REPUBLIQUE DU CAMEROUN
Paix – Travail – Patrie
****

REPUBLIC OF CAMEROON
Peace – Work – Fatherland
****

DEPARTEMENT DE GENIE CIVIL
DEPARTMENT OF CIVIL ENGINEERING

DEPARTMENT OF CIVIL, ARCHITECTURAL
AND ENVIRONMENTAL ENGINEERING

THE INFLUENCE OF THE TYPE OF FLOOR (IN CONCRETE AND WOOD) ON THE STRUCTURAL BEHAVIOR OF TALL BUILDINGS: CONCEIVED CASE STUDY

A Thesis submitted in partial fulfilment of the requirements for the degree of Masters of Engineering (MEng) in civil Engineering

Curriculum**: Structural Engineering**

Presented by:

DJOMOU TENOU KEVIN LOIC

Student number: 15TP21007

Supervised by:

Pr. Carmelo MAJORANA

Co-supervised by:

Dr. Eng. POH'SIE Guillaume Hervé

Eng. CARDILLO Giuseppe

Année Académique 2020/2021

DEDICATION

I dedicate my work

To

My family.

In gratitude for all the love with which you cover me and the support that you bring to me for the success of my studies and my accomplishment as a man.

ACKNOWLEDGEMENTS

I would like to thank above all the ALMIGHTY GOD for his protection and his breath without which I wouldn't have achieved this work and for the blessings he keeps flooding me. My special thanks to:

- The President of the jury Prof. NKENG Georges, Prof. Carmelo MAJORANA Elambo for the honor you give me in being the president of the jury in my thesis presentation;
- The Examiner Prof. TALLA André for the honor you give me in being the examiner of this work and adding to it by criticizing to ameliorate;
- My supervisors Prof. Carmelo MAJORANA, Dr. Eng. POH'SIE Guillaume Hervé and Eng. Giuseppe CARDILLO for their guidance and availability;
- Director of the National Advanced School of Public Works (NASPW) of Yaoundé Prof. NKENG George ELAMBO, and Prof. Carmelo MAJORANA of University of Padua in Italy for all their academic and administrative support during these five years spent at NASPW;
- Prof. MBESSA Michel, Head of Department of civil engineering for his availability, his advices and tutoring.
- All the teaching staff of NASPW and University of Padua for their good quality teaching and motivation they administrated us.
- All my classmates and my friends of the 6th batch of NASPW who were a source of motivation and joy especially to "La team";
- My parents Mrs. NGAMGOUM Marie, Mrs FANKAM Elvisse Lune, Mrs TCHAWO YAMDEU Clarisse and Mr. TENE Celestin for the trust, love, unconditional support, valuable advices and efforts invested in me for the obtaining of this degree.
- My family for the moral support during all these years
- To all people who, from near or far have given me their heartfelt help in carrying out this work.

ABSTRACT

The main objective of this work was to study the influence of the types of floors on the structural behaviour of buildings. To achieve this goal, an evaluation of the behaviour of a building with different type of floor namely (i) the hollow block floor and (ii) the wood-concrete floor was carried out initially considering the fixed base and then raft foundation . A conceived building of six (6) levels taken as a case study was analysed and dimensioned to resist static and dynamic loads according to Eurocodes 2 and 5. The underneath soil was modelled using Winkler springs approach to depict soil flexibility. The static and dynamic analysis of our building was carried out using the structural analysis software SAP 2000 (Structural Analysis Program) version 22. The results obtained were compared in terms of period of vibration, base reaction, lateral deformation, and the inter-storey deviation. It has been noticed that the wood-concrete floor gives the vibration period larger than hollow block floor and this for the fixed base and raft foundation. Concerning the lateral deformation, and the inter-storey deviation, the hollow block floor is the best favourable because it gives less deformations (119 mm to $5^{éme}$ story) compared to the wood-concrete floor (150 mm to $5^{éme}$ story) with the two types of foundations considered. Regarding to base shear, the wood concrete floor give large values compared to hollow block floor with fixed base and raft foundation. These results show that the hollow block floor is the most suitable in the building compared to wood concrete floor.

Keywords: Influence of floors, structural behaviour, hollow block floor, wood-concrete floor.

RESUME

L'objectif principal de ce travail était d'étudier l'influence des types de planchers sur le comportement structurel des bâtiments. Pour atteindre cet objectif, une évaluation du comportement d'un bâtiment avec des différents types de planchers à savoir (i) le plancher à corps creux et (ii) le plancher bois-béton a été mené en considérant initialement une fondation avec les encastres à la base (semelles isolées) et ensuite un radier général. Un immeuble conçut de six (6) niveaux pris comme cas d'étude a été analysé et dimensionné pour résister aux charges statiques et dynamiques suivant les Eurocodes. Les différents planchers ont été dimensionnés suivant les prescriptions des Eurocodes 2et 5. Le sol support a été modélisé en utilisant des ressorts. L'analyse statique et dynamique de notre bâtiment a été réalisée à l'aide du logiciel de calcul de structures SAP 2000 (Structural Analysis Program) version 22. Les résultats obtenus ont été comparé en termes de la période de vibration, de la déformation latérale, de l'effort tranchant à la base et la déviation inter-étage. Il a été remarqué que le plancher bois béton donne des périodes de vibration plus grandes comparées à la dalle à corps creux et ceci avec les supports fixes à la base du bâtiment dans un premier temps et le radié général dans un second temps. Concernant la déformation latérale, l'effort tranchant à la base et la déviation inter-étage, le plancher à corps creux est le mieux favorable pour les deux types de fondations considérées car il donne des déformations moindres (119 mm au 5^{éme} étage) comparées au plancher bois béton (150 mm au 5^{éme} étage). Ces résultats démontrent que le plancher à corps creux est le plus approprié dans le bâtiment avec les deux types de fondations considérées.

Mots clés : Influence des planchers, comportement structurel, plancher à corps creux, plancher bois-béton

SUMMARY

GLOSSARY

NOTATIONS AND SYMBOLS

BLC	Bois Lamellé-collé
C	Coniferous
CLT	Cross Laminated Timber
D	Deciduous
DCH	Ductility Class High
EC	Eurocode
H	Heigth
ISO	International organization for Standardization
RC	Reinforced Concrete
SAP	Structural Analysis Program
SLS	Service Limit State
TMD	Tuned mass damper
ULS	Ultimate Limit State
3D	3 Dimension

A_1	Cross-section area of concrete slab
A_2	Cross-section area of timber beam
Ac	Area of the concrete cross section
As	Area of the steel reinforcement section
Asmin	Minimum section area
E_1	Elastic modulus of concrete
E_2	Elastic modulus of timber beam
$(EI)_{eff}$	Effective stiffness
F_d	Design value of the effect
F_{SLS}	Design value of the effect at SLS
F_{ULS}	Design value of the effect at ULS
$G_{0,c,k}$	Self-weight of concrete slab
$G_{0,t,k}$	Self-weight of timber beam
G_1	Characteristic value of dead load
H	Structure height or the effective structure height
I_1	Inertia moment of concrete slab
I_2	Inertia moment of timber beam
L	Length of the beam
$M_{ed,max}$	Bending moment value
M_{Rd}	Resisting moment
P_{Rd}	Design shear strength value of the connection
P_{Rk}	characteristic shear strength value of the connection
Q_k	Characteristic variable load value
T	Period of the structure
$V_{ed,max}$	Shear value
a_1	Distance between the centroid of concrete slab and the center of gravity
a_2	Distance between the centroid of timber joist and the center of gravity
b	Width of the element
b_1	Width cross-section of concrete slab
b_2	Width of timber beam
C_{min}	Minimum concrete cover
$C_{min,b}$	Minimum cover due to bond requirement
$C_{min,dur}$	Minimum cover due to environmental conditions

d	Effective height of the section
d_r	Design inter-storey drift
f	deflection
$f_{c,t,d}$	Design tensile strength of concrete
$f_{c,t,m}$	Mean value of tensile strength of concrete
f_{cd}	Design compression strength of concrete
f_{ck}	Characteristic compression strength
$f_{m,d}$	Design bending strength of timber
$f_{m,k}$	Characteristic bending strength of timber
$f_{t,0,d}$	Design tensile strength of timber
$f_{t,0,k}$	Characteristic tensile strength of timber
$f_{v,0,k}$	Design shear strength
f_{vd}	Characteristic shear strength
f_{yd}	Design yield strength of steel
f_{yk}	Characteristic yield strength
h_1	Thickness of concrete slab
h_2	Thickness of timber beam
i	Gyration radius of the uncracked concrete section
k_{def}	Modification factor of timber
k_{mod}	Modification factor of timber
k_{ser}	Stiffness at SLS
k_u	Stiffness at ULS
k_u	Stiffness at ULS
s	Spacing of the connection
γ_c	Density of concrete
γ_c	Partial factor of concrete
γ_M	Partial factor of timber
γ_t	Partial factor of timber
γ_t	Density of timber
$\Delta C_{dur,add}$	Add reduction of minimum cover for use of additional protection
$\Delta C_{dur,st}$	Reduction of minimum cover for use of stainless steel
$\Delta C_{dur,\gamma}$	Additive safety element
λ	Slenderness

λ_{lim}	Limit value of slenderness
σ	stress
$\sigma_{1,M}$	Normal bending stress without connection in concrete
$\sigma_{1,N}$	Normal bending stress with connection in concrete
$\sigma_{2,M}$	Normal bending stress without connection in timber
$\sigma_{2,N}$	Normal bending stress with connection in timber
ψ_2	Factor for quasi-permanent value of a variable action

LIST OF FIGURES

LIST OF TABLES

GENERAL INTRODUCTION

The expansion in the field of science and technology that the world is witnessing in the twenty-first century is leading its inhabitants to become more interested in civil engineering. The latter being the set of techniques applied to the design, production and exploitation by civil engineers. When designing a building, the engineers must choice the type of floor will use for construction. The wood-concrete composite is one of these new techniques developed firstly to improve pre-existing wooden structures in term of performance and lifespan of structures and thus meet expectations in terms of infrastructure design and construction with a sustainable development objective. The advantage of wood-concrete floor process is that it increases the life span of existing wooden floors and increases their load-bearing capacity, as well as having lighter floors compared to concrete floors or steel structures offering the same or even better performance. Other advantages for this technique include easy to installation and it can be economic. It is a technique already used in large countries but little known in underdeveloped countries. In our country, this technique could be applied and popularized using the type wood-concrete floor adapted to the different materials and tools found in Cameroon. It could also be demonstrated that it can be less expensive with a less complex implementation than the processes that already exist in the construction of floor.

The choice of the type of floor for the analysis and the dimensioning of the building must be judicious so that the building has a good structural behaviour. Thus, it is very important to carry out a good study on the structure to analyse the behaviour of its elements with each type of floor.

The aim of this work is to evaluate and quantify the period of vibration, base reaction, lateral deformation, and the inter-storey deviation of building with hollow block floor and building with wood-concrete floor. To attain this objective, the work is divided in three chapters. The first chapter is about the state of the art and will permit to master the basic concepts related to the floor. The second chapter entitled methodology will present the steps adopted to achieve the objective of this work. Finally, at chapter three the results of the comparison of the structural behaviour of building for the different types of foundation will be presented.

CHAPTER 1: LITERATURE REVIEW

Introduction

A floor, in the building sector is a work in concrete, wood-concrete, steel-concrete forming a horizontal platform and a separation between the floors of a construction. It is a predominant element in the construction of a building as much in his promising roles, however, the designer has before him a wide range of construction processes and he must evaluate, estimate and verify the feasibility of his choices which directly influence his work schedule and also the cost of carrying out his project. Tall buildings have a unique appeal, even an air of romance and mystery associated with their design. The adoration that super and ultratall buildings command lies in their apparent freedom from gravity loads they do not just stand tall, they seem to do so effortlessly resisting gravity as well as laterally directed force generated by wind gusts and seismic ground motions. Our study being essentially dedicated to concrete floor and wood-concrete floor, this chapter will consist of the presentation from functions of the floors, the types of floor and tall building.

1.1. Functions of the floors

Floor is a two-dimensional planar structural component of building having a very small thickness compared to its other two dimensions. It provides a covering shelter or working flat surface in building, bridges and other types of structures. Floors have many functions.

1.1.1. Strength and stability

The strength of floor structure should be adequate to carry dead load of the floor, finishes, fixtures, partitions, services and expected imposed loads of occupants. With regard to stability of floors, the stiffness of floor should be enough to make the floor stay stable and level under its self-weight and expected dead and live loads. Furthermore, the floors need to support and accommodate ventilation, water, electrical, heating services without causing detrimental effect on the stability of the floor; do not bend (limit the deflection during the formwork then during use), be durable.

1.1.2. Thermal and acoustic insulation

The floors must be thermally insulating (for example above a garage), acoustically insulating (impact noise). It should be possible for a building to maintain constant temperature or heat the inside the building irrespective of the temperature changes outside (Terzi and Elçi, 2006). According to modern building concepts, a floor should neither create noise when used nor transmit noise. Sometimes, it is required that any movement on the top floor should not disturb the persons working on the other floors. Suitable flooring is provided which is somewhat noiseless when travelled over.

1.2 Types of floor

There are many types of floor and can be classified like reinforced concrete floor, prestressed concrete floor and composite floor.

1.2.1 Reinforced Concrete floor

A reinforced concrete floor is a planar structural element and is used to provide a flat surface in buildings. There are several types of concrete floor.

1.2.1.1 Flat floor

The flat floor is a reinforced concrete floor supported directly by concrete columns or caps. Flat floor doesn't have beam so it is also called a beam-less floor. Flat floor is defined as one sided or two-sided support system with sheer load of the floor being concentrated on the supporting columns and a square floor called drop panels. Drop panels play a significant role here as they augment the overall capacity and sturdiness of the flooring system beneath the vertical loads there by boosting cost effectiveness of the construction. Usually the height of drop panels is about two times the height of floor. Load are directly transferred to columns. In this type of construction, a plain ceiling is obtained thus giving an attractive appearance from an architectural point of view. The plain ceiling diffuses the light better and is considered less vulnerable in the case of fire than the traditional beam floor construction. The flat floor is easier to construct and requires less formwork. Flat floor are most suitable for spans of 6 to 9 m, and for live loads of 4-7 kN/m^2. It can be constructed as a post-tensioned flat floor. The thickness of the flat floor is 20 cm (www.civilread.cm). The figure 1.1 and 1.2 show the flat floor.

Figure1.1. Flat floor construction (www.civilread.cm, 2021)

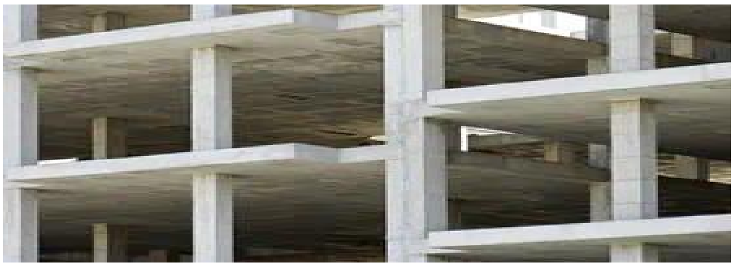


Figure1.2. Flat floor construction (www.civilread.cm, 2021)

Flat floor are used at to provide plain ceiling surface giving better diffusion of light; easy constructability with the economy in the formwork; larger headroom or shorter storey height and pleasing appearance (Ferrareto et al.2015). This kind of floors are provided in the parking. Flat floors are generally used in parking decks, commercial buildings, hotels or places where beam projections are not desired for height restrictions. Flat floor minimizes floor-to-floor heights when there is no requirement for a deep false ceiling. Building height can be reduced; he have less construction time; it increases the shear strength of the floor, reduce the moment in the floor by reducing the clear or effective span. In a flat floor system, it is not possible to have a large span, not suitable for supporting brittle partitions. There are different types of concrete flat floors like simple flat floor, flat floor with drop panels and flat floor with both drop panels and column heads. The column heads increase shear strength of floor; it reduce the moment in the floor by reducing the clear or effective span. The drops panels increase shear strength of floor; it increase negative moment capacity of floor; it stiffen the floor and hence reduce deflection.

1.2.1.2. Conventional slab

The floor which is supported on beams and columns is called a conventional floor. In this kind, the thickness of the floor is small whereas the depth of the beam is large and load is transferred to beams and then to columns. It requires more formwork when compared with the flat floor. In the conventional type of floor there is no need for providing column caps. The thickness of conventional floor is 10 cm. 13 to 16 cm is recommended if the concrete will receive occasional heavy loads, such as motor homes or garbage trucks (www.civilread.cm). Reinforcement is provided in conventional floor and the bars which are set in horizontal are called main reinforcement bars and bars which are set in vertical are called Distribution bars. Based on the length and breadth of conventional floor is classified into two types.

a) One way floor

One way floor is supported by beams on the two opposite sides to carry the load along one direction. The ratio of longer span (l) to shorter span (b) is equal or greater than 2 considered as one-way floor. In this type, the floor will bend in one direction (in the direction along its shorter span). However minimum reinforcement known as distribution steel is provided along the longer span above the main reinforcement to distribute the load uniformly and to resist temperature and shrinkage stresses. The equation 1.1 illustrates the condition on ratio of longer span (l) to shorter span (b).

$$\frac{\text{Longer span}}{\text{short span}} \geq 2 \qquad \text{Eq.(1.1)}$$

In general, the length of the floor is 4m. But in one way floor, one side length is 4m and another side length is more than 4m. So it satisfies the above equation. A live load is to 3 to 5kN/m^2. He can also be used for larger spans which relatively higher cost and higher floor deflection. Additional formwork for the beams is however needed. Main reinforcement is provided in shorter span and distribution reinforcement is provided in the longer span. Main bars are cranked to resist the formation of stresses. Generally all the cantilever slabs are one way floor.

b) Two way floor

In two way floor, the ratio of longer span (l) to shorter span (b) is less than 2. The floors are likely to bend along both the directions to the four supporting edges and hence distribution reinforcement is provided in both the directions. The equation 1.2 illustrates the condition on ratio of longer span (l) to shorter span (b).

$$\frac{\text{Longer span}}{\text{short span}} < 2 \qquad \text{Eq.(1.2)}$$

The construct of this type of floor is similar to that of one way floor, but it may need more formworks since two way floor are supported on all sides. In this kind of slab, the length and breadth of the floor are more than 4m. Live loads is 3-6kN/m^2. To resist the formation of stresses distribution bars are provided at both the ends in two way floor. The beams increase the stiffness of the slabs, producing relatively low deflection. Additional formwork for the beams is needed. These types of floors are used in constructing of multi-storeyed building. The figure 1.3 shows the construction of two way floor. The figure 1.4 show the difference between one and two way floor.

Figure1.3. Two way floor construction (www.civilread.cm, 2021)

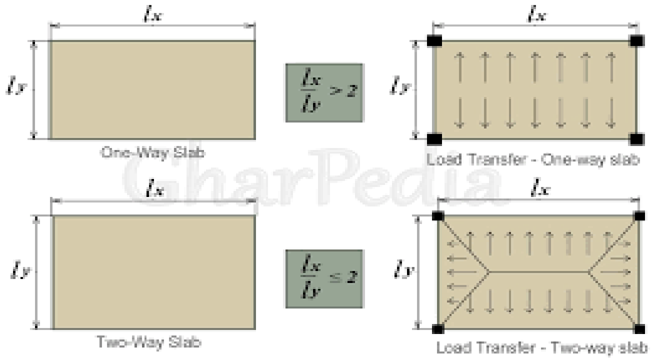


Figure1.4. Difference between one and two way floor (www.civilread.cm, 2021)

1.2.1.3. Waffle floor (Grid floor)

Concrete floor made of reinforced concrete with concrete ribs running in two directions on its underside. It is a type of reinforcement concrete floor that contains square grids with deep sides. The name Waffle comes from the grid pattern created by the reinforcing ribs. Waffle floor construction process includes fixing forms, placement of pods on shuttering, installation of reinforcement between pods, installation of steel mesh on top of pods, and pouring of concrete. After concrete sets, the formwork is removed and pods are not removed. It is usually used where large spans are required (auditorium, cinema halls) to avoid many columns interfering with space. The main purpose of employing this technology is for its strong foundation characteristics of crack and sagging resistance. This kind of floor is majorly used at the entrance of hotels, malls, restaurants for good pictorial view and to install artificial lighting. Grid floor are suitable for spans of 9-15 m and live loads of 4-7 kN/m^2 (www.civilread.cm). Formwork, including the use of pans, is quite expensive. Waffle floor also holds a greater amount of load compared with conventional concrete floor. Based on the shape of pods waffle floor are classified into triangular pod system and square system.

Waffle floors are able to carry heavier loads and span longer distances than flat floor as these systems are light in weight; these systems are light in weight and hence considerable saving is ensured in the framework as the light framework is required. Waffle floor is not used in typical construction projects; the casting forms or moulds required for precast units are very costly and hence only economical when large scale production of similar units are desired (Ferrareto et al., 2015); construction requires strict supervision and skilled labour. The figure 1.5 shows the construction of waffle floor.

Figure1.5. Waffle floor construction (www.civilread.cm, 2021)

1.2.1.4. Hardy floor

It is a type of concrete floor constructed using hardy bricks which significantly decline the amount of concrete and eventually the slab's self-weight. Hardy bricks are the hollow bricks and made up of concrete. The thickness of hardy floor is commonly greater than conventional floor and around 270 mm. The dimensions of hardy brick is 40 cm × 20cm × 20 cm. The construction of hardy floor involves formwork installation, hardy block placement, placement of reinforcement into gaps between blocks, placement of steel mesh on the blocks, and finally pouring of concrete. It is economical for spans of length up to 5 m, and it reduces the quantity of concrete below neutral axis, and moderate live loads shall be imposed; easy of construction, especially when all beams are hidden beams; improved insulation for sound and heat. Hardy floors are further classified into two types (one way hardy floor and two way hardy floor). It is constructed at locations where the temperatures are very high. To resist the temperature from the top of the floor thickness is increased. The application of this type of floor can be seen in Dubai and China. The figure 1.6 shows the construction of hardy floor.

Figure1.6. Hardy floor construction (www.civilread.cm, 2021)

1.2.1.5. Hollow core floor

It is a type of precast floor through which cores are run. Not only do these cores decline floor self-weight and increase structural efficiency but also act as service ducts. He is manufactured using high tensile strength prestressed strands or single wire which are embedded within the element. Floors in prestressed concrete are usually produced in lengths of up to 200 meters. The process involves extruding wet concrete along with the prestressed steel wire rope from a moving mold. The floor has been especially popular in countries where the emphasis of home construction has been on precast concrete, including Northern Europe and former socialist countries of Eastern Europe. The production of these element is achieved using our extruder and slipformer machines that cast in one phase along a production bed without the need of any formworks. Hollow core floors are widely used with different supporting structures (concrete constructed walls, brick built walls, steel structure, on-site concrete cast structures, and prefabricated beams. It is suitable for cases where fast constructions are desired. Moreover hollow core floors process certifiable fire resistance and offer an economic solution to construct fire stop wall for warehouses, industrial buildings.

There is no restriction on the span of the hollow core floor units, and their standard width is 120mm and depth ranges from 110mm to 400mm. The floor units are commonly installed between beams using cranes and the gaps between units are filled with screeds. It has been observed that, hollow core floor can support 2.5 kN/m^2 over a 16m span (www.civilread.cm). It is suitable for offices, retail or car park developments. It is typically used in the construction of floors in multi-storey apartment buildings. Another fabrication system produces hollow-core floor in reinforced concrete (not prestressed). These are made on carousel production lines, directly to exact length, and as a stock product. However, the length is limited to about 7-8 meters. Especially in Belgium, this method is widely used in private housing. The figures 1.7 shows the construction of hollow core floor.

Figure1.7. Hollow core floor construction (www.civilread.cm, 2021)

Hollow core floor not only reduces building costs it also reduces the overall weight of the structure; excellent fire resistance and sound insulation are other attributes of hollow floor due to its thickness; it eliminates the need to drill in floor for electrical and plumbing units; it is easy to install and requires less labour; he is fast in construction; no additional formwork or any special construction machinery is required for reinforcing the hollow block masonry. The voids of the hollow core can be used as conduit for installations. The interior of the core can be coated in order to use it as a ventilation duct. Precast concrete popularity is linked with low-seismic zones and more economical constructions because of fast building assembly. If not properly handled, the hollow core floor units may be damaged during transport; it becomes difficult to produce satisfactory connections between the precast members;

1.2.2. Prestressed concrete floor

Prestressed concrete is a method for overcoming the concrete's natural weakness in tension. It can be used to produce beams, floors or bridges with a longer span than is practical with ordinary reinforced concrete. Within the field of building structures, most prestressed concrete applications are in the form of simply supported precast floor. Prestressing floor can be accomplished in pre-tensioned floor and post-tensioned floor.

1.2.2.1. Pre-tensioned floor

The floor in which steel is tensioned before placing the concrete is called pre-tensioned floor. Pre-tensioning is accomplished by stressing wires or strands, called tendons, to predetermined amount by stretching them between two anchorages prior to placing concrete. The concrete is then placed and tendons become bonded to concrete throughout their length. After concrete has hardened, the tendons are released by cutting them at the anchorages. The tendons tend to regain their original length by shortening and in this process transfer through bond a compressive stress to the concrete. The tendons are usually stressed by the use of hydraulic jacks. The stress in tendons is maintained during the placing and curing of concrete by anchoring the ends of the tendons to abutments that may be as much as 200 m apart. The abutments and other formwork used in this procedure are called prestressing bench or bed. Most of the pre-tensioning construction techniques are patented although the basic principle used in all of them is common and is well known. In pre-tensioning the reinforcement, I the form of tendons or cables, is stretched (put into tension) across the concrete formwork before the concrete placed. After the concrete has hardened and a suitable strength developed, the tendons are released. Pre-tensioning is generally done in precasting plants in permanent beds, which are used to produce pre-tensioned precast concrete elements for the building industry. The floor has same features of post tensioning. The figure 1.8 shows the construction of pre-tensioned floor.

Figure1.8. Pre-tensioned floor construction (www.civilread.cm, 2021)

1.2.2.2. Post-tensioned floor

The floor which is tensioned after constructing a floor is called post tension floor. Reinforcement is provided to resist the compression. In a post-tensioned beam, the tendons are stressed and each end is anchored to the concrete section after the concrete has been cast and has attained sufficient strength to safety withstand the prestressing force. In post-tensioning method, tendons are coated with grease or a bituminous material to prevent them from becoming bonded to concrete. Another method used in preventing the tendons from bonding to the concrete during placing and curing of concrete is to encase the tendon in a flexible metal hose before placing it in the forms. The metal hose is referred to as sheath or duct and remains in the structure. After the tendon has been stressed, the void between the tendon and the sheath is filled with grout. Thus the tendons become bonded to concrete and corrosion of steel is prevented. A tendon is generally made of wires, strands, or bars. Wires and strands can be tensioned in groups, whereas bars are tensioned one at a time. Post tensioning is performed by two main operations (tensioning the steel wires or strands by hydraulic jacks that stretch the strands while bearing against the ends of the member and then replacing the jacks by permanent anchorages that bear on the member and maintain the steel strands in tension). Post-tension prestressing can be done at site. This procedure may become necessary or desirable in certain cases. Post-tensioning provides a means to overcome the natural weakness of concrete in tension and to make better use of its strength in compression.

The principle is easily observed when holding together several books by pressing them laterally. In a post-tensioned beam, the tendons are stressed and each end is anchored to the concrete section after the concrete has been cast and has attained sufficient strength to safely withstand the prestressing force. In concrete structures, this is achieved by placing high-tensile steel tendons/cables in the element before casting. When concrete reaches the desired strength the tendons are pulled by special hydraulic jacks and held in tension using specially designed anchorages fixed at each end of the tendon. This provides compression at the edge of the structural member that increases the strength of the concrete for resisting tension stresses. If tendons are appropriately curved to a certain profile, they will exert in addition to compression at the perimeter, a beneficial upward set of forces (load balancing forces) that will counteract applied loads, relieving the structure from a portion of gravity effects. This is one of the types of concrete floors. In this type of floor, cables are tied instead of reinforcement. In steel reinforcement, the spacing between bars is 10.5 cm to 15.5 cm whereas in post tension slab the spacing is more than 2 m (www.civilread.cm). The post-tension floors can be in many types (Flat slabs with/without drop panels; ribbed and waffles floors; one way/two way floors.)

Post tension floor allows floors and other structural members to be thinner; it allows us to build floor on expansive or soft soils; cracks that do form are held tightly together; post tension floors are excellent ways to construct stronger structures at an affordable price; it reduces or eliminates shrinkage cracking-therefore no joints, or fewer joints, are needed; it lets us design longer spans in elevated members, like floors or beams. The post tension floor can be made only by skilful professionals; the main problem with using post tension floor is that if care is not taken while making it, it can lead to future mishaps. For heavy loads and large spans in buildings or bridges, it may be very difficult to transport a member from precasting plant to a job site. In post-tensioning it is necessary to use some types of device to attach or anchor the ends of the tendons to the concrete section. Many times, ignorant workers do not fill the gaps of the tendons and wiring. These gaps cause corrosion of the wires which may break untimely, leading to some failures unexpectedly. The figure 1.9 shows the construction of pre-tensioned floor.

Figure1.9. Post-tensioning floor construction (www.civilread.cm, 2021)

1.2.3. Composite floor

Composite floor is a floor involving multiple dissimilar materials bounded together so strongly that they act together as a single unit from in order to make the most of this association. There are many types of composite floor.

1.2.3.1. Bubble Deck floor

Bubble deck slab is a kind of slab in which voids are created in it (Immanuel Joseph Chacko, 2016). Bubble Deck is the invention of Jorgen Bruenig in the 19190s, who developed the first biaxial hollow floor (now known as Bubble Deck) in Denmark. It is constructed by placing plastic bubbles which are prefabricated and the reinforcement is then placed between and over plastic bubbles and finally, fresh concrete is poured. The plastic bubbles replace the ineffective concrete at the center of the floor. Bubble Deck floor has a superiority over the conventional floor. Bubble Deck floor reduce weight, increase strength, larger spans can be provided, fewer columns needed, no beams or ribs under the ceiling are required. Consequently, not only does it decline the total cost of construction but it is also environmentally friendly since it reduces the amount of concrete (35%). These types of floors are used in constructing of residential living, offices, industrial buildings, offices, hotels, schools, parking, hospitals and laboratories. There are many materials used in Bubble Deck floor: concrete, reinforcement bars and hollow bubbles. The figure 1.10 shows the construction of Bubble Deck floor.

Figure1.10. Bubble deck floor construction (www.civilread.cm, 2021)

1.2.3.2. Wood-concrete floor

The materials that constitute this type of floor are wood and concrete.

a) Wood

Also called timber, Wood is a natural, renewable, heterogeneous, anisotropic material with a high hygroscopicity. The part taken from the tree to make the wood is the trunk. The trunk has two functions, one of which is to carry sap from the roots to the leaves for the raw sap (water) and from the leaves to the roots for the nourishing sap; the second is to support mechanical loads (self-weight of the tree, its leaves, fruit, snow and wind loads). From the outside to the inside we can distinguish the bark, the bast which is used to transport the nourishing sap, the cambium, the sapwood which transports the raw sap, the heartwood or duramen which is used as construction wood. The figure 1.11 shows the structure of the wood.

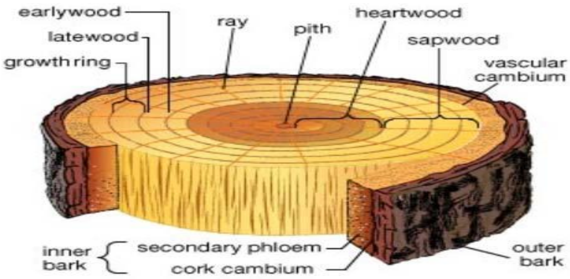


Figure1.11. Structure of trunk (https://www.pinterest.com/pin/177610779029001937,Merriam-Webster (2006))

The trunk is composed of several types of cells, of which the fibers that make up the supporting tissue and play a mechanical role. They develop according to the direction of growth of the tree (axial or longitudinal direction); the vessels make up the sap conduction tissues and finally the cells that serve as reserve accumulation. This organization results from the specific properties (appearance, density, mechanical characteristics...).

Wood species are classified into two main groups of which the coniferous (C) ones with noncaduccous leaf (ex: fir, spruce, larch, pine, etc.) and the deciduous (D) with deciduous leaf which have a biological diversity much higher than that of the coniferous ones. They are more evolved and have a more complex structure than the coniferous ones. (Example: chest nut, oak, maple, linden, frene...).

In the construction industry, wood represents various percentages as building materials, for example, in France 10% compareddtoo15% in Germany. By construction wood is meant the wood used to make frames, trusses, roof supports; the framework wood of industrial, leisure and school buildings; the structural wood of works of art.

Wood is not used for structures in direct contact with the ground (foundations). Until recently, it was also not used for high-rise buildings (at least R+3). Under the effect search for competitive environmental balances and the development of the sector, higher buildings are beginning to emerge. For example, glued timber makes it possible to erect wooden buildings up to R+8, such as the Stadthaus in London, which has 29 apartments.

Many wood-based products have been developed to reduce the intrinsic variability of solid wood. These products are more reliable (standard deviations of lower mechanical characteristics) because they are more homogeneous and often less anisotropic. All these products use adhesives to join together more or less large wood elements. A first family of products assembles plank-like elements (BLC glulam and CLT cross laminated timber). The second family assembles smaller-sized pieces of wood (sheets of wood for Lamibois or fines lamellas for OSB). The figures 1.12 and 1.13 show two traditional wood floors.

Figure1.12. Timber slab construction (http://www.architectmagazine.com/techology/somtests a mass timber composite system for high rise construction and explores a steel option, 2012)

Figure1.13. Timber slab construction (http://www.architectmagazine.com/techology/somtests a mass timber composite system for high rise construction and explores a steel option, 2012)

b) Concrete

Concrete is an artificial stone which is obtained after hardening the homogenous mixtures of cement, water, aggregates and sometime admixture and/or additive (to modify the fresh and hardened concrete property). The concrete composition must be established in order to assure the resistance and durability of building elements, by using less cement dosage. The concrete composition is established by concrete class which is indicated by the designer; cement type, additive type according to the requirements from the project; concrete consistency according to the element type which is to be realized, Impermeability, freeze proofness, maximum water/ cement ratio (W/C) and the minimum cement dosage according to the minimum requirements for durability assurance; Aggregate grading zones, aggregate grading and maximum granule shape according to the shape and dimension elements and the distance between reinforcements. The composition is expressed in dosages which represent the materials' quantities necessary for 1m^3 of concrete: cement, water, aggregates, admixture and/or additive.

1.3. Tall buildings

A building is considered a tall building depending on the effect of a lateral force on the building.

1.3.1. Structural systems of tall buildings

The main structural systems of tall buildings consist of gravity load resisting systems and lateral load resisting systems. In real design practice, the main lateral forces that need to be considered are wind and earthquakes. As it has a longer period than low rise buildings, the earthquake load is less onerous. Therefore, in most cases, wind is the governing load in the design.

The major components associated with the vertical load resisting systems of high rise buildings are the floor system, columns, transfers beams and shear walls. Shear walls are primarily used for resisting lateral forces but are also used to resist vertical loads. The floor system is part of the gravity load resisting system that remains relatively similar in both high rise and low rise buildings. However, for tall buildings, it is imperative to minimise the weight of the entire structure. Therefore, lightweight floor systems, such as post-tension slabs, are the dominant selection in the design. The floor system also plays an important role in resisting lateral loads. Due to its large in-plane stiffness, the lateral loads are transferred through the floor plate via a diaphragm action by allowing a constant pathway to the lateral resisting system, such as core walls and bracings; therefore, the design of adequate floor diaphragms is also an important issue in tall building design. For floor plates with large openings, special attention needs to be paid to these as they weaken the diaphragm action in the floor slab. In some projects, planar trusses need to be employed to enhance diaphragm actions.

1.3.2. Behaviour of tall buildings subjected to wind

The traditional approach to wind loading that has been used for the design of tall buildings since the 1960s is one in which the wind pressure is assumed to act statically. This is convenient in that it has enabled the appropriate coefficients of pressure to be estimated from wind tunnel tests carried out in a uniform steady velocity in a wind tunnel. The velocity used in design has been variously determined from maximum gust speeds or average velocities measured over a minute of a mile of wind, depending on the nature of the routine meteorological measurements. There is usually an adjustment for the variation of velocity with height, which is sometimes based on the maximum gust variation with height and sometimes on the mean speed variation with height. Although convenient, this quasistatic approach to wind loading is unrealistic in several respects. Furthermore, there is a need for broadening the basis of design against wind to include more explicitly such factors as allowable deflections, comfort of occupants, and strength. The diversity of structures now being built makes the problem of formulating wind loads using simplified umbrella loadings more and more difficult if at the same time they are to be economical and satisfactory from a performance standpoint.

The use of static wind loads in the design of tall buildings, although convenient, can at times lead to false impression as to the real behaviour of a tall building or any structure for that matter in the wind. In the natural wind over a city, a tall building is constantly buffeted by gusts and other aerodynamic forces, and although it does tend to deflect toward a mean position, it is continuously swaying with an amplitude that may be at least as large as the steady deflection. In describing the observation on the Empire State Building, Rathbun (1940) states that the "building tended to vibrate continuously like the tines of a turning fork." All the traces show that the sway motion occurs primarily in what turns out to be the fundamental frequency of the building with little evidence of the higher harmonics.

The sway motion is in fact probably the most significant aspect of the behavior determining Whether a structure performs satisfactorily or not in service. The sway acceleration directly determines the comfort of occupants and a large proportion of the total deflection, which governs the cracking of walls is due to sway. The fact that the dynamic response is influenced by factors other than stiffness alone, such as the mass and damping, and the fact that the criterion of performance cannot be based on stress and strength only but also on other factors such as deflection and acceleration considerations makes design for the dynamic effects of wind more subtle in order to develop an approach for the design of tall buildings against the wind, it is first necessary to understand in general terms the character of the wind and the important factors that influence its action on buildings. While it cannot be pretended that the understanding is yet complete, research has clarified a great deal and provided a framework in which to fit ideas.

Every structure has a natural frequency of vibration. Should dynamic loading occur at or near its natural frequency, structural damage out of all proportion to size of load may result. It is well known, for example, that bridges capable of carrying far greater loads than the weight of a company of soldiers may oscillate dangerously and may even break down under dynamic loading of soldiers marching over them in step. Similarly, certain periodic gust within the wide spectrum of gustiness in wind may find resonance with natural vibration frequency of a building, and although the total force caused by that particular gust frequency would be much less than the static design load for the building, dangerous oscillations may be set up. This applies not only to the structure as a whole but also to components such as curtain wall panels and sheets of glass. A second dynamic effect is caused by instability of flow around certain structures. Long narrow structures such as smoke stacks, light standards, and suspension bridges are particularly susceptible to this sort of loading, causing an alternating pattern of eddies to form in its wake. A side thrust is thus exerted on the object similar to the lift on an aerofoil, and since this thrust alternates in direction, a vibration may result. Side-to-side wobbling effect of a straight stick pulled through water is an example of this phenomenon.

Generally, in tall building design, the crosswind motion perpendicular to the direction of wind is often more critical than along-wind motion. At low wind speeds, since the shedding occurs at the same instant on either side of the building, there is no tendency for the building to vibrate in the transverse direction. Therefore, the building experiences only along-wind oscillations parallel to wind direction. However, at higher speeds, vortices are shed alternately, first from one and then from the other side. When this occurs, there is an impulse in the along-wind direction as before, but in addition, there is an impulse in the transverse direction. However, the transverse impulse occurs alternately on opposite sides of the building with a frequency that is precisely half that of the along-wind impulse. This impulse due to transverse shedding gives rise to vibrations in the transverse direction. The Phenomenon is called vortex shedding.

1.3.3. Building sway

The questions regarding building sway may be asked in many different ways but, in essence, boils down to this: “How much can we allow a building to sway in the wind?” It seems no one proposes to answer the question for the most part because a reasonably simple answer is exceedingly difficult to formulate. We will, however, examine several of its facets.

Let us start by considering how the engineers start their wind design. First, they must establish the nature of the wind that is the environment of the particular building in question. For a very tall building, the engineer would like the characteristics of the wind to be described in relatively great detail. Second, the engineer must determine the response of the structure to the turbulent wind environment. In spite of the advances in computer field, it is not yet possible to select a structure that accepts the wind environment and meets the acceptable deflection and acceleration criteria. We must still resort to a trial-and-error procedure in which the structure response is calculated, compared with the acceptance criteria and modified as required. The procedure is repeated over and over until the acceptance criteria are just met. At the moment, we lack sufficient background and knowledge to go about a direct design.

Great steps forward are being made both in describing the nature of the wind and the effects of that wind on a structural system. Much less is known as to what levels of structural response can be tolerated. Deflection, as such, and particularly total deflection or sway appears to have little value in an engineering sense. The three things that do matter are the integrity of the structural system, the integrity of the architectural finishes, and the comfort of the building occupants. All are related to the dynamic and to the static response of the structure to its wind environment. The disposition of the natural wind particularly over large built up areas is exceedingly complex to quantify. Tall buildings in these terrains are buffeted by gusts and other aerodynamic forces and are set into oscillation. In addition, the building tends toward a steady deflection. The sway amplitudes can equal or exceed the steady deflection. This is almost always true for very tall buildings. It follows, then, that the occupants of upper levels of tall buildings are constantly subjected to motion. The design challenge therefore is to only limit motion to the extent that it should not adversely affect the integrity of the structure nor cause discomfort to occupants due to high sway accelerations. To determine acceptable levels of acceleration, we must consider the extremes panic levels, so to speak as well as various working and perception levels. It would be convenient if we could establish a simple number as an acceptance level for acceleration and then limit building response to that level. At first glance, it would seem to be the simplest way, but in reality, it is not.

In attempting to evaluate this swaying motion of a structure, the engineer needs to know how much and how often the swaying motion occurs. Knowing the periods of vibration of the structure, the engineer can readily translate sway amplitudes into accelerations. As with amplitude, knowledge of modes shapes will allow an evaluation of acceleration to be made throughout the height of the structure. An increase in structural damping will decrease structures response and decrease accelerations. It is interesting to recognize that a change in structure stiffness will affect response but will have little effect on acceleration levels. Now, should the engineer have knowledge of acceptable levels of acceleration, he or she can adjust the stiffness of his structure and the damping contained therein until the response of the structure is within the desired limits. At this stage in the technology, it remains to the individual engineer to set his own criteria for swaying motion. Guidance, however, is available in several published papers and standards.

1.3.4. Seismic design

The seismic design is based on three main design approaches namely: displacement-based design, capacity-based design and performance-based design.

1.3.4.1. Displacement-based design method

Earthquake engineering performance assessment commonly involves consideration of nonlinear response. Displacement-based design is a process by which earthquake-induced displacement demands are first estimated, followed by an assessment of the local deformation demands and capacities. The concept was first expressed by Muto et al. (1960) and later presented as a general design and assessment approach (Moehle, 1992). Displacement-based concepts are now firmly embedded as design features of codes and assessment standards. In displacement-based design, the first step is to analyze a structure under earthquake shaking to establish the earthquake-induced lateral displacements. Structural analysis is then used to estimate the local inelastic deformation demands. Flexural hinge rotations determined by this approach can be compared with rotation capacities at various limit states. The most convenient quantity to measure, either displacement imposed on a structure by an earthquake, or the capacity of the structure to develop ductility, is displacement (Paulay and Priestley, 1992). As observed by Newmark and Velestos (1990), the maximum lateral displacements of a structure are independent of the level of resistance. This ideal behavior is illustrated in figure 1.14.

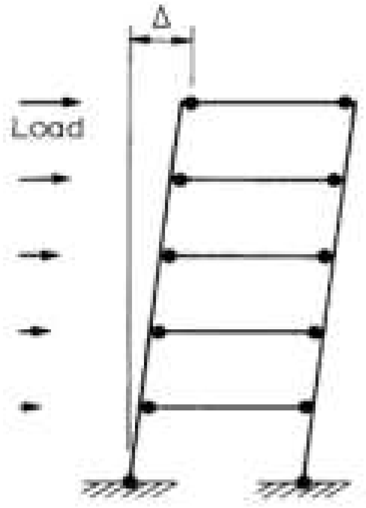


Figure1.14. Sway mechanism in laterally loaded multistorey frame (Paulay, 1983)

Therefore, a structure can be designed for a lower lateral load than the maximum induced lateral load, provided that some of the members plastify and the structural integrity never get compromised. Thus, the structure must be designed for a maximum level of deformations and not for maximum loading.

1.3.4.2. Capacity based design

Capacity design is a design method for controlling the yielding mechanism of a structure that is expected to respond inelastically to a design loading or an overload. Capacity design was introduced in structural earthquake engineering in Blume et al. (1961) (Moehle, 2015). Since that time, it has become widely adopted in earthquake engineering practice. In capacity design, structures are designed for many limit states. Just like the load design, it is designed for all the limit states except that one limit state is chosen to occur before any other. Structures for earthquake resistance, distinct elements of the primary lateral force resisting system are chosen and suitably designed and detailed for energy dissipation under severe imposed deformations.

1.3.4.3. Shear failure

The concept is based on the energy dissipation capacity of the structure during a seismic event. The design of the structure follows a design sequence which goes thus:

a) Beam flexural design

After the redistribution process of the moments, beams are proportioned such that their dependable flexural strength at selected plastic hinge locations is as close as possible to the moment requirements resulting from the redistribution process. Generally plastic hinges are chosen to be at the column faces but not always is that the case. Flexural strength of other regions of the beam are designed to ensure that no plastic hinges forms where it has not been provided for.

b) Beam shear design

The shear strength at all sections along the beam is designed to be higher than the shear corresponding to maximum feasible flexural strength at the beam plastic hinges. This is so because, according to Paulay and Priestley (1992), the inelastic shear deformation does not exhibit the prerequisite characteristic of energy dissipation.

c) Column flexural strength

To determine the maximum feasible column moments corresponding to beam flexural overstrength, considerations of the joint moment equilibrium and possible higher-mode structural response are used.

d) Transverse reinforcements for columns

In the column design, determination of the amount of transverse reinforcements is based on the stringent of the requirements for shear strength, confinement of compressed concrete, stability of compression reinforcement, and lapped bar splices.

Conclusion

All through this chapter, the aim was to present the functions of the floors, the different types of floor, the notions about tall buildings, the effect of wind load on tall building and building sway. s. The design of such structural element is done based on theories and concepts taken into account by the various international design codes presented in this chapter. It has been noticed that important disparities exist among the design codes in the requirement for the design and detailing of such structural element. The functions of the floors are strength, stability, thermal, acoustic insulation. For the types of floors we have spoken of the reinforced concrete floor, composite floor. We also talked about structural systems of tall buildings, wind loads on tall buildings, behaviour of tall buildings subjected to wind and building sway. What follows is the methodology in which are found the step by step procedure for the achievement of the set objectives of this work.

