0.32	24.61	12.30	348	274.05	136.9
Secondary without wall	0.35	0.32	6.78	6.78	348	75.5	75.5

Table IV.25: Determination of the moment of resistance in the beams

Columns :

Levels	h(m)	Z(m)	A_S	f_e	M_R
Ground ,1st,2nd ,3rd ,4th floor	0.6	0.54	31.4	400	678.2

Table IV.26: Determination of the moment of resistance in the columns

Verification of nodal zones in the main direction:

Levels	Main beam						observation
	M_n	M_s	$M_n + M_s$	M_R	M_E	$1.25\ \lvert M_W\rvert + \lvert M_e\rvert$	
Ground	678.2	678.2	1356.4	231.2	115.6	404.6	**Verified**
1st floor	678.2	678.2	1356.4	231.2	115.6	404.6	
2nd floor	678.2	678.2	1356.4	231.2	115.6	404.6	
3rd floor	678.2	678.2	1356.4	231.2	115.6	404.6	
4th floor	678.2	678.2	1356.4	231.2	115.6	404.6	

Table IV.27: Verification of nodal zones in the main direction main beam

levels	Secondary beam						Observation
	M_n	M_s	$M_n + M_s$	M_R	M_E	$1.25\ \lvert M_W\rvert + \lvert M_e\rvert$	
Ground	678.2	678.2	1356.4	274.05	136.9	479.46	**Verified**
1st	678.2	678.2	1356.4	274.05	136.9	479.46	
2nd	678.2	678.2	1356.4	274.05	136.9	479.46	
3rd	678.2	678.2	1356.4	274.05	136.9	479.46	
4th	678.2	678.2	1356.4	274.05	136.9	479.46	

Table IV.28: Verification of nodal zones in the main direction secondary beam

IV.10.4 Reinforcement diagrams for the columns:

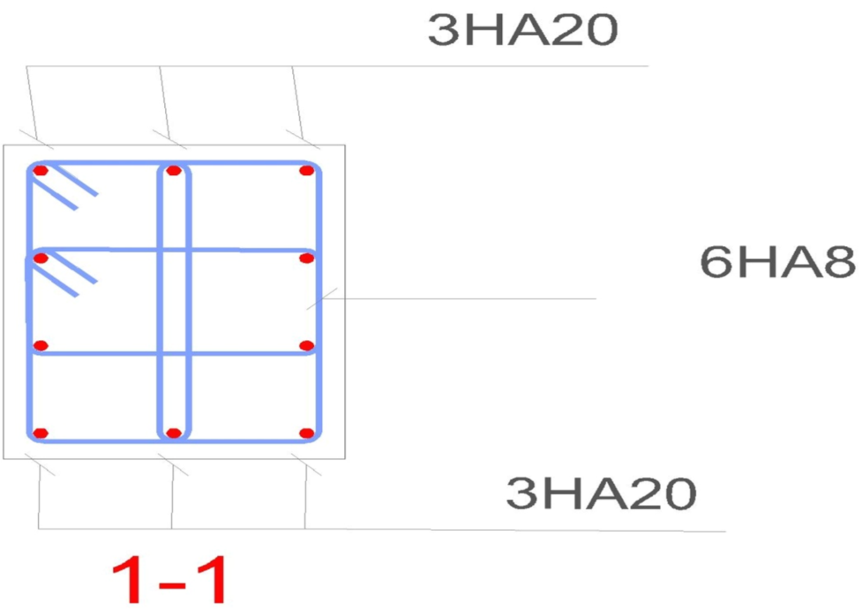


Figure IV.16: Columns reinforcement's diagram

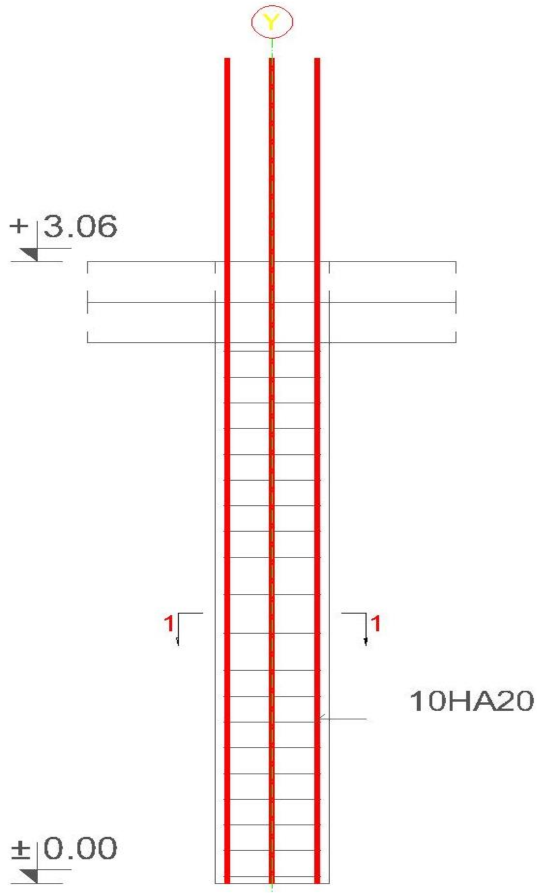


Figure IV.17: Longitudinal section of column reinforcement

IV.11 Study of shear walls:

Shear walls or bracing walls can be defined as vertical elements intended to withstand, in addition to vertical loads (at most 20 %), horizontal forces (at most 75%) for mixed bracing, and thanks to their significant stiffness in their plane. They have two planes, one of low inertia and the other of high inertia, which requires a layout in both directions (x and y).

The shear wall is considered as a cantilever fixed at its base, subjected to combined bending with a shear force, hence we can cite the main modes of failure in a slender shear wall caused by these stresses:

Failure by bending

Failure in bending by shear force

Failure by crushing or tensile stress of the concrete

The calculation will be done according to the usual calculation combinations:

$$\begin{cases} 1.35G + 1.5Q \\ G + Q \pm Ex, y \\ 0.8G + Ex, y \end{cases}$$

IV.11.1 Recommendations of RPA:

According to the requirements of RPA99.The reinforcement of the walls is composed of vertical reinforcements, horizontal reinforcements and distribution reinforcements.

Vertical reinforcements: R.P.A 99 (A7.7.4.1):

When part of the wall is stretched under the action of vertical and horizontal forces, the tensile force must be taken up entirely by the reinforcements.

The minimum percentage of vertical reinforcement over the entire stretched area is 0.20%.

It is possible to concentrate the tensile reinforcements at the end of the wall or the pier, the total section of vertical reinforcements of the tense zone having to remain at least equal to 0.20% of the horizontal section of the concrete in tension.

If significant compressive forces act on the end of the wall, the vertical bars must comply with the conditions imposed on the posts.

The vertical bars of the last level must be equipped with a hook at the top.

All the other bars have no hooks (joint only by lap).

At each end of the wall or pier the spacing of the bars must be at most equal to 15cm.

The vertical bars of the extreme zones should be bound horizontal frames whose spacing should not be greater than the thickness of the wall.

Horizontal reinforcement: RPA 99 (art 7.7.4.2)

The horizontal bars must be equipped with hooks at 135° having a length of 10Φ. If there are stiffness heels, the horizontal bars must be anchored without a hook if the dimensions of the heels allow for a straight anchor.

Common rules: R.P.A 99 (art 7.7.4.3)

The minimum percentage of vertical and horizontal reinforcement in the piers is given as follows:

Overall in the veil section: 0.15%

In current zone: 0.10%

The spacing of the vertical and horizontal bars must be less than the smaller of the following values:

$\Longrightarrow \quad \begin{cases} s_t \leq 1.5a \\ s_t \leq 30cm \end{cases} \Longrightarrow$ St ≤ Min (30cm; 1.5a), with a: wall spacing

The two layers of reinforcement must be connected together with at least four (4) pins per square meter; their main role is to connect the two layers of reinforcement so as to ensure their stability during the pouring of the concrete.

The diameter of the vertical and horizontal bars of the sails (with the exception of the end zones) must not exceed 1/10 of the thickness of the sail.

The lap lengths must be equal to:

40 Φ: for bars located in areas where overturning designates efforts is possible

20Φ: for bars located in areas compressed under the action of all possible load combinations.

Along the casting joints, the shear force must be taken for the seam steels, the section of which must be calculated with the following formula: $A_w = 1.1\frac{V}{F_e}$

With: V=1.4$V_{calculated}$

IV.11.2. Calculation example:

The shear wall is calculated vertically along its mean plane for combined bending and shear. Our example will be limited to the "P1" wall with a section of (20×200 cm²) which is the most solicited under accidental combinations.

According to Article 7.7.4 of RPA99v2003, the calculation of the vertical and horizontal reinforcement of the walls is done in the direction of their average plane by applying the classical rules of reinforced concrete.

Calculation of reinforcement for combined bending:

M_{max} = 1072.2743 KN.m

N_{corr} = -190.2306 KN

M_Q = 0.692 KN.m

M_G = 5.0472 KN.m

M_{ser} = 5.7392 KN.m

N_{ser} = -363.8196 KN

H_e = 2.71 m (clear height)

First order eccentricity:

$e_1 = \frac{Mmax}{Ncorr} + e_a$

e_a : Additional eccentricity reflecting initial geometric imperfections after construction.

$e_a = \max\left(2\ cm\ ;\ \frac{He}{250}\right) \rightarrow e_a = 2\ cm = 0.02\ m$

$e_1 = \frac{1072.2743}{190.2306} + 0.02 = 5.657\ m$

Ultimate load corrected by buckling:

$l_f = l_0 = 2.76\ m$

$\lambda_g = \frac{l_f}{h} = \frac{2.76}{2} = 1.38$

$\lambda_g >< \max\left(15; \frac{20\ e_1}{h}\right) = \max(15\ ; 56.57) \rightarrow \lambda_g = 1.38 < \ 56.57$

So, the calculation will be carried out for combined bending, taking into account the second-order eccentricity in a fixed manner.

2nd order eccentricity:

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

e_2: Eccentricity due to second order effects related to the deformation of the structure.

$\alpha = \frac{M_G}{M_Q + M_G} = \frac{5.0472}{5.0472+0.692} = 0.87$

$\varphi = 2$ General case.

Ultimate stress corrected by calculation in compound bending:

$e_0 = e_1 + e_2 = 5.657 + 0.004 = 5.661\ m$

$M = N \times e_0 = 190.2306 \times 5.661 = 1076.895\ KN.m$

Stress brought back to the center of gravity of the tense steels:

$e_A = e_0 + \left(d + \frac{h}{2}\right) = 5.661 + \left(1.8 - \frac{2}{2}\right) = 6.46\ m$

with d = 0.9 * 2 = 1.8 m (height)

$M_{uA} = N \times e_A = 190.2306 \times 6.46 = 1228.889\ KN.m$

At serviceability limit state (SLS):

$N_{ser} = -363.8196$

$M_{ser} = 5.7392$

$e_{0ser} = \frac{M_{ser}}{N_{ser}} = \frac{5.7392}{363.8196} = 0.016m$

Stress brought back to the center of gravity of the tense steels:

$e_A = e_{0\ ser} + \left(d - \frac{h}{2}\right) = 0.016 + (1.8 - 1) = 0.816\ m$

$M_{ser\ A} = N_{ser} \times e_A = 363.8196 \times 0.816 = 296.876\ m$

Type de section pour le calcul des armatures longitudinales :

$\mu_{bc} = 0.8\frac{h}{d}\left(1 - 0.4\frac{h}{d}\right) = 0.8\ \frac{2}{1.8}\left(1 - 0.4\frac{2}{1.8}\right) = 0.494$

$\mu_{bu} = \frac{M_{uA}}{b \times d^2 \times f_{bu}} = \frac{1228.889 \times 10^6}{200 \times 1800^2 \times 14.2} = 0.133$

$$\mu_{bu} = 0.133 < \mu_{bc} = 0.494 \rightarrow \boldsymbol{partially\ stretched\ section}$$

So, the calculation is done in simple bending

$\mu_{bu} = 0.133 < 0.186 \rightarrow A' = 0$

$\alpha = 1.25\left(1 - \sqrt{1 - 2 \times \mu_{bu}}\right) = 0.179$

$z = d(1 - 0.4\alpha) = 1.8 \times (1 - 0.4 \times 0.179) = 1.67\ m$

$A_1 = \frac{M_{uA}}{z \times \frac{f_e}{\gamma_s}} = \frac{1228.889 \times 10^6}{1670 \times 348} = 2114.545\ mm^{\ 2} = 21.14 cm^2$

Reinforcements in compound bending:

$A = A_1 - \frac{N_u}{f_{su}} = 2114.545 - \frac{190.2306 \times 10^3}{348} = 1567.905\ mm^2 = 15.67\ cm^2$

$A' = 0$: It is not necessary to put reinforcements; the concrete alone will suffice.

The reinforcement must be arranged so that the center of gravity of the reinforcement corresponds to a useful height of 1.8 m.

The "Lt" zone in which the reinforcement must be placed is determined:

$L_t = (2 - 1.8 - 0.03) \times 2 = 0.34\ m$

In 0.34 m if we have reinforcements spaced 10 cm apart, we will have:$3 \times 2 = 6\ bars$

We will therefore have:$\frac{15.67}{6} = 2.61 cm^2$ Which corresponds to bars in HA 20

$(A_{HA20} = 3.14\ cm^2)$

Minimum section according to RPA99v2003:

In the towed area:

$A_{min-1} = 0.2\%\ b \times h = 0.0020 \times 20 \times 100 = 4\ cm^2$

Overall in the wall:$A_{min-2} = 0.15\%\ b \times h = 0.00150 \times 20 \times 100 = 3\ cm^2$

We adopt for the current zone: T12 spaced with 20 cm for 1ml we have $2 \times 5 \times 1.13 = 11.3\ cm^2$.)

walls	section	M_{max}	N_{corr}	A_{calcul}	A_{min}	$l_t$zone		Current zone	
						Choice of bars	st	Choice of bars	st
Vy1	(20*200)	1072.2743	190.2306	15.67	3	6HA20	10	5T12	20
Vy2	(20*200)	1062,799	60,9736	19.50	3	6HA20	10	5T12	20
Vy3	(20*200)	658.4059	82.1997	12.61	3	6HA20	10	5T12	20
Vy4	(20*200)	651.2683	165.9453	14.35	3	6HA20	10	5T12	20
Vx1 (p5)	(20*200)	982.8803	158.1085	20.39	3	8HA20	10	5T12	20
Vx2 (p6)	(20*200)	964.4883	283.3295	23.42	3	8HA20	10	5T12	20

Table IV.29: Reinforcements of the shear walls.

Calculate the shear force horizontally in plane 1-2:

We hang the maximum shear force applied to the section of the same veil p1 under accidental combinations:

$V_{max} = 311{,}0317\ KN$

Vérification de la contrainte de cisaillement :

According to RPA99V2003 (Art 7.7.2) the limit shear stress in concrete for spandrel and piers is given by the formula:

$$\tau_b = \frac{\bar{V}}{b_0 \times d} \leq \bar{\tau_b} = 0.2 \times f_{c28} = 5\ Mpa$$

With:

$\bar{V} = 1.4 \times V_{max}$

b_0 : Wall thickness

d : Useful height $= 0.9 \times h = 0.9 \times 2 = 1.8\ m$

h : total height of gross section

We will therefore have:

$\tau_b = \frac{\bar{V}}{b_0 \times d} = \frac{1.4 \times 311.0317 \times 10^3}{200 \times 1800} = 1.21\ Mpa$

$\tau_b \leq \bar{\tau_b}$ ... ... $v.c$

Calculation of transverse reinforcements:

First, we set the spacing:

$S_t \leq \min(1.5 \times a; 30cm) = 30cm$

We take $S_t = 20\ cm$

We now calculate the transverse reinforcement section:

According to BAEL91 (article A.5.1.23)

$\frac{A_t}{b_0 \times S_t} \geq \frac{\tau_u - 0.3 f_{tj} k}{0.8 f_e (cos\alpha + sin\alpha)}$

τ_u : Conventional tangent constraint.

With:

$\tau_u = \frac{V_{max}}{b_0 \times d} = \frac{311.0317 \times 10^3}{200 \times 1800} = 0.86\ Mpa$

S_t : Spassing.

f_{tj} : Tensile strengths.

f_e : Elastic limit.

$k = 1 + 3\frac{\sigma_{cr}}{f_{cj}}$, in compound bending with compressionσ_{cr} designating the average stress compression of the total section of the concrete, under the normal force of calculation

So: $k = 1 + 3\frac{N_u}{B \times f_{cj}}$

$N_u = 420{,}901\ KN$

$B = 0.2 \times 2 = 0.4\ m^2$

$k = 1 + 3\frac{420.901 \times 10^3}{400000 \times 25} = 1.13$

α : Angle of inclination of transverse reinforcement ($\alpha = 90°$)

$A_t \geq b_0 \times S_t \times \frac{\tau_u - 0.3 f_{tj} k}{0.8 f_e (cos\alpha + sin\alpha)} = 200 \times 200 \times \frac{0.86 - 0.3 \times 2.1 \times 1.13}{0.8 \times 400 \times (cos90 + sin90)} = 1.85 \times 10^{-5}\ cm^2$

So $A_t \approx 0$ we adopt the minimal section

Regarding the minimum section

According to RPA99-2003:

For a strip of 1 ml

$A_{t-min} = 0.15\% \times 20 \times 100 = 3\ cm^2$

According to BAEL 91 (Article A.5.1.1.22) :

$\frac{A_{min}}{b_0 \times S_t} = \frac{0.4}{f_e} \rightarrow A_{mi,} = \frac{0.4}{400} \times 20 \times 20 = 0.4\ cm^2$

We adopted: $A_{t-min-RPA} = 3\ cm^2$

So, we adopt a horizontal reinforcement in T10 with a spacing of 20 cm, either a section of steel of (100/20= 5) 5*2*0.785=7.85cm²

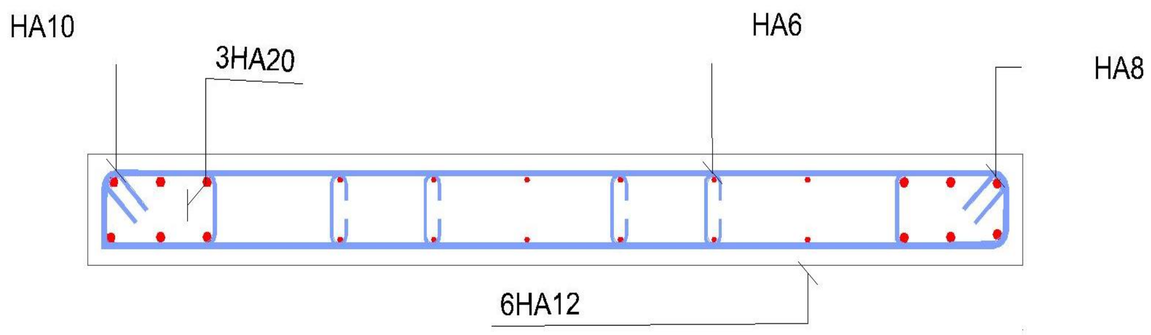


Figure IV.18: Shear walls (Vy) reinforcement diagram.

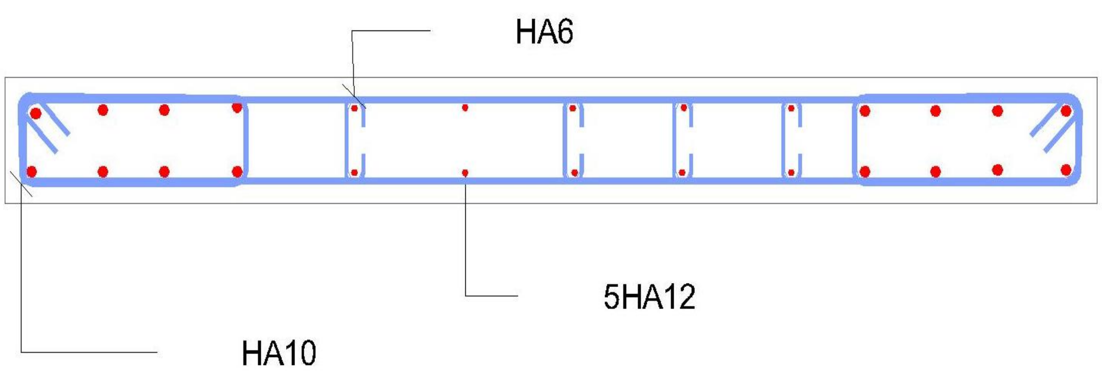


Figure IV.19: Shear walls (Vx) reinforcement diagram.

IV.12. Conclusion:

In this part, the structural elements (columns, beams, walls) are reinforced according to various regulations (RPA 99V2003, BAEL91 modified99).

The maximum solicitations for the three elements (columns, beams, walls) are extracted from the ETABS software in order to calculate the maximum reinforcements.

We used ROBOT expert to calculate the reinforcement of the beams to simplify and accelerate the work.

For economic reasons, we adopted different reinforcement section for the beams (main beams with/without walls, secondary beams with/without walls).

For the columns, we generated the reinforcement section adopted for all columns.

For the shear walls, we changed the reinforcement section in Y-direction for economic reasons.

Chapter V:

Non linear analysis:

V.1 INTRODUCTION:

During an earthquake, structures experience various degrees of deformation and stress distribution. Linear analysis assumes that the structural response remains within the elastic range, meaning the deformation is directly proportional to the applied load. However, in reality, structures exhibit nonlinear behavior when subjected to large displacements and forces.

Nonlinear analysis plays a significant role in providing a more accurate and realistic understanding of the behavior of structures under various loading conditions., This advanced analysis technique allows engineers to evaluate the performance and safety of structures more comprehensively, particularly in situations involving large deformations, material nonlinearity, and complex structural behavior.

In this chapter we are going to perform both non linear time history and static pushover analysis.

Our study will be based on the application of the nonlinear analysis to understand well the behavior of the structure in the nonlinear field and to determine the critical zones which are fragile and likely to undergo damage by the determination of the index of damage to this structure.

PART I: NON LINEAR STATIC ANALYSES:

V. 2 STATIC PUSHOVER ANALYSES:

Pushover analysis, also known as nonlinear static analysis or capacity spectrum method, is a simplified yet effective technique to capture the nonlinear behavior of structures. It involves gradually applying increasing lateral forces or displacements to the structure and assessing its response at each stage. The goal is to determine the ultimate capacity of the structure, identify potential failure modes, and evaluate the distribution of forces throughout the system.

Pushover analysis provides several benefits in seismic design and evaluation:

1. Nonlinear behavior: It captures the actual nonlinear behavior of structures and provides insights into potential failure mechanisms.
2. Performance-based design: It enables performance-based seismic design by evaluating the structure's response beyond code-prescribed limits.
3. Retrofitting and strengthening: Pushover analysis can aid in identifying vulnerable elements and regions in existing structures, facilitating retrofitting and strengthening efforts.
4. Design optimization: It allows assessing and comparing the performance of different design alternatives and selecting the most suitable one.

V.2.1 Non linear static procedures:

V.2.1.1. Capacity curve:

The capacity curve is generated by incrementally applying increasing lateral forces or displacements to the structure. At each step, the structural response is computed, typically in terms of a performance parameter such as base shear, roof drift, or displacement. The analysis continues until the structure reaches the target displacement or a predefined performance limit. Once the pushover curve has been obtained, we seek to transform it into an equivalent capacity curve linking the acceleration of a structure with one degree of freedom to its displacement “Acceleration spectrum Sa – Displacement spectrum Sd”.

The axes of the capacity curve must therefore be transformed in order to have the same units:

Base reaction / mass = acceleration

Displacement / Modal participation factor = displacement

$$\begin{cases} \mathbf{S_{ai}} = \dfrac{V/W}{\propto_1} \\ \mathbf{S_{di}} = \dfrac{\delta_i}{(PF_{1\times}\varphi_{1,roof})} \end{cases}$$

$\mathbf{W}$: Seismic mass of the structure.

$\boldsymbol{\alpha_1}$: The modal mass coefficients

$\mathbf{PF_1}$: The modal participation factor of the first mode of the structure

$\boldsymbol{\varphi_{1,roof}}$: Amplitude of the first mode of vibration at the top (Amplitude of the fundamental mode)

$$\mathbf{PF_1} = \frac{\sum_{i=1}^{n}(w_i\varphi_{i,1})/g}{\sum_{i=1}^{n}(w_i{\varphi_{i,1}}^2)/g}$$

And $$\alpha_1 = \frac{(\sum_{i=1}^{n}\frac{w_i\varphi_{i,1}}{g})^2}{\sum_{i=1}^{n}(w_i/g)\sum_{i=1}^{n}(w_i{\varphi_{i,1}}^2)/g}$$

$\mathbf{w_i}$: Level weight.

