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DEPARTEMENT DE GENIE CIVIL

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Università degli Studi di Padova

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DEPARTMENT OF CIVIL, ARCHITECTURAL AND ENVIRONNEMENTAL ENGINEERING

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INFLUENCE OF THE TYPOLOGY OF INFILL WALLS IN THE DESIGN OF REINFORCED CONCRETE BUILDINGS:CONCEIVED CASE STUDY

A thesis submitted in partial fulfilment of the requirements for the degree of Master of Engineering (MEng) in Civil Engineering

Curriculum: Structural engineering

Presented by

Ndjeutcha Tieyam Joelle Natacha

Student number: 15TP21041

Supervised by:

Prof. Carmelo MAJORANA

Co-Supervised by:

Eng. Giuseppe CARDILLO

Dr. Eng. Guillaume Hervé POH'SIE

ACADEMIC YEAR: 2020/2021

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DEDICATION

To

MY PARENTS

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

ACKNOWLEGEMENTS

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:

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- All people who, from near or far have given me their heartfelt help in carrying out this research

LIST OF ABBREVIATIONS AND SYMBOLS

LIST OF ABBREVIATIONS

EC Eurocode

FEMA Federal Emergency Management Agency

RC Reinforced Concrete

SAP Structural Analysis Program

SLS Serviceability Limit State

SSI Soil-Structure-Interaction

ULS Ultimate Limit State

LIST OF SYMBOLS

A Area of the cross section

a infill wall strut representation width

Ac Area of the concrete cross section

$Amin$ Minimum section area

$Anet$ Net area of the cross section

As Area of the steel reinforcement section

Asw Cross sectional area of the shear reinforcement

B Foundation half width

C Modulus of subgrade reaction of the soil

$Cmin$ Minimum concrete cover

D Embedment depth

FEd, Design support reaction

Fb Shear force

G Shear modulus

G1k Structural load of the building

G2k Non-structural load apply on the building

Ixx Moment of inertia of the section along x axis

Iyy Moment of inertia of the section along y axis

Kj Static stiffness

L Foundation half length

Nc Bearing capacity factor

N_q Bearing capacity factor

M_{sd+} Positive moment at mid-span in hyperstatic condition

M_{sd-} Negative moment at support in hyperstatic condition

M_{Ed} Bending moment at support

M_{Rd} Resisting moment

M_{Sd} Soliciting bending moment

T Period of the structure with fixed base

V_s Shear velocity

W Weight of the structure

b Width of the element

bt Mean width of the tension zone

b_w Smallest width of the cross section in the tensile area

c Concrete cover

d Effective height of the section

f_{cd} Design resisting strength of the concrete

f_{ctm} Tensile strength of the concrete

f_y Design yielding strength of the steel

f_{yd} Design yielding strength of the steel

f_{yk} Characteristic yield strength

f_{ywd} Design yield strength of the shear reinforcement

g_k Permanent load applied on the building

h Height of the level i

h_{wi} height of the wall i;

i Gyration radius of the uncracked concrete section

j Index denoting the mode of vibration

k stiffness of the structure

l is the span length of the beam

lo Effective length of the element

l_{wi} is the length of the section of the wall i

sl, Maximum longitudinal spacing

st, Maximum transversal spacing

$\Delta Cdur\ add$ Add reduction of minimum cover for use of additional protection

ΔC_{dur}, Reduction of minimum cover for use of stainless steel

ΔC_{dur}, Additive safety element

$\emptyset l$, Minimum diameter of the longitudinal bars

σ Contact pressure

σ_{adm} Admissible pressure on the soil

γ Specific weight

γc Partial factor for concrete

γs Partial safety factor for steel

Ψ_{E}, Combination coefficient for variable action

λ Slenderness

λ_1 strut width coefficient

λ_{lim} Limit value of slenderness

ν Poisson's ratio

ρ_w Shear reinforcement ratio

ρ_w, Minimum shear reinforcement ratio

σ_c Stress in the concrete

σ_s Stress in the reinforcement

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

S_r the recovery area of the column;

n the number of stories above the considered column

N_R is the design axial compression force

ABSTRACT

The main objective of this work was to study the influence of infill walls in the design of reinforced concrete buildings. A literature review was first carried out to highlight about the importance of infill walls, their failure mechanisms, the characteristics of most used infill walls, the type of RC buildings and the different types of failure mechanisms of RC buildings. Secondly our conceived case study which was a G+3 reinforced concrete residential building was modelled under static load condition. Three types of infill walls namely Drywall, compressed earth brick wall and masonry wall have been chosen for the analysis because of their frequency of use in the construction field. Infill walls were modelled as thin shell for the micro model and modelled as strut for the macro model. The struts characteristics were computed according to the FEMA 356 and horizontal and vertical structural elements were designed according to the Eurocodes. A finite element analysis was performed on the structure with fixed base in other to study the lateral and the vertical deformability at each level. The different analysis have been performed on a frame of the building and the whole building with and without infill walls using the software SAP2000 v22 . A comparative study has been performed between frame without infill wall and frame with infill wall especially compressed earth brick wall, drywall and masonry wall. The results showed that the presence of infill wall increases the global stiffness and reduces the horizontal deformability of the building. It also affects the deflection of the horizontal elements. In addition, drywall is lighter and offers a better resistance than the masonry and brick. This work is only limited to the structural aspect, and did not take into account the economical and constructional aspects.