CHAPTER 2: METHODOLOGY

Introduction

The principal objective of the method is to clarify how the study will be conducted thus it is therefore an indispensable element in the analysis. The aim of this section is to present the elements of our work. Then after the presentation of the different standards used to develop this work, the different design procedures that will be followed to achieve this work will be presented.

2.1. Site recognition

The site recognition will be carried out from a documentary research whose essential goal is to know the location of the site, the climate, the hydrology and socio-economic parameters in the region.

2.2. Site visit

The purpose of this activity is the building description results from the observation and the presentation of the use category, the dimension, the floor plans and elevation configuration.

2.3. Data collection

Two types of information will be collected namely those related to the soil and those related to the structure.

2.3.1. Geotechnical data

Geotechnical data will be extracted from in-situ and laboratory tests done at the site. From these tests the soil stratigraphy will be reconstructed, and for each layer their main characteristics.

2.3.2. Structural data

Structural data are related to the structural plan of each level where we can identify the position of structural elements namely beams, columns and the concrete core and the characteristics of materials used.

2.4. Codes

The norms that will be used for the design of elements are the Eurocode 0, basis of structural design Eurocode 1, actions on structure, Eurocode 2, design of concrete structures, Eurocode 5, design of timber structure and Eurocode 8 design for earthquake resistance. These European standards define the loads and the combination of loads for the design.

2.5. Actions

The following loads will be considered in the calculation.

2.5.1. Permanent action

These are actions acting during the whole nominal life of the structure with negligible time variation of their intensity (that can be considered as constant in time).

- Self-weight of structural elements (G1); self-weight of the soil, if present; forces due to the soil (excluding the effects of the service loads applied to the soil); forces due to water pressure (when they are constant in time);
- Self-weight of non-structural elements (G2); imposed displacements and deformations determined by the designer and realized in-situ;
- Prestressing
- shrinkage and creep (fluage);
- differential displacements.

Permanent action or load consist essentially of the weight of the element, whether structural or not. Provisions for the evaluation of the self-weight of these elements are given in Eurocode1.

2.5.2. Variable action

Variable actions are those which, as the name goes, vary with respect to time. They consist of actions on the structure (or on the structural element) with instantaneous values which can be significantly different in time: That is, their magnitude is time dependent.

- with long duration: acting with a significant intensity, also if non-continuously, for a not negligible time compared to the nominal life of the structure;
- with brief duration: acting with brief duration compared to the nominal life of the structure;

This variation is nonnegligible and monotonic. Variable loads fall under two main kinds; Imposed loads and seismic- induced loads.

2.6. Combination of actions

A combination of actions, as the name indicates, consist of a set of load values applied to the structure simultaneously to verify its structural reliability for a given limit state (design limit states). They are defined as follows, when designing a building.

- At the Ultimate Limit State (ULS)

$$\Sigma_{j\geq1}\gamma_{G,j}G_{k,j} + \gamma_{Q,1}Q_{k,1} + \Sigma_{i>1}\gamma_{Q,i}\Psi_{0,i}Q_{k,i} \quad \text{(Eq. 2.1)}$$

Eq. 2.1 above is the fundamental load combination at Ultimate Limit State (ULS) design situation, where the coefficients $\gamma G,j$ and $\gamma Q,i$ are partials factors or again safety coefficients, which minimize the action which tends to reduce the solicitations and maximize the one which tends to increase it. The values of these partial factors recommended by the Eurocode 0 for the structural and Geotechnical verifications are:

$\gamma G,$ = 1.35

$\gamma G,$ = 1

$\gamma Q,1$ = 1.50 Where unfavorable (0 where favorable)

$\gamma Q,$ = 1.50 Where unfavorable (0 where favorable)

- At the Service-ability Limit State (SLS),

2.6.1. Characteristic combination

This is the load combination for non-reversible Serviceability Limit State (SLS) for the verifications with allowable stress method.

$$\Sigma_{j\geq1}G_{k,j} + Q_{k,1} + \Sigma_{i>1}\gamma_{Q,i}\Psi_{0,i}Q_{k,i} \quad \text{Eq. (2.2)}$$

2.6.2. Quasi-permanent combination

Normally, this type is used for long-term effect and appearance of the structure

$$\Sigma_{j\geq1}G_{k,j} + \Sigma_{i>1}\gamma_{Q,i}\Psi_{2,i}Q_{k,i} \quad \text{Eq. (2.3)}$$

Ψ_2, is the factor for combination value of a variable action for quasi-permanent, (see table A.1.1 of EN 1990).

2.7. Static design methodology

The static design is done based on the static analysis. Static analysis studies the behaviour of the structure under static loads application. The analysis starts with the modelling of the structural members. In that line, the concrete cover, the design and verification of one horizontal and one vertical (column) structural element, both considered as representative of the other elements of their type.

2.7.1. Concrete cover and durability

Concrete cover is most importantly a measure to protect structural reinforcements against environmental actions. The durability of a structure depends on how long the later can be protected from external factors like environmental actions and others but also the capacity of the structure to perform its function throughout its working life without severe damage. Concrete cover is defined in Eurocode 2 as the distance between the surface of the reinforcement closest to the nearest concrete surface and the nearest concrete surface (Figure 2.1). The nominal cover is defined as, according to EN 1992-1-1_E_2004, a minimum cover, C_{min}, plus an allowance in design for deviation, ΔC_{dev}.

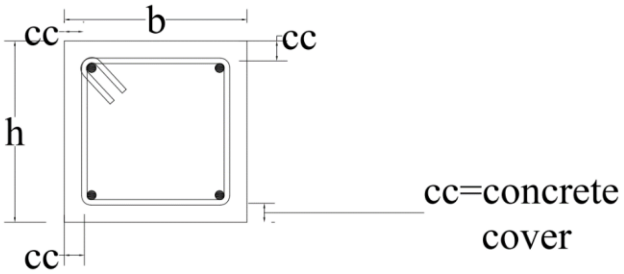


Figure 2.1. Concrete cover

$$C_{nom} = C_{min} + \Delta C_{dev} \quad \text{Eq. (2.4)}$$

With,

ΔC: The allowance in design for deviation with a recommended value of 10 mm

C_{min}: Is the minimum concrete cover

$$C_{min} = \max(C_{min,b};\ C_{min,dur} + \Delta C_{dur,\gamma} - \Delta C_{dur,st} - \Delta C_{dur,add};\ 10\text{mm} \quad \text{Eq. (2.5)}$$

Where:

$C_{min,b}$: Is the minimum cover due to bond requirement, equal to the diameter of the bars or the equivalent diameter in the case of bundled bars

$\Delta C_{dur,\gamma}$: Is the additive safety element with a recommended value of 0 mm

$\Delta C_{dur,st}$: Reduction of minimum cover for use of stainless steel

$\Delta C_{dur,add}$: Is the add reduction of minimum cover for use of additional protection

$C_{min,dur}$: Is the minimum cover due to environmental conditions in function of the exposure and the structural class of the building.

2.7.2. Horizontal structural element design

The horizontal structural element to be designed, in this section is a hollow block floor, two way floor, wood-concrete floor and beam. The latter is designed under Ultimate Limit State (ULS) and Serviceability Limit State (SLS). The modelling and design are done with the use of the software SAP 2000 V22. Materials, section properties, loads, loads combinations, restrains and constrains and other design parameters are defined and assigned to the beam to obtain the solicitation parameters and curves. The envelop curve is then obtained for each solicitation parameter.

2.7.2.1. Predesign of the hollow block and two way floor

For the hollow-block slab, the predesign is given by the deflection condition.

$$e > \frac{L}{25} \qquad \text{Eq. (2.6)}$$

From which, L= min ($L_{max,x}$; $L_{max,y}$). The unit is the centimeter (cm).

2.7.2.2. Predesign of the wood-concrete floor

The analytical analysis of a composite concrete wood slab following the norm EN1995-1-1, consist of calculating a composite slab compose of secondary beams made of timber and a reinforced concrete slab link with connectors. The analysis is made on a timber wood of length L submitted to the self-weight of the slab, the weight of the finishing and the live load. For the condition at the support, it will use a hinge at one end and a roller support at the other. The step to design such slab as shown in the figure 2.2 will be presented as proceed in the work. The analytical analysis of a mixed wood-concrete slab according to the dimensioning steps of the EN1995-1-1 standard consists of the calculation of a mixed slab composed of solid wood joists in our case and associated with a reinforced concrete slab by means of connectors. The analysis is carried out on a wooden beam of length L subjected to the dead weight of the concrete slab, the covering and the operating load. As a boundary condition for simple supports. The steps for dimensioning such a slab will be presented later on as shown in the figure 2.2.

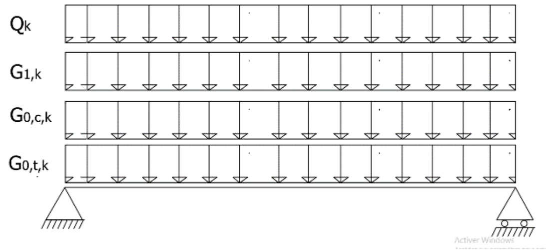


Figure 2.2. Mechanical scheme of a wooden beam subjected to different load

a) Loads

The figure 2.3 shows the cross section of the composite timber-concrete structure.

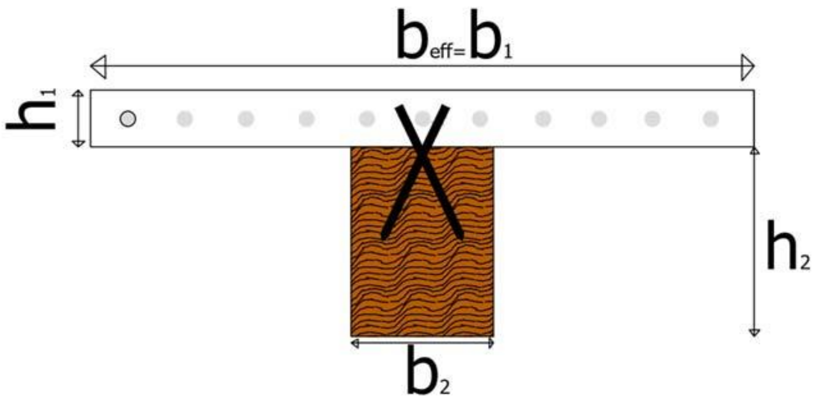


Figure 2.3. T-beam cross section of the composite timber-concrete structure

i) Self-weight of timber joist

$$G_{0,t,k} = A_2.\gamma_t \quad \text{Eq. (2.7)}$$

Where,

$G_{0,t,k}$ in kN/m

A_2 in m^2

γ_t in kN/m^3

ii) Self-weight of concrete slab

$$G_{0,c,k} = \gamma_c.A_1 \quad \text{Eq. (2.8)}$$

$G_{0,c,k}$ is the characteristic value of concrete slab self-weight in kN/m

A_1 in m^2

γ_c in kN/m^3

iii) Dead load

$$G_{1,k} = g_{1,k}.b_c \qquad \text{Eq. (2.9)}$$

$G_{1,k}$ in kN/m

$g_{1,k}$ in kN/m^2

b_c in m

iv) Variable loads

$$Q_k = q_{k \text{ of category B}}.b_1 \qquad \text{Eq. (2.10)}$$

Where,

$q_{k \text{ of category B}}$ is the characteristic value of imposed loads on floors in kN/m^2

b) Loads combinations

i) At the Ultimate Limit State (ULS)

$$F_{ULS} = \gamma_g.(G_{0,k} + G_{1,k}) + \gamma_q Q_k \qquad \text{Eq. (2.11)}$$

Where,

F_{ULS} in kN/m

ii) At the Service-ability Limit State (SLS)

$$F_{sls,rare} = G_{0,k} + G_{1,k} + Q_k \qquad \text{Eq. (2.12)}$$

$$F_{sls,quasi\text{-}permanent} = G_{0,k} + G_{1,k} + \psi_{2,1} Q_k \qquad \text{Eq. (2.13)}$$

c) Short-term analysis at the Ultimate Limit State

i) Design moment and shear (M_{ed}, V_{ed})

The analysis is made on a beam on two simple supports of length .The following equation give the value of the moment and at a middle respect to the origin O of frame. The situation is shown in the figure 2.4.

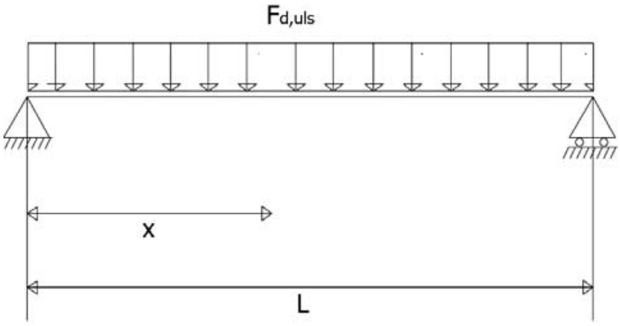


Figure 2.4. Timber beam under the total load actions, at ULS

At the middle of the beam L/2, we have the expression of the maximum design bending moment given by the equation 2.14.

$$M_{ed,max} = \frac{F_{uls}L^2}{8} \qquad \text{Eq. (2.14)}$$

The expression of the maximum design shear become

$$V_{ed,max} = \frac{F_{uls}L}{2} \qquad \text{Eq. (2.15)}$$

ii) Calculation of effective rigidity

The figure 2.5 shows the necessary parameters to compute the effective rigidity of the composite structure.

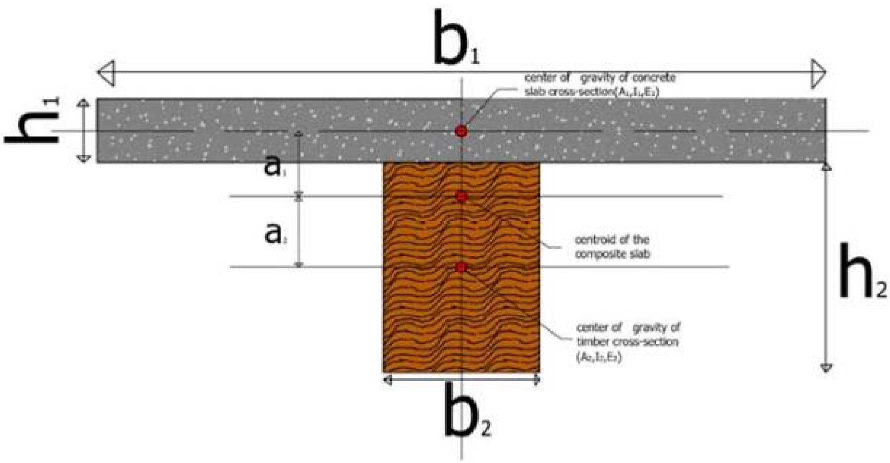


Figure 2.5. Cross-section showing the two centers of gravity for each element and the centroid of composite floor

$$(EI)_{eff} = E_1I_1 + \gamma_1E_1A_1a_1^2 + E_2I_2 + \gamma_2E_2A_2a_2^2 \qquad \text{Eq. (2.16)}$$

Where,

$(EI)_{eff}$ in $N.mm^2$

$$\gamma_1 = (1+\frac{\Pi^2 E_1 A_1 s}{k_u L^2})^{-1} \quad \text{Eq. (2.17)}$$

L in m

K_u in N/mm

$$K_u = \frac{2}{3}k_{ser} \quad \text{Eq. (2.18)}$$

k_{ser} in N/mm

a_1 and a_2 in mm

$$a_2 = \frac{\gamma_1 E_1 A_1.(\frac{h_1}{2}+t+\frac{h_2}{2})}{\gamma_1 E_1 A_1 + \gamma_2 E_2 A_2} \quad \text{Eq. (2.19)}$$

$$a_1 = 0.5(h_1 + h_2) - a_2 \quad \text{Eq. (2.20)}$$

h_1 and h_2 in mm

iii) Calculation of bending stresses

$$\sigma = \frac{MY}{I} \quad \text{Eq. (2.21)}$$

Two types of normal bending stresses are defined, one of which is composed by the normal bending stresses without collaboration of the concrete and the timber ($\sigma_{1,M}$, $\sigma_{2,M}$) and the other is composed by normal bending stresses introduced by the collaboration ($\sigma_{1,N}$, $\sigma_{2,N}$). The figure 2.6 shows the stress distribution in the timber and concrete.

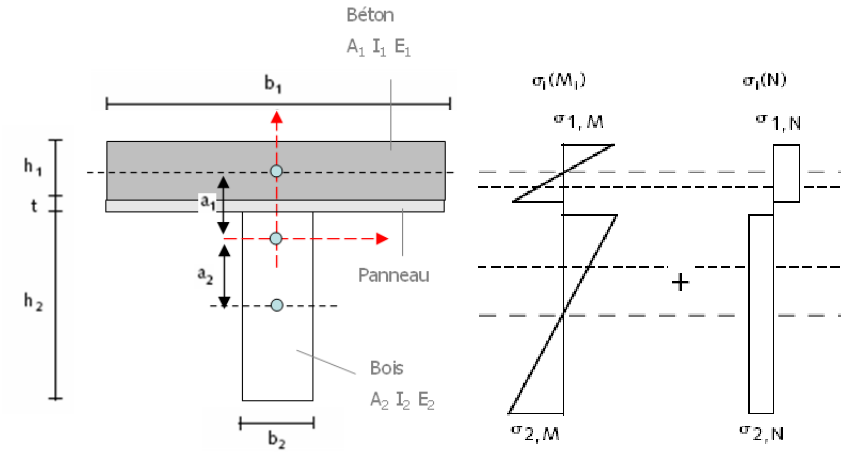


Figure 2.6. Stress distribution in timber and concrete without and with collaboration

Stress in the concrete

$$\sigma_{1,M} = 0.5\frac{E_1 h_1 M_{ed}}{(EI)_{eff}} \quad \text{Eq. (2.22)}$$

$$\sigma_{1,N} = \frac{\gamma_1 E_1 a_1 M_{ed}}{(EI)_{eff}} \quad \text{Eq. (2.23)}$$

Stress in the timber

$$\sigma_{2,M} = 0.5\frac{E_2 h_2 M_{ed}}{(EI)_{eff}} \quad \text{Eq. (2.24)}$$

$$\sigma_{2,N} = \frac{\gamma_2 E_2 a_2 M_{ed}}{(EI)_{eff}} \quad \text{Eq. (2.25)}$$

iv) Stress rate verification in concrete and timber

Concrete verification

The maximal compression in timber (Upper fiber) is given by equation 2.26.

$$\sigma_{c,d} = (\sigma_{1,M} + \sigma_{1,N}) \leq f_{c,d} \quad \text{Eq. (2.26)}$$

$$f_{c,d} = \frac{f_{c,k}}{\gamma_c} \quad \text{Eq. (2.27)}$$

$\sigma_{c,d}$ is the design bending stress value in the concrete;

$f_{c,d}$ is the design compression strength value of the concrete.

The maximal tensile in concrete (lower fiber) is given by the equation 2.28

$$\sigma_{t,d} = ((\sigma_{1,M} - \sigma_{1,N}) \leq f_{c,t,d} \quad \text{Eq. (2.28)}$$

$$f_{c,t,d} = \frac{f_{ctm}}{\gamma_c} \quad \text{Eq. (2.29)}$$

$\sigma_{t,d}$ is the design tensile stress value in the concrete;

$f_{c,t,d}$ is the design tensile strength value of the concrete;

f_{ctm} is the mean tensile strength value of the concrete.

Timber verification

$$\frac{\sigma_{2,N}}{f_{t,0,d}} + \frac{\sigma_{2,M}}{f_{m,d}} \leq 1 \quad \text{Eq. (2.30)}$$

$$f_{t,0,d} = k_{mod}\frac{f_{t,0,k}}{\gamma_M, b} \quad \text{Eq. (2.31)}$$

$$f_{m,d} = k_{mod}\frac{f_{m,k}}{\gamma_M, b} \quad \text{Eq. (2.32)}$$

d) Long term analysis at the ULS

i) Calculation of the equivalent rigidity

$$E_{1,end} = \frac{E_1}{1 + \varphi_{\infty,t0}} \quad \text{Eq. (2.33)}$$

$$E_{2,end} = \frac{E_2}{1 + \psi_2 k_{def}} \quad \text{Eq. (2.34)}$$

$$K_{u,end} = \frac{k_u}{1 + \psi_2 . k_{def,conexion}} \quad \text{Eq. (2.35)}$$

$$\gamma_{1,end} = (1 + \frac{\Pi^2 E_{1,end} A_1 s}{k_{ser,end} L^2})^{-1} \quad \text{Eq. (2.36)}$$

$$(EI)_{eff} = E_{1,end} I_1 + \gamma_{1,end} E_{1,end} A_1 a_1^2 + E_{2,end} I_2 + \gamma_{2,end} E_{2,end} A_2 a_2^2 \quad \text{Eq. (2.37)}$$

ii) Stress verification in the elements

The process is the same as that for short-term analysis, we must to verify the concrete and timber section according to the formulas given in short-term.

e) Verification of the shear in timber at short and long term analysis

The process consist to verify the maximum design shear stress $\tau_{z,d}$ which must be smaller than the design shear strength value of the timber $f_{v,d}$.

$$\tau_{z,d} \leq f_{v,d} \qquad \text{Eq. (2.38)}$$

$$\tau_{z,d} = \frac{3}{2}\frac{V_{ed,max}}{A_2} \qquad \text{Eq. (2.39)}$$

$$f_{v,d} = k_{mod}\frac{f_{v,0,k}}{\gamma_{M},b} \qquad \text{Eq. (2.40)}$$

f) Verification of the connection at the short-term

The process consist to verify that the design load on the fasteners F_d in a row is smaller than the load capacity of the fastener P_{Rd}.

$$F_d \leq P_{Rd} \qquad \text{Eq. (2.40)}$$

$$F_d = \frac{\gamma_1 E_1 A_1 a_1 s}{(EI)eff}.V_{ed} \qquad \text{Eq. (2.41)}$$

$$P_{Rd} = \frac{P_{Rk}}{\gamma_M} \qquad \text{Eq. (2.42)}$$

P_{Rd} is the characteristic shear strength value of the connection. We work with steel rebar and the value is taken according to results of shear test in short-term on this type of fasteners.

g) Calculation at the Service-ability Limit State

i) Short-term analysis

Calculation of the equivalent rigidity

At the SLS, the process to obtain the equivalent rigidity is the same. Taking the slip modulus at SLS equal to K_{ser} (EN1995-1-1, section2). Then, only γ_1 change in the formula

$$\gamma_1 = (1 + \frac{\Pi^2 E_1 A_1 s}{k_{ser} L^2})^{-1} \qquad \text{Eq. (2.43)}$$

The equivalent rigidity is given with the same formula

$$(EI)_{eff} = E_1 I_1 + \gamma_1 E_1 A_1 a_1^2 + E_2 I_2 + \gamma_2 E_2 A_2 a_2^2 \qquad \text{Eq. (2.44)}$$

Calculation of deflection at short-term

$$f_{max} = \frac{5}{384} \cdot \frac{F_{ELS,rare} L^4}{(EI)_{eff}} \qquad \text{Eq. (2.45)}$$

According to EN1995-1-1 (section 7), the admissible value of deflection at the short-term is givens as

$$f \leq L/150 \text{ to } L/500 \qquad \text{Eq. (2.46)}$$

ii) Long-term analysis

Equivalent rigidity

$$E_{1,end} = \frac{E_1}{1 + \varphi_{\infty,t0}} \qquad \text{Eq. (2.47)}$$

$$E_{2,end} = \frac{E_2}{1 + k_{def}} \qquad \text{Eq. (2.48)}$$

$$K_{ser,end} = \frac{k_u}{1 + k_{def,conexion}} \qquad \text{Eq. (2.49)}$$

$$\gamma_1 = (1 + \frac{\Pi^2 E_{1,end} A_1 s}{k_{ser,end} L^2})^{-1} \qquad \text{Eq. (2.50)}$$

$$(EI)_{eff} = E_{1,end} I_1 + \gamma_{1,end} E_{1,end} A_1 a_1^2 + E_{2,end} I_2 + \gamma_{2,end} E_{2,end} A_2 a_2^2 \qquad \text{Eq. (2.51)}$$

2.7.2.3. Predesign of the beam

For the beam, the predesign is given by

$$H \geq \frac{L}{14} \qquad \text{Eq. (2.52)}$$

$$0.3H \leq b \leq 0.5H \quad \text{Eq. (2.53)}$$

For the beam in the case of wood-concrete floor we have.

$$R_1 = \frac{GL_1}{2} + \frac{QL_1}{2} \quad \text{Eq. (2.54)}$$

$$R_{1,distributed} = \frac{GL_1}{2b_{eff}} + \frac{QL_1}{2b_{eff}} \quad \text{Eq. (2.55)}$$

$$R_2 = \frac{GL_2}{2} + \frac{QL_2}{2} \quad \text{Eq. (2.56)}$$

$$R_{2,distributed} = \frac{GL_2}{2b_{eff}} + \frac{QL_2}{2b_{eff}} \quad \text{Eq. (2.57)}$$

The total load is

$$F = R_{1,distributed} + R_{2,distributed} + W_{wall}.h_{wall} \quad \text{Eq. (2.58)}$$

a. Ultimate Limit State design

Under ULS, the floor and beam will be verified for bending moment and shear force solicitations as there are no axial forces on the floor and beam.

i. Bending moment design

Design for bending moment is done with the envelope curve of the bending moment solicitation parameter. Provisions given by Eurocode 2 recommend moment reduction at the support as shown in Figure 2.7. For continuous beams, the value depends on the connection between the beam and the support.

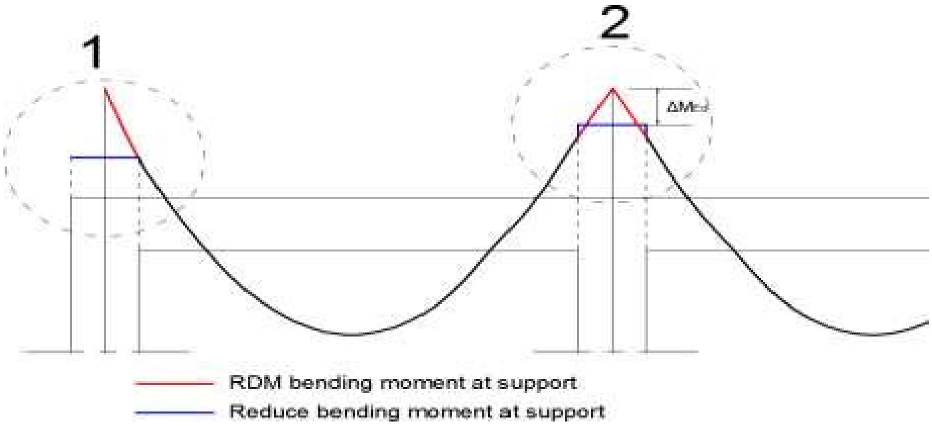


Figure 2.7. Moment reduction at supports (Djeukoua, 2019)

The amount of this reduction s given by:

$$\Delta M_{Ed} = F_{Ed,sup}.t/8 \qquad \text{Eq. (2.59)}$$

Where:

t is the breadth of the support

FEd, is the design of the support

ii. Longitudinal steel reinforcement

The section of the beam is a rectangular one. The longitudinal steel reinforcement is computed using Eq. 2.60.

$$A_s = \frac{M_{Ed}}{0.9df_{yd}} \qquad \text{Eq. (2.60)}$$

The figure 2.8 shows the steel reinforcements in the beam.

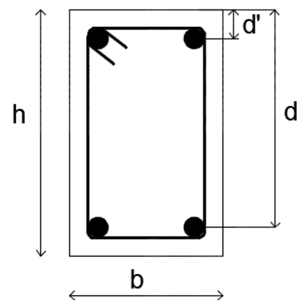


Figure 2.8. Example for beam section with longitudinal reinforcements

The provisions of the Eurocode 2 to be verified by the beam cross section for minimum and maximum reinforcement areas are given in Eq. 2.61 and Eq. 2.62.

$A_{s,min} = \max(0.26\frac{f_{ctm}}{f_{yk}}.b.d; 0.0013.b.d)$ Eq. (2.61)

$A_{s,max} = 0.004A_c$ Eq. (2.62)

Where:

b is the mean width of the tension zone

d is the effective depth of the section

f_{yk} is tensile strength of the concrete

iii. Verification of the steel reinforcement

We compute the number of reinforcement bars needed and the corresponding area of reinforcement. The section is verified, using the position of the neutral axis inside the section, by calculating the resisting bending moment of the section as presented in the figure 2.9.

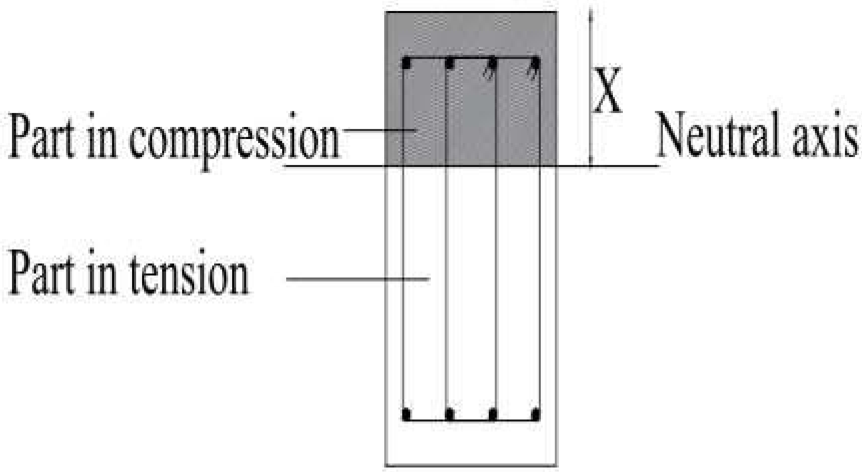


Figure 2.9. Neutral axis position in the beam section

Below is the computation of the neutral axis position.

$X = \frac{A_s f_{yd}}{0.8bf_{cd}}$ Eq. (2.63)

Where:

A_s is the adopted area of steel

F_{yd} is design yielding strength of the steel

Fcd is design concrete compressive strength b is the mean width of the tension zone

We then calculate the resisting moment as

$M_{Rd} = As.f_{yd}(d-0.4x)$ Eq. (2.64)

With

A_s is effective area of the steel section

f_{yd} is design yielding strength of the steel

x is neutral axis

b. Grashof principle for two way floor

The figure 2.10 shows one panel of two way floor.

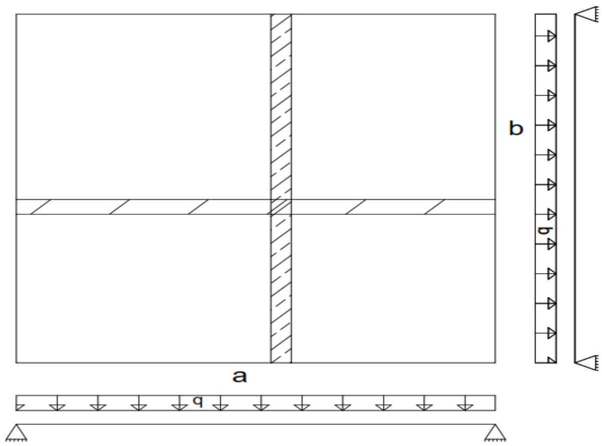


Figure 2.10. One panel of two way floor

For two way floor the distribution of loads is in directions (a) and (b). To design the floor in each direction we apply Grashof principle. Let q the distributed load apply to the two way floor. In each direction the load we will use to design the two way floor is

$$q_a = \frac{b^4}{a^4+b^4} q$$ Eq. (2.65)

$$q_b = \frac{a^4}{a^4+b^4} q$$ Eq. (2.66)

When we have many panels, we take the maximum load in each direction.

c. Shear verification

The beam element needs to resist shear forces. Transversal steel reinforcement is therefore needed inside the beam. These reinforcements are called stirrups and can be determined from the minimum of V_{Rsd} and VRcd given in Eq 2.67 and Eq 2.68. Whether shear reinforcement is needed or not is agreed on, on comparison of the acting shear, V_{Ed}, with the shear resistance of the members without shear reinforcements, $V_{Rd,C}$.

Without shear reinforcement

$$V_{Rd,c} = \max\{[C_{Rd,c}k(100\rho_l f_{ck})^{1/3} + k_1\sigma_{cp}]b_w d;\ (V_{min} + k_1\ \sigma_{cp})\ b_w d\} \qquad \text{Eq. (2.67)}$$

With

$V_{Rd,}$ is the characteristic strength of the reinforcement

d is the is the effective depth of the section

is the smallest width of the cross section in the tensile area

$\sigma_{cp} = N_{Ed}/A_c < 0.2\ \ [N/mm^2]$

N_{Ed} is axial force of the cross section due to loading or prestressing

b_{wd} is concrete cross-sectional area

$k = 1 + \sqrt{\frac{200}{d}} \leq 2.0$ With d in mm

$\rho_l = \frac{A_{sl}}{b_w d} \leq 0.02$

With shear reinforcement

$$V_{Rd} = \min(\frac{A_{sw}}{s} Z f_{ywd} cot\Theta;\ \alpha_{cw} b_w Z V_1 f_{cd}/(\cot\Theta + \tan\Theta) \qquad \text{Eq. (2.68)}$$

2.7.2.4. Serviceability Limit State

The parameters of interest in this section are the stress limitations, the crack and the deflection control. The rare combination mentioned earlier in the previous sections, is the combination used for the stress verification because it permits to avoid inelastic deformation of the reinforcement and longitudinal cracks in concrete. Long-term and short-term effects are accounted for by the modular ratio, on which depends the stress value.

$$n_0 = \frac{E_s}{E_c} \qquad \text{Eq. (2.69)}$$

$$n_\infty = n_0(1+\varphi_L \times \rho_\infty) \qquad \text{Eq. (2.70)}$$

Where:

φ_L = 0.55 for shrinkage of concrete and the parameter $\rho_\infty = 2 \div 2.5$.

For an uncracked concrete section, the neutral axis is computed using equation 2.71.

$$X = \frac{nA_s}{b}[-1 + \sqrt{1 + \frac{2bd}{nA_s}}] \qquad \text{Eq. (2.71)}$$

The moment of inertia of the uncracked section is given by

$$J = \frac{bX^3}{3} + NA_s(d - X)^2 \qquad \text{Eq. (2.72)}$$

The stresses in the concrete and steel can now be computed using Eq 2.73 and Eq 2.74

$$\sigma_c = \frac{M}{J}X \qquad \text{Eq. (2.73)}$$

$$\sigma_s = \frac{n_\infty M(d-X)}{J} \qquad \text{Eq. (2.74)}$$

The verifications of these stresses, as provided by Eurocode 2, are given below.

$$\sigma_c \leq k_1 \times f_{ck} \qquad \text{Eq. (2.75)}$$

$$\sigma_c \leq k_2 \times f_{ck} \qquad \text{Eq. (2.76)}$$

$$\sigma_c \leq k_3 \times f_{yk} \qquad \text{Eq. (2.77)}$$

With k_1 = 0.6, k_2 = 0.45 and k_3 = 0.8

2.7.3. Vertical structural element design

For the column design, a 3D modelling of the building in the software SAP2000 will be done. Also, different loads arrangements will be considered to obtain the envelope curve for each solicitation. The pre-dimensioning is done and the design at ULS for the axial force, the bending moment and the shear force and the verification is done for the slenderness.

2.7.3.1. Column pre-dimensioning

The preliminary design of the column is done in two steps. The first step is based the axial loads resistance to determine the minimum area section and the second one on the modal analysis of the 3D model of the structure to verify the global dynamic behaviour.

In a seismic area, the preliminary design of the column considers that 60% of the concrete resistance is used to take over the axial force. Then we can estimate the minimum area section of the column using equation 2.78.

$$N_{Rd} = 0.6 \times f_{cd} \times A_c \geq N_{sd} \qquad \text{Eq. (2.78)}$$

Where:

Ac: is the concrete section area;

Nsd: is the axial load computed using the recovery area of the column

The axial load is computed using equation 2.74

$$N_{sd} = q \times S_r \times n \qquad \text{Eq. (2.79)}$$

Where:

q: is the uniform distributed loads on each floor computed at ULS;

Sr: is the recovery area of the column;

n: is the number of stories above the considered column

2.7.3.2. Bending moment-axial force verification

For a column line, from the ground floor to the roof, the envelope curve is derived. Shear forces put aside, each column at each level is subjected to moment and axial force; these solicitations are obtained from the respective envelope curves. Each couple of points, M-N (moment-axial force) should belong to the section M-N interaction diagram. The interaction is a diagram representing all the limit situations deterministic of the section failure and is computed through the determination of points, which will be plotted to obtain the curve. Points lying within the diagram are respectful of the design criteria otherwise failure occurs. When considering a rectangular section as shown in Figure 2.11, the computation of the points is done as presented in the subsequent sections.

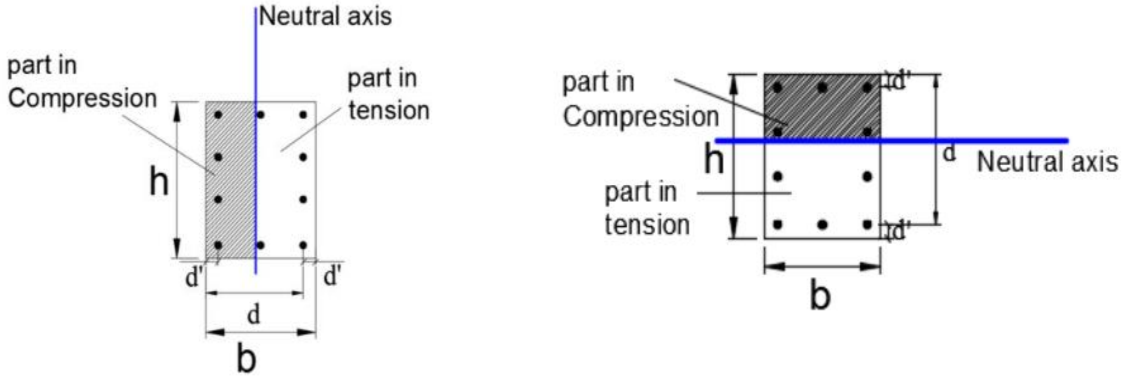


Figure 2.11. Rectangular section to illustrate the computation of the M-N diagram for different direction of the neutral axis (Djeukoua,2019)

a. First point

The section is completely subjected to tension, hence, the concrete is not reacting. We impose $\varepsilon_s = \varepsilon_{su}$, $\varepsilon'_s = \varepsilon_{syd}$ then the stress inside the element correspond to the design yielding strength of the steel reinforcement and the limit axial force and bending moment are obtained from the equations 2.80 and 2.81.

$$N_{Rd} = f_{yd}.A_s + f_{yd}.A'_s \quad \text{Eq. (2.80)}$$

$$M_{Rd} = f_{yd}.A_s.(\frac{h}{2} - d') - f_{yd}.A'_s.(\frac{h}{2} - d') \quad \text{Eq. (2.81)}$$

b. Second point

The section is completely subjected to tension. We impose: $\varepsilon_s = \varepsilon_{su}$, $\varepsilon_c = 0$. We should verified if the upper steel is yielded or not by determining the strain ε'_s. The limit axial force and bending moment is obtained from the equations 2.80 and 2.81.

c. Third point

We impose that the failure is due to concrete and the lower reinforcement is yielded. We assume $\varepsilon_s \geq \varepsilon_{syd}$, $\varepsilon_c = \varepsilon_{cu2}$ and we determine the neutral axis position. Then we should verify if the upper steel is yielded or not by determining the strain ε'_s. In order to determine the corresponding stress. The limit axial force and bending moment are obtained from the equations 2.82 and 2.83.

$$N_{Rd} = -\beta_1.b.x.f_{cd} + f_{yd}.A_s - f_{yd}.A'_s \quad \text{Eq. (2.82)}$$

$$M_{Rd} = f_{yd}.A'_s.(\frac{h}{2} - d') + f_{yd}.A_s.(\frac{h}{2} - d') + \beta_1.b.x.f_{cd}(\frac{h}{2} - \beta_2.x)$$ Eq. (2.83)

d. Fourth point

We impose that the failure is due to concrete and the lower reinforcement reaches exactly $\varepsilon_s = \varepsilon_{syd}$. As for the previous point, we determine the neutral axis position and the strain $'_s$. The limit value of the acial force and the bending moment is determined using the equations 2.82 and 2.83.

e. Fifth point

We impose that the failure is due to concrete and the lower reinforcement reaches exactly $\varepsilon_s = 0$ then the neutral axis position is equal to the effective depth of the section. The limit axial force and bending moment id obtained from the equations 2.84 and 2.85.

$$N_{Rd} = -\beta_1.b.x.f_{cd} - f_{yd}.A'_s$$ Eq. (2.84)

$$M_{Rd} = f_{yd}.A'_s.(\frac{h}{2} - d') + \beta_1.b.d.f_{cd}(\frac{h}{2} - \beta_2.x)$$ Eq. (2.85)

f. Sixth point

We impose that the section is uniformly compressed. We assume $\varepsilon_s = \varepsilon_c \geq \varepsilon_{c2}$. The limit axial force and bending moment is obtained from the equations 2.86 and 2.87 respectively.

$$N_{Rd} = -b.h.f_{cd} - f_{yd}.A_s - f_{yd}.A'_s$$ Eq. (2.86)

$$M_{Rd} = f_{yd}.A'_s.(\frac{h}{2} - d') - f_{yd}.A_s.(\frac{h}{2} - d')$$ Eq. (2.87)

The steel reinforcement of the column is considered taking into account the limitations of the Eurocode 2 defined by equation 2.88 and 2.89.

$$A_{s,min} = \max(0.10N_{Ed} / f_{yd} ; 0.003\ A_c)$$ Eq. (2.88)

$$A_{s,max} = 0.04A_c$$ Eq. (2.89)

Where:

NEd: is the design axial compression force

: is the design yield strength of the longitudinal reinforcement

2.7.3.3. Shear verification

The verification procedure is the same for the beam. The detailing of members prescribed by the Eurocode 2 imposed a minimum diameter of 6 mm or one quarter the maximum diameter of the longitudinal bars. The maximum spacing of the transverse reinforcement is given by the equation 2.90.

$$S_{cl,max} = \min(20\varnothing_{l,min};b;400mm) \quad \text{Eq. (2.90)}$$

Where:

$\varnothing_{l,min}$: is the minimum diameter of the longitudinal bars

b: is the lesser dimension of the column

This maximum spacing has to be reduced by a factor 0.6 in sections within a distance equal to the larger dimension of the column cross-section above or below the beam.

2.7.3.4. Slenderness verification

The slenderness verification permits to know if we have to consider the second order effect or not. It consists in verifying if the slenderness of the element is below a limit value, defined by the Eurocode 2 as

$$\lambda_{lim} = 20.A.B.C/\sqrt{n} \quad \text{Eq. (2.91)}$$

Where:

$A = \frac{1}{1+0.2\varphi_{ef}}$ (φef is the effective creep ratio; A= 0.7 if φef is not known)

$B = \sqrt{1+2\omega}$ ($\omega = A_s f_{yd} /(A_c f_{cd})$: is the mechanical reinforcement ratio;

$C = 1.7 - r_m$ ($r_m = M_{01}/M_{02}$: is the moment ratio; equal to 1 for unbraced system)

$n = N_{Ed}/(A_c f_{cd})$: relative normal force

The slenderness of an element is evaluated by the formula

$$\lambda = l_0/i \quad \text{Eq. (2.92)}$$

Where

i is the gyration radius of the uncracked concrete section

lo is the effective length of the element (lo = $0.7l$)

The gyration radius of the uncracked section is given by

$$i = \sqrt{\frac{I}{A}} \quad \text{Eq. (2.93)}$$

Where I is the moment of inertia and A is the area of the section.

2.8. Analysis criteria

The results of the study will be analysed on four parameters that are: the period of the three first vibration modes of the structure, the lateral deformation, the inter-story drift and the shear.

2.8.1. Period and vibrations modes

The fundamental period of a building is an inherent properties who depend on the properties of the structure. In seismic zone, the spectral acceleration of a building is function of this period of the structure then this properties of the structure can increase or reduce this spectral acceleration and then become an important parameter in the response of a structure.

Some empirical formulas permit to estimate this period. It’s the case of the one defined by the Eurocode 8 for buildings with heights up to 40m expressed by.

$$T_1 = C_t.H^{3/4} \quad \text{Eq. (2.94)}$$

Where:

C_t: is a coefficient that depends on the moment resisting type of the structure

H: is the total height of the building above the foundations in meters

The real values of this properties of the building can be obtained through a dynamic analysis of the structure and this parameter is associated to a mode shapes (vibration mode) and a mass participating ratio. The mode shape of the structure describes the configurations into which a structure will deform naturally while the mass participation ratio indicated the contribution of this mode shapes to the structural response. The vibration mode hence indicate the deformations modes of the structure.

2.8.2. Storey displacement

The storey displacement is the absolute value of displacement of the storey under action of the lateral forces. It permits also a proper estimation of the separation distance between buildings. Figure 2.13 shows the displacement of the last storey of a building.

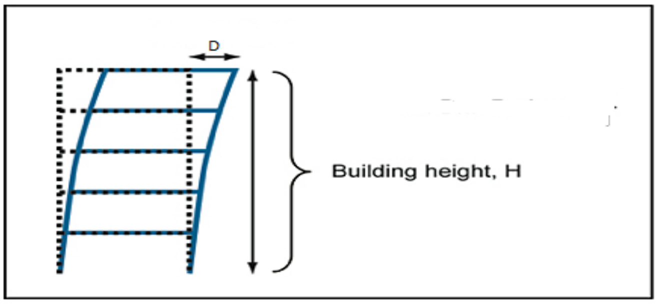


Figure 2.12. Storey displacement of the building (Zahura et al, 2016)

The total displacements must be controlled to mitigate the effects of secondary P-Δ effects and overall stability of the building. In seismic design, this displacement can affect both the structural elements that are part of the lateral force resisting system and structural elements that are not part of the lateral force resisting system. This parameter are better evaluate through the interstorey drift ratio presented in the next part.

2.8.3. Storey drift ratio

Storey drift ratio is the maximum relative displacement of each floor divided by the height of the same floor and is an important parameter that will be evaluated. Inter-story drift represents the most important parameter to be analysed as it is strictly connected to the damage suffered by both structural and non-structural elements. The inter-story drift has been employed as an index to evaluate the deformation capacity of a building and to further determine its performance. This parameter is evaluated as the difference of the average lateral displacements at the top and the bottom of a storey.