$\boldsymbol{\varphi_{i,1}}$: The modal amplitude at level, for mode 1.

$\mathbf{g}$: is gravity.

V.2.1.2. Lateral Load Distribution for NSP

Lateral loads shall be applied to the mathematical model in proportion to the distribution of mass in the plane of each floor diaphragm. The vertical distribution of these forces shall be proportional to the shape of the fundamental mode in the direction under consideration.

V.2.1.3. Target displacement:

The target displacement refers to the maximum displacement that the structure is expected to achieve during a seismic event. It is an important parameter used to define the performance objectives of the structure and to assess its seismic capacity.

The target displacement is typically specified based on the design criteria and the desired performance level of the structure. The performance level can vary depending on factors such as the importance of the structure, its occupancy, and the local building codes or design standards.

V.2.1.3.1. According to ASCE41-13:

Determine the performance point by the method of coefficients:

ASCE41-13 proposes to stop the loading when a target displacement δt is reached

(At control point) defined according to: $\delta t = C_0 C_1 C_2 S_a \frac{{T_e}^2}{4\pi^2} g$

Where:

S_a = Response spectrum acceleration at the effective fundamental period and damping ratio of the building in the direction under consideration

g = acceleration of gravity;

C_0 = Modification factor to relate spectral displacement of an equivalent single-degree-of-freedom (SDOF) system to the roof displacement of the building multidegree-of-freedom (MDOF) system calculated using one of the following procedures:

The first mode mass participation factor multiplied by the ordinate of the first mode shape at the control node;

The mass participation factor calculated using a shape vector corresponding to the deflected shape of the building at the target displacement multiplied by

Ordinate of the shape vector at the control node; or the appropriate value from (Table 7-5 ASCE41-13)

$C1$ = Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response.

Te = Effective fundamental period of the building in the direction under consideration, in seconds

$$T_e = T_i \sqrt{\frac{K_i}{K_e}}$$

Where:

T_i= Elastic fundamental period (in seconds) in the direction under consideration calculated by elastic dynamic analysis.

K_i = Elastic lateral stiffness of the building in the direction under consideration.

K_e= Effective lateral stiffness of the building in the direction under consideration.

$C2$ = Modification factor to represent the effect of pinched hysteresis shape, cyclic stiffness degradation, and strength deterioration on the maximum displacement response.

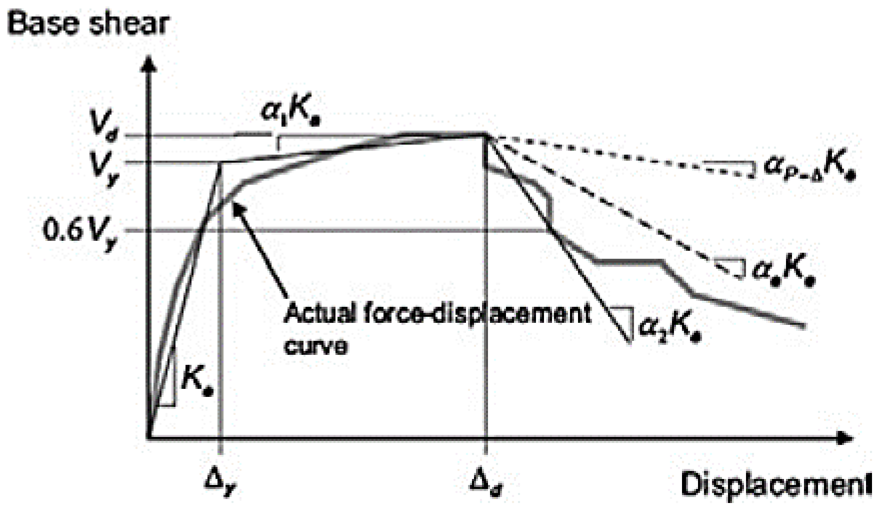


Figure V.1: Idealized Force–Displacement Curves

- **General Requirements for NSP*:***

The requirement to carry out the analysis to at least 150% of the target displacement is meant to encourage the engineer to investigate likely building performance and behavior of the model under extreme load conditions that exceed the analysis values of the Seismic Hazard Level under consideration. The engineer should recognize that the target displacement represents a mean displacement value for the selected Seismic Hazard Level and that there is considerable scatter about the mean. Estimates of the target displacement may be unconservative for buildings with low strength compared with the elastic spectral demand.

V.2.1.3.2. According to ATC-40:

Performance point allows evaluating the maximum displacement that the structure will undergo and subsequently its degree of penetration in the plastic domain. The calculation of the performance point requires the availability of two distinct curves: the seismic stress (demand) curve and the capacity curve derived from a nonlinear static analysis. We compare the capacity curve with the demand curve to determine the point of intersection. This intersection point can be considered as a performance point and provides information on the spectral acceleration (forces) and displacement observed for a given earthquake.

1. **Seismic demand spectra:**

Seismic demand spectra are elastic response spectra presented in Acceleration Spectrum vs. Displacement Spectra (A-D) format. These elastic seismic demand spectra are obtained using the following formula: $S_{di} = \frac{{T_i}^2}{4\pi^2} S_{ai} g$

After converting the response spectrum in ADRS format, we go to an inelastic spectrum scaled down

- **The inelastic seismic demand (inelastic spectrum):** is obtained by reducing the elastic response spectrum damped to 5% by factors which depend on the effective damping of the structure.

a- Determination of inelastic seismic demand:

A bilinear representation of the capacity curve is needed to estimate the effective damping (β_{eff}), it requires the definition of the point (a_{pi}, d_{pi}), which is usually defined using the principle of equality of maximum displacements. To construct the bilinear representation of the capacity curve, the first segment is drawn from the origin with a slope corresponding to the initial stiffness of the structure (elastic segment). The second segment is drawn in connecting point (a_{pi}, d_{pi}) to a point (a_y, d_y) which is defined so as to have equal areas.

b- Estimation of damping and reduction of 5 percent damped response spectrum:

The damping that occurs when earthquake ground motion drives a structure into inelastic range can be viewed as a combination of viscous dumping that is inherent in the structure and hysteretic damping. Hysteretic damping is related to the area inside the loops that are formed when the base shear is plotted against the structure displacement.

The equivalent viscous damping β_{eq}, associated with a maximum displacement of d_{pi}, can be estimated from the following equation:

$$\beta_{eq} = \lambda\beta_0 + 0.05$$

Where:

$$\beta_0 = \frac{1}{4\pi}\frac{E_D}{E_{S0}} = \frac{63.7(a_y d_{pi} - a_{pi} d_y)}{(a_{pi} d_{pi})}$$

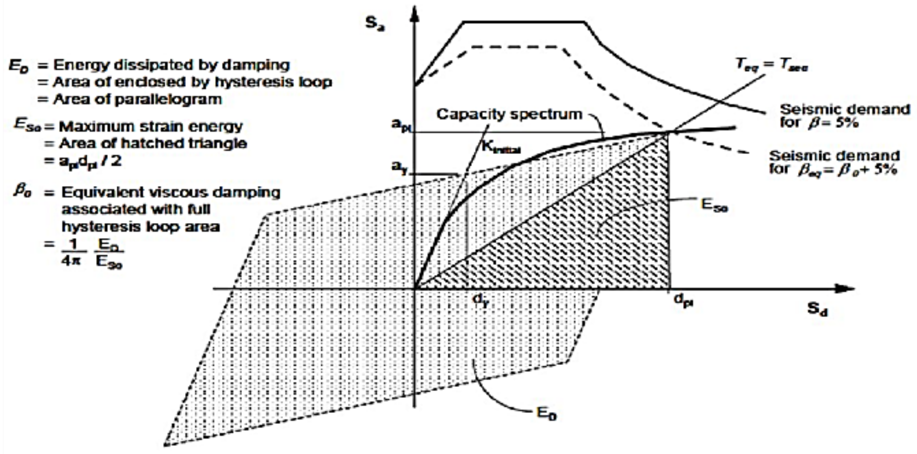


Fig V.2: derivation of damping for spectral reduction

λ : Depends on the structural behavior of the building

Structural behavior type	β_0	λ
Type A	≤ 16.25 > 16.25	1 $\frac{1.13 - 0.51(a_y d_{pi} - a_{pi} d_y)}{a_{pi} d_{pi}}$
Type B	≤ 25 > 25	0.67 $\frac{0.845 - 0.446(a_y d_{pi} - a_{pi} d_y)}{a_{pi} d_{pi}}$
Type C	Any value	0.33

Table V.1 values for damping modification factor

The selection of structural behavior type depends on both the quality of the primary elements of the seismic resisting system and the duration of shaking as shown in **TableV.2**

Shaking duration	Essentially new building	Average existing building	Poor existing building
short	Type A	Type B	Type C
long	Type B	Type C	Type C

Table V.2 Structural behavior types

$\boldsymbol{S_{Ra}}$ *and* $\boldsymbol{S_{Rv}}$ Are the spectral reduction factors formulated as follows:

$$\begin{cases} S_{Ra} = \dfrac{3.21 - 0.681 ln\beta_{eff}}{2.12} \\ S_{Rv} = \dfrac{2.31 - 0.41 ln\beta_{eff}}{1.65} \end{cases}$$

The soil seismic coefficients Ca and Cv used to define the response spectrum depend on the mechanical properties of the soil and its peak acceleration (PGA). The classification of sites has been essentially based on the speed of the shear waves passing through this soil.

The seismic site coefficients Ca and Cv according to the American code (ATC-40) are given in the tables below for each type of ground classified according to the same American code. An equivalence was established between the classifications of soils according to ATC-40 and RPA(2003) by comparing the speeds of the shear waves according to different values of the zone acceleration coefficient to thus allowing the determination of these coefficients for the type of ground serving as a base for the structure under study. The results are written below:

Soil type	V_s	Shaking intensity				
		Z=0.075	Z=0.15	Z=0.20	Z=0.30	Z=0.40
S_a	760-1500	0.08	0.15	0.20	0.30	0.40
S_B	360-760	0.09	0.18	0.24	0.33	0.40
S_D	180-360	0.12	0.22	0.28	0.36	0.44
S_E	< 180	0.19	0.30	0.34	0.36	0.36

Table V.3 Seismic coefficient Ca

Soil type	V_s	Shaking intensity				
		Z=0.075	Z=0.15	Z=0.20	Z=0.30	Z=0.40
S_a	760-1500	0.08	0.15	0.20	0.30	0.40
S_B	360-760	0.13	0.25	0.32	0.45	0.56
S_D	180-360	0.18	0.32	0.40	0.54	0.64
S_E	< 180	0.26	0.50	0.64	0.84	0.96

Table V.4 seismic coefficient Cv

V.2.1.4. Performance point:

Performance point allows us to assess the maximum displacement that the structure will undergo and subsequently its degree of penetration into the plastic range.

In the ATC 40, three procedures (A, B, C) are proposed for the determination of the performance point , procedures A and B are analytical methods, based on mathematical formulas, while procedure C is graphical.

In our case we choose method A for its accuracy and clarity.

- ✓ **Procedures of method A :**

1 Develop the 5% damped response spectrum (elastic).

2 Transform the capacity curve into a capacity spectrum using the relations cited previously.

3 Select a trial performance point$a_{pi}d_{pi}$

4 Develop a bilinear representation of the capacity spectrum.

5 Calculate the spectral reduction factors and plotting the elastic response spectra and inelastic (reduced) as well as the capacity spectrum in the same graph.

6 We check if the point of intersection between the reduced response spectrum and the capacitance spectrum is acceptable, with:

$0.95d_{pi} \leq d_i \leq 1.05d_{pi}$ …………(*)

7 If the condition (*) is not verified, a new point is selected (a_{pi+1}, d_{pi+1})

With: $d_{pi+1} = \frac{d_{pi}+d_i}{2}$

And we return to step 4.

8 If the condition (*) is verified then the test performance point is the performance point.

V.2.2 Results of nonlinear static analysis:

V.2.2.1 Push over simulation:

The pushover analysis is performed by applying an incrementally increasing lateral force distribution (triangular) to the structure until the displacement at the top reaches the defined target displacement.

We opted to use new software known as Seismostruct 2022 which is a finite element package for structural analysis, capable of predicting the large displacement behavior of space frames under static or dynamic loadings, considering both geometric nonlinearities and material inelasticity.

The software consists of three main modules: a **Pre-Processor**, in which it is possible to define the input data of the structural model, a **Processor,** in which the analysis is carried out, and finally a **Post-Processor**, to output the result; all is handled through a **completely visual interface.**

No input or configuration files, programming scripts or any other time-consuming and complex text editing are required.

The processor, moreover, features real-time plotting of displacement curves and deformed shape of the structure, together with the possibility of pausing and re-starting the analysis, whilst the post-processor offers advances post-processing facilities, including the ability to custom-format all derived plots and deformed shapes, thus increasing productivity of users; it is also possible to create AVI movie files to better illustrate the sequence of structural deformation.

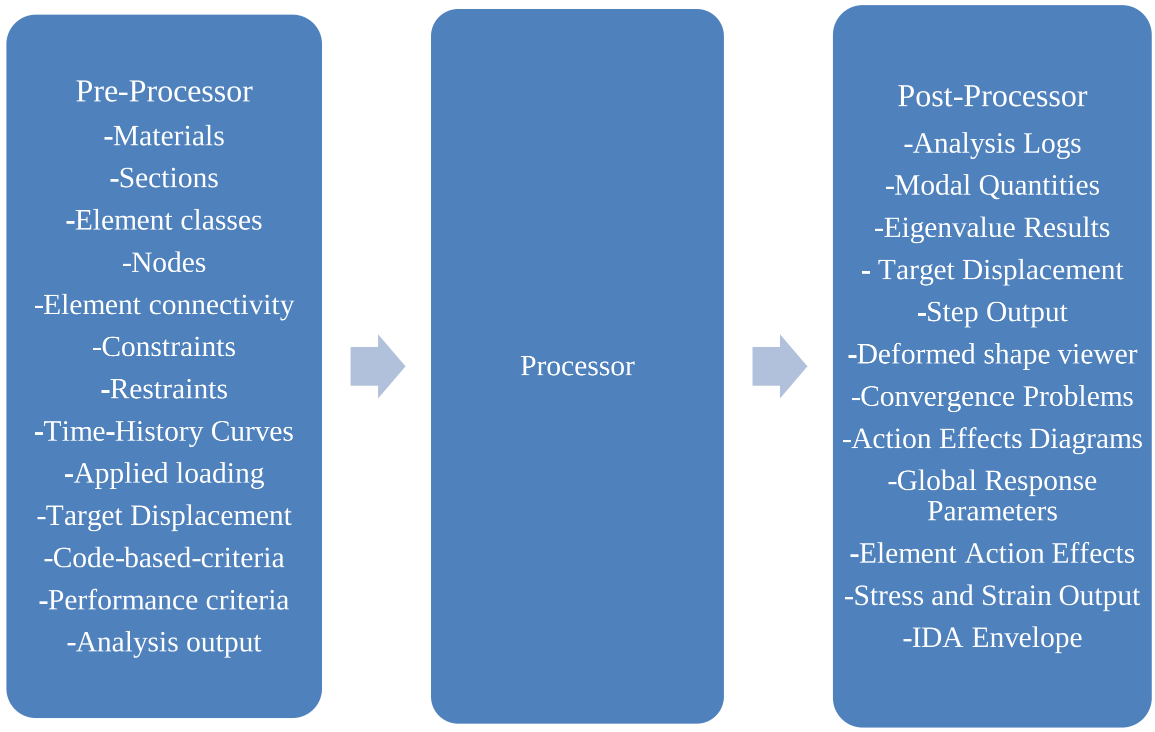


Figure V.3: Structure of the software

V.2.2.1.1. Introduction information steps:

- **Getting started: a new project**

After you open a new file first of all, select static pushover analysis from the drop-down menu at the top left corner on the Pre-Processor window (see picture below). Once the type of analysis has been selected, you can start to create the model.

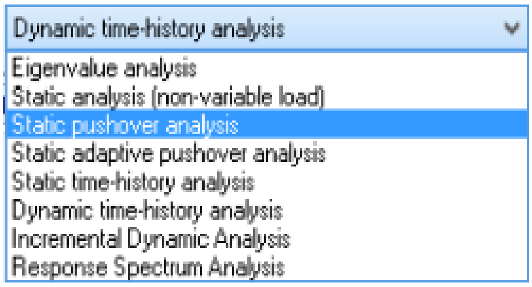


Figure V.4: Selection of the analysis type

- **Pre-Processor – Materials:**

The Materials module is the first module you have to fill in. You have two options of inserting a new material: (i) clicking on the Add Material Class button in order to select a predefined material class or (ii) clicking on the Add General Material button if you are interested in defining all the material parameters.

So, in this step, we introduce the different materials required in our analysis. The materials to be introduced are:

- Concrete ⟶ the concrete type is **con_ma** (Mander et al).
- Steel ⟶ the steel type is **stl_mp** (Menegotto et pinto).

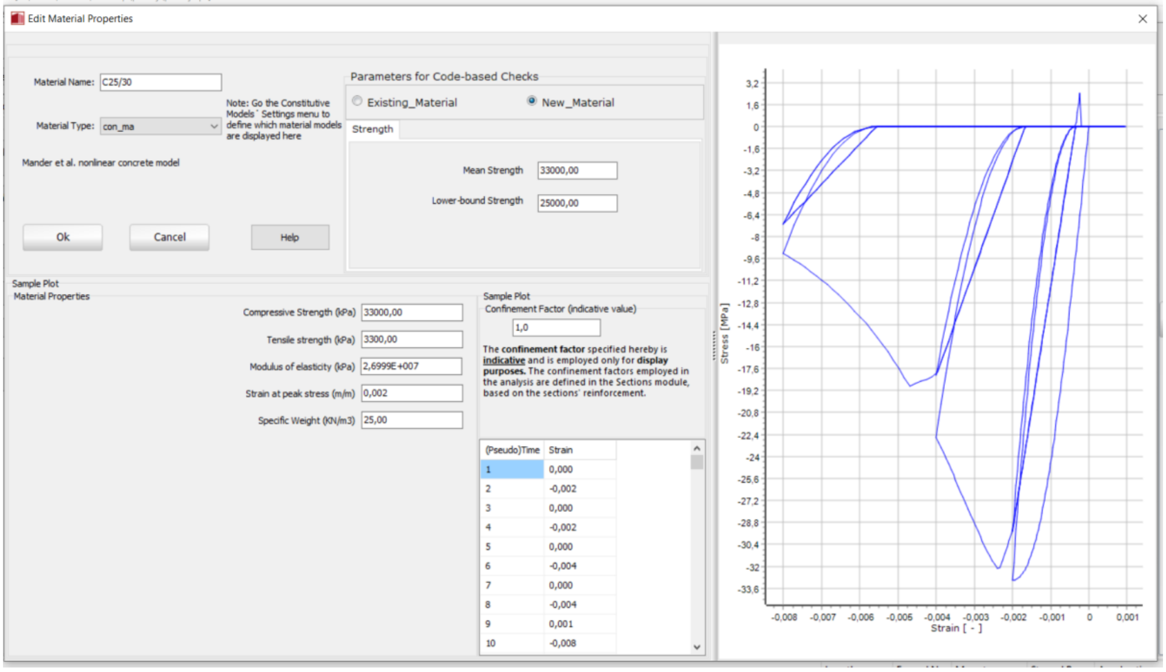


Figure V.5: concrete material

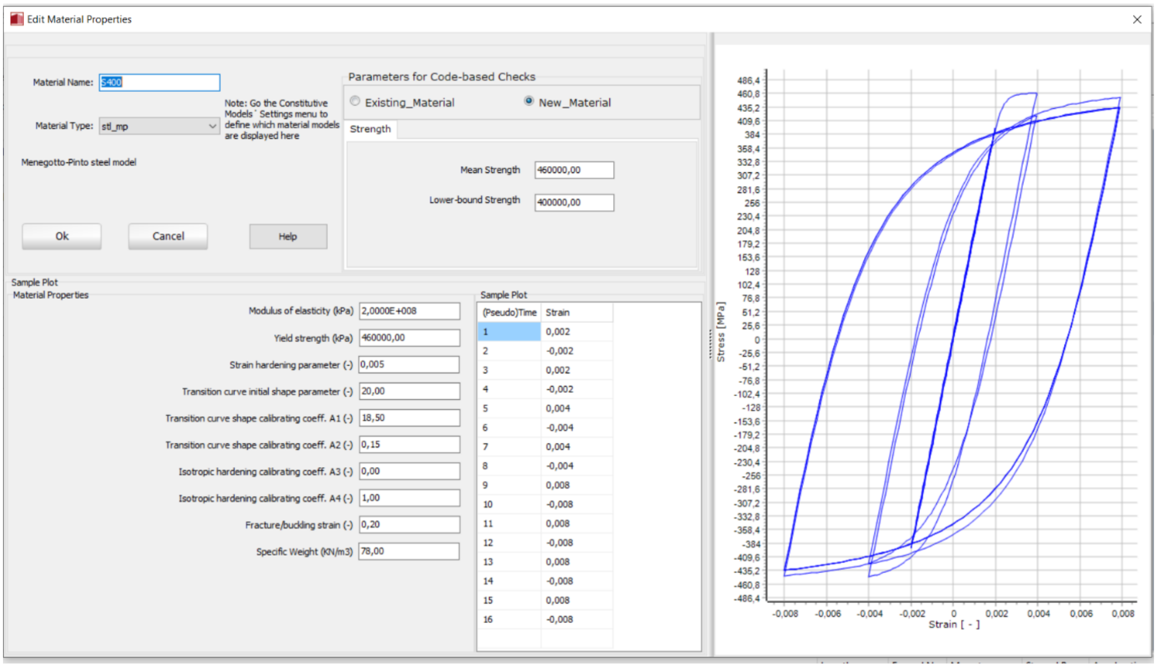


Figure V.6: steel material

- **Pre-Processor – Sections:**

Once the materials have been defined, move to the Sections module and click on the Add button in order to define the sections properties of structural elements.

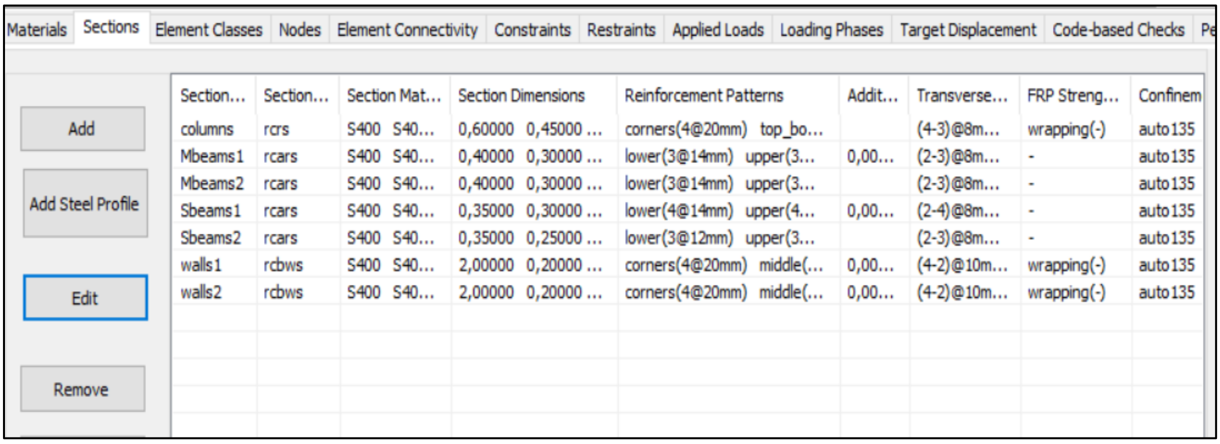


Figure V.7: Sections module

In this step we introduced different sections of beams, columns and shear walls.