Key words: infill wall, reinforced concrete, stiffness, lateral deformability, deflection.

RESUME

Le but de ce travail était de montrer l'influence qu'ont les murs de remplissage dans le dimensionnement des structures en béton armé. Les murs de remplissage sont des éléments non porteurs servant à séparer les pièces d'un bâtiment et par conséquent sont considérés comme des charges distribuées lors du dimensionnement des structures en béton armé. Dans le but de mener à bien notre étude sur l'influence des murs dans le dimensionnement, plusieurs étapes ont été suivies. Tout d'abord une revue de la littérature portant sur l'importance des murs de remplissages et leurs mécanismes de ruines, les différents caractéristiques des types de murs de remplissage les plus utilisés, les types de bâtiments en béton armé et leurs caractéristiques. Par la suite, le cas d'étude qui est un immeuble R+3 à usage d'habitation a été modélisé et dimensionné sous chargement statique selon la norme européenne Eurocode. Trois types de mur de remplissage ont été sélectionnés pour les analyses structurelles notamment les murs en maçonnerie, les murs en briques de terre compressées, et les murs en Placoplatre ceci à cause de leurs fréquences d'utilisation dans le domaine de la construction. Les murs furent modélisés comme des coques minces à courbature négligeable pour le modèle microscopique, et comme des étais pour le modèle macroscopique. Les dimensions des étais ont été calculé selon la norme américaine FEMA 356. En utilisant la méthode des éléments finis, une analyse a été effectuée dans le but d'évaluer la déformation latérale et verticale de chaque niveau du batiment.les analyses étaient effectuées sur un modèle simple et le modèle de la structure entière à l'aide du logiciel SAP 2000 version 22 en considérant d'une part les murs de remplissage et d'autre part en les ignorants.Enfin, l'étude comparative effectuée en variant les types de murs lorsqu'ils sont considérés dans les analyses a montré que la présence des murs de remplissage augmente la raideur du bâtiment et réduit la déformation latérale du bâtiment. De plus, les murs de remplissage influence aussi la déflexion des poutres.il a été aussi montrer que le Placoplatre offre une meilleure résistance que la brique de terre compressé, suivie de la maçonnerie. Ce travail est limité à l'aspect structurel et ne prend pas en compte l'aspect économique.

Mots-clés : murs de remplissage, béton armé, raideur, déformation latérale, déflexion

LIST OF FIGURES

LIST OF TABLE

TABLE OF CONTENT

GENERAL INTRODUCTION

Concrete frame buildings are more and more used around the world especially in Africa. Hence, researches have been carried out to improve the rigidity and resistance of against wind, earthquakes and fire.Thus, to improve the performance of structural elements,several researchers have been lead to study the behaviour and the influence of structural elements and secondary elements. Non-structural elements especially infill walls are not considered in the design process due to their modelling which is hard and complex. Infill walls are non load bearing walls used to seprate rooms in a buiding. The effect of these walls are modelled as mass and weight which are applied to the floors and the beams of each level.

The phenomenon that grab the attention of the researcher was the cracking observed on infill wallls when the structural frame was subjected to lateral of vertical loads because, according to the design process, all the loads are carried by the structural elements,which transmit loads to each others. Thus the failure should only occurs on structural elements but that is not the case. The first study about the behaviour of frames with infill walls was made by Polyakov in 1956 while considering infill masonry walls. This research showed that the presence of masonry wall influence the stiffness and the period of the building. Holmes (1961) proposed a representation of masonry infill wall as diagonal strut, then Mehrabi et al (1996) proposed a finite element analysis to show the behaviour of masonry walls in RC buildings.

However,masonry is not the only material used for filling and some studies discovered that masonry infill wall present a higher capacity to failure when subjected to earthquake or hurricanes . Therefore some researchers like Fi Wang et al (2021) studied the effect of other types of infill walls on steel and RC buildings.The analysis results were concluent depending on the type of wall and the analysis performed.

The aim of this thesis is to evaluate the effect of different types of infill wall in the design of reinforced concrete building. To attend this objective,this work is divided in three chapters.The first chapter will be a literature review on basic concepts about the infill walls, the type of RC buildings and failure mechanism.The second chapter will present a methodology, which will be a guideline for obtention of our results. Finally, the third and last chapter will display the result of the analysis on different frames with and without infill walls.

CHAPTER 1: LITERATURE REVIEW

Introduction

The design and construction technics have evolved with the time passing by as a result of the evolution of the technology, climate changes, increase of the population (there are more and taller building because of the lack of space), and the permanent pursuit of comfort and durability of structures. Thus, the research lead during the last decades on the increase of the RC resistance to external loads in particular to earthquake showed that infill walls have a great impact on the seismic response of the building. Understanding the response of infill walls in reinforced concrete building frames during loading is crucial to the development of an overall efficient and safe structure. However, the behaviour of infill wall depends mostly on the type of material use for infill, and the configuration of the building. This chapter intends to present the concept that are involved in the contribution of infill walls in the resistance of a RC building.