The limitation of the inter-storey drift by the Eurocode 8 for buildings having non-structural elements of brittle material attached to the structure is given by the equation 2.95.

$$d_r \, \nu \leq 0.005h \qquad \text{Eq. (2.95)}$$

Where

dr: is the inter-storey drift defined as:

$$d_r = d_{i+1} - d_i \qquad \text{Eq. (2.96)}$$

d_{i+1} is the deflection at the (i+1) level;

d_i is the deflection at the (i) level;

ν: is a reduction factor that depends on the importance class of the building.

2.8.4. Shear forces

The seismic response of the structure in terms of the story shear as well as internal forces over the height of the structural elements is selected as response parameters of interest as these are generally considered the most important response parameters in seismic design practice.

Conclusion

This chapter had as objective to present the different codes, the different procedures that will be used in this work and the seismic performance on which will be based the analysis. The analysis will be performed using a structural analysis software SAP 2000 version 22 while the different designs will be done manually through the software Excel applying the European standards. After all the different procedures have been well described, the case study will be presented, analysed in the software SAP 2000 and statically design following the process presented in this section.

CHAPTER 3: RESULTS AND INTERPRETATIONS

Introduction

After the presentation of the methodology, this chapter will consists of a preliminary part concerning the presentation of the case study and the different characteristics considered for its analysis. This will be followed by a static analysis of the case study in order to determine the different characteristics of the structural elements of the structure. At the end, in order to present the different information necessary for the next section, some results of the dynamic analysis notably the period will be presented.

3.1. General presentation of the site

Here, we present the study area through its location, geology, relief and soil.

3.1.1. Geographic location

Yaoundé is the political capital of Cameroon and chief town of the Centre region; situated at latitude 3.87^{o} North and longitude 11.52^{o} East at an elevation of 760 metres above sea level. Located 300km from the Atlantic Ocean and surrounded by 7 hills, Yaoundé belongs to the Mfoundi division of the Centre region and measures a total surface area of 183 km^2.

3.1.2. Geology and relief

The bedrock in Yaoundé is mainly composed of gneiss. This rock is neither porous nor soluble, but it is its discontinuities (faults, diaclases) that give fissure permeability to the formation. The hydrogeology is characterized by continuous aquifers, approximately exploitable overlying water bearing fissures or fracture aquifers in the bedrock; these types of aquifers are superimposed or isolated. Concerning the relief, the land rises gently in escarpments from the southwestern coastal plain before joining the Adamawa Plateau via depressions and granite massifs. The field is characterised by rolling, forested hills, the tallest of which have bare, rocky tops.

3.2. Presentation of the case study

The case study, is a six-storey reinforced concrete building frame for office use. It is a rectangular floor, with its length being 21 m and the width 16 m. The total height of the building is 18m with the height of the ground floor being equal to 3m. The building is regular in plan and in elevation without setback. The floor is symmetric by the x-axis. Beams on the same floor do not have the same design. They have the same sections but not the same lengths, and this is the case in either directions (x-direction and y-direction). The structure has identical floor plan as from level one to level six. The plan view for all level is plotted in the figure 3.1.

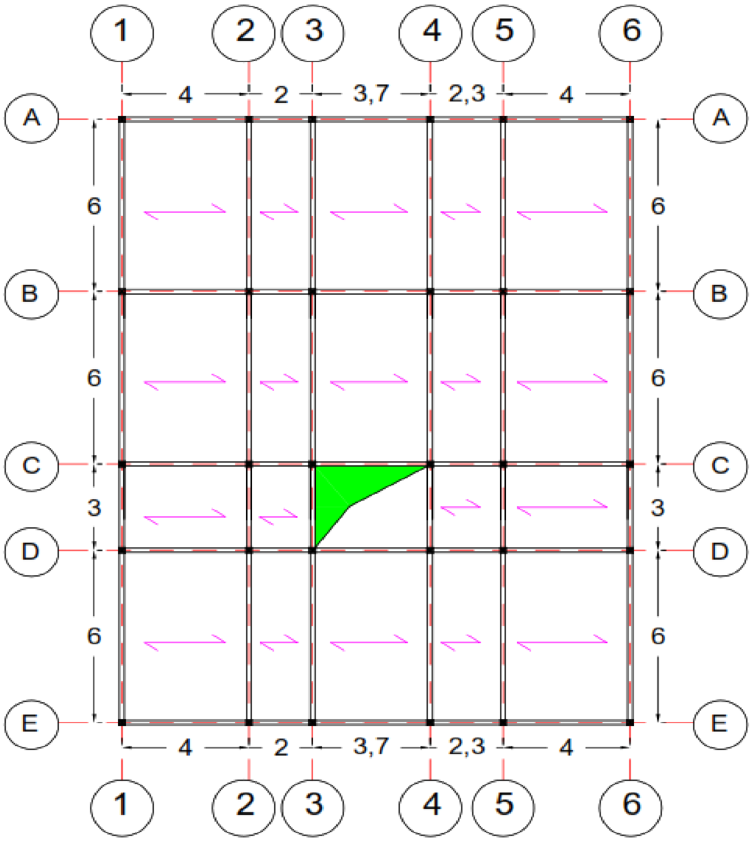


Figure 3.1. Floor plan view

Figure 3.2 presents the model of the structure in SAP 2000.

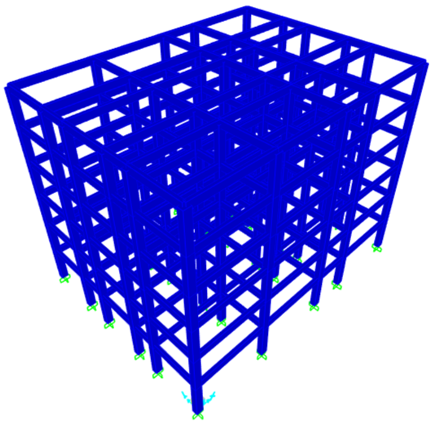


Figure 3.2. Model of the structure in SAP 2000.

3.3. Design of the floors

The floors will be design here are hollow-block floor, two way floor and wood-concrete floor.

3.3.1. Hollow-block floor

The plan view is plotted in the figure 3.1.

3.3.1.1. Material properties

For the analysis and the design, the concrete class adopted for the superstructure is C25/30 and the longitudinal steel reinforcement is Fe400B. For the transversal reinforcement, we consider a characteristic yield strength of 235 N/mm². The main characteristics of these materials for linear analysis and design of the structure are given in table 3.1 for the concrete and table 3.2 for the steel reinforcement.

Table 3.1. Concrete characteristics, Eurocode 2

Property	Value	Unit	Definition
Class	C25/30	-	Concrete class
f_{ck}	25	N/mm^2	Characteristic compressive strength at 28 days
$f_{cm} = f_{ck} + 8$	33	N/mm^2	Mean value of concrete cylinder compressive strength
γ_c	1.5	-	Partial safety factor for concrete
$f_{cm} = \dfrac{\alpha_{cc} f_{ck}}{\gamma_c}$	14.16	N/mm^2	Design value of compressive strength
$f_{ctm} = 0.3(f_{ck})^{\frac{2}{3}}$	2.56	N/mm^2	Mean value of axial tensile strength of concrete
$f_{ctd} = \dfrac{0.7 f_{ctm}}{\gamma_c}$	1.2	N/mm^2	Design resistance in traction
$E_{cm} = 22000(f_{cm}/10)^{0.3}$	31476	N/mm^2	Secant modulus of elasticity
v	0.5	-	Poisson ratio
G	13115	N/mm^2	Shear modulus
γ	25	kN/m^3	Specific weight of concrete

Table 3.2. Longitudinal reinforcement characteristics

Property	Value	Unit	Definition
Class	B400B		Steel class
f_{yk}	400	N/mm^2	Characteristic yield stress
γ_s	1.15	-	Partial safety factor for steel
γ	78.5	kN/m^3	Specific weight of steel
ν	0.3	-	Poisson ratio

The hollow block floor section is plotted in the figure 3.3.

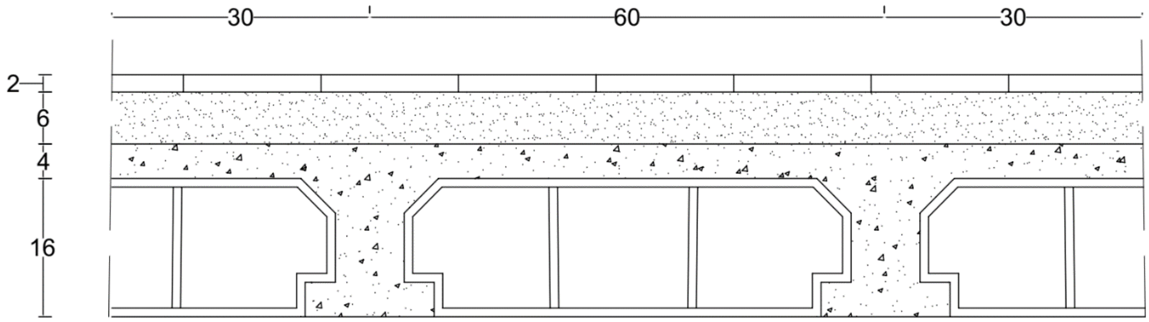


Figure 3.3. Hollow block floor section

3.3.1.2. Definition of the loads

The Different loads applied on our structure are shown in the table 3.3.

Table 3.3. Designation of the loads

Designation	Symbols	Values (kN/m²)
Hollow-block slab (16cm + 4cm)	g_1	2.85
Ceramic stoneware Tiles(2 cm)	g_2	$0.22 \times 2 = 0.44$
Screed	g_2	$0.22 \times 6 = 1.32$
Imposed loads	q	3

The influence length is l= 0,6m. The loads applied are calculated by the equation

$$Q = q \times l \quad \text{Eq. (3.1)}$$

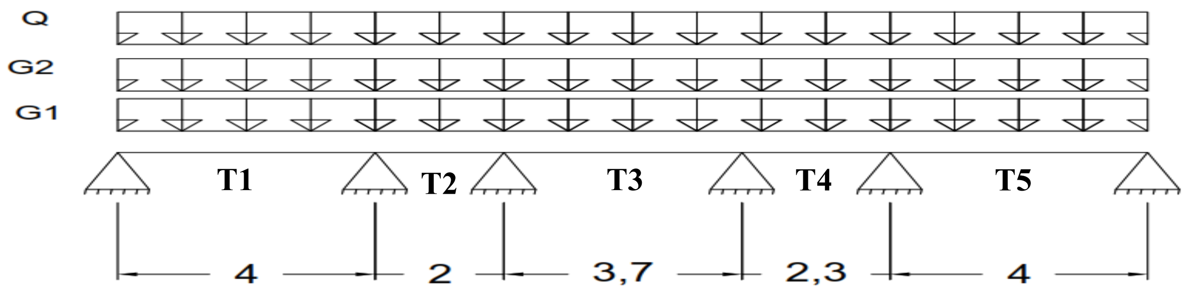
Table 3.4 shows the loads.

Table 3.4. Loads of design

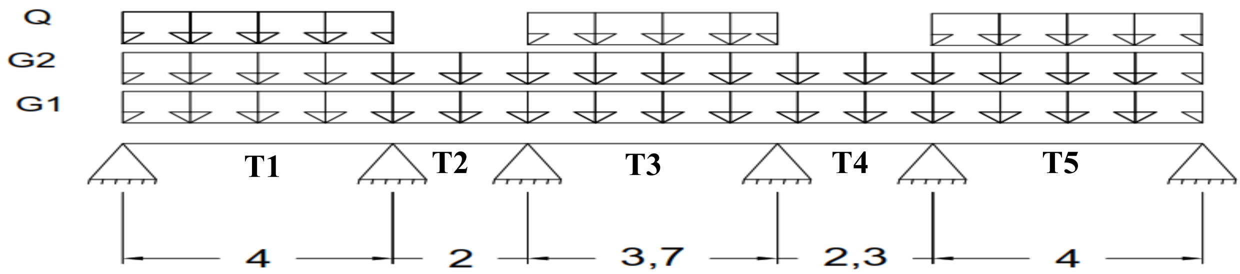
Designation	Value (kN/m)
Self-weight of the slab	1.71
Non-structural permanent loads	1.66
Variable loads	1.8

3.3.1.3. Combinations

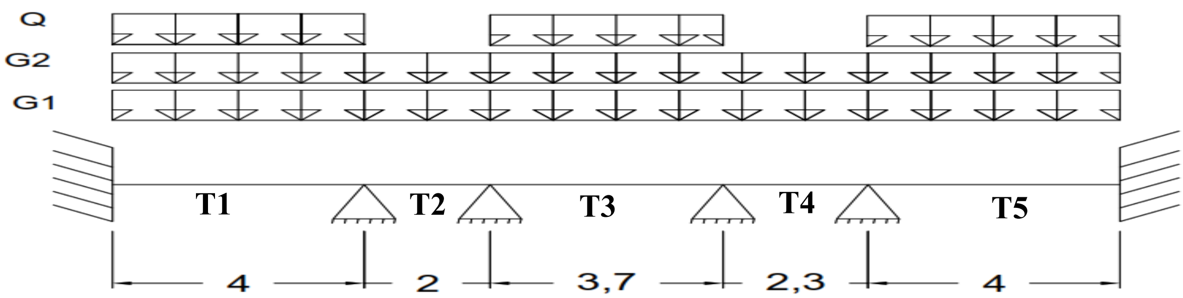
From this model, tree loads arrangements are defined for the design and are presented in the figure 3.4.



Load combination ULS T1T2T3T4T5



Load combination ULS T1T3T5



Load combination ULS T1T3T5 embedded

Figure 3.4. Load combination on the floor

3.3.1.4. Ultimate limit state

The envelope curve of bending moment is given in the figure 3.5.

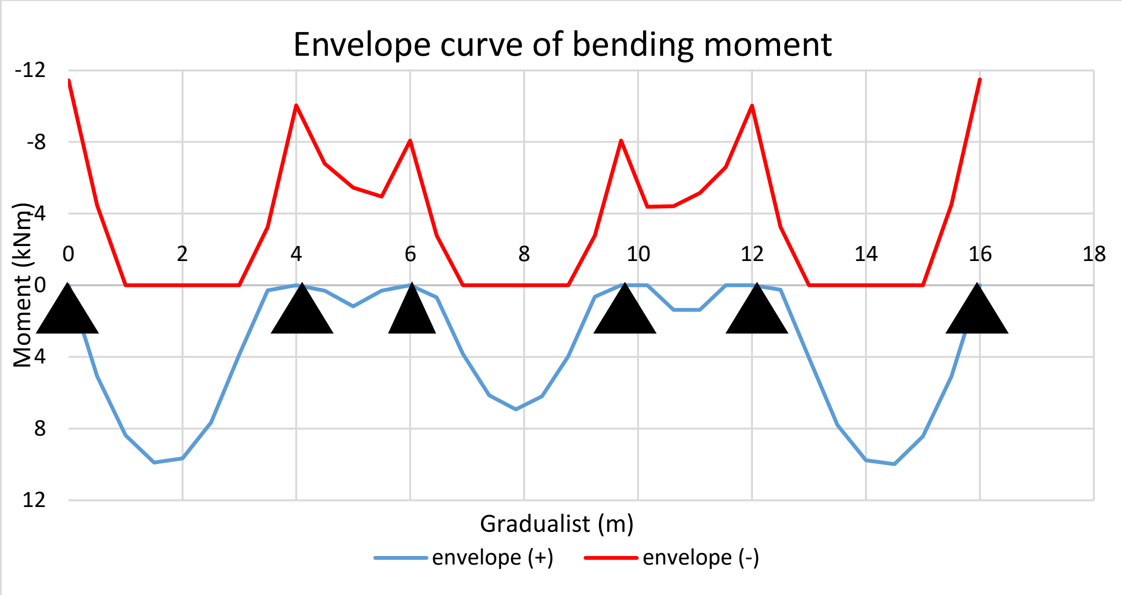


Figure 3.5. Envelope curve of bending moment

The envelope curve of shear is given in the figure 3.6.

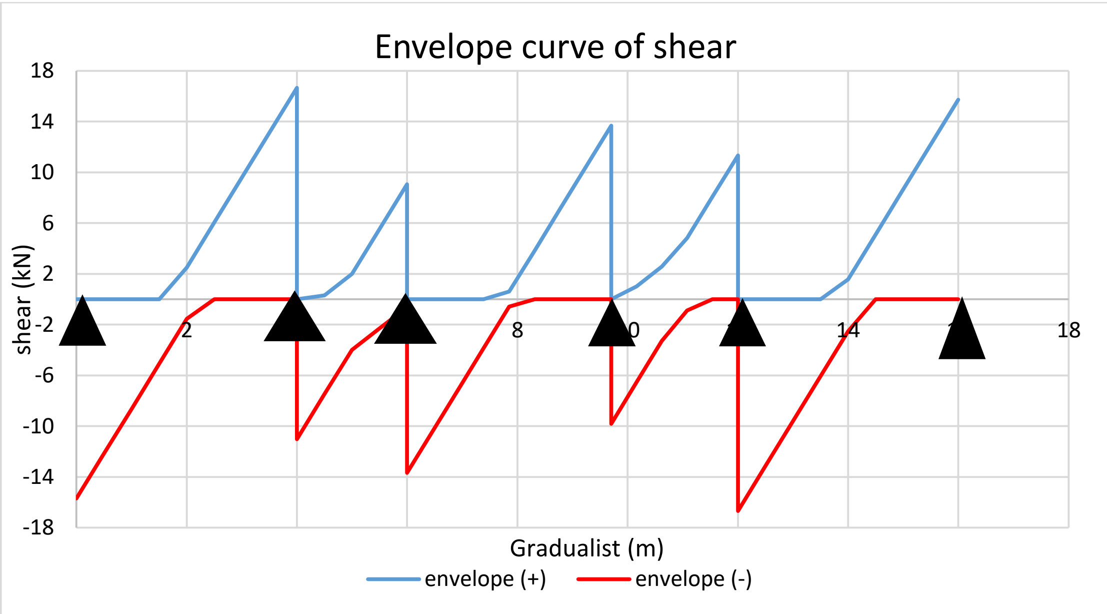


Figure 3.6. Envelope curve of shear

From the envelope curve of the bending moment presented in figure 3.4, we apply the procedure detailed in the section 2.4.2.4 to obtain the final solicitation curve presented in the figure 3.7.

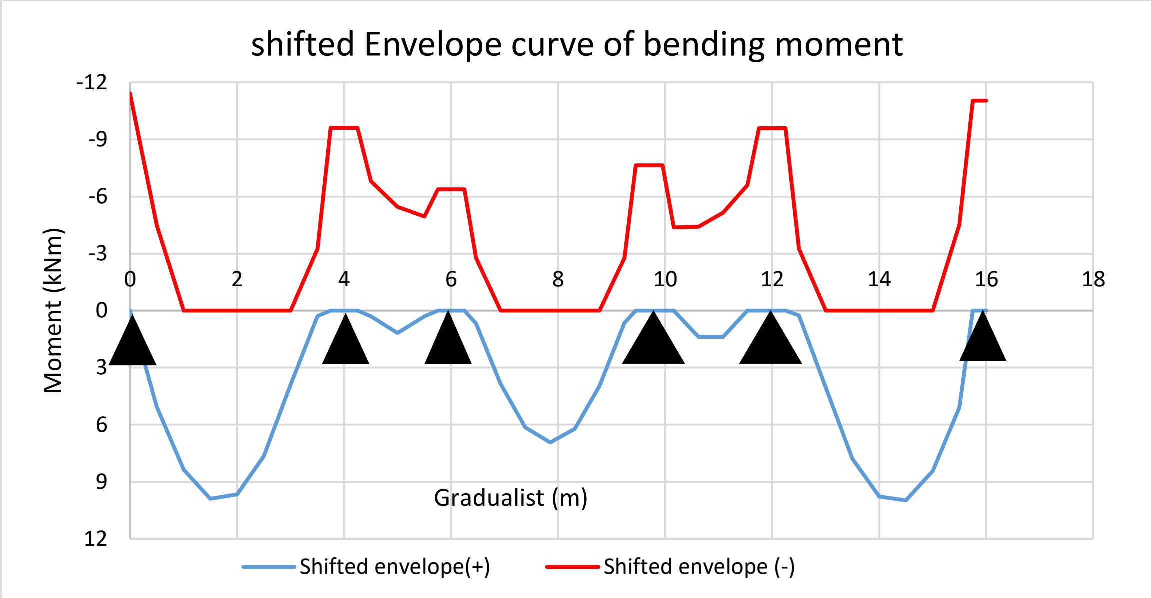


Figure 3.7. Shifted envelope curve of bending moment

3.3.1.5. Calculation of the steel bars reinforcement

The steel reinforcement is evaluated using the equation 2.60 and the section obtained is verified for the detailing of member presented in the equations 2.61 and 2.62. At the end, the steel section is evaluated and verified and the results obtained is presented in the figure 3.8.

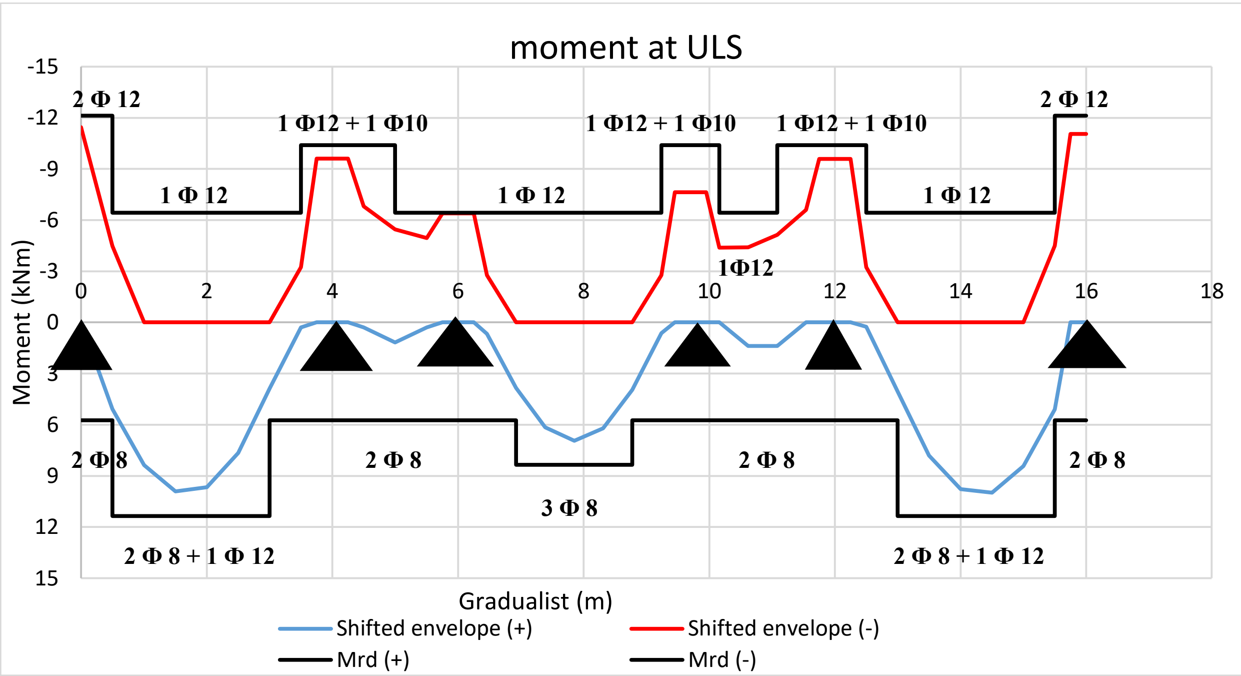


Figure 3.8. Recapitulative curve of the bending moment verification of the rib

For the transversal reinforcement, considering a diameter of 6 mm the design procedure presented on the section 2.4.2.4.c permits to obtain the spacing of the stirrups necessary to resist to the envelope of the shear solicitations. Figure 3.9 presents a recapitulative of these stirrups spacing along the rib.

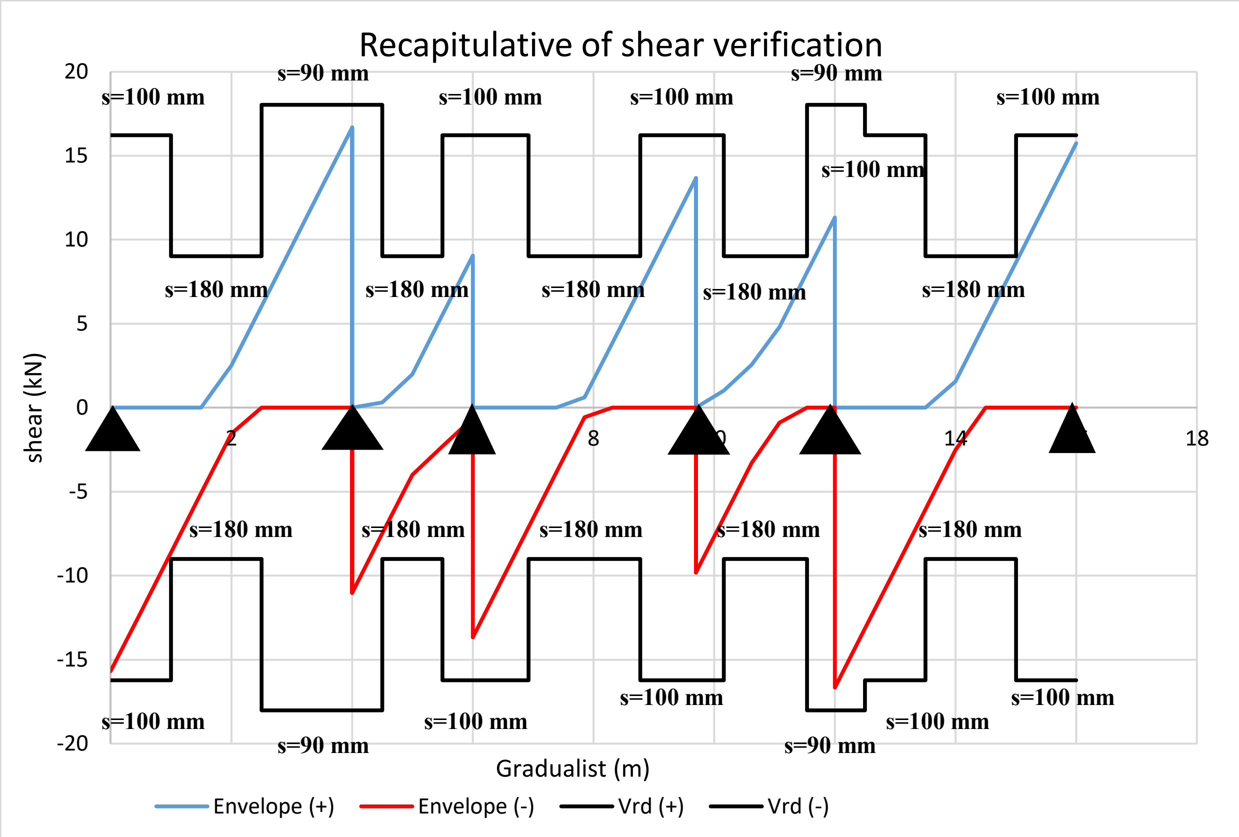


Figure 3.9. Recapitulative curve of the shear verification of the rib

3.3.1.6. Service Limit States (SLS)

The envelope curve of bending moment is given in the figure 3.10.

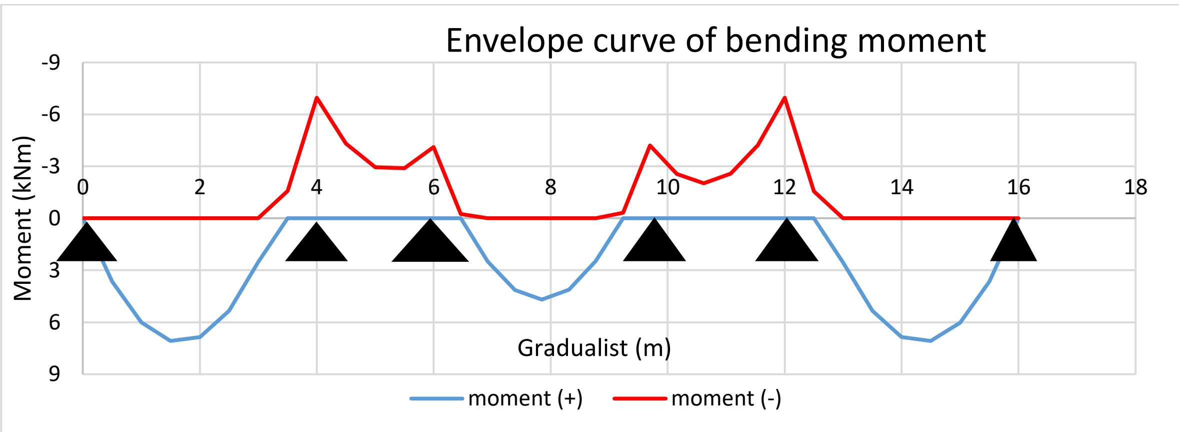
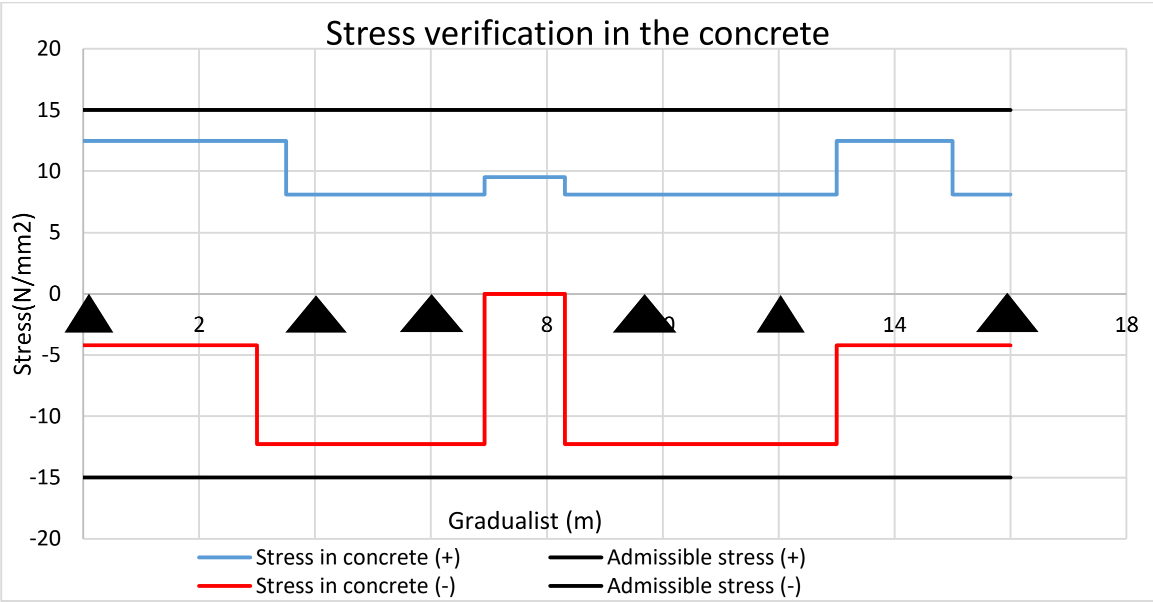
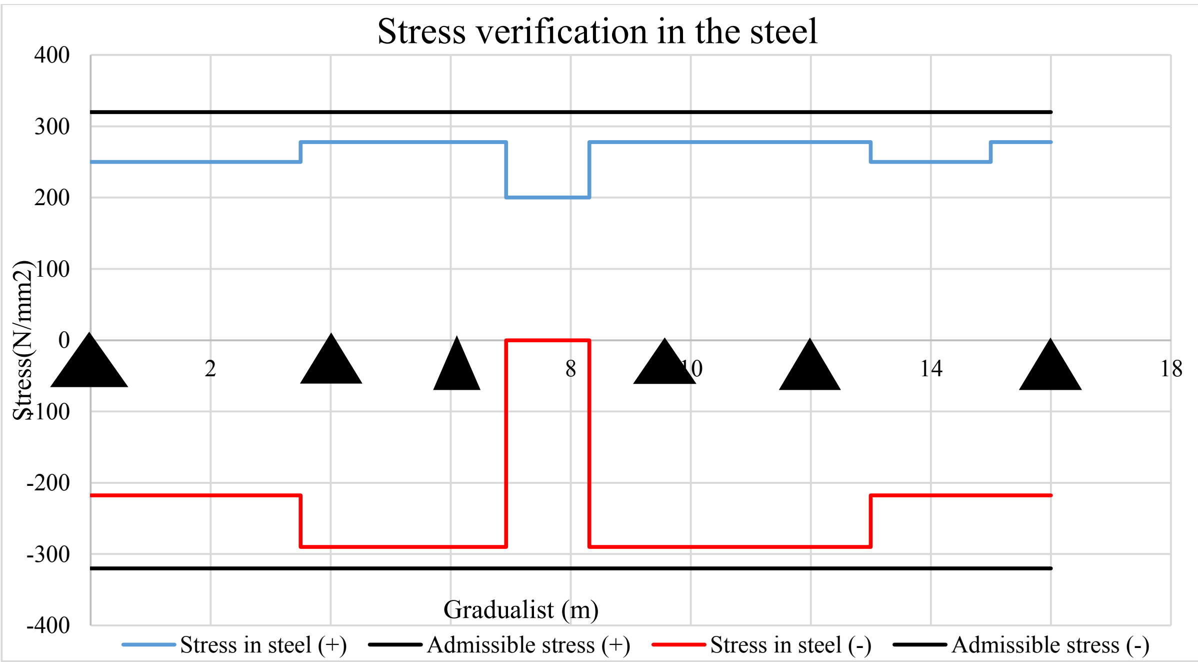


Figure 3.10. Envelope curve of bending moment.

With this envelope curve for bending moment at serviceability limit state the stress in the concrete and in the reinforcement are obtained using the equations 2.73 and 2.74. The limit value on the stress is evaluated from the equations 2.75 and 2.76 using the recommended values of the Eurocode 2, means taking $k_1 = 0.6$ and $k_3 = 0.8$. Figure 3.11 shows a comparison of the stress inside the concrete and the steel reinforcement to the admissible stress.



(a) in concrete

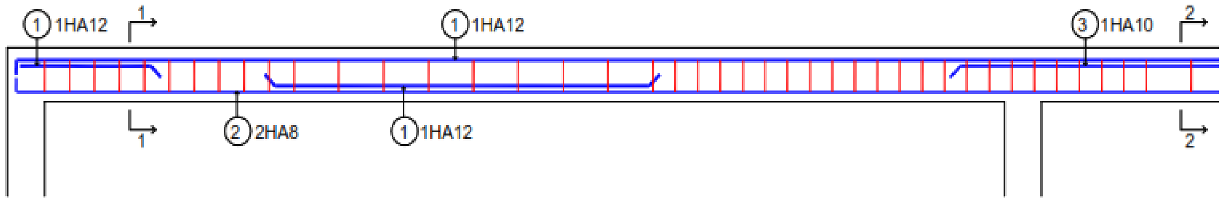


(b) in steel

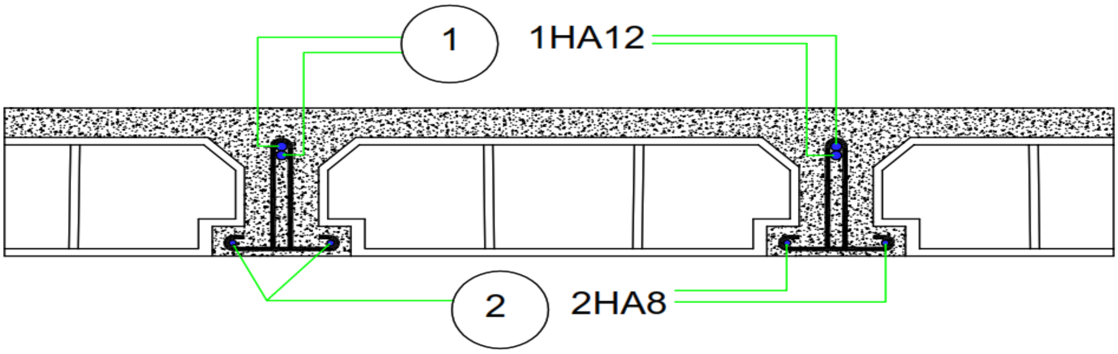
Figure 3.11. Recapitulative curve of stress verification of the beam: (a) in the concrete; (b) in the steel

The figure 3.12 shows the distribution of the steel bars reinforcement in the rib.

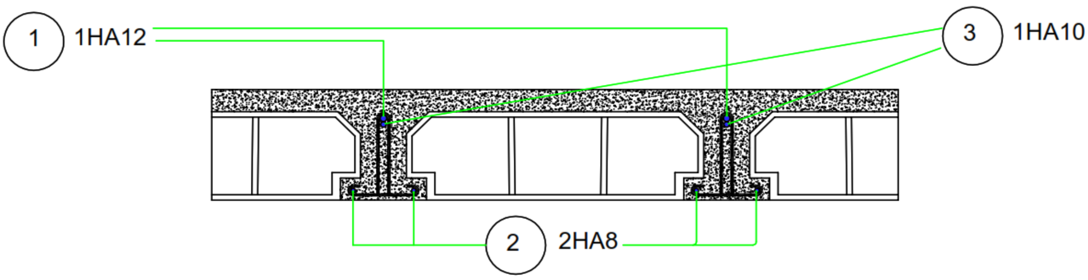
Span 1



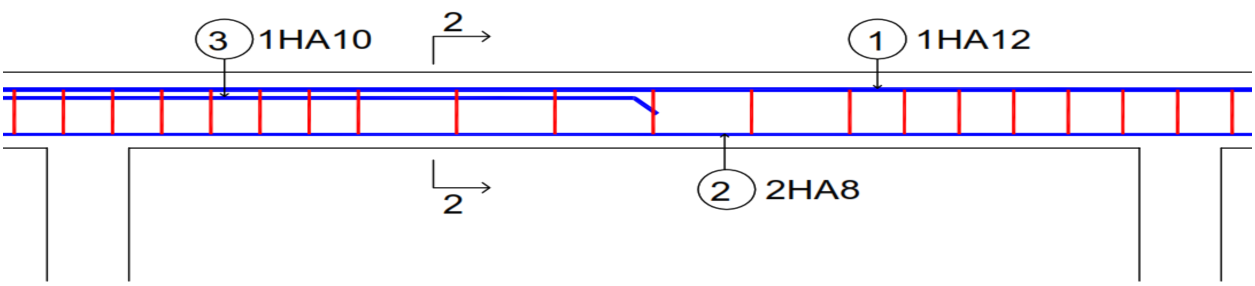
1 - 1



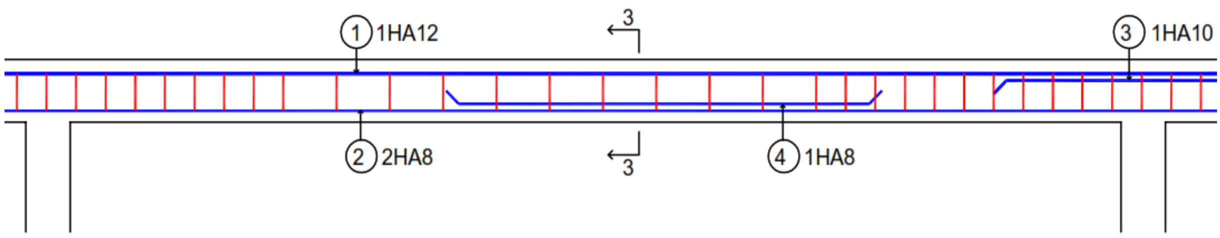
2 - 2

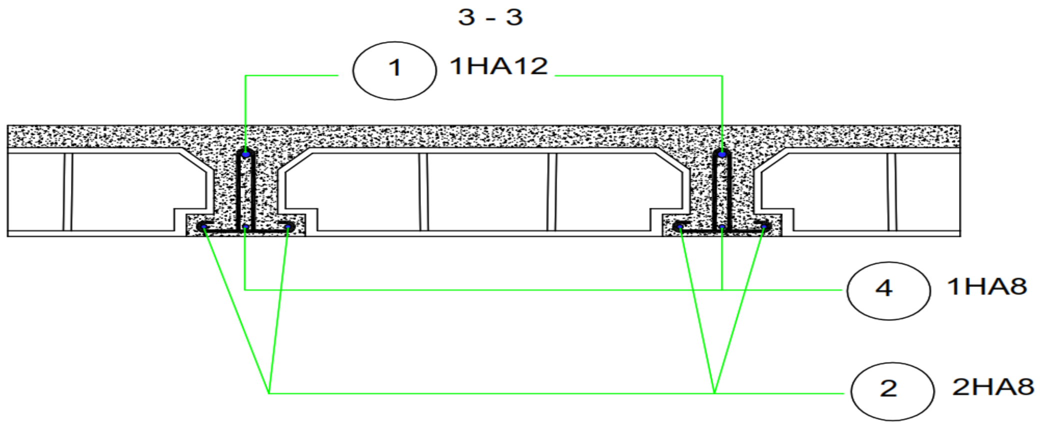


Span 2

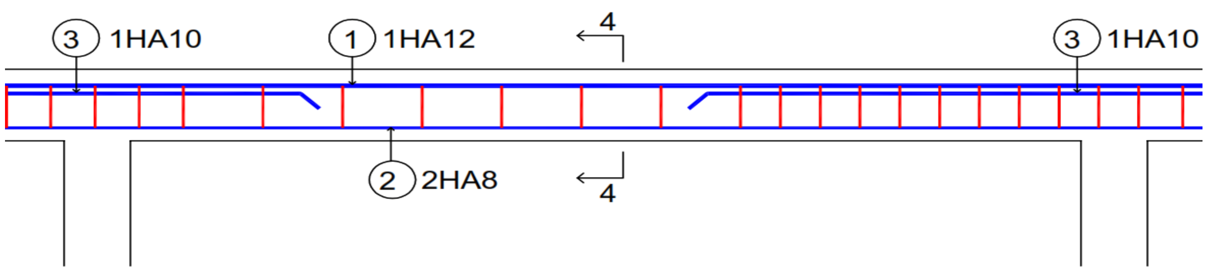


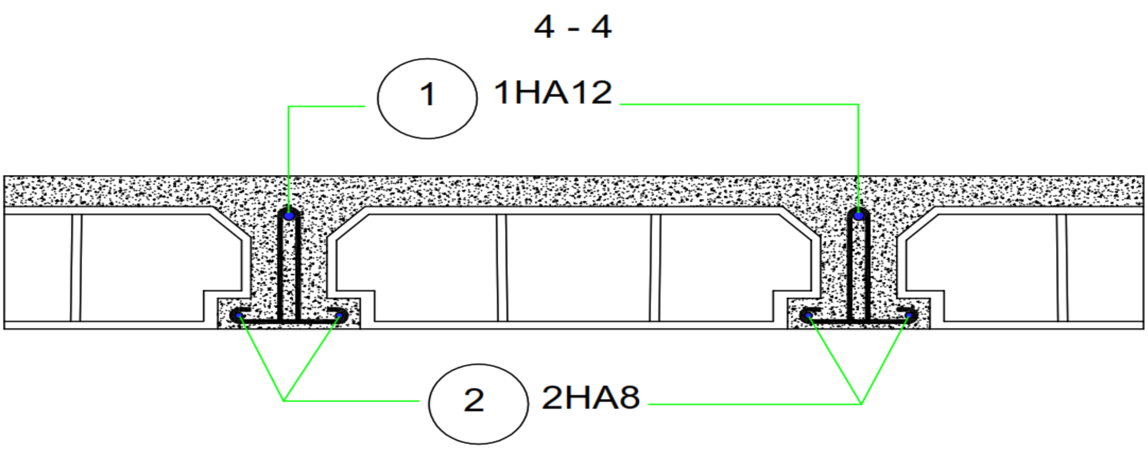
Span 3


1 1HA12
3
3 1HA10
2 2HA8
3
4 1HA8


3 - 3
1 1HA12
4 1HA8
2 2HA8

Span 4


3 1HA10
1 1HA12
4
3 1HA10
2 2HA8
4


4 - 4
1 1HA12
2 2HA8

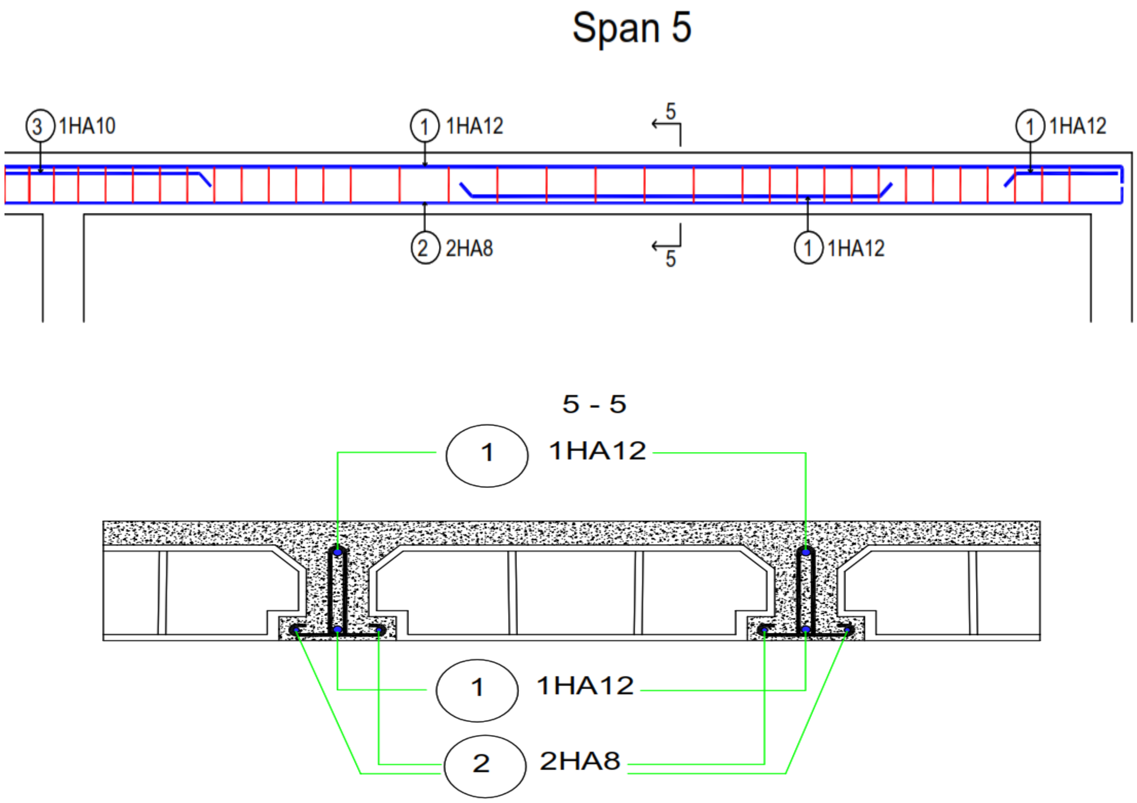


Figure 3.12. The distribution of the steel bars reinforcement in the rib

The figure 3.13 shows the steel bars reinforcement inside the compression table of the slab

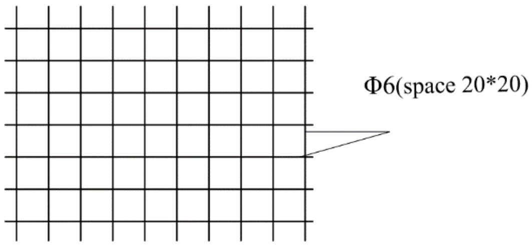


Figure 3.13. The steel bars reinforcement inside the compression table of the slab

The figure 3.14 shows the distribution of ribs. The base of rib is 15cm and distance between two ribs is 60cm.