<u>Columns:</u>

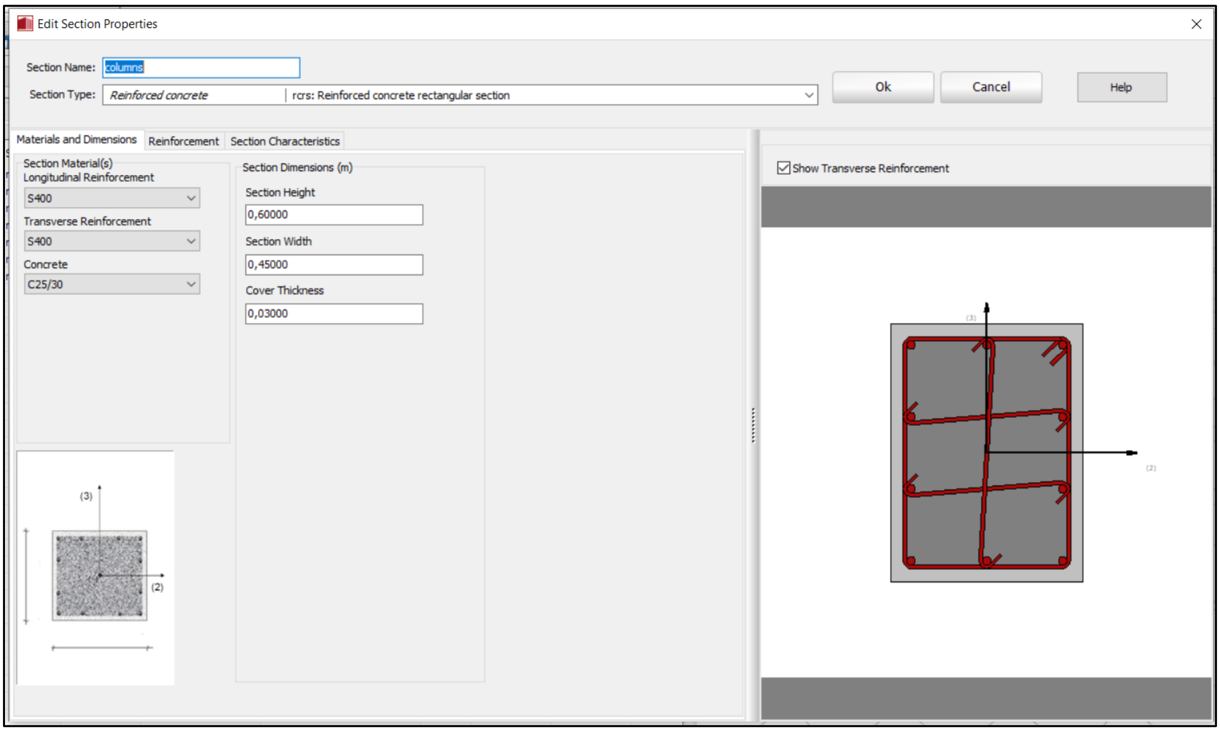


Figure V.7.1: column section

Beams:

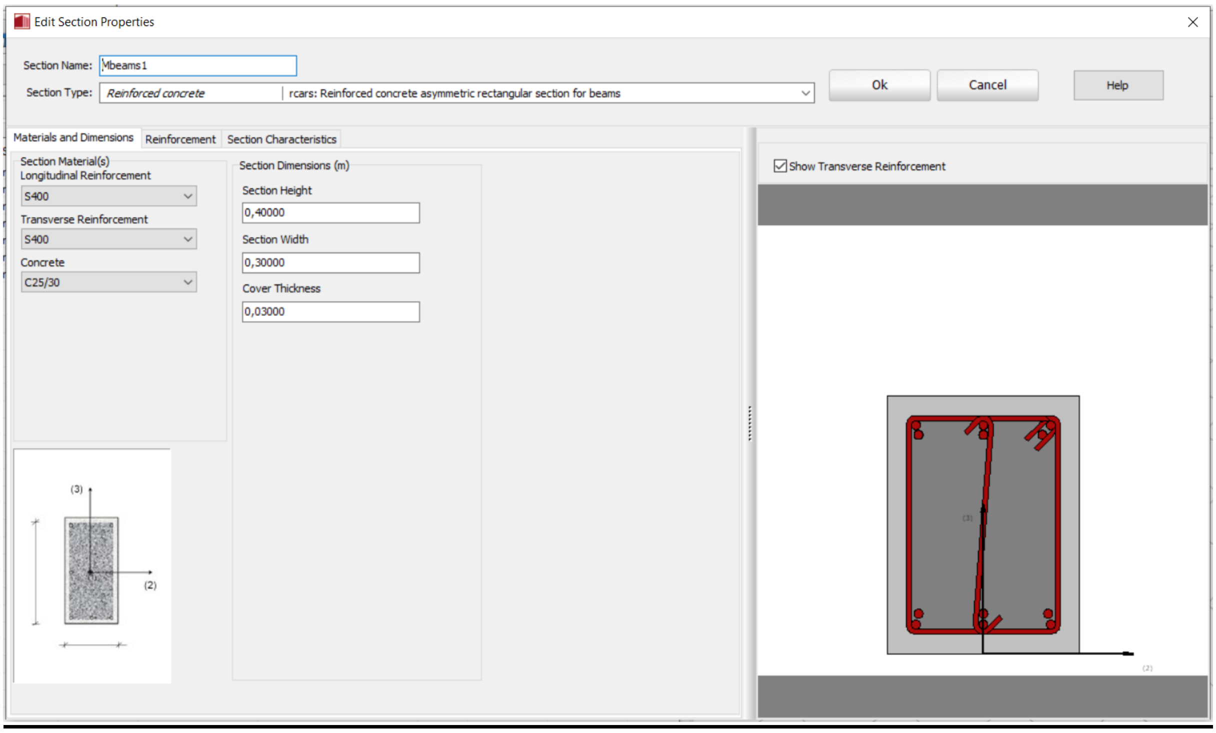


Figure V.7.2: Beam section.

Shear walls :

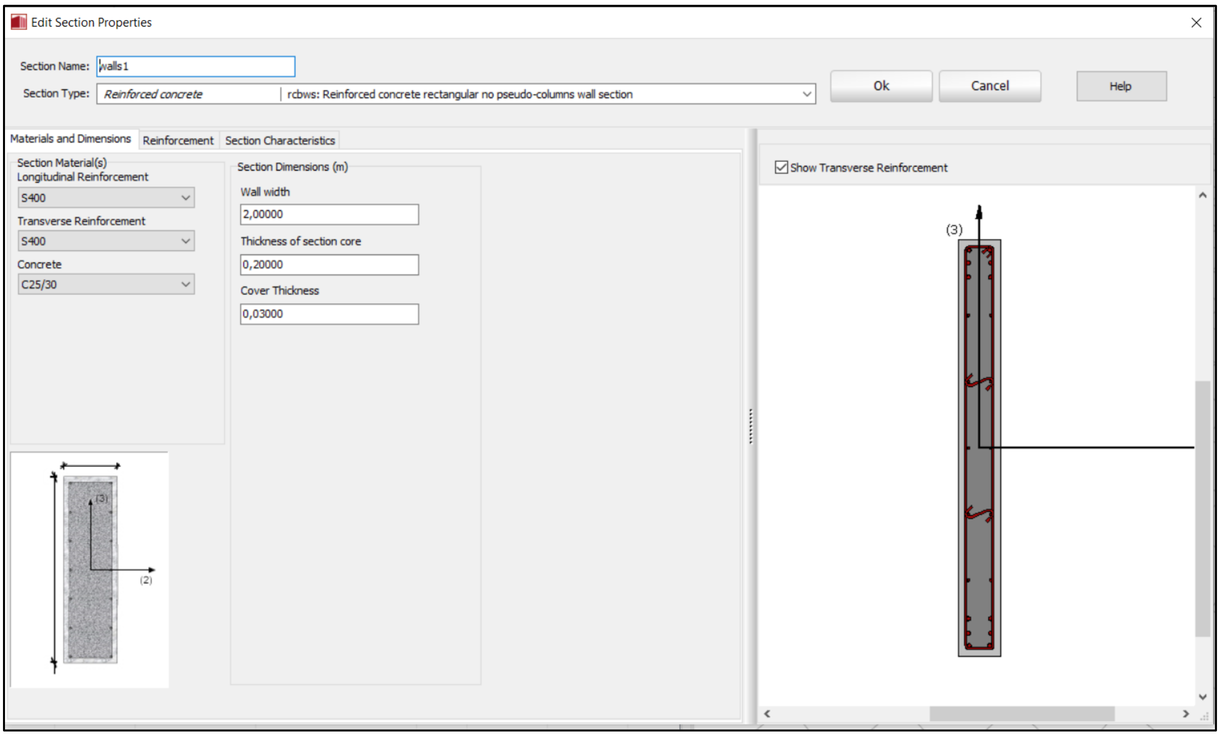


Figure V.7.3: Shear wall section.

- **Pre-processor – Element Classes**

For each section described above, you have to define an element class in the Element Classes module. Hence, click on the Add button related to the Beam-Column Element Types: a dialogue window will be opened.

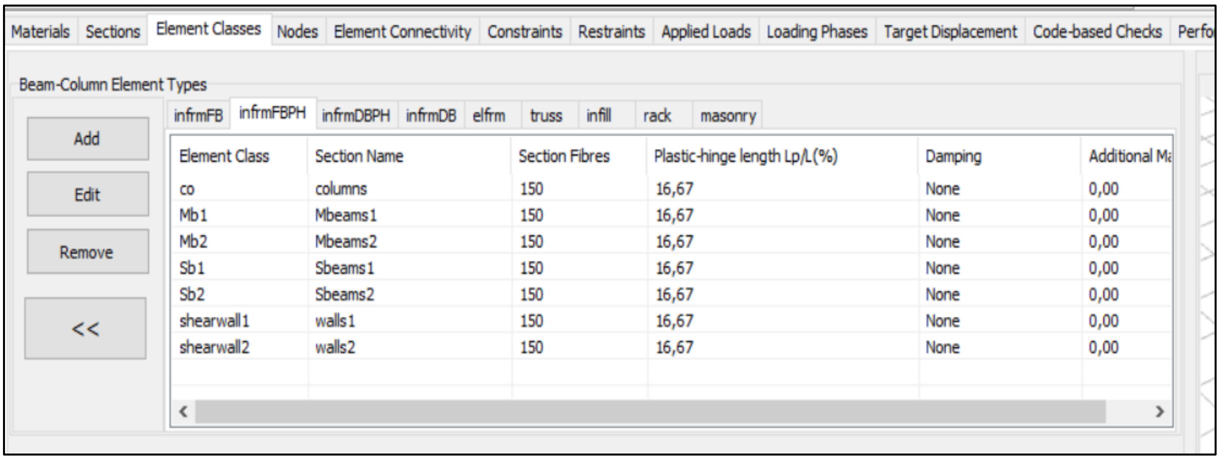


Figure V.8: Element classes module.

In our case we choose inelastic plastic-hinge force-based frame element.

- **Pre-processor – Nodes:**

At this point it is necessary to define the geometry of the structure. Hence, move to the Nodes module in order to define the nodes.

The first node you are going to define is a structural node. Click on the Add button. Then, in the new node dialogue window (i) assign the node name (N1), (ii) introduce the coordinates (x=0, y=0, z=0) and (iii) select the node type from the drop-down menu (structural node).

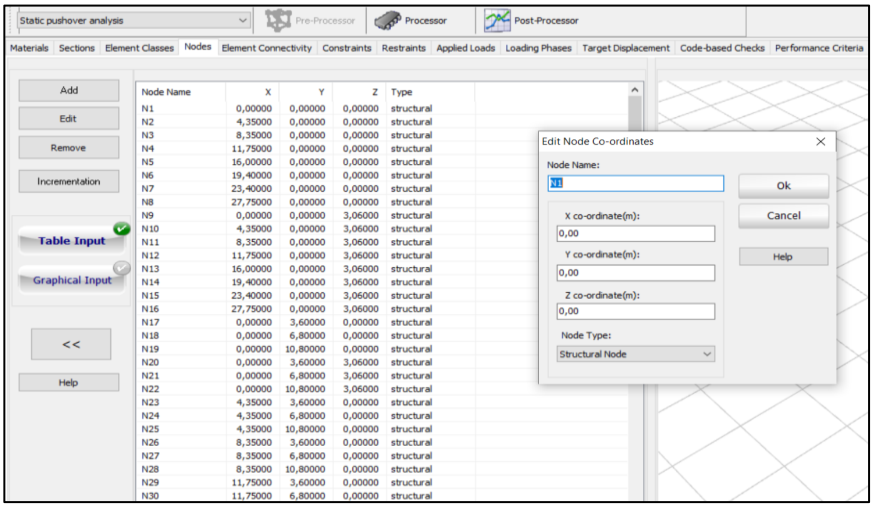


Figure V.9: Nodes module and definition of a new node.

- **Pre-processor – Element Connectivity**

Now, move to the Element Connectivity module in order to add the structural elements (i.e. columns, beams and shear walls). The first element you are going to define is a column. Hence, click on the Add button.

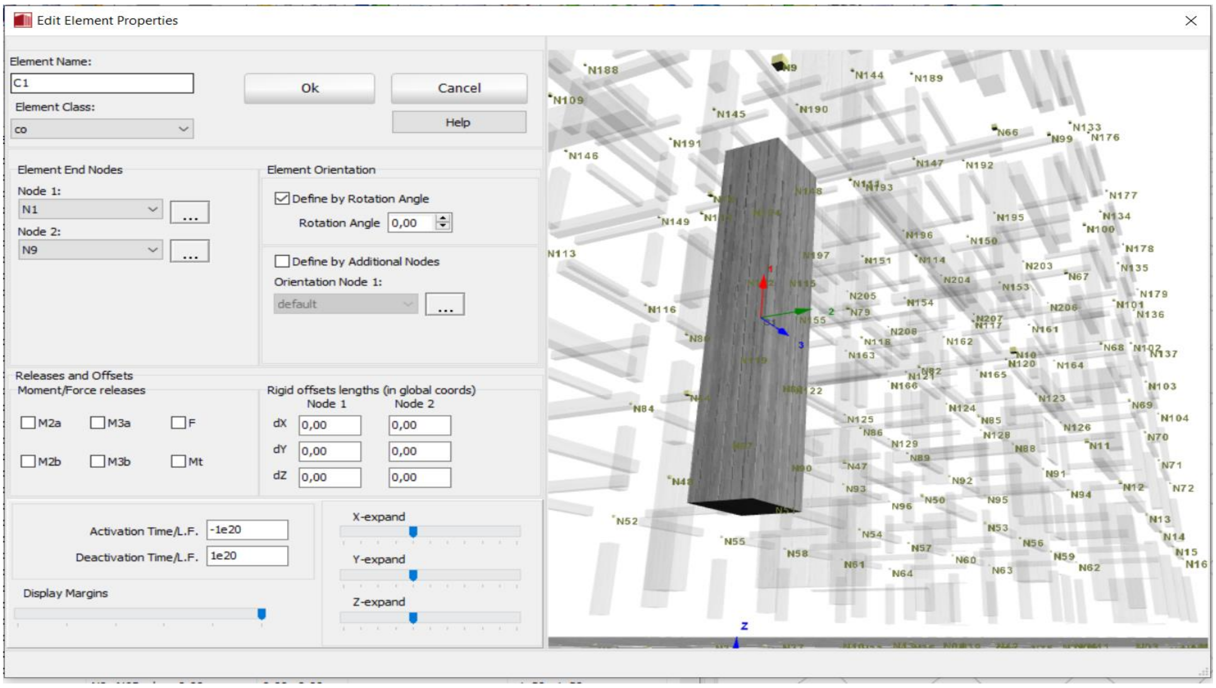


Figure V.10: Element connectivity module

- **Pre-processor – Constraints**

Now you have to define the constraining conditions of the structure. Two rigid diaphragms need to be created. Hence, go to the Constraints module and click on the Add button.

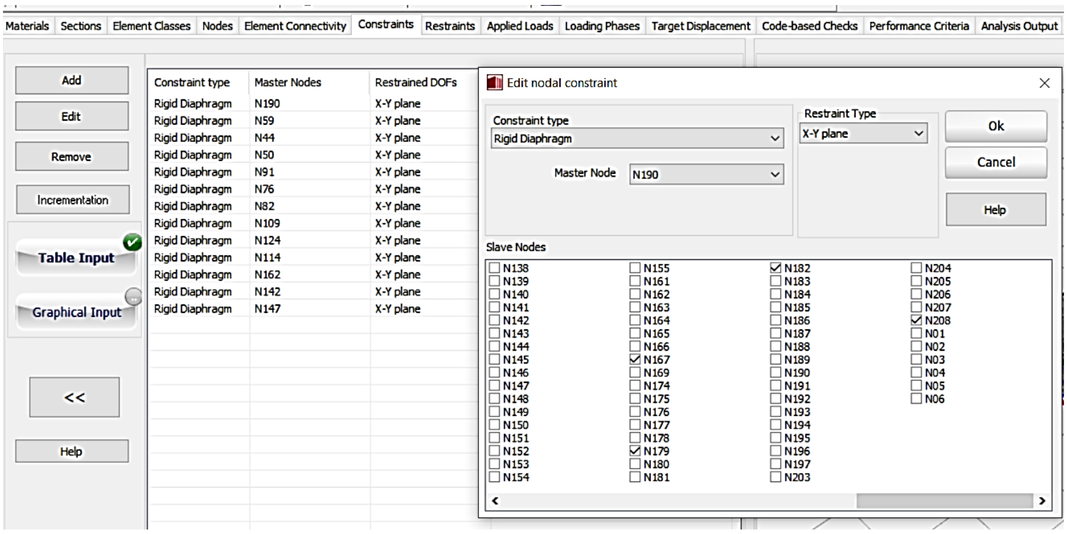


Figure V.11: Constraints module

- **Pre-processor – Restraints**

The last step related to the "structural geometry" is the definition of the restraining conditions. In this tutorial you have to fully restrain the base nodes of the structure. To do this, (i) move to the Restraints module, (ii) select the nodes you wish to restrain (-> base nodes) and (iii) click on the Edit button.

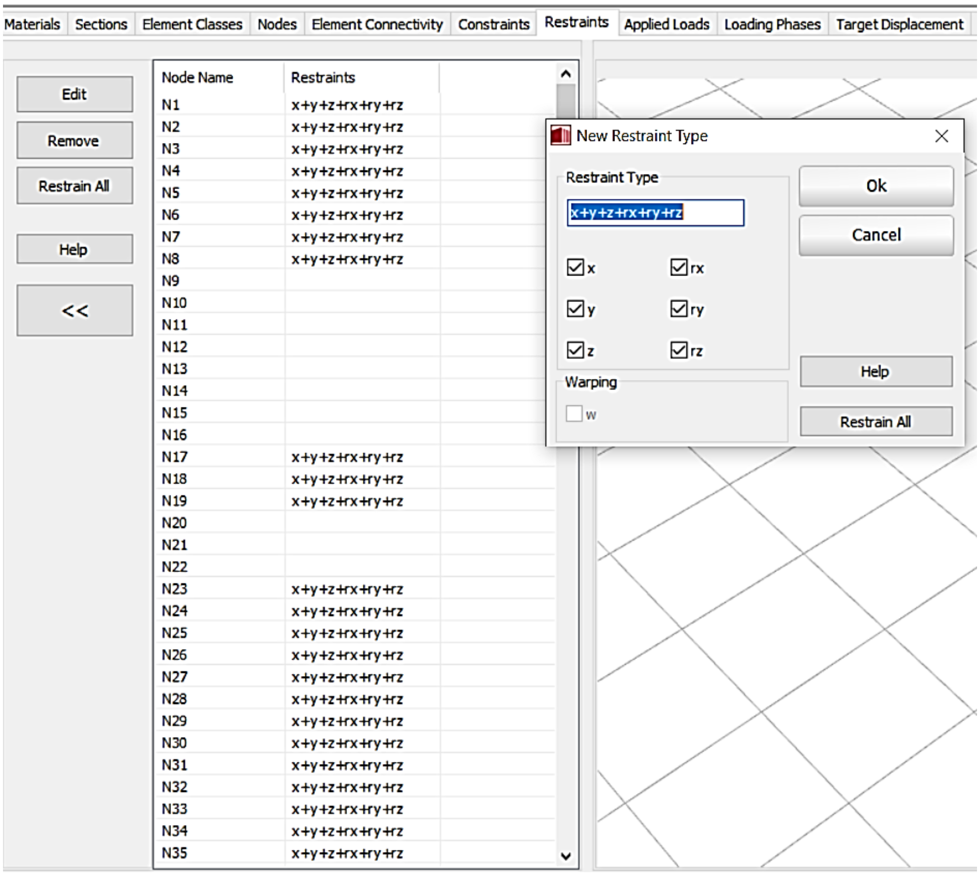


Figure V.12: Restraints module

- **Pre-processor – Applied**

Loads Since a pushover analysis needs to be carried out; you have to apply the appropriated loads (i.e. incremental loads) to the structural model. Hence, go to the Applied Loads module and click on the Add button for Load Curves.

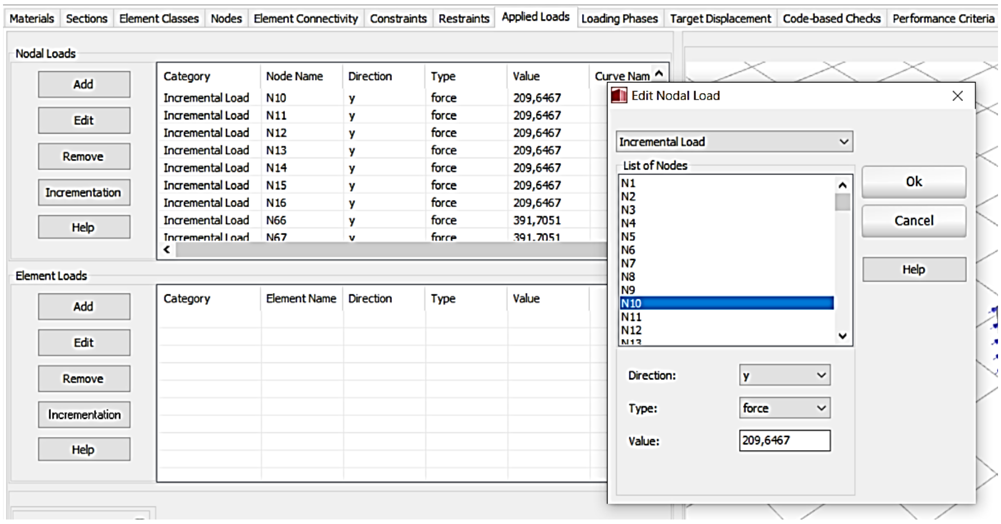


Figure V.13: Applied loads module

- **Pre-processor – Loading Phases:**

In this step we will introduce the value of the target displacement and the control node (generally the node of the roof). For this; click on the **Add button** then chose **response control**, we introduce the target displacement, control node and the direction, (target displacement = structure height /25).

- **Pre-processor –Performance criteria:**

In this step we will define the performance criteria. In this example we are interested in the criteria bellow :

– Yielding of steel
– Fracture of steel
– Spalling of cover concrete
– Crushing of core concrete

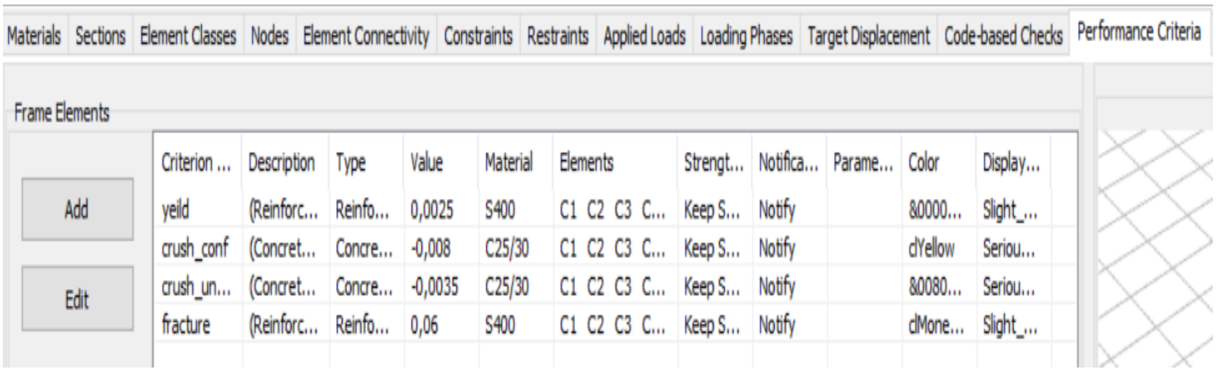
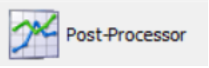


Figure V.14: performance criteria module

Processor: In the Processor area you are allowed to start the analysis. Hence, click on the **Run button** 

Post-Processor: Click on 

to see the different results of the structure.