1.1. Infill walls

An infill wall is a supported wall that close the perimeter of a building constructed with three-dimensional framework structure. It serves to separate inner and outer space. Infill walls can be classified according to their function or the material used for their conception. Infill panel walls are a form of cladding built between the structural members of a building. The structural frame provides support for the cladding system, and the cladding provides separation of the internal and external environments. Infill walling is different to other forms of cladding panel in that it is fixed between framing members rather than being attached to the outside of the frame. Moreover, infill walls are not considered to be load bearing, although they are required to resist wind loads applied to the façade, as well as supporting their own weight. Other functional requirements for infill walls include:

- supportiveness between structural framing members;
- weather-resistance;
- thermal and sound insulation;
- fire resistance;
- sufficient openings for natural ventilation and glazing;
- accommodation to differential movements between themselves and the frame;

1.1.1. Importance of infill wall in the structural response of a building

Infill walls are widely acknowledged as a non-structural element. Therefore, only their weight is considered for the design as distributed loads, acting on the structural frame of the buildings. however, the behaviour of an infill panel when the structure is submitted to horizontal force due to earthquakes, tired up the curiosity of the researcher.As shown in Figure 1.1, the infill wall is submitted to compression.

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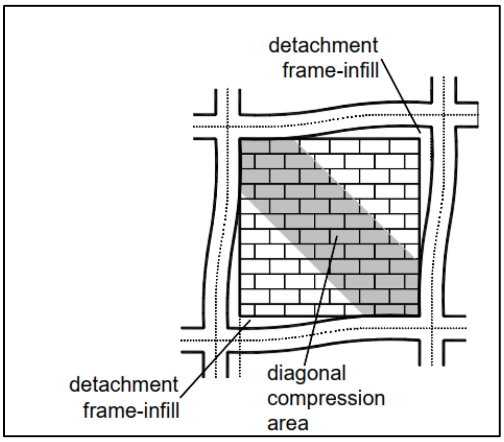


Figure 1.1. Strut model of an infill frame (P.G Asteris et al, 2011)

1.1.2. Failure mechanism of infill walls

Basically, the frame structure takes of the load and transmit it to the soil but when it comes to failure, the behaviour of the structure varies in accordance with the presence of infill panel or not. Some research showed that infill walls are submitted to failure when the frame is submitted to horizontal forces. According to Asteris et al (2011) there are five failure modes of infill wall.

1.1.2.1. The Corner Crushing (CC) mode

This mode represents the crushing of the infill in at least one of its loaded corners, as shown in figure 1.2. This mode is usually associated with in-filled frames consisting of a weak masonry infill panel surrounded by a frame with weak joints and strong members.

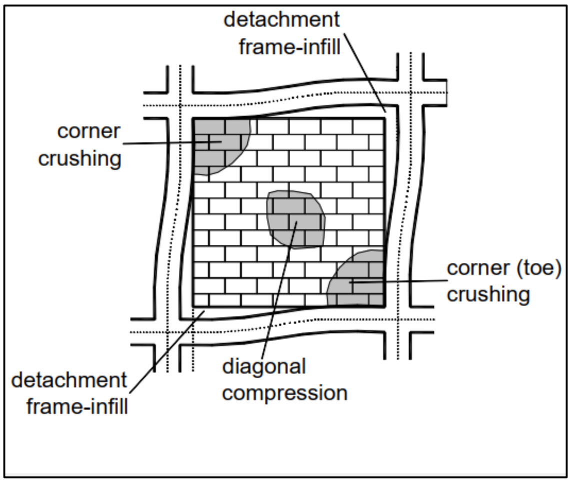


Figure 1.2. Representation of CC mode (Asteris et al, 2011)

1.1.2.2. The Diagonal Compression (DC) mode

This mode represents the crushing of the infill within its central region, as illustrated in figure 1.3, it is associated with a relatively slender infill, where failure results from out-of-plane buckling of the infill.

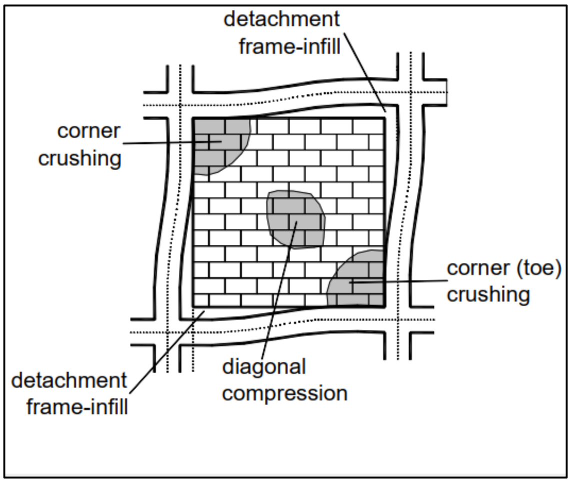


Figure 1.3. Representation of DC mode (Asteris et al, 2011)

1.1.2.3. The Sliding Shear (SS) mode

The sliding shear mode represents horizontal sliding shear failure through bed joints of a masonry infill, as shown in Figure1.4. This mode is associated with infill of weak mortar joints and a strong frame.

1.1.2.4. The Diagonal Cracking (DK) mode,

It is characterized by the formation of a crack across the compressed diagonal of the infill panel and often takes place with simultaneous initiation of the SS mode, as shown in figure1.4 . This mode is associated with a weak frame or a frame with weak joints and strong members in-filled with a rather strong infill.