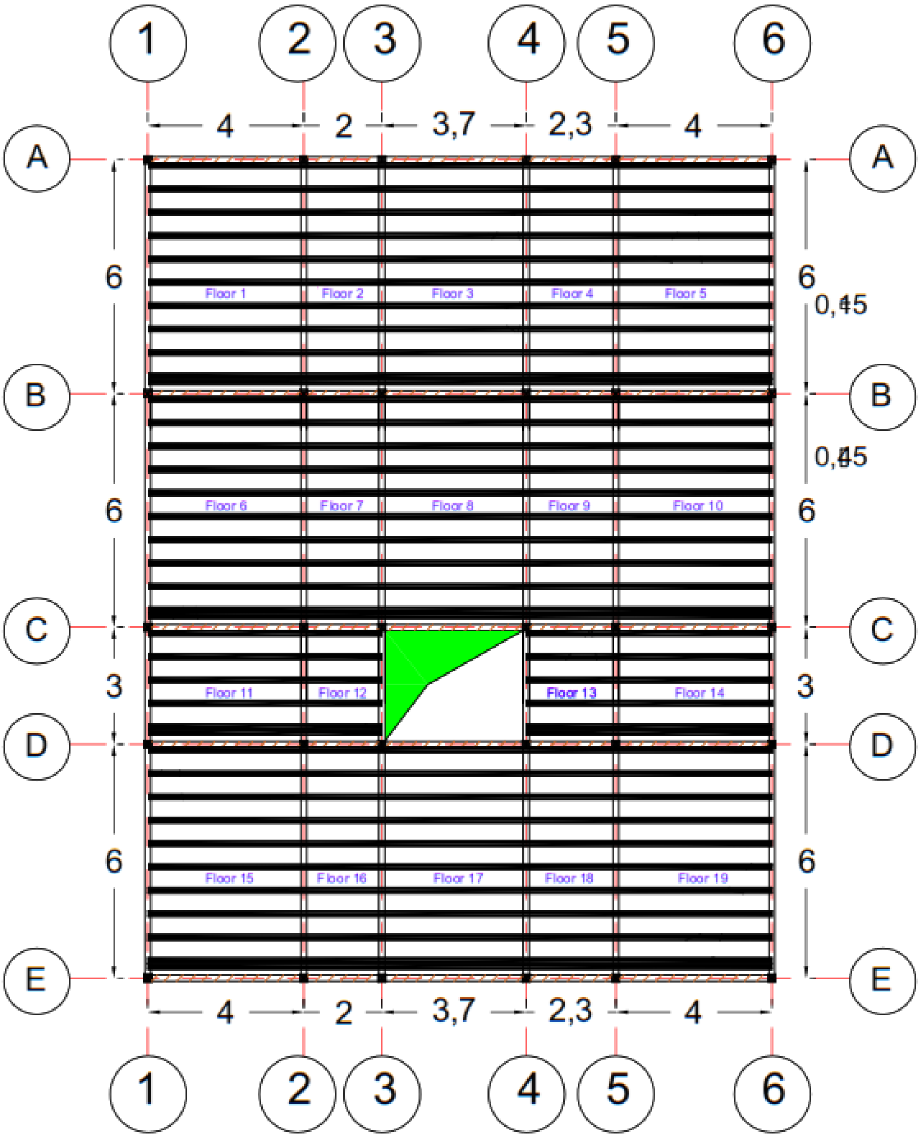


Figure 3.14. The distribution of ribs

3.3.2. Two way floor

Using the equation 2.6 the thickness of the floor is 25cm. The figure 3.15 shows the section of two way floor.

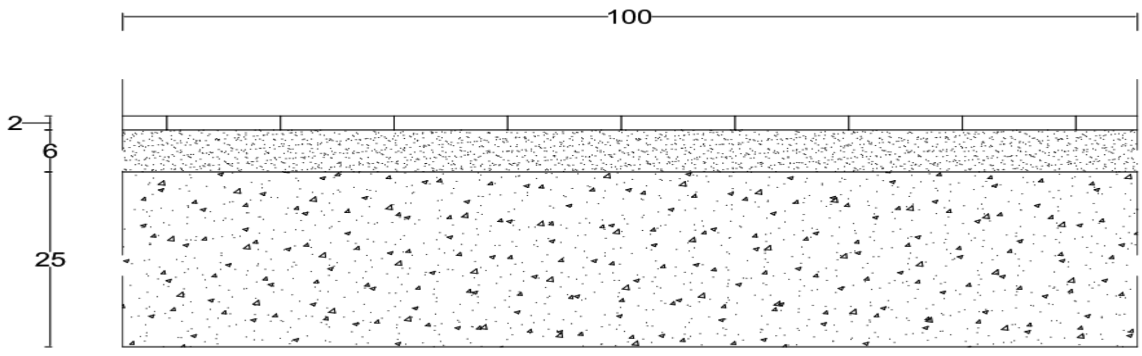


Figure 3.15. Two way floor section

3.3.2.1. Definition of the loads

The Different loads applied on our structure are shown in the table 3.5.

Table 3.5. Designation of the load

Designation	Symbols	Values (kN/m²)
Two way slab	g_1	25×0.25= 6.25
Ceramic stoneware Tiles(2 cm)	g_2	0.22 × 2 = 0.44
Screed	g_2	0.22 × 6 = 1.32
Imposed loads	q	3

To determine the linear loads in a two way slab, the Grashof method is used. The influence line is 1m.

3.3.2.2. Grashof principle

The figure 3.16 shows the plan view and the slab bands to design.

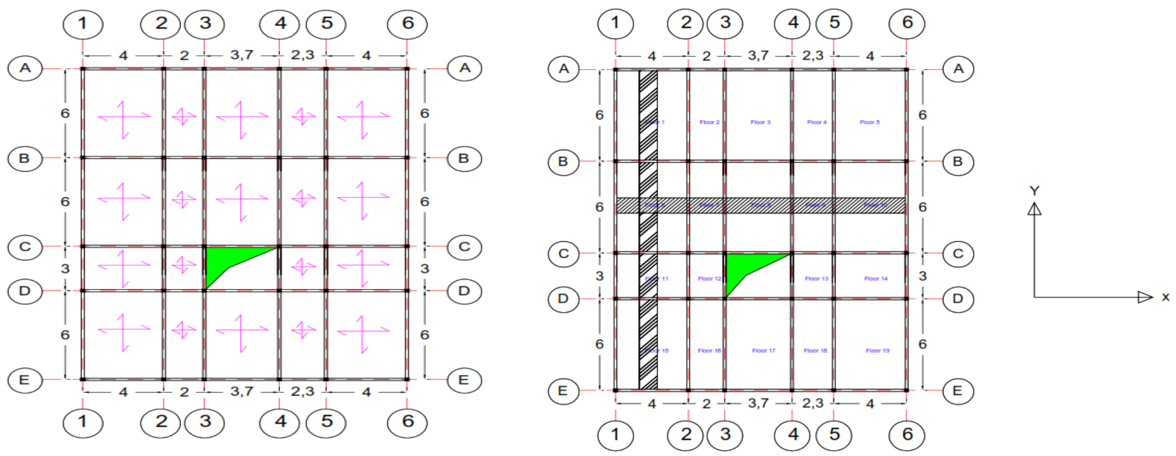


Figure 3.16. Two way floor

The table 3.6 and 3.7 shows the Grashof coefficients.

Table 3.6. The Grashof coefficients in X direction

Panels	Grashof coefficients
Panel 1	0.84
Panel 2	0.99
Panel 3	0.88
Panel 4	0.98
Maximum value	0.99

Table 3.7. The Grashof coefficients in Y direction

Panels	Grashof coefficients
Panel 1	0.17
Panel 5	0.76
Maximum value	0.76

Table 3.8 and 3.9 shows the loads.

Table 3.8. Loads of design in X direction

Designation	Value (kN/m)
Self-weight of the slab	6.19
Non-structural permanent loads	1.75
Variable loads	2.97

Table 3.9. Loads of design in Y direction

Designation	Value (kN/m)
Self-weight of the slab	4.75
Non-structural permanent loads	1.34
Variable loads	2.28

a) Design in the X direction

We will use the same combinations like in 3.3.1.3.

i) Ultimate limit state design

The envelope curve of bending moment is given in the figure 3.17.

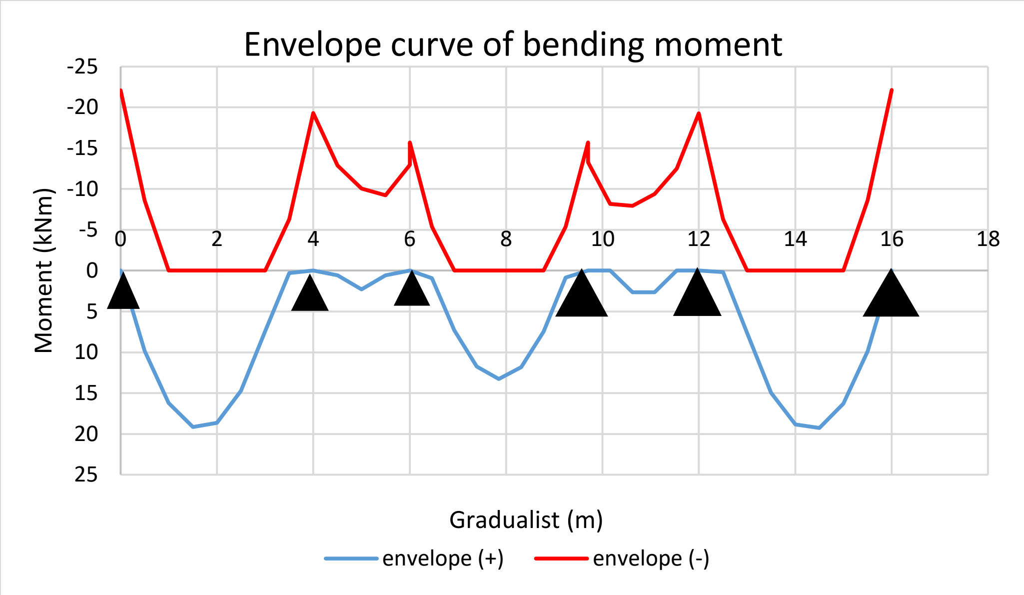


Figure 3.17. Envelope curve of bending moment

From the envelope curve of the bending moment presented in figure 3.16, we apply the procedure detailed in the section 2.4.2.4 to obtain the final solicitation curve presented in the figure 3.18.

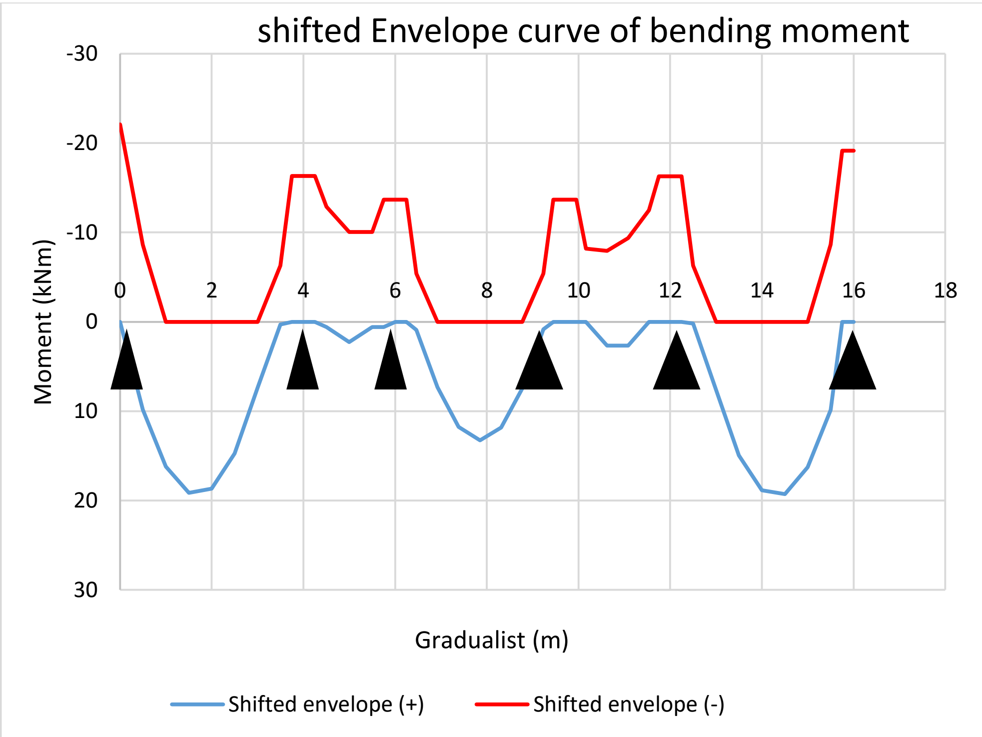


Figure 3.18. Shifted envelope curve of bending moment

ii) Calculation of the steel bars reinforcement

The steel reinforcement is evaluated using the equation 2.60 and the section obtained is verified for the detailing of member presented in the equations 2.61 and 2.62. At the end, the steel section is evaluated and verified and the results obtained is presented in the figure 3.19.

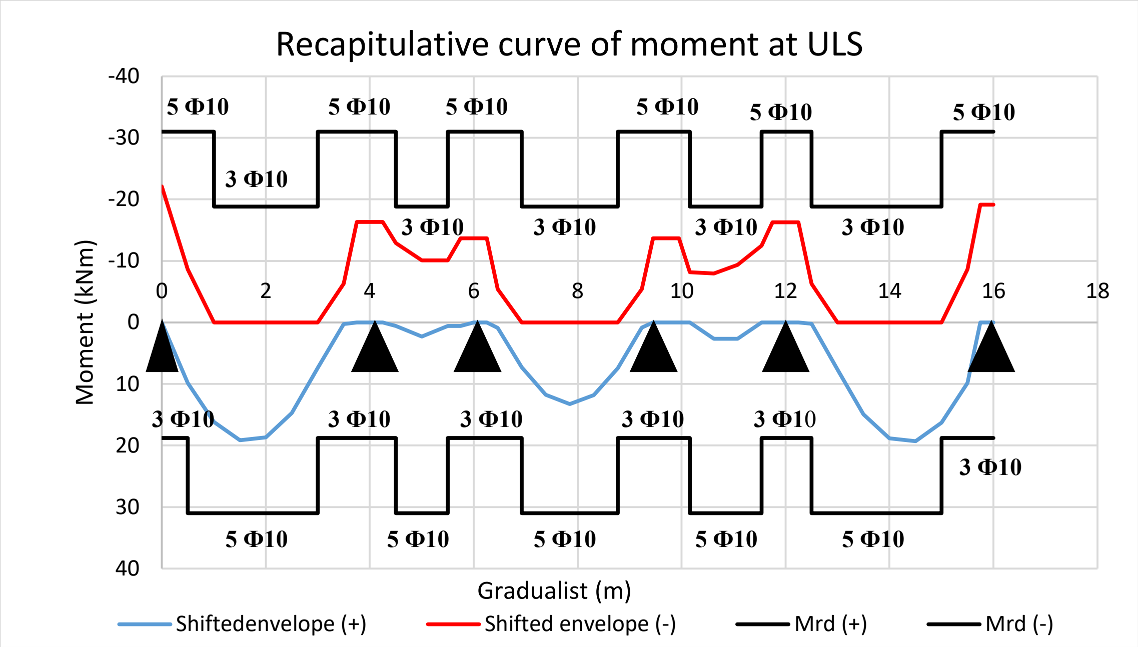


Figure 3.19. Recapitulative curve of the bending moment verification of the slab band in x direction

iii) Service Limit State (SLS)

The figure 3.20 shows the envelope curve of bending moment.

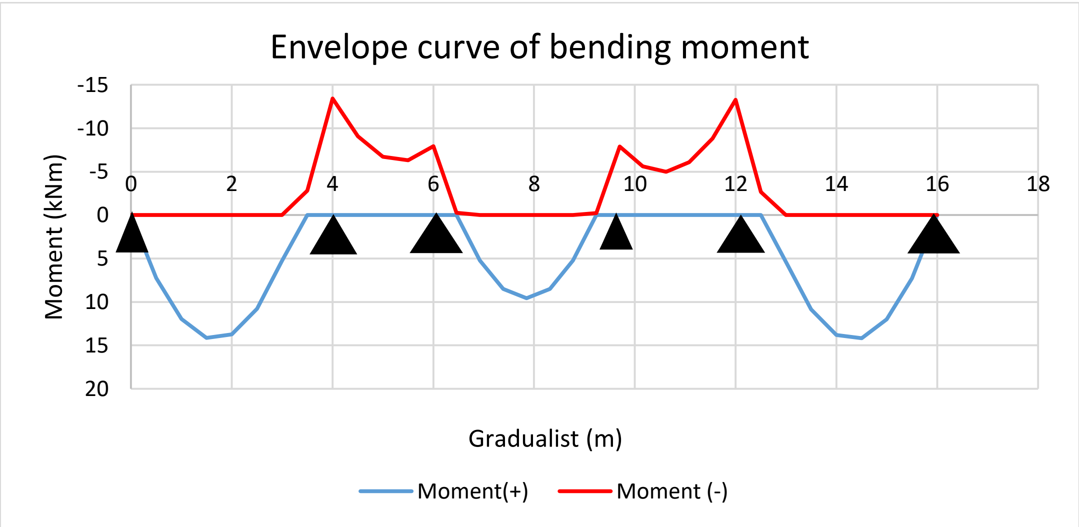
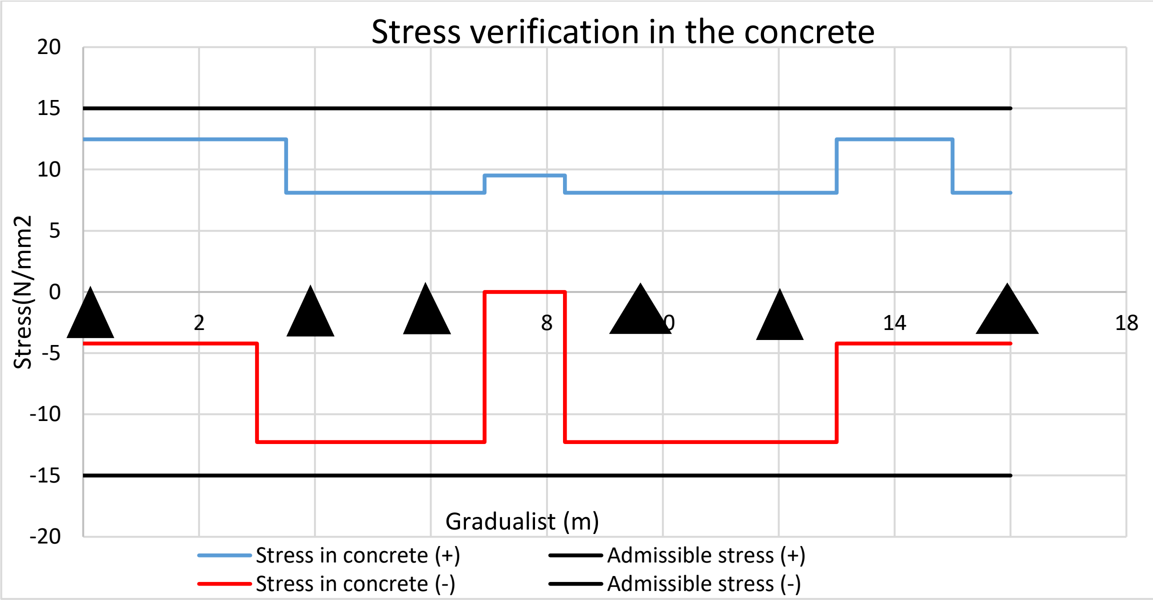
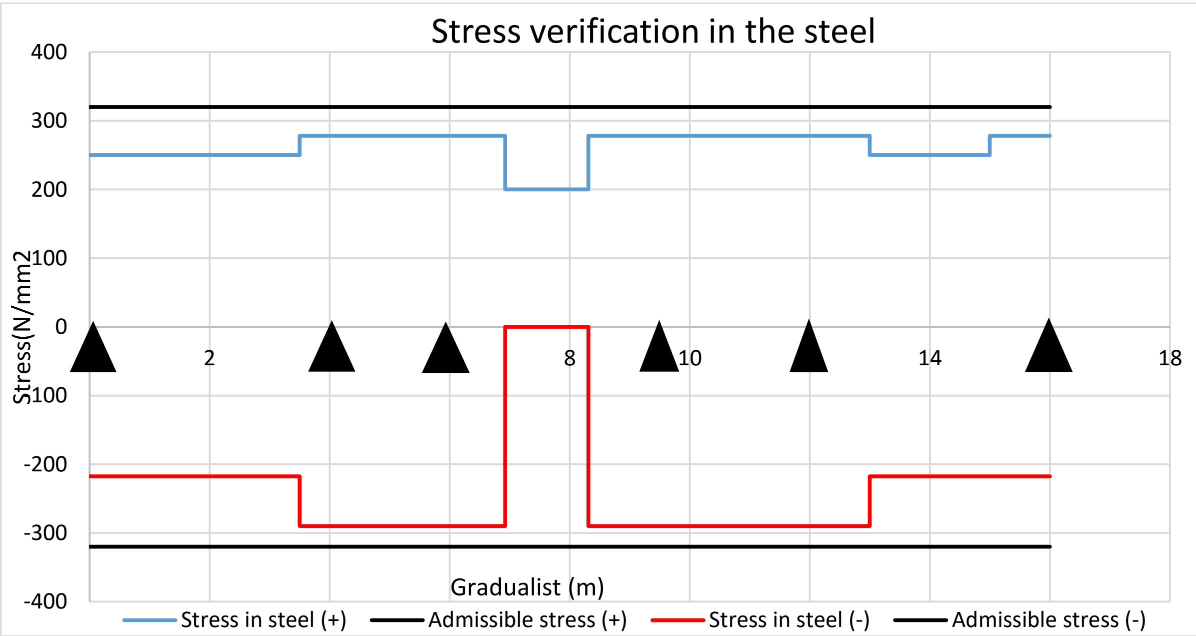


Figure 3.20. Envelope curve of bending moment

With this envelope curve for bending moment at serviceability limit state the stress in the concrete and in the reinforcement are obtained using the equations 2.73 and 2.74. The limit value on the stress is evaluated from the equations 2.75 and 2.76 using the recommended values of the Eurocode 2, means taking $k_1 = 0.6$ and $k_3 = 0.8$. Figure 3.21 shows a comparison of the stress inside the concrete and the steel reinforcement to the admissible stress.



(a) in concrete



(b) in steel

Figure 3.21. Recapitulative curve of stress verification of the beam: (a) in the concrete; (b) in the steel

b) Design in Y direction

i) Ultimate limit state design

The envelope curve of bending moment is given in the figure 3.22.

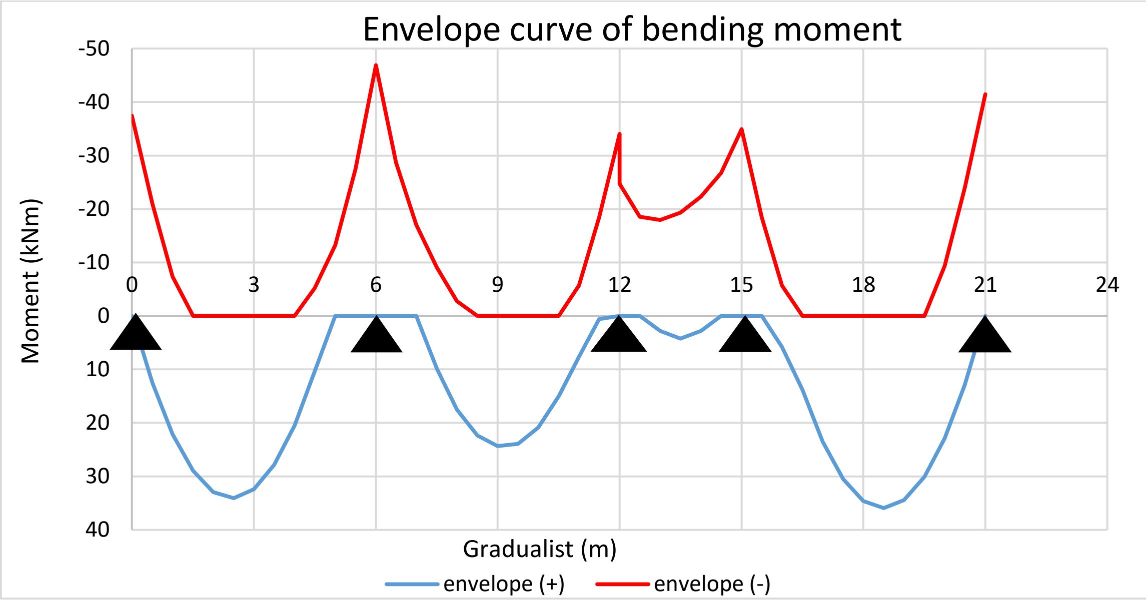


Figure 3.22. Envelope curve of bending moment

From the envelope curve of the bending moment presented in figure 3.21, we apply the procedure detailed in the section 2.4.2.4 to obtain the final solicitation curve presented in the figure 3.23.

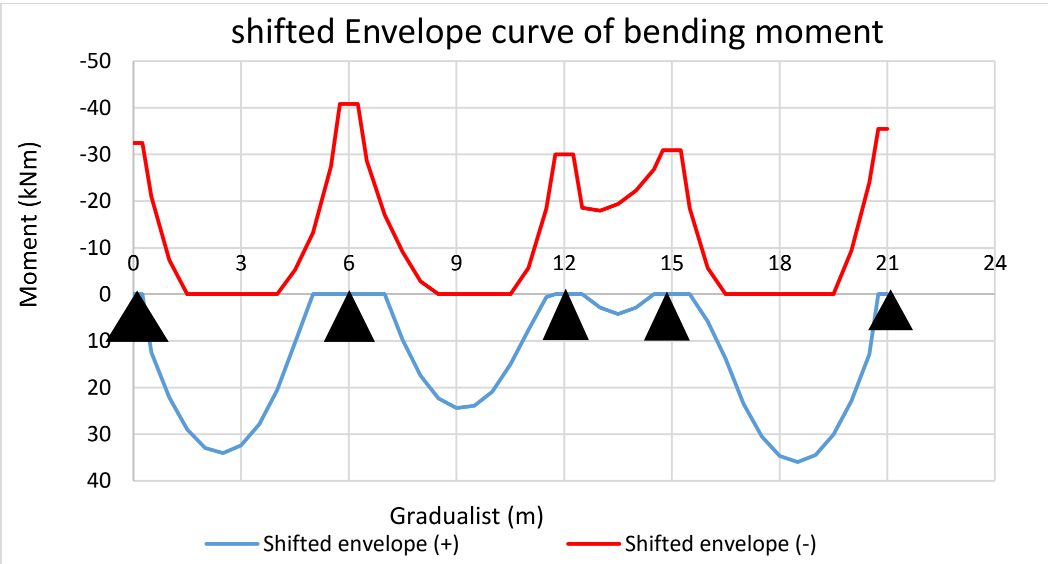


Figure 3.23. Shifted envelope curve of bending moment

ii) Calculation of the steel bars reinforcement

The steel reinforcement is evaluated using the equation 2.60 and the section obtained is verified for the detailing of member presented in the equations 2.61 and 2.62. At the end, the steel section is evaluated and verified and the results obtained is presented in the figure 3.24.

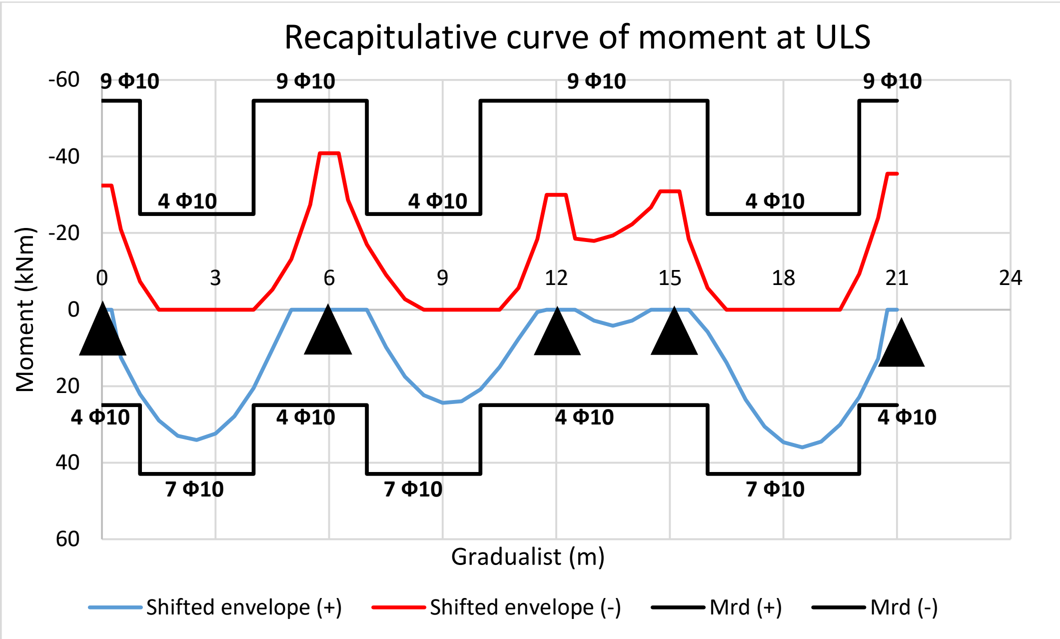
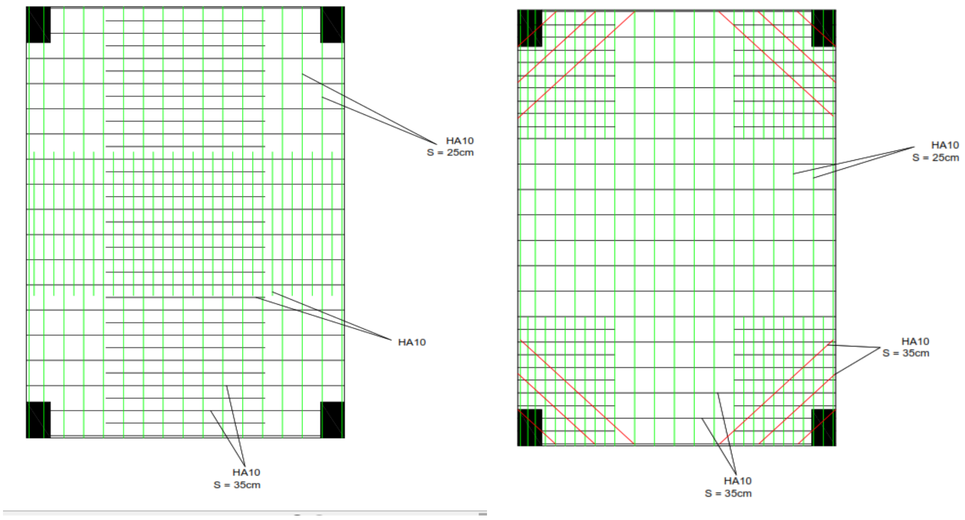


Figure 3.24. Recapitulative curve of the bending moment verification of the slab band in y direction

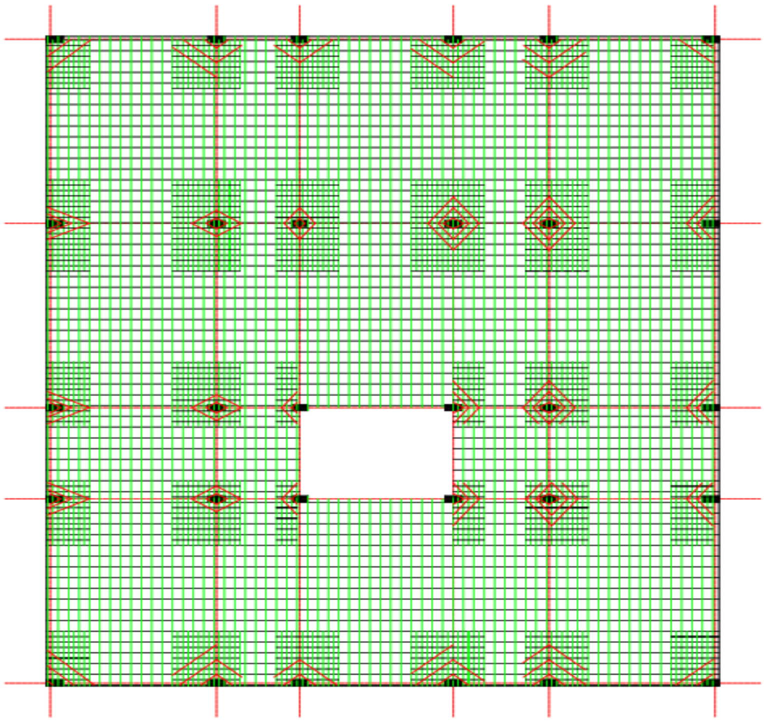
The figure 3.25 shows the distribution of the steel bars reinforcement in the floor 1.



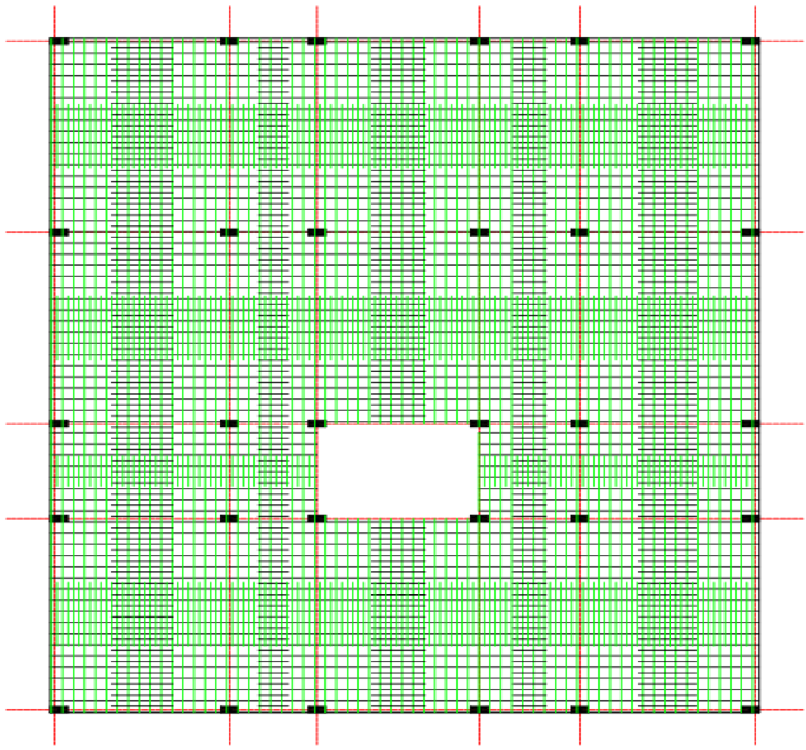
(a) lower layer (b) upper layer

Figure 3.25. The distribution of the steel bars reinforcement in lower layer (a) and upper layer (b) of floor 1.

The figure 3.26 shows the distribution of the steel bars reinforcement inside all the floor.



(a) upper layer



(b) lower layer

Figure 3.26. The distribution of the steel bars reinforcement in upper layer (a) and lower layer (b) of floor.

3.3.3. Wood-concrete floor

3.3.3.1. Material properties

Recomposed wooden beam (parallam) is the species chosen for this work. The table 3.10 shows the characteristic of the materials.

Table 3.10 Characteristics of the materials.

Materials property	Symbols	Values	References
Concrete slab			
width (influence length)	b_1	500 mm	
thickness	h_1	40 mm	
cross section area	A_1	20000 mm	
inertia	I_1	2666666,67 mm^4	
modulus of elasticity	E_1	31000 MPa	[EN 1992-1-1] Tab. 3.1
characteristic compressive cylinder strenght	f_{ck}	25 MPa	[EN 1992-1-1] Tab. 3.1
characteristic axial tensile strenght	f_{ct}	1.8 MPa	[EN 1992-1-1] Tab. 3.1
partial factor for concrete	γc	1.5	[EN 1992-1-1] Tab. 2.1
density	$\gamma concrete$	25 kN/m^3	[EN 1991-1-1] Tab. A.1
distance between connectors	s	100 mm	
Timber joist			
Grade class	**parallam**		
width	b_2	100 mm	
thickness	h_2	250 mm	
cross section area	A_2	25000 mm^2	
inertia	I_2	13020833,3 mm^4	
modulus of elasticity (E_{cm})	E_2	14000 MPa	([NF EN 338])Tab.2.1 (2009)
characteristic bending strenght	$f_{m,k}$	40 N/mm^2	([NF EN 338])Tab.2.1 (2009)
characteristic tensile strenght ($f_{t,0,k}$)	$f_{t,0,k}$	24 N/mm^2	([NF EN 338])Tab.2.1 (2009)
characteristic shear strenght ($f_{v,k}$)	$f_{v,k}$	2.4 N/mm^2	([NF EN 338])Tab.2.1 (2009)
partial factor for timber (γ_M)	γ_M	1.3	([EN 1995-1-1])tab.2.3
service class		1	
load duration class		medium term	
modification factor	k_{mod}	0.8	([EN 1995-1-1])Tab.3.1
deformation factor(k_{def})	k_{def}	0.6	([EN 1995-1-1])Tab.3.2
density	ρ	767 kg/m^3	([NF EN 338-1-1])Tab.3.3
Connectors properties (steel rebars)			
diameter	Ø	12 mm	
slip modulus at SLS	k_{ser}	6000 N/mm	experimental test (CSTC report n°13-2010)
slip modulus at ULS	ku=2/3 . kser	4000 N/mm	experimental test (CSTC report n°13-2010)
Charateristic resistance	P_{Rk}	10000 N	experimental test (CSTC report n°13-2010)

The figure 3.27 shows the cross- section of the composite slab and its dimensions. The total length of connector is 14cm, embedded to 9cm inside wood.

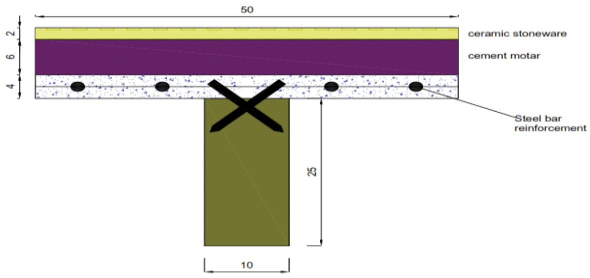


Figure 3.27. Timber-concrete slab cross-section

3.3.3.2. Loads, characteristics value of load

The table 3.11 shows the loads.

Table 3.11. Loads, characteristics value of load

Loads calculations values		
Names	**Values**	**Units**
self-weigth of timber	0.25	kN/m
Self-weigth of concrete slab	0.5	kN/m
dead load	0.88	kN/m
variable load	1.5	kN/m
characteristic combination loads		
F_{ELU}	4.37	kN/m
$F_{ELS,quasi}$	2.078	kN/m
$F_{ELS,rare}$	3.128	kN/m

The curve of bending moment is given in the figure 3.28.

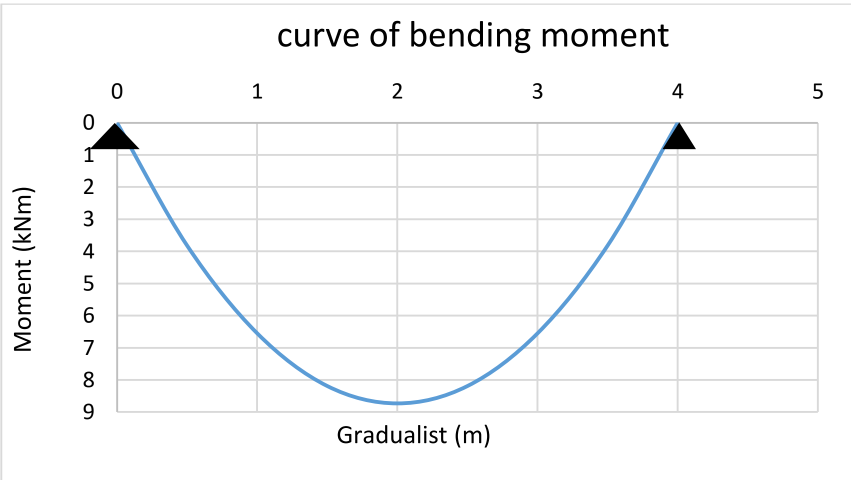


Figure 3.28. Curve of bending moment

The curve of shear is given in the figure 3.29.

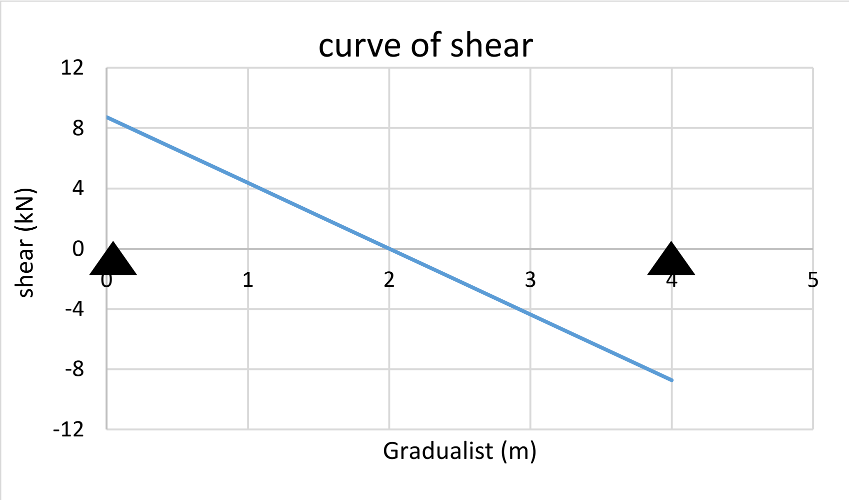


Figure 3.29. Curve of shear

3.3.3.3. ULS Analysis

Here we verify the stress in the concrete, in the wood, the shear stress in the timber and we verify the load bearing capacity of fasteners. The analysis was done in short and long term analysis.

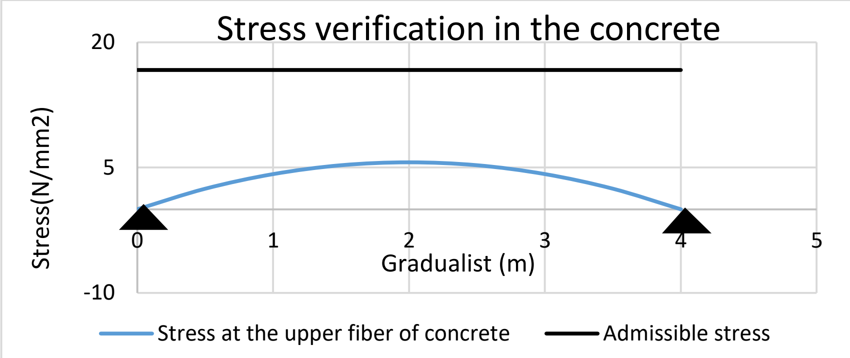
a) Short-term analysis

The table 3.12 shows the equivalent rigidity.

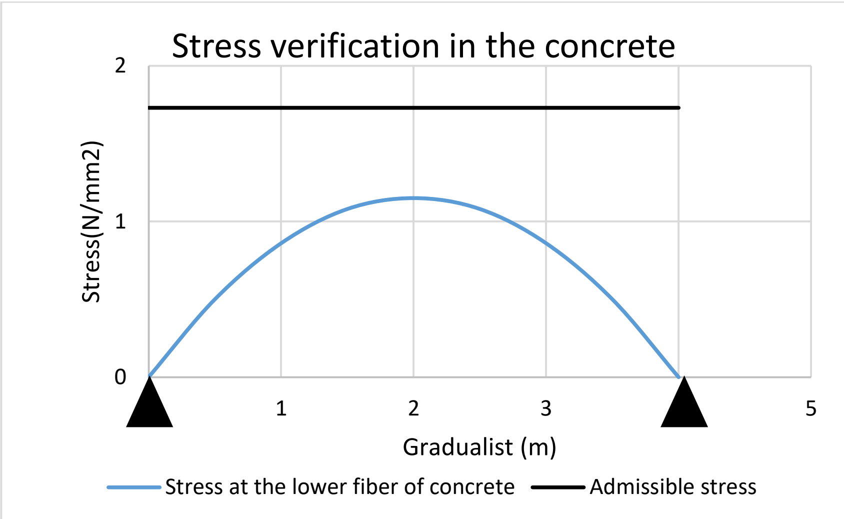
Table 3.12. Equivalent rigidity

Notations	Values	Units
γ_1	0.09	
γ_2	1	
a_2	23.41	mm
a_1	139.59	mm
$(EI)_{eff}$	1.60×10^{12}	Nmm^2

The figure 3.30 shows the stress verification in the concrete at short term.



(a) upper fiber



(b) lower fiber

Figure 3.30. Verification of the stress in upper fiber (a) and lower fiber (b) of concrete at short term

The figure 3.31 shows the stress verification in the wood at short term.

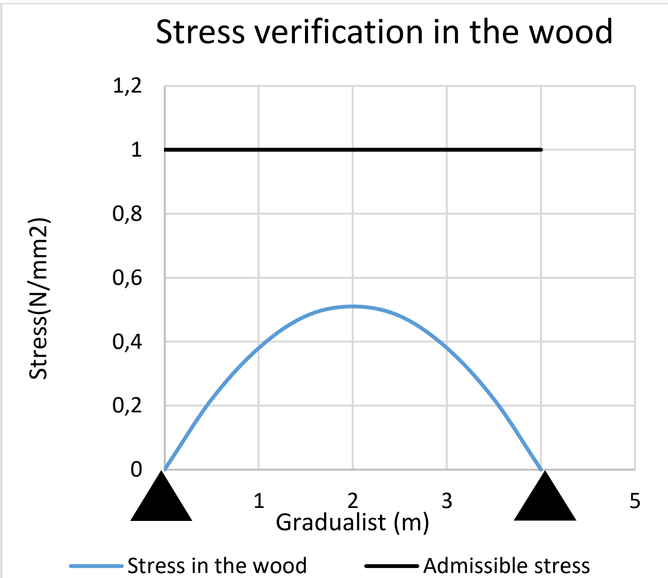


Figure 3.31. Verification of the stress in the wood at short term

b) Checking the loading rate in the short term stud

The figure 3.32 shows the verification of the stud at short term.