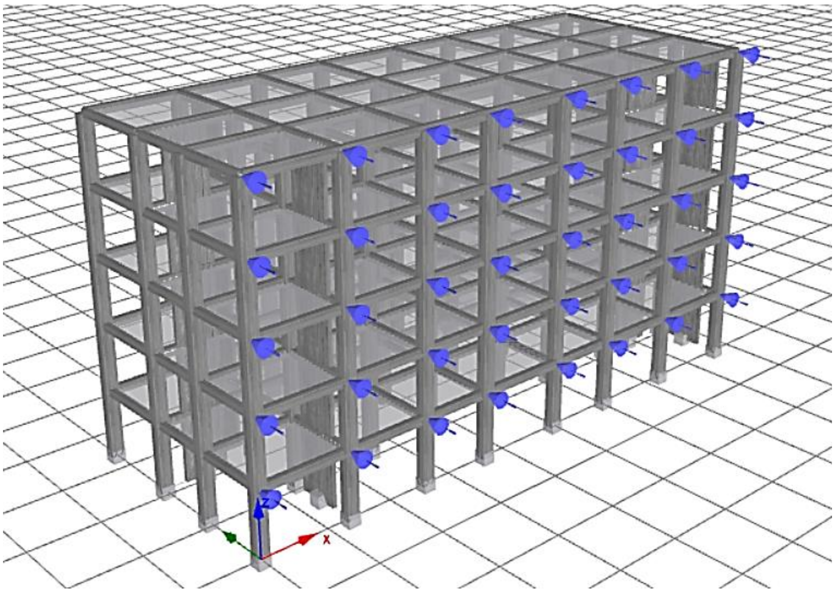


Figure V.15: Structure Shape 3D

The results of the nonlinear static analysis are:

<u>Triangular loading:</u>

✓ **Pushover Curve (Capacity Curve)**

Direction X-X:

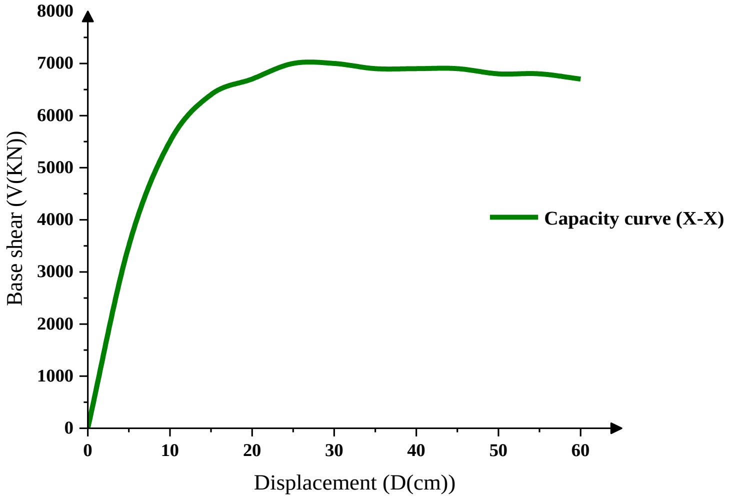


Figure V.16: Capacity Curve X-X

Direction Y-Y:

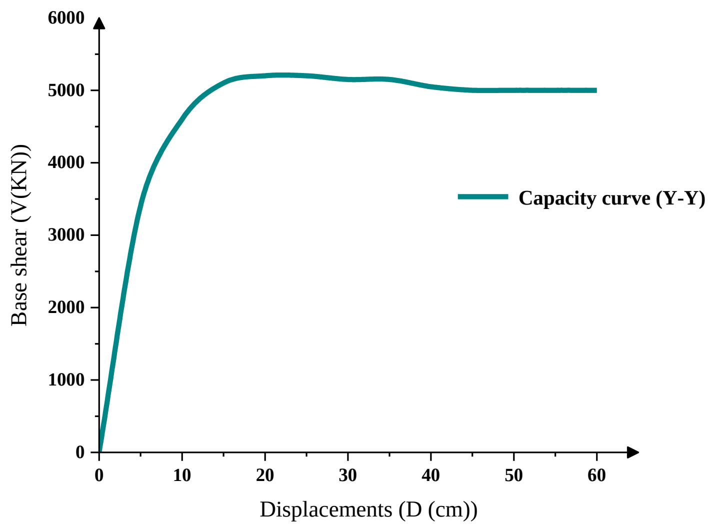


Figure V.17: Capacity Curve Y-Y

V.2.2.1.2 Transformation of push over capacity curve to an equivalent capacity curve (acceleration-displacement)

We will transform it into an equivalent capacity curve relating acceleration and displacement "Acceleration spectrum Sai – Displacement spectrum Sdi" using the following relationships:

$$Sa_i = \frac{V_i/W}{\alpha_1} \qquad Sd_i = \frac{\delta_i}{(PF_1 \times \varphi_{1,roof})}$$

With:

$$PF_{1m} = \frac{\sum_{i=1}^{n}(w_i\varphi_{i.1})/g}{\sum_{i=1}^{n}(w_i\varphi_{i,1}^2)/g} \times \varphi_{roof} \quad \ldots\ldots\ldots\ldots \text{Modal participation factor.}$$

$$\alpha_1 = \frac{[\sum_{i=1}^{n}(w_i\varphi_{i,1})/g]^2}{[\sum_{i=1}^{n} w_i/g]\,[\sum_{i=1}^{n}(w_i\varphi_{i,1}^2)/g]} \ldots\ldots\ldots\ldots \text{Modal mass coefficient.}$$

First we have to calculate the $\boldsymbol{\varphi}$ factor and to do this we need to solve this equation:

$$|[K] - w^2[M]| \times \begin{pmatrix} \varphi_{1.1} \\ \varphi_{2.1} \\ \varphi_{3.1} \\ \varphi_{4.1} \\ \varphi_{5.1} \end{pmatrix} = 0$$

With:

$[M]$: The mass matrix.

$[K]$: The stiffness matrix.

w : Pseudo period (rad/s)

According to the linear analysis, we have:

<u>Direction X-X:</u>

First Mode		
T1 = 0.424 s		
Sa =0.125g		
Floors	**Masses(t)**	**Stiffness(KN)**
Fifth floor	301,0416	474201.201
Fourth floor	580.921	591395.559
Third floor	860.800	658054.747
Second floor	1140.679	810445.64
First floor	1420.559	1384994.56
$\boldsymbol{\alpha_1}$	0.5115	
$\boldsymbol{PF_{1m}}$	1.4668	

Table V.5: 1st mode linear analysis results direction X-X

$T = \frac{2\pi}{w} \quad \Rightarrow \quad w = \frac{2\pi}{T}$

We have $T_1 = 0.424s \;\Rightarrow\; w = \frac{2\pi}{0.424} = 14.81\,\frac{rad}{s} \Rightarrow w^2 = 219.336\,(\frac{rad}{s})^2$

$$[K] = \begin{bmatrix} 2195440.2 & -810445.64 & 0 & 0 & 0 \\ -810445.64 & 1468500.39 & -658054.747 & 0 & 0 \\ 0 & -658054.747 & 1249450.31 & -591395.559 & 0 \\ 0 & 0 & -591395.559 & 1065596.76 & -474201.201 \\ 0 & 0 & 0 & -474201.201 & 474201.201 \end{bmatrix}$$

$$[M] = \begin{bmatrix} 1420.559 & 0 & 0 & 0 & 0 \\ 0 & 1140.679 & 0 & 0 & 0 \\ 0 & 0 & 860.800 & 0 & 0 \\ 0 & 0 & 0 & 580.921 & 0 \\ 0 & 0 & 0 & 0 & 301.0416 \end{bmatrix}$$

$$|[K] - w^2[M]| \times \begin{pmatrix} \varphi_{1.1} \\ \varphi_{2.1} \\ \varphi_{3.1} \\ \varphi_{4.1} \\ \varphi_{5.1} \end{pmatrix} = 0$$

$$\left[\begin{bmatrix} 2195440.2 & -810445.64 & 0 & 0 & 0 \\ -810445.64 & 1468500.39 & -658054.747 & 0 & 0 \\ 0 & -658054.747 & 1249450.31 & -591395.559 & 0 \\ 0 & 0 & -591395.559 & 1065596.76 & -474201.201 \\ 0 & 0 & 0 & -474201.201 & 474201.201 \end{bmatrix} - 219.336 \times \begin{bmatrix} 1420.559 & 0 & 0 & 0 & 0 \\ 0 & 1140.679 & 0 & 0 & 0 \\ 0 & 0 & 860.800 & 0 & 0 \\ 0 & 0 & 0 & 580.921 & 0 \\ 0 & 0 & 0 & 0 & 301.0416 \end{bmatrix}\right] \times \begin{pmatrix} \boldsymbol{\varphi_{1.1}} \\ \boldsymbol{\varphi_{2.1}} \\ \boldsymbol{\varphi_{3.1}} \\ \boldsymbol{\varphi_{4.1}} \\ \boldsymbol{\varphi_{5.1}} \end{pmatrix} = 0$$

$$\begin{bmatrix} 1883860.471 & -810445.64 & 0 & 0 & 0 \\ -810445.64 & 1218308.421 & -658054.747 & 0 & 0 \\ 0 & -658054.747 & 1060645.881 & -591395.559 & 0 \\ 0 & 0 & -591395.559 & 938179.8715 & -474201.201 \\ 0 & 0 & 0 & -474201.201 & 408171.9406 \end{bmatrix} \times \begin{pmatrix} \boldsymbol{\varphi_{1.1}} \\ \boldsymbol{\varphi_{2.1}} \\ \boldsymbol{\varphi_{3.1}} \\ \boldsymbol{\varphi_{4.1}} \\ \boldsymbol{\varphi_{5.1}} \end{pmatrix} = 0$$

$$\begin{cases} 1883860.471\varphi_{1,1} - 810445.64\varphi_{2,1} = 0 \\ -810445.64\varphi_{1,1} + 1218308.421\varphi_{2,1} - 658054.747\varphi_{3,1} = 0 \\ -658054.747\varphi_{2,1} + 1060645.881\varphi_{3,1} - 591395.559\varphi_{4,1} = 0 \\ -591395.559\varphi_{3,1} + 938179.8715\varphi_{4,1} - 474201.201\varphi_{5,1} = 0 \\ -474201.201\varphi_{4,1} + 408171.9406\varphi_{5,1} = 0 \end{cases}$$

We assume: $\varphi_{5,1} = 1$

So we will have:

$$\begin{pmatrix} \varphi_{1.1} = 0.05 \\ \varphi_{2.1} = 0.129 \\ \varphi_{3.1} = 0.56 \\ \varphi_{4.1} = 0.86 \\ \varphi_{5.1} = 1 \end{pmatrix}$$

As results we have:

$\boldsymbol{PF_{1m} = 1.4668}$ $\quad$ $\boldsymbol{,\alpha_1 = 0.5115}$

So, the capacity spectrum is represented by Figure V.18:

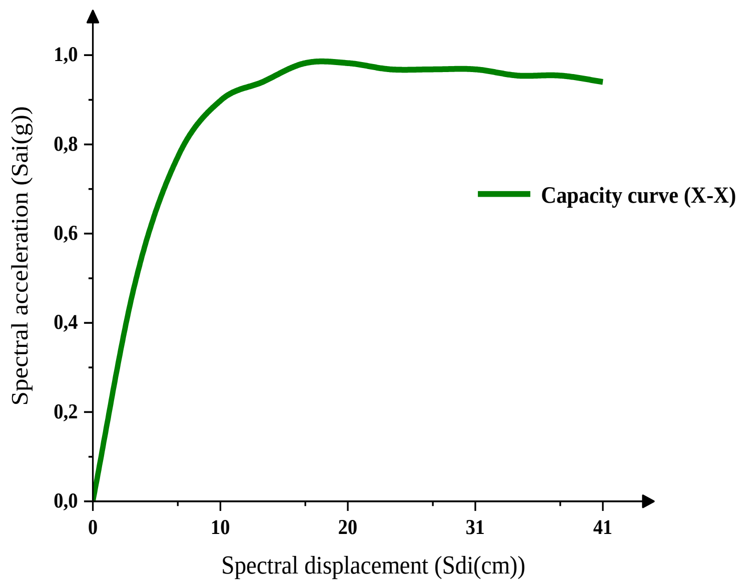


Figure V.18: Capacity Spectrum X-X

Direction Y-Y:

First Mode		
T1 = 0.468 s		
Sa =0.125g		
Floors	**Masses(t)**	**Stiffness(KN)**
Fifth floor	301,0416	264787.502
Fourth floor	580.921	373320.759
Third floor	860.800	458507.833
Second floor	1140.679	611278.104
First floor	1420.559	1301267.402
α_1	0.62	
PF_{1m}	1.51	

Table V.6: 1^{st} mode linear analysis results direction Y-Y

So, the capacity Spectrum is represented by **Figure V.19**

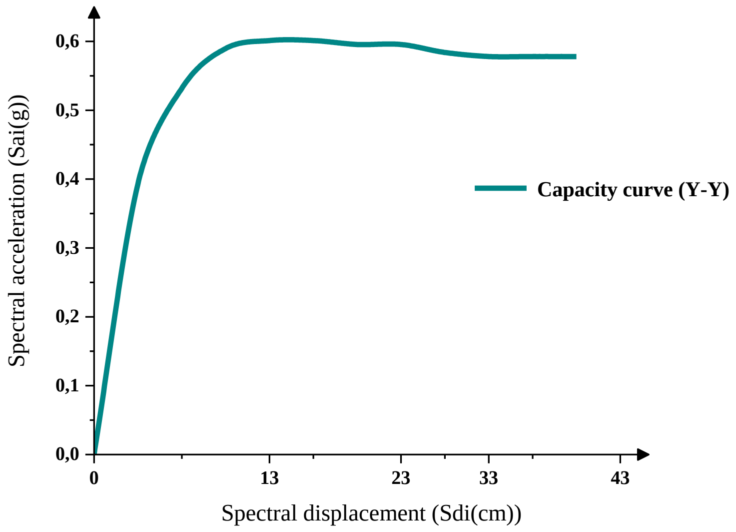


Figure V.19: Capacity Spectrum Y-Y

After determining the capacity spectrum, we move on to the determination of the elastic response spectrum.

V.2.2.1.3 Determination of performance points using a 5% damped elastic spectrum:

V.2.2.1.3.1 According to RPA:

The response spectrum defined in the Algerian code (RPA99v2003) is a curve of maximum acceleration response (Sa/g) for a single degree of freedom system subjected to a given excitation for successive values of natural periods T. According to RPA99v2003, the response spectrum is calculated using the following relationships:

$$\frac{S_a}{g} = \begin{cases} 1.25A\left(1+\frac{T}{T_1}\left(2.5\eta\frac{Q}{R}-1\right)\right) & 0 \leq \mathrm{T} \leq \mathrm{T}_1 \\ 2.5\eta(1.25A)(\frac{Q}{R}) & T_1 \leq T \leq T_2 \\ 2.5\eta(1.25A)\left(\frac{Q}{R}\right)(\frac{T_2}{T})^{\frac{2}{3}} & T_2 \leq T \leq 3s \\ 2.5\eta(1.25A)(\frac{T_2}{3})^{2/3}(\frac{3}{T})^{5/3}\left(\frac{Q}{R}\right) & T > 3s \end{cases}$$

With: $A = 0.25$, $Q = 1$, $R = 1$, $T_1 = 0.15s$, $T_2 = 0.5s$, $\varepsilon = 5\%$, $\eta = 1$, and we take T from “0 sec” to “3sec” with 0.05 sec step

Now we have to transform the elastic response spectrum (Acceleration-Time) to a response spectrum (Acceleration-Displacement) and as we know only spectrum acceleration “Sai” in function of period T, so the spectrum displacement is given by the following relation: $S_{di} = \frac{T_i^2}{4\pi^2} . S_{ai} . g$

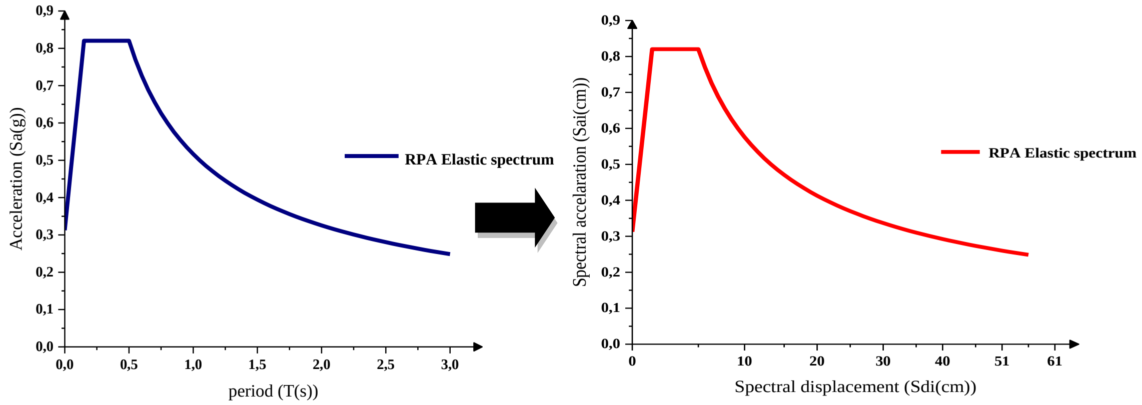


Figure V.20: transformation of the elastic response spectrum

✓ **Determination of performance point:**

According to procedure (A) of ATC40, the performance point is found after several iterations:

We take: $\begin{cases} C_a = 0.32 \\ C_v = 0.47 \end{cases}$

Direction X-X:

1st iteration:

$\begin{cases} a_{pi} = 0.977\ g \\ d_{pi} = 20.45\ cm \end{cases}$ $\begin{cases} a_y = 0.883\ g \\ d_y = 6.37\ cm \end{cases}$

PS: to obtain (d_y, a_y) we projected the points(d_{pi}, a_{pi}) and after an approximated reading from the graph we found the following points $\begin{matrix} a_y = 0.883\ g \\ d_y = 6.37\ cm \end{matrix}$

/	Direction X-X
$\beta_0 = \frac{63.7(a_y d_{pi} - d_y a_y)}{a_{pi} d_{pi}}$	37.73%
$\beta_{eff} = \lambda\beta_0 + 5$	26.88%
$SR_a = \frac{3.21 - 0.681 ln\beta_{eff}}{2.12}$	0.46 g
$SR_v = \frac{2.31 - 0.41 ln\beta_{eff}}{1.65}$	0.58 g
$T_s = {SR_v.C_V}/{2.5.SR_a.C_A}$	0.74 s
$S_A = 2.5.SR_a.C_A$	0.37g
$S_d(T_s) = S_A.g.100(\frac{T_s}{2\pi})^2$	5.034 cm

Table V.7: Results of first iteration (direction X-X)

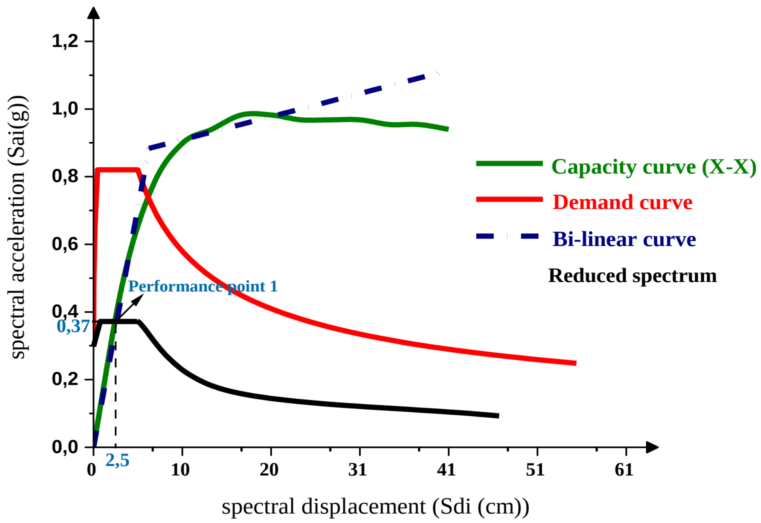


Figure V.21: Performance point of the 1st iteration (direction X-X)

Performance point 1: $\begin{cases} a_{p1} = 0.37g \\ d_{p1} = 2.5\ cm \end{cases}$

Error: $100\left(\frac{20.45-2.5}{20.45}\right) = 87.77\% > 5\%$ $\Rightarrow$ a second iteration is required.

<u>2nd iteration:</u>

$$\begin{cases} \mathrm{a_{pi}} = \frac{0.37+0.977}{2} = 0.67 \\ \mathrm{d_{pi}} = \frac{20.45+2.5}{2} = 11.47 \end{cases} \Rightarrow \begin{cases} \mathrm{a_{py}} = 0.4\ \mathrm{g} \\ \mathrm{d_{py}} = 4.7\ \mathrm{cm} \end{cases}$$

/	Direction X-X
$\beta_0 = \dfrac{63.7(a_y d_{pi} - d_y a_y)}{a_{pi} d_{pi}}$	11.92%
$\beta_{eff} = \lambda\beta_0 + 5$	12.98%
$SR_a = \dfrac{3.21 - 0.681 ln\beta_{eff}}{2.12}$	0.69 g
$SR_v = \dfrac{2.31 - 0.41 ln\beta_{eff}}{1.65}$	0.76 g
$T_s = {SR_v.C_V}/{2.5.SR_a.C_A}$	0.64 s
$S_A = 2.5.SR_a.C_A$	0.55g
$S_d(T_s) = S_A.g.100(\frac{T_s}{2\pi})^2$	5.6 cm

Table V.8: Results of the second iteration (direction X-X)

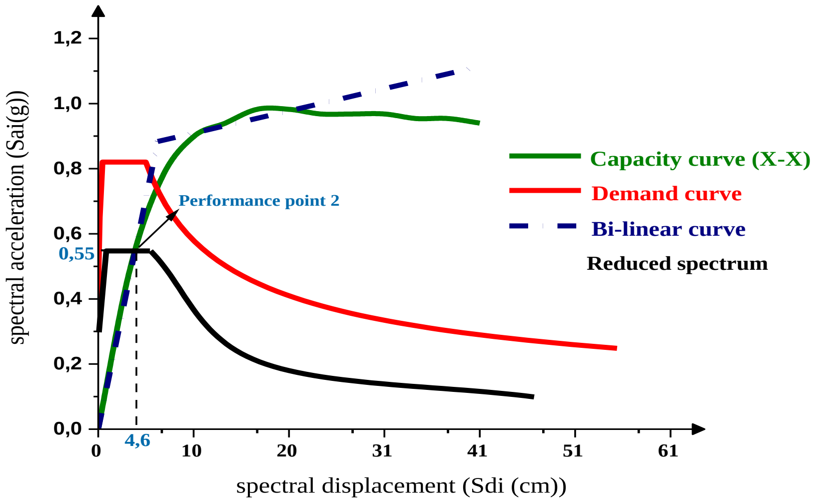


Figure V.22: Performance point of the 2nd iteration (direction X-X)

Performance point 2: $\begin{cases} a_{p2} = 0.55g \\ d_{p2} = 4.6cm \end{cases}$

Error: $100\left(\frac{11.47-4.6}{11.47}\right) = 59.89\% > 5\%$ $\Rightarrow$ a third iteration is required.