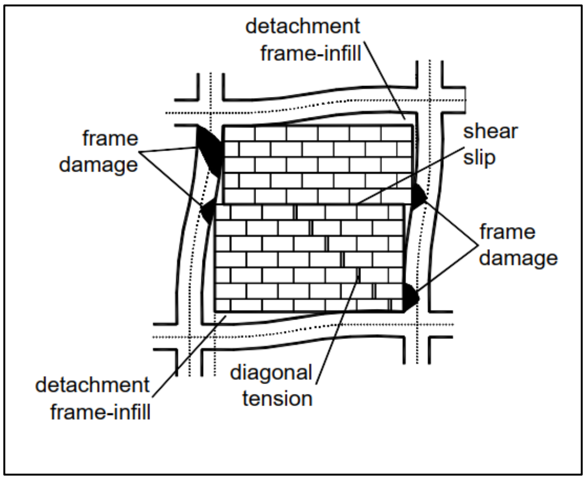


Figure 1.4. Representation of DK mode (Asteris et al. 2011)

1.1.2.5. The Frame Failure (FF) mode,

This type of failure is seen in the form of plastic hinges developing in the columns or the beam-column connections, as shown in figure1.5 . This mode is associated with a weak frame or a frame with weak joints and strong members in-filled with a rather strong infill.

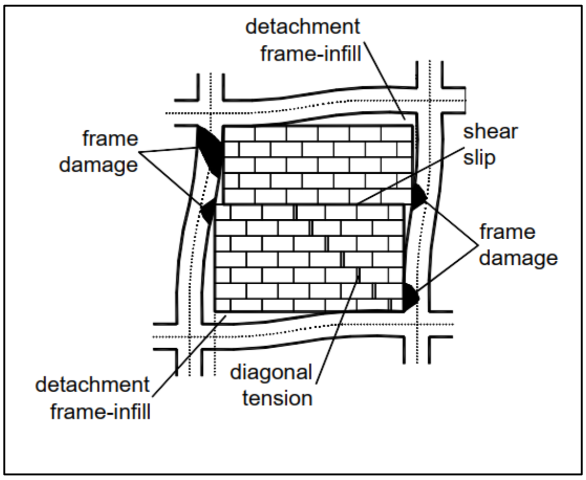


Figure 1.5. Representation of FF mode (Asteris et al. 2011)

1.1.3. Rigidity and deformability of the building

Albayrak et al (2017) presented an overview on the modelling of infill wall in framed structures. it actually showed that the infill walls distribution influences the position of the center of rigidity and the center of mass of the building. The seismic behaviour considering infill wall and without is totally different: the stiffness of the building increase of 40% when infill walls are considered in the analysis.

1.1.4. The structural durability

The durability of a structure is its ability to serve its intended purposes for sufficiency long period of time, or at least during its expected service of life. The structural durability deals with the capability of structure or components to withstand the loads encountered in service over specified period of use without failure occurring in the form of incipient cracks or inacceptable degradation.

1.1.5. Acoustic and thermal comfort

The acoustic and thermal comfort are important parameters that are part of building's comfort and utility criteria. Infill walls have an influence on this characteristic on these elements because the characteristic of the materials use for infill panels vary from one to another. Therefore, it is important to master the acoustic characteristic of the material use for infill in order to know which the best material for each particular building.

1.1.6. Typology of infill walls

There are several types of infill walls and their properties vary from one to another. This paragraph presents the most used types of infill walls.

1.1.6.1. Drywalls

Drywall also known as plasterboard, wall boards, gypsum panel or custard panel) is a panel made of calcium sulphate dehydrate (gypsum) with or without additive, pressed between a facer and a backer.It was invented in 1916 by a company base in America known as the United States Gypsum. Initially, it was used for fireproofing specific areas but within few years, it almost replaced the plaster.

a. Thermal properties

Extensive studies show that the permanent the permanent temperature that damage plaster board is above 81.7°C. However, the plasterboard experiences dehydrating between 60°C and 80°C but restore to ambient level without decreasing the quality level. Nevertheless, with rising temperatures the exposed surface of gypsum loses water and turns into calcium sulphate anhydrite, which falls off the unaltered substrate. As heat penetrates through the thickness, more material transforms to anhydrite powder and consecutive layers are shed. Using glass fiber reinforcements in fire-rated boards delay the ablation of the exposed surface.

b. Wind resistance properties

R.Wolfe (1983) made a research to characterize the resistance of gypsum board to racking force induce by wind. The Results of this study show that gypsum wallboard can provide a significant contribution to wall racking performance. This contribution does not appear to be affected by interactions with wind bracing; however, it does vary with panel orientation and wall length.

c. Acoustic properties

Gypsum board is widely used for any application that requires enhanced soundproofing and sound control. Ideal for interior walls in hotels, restaurants, hospitals, schools and even in home theatres, music rooms, and bedrooms. Whether protecting an inside room from an outside noise, or noise proofing an inside space, acoustic drywall is for wall assemblies and spaces high sound transmission class (STC) performance.

d. Deformability and failure mechanism properties

Experimental tests have confirmed that the as built drywall systems adopted in the current practice for commercial buildings are susceptible to a level of damage which would require repairing interventions at low drift levels. The as built steel framed drywall specimen lost serviceability at 0.3% inter-storey drift level with a ductile post-yield behavior. On the other hand, the timber framed drywall lost serviceability at a higher drift level of 0.75% with a brittle behavior.