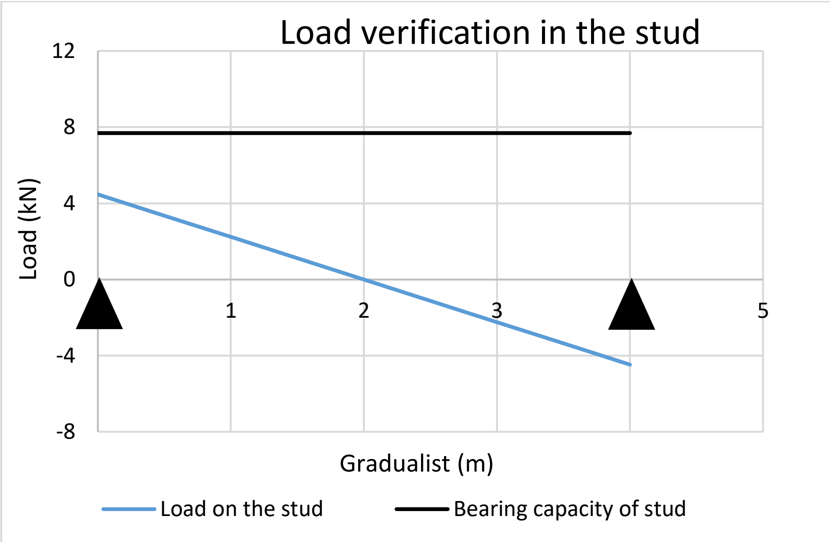


Figure 3. 32. Verification of the stud at short term

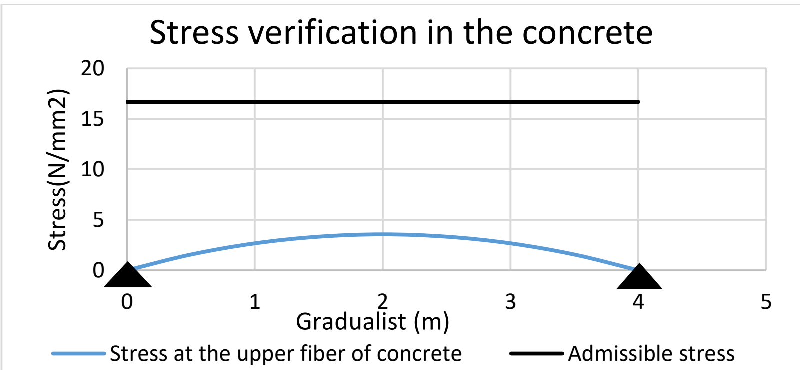
c) Long-term analysis

The table 3.13 shows the equivalent rigidity.

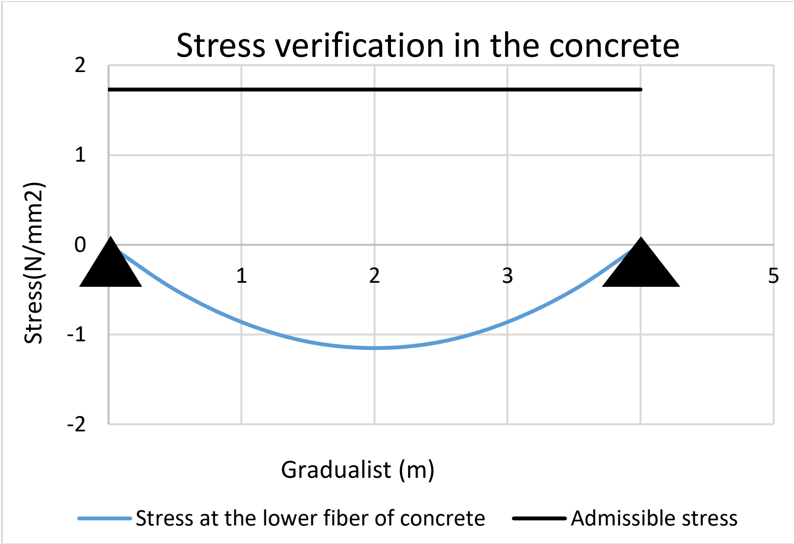
Table 3.13. Equivalent rigidity

Notations	Values	Units
$E_{1,End}$	10333.33	N/mm^2
$E_{2,End}$	11864.41	N/mm^2
$K_{u,End}$	3669.72	N/mm
γ_1	0.29	
γ_2	1	
a_2	27.25	mm
a_1	135.75	mm
$(EI)_{eff}$	1.49971×10^{12}	Nmm^2

The figure 3.33 shows the stress verification in concrete at long term.



(a) upper fiber



(b) lower fiber

Figure 3.33. Verification of the stress in upper fiber (a) and lower fiber (b) of concrete at long term

The figure 3.34 shows the stress verification in the wood at long term.

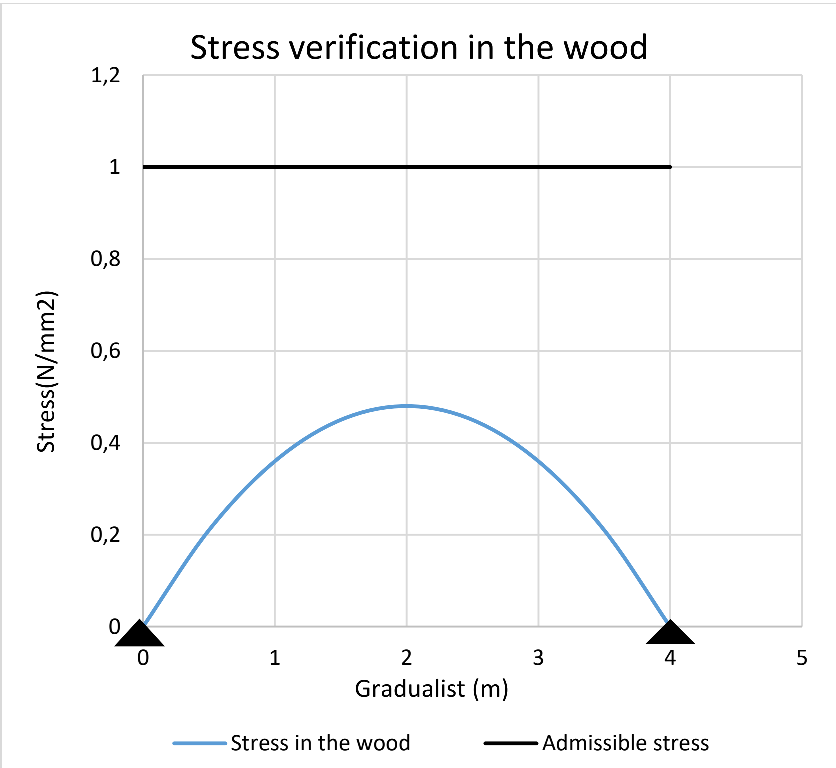


Figure 3.34. Verification of the stress in the wood at long term

The figure 3.35 shows the shear verification in the wood at short and long term.

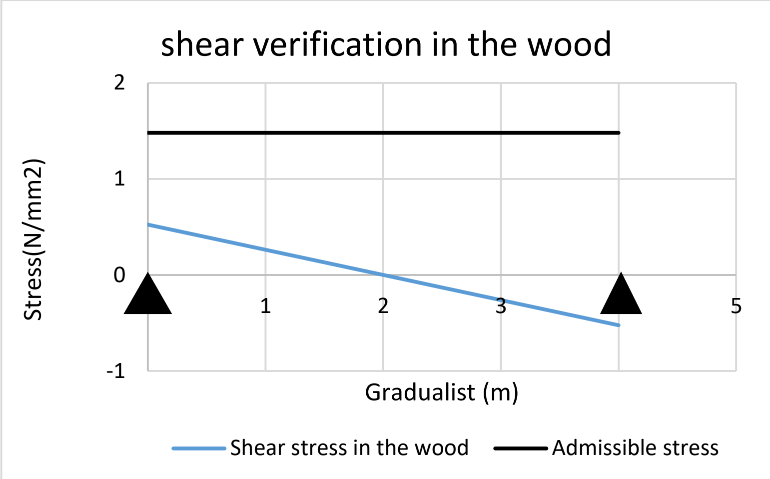


Figure 3.35. Shear verification in timber

d) Checking the loading rate in the long term connectors

The figure 3.36 shows the verification of the stud at long term.

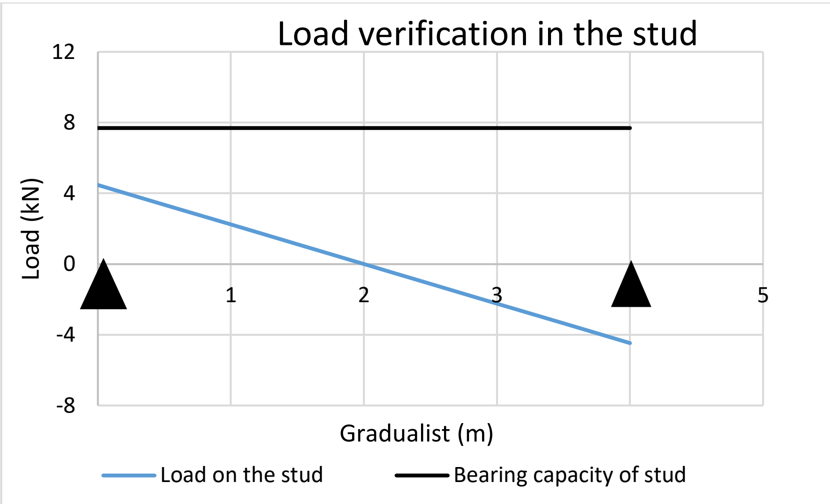


Figure 3.36. Verification of the stud at long term

3.3.3.4. SLS Analysis

As at ULS, the analysis was done in short and long term analysis.

a) Short-term

The table 3.14 shows the deflection of timber beam at short-term.

Table 3.14. Equivalent rigidity and deflection

Notations	Values	Units
γ_1	0.14	
γ_2	1	
a_2	31.59	mm
a_1	131.41	mm
$(EI)_{eff}$	2.06741×10^{12}	Nmm^2
f_{max}	5.04	mm
L/150	26.67	mm
L/500	8	mm
$f_{max} \leq L/150$ satisfy		
$f_{max} \leq L/500$ satisfy		

b) Long-term

The table 3.15 shows the deflection of timber beam at long-term.

Table 3.15. Equivalent rigidity and deflection

Notations	Values	Units
$E_{1,End}$	10333.33	N/mm^2
$E_{2,End}$	8750	N/mm^2
$K_{ser,End}$	4615.38	N/mm
γ_1	0.27	
γ_2	1	
a_2	32.74	mm
a_1	130.26	mm
$(EI)_{eff}$	1.30876	Nmm^2
f_{max}	7	mm
$f_{max} \leq L/150$ satisfy		
$f_{max} \leq L/500$ satisfy		

In conclusion the section of wood-concrete floor is verify.

The figure 3.37 shows the distribution of joist. The base of joist is 10cm and distance between two joits is 50cm.

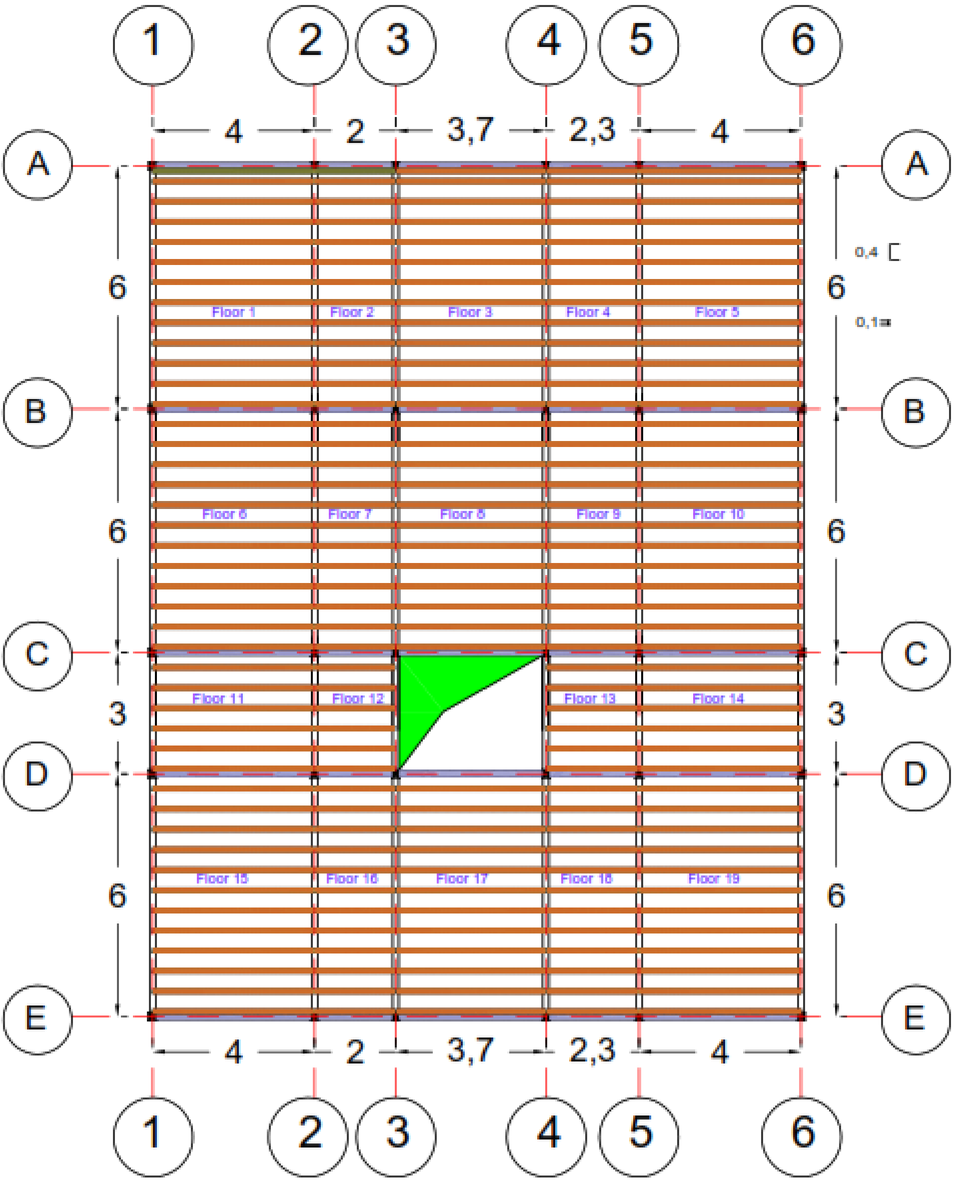


Figure 3.37. The distribution of joist

3.4. Design of the beams

Here only the beams in case of hollow block floor and wood-concrete floor will be design. Figure 3.38 shows the choice of the beam under study.

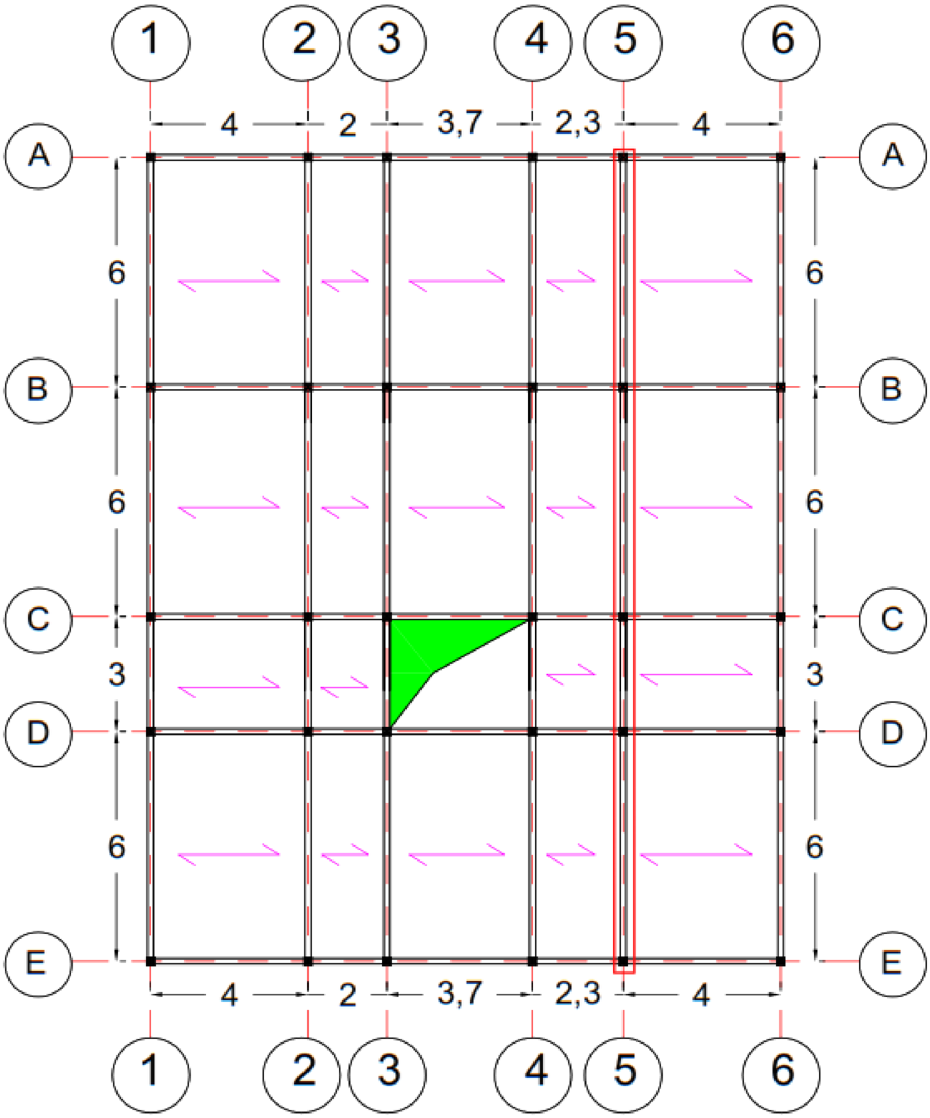


Figure 3.38. Selected beam for design

3.4.1. Case of hollow block floor

The preliminary section is obtained by assuming the value of the section height and the section depth. The section height is obtained from the maximum span using the equations 2.52 and 2.53. we will take H = 55 cm and b = 20 cm.

3.4.1.1. Definition of the loads

The Different loads applied on our structure are shown in the table 3.16.

Table 3.16. Designation of the loads

Designation	Symbols	Values (kN/m^2)
Permanent load	g_2	4.61
Imposed load	q	3

The weight of the wall is included in the loads designation W_{wall} = 7.69 kN/m. The influence area is 3,15m . The loads applied are calculated by the equation

$$Q = q \times i_a \quad \text{Eq. (3.2)}$$

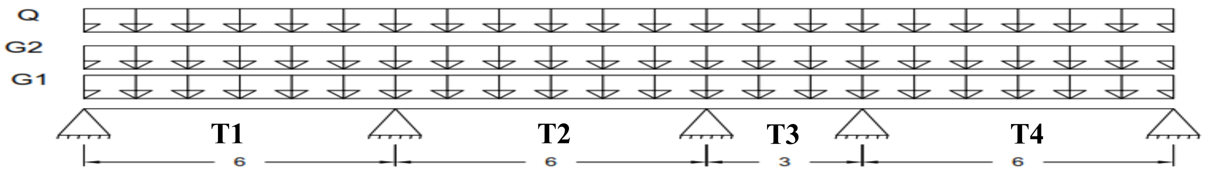
The values of the loads are shown in the table 3.17.

Table 3.17. Loads of design

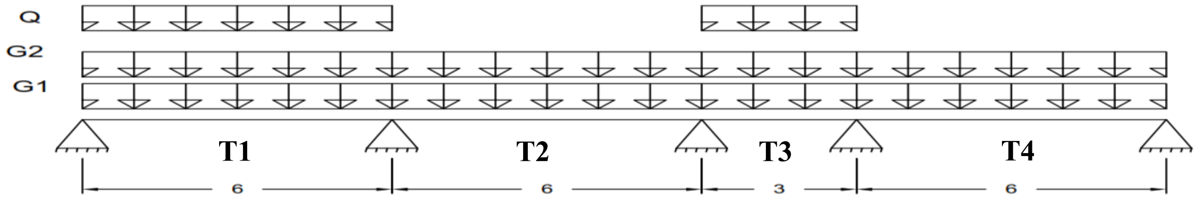
Designation	Value (kN/m)
Permanent load	22.22
Imposed load	9.45

3.4.1.2. Combinations

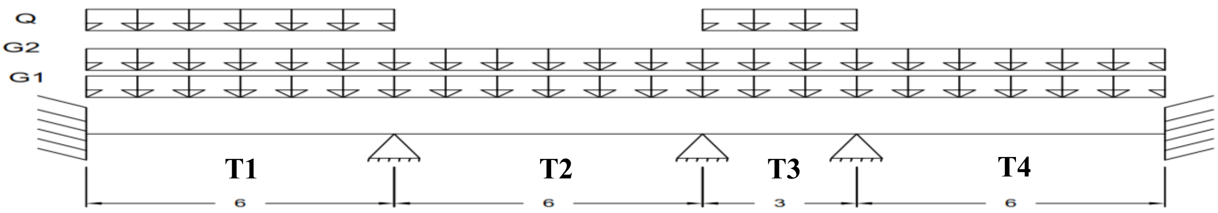
From this model, tree loads arrangements are defined for the design and are presented in the figure 3.39.



Load combination ULS T1T2T3T4



Load combination ULS T1T3



Load combination ULS T1T3 embedded

Figure 3.39. Load combination on the beam

3.4.1.3. Ultimate limit state design

The envelope curve of bending moment is given in the figure 3.40.

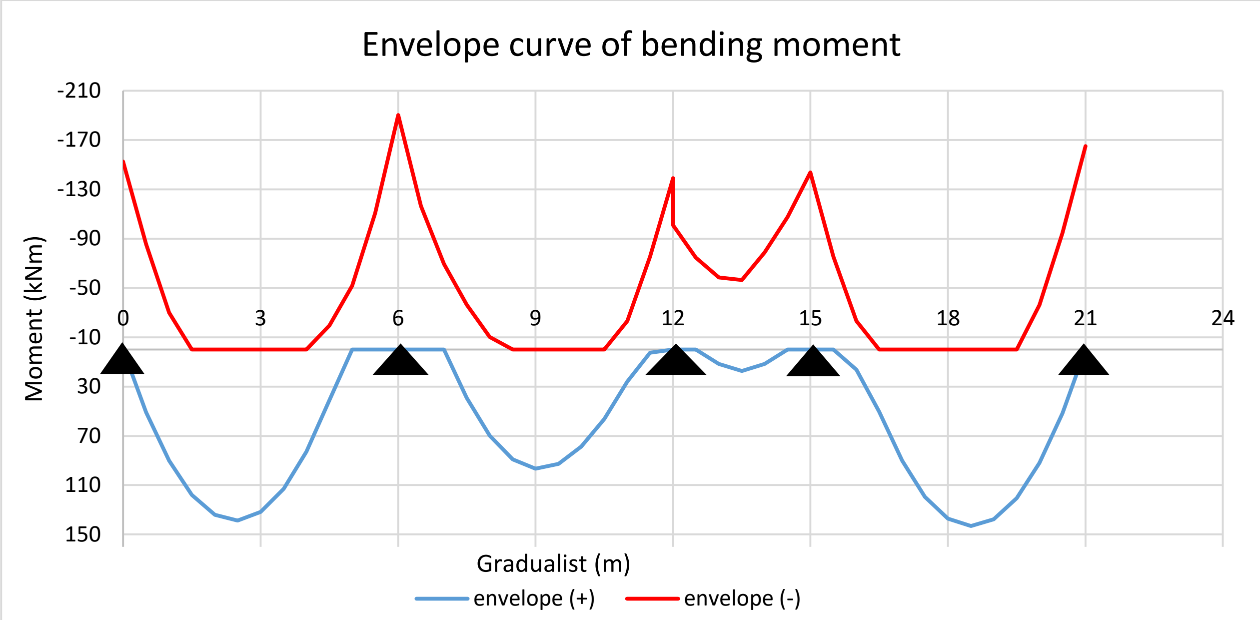


Figure 3.40. Envelope curve of bending moment

The envelope curve of shear is given in the figure 3.41.

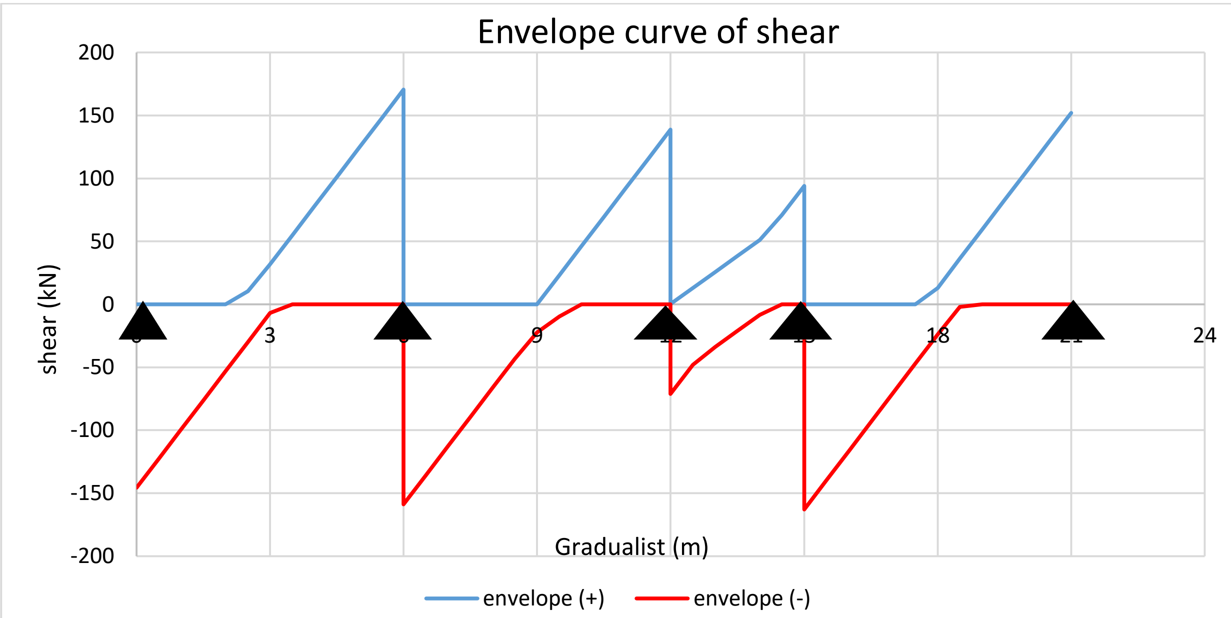


Figure 3.41. Envelope curve of shear

From the envelope curve of the bending moment presented in figure 3.41, we apply the procedure detailed in the section 2.4.2.4 to obtain the final solicitation curve presented in the figure 3.42.

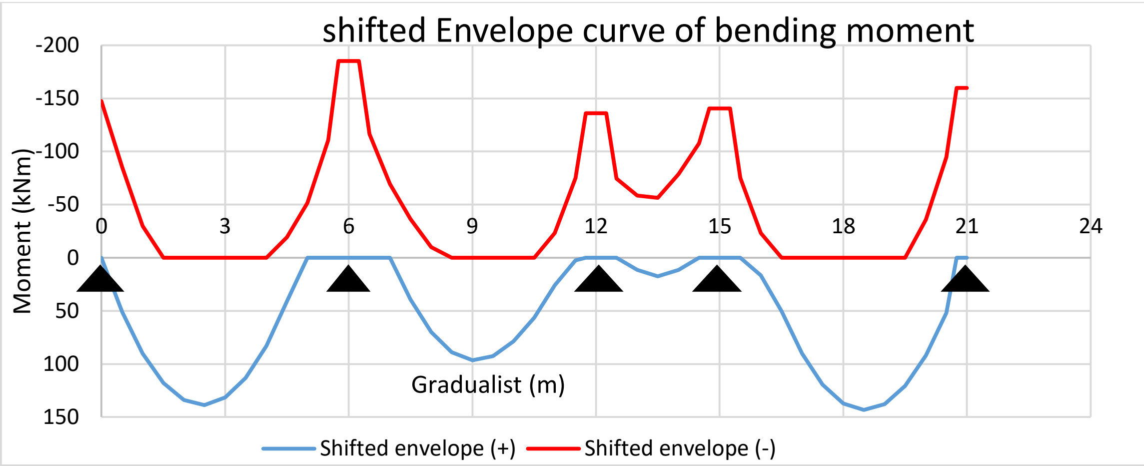


Figure 3.42. Shifted envelope curve of bending moment

3.4.1.4. Calculation of the steel bars reinforcement

The steel reinforcement is evaluated using the equation 2.60 and the section obtained is verified for the detailing of member presented in the equations 2.61 and 2.62. At the end, the steel section is evaluated and verified for a beam section of 20cm×55cm and the results obtained is presented in the figure 3.43.

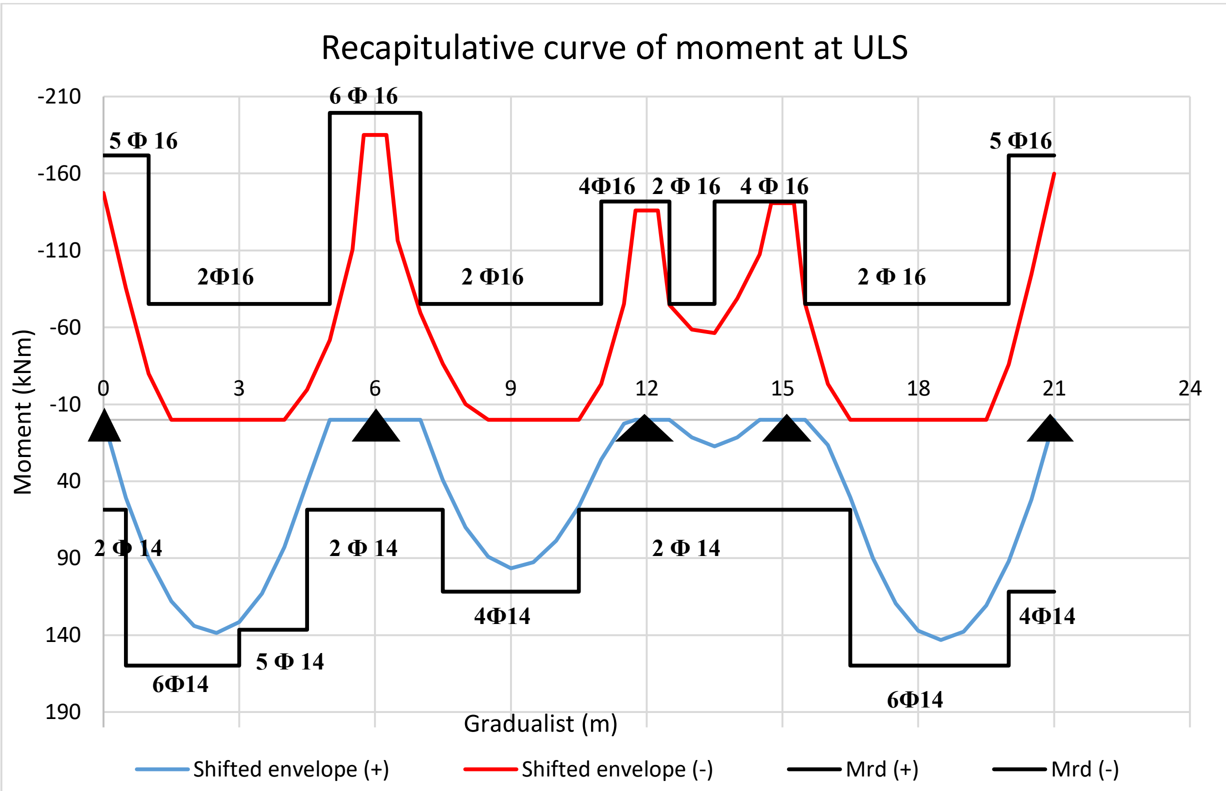


Figure 3.43. Recapitulative curve of the bending moment verification of the beam

For the transversal reinforcement, considering a diameter of 6 mm the design procedure presented on the section 2.4.2.4.c permits to obtain the spacing of the stirrups necessary to resist to the envelope of the shear solicitations. Figure 3.44 presents a recapitulative of these stirrups spacing along the beam.

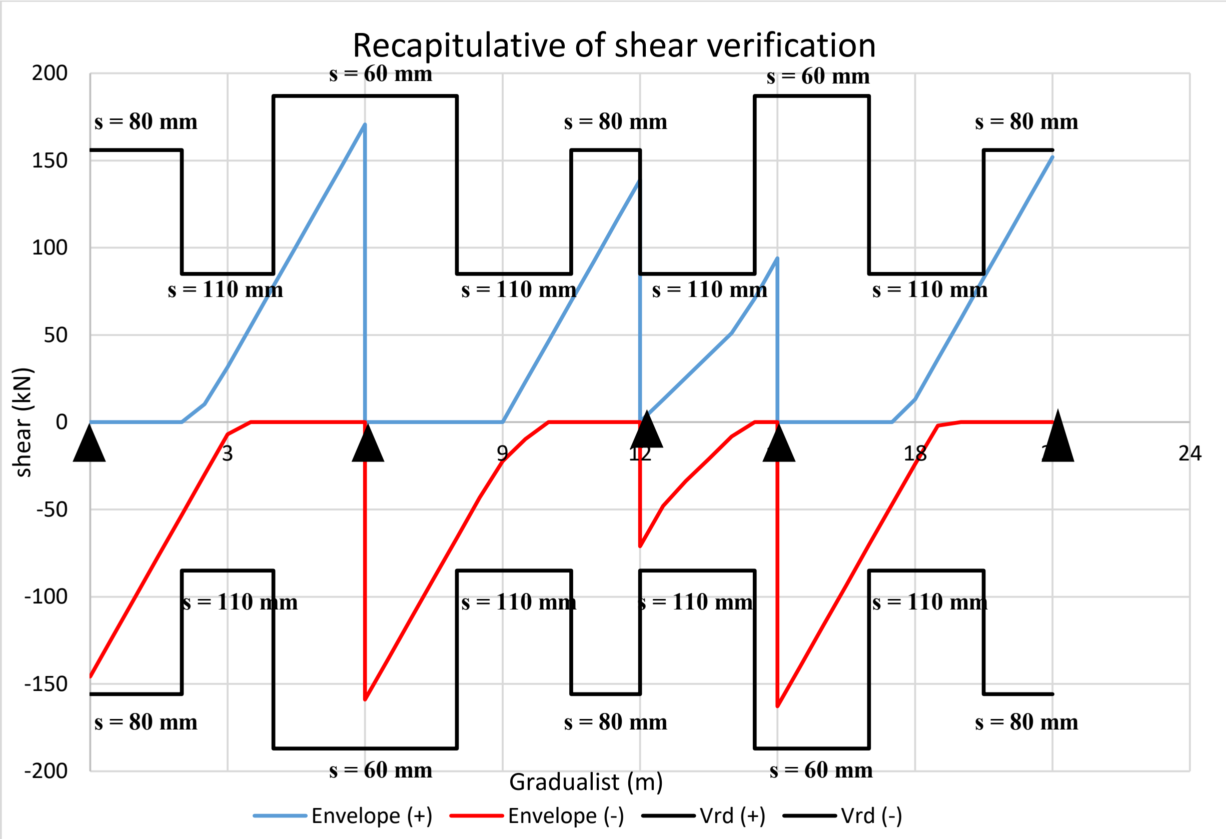


Figure 3.44. Recapitulative curve of the shear verification of the beam

3.4.1.5. Service Limit State (SLS)

The figure 3.45 shows the envelope curve of bending moment.

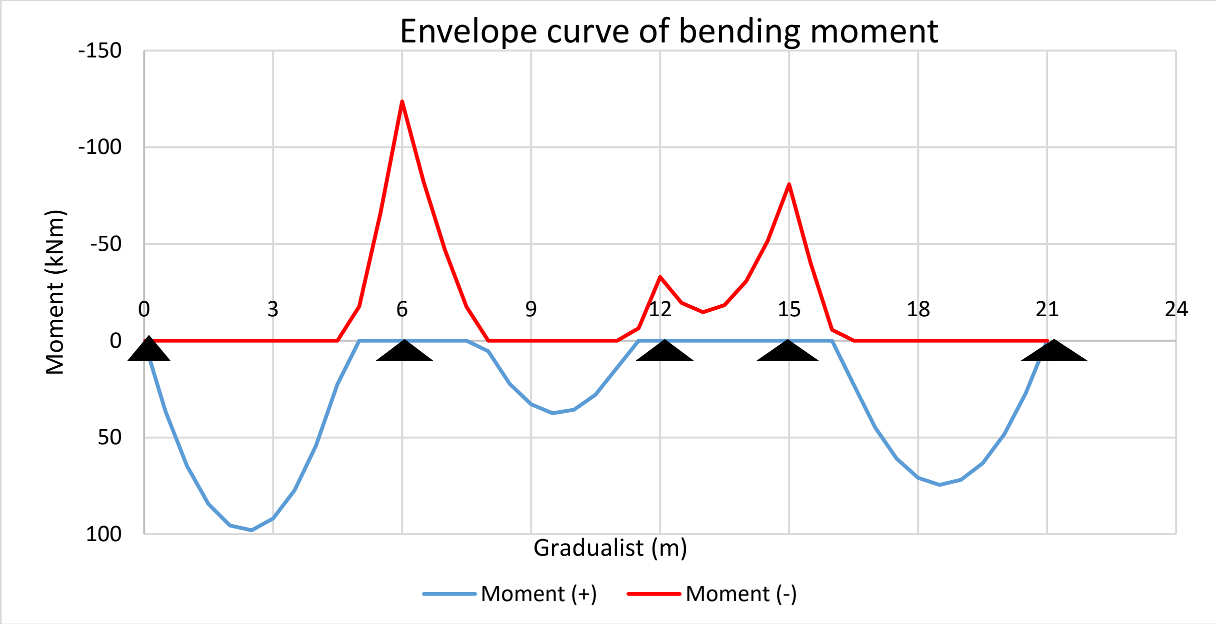
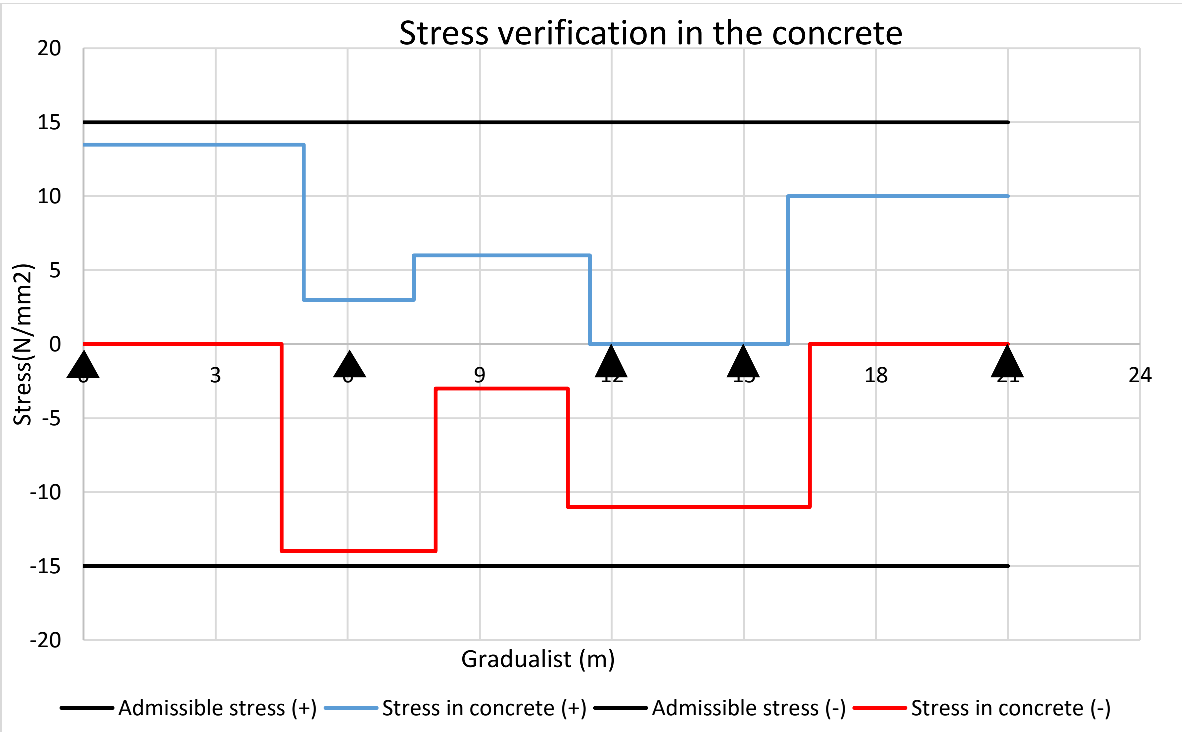
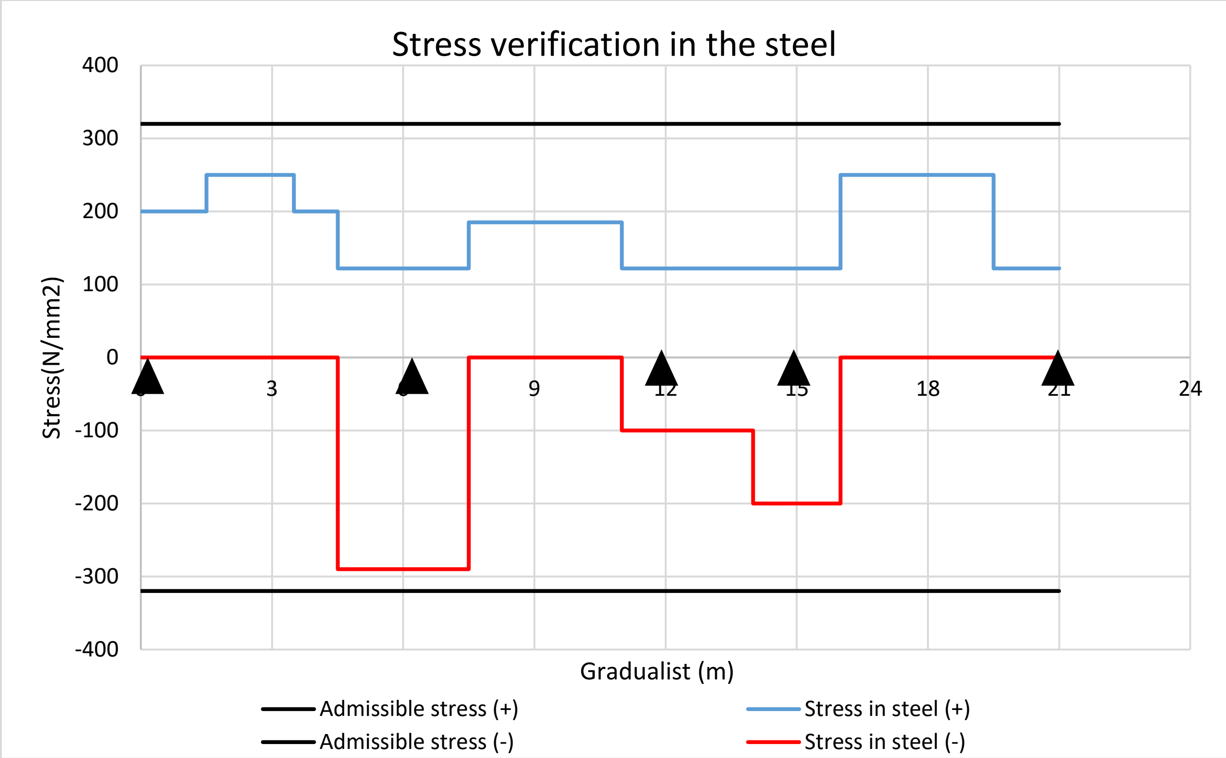


Figure 3.45. Envelope curve of bending moment

With this envelope curve for bending moment at serviceability limit state the stress in the concrete and in the reinforcement are obtained using the equations 2.73 and 2.74. The limit value on the stress is evaluated from the equations 2.75 and 2.76 using the recommended values of the Eurocode 2, means taking $k_1 = 0.6$ and $k_3 = 0.8$. Figure 3.46 shows a comparison of the stress inside the concrete and the steel reinforcement to the admissible stress.