3rd iteration:

$\mathrm{a_{pi}} = \frac{0.55+0.67}{2} = 0.61$
$\mathrm{d_{pi}} = \frac{4.6+11.47}{2} = 8.035$ $\Rightarrow$ $\begin{cases} \mathrm{a_{py}} = 0.3 \text{ g} \\ \mathrm{d_{py}} = 3.5\text{cm} \end{cases}$

/	Direction X-X
$\beta_0 = \dfrac{63.7(a_y d_{pi} - d_y a_y)}{a_{pi} d_{pi}}$	3.58%
$\beta_{eff} = \lambda\beta_0 + 5$	7.4%
$SR_a = \dfrac{3.21 - 0.681 ln\beta_{eff}}{2.12}$	0.87 g
$SR_v = \dfrac{2.31 - 0.41 ln\beta_{eff}}{1.65}$	0.90 g
$T_s = {SR_v.C_V}/{2.5.SR_a.C_A}$	0.60 s
$S_A = 2.5.SR_a.C_A$	0.696g
$S_d(T_s) = S_A.g.100(\frac{T_s}{2\pi})^2$	6.226 cm

Table V.9: Results of the third iteration (direction X-X)

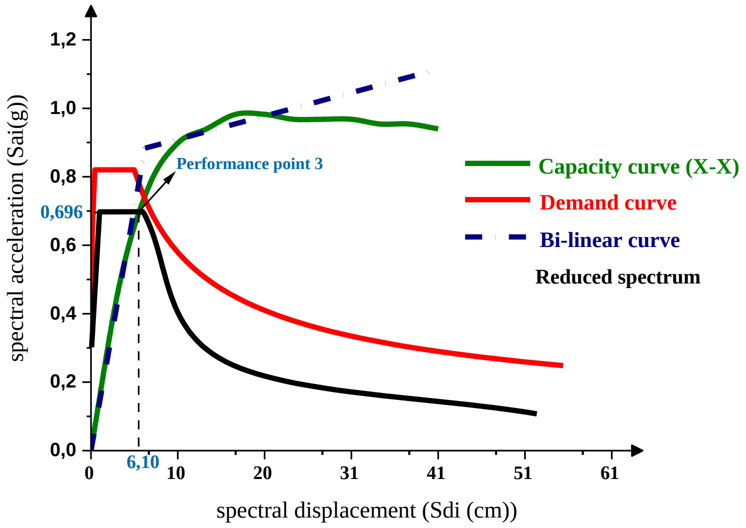


Figure V.23: Performance point of the 3rd iteration (direction X-X)

Performance point 3: $\begin{cases} a_{p3} = 0.696g \\ d_{p3} = 6.10cm \end{cases}$

Error: $100\left(\frac{8.035-6.1}{8.035}\right) = 24.08\% > 5\%$ $\Rightarrow$ a fourth iteration is required

4th iteration:

$$\mathrm{a_{pi}} = \frac{0.696+0.61}{2} = 0.65 \qquad \mathrm{d_{pi}} = \frac{6.10+8.035}{2} = 7.07 \quad \Longrightarrow \quad \begin{cases} \mathrm{a_{py}} = 0.65\ \mathrm{g} \\ \mathrm{d_{py}} = 7.07\mathrm{cm} \end{cases}$$

/	Direction X-X
$\beta_0 = \dfrac{63.7(a_y d_{pi} - d_y a_y)}{a_{pi} d_{pi}}$	0%
$\beta_{eff} = \lambda\beta_0 + 5$	5%
$SR_a = \dfrac{3.21 - 0.681 ln\beta_{eff}}{2.12}$	0.99 g
$SR_v = \dfrac{2.31 - 0.41 ln\beta_{eff}}{1.65}$	1 g
$T_s = {SR_v.C_V}/{2.5.SR_a.C_A}$	0.59 s
$S_A = 2.5.SR_a.C_A$	0.79g
$S_d(T_s) = S_A.g.100(\frac{T_s}{2\pi})^2$	6.83 cm

Table V.10: Results of the fourth iteration (direction X-X)

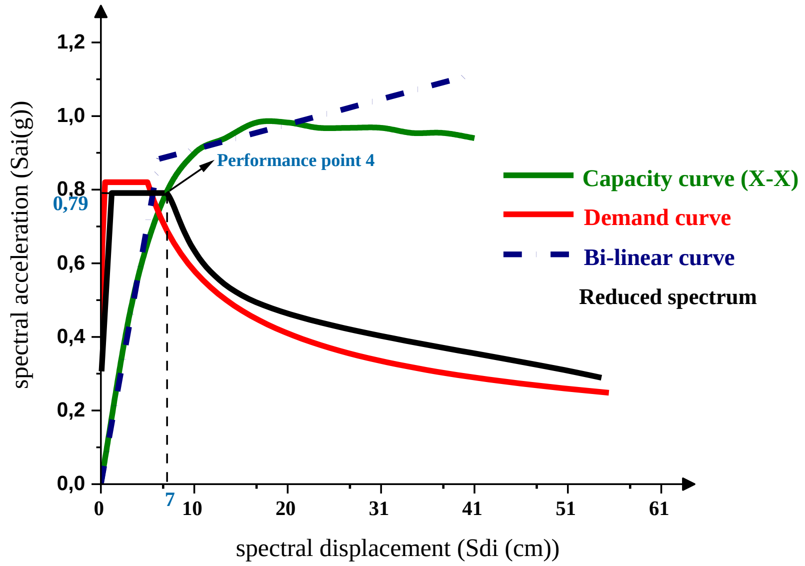


Figure V.24: Performance point of the 4th iteration (direction X-X)

Performance point 4: $\begin{cases} a_{p4} = 0.78\ g \\ d_{p4} = 7\ cm \end{cases}$

Error: $100\left(\frac{7.07-7}{7.07}\right) = 1\% < 5\%$ $\Rightarrow$ verified condition.

So the performance point is defined by a displacement: $\boldsymbol{d = 7\ cm}$

<u>Direction Y-Y:</u>

<u>1st iteration:</u>

$$\begin{cases} a_{pi} = 0.6g \\ d_{pi} = 15.42cm \end{cases} \Rightarrow \begin{cases} a_{py} = 0.54\ g \\ d_{py} = 4.05cm \end{cases}$$

/	Direction Y-Y
$\beta_0 = \frac{63.7(a_y d_{pi} - d_y a_y)}{a_{pi} d_{pi}}$	40.599%
$\beta_{eff} = \lambda\beta_0 + 5$	27.765%
$SR_a = \frac{3.21 - 0.681 ln\beta_{eff}}{2.12}$	0.446 g
$SR_v = \frac{2.31 - 0.41 ln\beta_{eff}}{1.65}$	0.574 g
$T_s = {SR_v.C_V}/{2.5.SR_a.C_A}$	0.756 s
$S_A = 2.5.SR_a.C_A$	0.3568g
$S_d(T_s) = S_A.g.100(\frac{T_s}{2\pi})^2$	5.074 cm

Table V.11: Results of the first iteration direction (Y-Y)

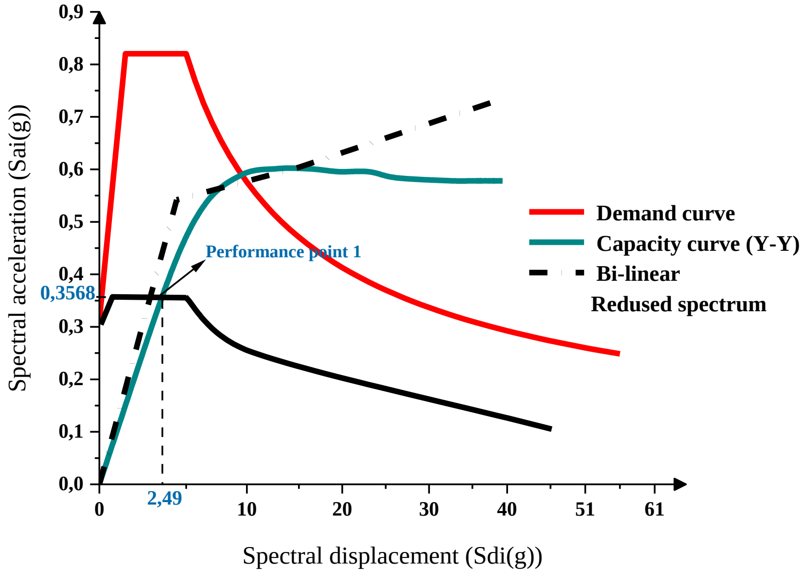


Figure V.25: performance point of the 1st iteration direction (Y-Y)

Performance point 1: $\begin{cases} a_1 = 0.3568\ g \\ d_1 = 2.49\ cm \end{cases}$

Error: $100\left(\frac{15.42-2.49}{15.42}\right) = 83.85\% > 5\%$ $\Rightarrow$ a second iteration is required.

<u>2nd iteration:</u>

$$\begin{cases} \mathrm{a_{pi}} = \frac{0.3568+0.6}{2} = 0.4784\mathrm{g} \\ \mathrm{d_{pi}} = \frac{15.42+2.49}{2} = 8.955\mathrm{cm} \end{cases} \Rightarrow \begin{cases} \mathrm{a_{py}} = 0.45\ \mathrm{g} \\ \mathrm{d_{py}} = 5.6\mathrm{cm} \end{cases}$$

/	Direction Y-Y
$\beta_0 = \dfrac{63.7(a_y d_{pi} - d_y a_y)}{a_{pi} d_{pi}}$	20%
$\beta_{eff} = \lambda\beta_0 + 5$	18.4%
$SR_a = \dfrac{3.21 - 0.681 ln\beta_{eff}}{2.12}$	0.57 g
$SR_v = \dfrac{2.31 - 0.41 ln\beta_{eff}}{1.65}$	0.68 g
$T_s = {SR_v.C_V}/{2.5.SR_a.C_A}$	0.7 s
$S_A = 2.5.SR_a.C_A$	0.46g
$S_d(T_s) = S_A.g.100(\frac{T_s}{2\pi})^2$	5.6 cm

Table V.12: Results of the second iteration direction (Y-Y)

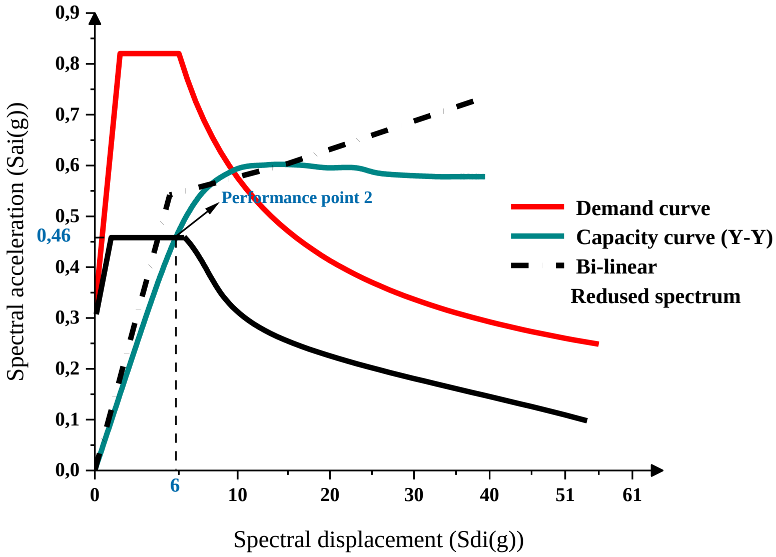


Figure V.26: Performance point 2nd iteration direction (Y-Y)

Performance point 2: $\begin{cases} a_2 = 0.46\ g \\ d_2 = 6\ cm \end{cases}$

Error: $100\left(\frac{8.955-6}{8.955}\right) = 32.99\% > 5\%$ $\Rightarrow$ a third iteration is required.

<u>3rd iteration:</u>

$$\begin{cases} \mathrm{a_{pi}} = \frac{0.46+0.4784}{2} = 0.469\mathrm{g} \\ \mathrm{d_{pi}} = \frac{8.955+6}{2} = 7.47\mathrm{cm} \end{cases} \Longrightarrow \begin{cases} \mathrm{a_{py}} = 0.4\ \mathrm{g} \\ \mathrm{d_{py}} = 4.5\mathrm{cm} \end{cases}$$

/	Direction Y-Y
$\beta_0 = \frac{63.7(a_y d_{pi} - d_y a_y)}{a_{pi} d_{pi}}$	15.95%
$\beta_{eff} = \lambda\beta_0 + 5$	15.68%
$SR_a = \frac{3.21 - 0.681 ln\beta_{eff}}{2.12}$	0.63 g
$SR_v = \frac{2.31 - 0.41 ln\beta_{eff}}{1.65}$	0.73 g
$T_s = {SR_v.C_V}/{2.5.SR_a.C_A}$	0.68 s
$S_A = 2.5.SR_a.C_A$	0.50g
$S_d(T_s) = S_A.g.100(\frac{T_s}{2\pi})^2$	5.74 cm

Table V.13: Results of the third iteration direction (Y-Y)

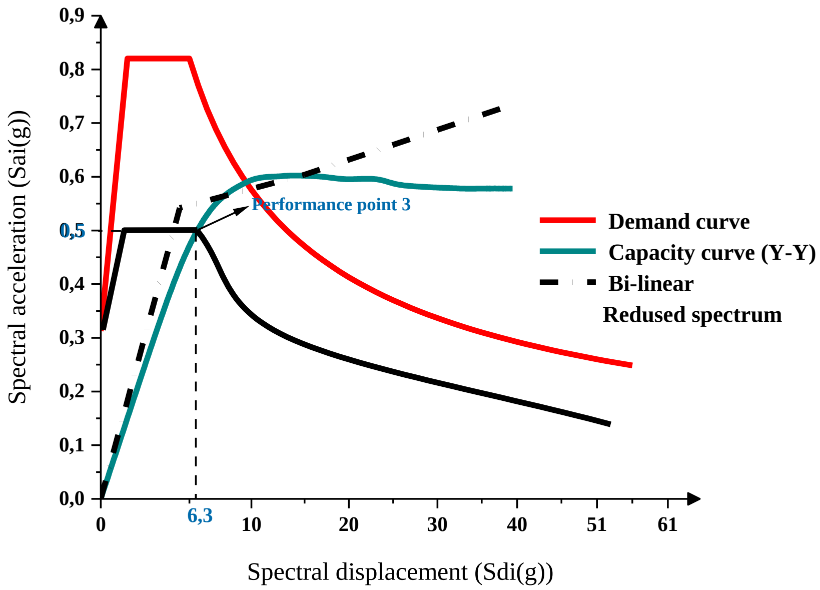


Figure V.27: performance point of the 3rd iteration direction (Y-Y)

Performance point 3: $\begin{cases} a_3 = 0.5\ g \\ d_3 = 6.3\ cm \end{cases}$

Error: $100\left(\frac{7.47-6.3}{7.47}\right) = 15.66\% > 5\%$ $\Rightarrow$ a fourth iteration is required.

4th iteration:

$$\begin{cases} \mathrm{a_{pi}} = \frac{0.469+0.5}{2} = 0.484\mathrm{g} \\ \mathrm{d_{pi}} = \frac{7.227+6}{2} = 6.89\mathrm{cm} \end{cases} \quad \Longrightarrow \quad \begin{cases} \mathrm{a_{py}} = 0.35\ \mathrm{g} \\ \mathrm{d_{py}} = 3.8\mathrm{cm} \end{cases}$$

/	Direction Y-Y
$\beta_0 = \dfrac{63.7(a_y d_{pi} - d_y a_y)}{a_{pi} d_{pi}}$	10.93%
$\beta_{eff} = \lambda\beta_0 + 5$	12.32%
$SR_a = \dfrac{3.21 - 0.681 ln\beta_{eff}}{2.12}$	0.72 g
$SR_v = \dfrac{2.31 - 0.41 ln\beta_{eff}}{1.65}$	0.8 g
$T_s = {SR_v.C_V}/{2.5.SR_a.C_A}$	0.64 s
$S_A = 2.5.SR_a.C_A$	0.576g
$S_d(T_s) = S_A.g.100(\frac{T_s}{2\pi})^2$	5.86 cm

Table V.14: Results of the fourth iteration direction (Y-Y)

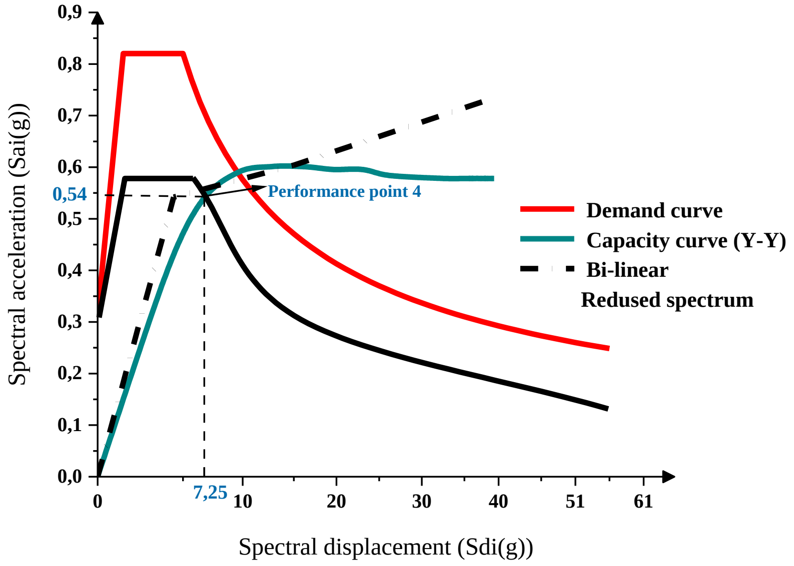


Figure V.28: performance point of the 4th iteration direction (Y-Y)

Performance point 4: $\begin{cases} a_3 = 0.54\ g \\ d_3 = 7.25\ cm \end{cases}$

Error: $100\left(\frac{7.25-6.89}{7.25}\right) = 4.96\% > 5\% \quad \Rightarrow$ verified condition.

So, the performance point is defined by a displacement: $\boldsymbol{d = 7.25\ cm}$

V.2.2.1.3.2 According to ASCE 41:

Transforming the elastic spectrum (Acceleration-Period) into elastic spectrum (Acceleration-Displacement) using the relation bellow: $S_{di} = \frac{T_i^2}{4\pi^2} . S_{ai} . g$

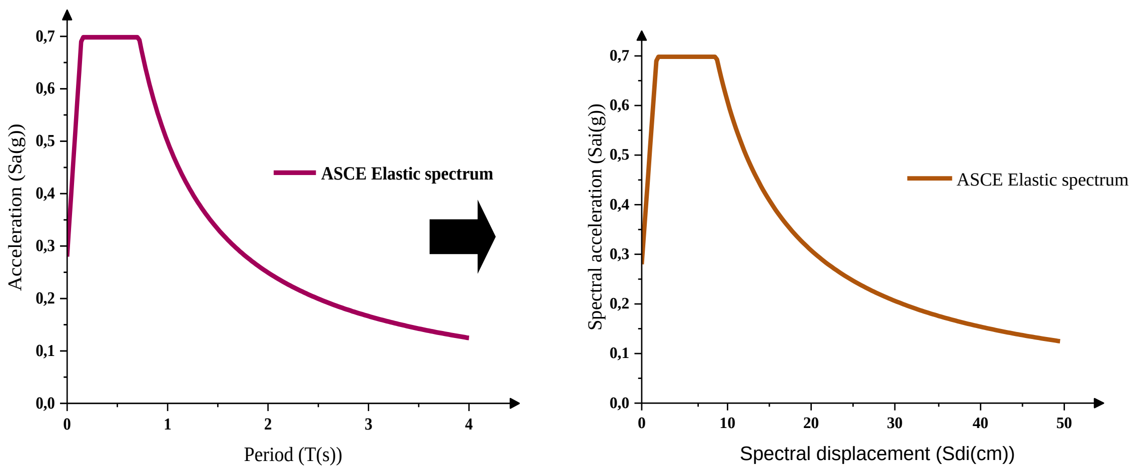


Figure V.29: Transformation of the elastic spectrum

✓ **<u>Determination of performance point:</u>**

According to procedure (A) of ATC40, the performance point is found after several iterations:

We take: $\begin{cases} C_a = 0.32 \\ C_v = 0.47 \end{cases}$

Direction X-X:

1st iteration:

$$\begin{cases} \mathrm{a_{pi}} = 0.977\mathrm{g} \\ \mathrm{d_{pi}} = 20.45\mathrm{cm} \end{cases} \Rightarrow \begin{cases} \mathrm{a_{py}} = 0.883\ \mathrm{g} \\ \mathrm{d_{py}} = 6.37\mathrm{cm} \end{cases}$$

/	Direction X-X
$\beta_0 = \dfrac{63.7(a_y d_{pi} - d_y a_y)}{a_{pi} d_{pi}}$	37.729%
$\beta_{eff} = \lambda\beta_0 + 5$	26.88%
$SR_a = \dfrac{3.21 - 0.681 ln\beta_{eff}}{2.12}$	0.45 g
$SR_v = \dfrac{2.31 - 0.41 ln\beta_{eff}}{1.65}$	0.58 g
$T_s = {SR_v . C_V}/{2.5 . SR_a . C_A}$	0.757 s
$S_A = 2.5 . SR_a . C_A$	0.36g
$S_d(T_s) = S_A . g . 100(\frac{T_s}{2\pi})^2$	5.13 cm

Table V.15: Results of the first iteration

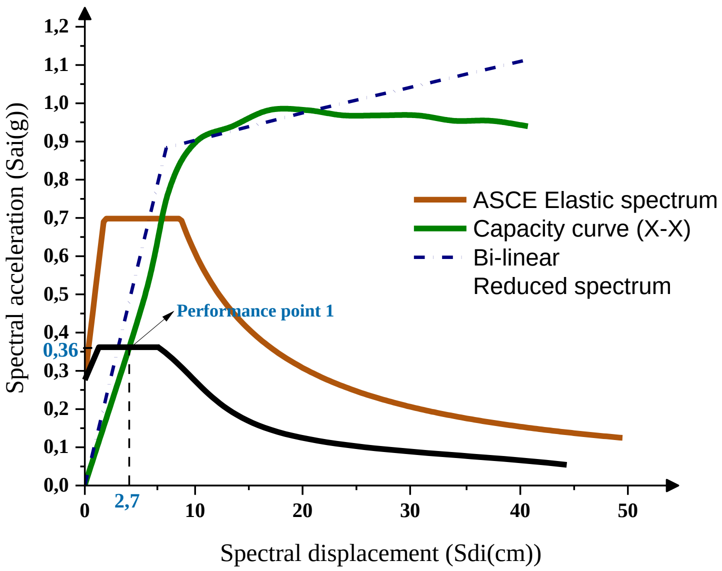


Figure V.30: performance point of the 1st iteration direction(X-X)

Performance point 1: $\begin{cases} a_1 = 0.36\ g \\ d_1 = 2.7\ cm \end{cases}$

Error: $100\left(\frac{20.45-2.7}{20.45}\right) = 86.79\% > 5\%$ $\Rightarrow$ so a second iteration is required.

2nd iteration:

$$\begin{cases} \mathrm{a_{pi}} = \frac{0.36+0.977}{2} = 0.668\mathrm{g} \\ \mathrm{d_{pi}} = \frac{20.45+2.7}{2} = 11.575\mathrm{cm} \end{cases} \Longrightarrow \begin{cases} \mathrm{a_{py}} = 0.6\ \mathrm{g} \\ \mathrm{d_{py}} = 7.6\mathrm{cm} \end{cases}$$

/	Direction X-X
$\beta_0 = \frac{63.7(a_y d_{pi} - d_y a_y)}{a_{pi} d_{pi}}$	14.29%
$\beta_{eff} = \lambda\beta_0 + 5$	14.57%
$SR_a = \frac{3.21 - 0.681 ln\beta_{eff}}{2.12}$	0.65 g
$SR_v = \frac{2.31 - 0.41 ln\beta_{eff}}{1.65}$	0.73 g
$T_s = {SR_v.C_V}/{2.5.SR_a.C_A}$	0.659 s
$S_A = 2.5.SR_a.C_A$	0.52g
$S_d(T_s) = S_A.g.100(\frac{T_s}{2\pi})^2$	5.61 cm

Table V.16: Results of the second iteration direction(X-X)

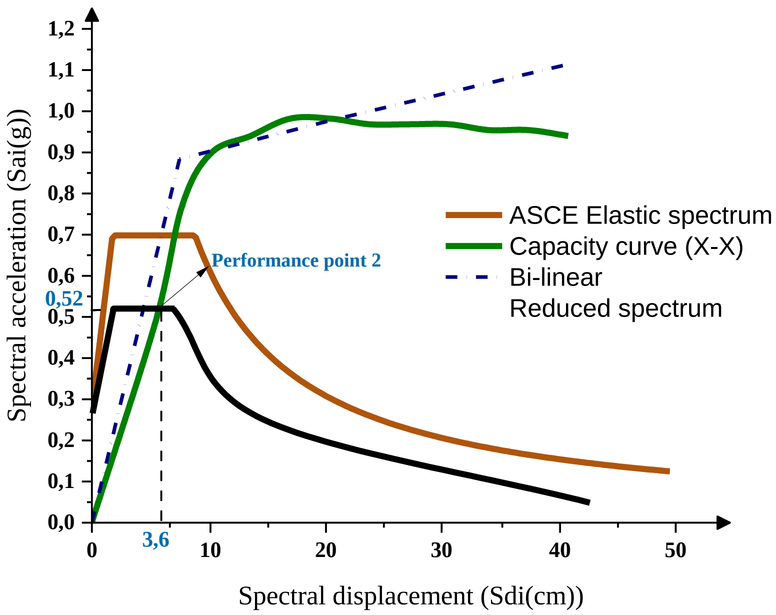


Figure V.31: Performance point of the 2nd iteration direction(X-X)

Performance point 2: $\begin{cases} a_2 = 0.52g \\ d_2 = 3.6\ cm \end{cases}$

Error: $100\left(\frac{11.575-3.6}{3.6}\right) = 68.89\% > 5\%$ $\Rightarrow$ So third iteration is required.