1.1.6.2. Masonry infill walls

Masonry walls are walls made of brick concrete blocks held together with mortar made of gypsum, lime, sand and water in the proper proportion. There are different types of masonry walls especially reinforced masonry walls, hollow/cavity masonry walls, composite masonry walls and post tensioned masonry walls. The most used masonry walls as infill walls are hollow/cavity walls because they are not load bearing. Therefore, the next sections will focus on the hollow cement walls properties.

a. Thermal properties

Hollow masonry walls are used to prevent moisture reaching the interior of the building by providing hollow space between outside and inside face of the wall. They also help in the temperature control inside as the hollow space restricts heat to pass through the wall.

b. Wind resistance properties

Masonry walls are susceptible to wind-caused damage especially when subjected to high internal pressures. Internal pressures result when doors or windows are breached on the windward side of the building. The combination of positive pressures on the interior sides of perimeter walls and negative pressures on exterior sides can lead to wall failure. Masonry walls that fail in the wind frequently rotate along horizontal hinge lines at the bases of the walls and separate along vertical joints at window and door openings. Window and door openings interrupt continuity and bridging action, thereby weakening walls.

c. Acoustic properties

The sound insulation of hollow masonry is complex. Nevertheless, masonry walls solution performs well in acoustic test. Mass provides good acoustic isolation and can be often exceed the minimum requirement for building. Therefore, the acoustic performance is achieved with the complement of other elements such as sand (the cavity is filled with the sand to improve the soundproof of the wall) or coating materials.

d. Deformability and failure mechanism properties

Past earthquakes and research demonstrated that the masonry infill walls have advantages in the improvement of energy dissipation as well as increase of stiffness and strength properties of RC structures when they are placed regularly throughout the structure and/or they do not cause shear failures of columns. The masonry wall has a low deformability it can be considered as a brittle element.

1.1.6.3. Glass wall

According to James Stevens Curl and Susan Wilson (2017), glass is a semi- or fully transparent hard, brittle, lustrous material made by igneous fusion of silica (usually sand) with an alkaline sodium or potassium salt and added ingredients. Glass blocks, or bricks, can be used to create architectural structures such as partition walls or screen walls across part of a room, around a shower or bath, as part of stair constructions and so on.

a. Thermal properties

The length and volume of a glass usually increases with increasing temperature. The thermal expansion curve measured by dilatometer gives three important properties – the thermal expansion coefficient (a), glass transformation temperature (Tg), and dilatometric softening temperature (Td). Coefficient of thermal expansion is a measure of the rate of expansion with temperature. For glasses, the measured expansion coefficient is one directional and measured value is linear thermal expansion coefficient (aL). The linear thermal expansion coefficient value of commercial glasses is measure over a specified temperature range and most cases from 0 to 300 °C, 20 to 300 °C, or 25 to 300 °C (Shelby & Lopes, 2005).

b. Wind resistance properties

The wind resistance of glass wall depends on the type of building, the position of the wall, wind of the site. Curtain glass wall are directly subjected to wind loads. There are specific types of glass wall systems such as the Nana Wall SL73-an operable glass system that is design to stand up to hurricane.

c. Acoustic properties

Because of their densities, glass walls are actually good for acoustic isolation. However, the thickness of glass is generally small which reduce their isolation properties.to obtain optimum sound insulation characteristics, insulation glass elements or special laminated safety glass should be used.

d. Deformability and failure mechanism

When glass is under load it will bend and accommodate stress to a certain level and then suddenly fail once its threshold is met. The failure can be sudden and spectacular. Once the crack starts, there is little within structure to stop it propagating. A positive point is that because the inner structure is not mobile, glass does not suffer from dynamic fatigue. Once stress is removed, the glass return unchanged by the experience.

1.1.6.4. Wood walls

These types of wall consist on wood frame-based wall on which is fixed plywood lath, or block wood fixed. Depending on the type of wood used there are plywood walls, timber walls, block board walls. Wood is a reusable material that capture carbon thus reduce the carbon footprint. However, the use of timber for infill of tall building structure decreasing because of the maintenance properties.

a. Thermal properties

Wood expand and contracts with varying temperature. Nevertheless these dimensional variations are small compared with shrinkage and swelling cause by varying moisture content.in most case, such temperature-related expansion and contraction are negligible and without practical importance. Only temperature below 0°C (32°F) have potential causes surface checks. Timber wall exhibits a low thermal conductivity (high heat insulating capacity) compare with other type of wall such as metal wall glass wall or concrete wall. However, when exposed to sufficiency high temperature, wood burns. This property make wood suitable for heating purpose but disadvantageous for technical purpose.

b. Wind resistance properties

Wind exerts pressure (inward or outward) on all exterior building surface. When structural wood panel such as plywood and oriented strand board are properly attached to lumber framing members, they form some of the most stable wall system. Wood is able to resist higher stresses when the load is applied for a short time; this feature enhance its performance in high wind events, which are typically of short duration.

c. Acoustic properties

Because of this internal friction, wood has a stronger sound dampening capacity than most structural materials. The natural acoustic properties of timber control this excessive echo, or reverberation, by reducing the transmission of sound vibrations. These properties of timber are why many public buildings, clad walls and ceilings are lined with acoustic timber panels or spaced timber battens. Timber acoustic paneling will often use holes or slots to increase the amount of sound absorption, essentially breaking up the energy of the sound wave. By breaking up the sound, the echoes are reduced. A classic example of what can be achieved with timber acoustics is the Sydney Opera House.

d. Deformability and failure mechanisms

According to the study of mechanical properties, wood is an elastoplastic material that presents different structural behavior when tensioned or compressed. Timber walls can present damage resulting to large deformation and degration of the material dues to different reasons which are mostly are biological and physical reason.