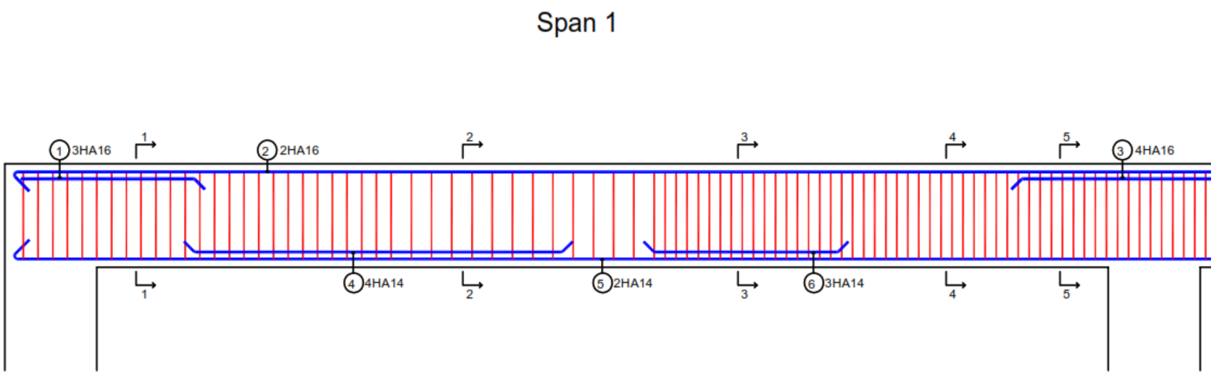
(a) in the concrete

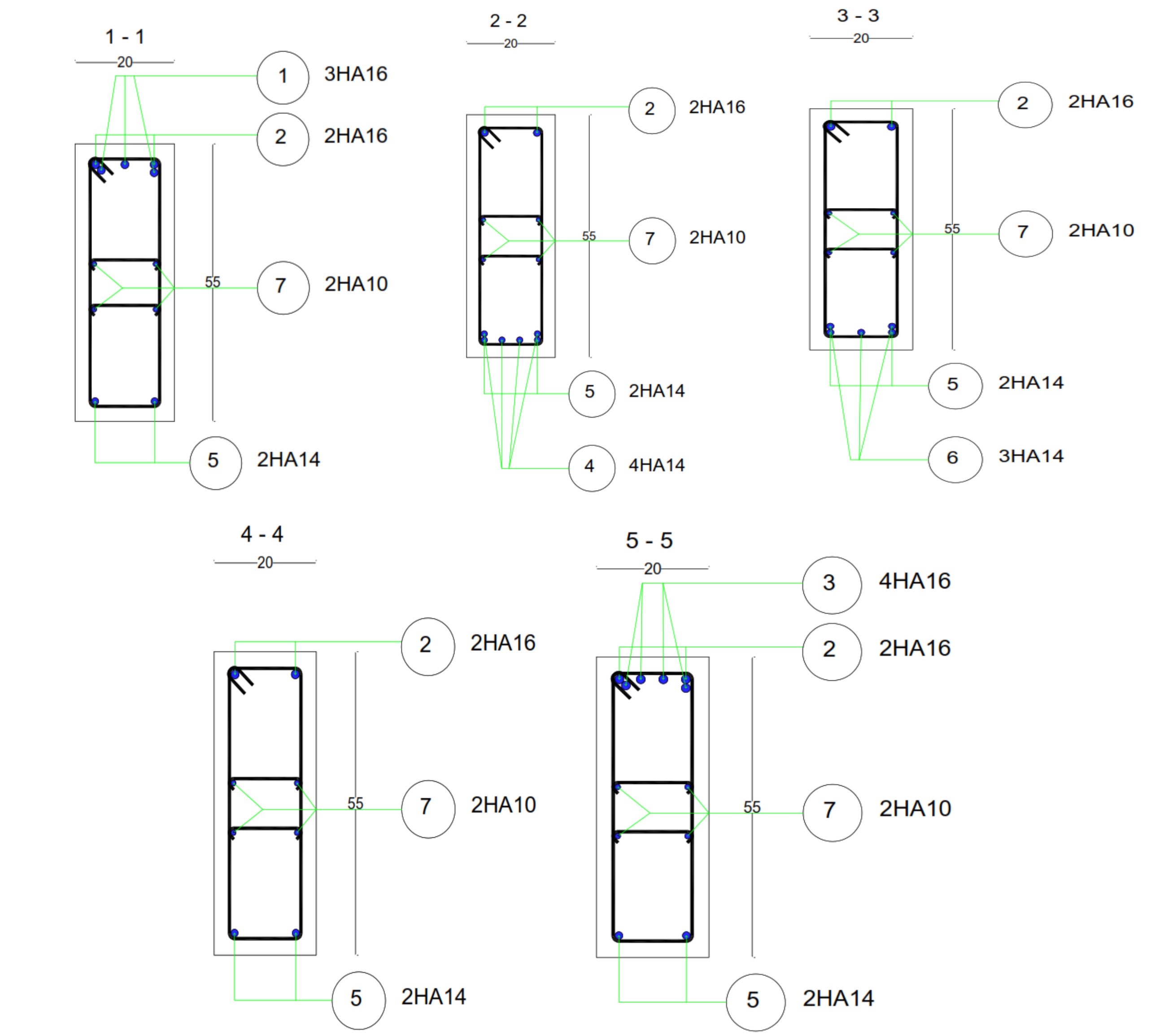


(b) in the steel

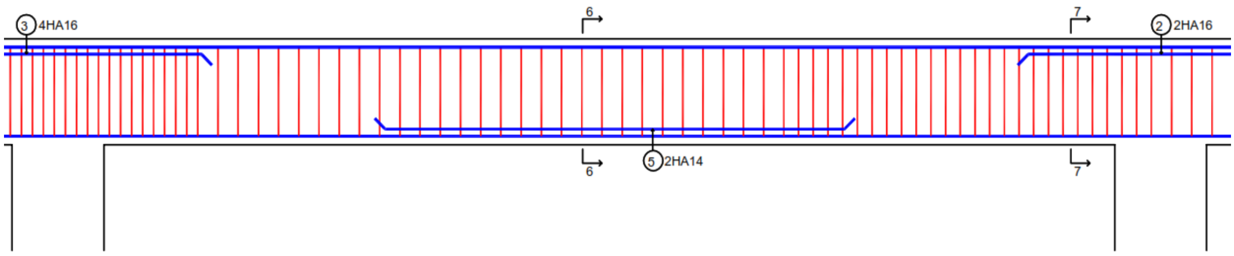
Figure 3.46. Recapitulative curve of stress verification of the beam: (a) in the concrete; (b) in the steel

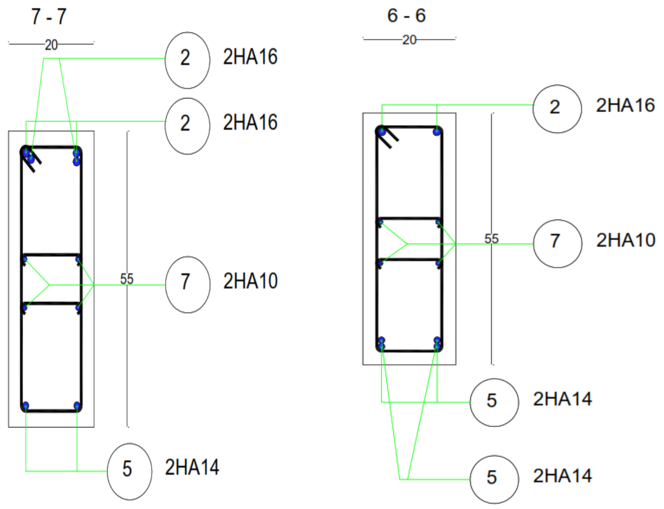
The figure 3.47 shows the distribution of the steel bars reinforcement in the beam.



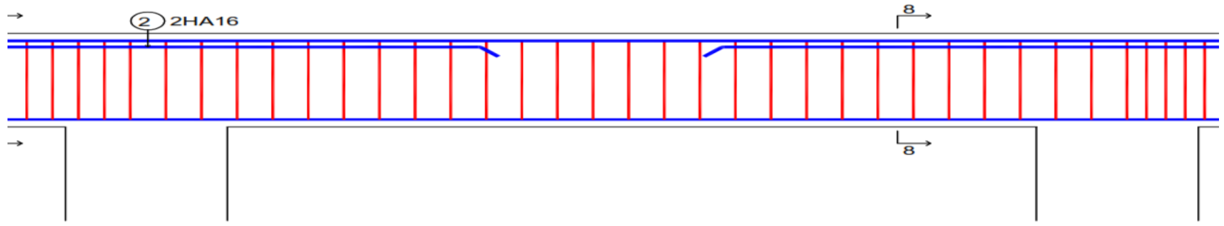

1 - 1
20
55
1 3HA16
2 2HA16
7 2HA10
5 2HA14
2 - 2
20
55
2 2HA16
7 2HA10
5 2HA14
4 4HA14
3 - 3
20
55
2 2HA16
7 2HA10
5 2HA14
6 3HA14
4 - 4
20
55
2 2HA16
7 2HA10
5 2HA14
5 - 5
20
55
3 4HA16
2 2HA16
7 2HA10
5 2HA14

Span 2

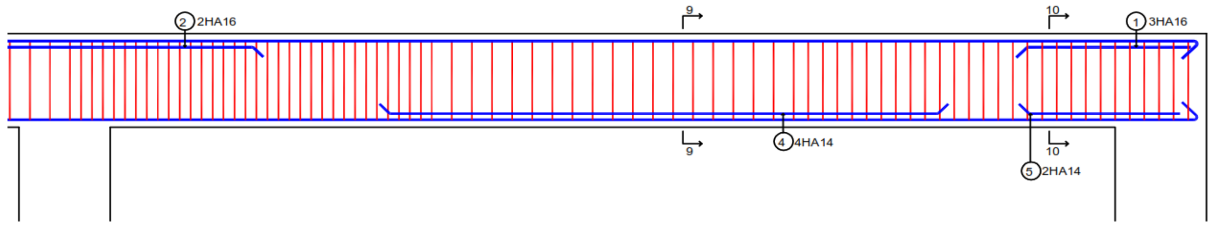

3 4HA16
2 2HA16
5 2HA14
6
7



Span 3



Span 4



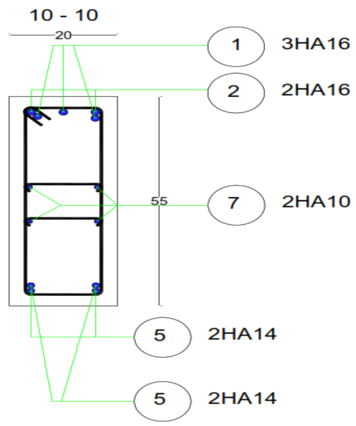


Figure 3.47. The distribution of the steel bars reinforcement in the beam

3.4.2. Case of wood-concrete floor

The cross section of beam is invented T beam of 45 cm width and 55 cm height.

3.4.2.1. Definition of the loads

Different loads applied on our beam are observed in the table 3.18.

Table 3.18. Loads of the design.

Designation	Values (kN/m)
permanent loads	18.01
Variable loads	9.45

We use the same combinations like in 3.3.1.2.

3.4.2.2. Ultimate limit state design

The envelope curve of bending moment is given in the figure 3.48.

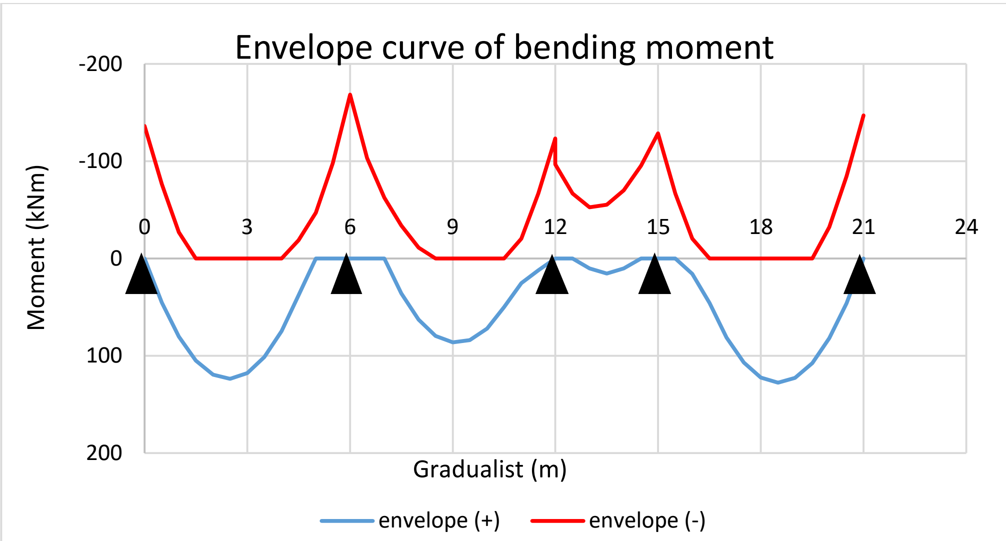


Figure 3.48. Envelope curve of bending moment

The envelope curve of shear is given in the figure 3.49.

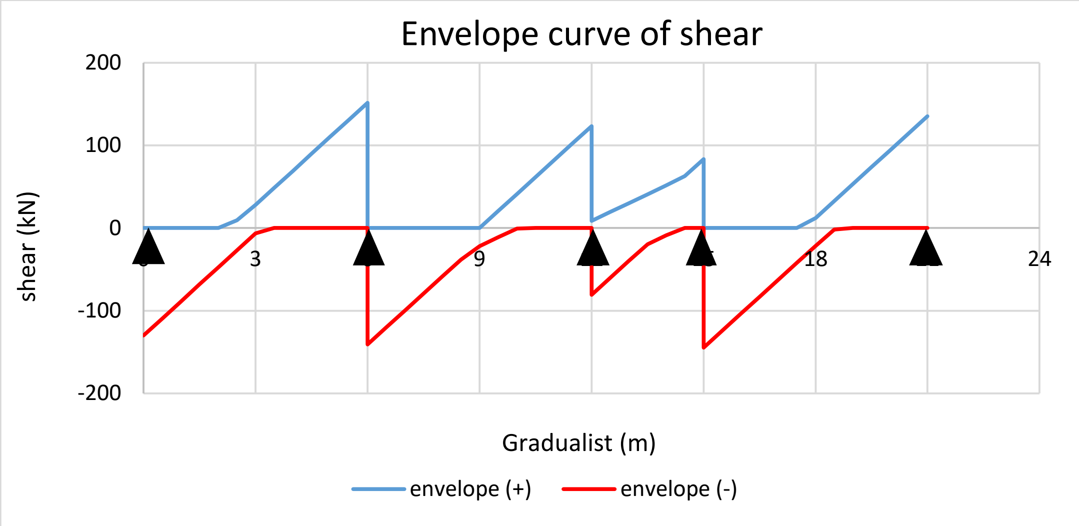


Figure 3.49. Envelope curve of shear

From the envelope curve of the bending moment presented in figure 3.48, we apply the procedure detailed in the section 2.4.2.4 to obtain the final solicitation curve presented in the figure 3.50.

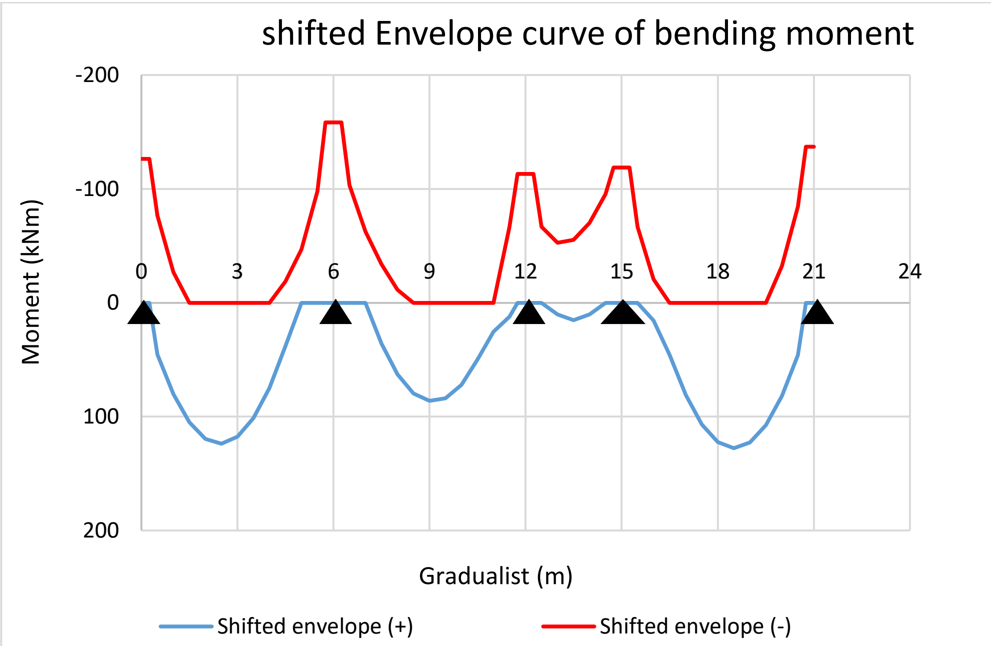


Figure 3.50. Shifted envelope curve of bending moment

3.4.2.3. Calculation of the steel bars reinforcement

The steel reinforcement is evaluated using the equation 2.60 and the section obtained is verified for the detailing of member presented in the equations 2.61 and 2.62. At the end, the steel section is evaluated, verified and the results obtained is presented in the figure 3.51.

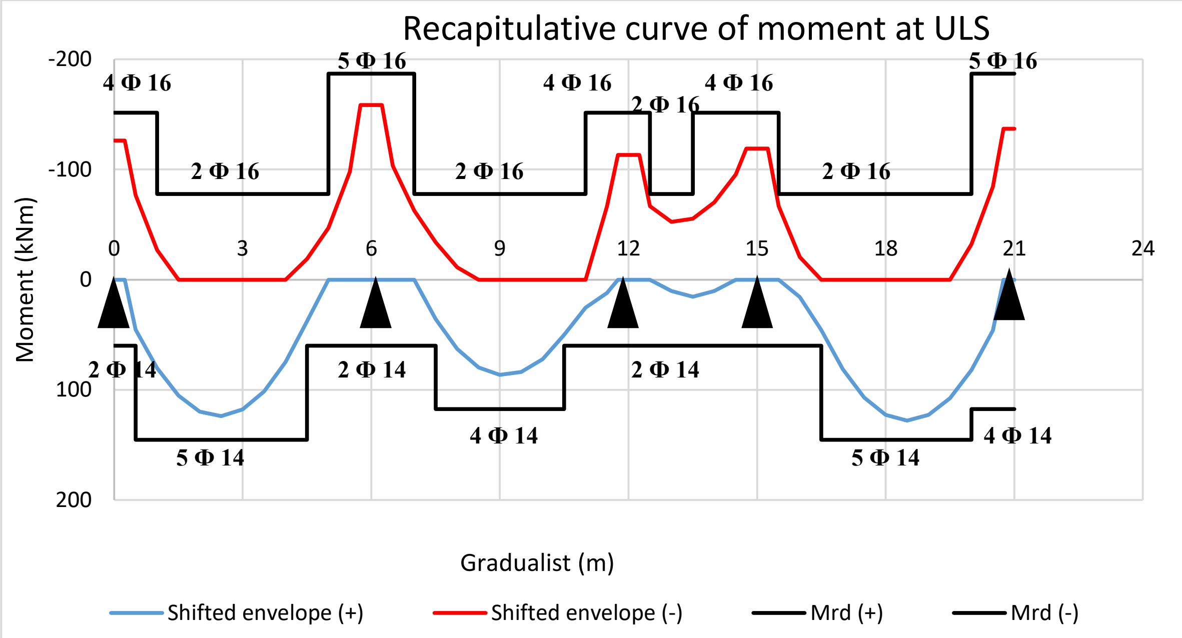


Figure 3.51. Recapitulative curve of the bending moment verification of the beam

For the transversal reinforcement, considering a diameter of 6 mm the design procedure presented on the section 2.4.2.4.c permits to obtain the spacing of the stirrups necessary to resist to the envelope of the shear solicitations. Figure 3.52 presents a recapitulative of these stirrups spacing along the beam.

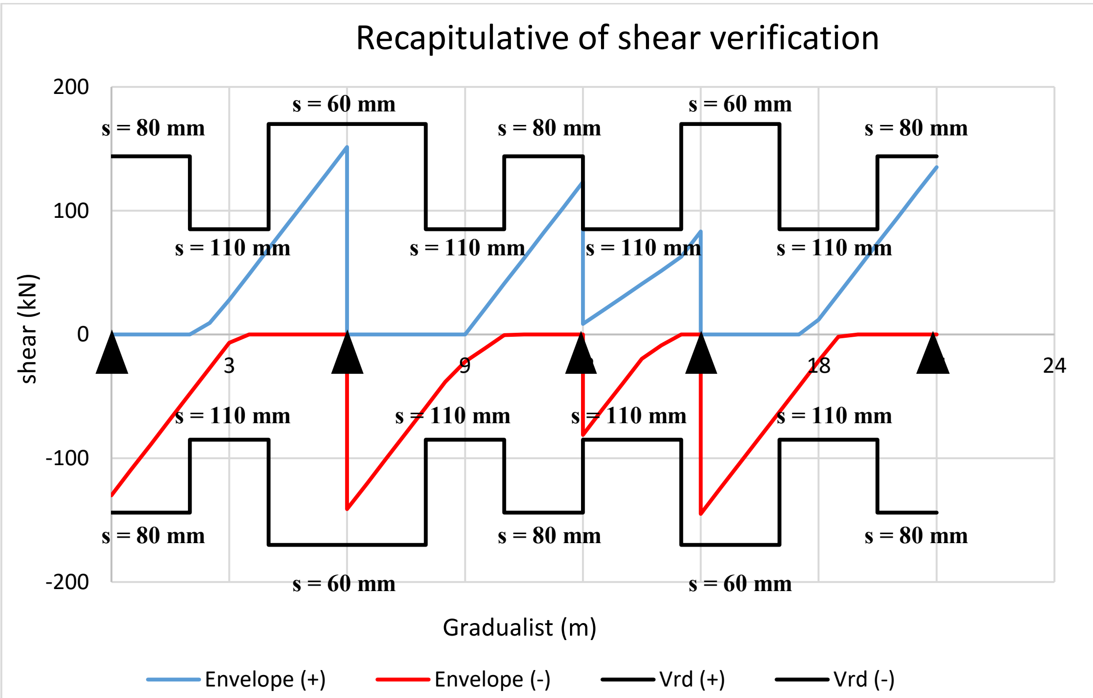


Figure 3.52. Recapitulative curve of the shear verification of the beam

3.4.2.4. Service Limit State (SLS)

The figure 3.53 shows the envelope curve of bending moment.

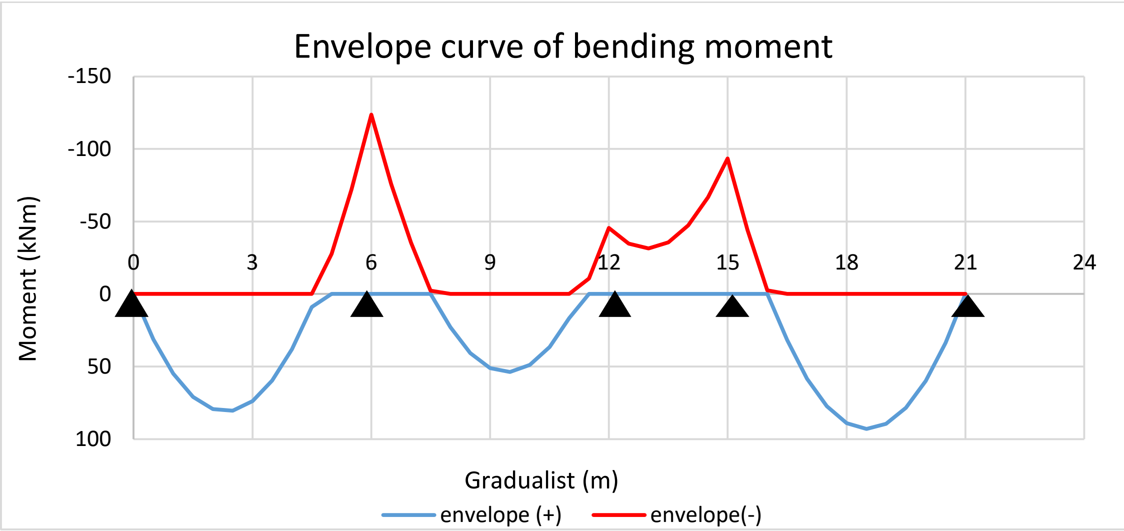
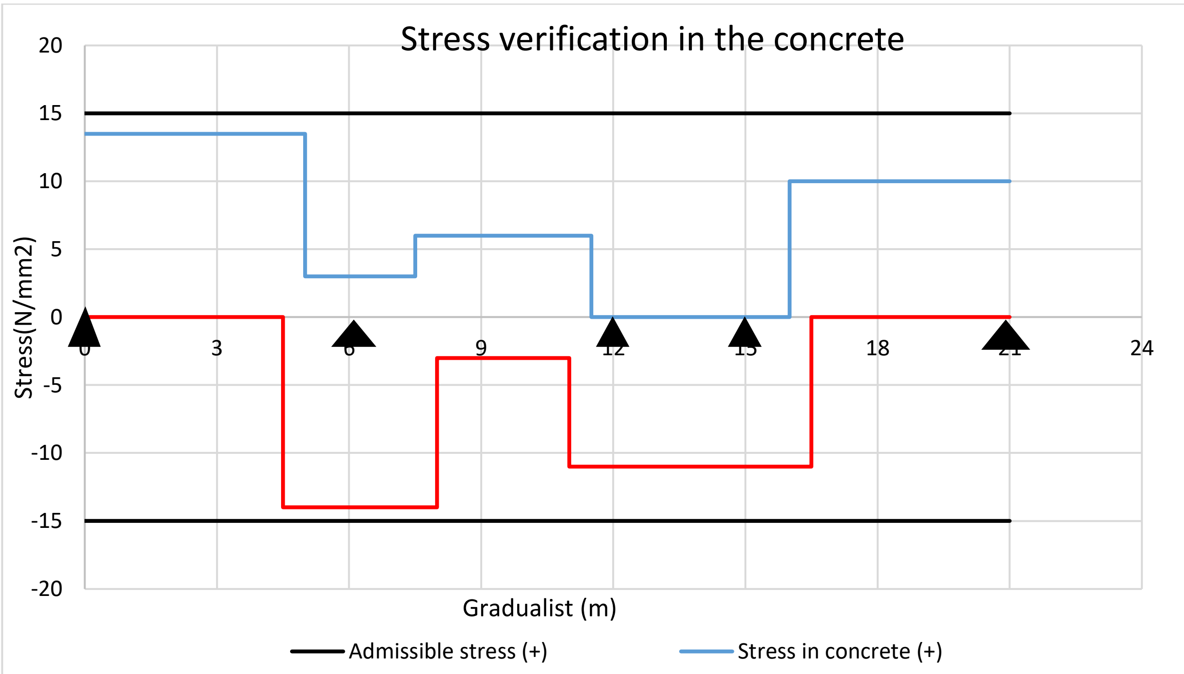
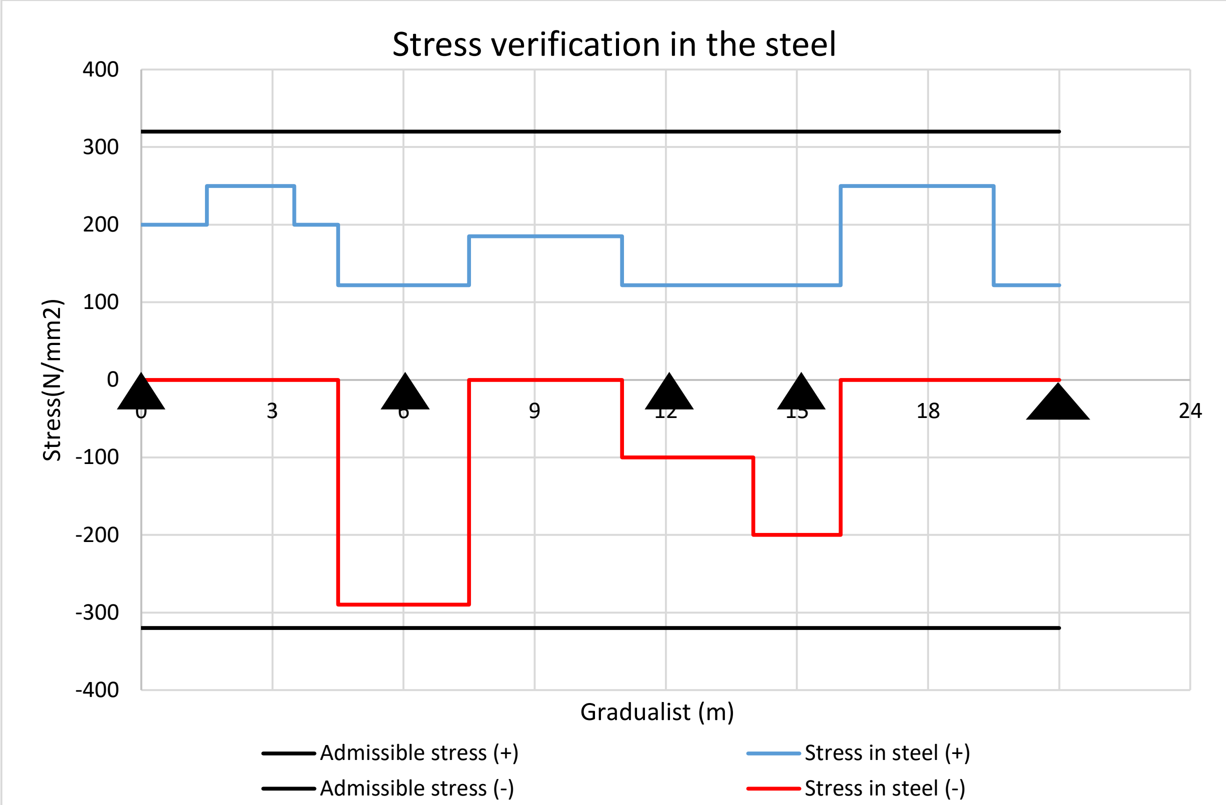


Figure 3.53. Envelope curve of bending moment

With this envelope curve for bending moment at serviceability limit state the stress in the concrete and in the reinforcement are obtained using the equations 2.73 and 2.74. The limit value on the stress is evaluated from the equations 2.75 and 2.76 using the recommended values of the Eurocode 2, means taking $k_1 = 0.6$ and $k_3 = 0.8$. Figure 3.54 shows a comparison of the stress inside the concrete and the steel reinforcement to the admissible stress.



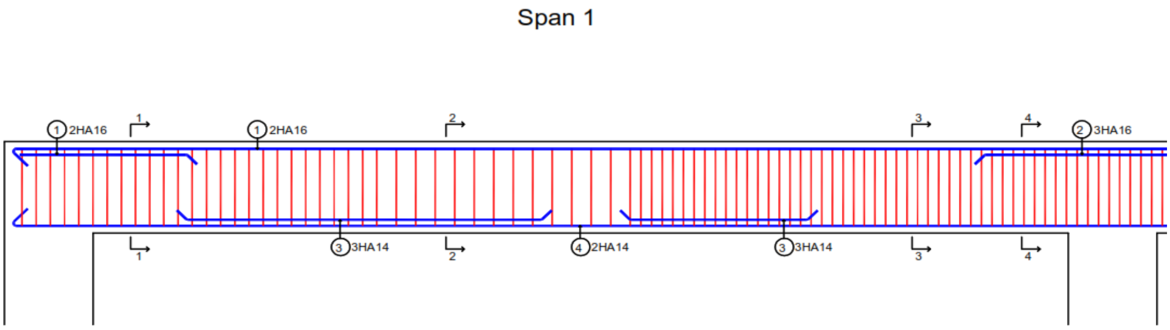
(a) in the concrete

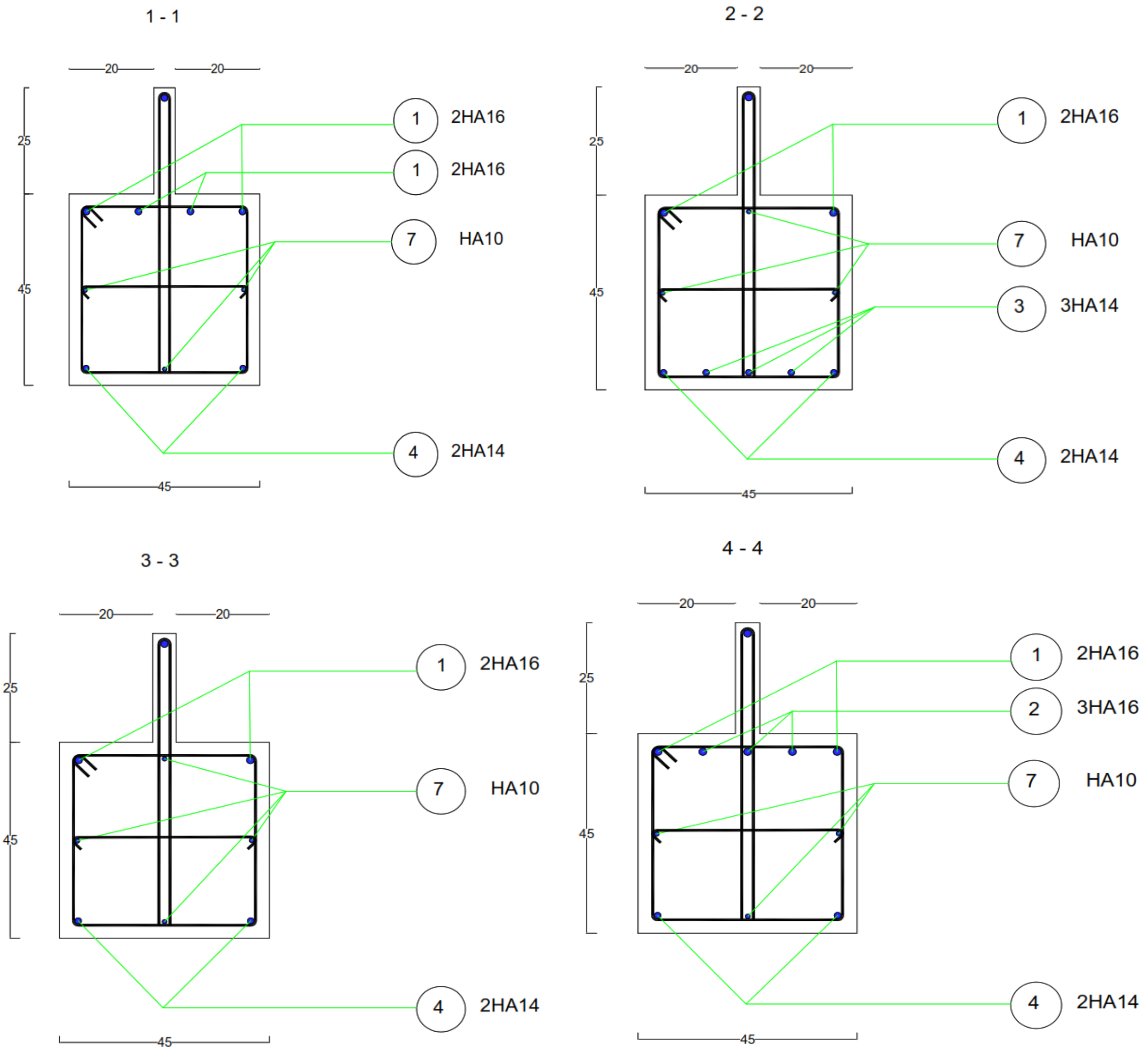


(b) in the steel

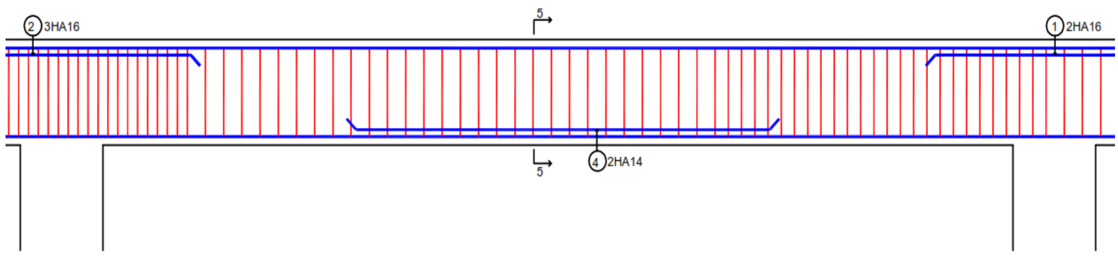
Figure 3.54. Recapitulative curve of stress verification of the beam: (a) in the concrete; (b) in the steel

The figure 3.55 shows the distribution of the steel bars reinforcement in the beam.

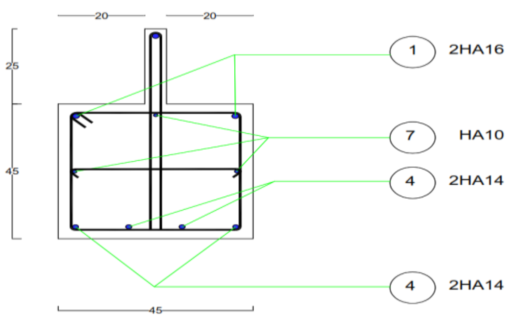



1 - 1
20
20
25
45
45
1 2HA16
1 2HA16
7 HA10
4 2HA14
2 - 2
20
20
25
45
45
1 2HA16
7 HA10
3 3HA14
4 2HA14
3 - 3
20
20
25
45
45
1 2HA16
7 HA10
4 2HA14
4 - 4
20
20
25
45
45
1 2HA16
2 3HA16
7 HA10
4 2HA14

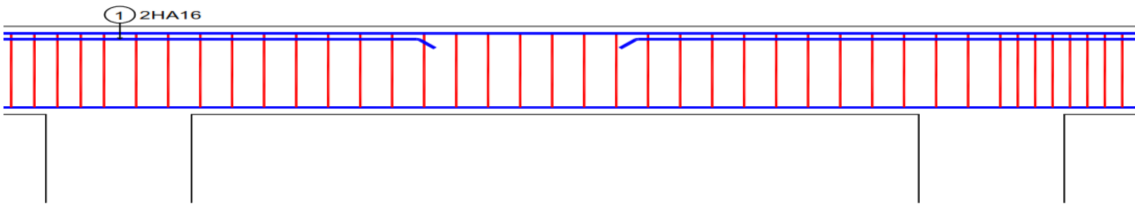
Span 2


2 3HA16
5
1 2HA16
5
4 2HA14

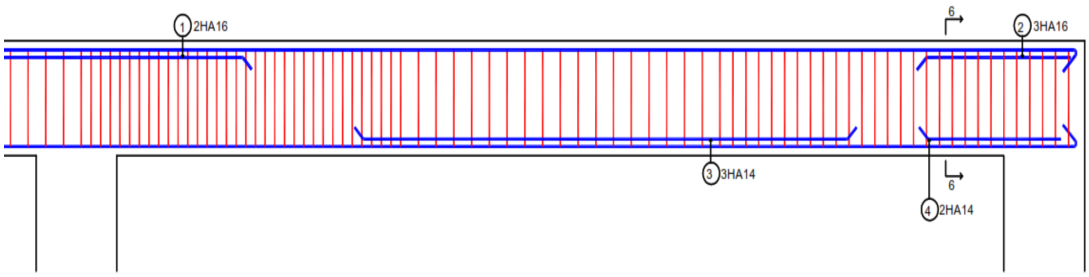
5 - 5



Span 3



Span 4



6 - 6

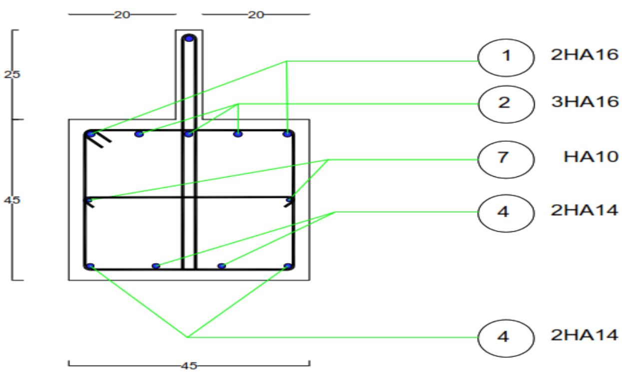


Figure 3.55. The distribution of the steel bars reinforcement in the beam

3.5. Design of the vertical structural elements

Here only the columns in case of hollow block floor and wood-concrete floor will be design. The column chosen for the design is the column B5 presented in figure 3.56.

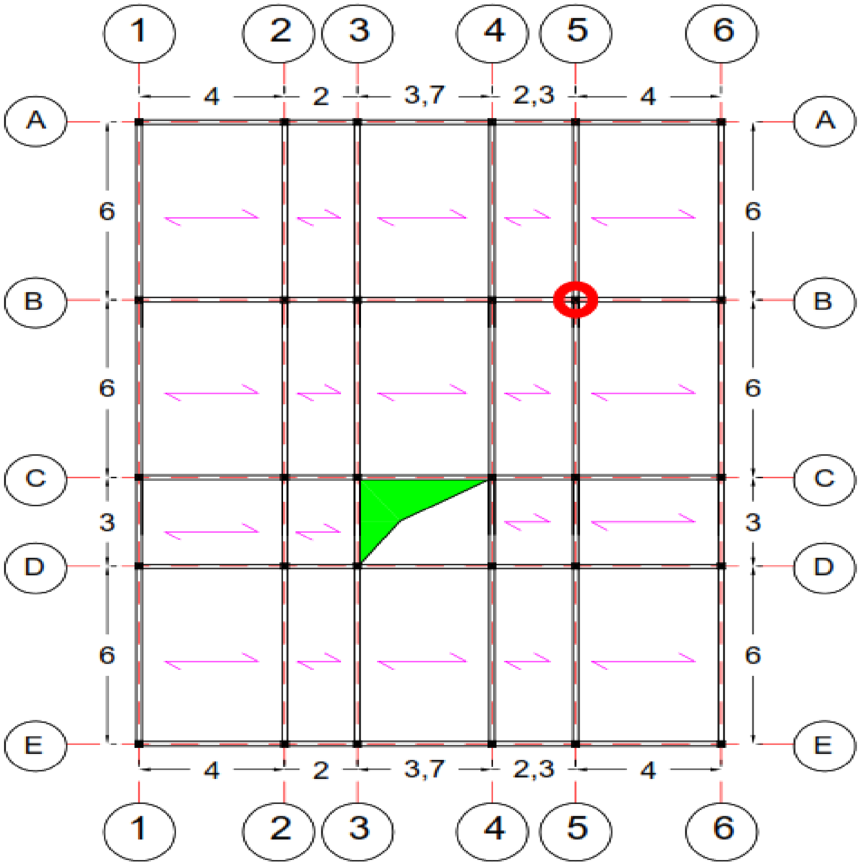


Figure 3.56. Choice of the study column

The modelling of the building in SAP 2000 is done as presented in the figure 3.57.

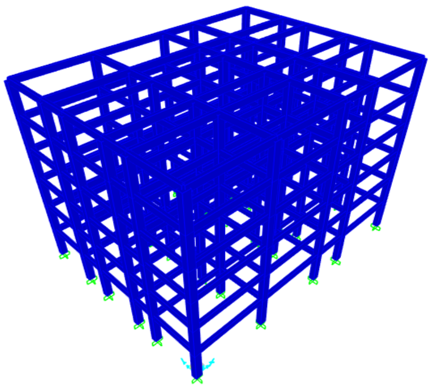


Figure 3.57. 3D model of the building

3.5.1. Case of hollow block floor

3.5.1.1. Preliminary design

Using equations 2.28 and 2.29, the section of columns for level 1 to 6 has to be $A_c \geq 101289\ mm^2$. The column section is then considered to be 400mm width by 500mm height.

For this structure which is a 6-storey reinforced concrete building with the total height of 18 m above the foundations, the fundamental period of the building is given using equation 2.44 T = 0.65s. The 3D modelling of the building in SAP 2000 with a fixed base and a percentage of participation of the imposed loads of 30 % permit to have the first mode as a translation (figure 3.58), the second mode as a translation (figure 3.59) and the third mode as torsion (figure 3.60). The first mode has a period of T_1 = 0.39974 s, T_2 = 0.38112s, T_3 = 0.36796s.