3rd iteration:

$$\begin{cases} a_{pi} = \frac{0.52+0.668}{2} = 0.594g \\ d_{pi} = \frac{11.575+3.6}{2} = 7.587cm \end{cases} \Rightarrow \begin{cases} a_{py} = 0.5\ g \\ d_{py} = 5\ cm \end{cases}$$

/	Direction X-X
$\beta_0 = \frac{63.7(a_y d_{pi} - d_y a_y)}{a_{pi} d_{pi}}$	11.639%
$\beta_{eff} = \lambda\beta_0 + 5$	12.798%
$SR_a = \frac{3.21 - 0.681 ln\beta_{eff}}{2.12}$	0.695 g
$SR_v = \frac{2.31 - 0.41 ln\beta_{eff}}{1.65}$	0.766 g
$T_s = {SR_v.C_V}/{2.5.SR_a.C_A}$	0.647 s
$S_A = 2.5.SR_a.C_A$	0.556g
$S_d(T_s) = S_A.g.100(\frac{T_s}{2\pi})^2$	5.789 cm

Table V.17: Results of the third iteration direction(X-X)

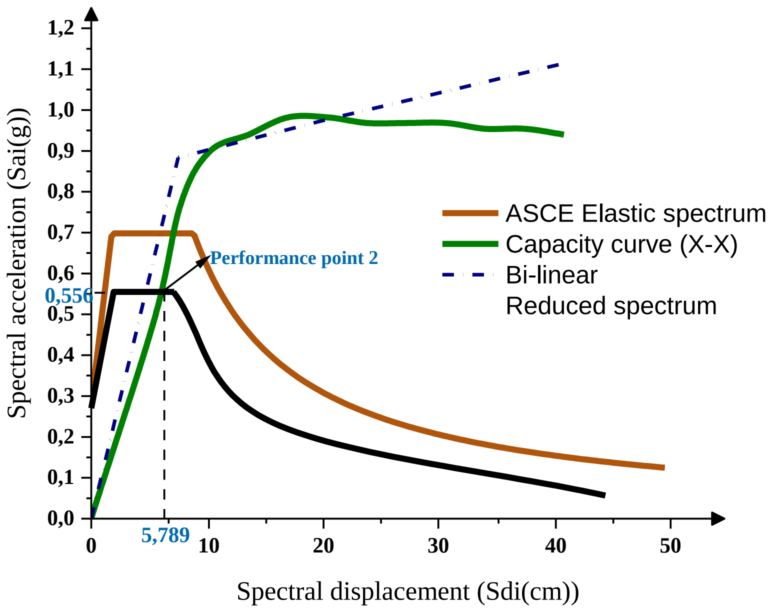


Figure V.32: Performance point of the 3rd iteration direction(X-X)

Performance point 3: $\begin{cases} a_3 = 0.556g \\ d_3 = 5.789cm \end{cases}$

Error: $100\left(\frac{7.587-5.789}{7.587}\right) = 44.64\% > 5\%$ $\Rightarrow$ So fourth iteration is required.

<u>4th iteration:</u>

$$\begin{cases} \mathrm{a_{pi}} = \frac{0.556+0.5949}{2} = 0.575\mathrm{g} \\ \mathrm{d_{pi}} = \frac{5.789+7.587}{2} = 5.89\mathrm{cm} \end{cases} \Rightarrow \begin{cases} \mathrm{a_{py}} = 0.40\mathrm{g} \\ \mathrm{d_{py}} = 3.5\ \mathrm{cm} \end{cases}$$

/	Direction X-X
$\beta_0 = \dfrac{63.7(a_y d_{pi} - d_y a_y)}{a_{pi} d_{pi}}$	6.08%
$\beta_{eff} = \lambda\beta_0 + 5$	9.328%
$SR_a = \dfrac{3.21 - 0.681 ln\beta_{eff}}{2.12}$	0.796 g
$SR_v = \dfrac{2.31 - 0.41 ln\beta_{eff}}{1.65}$	0.845 g
$T_s = {SR_v.C_V}/{2.5.SR_a.C_A}$	0.62 s
$S_A = 2.5.SR_a.C_A$	0.636g
$S_d(T_s) = S_A.g.100(\frac{T_s}{2\pi})^2$	6.08 cm

Table V.18: Results of the fourth iteration direction(X-X)

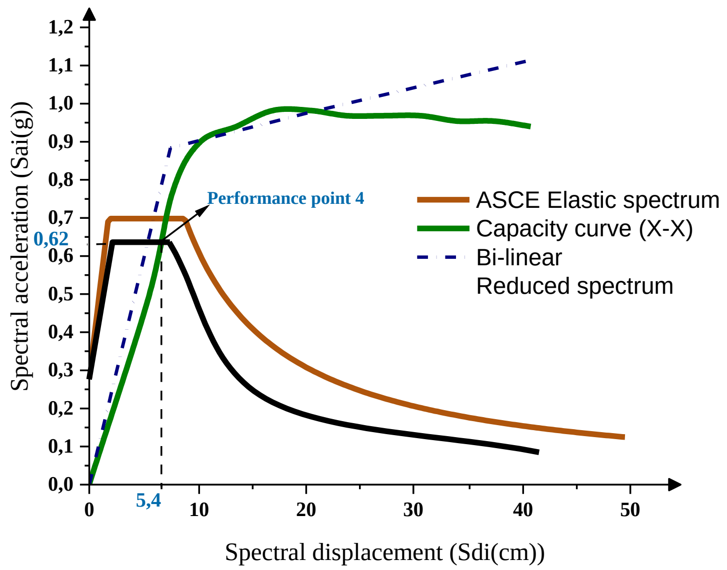


Figure V.33: Performance point of the 4th iteration direction(X-X)

Performance point 4: $\begin{cases} a_4 = 0.62g \\ d_4 = 5.4cm \end{cases}$

Error: $100\left(\frac{5.89-5.4}{5.89}\right) = 8.319\% > 5\%$ $\Rightarrow$ So fifth iteration is required.

5th iteration:

$$\begin{cases} \mathrm{a_{pi}} = \frac{0.62+0.575}{2} = 0.597\mathrm{g} \\ \mathrm{d_{pi}} = \frac{5.4+5.89}{2} = 5.7\mathrm{cm} \end{cases} \Rightarrow \begin{cases} \mathrm{a_{py}} = 0.3\mathrm{g} \\ \mathrm{d_{py}} = 2.6\ \mathrm{cm} \end{cases}$$

/	Direction X-X
$\beta_0 = \dfrac{63.7(a_y d_{pi} - d_y a_y)}{a_{pi} d_{pi}}$	2.67%
$\beta_{eff} = \lambda\beta_0 + 5$	6.788%
$SR_a = \dfrac{3.21 - 0.681 ln\beta_{eff}}{2.12}$	0.89 g
$SR_v = \dfrac{2.31 - 0.41 ln\beta_{eff}}{1.65}$	0.92 g
$T_s = {SR_v.C_V}/{2.5.SR_a.C_A}$	0.60s
$S_A = 2.5.SR_a.C_A$	0.69g
$S_d(T_s) = S_A.g.100(\frac{T_s}{2\pi})^2$	6.32 cm

Table V.19: Results of the fifth iteration direction(X-X)

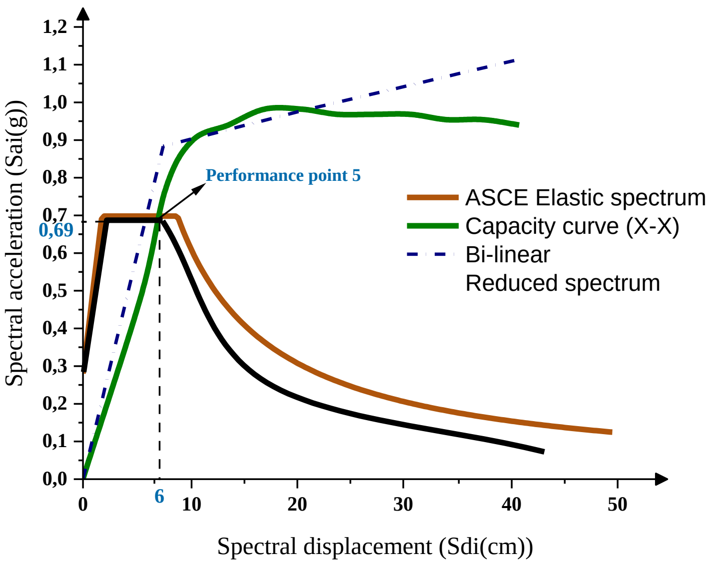


Figure V.34: Performance point of the 5th iteration direction(X-X)

Performance point 5: $\begin{cases} a_4 = 0.69g \\ d_4 = 6cm \end{cases}$

Error: $100\left(\frac{6-5.7}{6}\right) = 5\% = 5\% \quad \Rightarrow$ verified condition.

So, the performance point is defined by a displacement: $\boldsymbol{d = 6cm}$

Direction Y-Y:

1st iteration:

$$\begin{cases} a_{pi} = 0.603g \\ d_{pi} = 15.429cm \end{cases} \Rightarrow \begin{cases} a_{py} = 0.54\ g \\ d_{py} = 4.048cm \end{cases}$$

/	Direction Y-Y
$\beta_0 = \frac{63.7(a_y d_{pi} - d_y a_y)}{a_{pi} d_{pi}}$	40.33%
$\beta_{eff} = \lambda\beta_0 + 5$	27.58%
$SR_a = \frac{3.21 - 0.681 ln\beta_{eff}}{2.12}$	0.448 g
$SR_v = \frac{2.31 - 0.41 ln\beta_{eff}}{1.65}$	0.575 g
$T_s = {SR_v.C_V}/{2.5.SR_a.C_A}$	0.754s
$S_A = 2.5.SR_a.C_A$	0.358g
$S_d(T_s) = S_A.g.100(\frac{T_s}{2\pi})^2$	5.06 cm

Table V.20: Results of the first iteration direction(Y-Y)

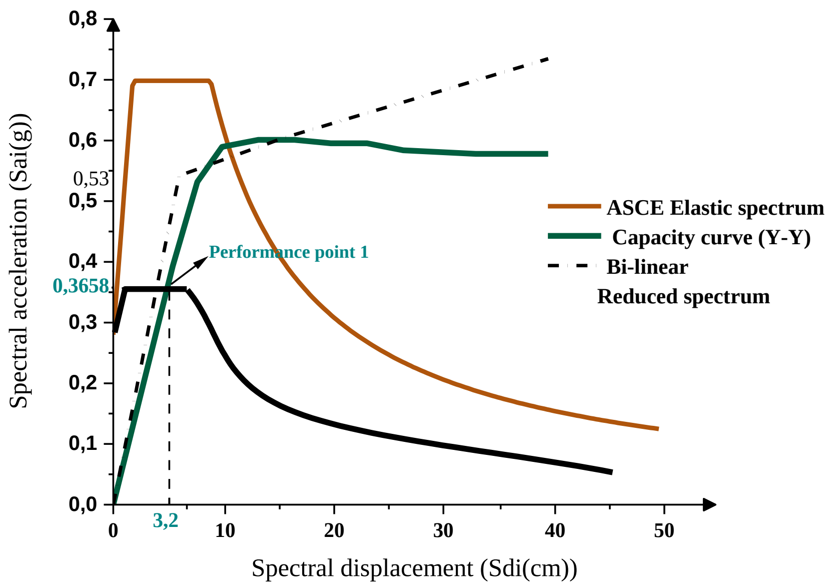


Figure V.35: Performance point of the first iteration direction(Y-Y)

Performance point 1: $\begin{cases} a_1 = 0.3658\ g \\ d_1 = 3.2cm \end{cases}$

Error: $100\left(\frac{15.429-3.2}{15.429}\right) = 79.25\% = 5\%$ $\Rightarrow$ a second iteration is required.

2nd iteration:

$$\begin{cases} a_{pi} = \frac{0.603+0.3658}{2} = 0.48g \\ d_{pi} = \frac{15.429+3.2}{2} = 9.31cm \end{cases} \Longrightarrow \begin{cases} a_{py} = 0.45\ g \\ d_{py} = 5.8cm \end{cases}$$

/	Direction Y-Y
$\beta_0 = \dfrac{63.7(a_y d_{pi} - d_y a_y)}{a_{pi} d_{pi}}$	20.03%
$\beta_{eff} = \lambda\beta_0 + 5$	18.42%
$SR_a = \dfrac{3.21 - 0.681 ln\beta_{eff}}{2.12}$	0.578 g
$SR_v = \dfrac{2.31 - 0.41 ln\beta_{eff}}{1.65}$	0.676 g
$T_s = {SR_v . C_V}/{2.5 . SR_a . C_A}$	0.687
$S_A = 2.5 . SR_a . C_A$	0.462g
$S_d(T_s) = S_A . g . 100(\frac{T_s}{2\pi})^2$	5.42 cm

Table V.21: Results of the second iteration direction(Y-Y)

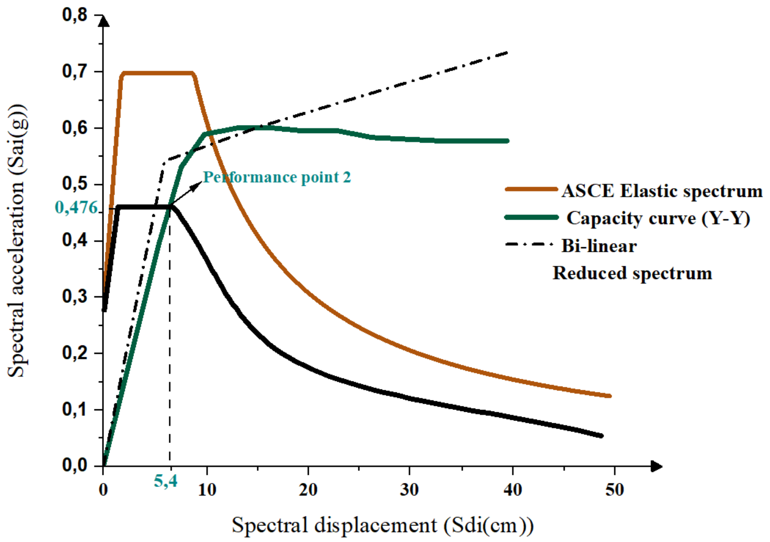


Figure V.36: Performance point of the 2nd iteration direction(Y-Y)

Performance point 2: $\begin{cases} a_2 = 0.476\ g \\ d_2 = 5.4cm \end{cases}$

Error: $100\left(\frac{9.31-3.2}{9.31}\right) = 41.99\% > 5\%$ $\Rightarrow$ a third iteration is required.

3rd iteration:

$$\begin{cases} a_{pi} = \frac{0.476+0.48}{2} = 0.478g \\ d_{pi} = \frac{9.31+5.4}{2} = 7.405cm \end{cases} \Rightarrow \begin{cases} a_{py} = 0.4\ g \\ d_{py} = 4\ cm \end{cases}$$

/	Direction Y-Y
$\beta_0 = \dfrac{63.7(a_y d_{pi} - d_y a_y)}{a_{pi} d_{pi}}$	18.89%
$\beta_{eff} = \lambda\beta_0 + 5$	117.65%
$SR_a = \dfrac{3.21 - 0.681 ln\beta_{eff}}{2.12}$	0.63 g
$SR_v = \dfrac{2.31 - 0.41 ln\beta_{eff}}{1.65}$	0.75 g
$T_s = {SR_v.C_V}/{2.5.SR_a.C_A}$	0.69 s
$S_A = 2.5.SR_a.C_A$	0.504g
$S_d(T_s) = S_A.g.100(\frac{T_s}{2\pi})^2$	5.96 cm

TableV.22: Results of the third iteration direction(Y-Y)

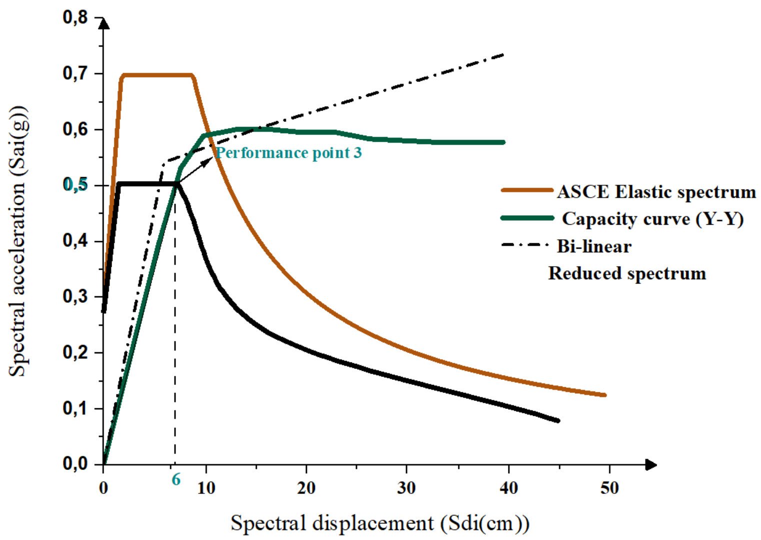


Figure V.37: Performance point of the 3rd iteration direction(Y-Y)

Performance point 3: $\begin{cases} a_3 = 0.5\ g \\ d_3 = 6cm \end{cases}$

Error: $100\left(\frac{7.405-6}{7.405}\right) = 18.97\% > 5\%$ $\Rightarrow$ a fourth iteration is required.

4th iteration:

$$\begin{cases} a_{pi} = \frac{0.5+0.478}{2} = 0.49g \\ d_{pi} = \frac{6+7.405}{2} = 6.7cm \end{cases} \Rightarrow \begin{cases} a_{py} = 0.35g \\ d_{py} = 3.5cm \end{cases}$$

/	Direction Y-Y
$\beta_0 = \frac{63.7(a_y d_{pi} - d_y a_y)}{a_{pi} d_{pi}}$	12.22%
$\beta_{eff} = \lambda\beta_0 + 5$	13.18%
$SR_a = \frac{3.21 - 0.681 ln\beta_{eff}}{2.12}$	0.69g
$SR_v = \frac{2.31 - 0.41 ln\beta_{eff}}{1.65}$	0.77 g
$T_s = {SR_v.C_V}/{2.5.SR_a.C_A}$	0.65 s
$S_A = 2.5.SR_a.C_A$	0.53g
$S_d(T_s) = S_A.g.100(\frac{T_s}{2\pi})^2$	5.78cm

Table V.23: Results of the fourth iteration direction(Y-Y)

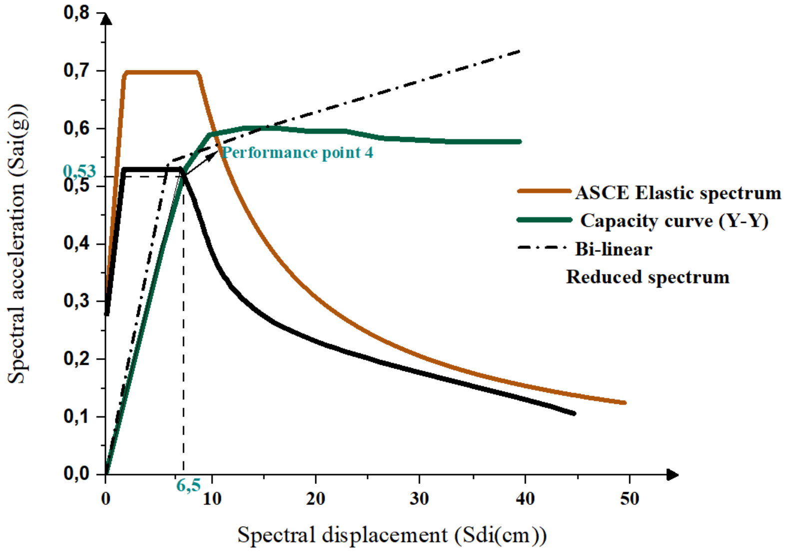


Figure V.38: Performance point of the 4th iteration direction(Y-Y)

Performance point 4: $\begin{cases} a_4 = 0.53\ g \\ d_4 = 6.5cm \end{cases}$

Error: $100\left(\frac{6.7-6.5}{6.7}\right) = 1.97\% < 5\%$ $\Rightarrow$ verified condition.

✓ The displacements (Performance point) from the pushover analysis are summarized it the following table:

	ATC40		RPA	
	Direction X-X	Direction Y-Y	Direction X-X	Direction Y-Y
Iteration number	5	4	4	4
Sdi(cm)	6	6.5	7	7.25
Sai(g)	0.69	0.53	0.78	0.54

Table V.24: Performance point results

Commentary:

According to Table (V.24), it can be observed that the results for both response spectrums (RPA and ATC) have practically the same results. Therefore, we will focus only on the results of the RPA response spectrum.

V.2.2.1.4 Determination of the performance level:

In order to determine the performance level of our structure, we compared the performance points with the performance levels obtained from the SeismoStruct software.

We have:

Direction X-X:

	Base shear (V)	Displacement (cm)
Operational Level	6700 KN	12.7 cm
Immediate Occupancy	6900 KN	16.22 cm
Performance Point	5560 KN	10.268 cm

Table V.25: Performance levels direction (X-X)

According to the **Table (V.25)** the performance level of the structure in direction (X-X) didn't exceed the operational level.

Direction Y-Y:

	Base shear (V)	Displacement (cm)
Operational Level	4200 KN	4.94 cm
Immediate Occupancy	4600 KN	6.20 cm
Performance Point	4671 KN	10.99cm

Table V.26: Performance levels direction (Y-Y)

According to the **Table (V.26**) the performance level of the structure in direction (Y-Y) exceeded the immediate occupancy level and reached the life safety level.

V.2.2.1.5 Determination of displacements (pushover):

Levels	h_k (cm)	Direction X-X	Direction Y-Y
		δ_{ek} (cm)	δ_{ek} (cm)
G floor	306	1.2	0.8
1st floor	306	2.9	2.2
2nd floor	306	4.6	3.9
3rd floor	306	6.1	5.7
4th floor	306	7	7.25

TableV.27: Inter story displacement

V.2.2.1.6 The influence of shear walls on the structural capacity:

Direction X-X:

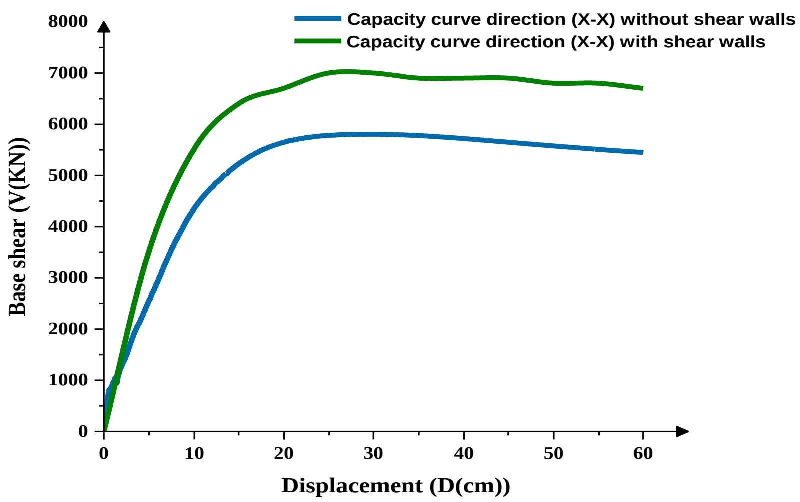


Figure V.39: Influence of the shear walls on the structural capacity direction (X-X)

Direction Y-Y:

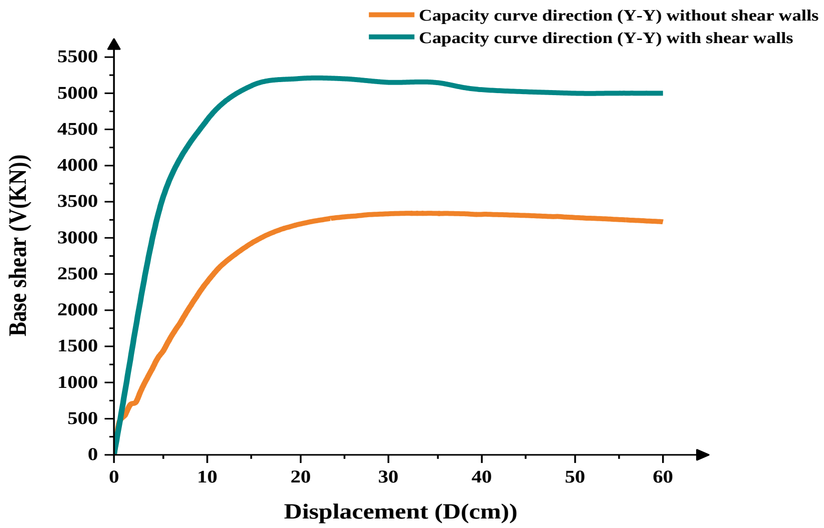


Figure V.40: Influence of the shear walls on the structural capacity direction (Y-Y)

Commentary:

- Despite the removal of shear walls, the structure reached the target displacement with no significant damage, the change was spotted in the capacity which means that our structure is capable of resisting with no need of shear walls.
- The inclusion of shear walls in a structure enhances its capacity (strength by stiffness).