1.1.6.5. Bricks walls

Bricks walls are one of the most common and budget friendly infill wall. They are obtained by assembling earth bricks (compressed earth bricks, terracotta bricks, clay block bricks etc.). There are 3 types of bricks wall available:

- Plain bricks walls: These types of walls are made with bricks that are laid in cement mortar and plastered on both sides.
- Brick noggin wall: They consist on brickwork built up within a wooden frame and plastered on both sides.

- Reinforced brick wall: In this type of wall, a reinforcement in the form of steel wire mesh or steel bar is provided for additional strength

The properties of bricks walls mostly depend on the characterization of the material use for their conception. The following characteristic belongs to compressed earth bricks (CEB) because it is the most used type of brick wall in Africa.

a. Thermal properties

Bricks generally offers better insulating capabilities than other building material. It helps to keep the interior temperature of buildings relatively constant because of its thermal moisture that the material absorbs, according to the Claybrics website. Bricks absorbs and releases heat slowly during the day, keeping the rooms cooler during the night and warmer at night. This ability to hold and release heat can be increased by the addition of insulating materials.

b. Wind resistance properties

Brick house may entail either solid brick construction or veneer over wooden frame. Brick walls do not stand windstorms although it appears to be rock solid. According to the FEMA, the main causes are the poor bonding between ties and mortar, brick ties corrosion, misalignment of ties and brick fastener pullout. Nevertheless brick wall can be designed according to the target wind pressure to wind stand.

c. Acoustic properties

According to A. Niampara Daza et al., brick wall offers a good acoustic performance. The observation have been made that the acoustic performance of the wall increase when the wall is thicker. With a thickness of 250 to 300 mm, the sound reduction index of brick wall exceed the minimum required value. However, the sound absorption of brick wall is poor compared to the other materials.

d. Deformability and failure mechanism properties

Exposure to harsh weather, poor construction and lack of maintenance can lead brick wall to failure. The common failure observed on Compressed Earth bricks walls are diagonal cracking, caused by the deterioration of the supporting shelf angle, the bowing, cause by the expansion of bricks joints, poor mortar quality. The deformability of brick wall are the same with masonry walls.

1.2. Type of reinforced concrete building

It is significant to know the purpose of use of a building before designing it. Because its influence shape and the structural system that has to be used. There are different type sof building that can be classify depending on a country norms and different purpose of use. In general buildings are classify according the following criteria.

1.2.1. According to the occupancy

Depending on the occupancy there are:

- **Residential buildings:** They are building used for living that include one or more dwelling which are dormitory, hotels, houses, apartment house.
- **Educational buildings:**These buildings include any building used for educational purpose such as college, school, assembly for Instruction, nursery.
- **Institutional buildings:**These buildings include any building where people are kept and being maintain in particular hospital orphanage etc.
- **Assembly buildings:**These buildings may include any building where many people can gather such as church , museum etc.
- **Business buildings**These type buildings may include any building used for business offices, court house etc.
- **Mercantile buildings:**These types of buildings include any sort of retails building or trades buildings.
- **Industrial buildings :**They are building used for industrial production and manufacturing of foods and other non-toxic substance.
- **Hazardous buildings:**These buildings are used for the production and the manufacturing of toxic, explosive, corrosive, radioactive and poisonous substances.

1.2.2. According to their design and height

There are 2 categories of that sort:

- **Detached building:**They are buildings that comprise roofs and walls which is detached from any other being and have open space within their boundaries.

- **Multi-storey or high-rise buildings:**These are buildings that comprise more than 4 stories and/or buildings with height more than 15 meters (without stilt) and 17.5 meters (with stilt).

1.2.3. According to the structural frame

1.2.3.1. Braced-frame structural system

This structural system refers to cantilevered vertical trusses that resist laterals loads primarily diagonal members together with girders, and forms the web of the vertical truss as shown in figure 1.6 . This is more used in steel construction and is suitable for multi-storey building in the low to mid-height range. This kind of system is also efficient and economical for enhancing the lateral stiffness and resistance of a rigid-frame system. One remarkable advantage of a braced-frame is that it can be repetitive up to the height of the building and is economical in design and fabrication.

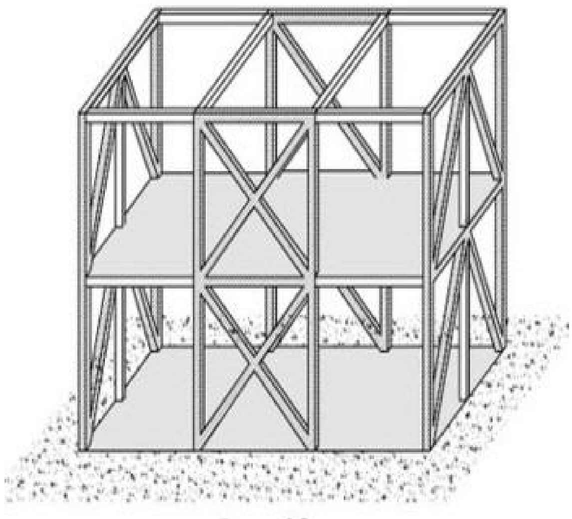


Figure 1.6. Representation Braced-frame structural system

1.2.3.2. Rigid-frame structural system

In this kind of system, beams and columns are constructed monolithically to withstand moments that are imposed due to loads. A rigid-frame system is more suitable for reinforced concrete buildings. Although this system may also be used in steel construction, the connections will be costly. However, there is the advantage of the likelihood of planning and fitting of windows due to open rectangular arrangement. Moreover, the members of a rigid-frame system can withstand bending moment, axial loads and shear force. Figure 1.7 illustrate the representation of a rigid –frame structural system.