3D configuration Plan view

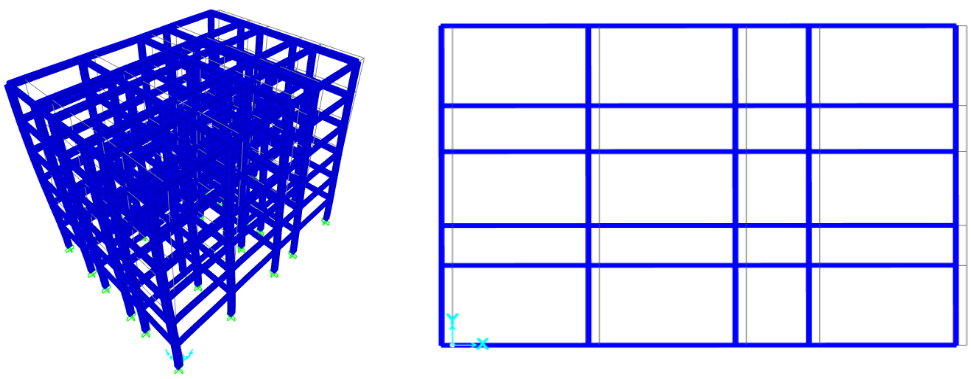


Figure 3.58. First vibration mode of the structure: translation in the x direction

3D configuration Plan view

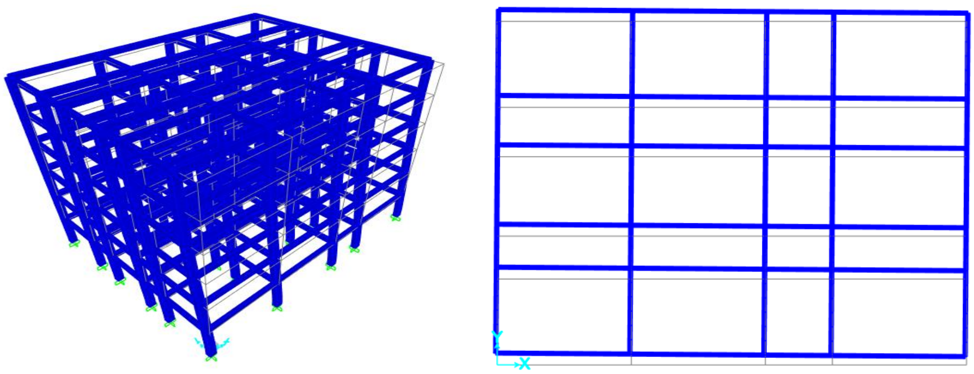


Figure 3.59. Second mode of vibration: translation in the y direction

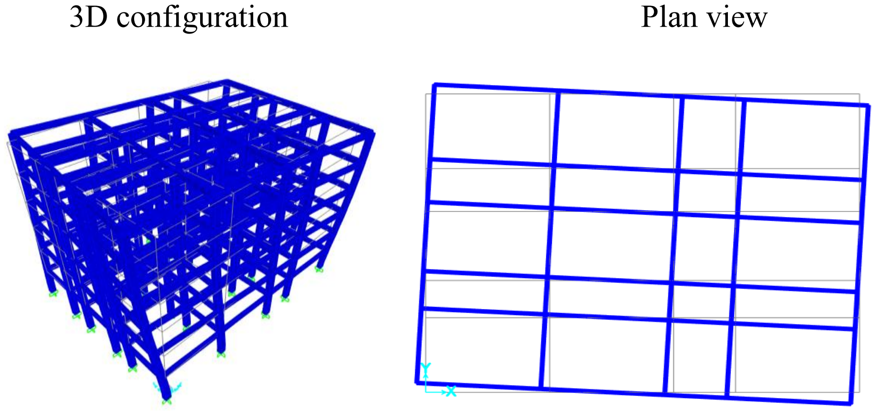
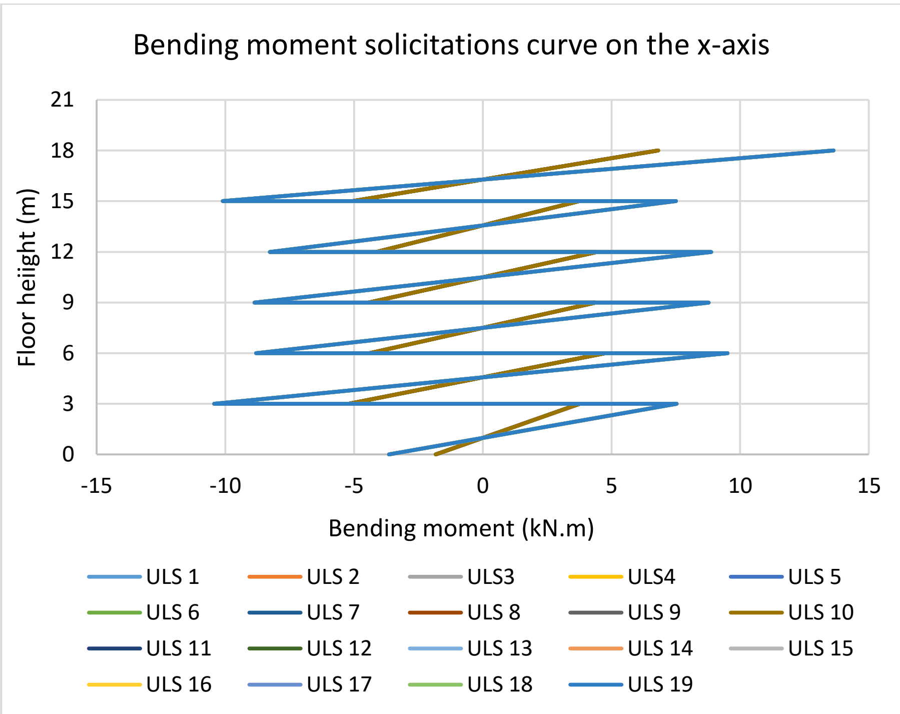


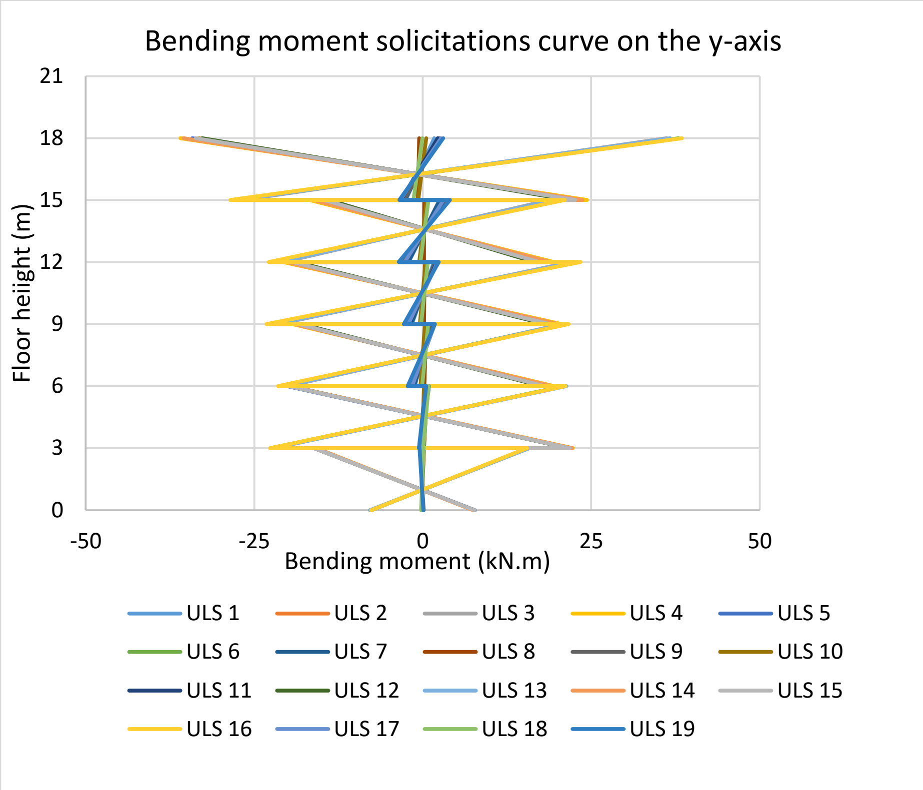
Figure 3.60. Third mode of vibration: torsion

3.5.1.2. Bending moment and axial force verification in the columns

Nineteen loads arrangements are considered for the principal and the secondary beams and permit to obtain different solicitation curves for the bending moment and the axial loads presented in figure 3.61 and figure 3.62 respectively.



a) Bending moment solicitation curves on x-axis



b) Bending moment solicitation curves on y-axis

Figure 3.61. Bending moment solicitations curves on the columns

Figure 3.62 shows the axial load solicitation curves for the load arrangements.

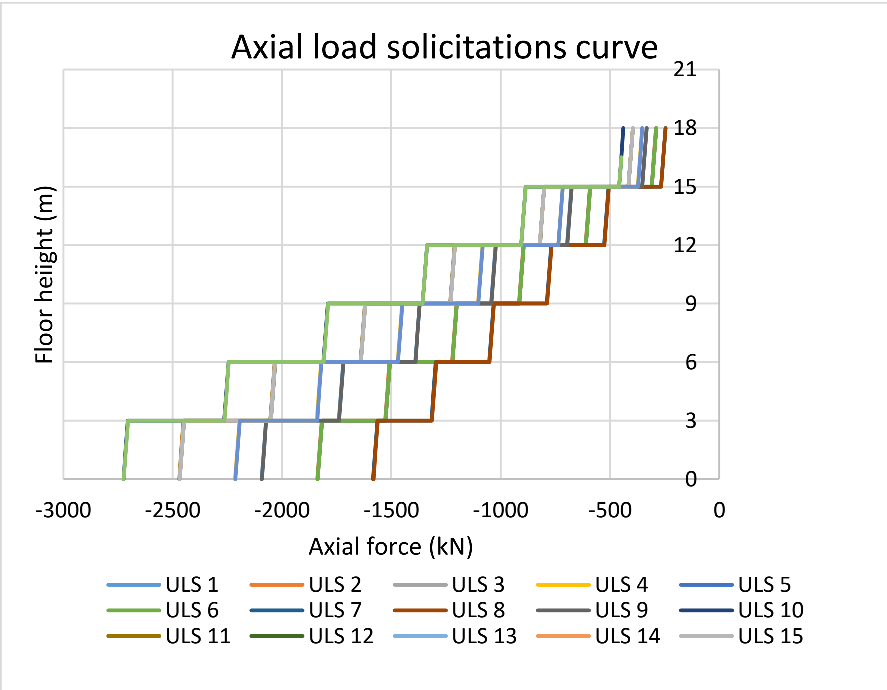


Figure 3.62. Axial load solicitation curves on the columns

From these solicitations the envelope curves of the bending moment in both x and y direction and the axial load solicitation are obtained and presented in the figure 3.63, figure 3.64 and figure 3.65 respectively.

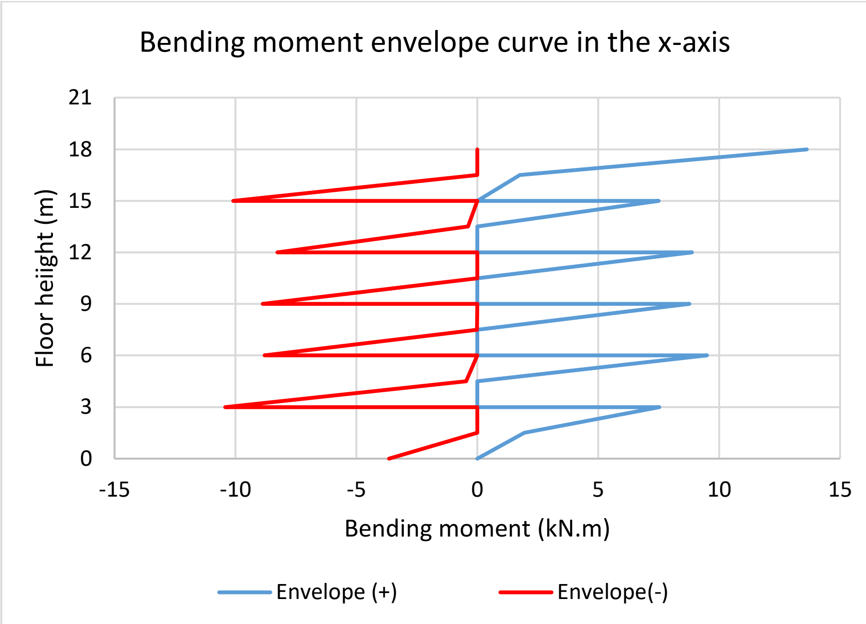


Figure 3.63. Bending moment envelope in x-axis

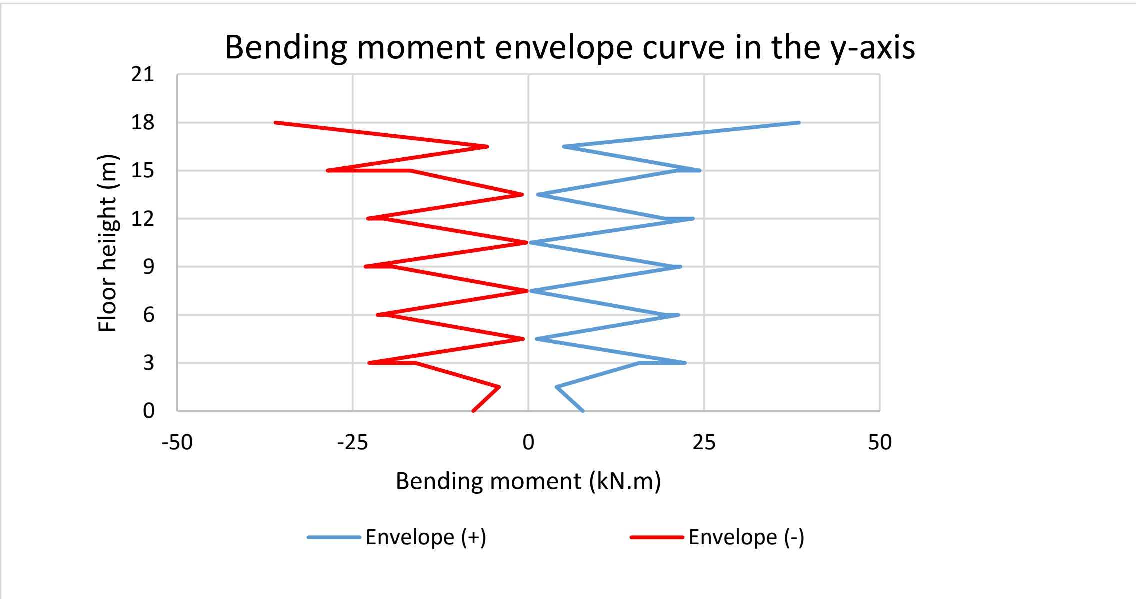


Figure 3.64. Bending moment envelope in y axis

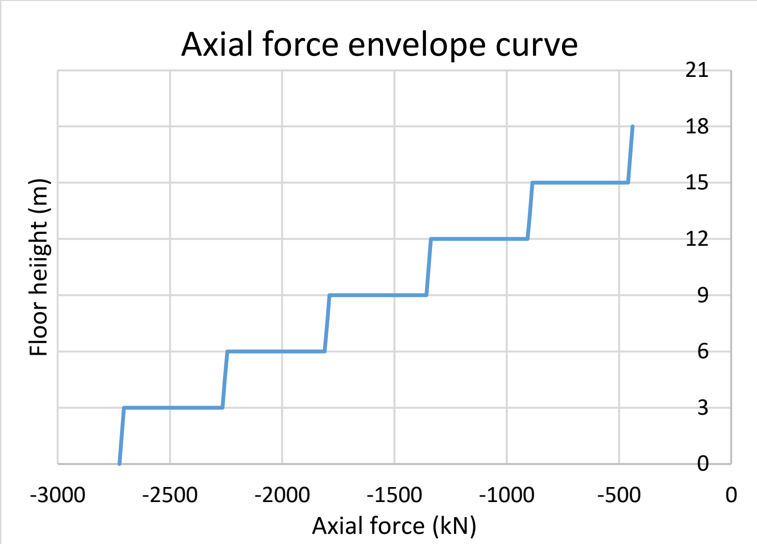


Figure 3.65. Axial load envelope curve

The verification of the axial loads and the bending moment is done through the interaction diagram as presented in the section 2.4.3.1. In this case study, considering a concrete section 40 cm × 50 cm, the limitations prescribes on the equations 2.38 and 2.39 result in the expression

$$698mm^2 \leq A_s \leq 8000mm^2$$

Which corresponds to a concrete section of 40 cm width and 50 cm height, to a minimum number of 7 bars of 12 mm diameter (7Ø12). The interaction diagram of the column in the two directions, considering a concrete section of **40*cm*** width and **50 *cm*** height and a steel reinforcement of **8Ø1**6 are presented in figures 3.66 and 3.67 for the two directions.

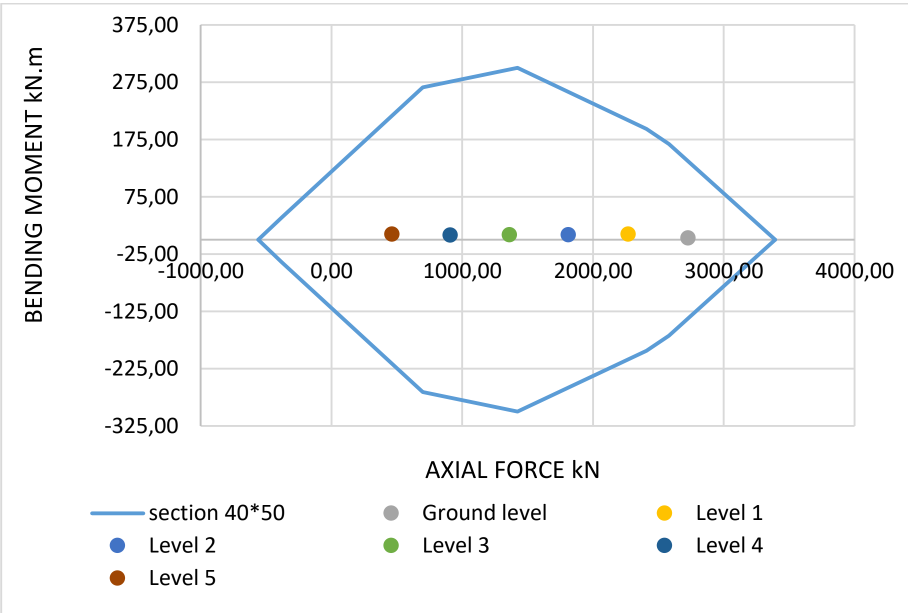


Figure 3.66. Interaction diagram of the column B5 in the x-direction

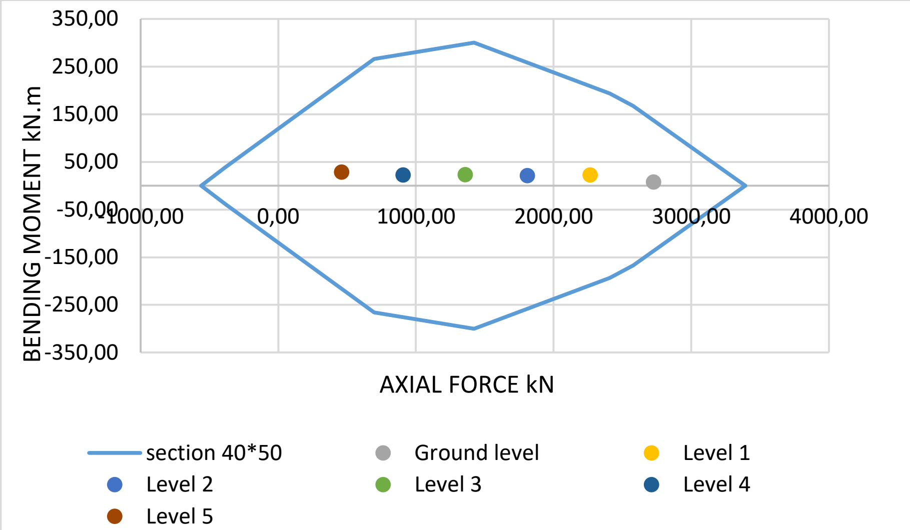
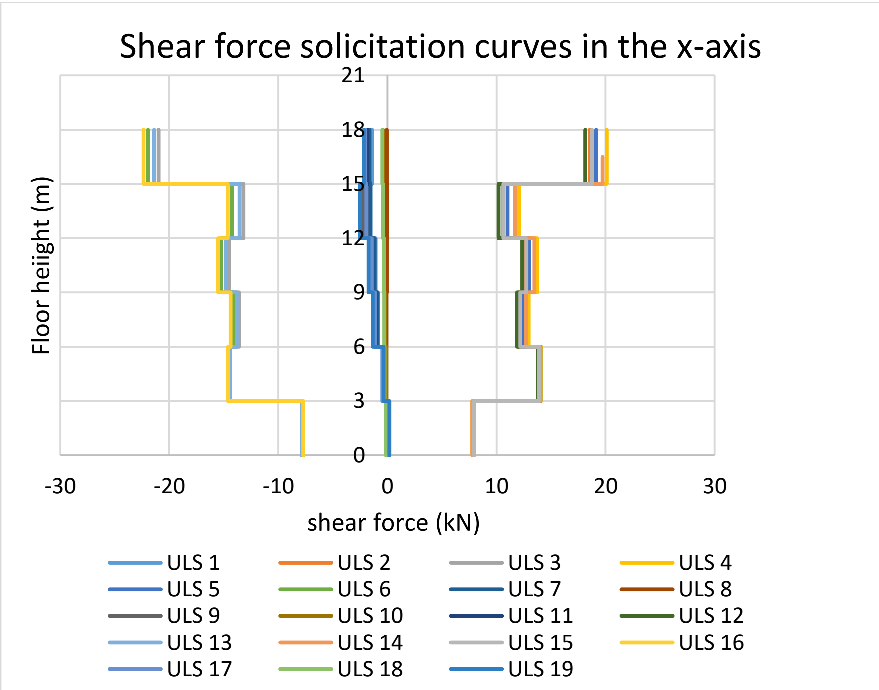


Figure 3.67. Interaction diagram of the column B5 in the y-direction

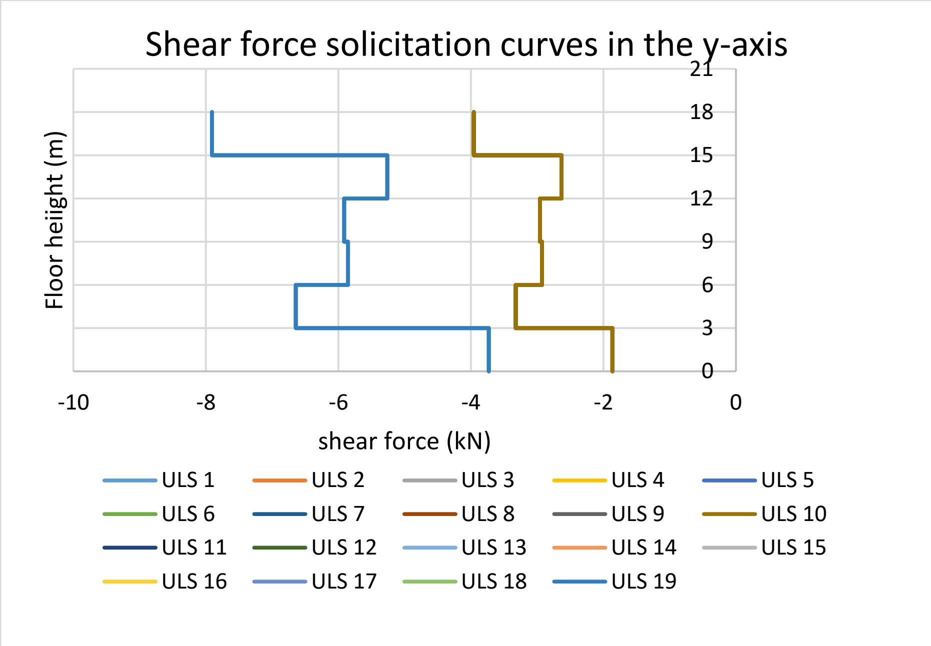
We observe that the bending moment and the axial force solicitations of the column is inside the diagram so the section is correct.

3.5.1.3. Shear verification

The different loads arrangements permits to obtain the shear solicitation curves in the x and y-direction as presented in figure 3.68



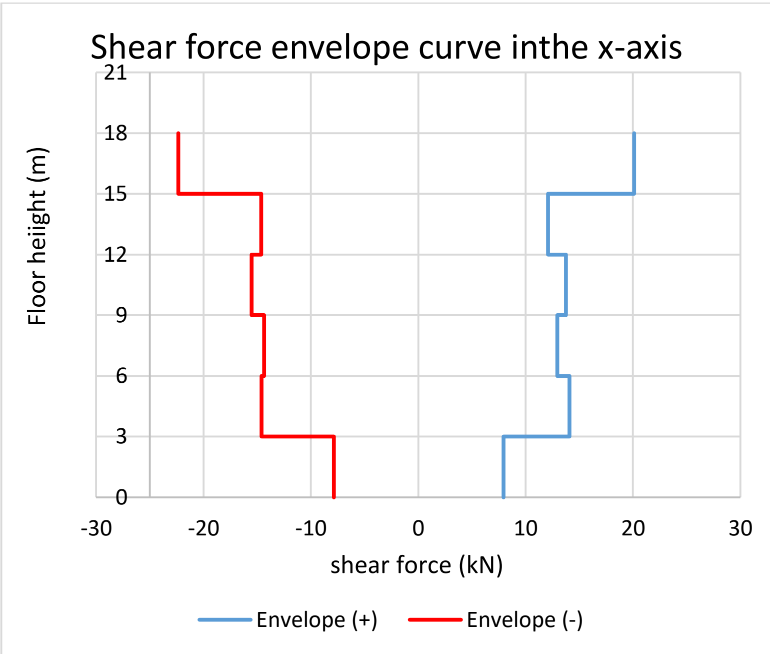
(a) in the y-axis



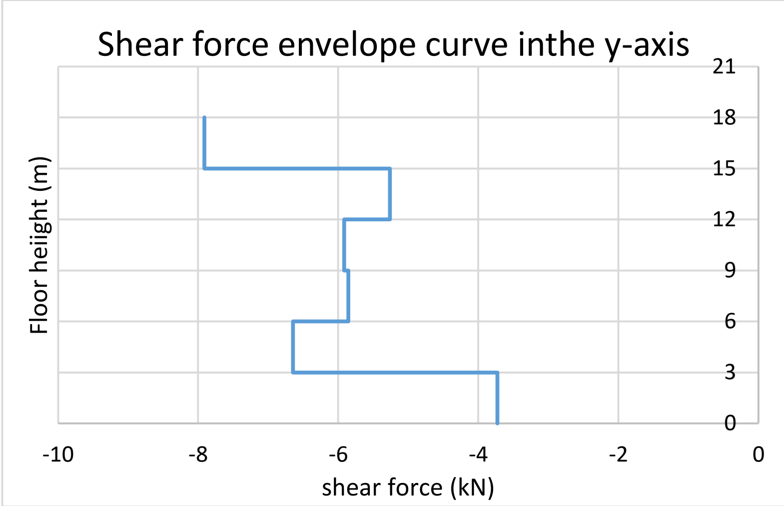
(b) in the x-axis

Figure 3.68. Shear force solicitation curve on the column: (a) in the y-axis; (b) in the x-axis

The envelope of these solicitation curves is obtained and presented in figure 3.69.



(a) in the y-axis



(b) in the x-axis

Figure 3.69. Shear force envelope curve on the column: (a) in the y-axis; (b) in the x-axis

Applying the procedure presented in the section 2.4.2.3.c, we observe that the shear resistance of the section without shear reinforcement is greater than the maximum shear solicitation on the column so the detailing of members has to be applied to have the spacing. In our case, we consider a diameter of 8 mm and the maximum spacing of the transverse reinforcement is given by:

$S_{cl,} = \min(320; 400; 400) = 320mm$

So, applying the prescriptions of the section 2.4.2.1.b, we obtain a spacing of the shear reinforcement of: 16 cm within 0.55 cm above and below the beams and 32 cm for the rest of the column.

3.5.1.4. Slenderness verification

Following the procedure presented on the section 2.4.3.4, the different parameters are evaluated and presented in table 3.19.

Table 3.19. Parameter for slenderness verification

A	B	C	n	λ	λ_{lim}
0.7	1.17	0.7	0.62	14	16

From the table 3.19 we have $\lambda < \lambda_{lim}$, so the slenderness of the column is verified. The figure 3.70 shows the distribution of the steel bars reinforcement in the column.

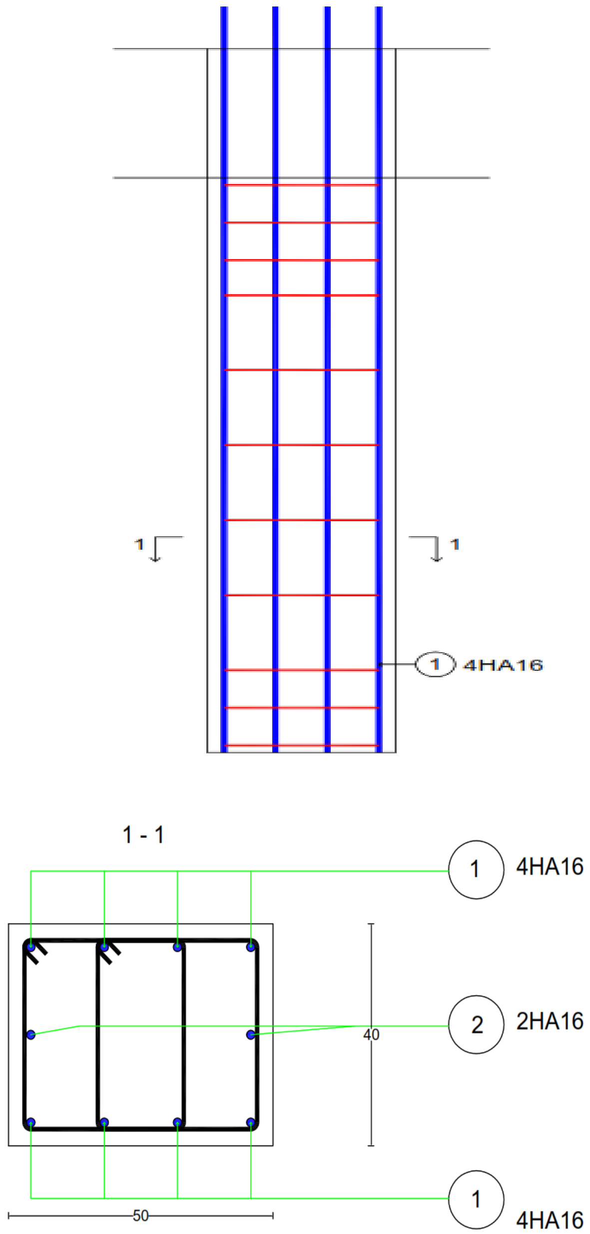
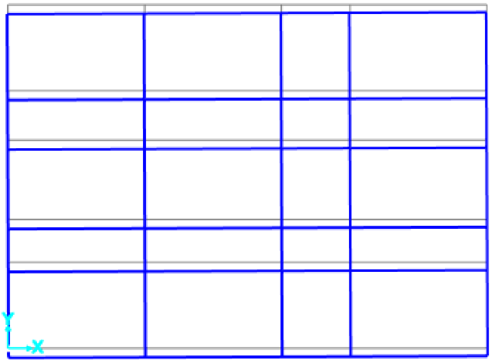


Figure 3.70. The distribution of the steel bars reinforcement in the column

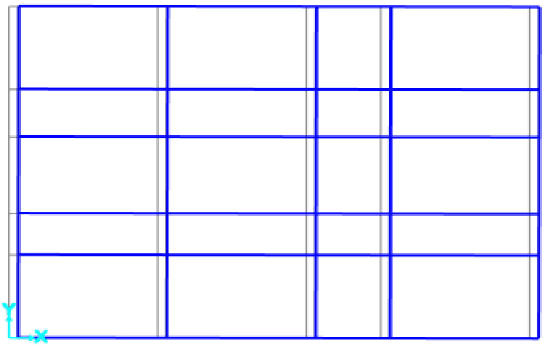
3.5.2. Case of wood-concrete floor

3.5.2.1. Preliminary design

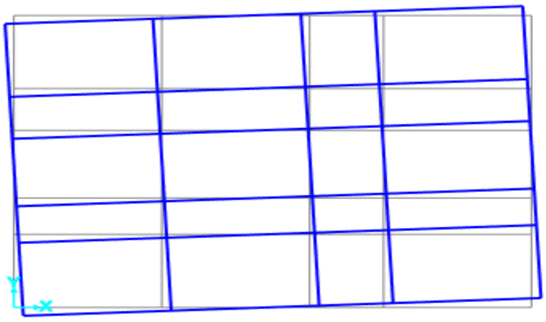
The section of column is the same like in the case of hollow-block floor. The 3D modelling of the building in SAP 2000 with a fixed base and a percentage of participation of the imposed loads of 30 % permit to have the first mode as a translation , the second mode as a translation and the third mode as torsion as shows in figure 3.71. The first mode has a period of $T_1 = 0.44564$ s, $T_2 = 0.41206$, $T_3 = 0.40622$s.



Mode 1 (translation along y axis)



Mode 2 (translation along x axis)

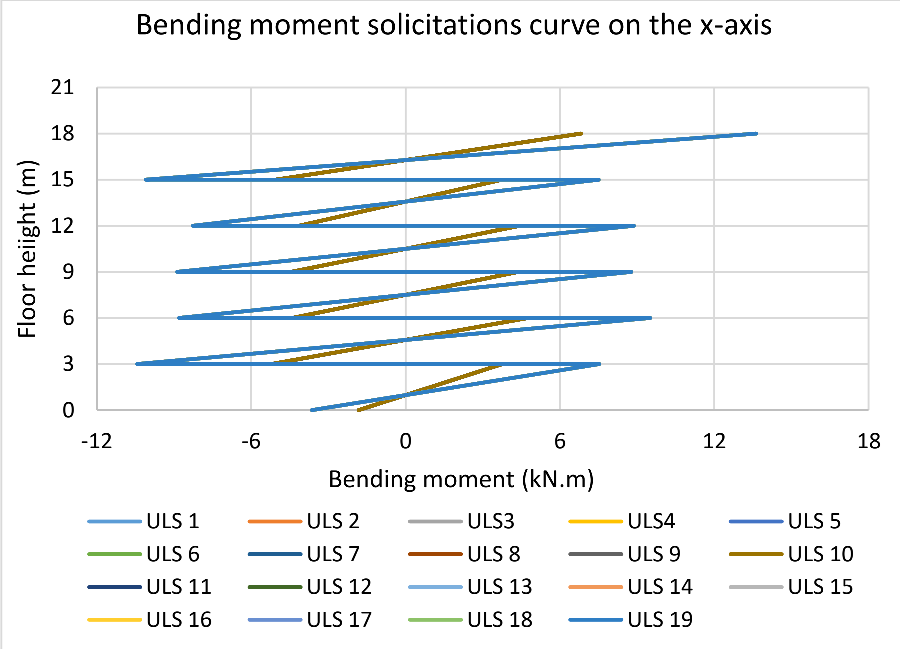


Mode 3 (torsion)

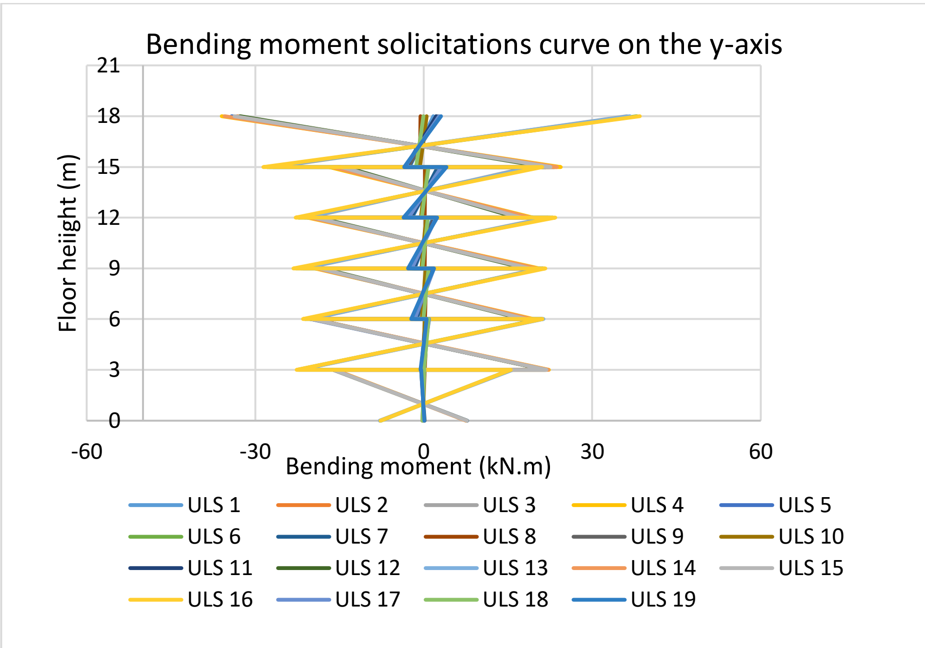
Figure 3.71. First three modes patterns of the modal analysis

3.5.2.2. Bending moment and axial force verification in the columns

Nineteen loads arrangements are considered for the principal and the secondary beams and permit to obtain different solicitation curves for the bending moment and the axial loads presented in figure 3.72 and figure 3.73 respectively.



a) Bending moment solicitation curves on x-axis



b) Bending moment solicitation curves on y-axis

Figure 3.72. Bending moment solicitations curves on the columns

Figure 3.73 shows the axial load solicitation curves for the load arrangements.

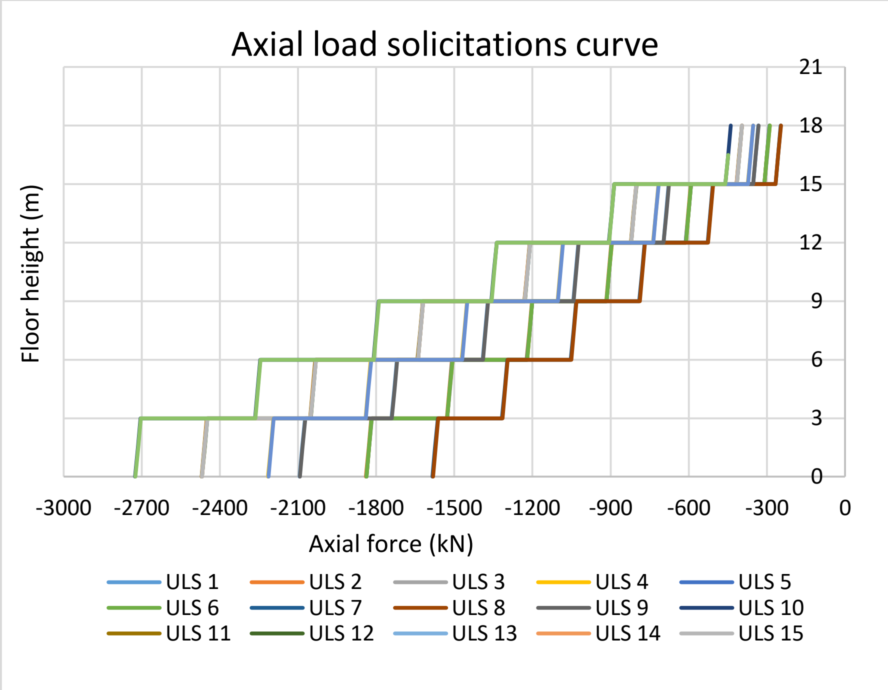


Figure 3.73. Axial load solicitation curves on the columns

From these solicitations the envelope curves of the bending moment in both x and y direction and the axial load solicitation are obtained and presented in the figure 3.74, figure 3.75 and figure 3.76 respectively.

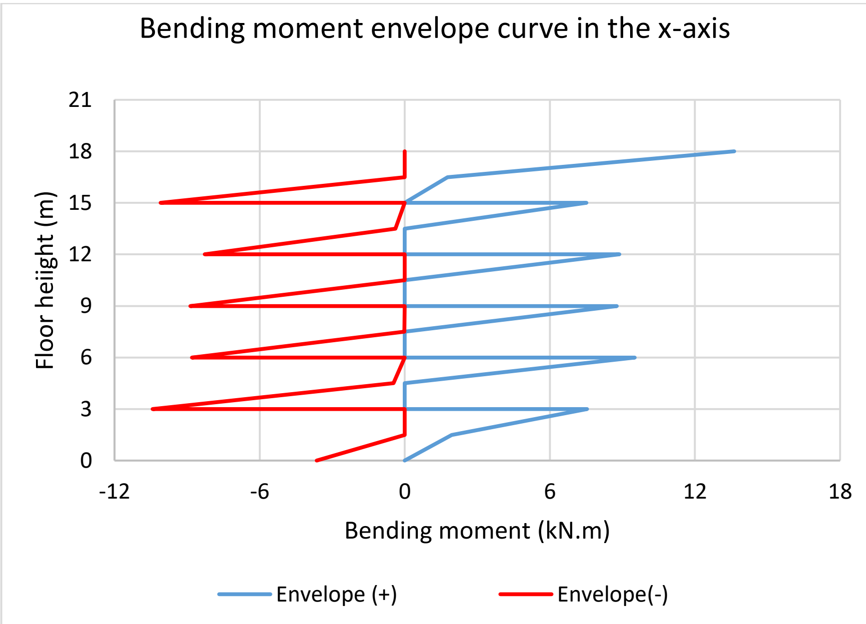


Figure 3.74. Bending moment envelope in x-axis

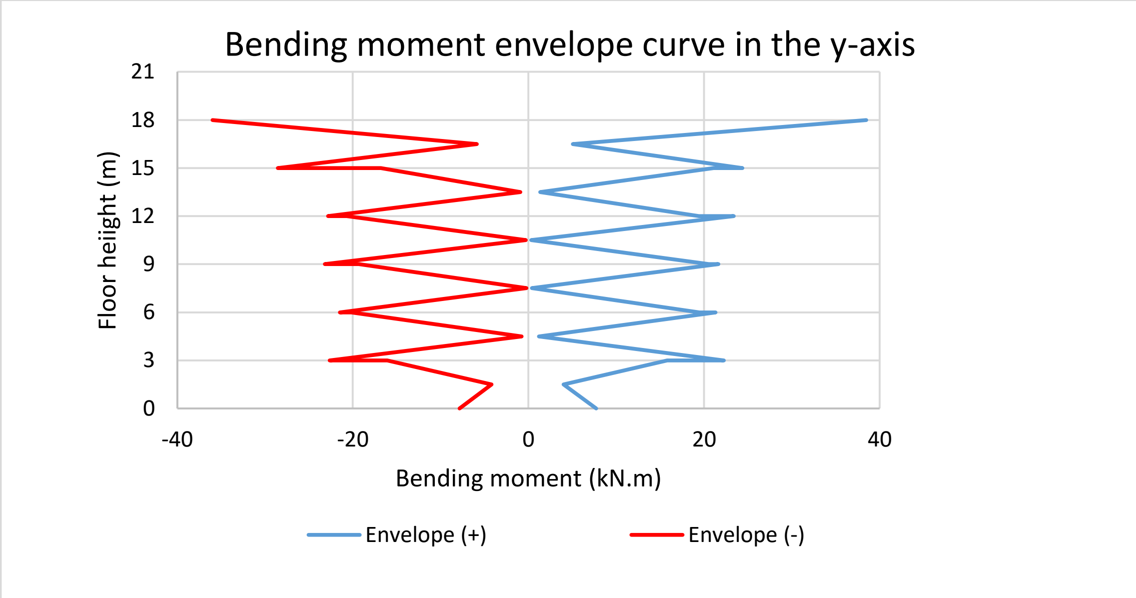


Figure 3.75. Bending moment envelope in y axis

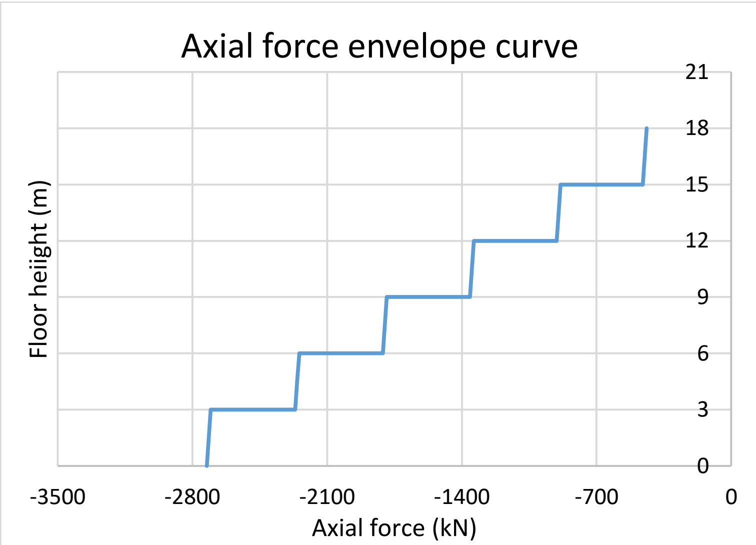


Figure 3.76. Axial load envelope curve

The verification of the axial loads and the bending moment is done through the interaction diagram as presented in the section 2.4.3.1. In this case study, considering a concrete section 40 cm × 50 cm, the limitations prescribes on the equations 2.38 and 2.39 result in the expression

$$500mm^2 \leq A_s \leq 6000mm^2$$

Which corresponds to a concrete section of 40 cm width and 50 cm height, to a minimum number of 5 bars of 12 mm diameter (5∅12). The interaction diagram of the column in the two directions, considering a concrete section of **40*cm*** width and **50 *cm*** height and a steel reinforcement of **6∅16** are presented in figures 3.77 and 3.78 for the two directions.

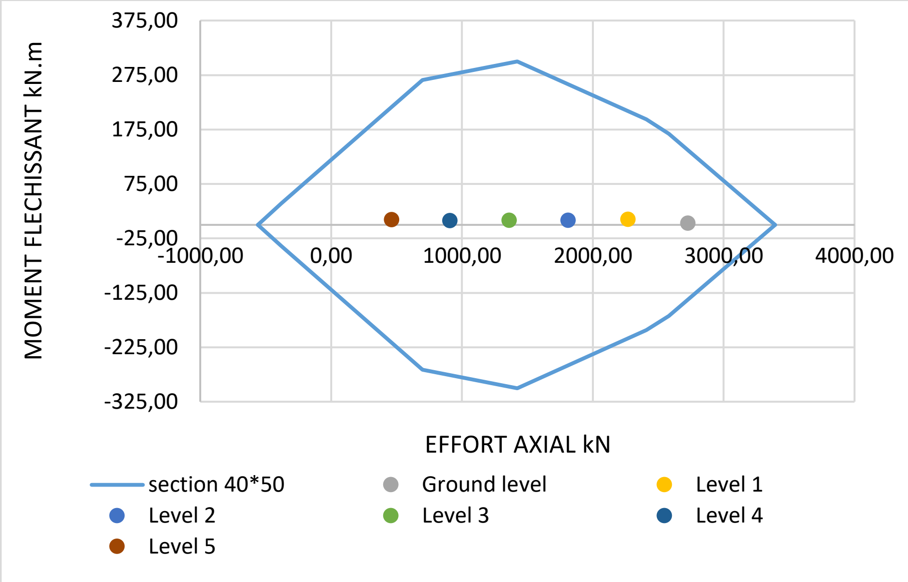


Figure 3.77. Interaction diagram of the column B5 in the x-direction

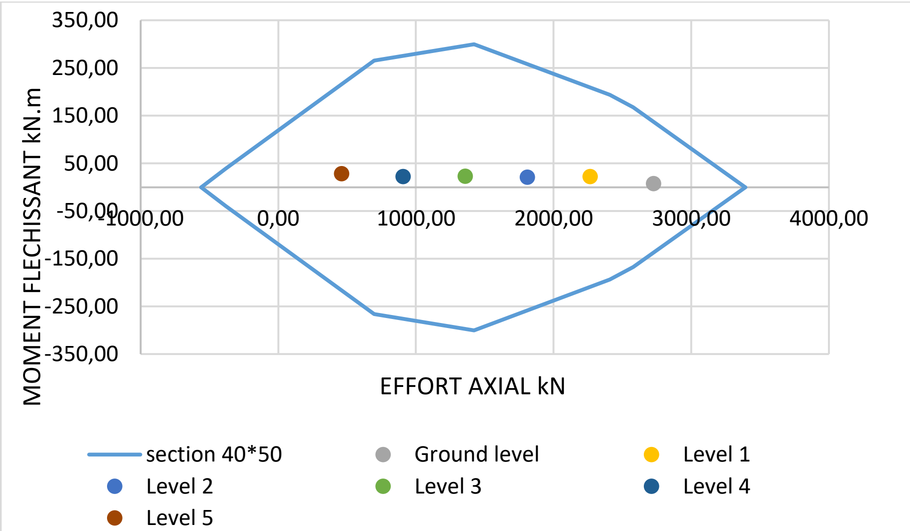
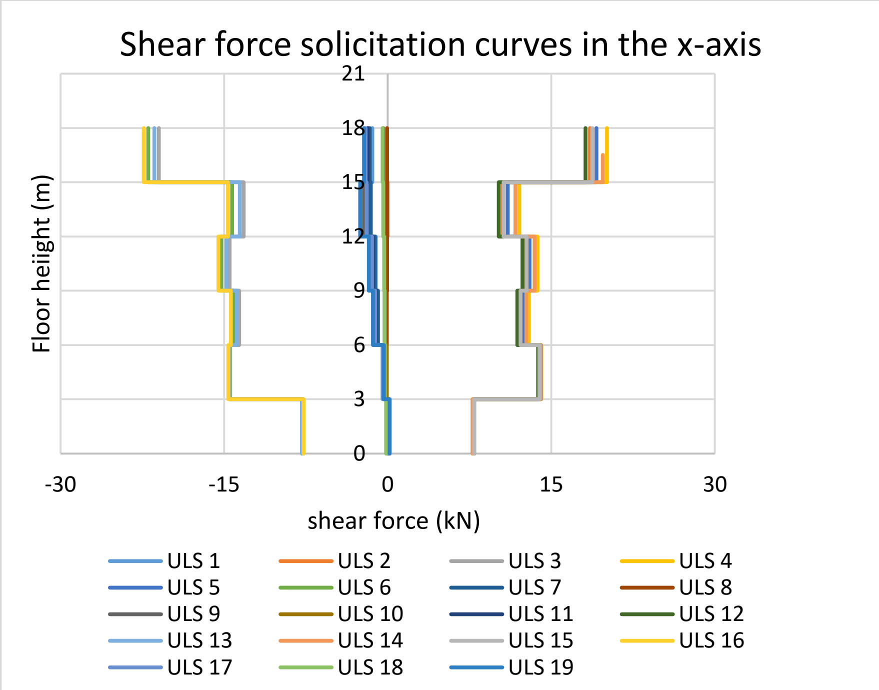


Figure 3.78. Interaction diagram of the column B5 in the y-direction

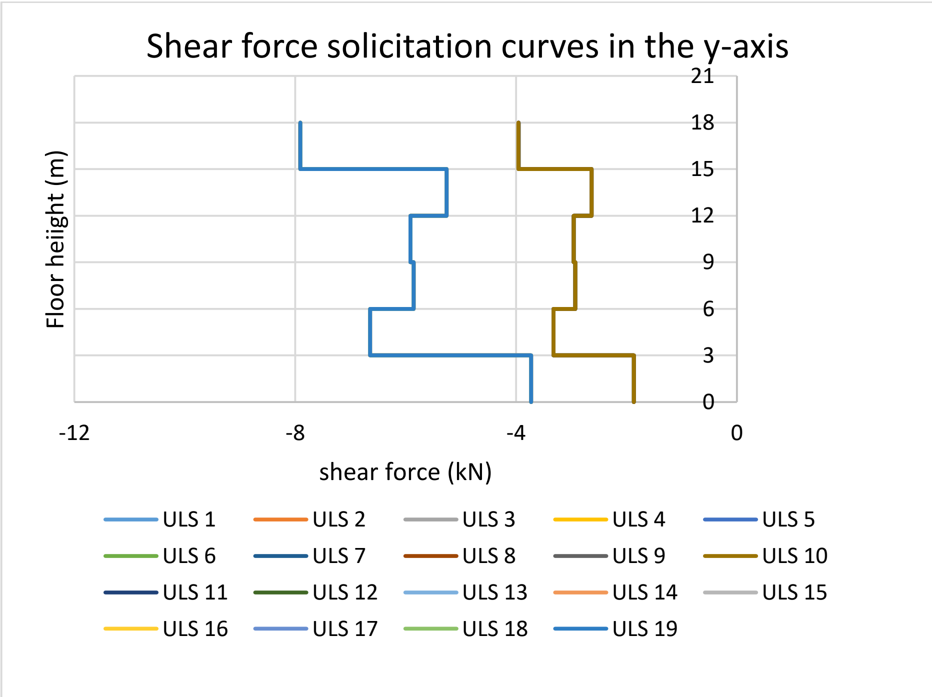
We observe that the bending moment and the axial force solicitations of the column is inside the diagram so the section is correct.