PART II: NON-LINEAR DYNAMIC ANALYSIS:

V.3 Dynamic time history analysis:

Accelerograms are recordings of ground acceleration during seismic events, such as earthquakes. Filtering and correction of accelerograms are essential steps in the processing and analysis of seismic data. These steps help enhance the quality and reliability of the recorded signals, allowing for better interpretation and understanding of the seismic event. Filtering accelerograms typically involves removing unwanted noise and frequency content that may interfere with the analysis. Commonly used filters include low-pass filters, high-pass filters, band-pass filters, and notch filters. The specific filter type and parameters depend on the characteristics of the recorded accelerograms and the intended analysis.

V.3.1 Time history simulation:

In this part we are going to correct the accelerograms using "seismoSignal software".

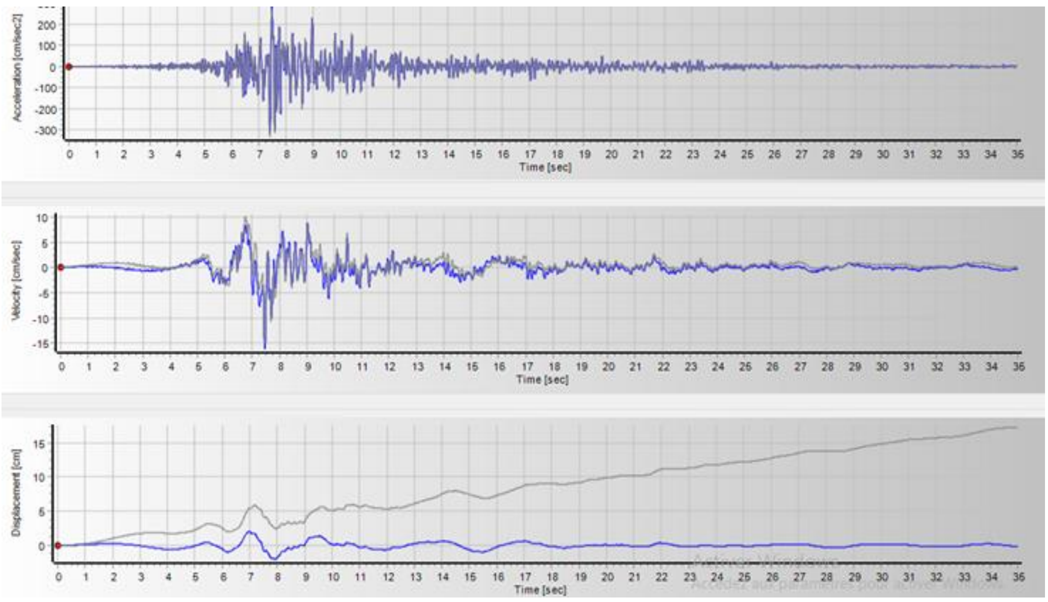
SeismoSignal: SeismoSignal constitutes an easy and efficient way for signal processing of strong-motion data, featuring a user-friendly visual interface and being capable of deriving a number of strong-motion parameters often required by engineer seismologists and earthquake engineers.

The program is able to read accelerograms saved in different text file formats, which may then be filtered and baseline-corrected. Polynomials of up to the 3rd order may be employed for the latter, whilst three different digital filter types are available, all of which capable of carrying out, causal or a-causal, high-pass, low-pass, band-pass and band-stop filtering.

V.3.1.1. The Steps of correcting an accelerogram by seismoSignal:

V.3.1.1.1. Filters:

Filters are used to attenuate or even eliminate certain frequency bands. That is usually necessary when, in the curve of speed or displacement versus time, an unusual ripple is observed. These filters are applied by decomposing the signal in Fourier series, eliminating certain amplitudes and recomposing the accelerogram corrected. It is possible to remove the low frequencies, the high frequencies or the frequencies contained in an intermediate band



FigureV.41: Acceleration, Velocity and Displacement of the Filtered Earthquake in bleu and unfiltered in gray.

V.3.1.1.1. Baseline correction:

This correction removes any linear drift or bias in the signal caused by instrumentation or environmental factors. It involves fitting a trend line to the baseline of the accelerogram and subtracting it from the original signal.

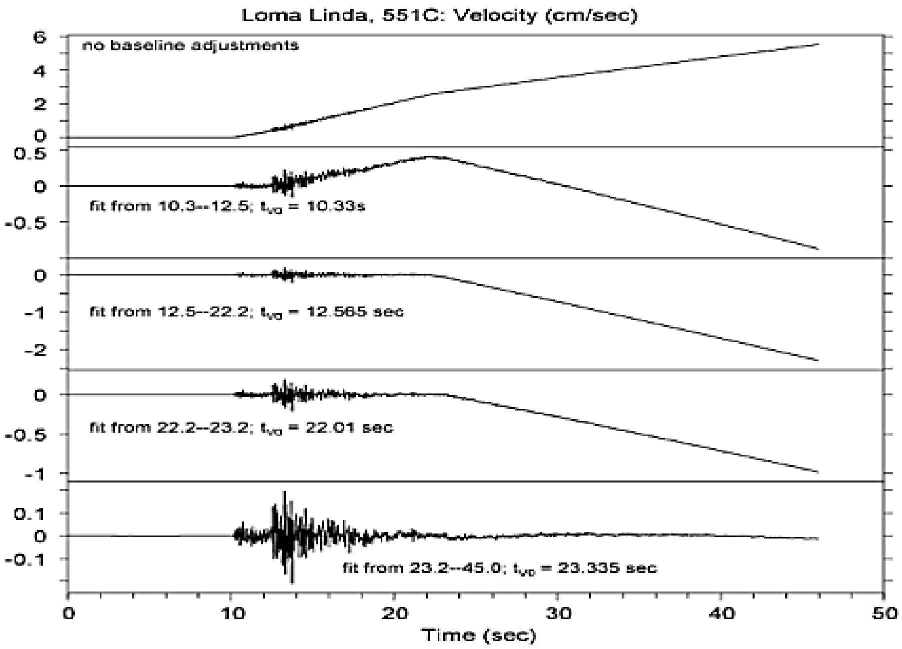


Figure.V.42: Velocity as a function of time of the earthquake recording before and after guideline correction.

V.3.1.2 Results of the non linear dynamic analysis:

- **Pre –processor:**

1. We choose dynamic non linear analysis: 

2. We define the accelerogram concerned by the analysis, in the tab **time-history curve**, after we select **load**.
3. a time step of 0.01 sec is introduced
4. a time step of 0.01 sec is introduced
5. Application of seismic loads in the form of acceleration at the level of the foundations.

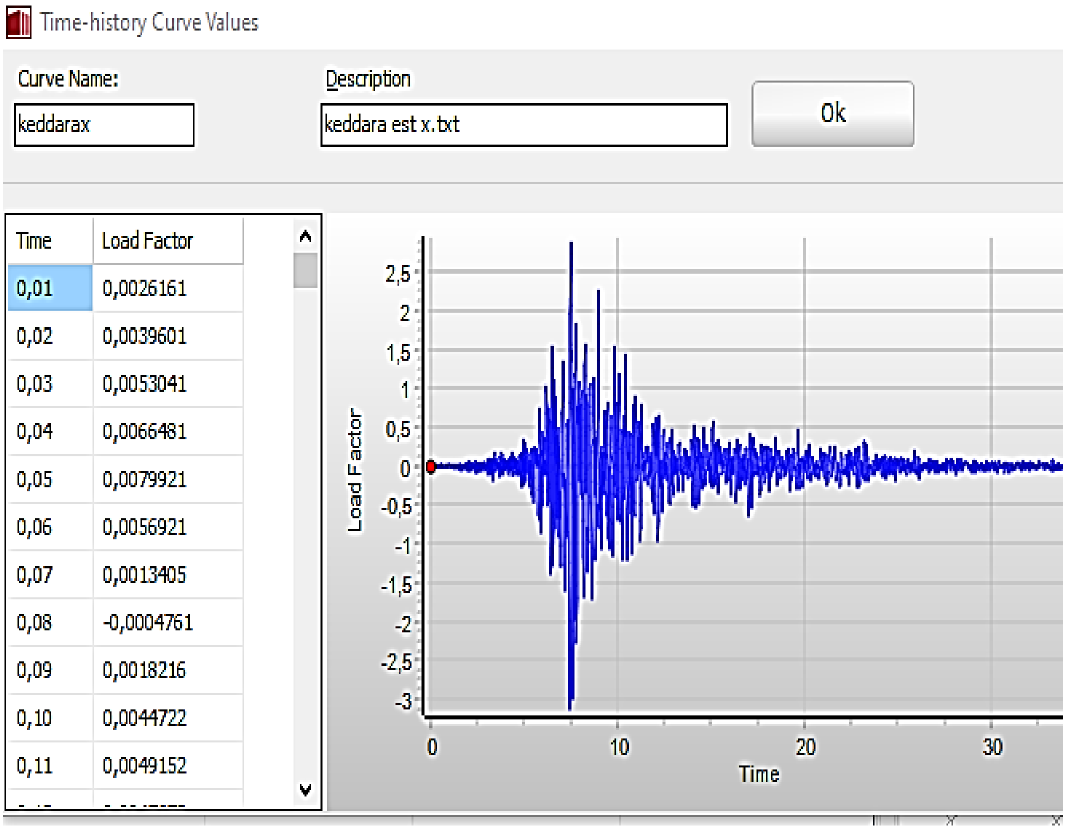


Figure V.43: Accelerogram definition

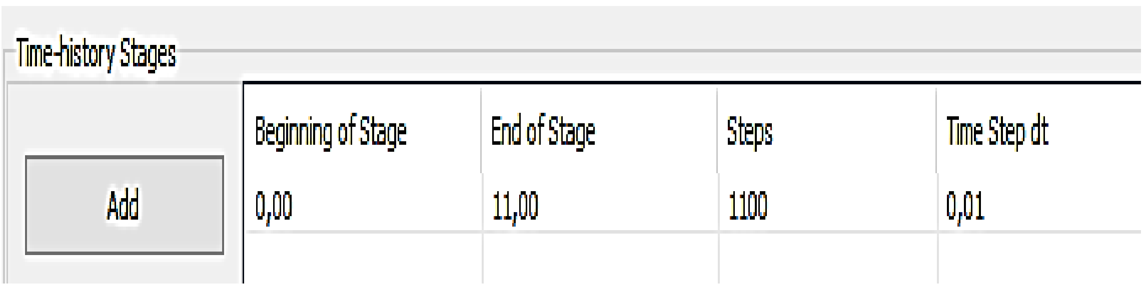
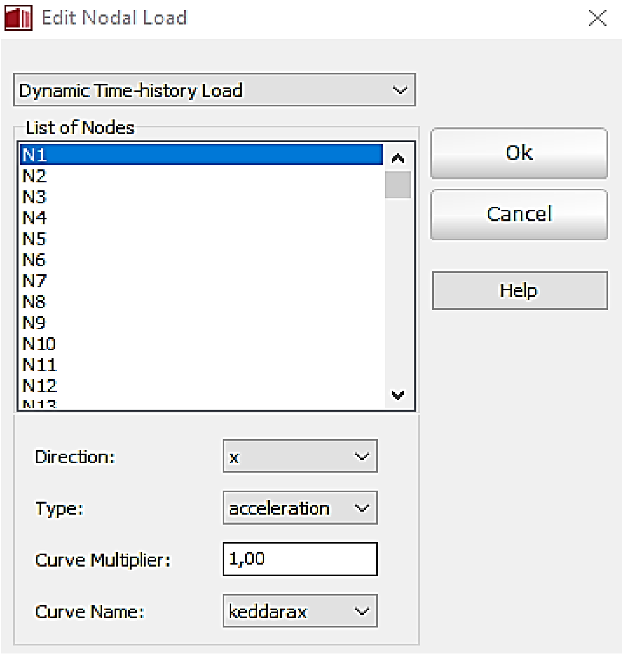


Figure V.44: Step definition



FigureV.45: application of seismic loads.

The accelerograms used:

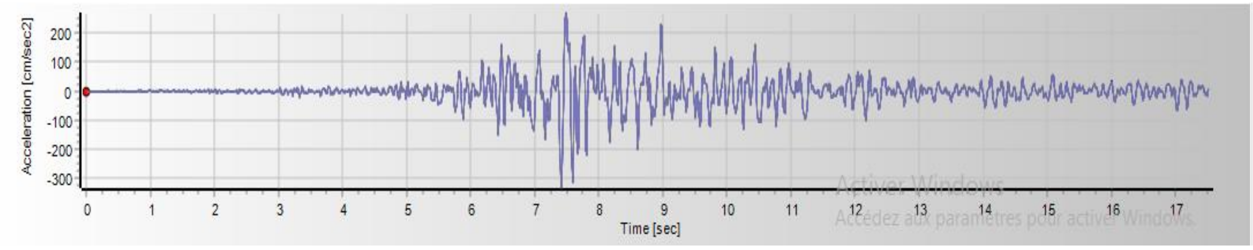


Figure V.46.1: Accelerogram 1 pga=0.3g.

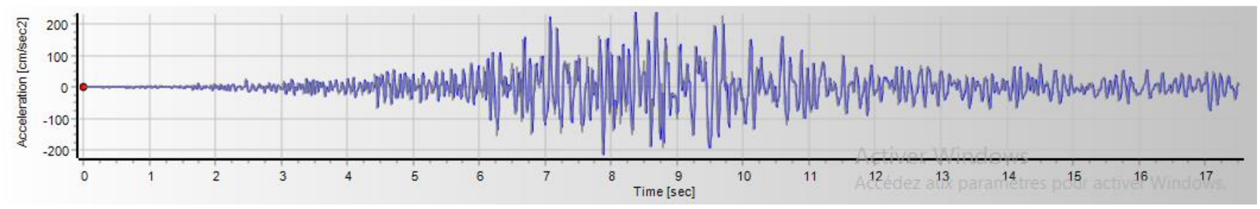


Figure V.46.2: Accelerogram 2 pga=0.21g.

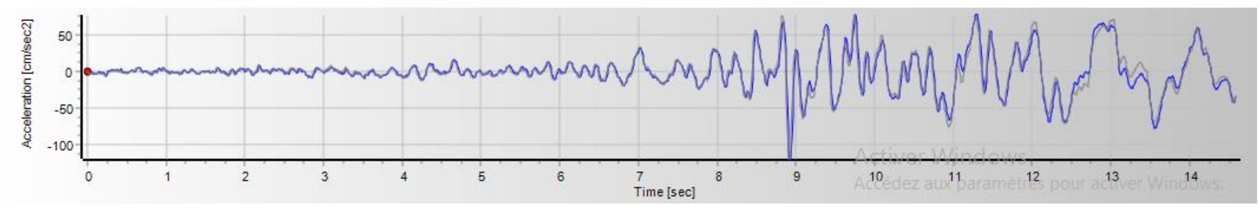
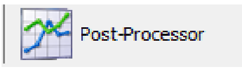


Figure.V.46.3: Accelerogram pga=0.17g

- **Processor:**

In this phase, you can launch the analysis by clicking on **run analysis** 

- **Post- processor:**

To see the different results of the structure we clic on 

<u>Displacement direction x-x:</u>

Floors	DNTHA Acc1	DNTHA Acc2	DNTHA Acc3
1	2	2	1.2
2	2.1	2.4	1.4
3	2.2	2.7	1.5
4	2.2	3	1.6
5	2.3	3.3	1.7

Table V.28: Displacement direction x-x

<u>Displacement direction y-y:</u>

Floors	DNTHA Acc1	DNTHA Acc2	DNTHA Acc3
1	2	2.6	1.3
2	2.4	2.8	1.4
3	2.9	3	1.7
4	3.3	3.3	1.8
5	3.6	3.5	2

Table V.29: Displacement y-y

V.4 Comparison between linear and non linear analysis:

Direction (X-X):

Floors	DNTHA Acc1	DNTHA Acc2	DNTHA Acc3	Push X	dynamic linear analysis
1	2	2	1.2	1.2	0,116
2	2.1	2.4	1.4	2.9	0,313
3	2.2	2.7	1.5	4.6	0,51
4	2.2	3	1.6	6.1	0,678
5	2.3	3.3	1.7	7	0,801

Table V.32: comparison of displacement direction (X-X)

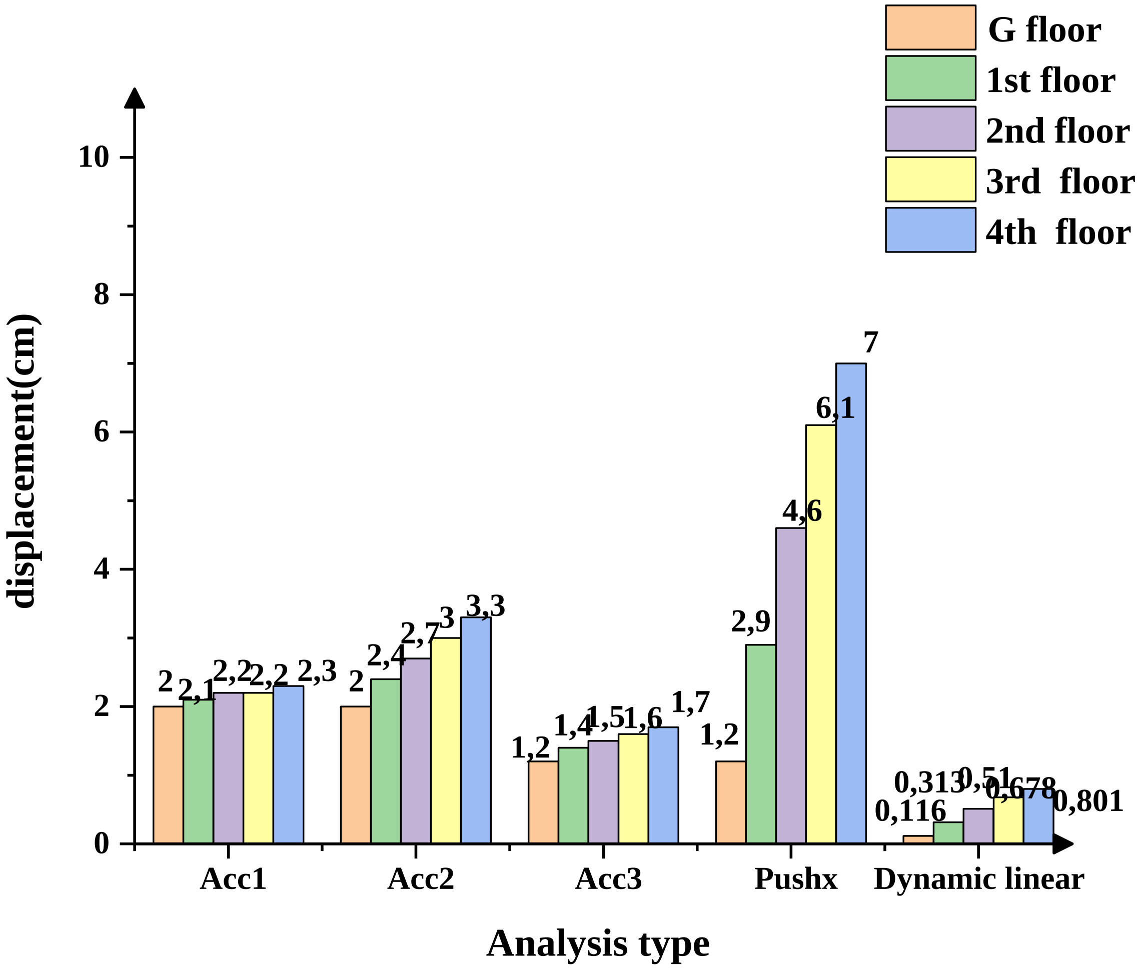


Figure V.47: comparison of displacement direction (X-X)

Direction (Y-Y):

Floors	DNTHA Acc1	DNTHA Acc2	DNTHA Acc3	Push Y	dynamic linear analysis
1	2	2.6	1.3	0.8	0,145
2	2.4	2.8	1.4	2.2	0,433
3	2.9	3	1.7	3.9	0,757
4	3.3	3.3	1.8	5.7	1,059
5	3.6	3.5	2	7.25	1,316

Table V.33: comparison of displacement direction (Y-Y)

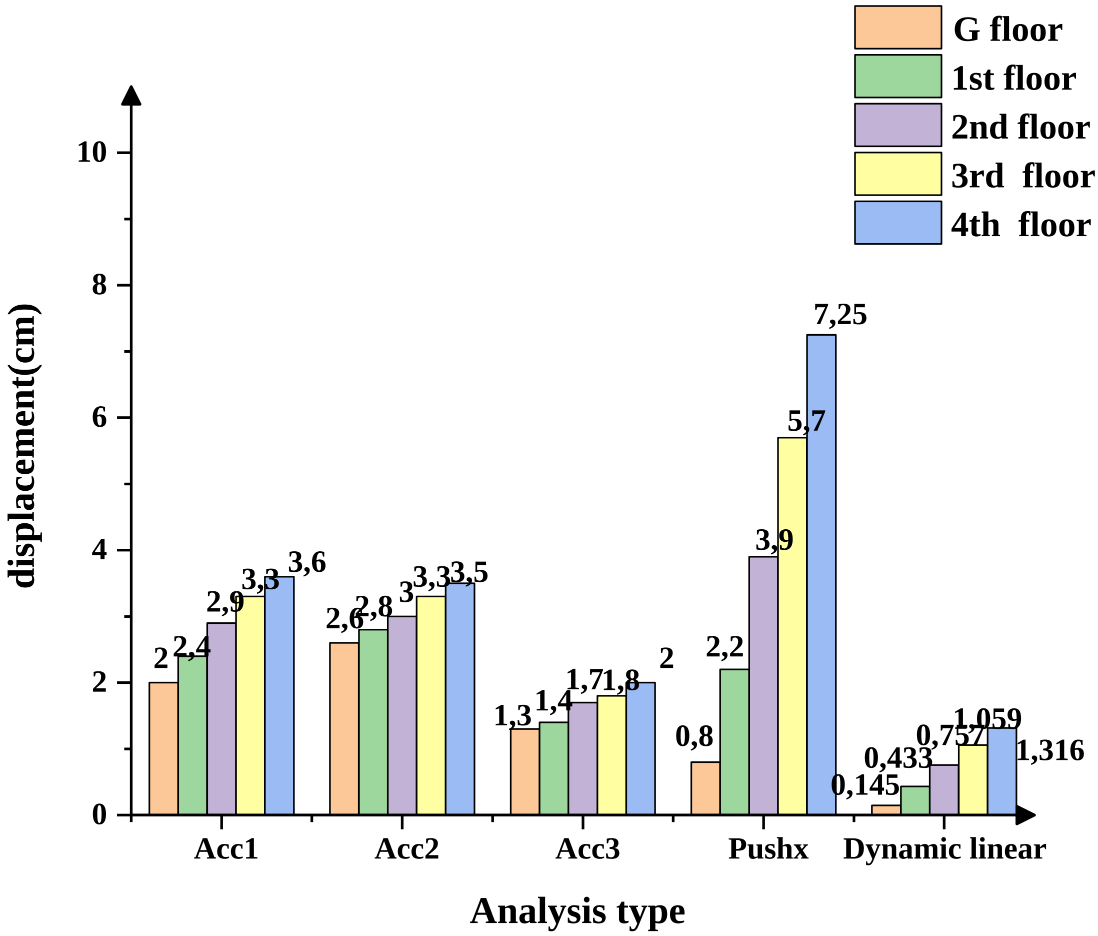


Figure V.48: comparison of displacement direction (Y-Y)

Commentary:

- According to the results of floor displacements of the three types of analysis, it can be seen that the response nonlinear dynamic oscillates between nonlinear static responses and the dynamic linear analysis.