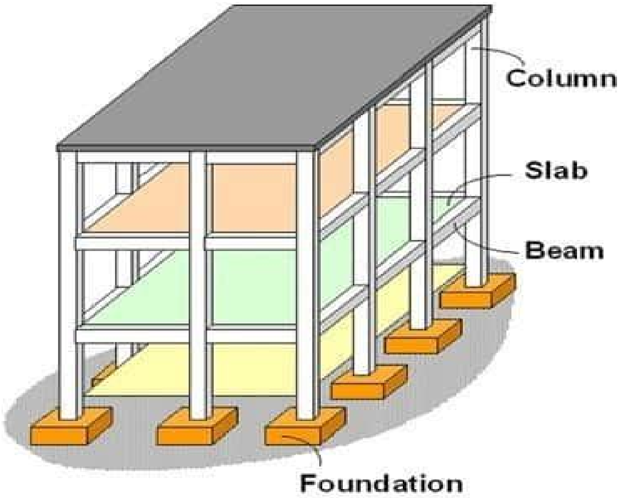


Figure 1.7. Typical RC frame building

1.2.3.3. Wall-frame system (dual system)

The wall-frame or dual system consists of a wall and frame that interact horizontally for a stronger and stiffer system as presented in figure 1.8. In this system, the walls are usually solid and can be found around elevator shafts, stairwells and/or at the perimeter of the building. Furthermore, the walls may also give a positive effect on the performance of the frames such as the prevention of a soft-story collapse.

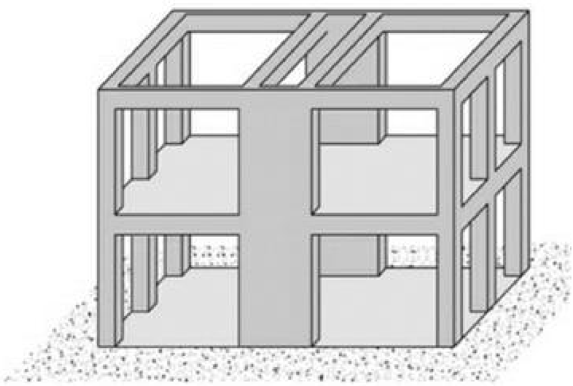


Figure 1.8. Typical dual frame systems

1.2.3.4. Shear wall system

This kind of system is a continuous vertical wall that's constructed from reinforced concrete or masonry wall as illustrated in figure 1.9. Shear walls are great at withstanding gravity and lateral loads, as well as acting as narrow-deep cantilever beams. This is commonly constructed as a core of buildings. When it comes to bracing tall buildings that are either reinforced concrete or steel structure, this system is highly suitable because shear walls are substantial in plane stiffness and strength. Moreover, a shear wall system is appropriate for hotel and residential building.

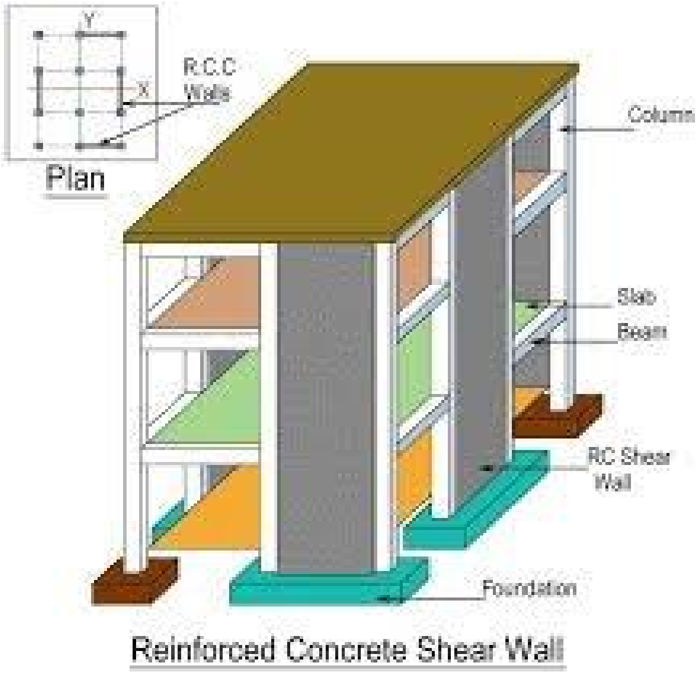


Figure 1.9. Representation of a shear wall frame system

1.3. Failure mechanisms of reinforced concrete buildings

Failure mechanisms are the physical, chemical, thermodynamic or other process that result to failure. They are categorized as either overstress, or wear out mechanisms. Overstress failure occurs because of a single load (stress) condition which exceed a fundamental strength property. Meanwhile, wear out failure is the result of cumulative damages related to loads apply during a define periods (from physic-of-failure prognostic for electronic product published in 2009). Earthquake causes the most devastating overstress failure on the buildings.