3.5.2.3. Shear verification

The different loads arrangements permits to obtain the shear solicitation curves in the x and y-direction as presented in figure 3.79.



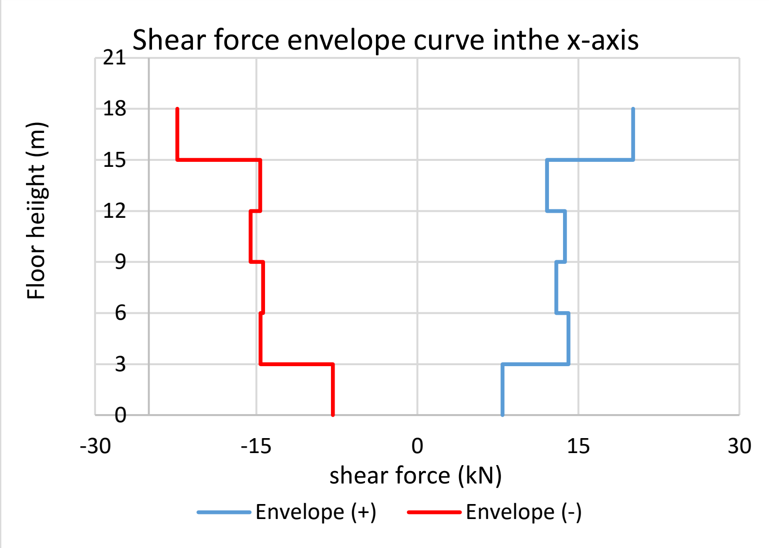
(a) in the x-axis



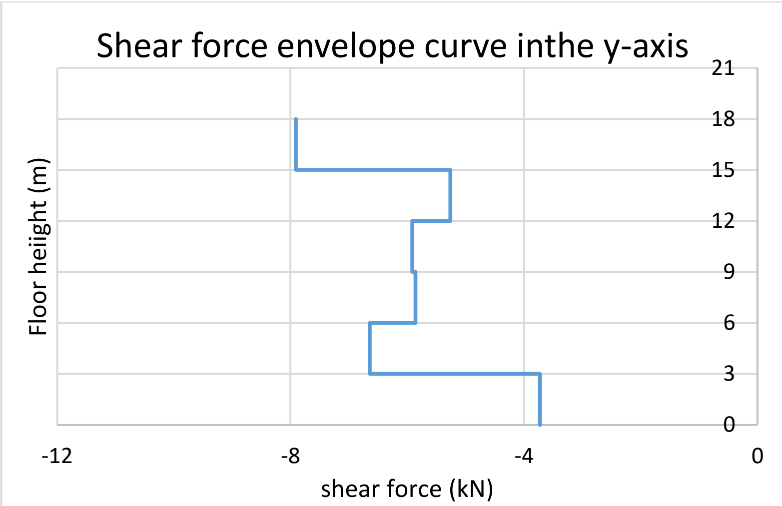
(b) in the y-axis

Figure 3.79. Shear force solicitation curve on the column: (a) in the x-axis; (b) in the y-axis

The envelope of these solicitation curves is then obtained and presented in figure 3.80.



(a) in the x-axis



(b) in the y-axis

Figure 3.80. Shear force envelope curve on the column: (a) in the x-axis; (b) in the y-axis

Applying the procedure presented in the section 2.4.2.3.c, we observe that the shear resistance of the section without shear reinforcement is greater than the maximum shear solicitation on the column so the detailing of members has to be applied to have the spacing. In our case, we consider a diameter of 8 mm and the maximum spacing of the transverse reinforcement is given by:

$S_{cl,} = \min(320; 400; 400) = 320mm$

So, applying the prescriptions of the section 2.4.2.1.b, we obtain a spacing of the shear reinforcement of: 16 cm within 0.55 cm above and below the beams and 32 cm for the rest of the column.

3.5.2.4. Slenderness verification

Following the procedure presented on the section 2.4.3.4, the different parameters are evaluated and presented in table 3.20.

Table 3.20. Parameter for slenderness verification

A	B	C	n	λ	λ_{lim}
0.7	1.12	0.7	0.50	14.	15.56

From the table 3.19 we have $\lambda < \lambda_{lim}$, so the slenderness of the column is verified.

3.6. Seismic load

3.6.1. Ground condition and seismic action

Soil investigations reveal that the soil consists of medium dense sand and gravel soil with. We can classify the soil as a soil type B from Eurocode 8 for the computation of the seismic action. The peak ground acceleration a_{gR} is equal to 0.25 g. The building is classified as importance class II and the corresponding importance factor amounts to $\gamma_I = 1.0$. The design ground acceleration on type B ground ag is defined as $a_g = \gamma_I * a_{gR} = 1.0 \times 0.25g = 0.25g$. Table 3.21 presents the different parameters used to define the seismic action.

Table 3.21. Seismic action characteristics.

Ground type	Damping %	Soil factor	$a_g(g)$	T_B (s)	T_C (s)	T_D (s)	Behaviour factor q
B	5	1.2	0.25	0.15	0.5	2	1

The elastic spectrum and plotted in the software Excel as presented in the figure 3.81.

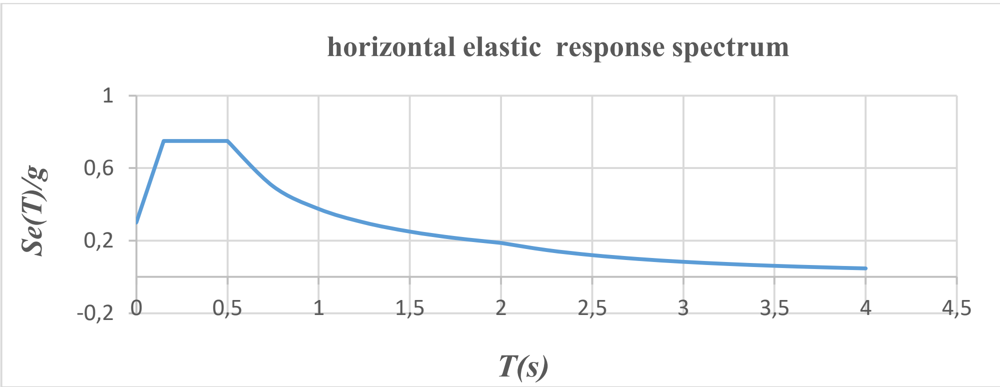


Figure 3.81. Elastic response spectrum used for the design.

3.6.2. Seismic load combination

Height combinations of actions were considered for seismic design of the shear wall that are

Seismic 1: $\sum_k G_k + 0.3 \sum_k Qk + Ex + 0.3Ey$ Eq. (3.3)

Seismic 2: $\sum_k G_k + 0.3 \sum_k Qk + Ex - 0.3Ey$ Eq. (3.4)

Seismic 3: $\sum_k G_k + 0.3 \sum_k Qk - Ex - 0.3Ey$ Eq. (3.5)

Seismic 4: $\sum_k G_k + 0.3 \sum_k Qk - Ex + 0.3Ey$ Eq. (3.6)

Seismic 5: $\sum_k G_k + 0.3 \sum_k Qk + 0.3Ex + Ey$ Eq. (3.7)

Seismic 6: $\sum_k G_k + 0.3 \sum_k Qk - 0.3Ex + Ey$ Eq. (3.8)

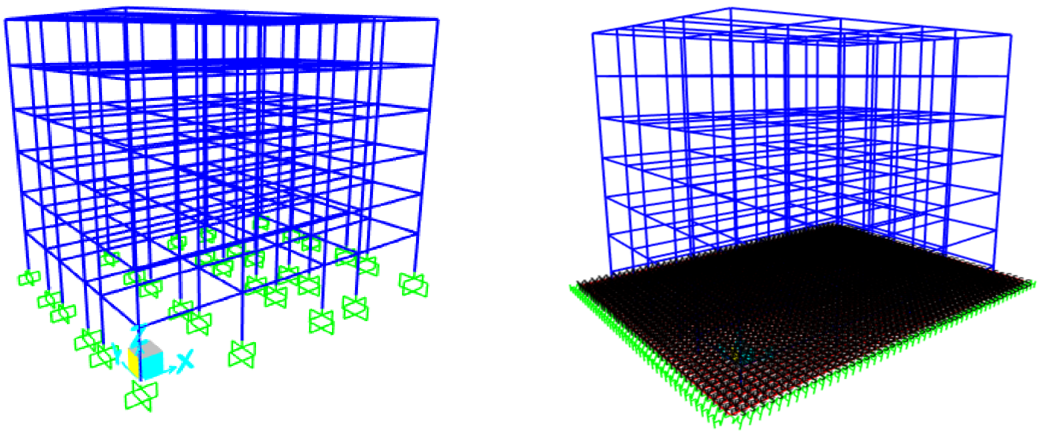
Seismic 7: $\sum_k G_k + 0.3 \sum_k Qk + 0.3Ex - Ey$ Eq. (3.9)

Seismic 8: $\sum_k G_k + 0.3 \sum_k Qk - 0.3Ex - Ey$ Eq. (3.10)

3.7. Analysis response

This parametric study carries out the response of the model in terms of the period, the lateral deformation of the building, the inter-story drift and the shear base. The analysis will be do with two different foundations (fixed base and raft foundation). Figure 3.82 and 3.83 show the hollow block floor model and the wood-concrete model for the fixed base and raft foundation respectively. The raft foundation is modelled with shell element is meshed for an element of 0.5 by 0.5 and a thickness of 0.5m with a spring constant in the x and y direction to be:

$k_{x,y,middle} = C \times A = 30000\ kN/m$ $\quad k_{x,y,edge} = 15000\ kN/m$



(a) model with fixed base (b) model with raft foundation

Figure 3.82. Model with fixed base (a) and raft foundation (b)

3.7.1. Building vibration period

The vibration period is an important parameter of a structure to estimate its seismic demand. Modern building codes generally use the period ratio (flexible base period, $\tilde{T}$, to the fixed-base period, T) of buildings to assess their response to seismic loadings. Table 3.22 and 3.23 provides the vibration periods in case of fixed base and raft foundation respectively of the building with different floors.

Table 3.22. Vibration periods of the building with fixed base

	Hollow block floor	Wood-concrete floor	Units
Mode 1	0.39974	0.44564	(s)
Mode 2	0.38112	0.41206	(s)
Mode 3	0.36796	0.40622	(s)

Table 3.23. Vibration periods of the building with raft foundation

	Hollow block floor	Wood-concrete floor	Units
Mode 1	0.45613	0.50845	(s)
Mode 2	0.42836	0.48902	(s)
Mode 3	0.41544	0.4648	(s)

As expected, the results of the analysis with fixed base presented in table 3.22 indicate that the building with hollow block floor is clearly stiffer than the building with wood concrete floor. For the raft foundation, the table 3.23 indicate that the models follows the same result like with fixed base. From the tables 3.22 and 3.23 we see that the raft foundation increases the periods of the structures compared to the fixed base.

3.7.2. Lateral deformation

The results of the 3D numerical predictions for the maximum lateral deflections of the structure with hollow block floor and wood-concrete floor are evaluated for each direction. The values of the lateral deflection for each story in x-direction are presented in table 3.24 and 3.25 and plotted in the figure 3.83 and 3.84. the figure 3.83 shows the definition of axis.

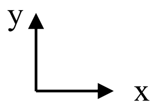


Figure 3. 83. Axis

Table 3.24. Relative displacement of the structure in x-direction with fixed base

Floor height (m)	Relative displacement in the x-direction (mm)	
h	Hollow block floor	Wood-concrete floor
0	0	0
3	13.6	15.9
6	36.1	40.1
9	58.2	63
12	76.5	81.9
15	89.6	95.5
18	97.3	103.2

Table 3.25. Relative displacement of the structure in x-direction with raft foundation

Floor height (m)	Relative displacement in the x-direction (mm)	
h	Hollow block floor	Wood-concrete floor
0	0	0
3	39.5	49.2
6	67.5	78.9
9	93.4	105.4
12	114.7	127
15	130.3	142.7
18	139.6	152.2

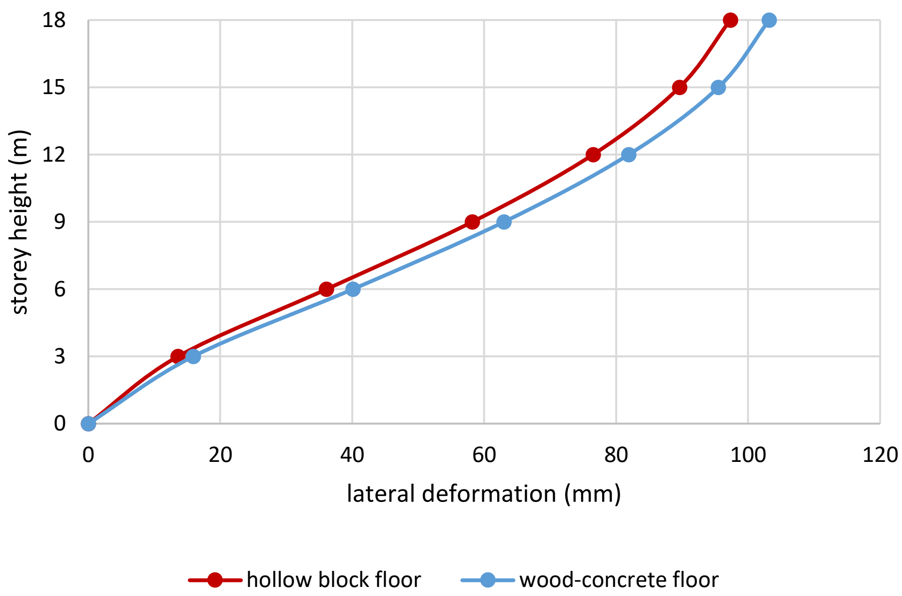


Figure 3.84. Maximum lateral deformation of the structure for the different types of floor in x-direction with fixed base

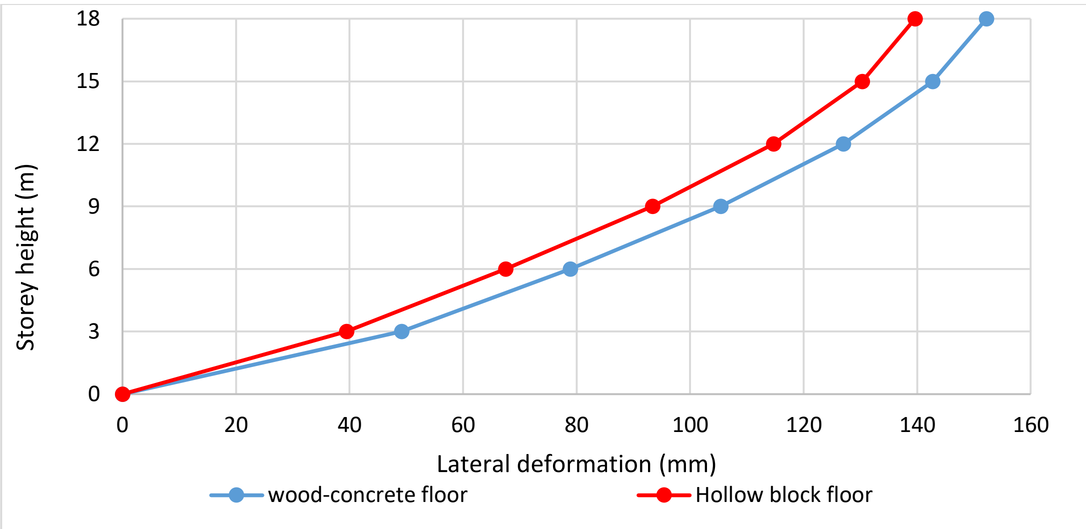


Figure 3.85. Maximum lateral deformation of the structure for the different types of floor in x-direction with raft foundation

Referring to figure 3.83, the building with wood-concrete floor has large values of lateral displacement compared to building with hollow block floor. So wood concrete floor increase the lateral deformation of the building.

We can conclude that for the two different foundations, the wood-concrete floor give the large values for lateral displacements compared to hollow block floor.

The relative displacements obtained in the y-direction are presented in the table 3.26 and 3.27 and plotted in the figure 3.85 and 3.86.

Table 3.26. Relative displacement of the structure in y-direction with fixed base

Floor height (m)	Relative displacement in the y-direction	
h	Hollow block floor (mm)	Wood-concrete floor (mm)
0	0	0
3	14	18.8
6	33.8	44.9
9	52.1	69.1
12	67.2	89
15	77.8	103.3
18	83.6	111.3

Table 3.27. Relative displacement of the structure in y-direction with raft foundation

Floor height (m)	Relative displacement in the y-direction	
h	Hollow block floor (mm)	Wood-concrete floor (mm)
0	0	0
3	34.7	44.6
6	58.5	75.7
9	79.8	103.4
12	97.3	126.2
15	110.1	142.9
18	117.4	152.9

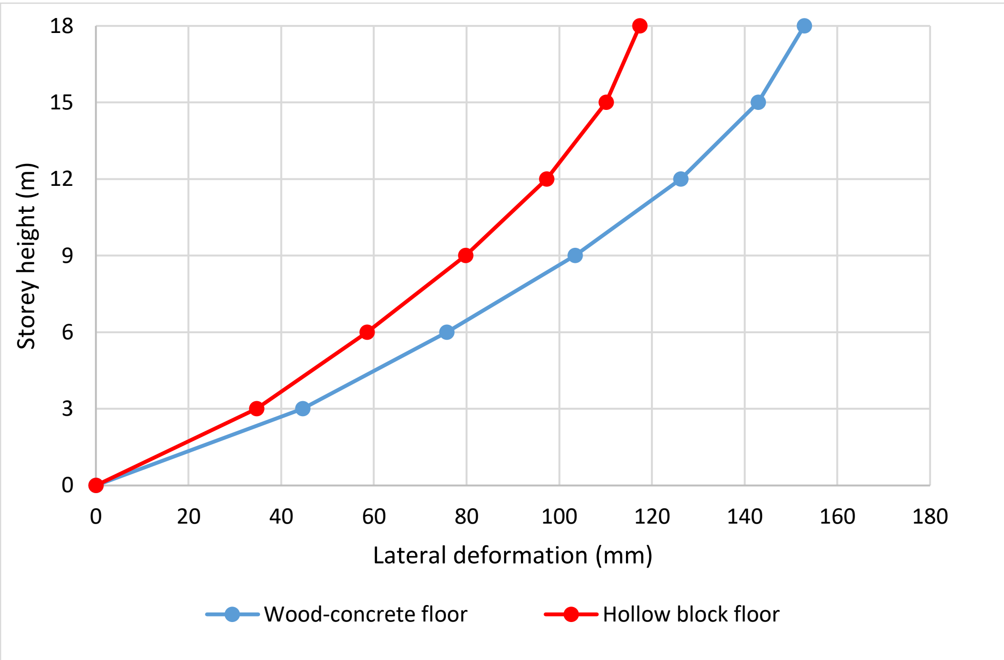


Figure 3.86. Maximum lateral deformation of the structure for the different types of floor in y-direction with fixed base

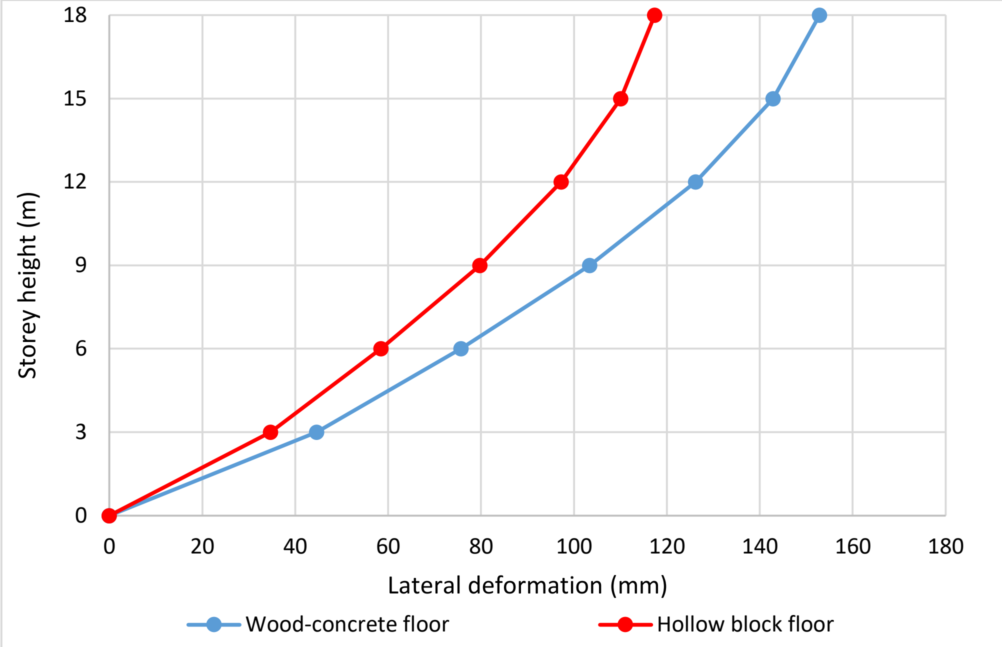


Figure 3.87. Maximum lateral deformation of the structure for the different types of floor in y-direction with raft foundation

Even in the y-direction the wood-concrete floor gives the largest values of the lateral deformation with fixed base and raft foundation. It can be concluded that the type of floor influences the lateral deformation of the structure. From the results above, choosing hollow block floor is an option to control the lateral deformation compared to the wood-concrete floor.

3.7.3. Inter-story drift

The inter-storey drifts are defined as the difference between the lateral deflections of two adjacent stories divided by the height of that storey. For this class of building with brittle materials and a reduction factor $\nu = 0.4$, the limit computed following Eurocodes is 1.25%. Table 3.28 and 3.29 presents the corresponding maximum inter-story drift for the different types of floor along x with fixed base and raft foundation respectively.

Table 3.28. Maximum inter-story drift for different types of floor along x with fixed base

Floor height (m)	Inter-story drift (%) along x	
h	Hollow block floor	Wood-concrete floor
0	0,453	0,53
3	0,75	0,806
6	0,736	0,763
9	0,61	0,63
12	0,436	0,453
15	0,256	0,256
18	0,256	0,256

Table 3.29. Maximum inter-story drift for different types of floor along x with raft foundation

Floor height (m)	Inter-story drift (%) along x	
h	Hollow block floor	Wood-concrete floor
0	1,316	1,64
3	0,933	0,99
6	0,863	0,883
9	0,71	0,72
12	0,52	0,523
15	0,31	0,316
18	0,31	0,316

These values permit to plot the inter-story drift in the x-direction presented in the figure 3.87 and 3.88 for fixed base and raft foundation respectively.

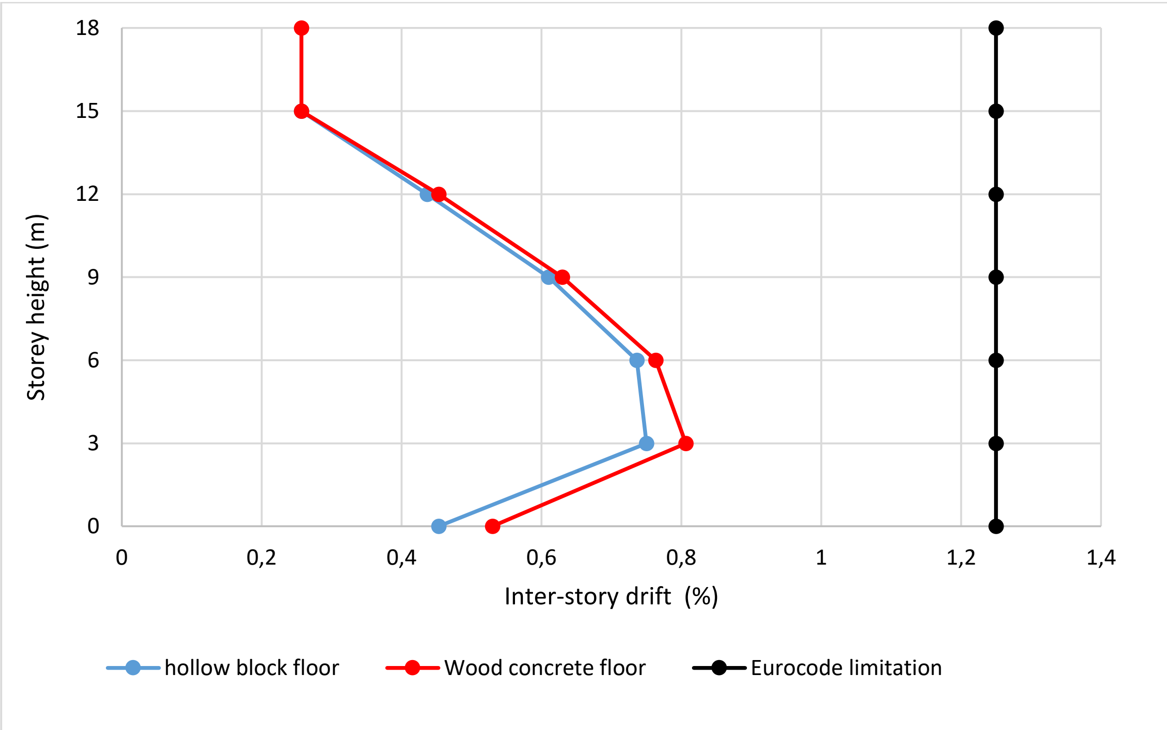


Figure 3.88. Maximum inter-story drift for the different type of floor along x with fixed base

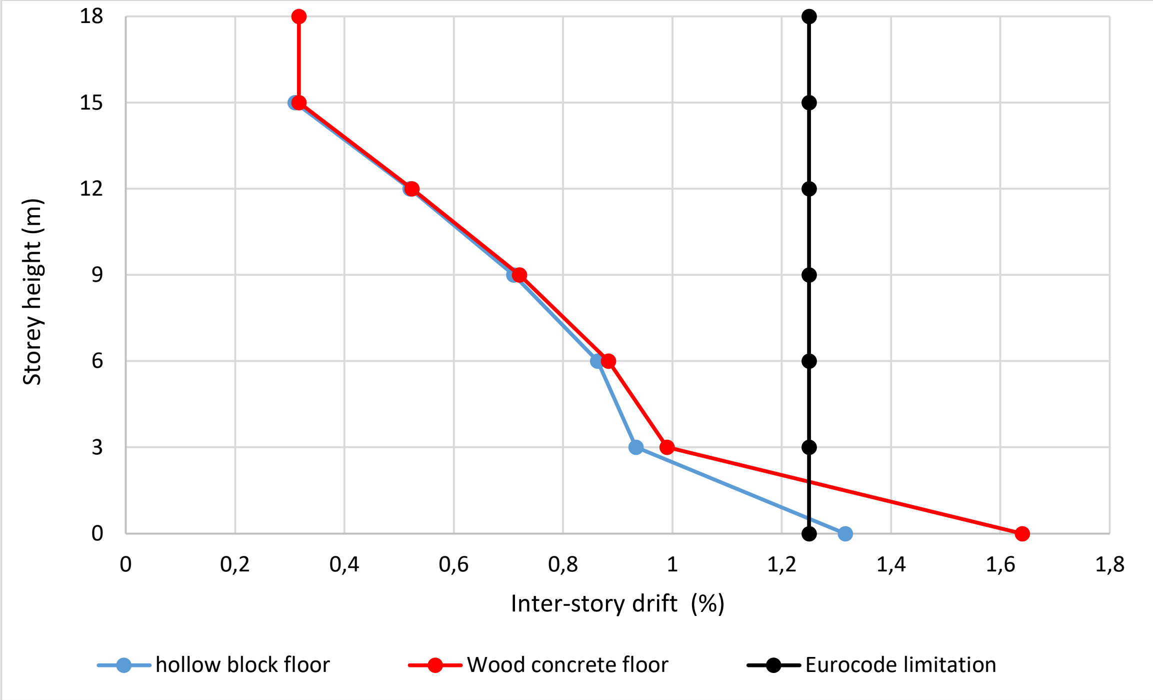


Figure 3.89. Maximum inter-story drift for the different type of floor along x with raft foundation

Table 3.30 and 3.31 presents the corresponding maximum inter-story drift for the different types of floor along y with fixed base and raft foundation respectively. The comparison of the drift along y-direction for the two floor is illustrated in the figure 3.89 and 3.90.

Table 3.30. Maximum inter-story drift for different types of floor along y with fixed base

Floor height (m)	Inter-story drift (%) along y	
h	Hollow block floor	Wood-concrete floor
0	0,466	0,626
3	0,66	0,87
6	0,61	0,806
9	0,503	0,663
12	0,353	0,476
15	0,193	0,266
18	0,193	0,266

Table 3.31. Maximum inter-story drift for different types of floors along y with raft foundation

Floor height (m)	Inter-story drift (%) along y	
h	Hollow block floor	Wood-concrete floor
0	1,156	1,486
3	0,793	1,036
6	0,71	0,923
9	0,583	0,76
12	0,426	0,556
15	0,243	0,333
18	0,243	0,333

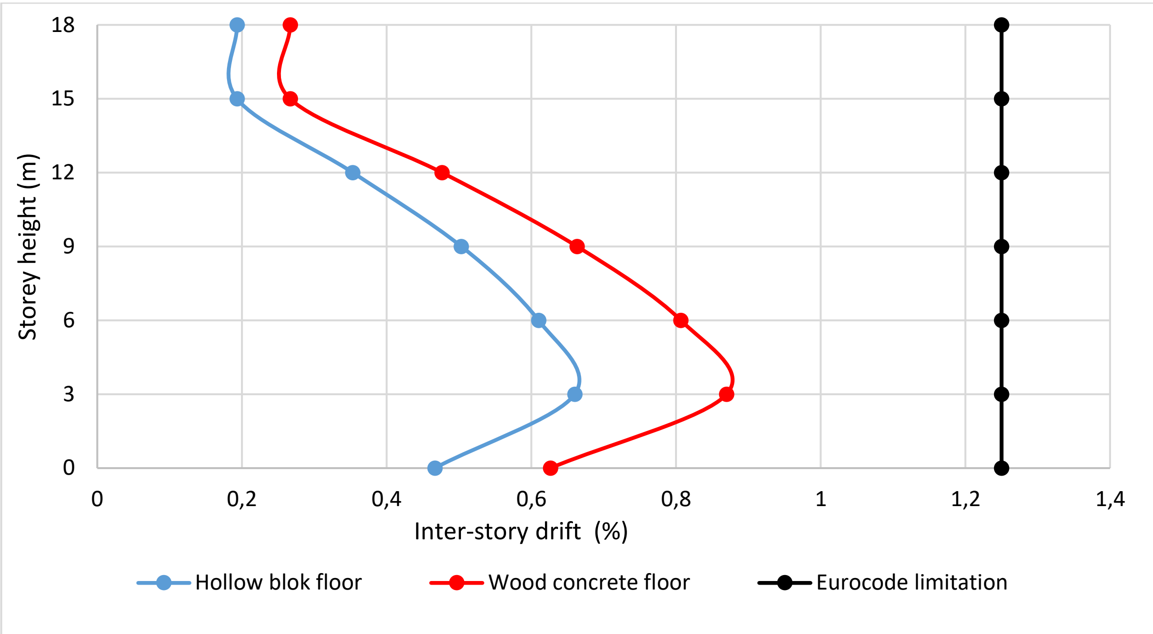


Figure 3.90. Maximum inter-story drift for the different type of floor along y with fixed bas

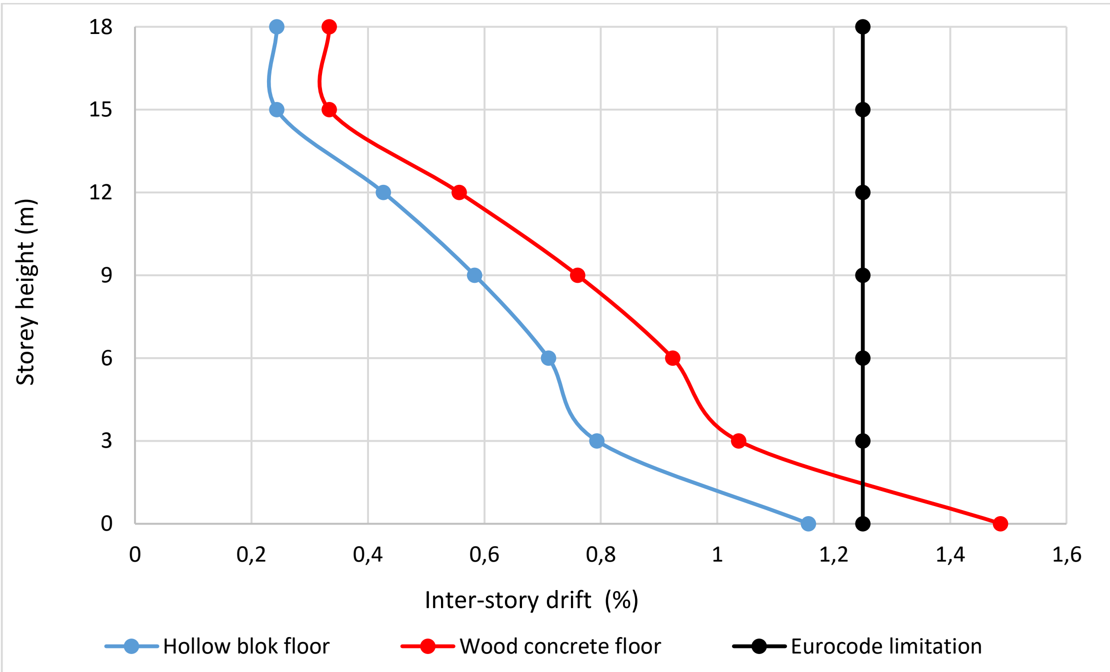


Figure 3.91. Maximum inter-story drift for the different type of floor along y with raft foundation

From the figure 3.89, wood-concrete floor tends to increase the inter-storey drifts of the superstructure, although the maximum inter-storey drift along x is less than 1.25 % which is the limit prescribed by Eurocode. For instance, at the heights 15 m and 18 m the values of inter-story are the same for the two floors. The maximum drift of the wood concrete floor along x is 0.806%, while the corresponding values for the hollow block floor is 0.75%. From the figure 3.87, wood concrete floor increase also the inter-storey drifts of the structure and the maximums inter-storey are greater than 1.25% for the two floor. So the fixed base is better than the raft foundation.

Along y-direction, the maximum inter-storey given by wood-concrete floor and hollow block floor are equal to 0.87% and 0.66 respectively, less than the limit obtained from Eurocode formulation. From the figure 3.89, we see that the maximum drift for wood concrete is greater than 1.25%. As a result, hollow block floor is an option to control the drift under shaking rather than using wood-concrete floor with fixed base and raft foundation.

3.7.4. Base shear

The shear forces experienced at the ground level due to response spectrum are presented in table 3.32 and 3.33 with fixed base and raft foundation respectively.

Table 3.32. Base shear from response spectrum with fixed base

Shear base (kN)	Hollow block floor	Wood-concrete floor
X direction	8418.725	12137.748
Y direction	8575.26	12228.395

Table 3.33. Base shear from response spectrum with raft foundation

Shear base (kN)	Hollow block floor	Wood-concrete floor
X direction	15027.188	16321.87
Y direction	14594.133	15634.438

From the table 3.32 and 3.33 we see that the shear forces in directions X and Y are largest in the wood-concrete floor. So the wood-concrete increase the shear force of the structure with fixed base and raft foundation.

Conclusion

The aim of this chapter was to present the case study, to perform the analysis and the design of the structural elements and to compare the influence of the type of floor on building. At the end a section of 54 cm height and 20 cm width is obtained for beams of hollow block floor and a section of 50 cm by 50 cm for the most solicitated column at the basement level. Concerning the structure, two types of floor were designed which are hollow block floor and wood-concrete floor. The analysis of building is done with fixed base and raft foundation. At the end, we found that the hollow block floor underestimates, the period, the lateral deformation, the base shear and the storey-drift compared to the wood concrete floor.

GENERAL CONCLUSION

The aim of our work was to carry out the influence of the type of floor (concrete, wood) on structural behaviour of buildings. This study has been done firstly through a literature review of types of floor and their functions. Secondary, the methodology of the analysis and design of structural elements was presented. Then following this methodology, a regular six storey building were designed. Two types of foundations were proposed for that building. The analysis was performed through the software SAP 2000 (version 22).

The results obtained from the analysis revealed that: **(1)** The vibration periods are large for wood-concrete floor compared to hollow block floor with fixed base and raft foundation. **(2)** The wood concrete floor amplifies the lateral deformation and the of the building when we have fixed base and raft foundation. **(3)** The inter-storey drift follows the same pattern as the lateral deformation with the largest value given by the wood concrete floor. **(4)** In general, the ratio of the structural shear for cases including the wood concrete floor amplifies the base shear compared to hollow block floor with the fixed-base and raft foundation, demonstrating that hollow block floor reduces shear forces in the structure with fixed base and raft foundation.

It can be concluded that the type of floor influences the structural behaviour of buildings. To assure continuity in the research it is suggested to design the building with hollow block floor. The latter would be an effective solution in our country. However, the study of its fire resistance, insulation and pushover of building could be the subject of another analysis.

BIBLIOGRAPHY

BOOKS

Davidovici, V., Corvez, D., Ghavamian, S., Corvec, V., Saintjean, C. (2015). *Pratique du calcul sismique : guide d'application de l'eurocode 8* (3 eme edition). Paris : Errolles.

Macginley, T. J., Choo, B. S., (1990). *Reinforced concrete: design theory and examples* (2nd edition). singapore : spon press.

Prab, B. T., Choo, B. S., (2006). *Reinforced concrete: design theory and examples* (3rd edition). USA and Canada : Taylors and francis.

Taranath, B. S. (2013). Tall Building Design: Steel, Concrete, and Composite Systems. In *Aquatic Procedia* (Vol. 1).

ARTICLES

NASSANİ, D. E. (2020). lateral loads resisting system in high-rise in reinforced buildings. *European Journal of Science and Technology*, *October*.

https://doi.org/10.31590/ejosat.808269

Viktor, C. S., & Stefan, S. (2016). *Methodology for Preliminary Design of High-Rise Buildings*. http://lup.lub.lu.se/luur/download?func=downloadFile&recordOId=8884631&fileOId=8885053%0Ahttp://files/1126/download.pdf

Viktor Castlen Rist, S. S. (2016). *Methodology For Preliminary Design Of High-Rise Buildings*.

NORMS

Comité Européen de Normalisation. (2002). *Eurocode 0: Basis of structural design. Norm EN 1990.* Brussels: Comité Européen de Normalisation.

Comité Européen de Normalisation. (2002). *Eurocode 1: Actions on structures. Norm EN 1991.* Brussels: Comité Européen de Normalisation.

Comité Européen de Normalisation. (2002*). Eurocode 2: Design of concrete structures. Norm EN 1992.* Brussels: Comité Européen de Normalisation.

Comité Européen de Normalisation. (2002). *Eurocode 7: Geotechnical design. Norm EN 1997.* Brussels: Comité Européen de Normalisation.

Comité Européen de Normalisation. (2002). *Eurocode 8: Design of structures for earthquake resistance.* Brussels: Comité Européen de Normalisation.

THESIS

Bile Bile, A. B. R. (2019). *Finite element method analysis applied to the study of joints in reinforced concrete structures* (master thesis). National advanced school of public work.

Biloa Bijanga, B. R. (2019). *Numerical modelling of composite timber-concrete structure: case study of timber concrete slab* (master thesis). National advanced school of public work.

Djeukoua Nathou, G. L. (2018). *Comparative analysis of seismic protection systems* (master thesis). National advanced school of public work.

Elsherbeny, H. A. (2016). *Behavior of Reinforced Concrete Flexible Floor Diaphragms under Seismic Loads By Eng . Hesham Ahmed El Sherbeny B . Sc . in Civil Eng . 1992 A Thesis Submitted to the Faculty of Engineering at Cairo University in Partial Fulfillment of Requirements for the Degree of in*. *April*. https://doi.org/10.13140/RG.2.1.1995.3046

Moustapha Housseni (2019). *Methods of analysis and seismic design of a 5 storey irregular reinforce concrete building* (master thesis). National advanced school of public work

Nsanji Bungong, D. (2018). *Methods of analysis and seismic design of a 5 storey irregular reinforce concrete building* (master thesis). National advanced school of public work

Viktor, C. S., & Stefan, S. (2016). *Methodology for Preliminary Design of High-Rise Buildings*. http://lup.lub.lu.se/luur/download?func=downloadFile&recordOId=8884631&fileOId=8885053%0Ahttp://files/1126/download.pdf

WEBOGRAPHY

https://Civilread.com//16-different-types –of-slab-construction

ANNEXES

Tables for methodology

Categories of use of the building (EC 1 Part 1)

Category	Specific Use	Example
A	Areas for domestic and residential activities	Rooms in residential buildings and houses; bedrooms and wards in hospitals; bedrooms in hotels and hostels kitchens and toilets.
B	Office areas	
C	Areas where people may congregate (with the exception of areas defined under category A, B, and D[1])	**C1**: Areas with tables, etc. e.g. areas in schools, cafés, restaurants, dining halls, reading rooms, receptions. **C2**: Areas with fixed seats, e.g. areas in churches, theatres or cinemas, conference rooms, lecture halls, assembly halls, waiting rooms, railway waiting rooms. **C3**: Areas without obstacles for moving people, e.g. areas in museums, exhibition rooms, etc. and access areas in public and administration buildings, hotels, hospitals, railway station forecourts. **C4**: Areas with possible physical activities, e.g. dance halls, gymnastic rooms, stages. **C5**: Areas susceptible to large crowds, e.g. in buildings for public events like concert halls, sports halls including stands, terraces and access areas and railway platforms.
D	Shopping areas	**D1**: Areas in general retail shops **D2**: Areas in department stores
[1] Attention is drawn to 6.3.1.1(2), in particular for C4 and C5. See EN 1990 when dynamic effects need to be considered. For Category E, see Table 6.3		
NOTE 1 Depending on their anticipated uses, areas likely to be categorised as C2, C3, C4 may be categorised as C5 by decision of the client and/or National annex. NOTE 2 The National annex may provide sub categories to A, B, C1 to C5, D1 and D2 NOTE 3 See 6.3.2 for storage or industrial activity		

Values of Minimum cover, $Cmin$,, requirements with regard to durability for reinforcement steel (EC2)

Environmental Requirement for $c_{min,dur}$ (mm)							
Structural Class	Exposure Class according to Table 4.1						
	X0	XC1	XC2 / XC3	XC4	XD1 / XS1	XD2 / XS2	XD3 / XS3
S1	10	10	10	15	20	25	30
S2	10	10	15	20	25	30	35
S3	10	10	20	25	30	35	40
S4	10	15	25	30	35	40	45
S5	15	20	30	35	40	45	50
S6	20	25	35	40	45	50	55

Values of subgrade modulus for different soil type.

	Nature du sol	*C* (t/m^3)
1	terrain légèrement tourbeux et marécageux	500- 1 000
2	terrain essentiellement tourbeux et marécageux	1 000- 1 500
3	sable fin	1 000- 1 500
4	remblais d'humus, sable et gravier	1 000- 2 000
5	sol argileux détrempé	2 000- 3 000
6	sol argileux humide	4 000- 5 000
7	sol argileux sec	6 000- 8 000
8	sol argileux très sec	10 000
9	terrain compacté contenant de l'humus du sable et peu de pierres	8 000-10 000
10	même nature que ci-dessus avec beaucoup de pierres	10 000-12 000
11	gravier fin et beaucoup de sable fin	8 000-10 000
12	gravier moyen et sable fin	10 000-12 000
13	gravier moyen et sable grossier	12 000-15 000
14	gros gravier et sable grossier	15 000-20 000
15	gros gravier et peu de sable	15 000-20 000
16	gros gravier et peu de sable mais très compacté	20 000-25 000