- The application of the Pushover has allowed us to follow the behavior of the structure and know the performance level.
- The error between the two nonlinear procedures probably amounts to the difference between the base data from both methods. The loading distribution developed in the time history varies during the earthquake according to the plastic deformation scheme of the structure, but in pushover analysis the distribution is constant (every increment).

- Using at least three accelerograms, precise the results of the time history analysis
- The displacements obtained through spectral modal analysis are close to those obtained through dynamic analysis using accelerograms, which once again validates the use of these accelerograms for three-dimensional structures.

V.5. Conclusion:

In this chapter we performed both non linear analyses to our structure, using the software “seismoStruct”, in the first part the pushover analysis was done manually using the capacity method mentioned in atc-40, the analysis was done in the two principal directions x-x/y-y and the performance point was reached allowing us to know the performance level of the structure. By the end of this part we have compared between the structure with and without shear walls concluding that we can leave the use the shear walls.

In the second part before applying the time history analysis we had to correct the accelerograms we choose, the correction and the filtering was done by “SeismoSignal”, the analysis results were compared to the results of the non linear static analysis (push over) and the dynamic linear analysis.

We have seen that the time history results are more precise then the two other methods because it’s a step-by-step direct integration over a time interval And base on real seismic load (accelerogram).

General conclusion

General conclusion:

In summary, this project provides a comprehensive analysis of a building's response, considering both linear and nonlinear effects. By employing the time history method and performance-based design principles, the study aims to enhance the seismic resilience of the structure.

this work involved a comprehensive study of a building consisting of a ground floor and four upper floors. The study was divided into two main parts, each addressing different aspects of the analysis and design process.

The project started with a linear dynamic analysis conducted by ETABS program, which generated crucial results for reinforcing the various structural elements of the building

The second part of the study was dedicated to investigating nonlinearity using both the nonlinear static and nonlinear dynamic methods. This analysis formed the foundation for the application of a performance-based design approach, which aims to replicate the realistic behavior of the building under seismic conditions.

In general, the nonlinear static method provides more comprehensive information compared static elastic or even dynamic analysis. However, it is not a universal solution for all cases. While useful for identifying weak points and potential design failures, it may not capture all possible failure mechanisms.

It is important to note that the analysis remains static, and it cannot provide an accurate representation of dynamic phenomena. For example, the method may fail to detect certain significant modes of deformation while overestimating others. The inelastic dynamic response can differ significantly from the response obtained using constant lateral load distributions. Significantly different results can be expected, especially for structures heavily influenced by high-frequency vibration modes.

In conclusion, the work carried out has highlighted the addressed issue. Several perspectives and recommendation on the current study can be stated:

- Based on the results of nonlinear analysis and for economic reasons, it is concluded that the reinforcement and cost of the building can be minimized by utilizing these methods.
- Nonlinear analysis can be employed to assess the seismic performance of existing structures and identify potential weaknesses that require attention.

- The push-over analysis can be expanded to an urban scale. It has been successfully utilized in previous studies to evaluate the seismic performance of urban areas and identify vulnerabilities that need to be addressed.
- It is crucial to verify and validate the results obtained from the nonlinear static analysis through experimental testing or comparison with real-life case studies. This will help in assessing the reliability and accuracy of the analysis method and ensuring the plausibility of the identified failure mechanisms.
- It is recommended to supplement the nonlinear static analysis with other methods, such as nonlinear dynamic analysis, to comprehensively capture the structural behavior under various loading scenarios. This will enhance the understanding of the structure's response.
- It is proposed to introduce the pushover method in the next revision of the Algerian seismic code (RPA).

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Annex

Annex: Soil repport

IV- ESSAIS AU LABORATOIRE

4-1 organisations des essais

Après la réception des échantillons en laboratoire, nous avons effectué des essais et des analyses au laboratoire sur les encroûtements calcaires sableux, pour la détermination des caractéristiques physico-mécaniques, ainsi que les analyses chimiques, à savoir :

Paramètres physiques

- Teneur en eau naturelle (Wn)
- Densité en place humide et sèche (γh) et (γd)
- Degrés de saturation (Sr %)
- Granulometrie

Paramètres mécaniques

- Cisaillement:
- Cohésion non drainée (C_{uu})
- Angle de frottement (Φ_{uu})

Analyse chimique

- Teneur en Sulfates ($So4^{-2}$)
- Teneur en carbonates ($CaCo_3$)

4-2 interprétations des résultats physiques

Sondage		Identification					
N°Sond	Prof. (m)	W (%)	γ (t/m³)	γd (t/m³)	Sr (%)	2 (mm)	0.08 (mm)
S1	2.5/3.0	6	1.68	1.57	24	92	32
	5.0/5.5	8	1.70	1.56	28	90	30
S2	2.5/3.0	6	1.70	1.60	25	87	28
	6.0/6.5	7	1.72	1.60	26	85	32
S3	2.5/3.0	6	1.68	1.59	23	87	35
	5.0/5.5	6	1.67	1.57	25	84	33
S4	2.5/3.0	8	1.74	1.60	34	86	30
	5.5/6.0	7	1.70	1.58	28	85	26
S5	2.5/3.0	6	1.65	1.56	25	87	30
	5.0/5.5	7	1.71	1.59	30	85	26
S6	2.5/3.0	5	1.66	1.57	23	88	30
	6.0/6.5	7	1.70	1.58	29	76	28

Les mesures de teneur en eau ont été effectuée sur les encroutements calcaires, valeurs trouvées sont comprises entre 5% à 8%. Le degré de saturation oscille entre 25 ... %. Une telle fourchette dénote un état hydrique naturel d'un sol sec.

***- Densité humide et sèche (γh) et (γd)**

Les valeurs des densités sèches varient pour les encroutements calcaires entre 1.56 et 1.6 t/m³. Pour Les densités humides apparentes sont de l'ordre de 1.66 à 1.74 t/m³. D'après la norme géotechnique, cette formation se situe dans les familles des sols semi denses.

Granulométrie

L'analyse granulométrie effectuée sur les encroutements calcaires a montré que le pourcentage des passants à 0.08 mm est inferieur de 40 %, il s'agit d'une formation grenue.

4-3.Analyses des caractéristiques mécaniques:

Les paramètres mécaniques permettent d'accéder, directement à la capacité portante des sols, compatibles avec une déformation (tassement) acceptables.

Nous avons utilisé pour déterminer ces paramètres la boite de Casagrande pour l'essai de cisaillement.

Sondage		Cisaillement	
N° Sond	Prof. (m)	C_{uu} (bars)	$Ø_{uu}$ (°)
S1	2.5/3.0	0.17	22
	5.0/5.5	0.21	21
S3	2.5/3.0	0.14	23
	5.0/5.5	0.24	19
S4	2.5/3.0	0.20	21
	5.5/6.0	0.18	22
S6	2.5/3.0	0.13	24
	6.0/6.5	0.19	23

***- Cisaillement (Selon NF P94-071-1):**

Par caractéristiques mécaniques de cisaillement, nous entendons la cohésion et l'angle de frottement interne qu'on déduit de l'essai de cisaillement. C'est ainsi que nous avons réalisé des essais de nature UU, ils ont été effectués à l'aide de la machine de cisaillement rectiligne à une vitesse de 1 mm/min.

Les composantes normales des contraintes totale (σ) et effective (σ') et la pression interstitielle ($\bar{u}$) de l'eau sont liées par la relation : σ = σ' + u

Qadm = Rp min/α

Qadm: contrainte admissible

Rp min: résistance de pointe minimale

α : coefficient variant de 20 à 30

Le sol en profondeur testé au pénétromètre dynamique lourd a indiqué une résistance minimale supérieur à 90 bars, l'estimation de la contrainte admissible d'un essai à un autre est comme suit :

Pénétromètre	P1	P2	P3	P4	P5	P6
Rp min (bars)	90	90	90	90	90	90
Q adm (bars)	**3.0**	**3.0**	**3.0**	**3.0**	**3.0**	**3.0**

6-3 Aptitude du sol aux fondations

Les deux méthodes de calcul de fondation ont aboutie à des valeurs de contrainte, ayant un écart moyenne entre eux, dont les valeurs varient entre 2.3 à 3.0 bars, donc la contrainte à prendre en considération est de l'ordre de **2.3 bars**, l'ancrage des fondations sera considéré à partir de **1.5 m** de profondeur.

6-4 évaluations des tassements

Etant donné que dans l'optique de la constructibilité, les sols appelés à constituer des assises de fondation courante sont essentiellement des sols pulvérulents notamment en profondeur, constitué par des formations villafranchiennes. Il s'agit des croutes et des encroutements calcaires. Ces formations étant réputées incompressibles, et ne poseront donc pas de problème d'instabilité vis à vis de la déformation verticale.

VIII- CONCLUSION GENERALE

Le projet de la Polyclinique à Tadjnanet, est implanté sur un terrain en pente dou orienté vers le Nord, le site est stable et ne pose aucun problème liée à la topographie.

La reconnaissance in situ que nous avons mené à montré que le sol est constitué par une formation qui appartienne au villafranchien, il s'agit d'une succession des croutes calcaires compactes et des encroutements calcaire devenant graveleux en profondeur.

La résistance à la pénétration dynamique varie d'une façon ordonnée à partir de la surface jusqu'à la profondeur de refus, la résistance minimale pour ces essais est très élevés, elles dépassent les 100 bars dans le premier mètre de profondeur. Le refus à la pénétration de la pointe, se situe proche de la surface, cependant, ces refus sont dus principalement à la forte compacité des croutes calcaires.

En laboratoire, les échantillons testés qui appartient à l'encroutement calcaire, ont montré qu'il s'agit d'un sol grenu où la fraction fine est faible, leur densité sèche caractérise d'un sol semi dense, leur degré d'humidité naturel est sec. Cette formation est caractérisée par des valeurs faibles de la cohésion, et des valeurs l'angle de frottement un peu élevées.

S'agissant de la composition chimique, les sols testés ne renfermant aucune présence de sulfate, sot une agressivité nulle, un ciment normal peut convenir dans la confection de béton de fondation, les taux de carbonates sont fortes.

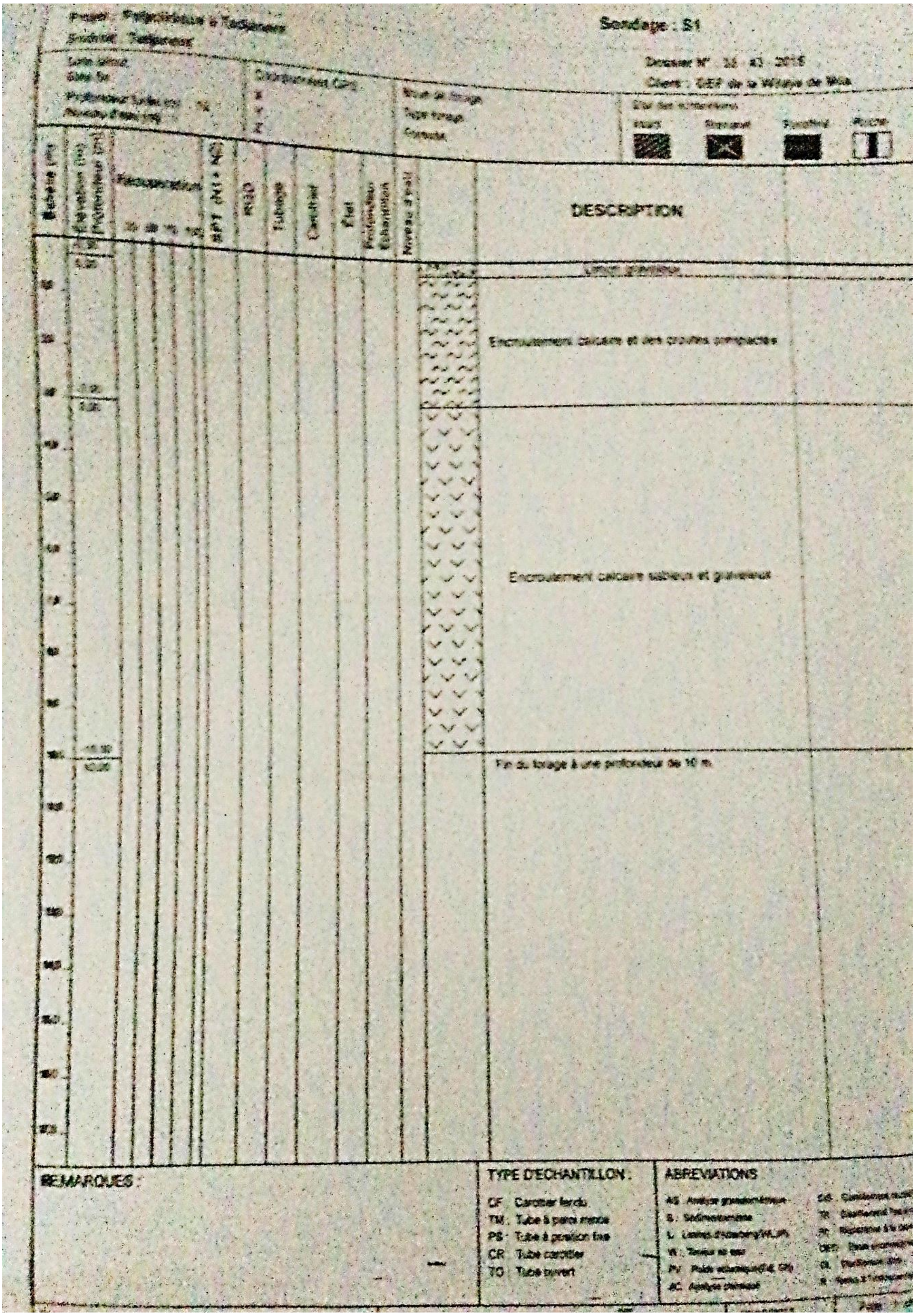
En basant sur toutes ces données, nous préconisons de retenir une contrainte admissible du sol de **2.3 bars** et de fonder l'ouvrage sur une semelle superficielle, ancrée à partir de **1.5 m**, les tassements seront négligeables.

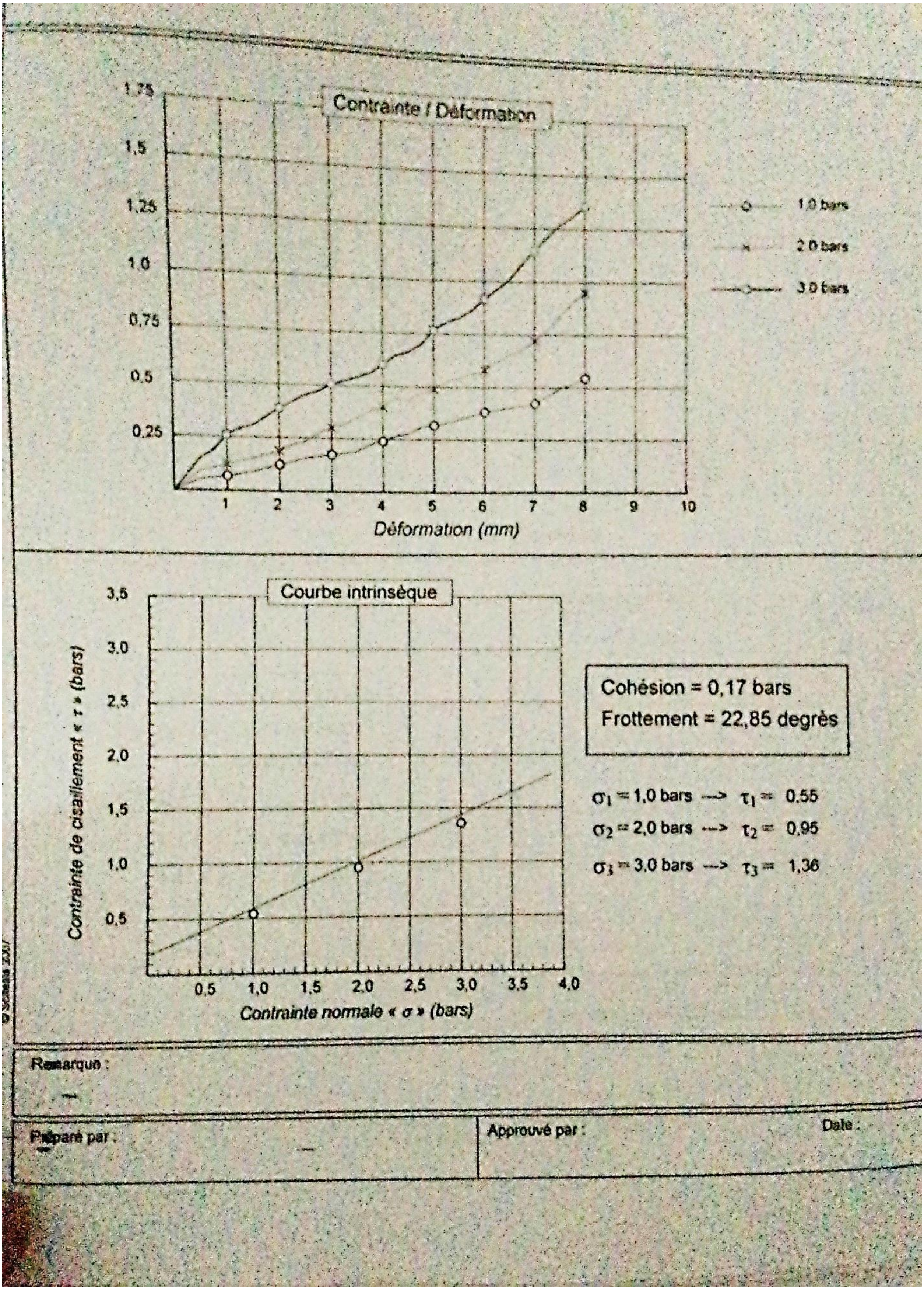
L'ensemble des donnés géologique et géotechnique permet de classer le site dans la catégorie S3 de RPA.

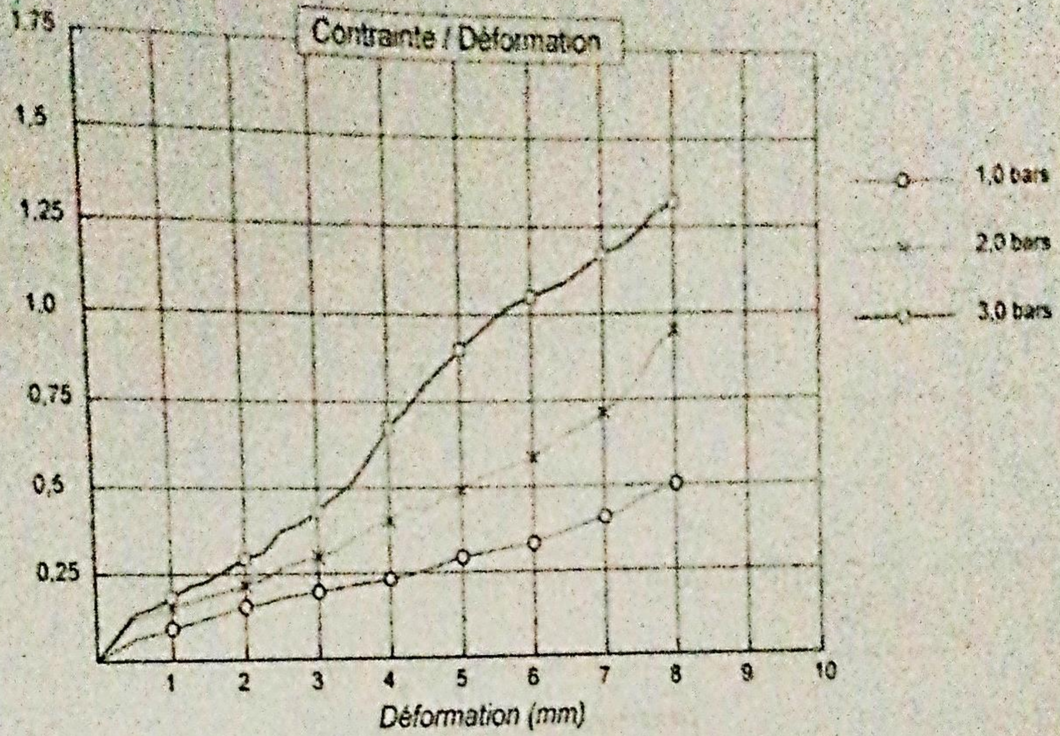
Nous restons, à disposition de notre client pour tout éclaircissement qu'il jugerait nécessaire.

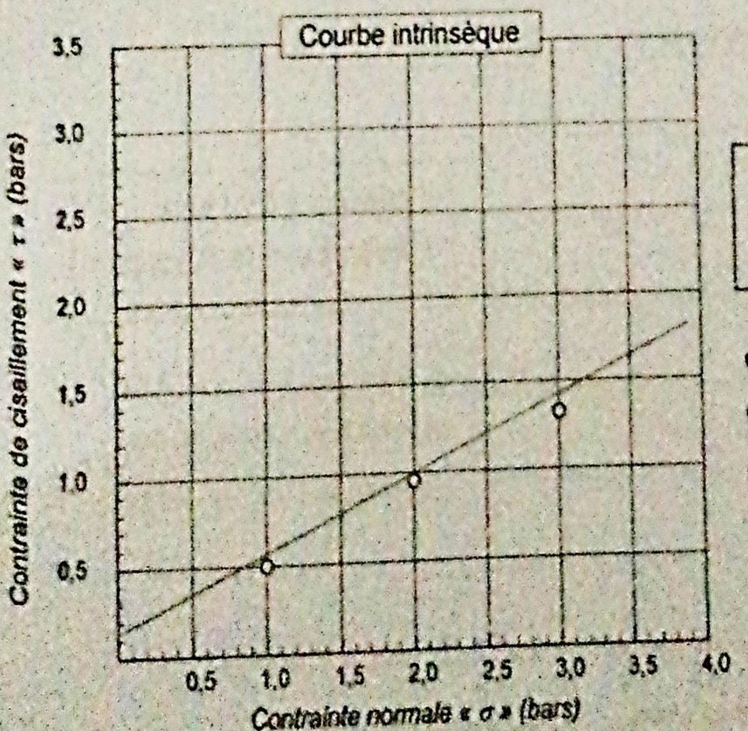
Le Directeur

N . Osmane


Sondage : S1
Dossier N° : 33 - 43 - 2015
DESCRIPTION
Encroutement calcaire et des croûtes compactes
Encroutement calcaire sableux et graveleux
Fin du forage à une profondeur de 10 m.
REMARQUES :
TYPE D'ECHANTILLON :
CF : Carottier fendu
TM : Tube à paroi mince
PS : Tube à position fixe
CR : Tube carottier
TO : Tube ouvert
ABREVIATIONS


Contrainte / Déformation
1.0 bars
2.0 bars
3.0 bars
Déformation (mm)
Courbe intrinsèque
Contrainte de cisaillement « τ » (bars)
Contrainte normale « σ » (bars)
Cohésion = 0,17 bars
Frottement = 22,85 degrés
σ1 = 1,0 bars --> τ1 = 0,55
σ2 = 2,0 bars --> τ2 = 0,95
σ3 = 3,0 bars --> τ3 = 1,36
Remarque :
Préparé par :
Approuvé par :
Date :





Cohésion = 0,14 bars

Frottement = 23,26 degrès

σ_1 = 1,0 bars --> τ_1 = 0,50

σ_2 = 2,0 bars --> τ_2 = 0,95

σ_3 = 3,0 bars --> τ_3 = 1,32

Remarque :