1.3.1. Soft and weak storey mechanism

In some RC buildings, especially at the ground floor, walls may not be continuous along to height of building for architectural, functional, and commercial reasons. While ground floor generally encloses with glass window instead of brick infill walls, partition walls are constructed above from this storey for separating rooms for the residential usage. This situation causes brittle failures at the end of the columns. In mid-rise reinforced concrete buildings, the most common failure mode is soft-storey mechanism, particularly at the first storey as illustrated in figure 1.10. Failures can be concentrated at any story called as weak storey in which the lateral strength changes suddenly between adjacent stories due to lack of or removing of partition walls or decreasing of cross section of columns. Thus, during an earthquake, partial and total collapses occur in these storeys.the weak story failure is presented in figure 1.11.

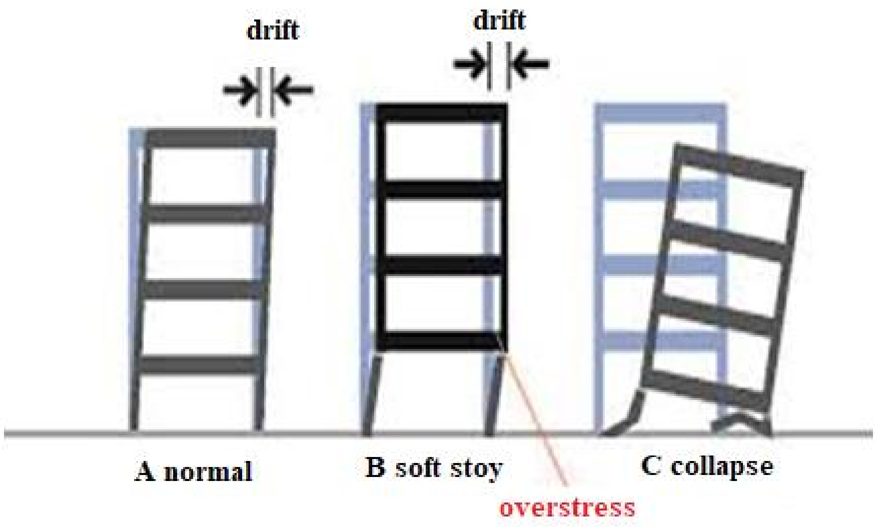


Figure 1.10. Soft story failure mechanism (FEMA)

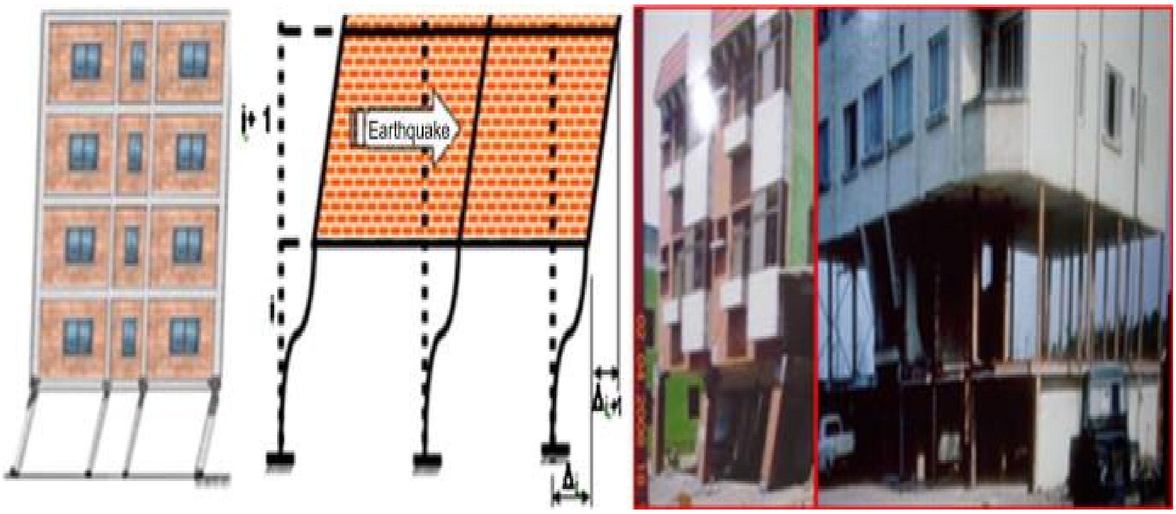


Figure 1.11. Weak storey failure mechanism (Nevzat Kirac et al. 2010)

1.3.2. Inadequate transverse reinforcement in columns and beams

Shear forces increase during an earthquake especially at columns and beam–column joints. Consequently, special attention should be paid to construction and design of beam–column joints and columns. Seismic design requires increasing of ductility of structures for performance-based design approach. In particular, columns of buildings can be having insufficient transverse reinforcement in the plastic hinge region. Therefore, structural elements which have such details show low performance against to dynamic loads and lost their shear and axial load carrying capacity. Figure 1.12 shows an exemple of failure caused by inadequate steel reinforcement.

Figure 1.12. Exemple of failure cause by inadequate steel reinforcement (Adem Dogangun 2013)

1.3.3. Short column failure

This type of mechanism can be developed due to structural adjustments and/or to continuous openings at the top of infill walls between columns. Lateral forces that occurred by an earthquake are carried by columns and shear walls. Length of column is an important factor for dissipation of these loads. When the length of column decreases, the column becomes stiffer and brittle than the other columns and this column attracts more shear forces. Thus, shear failure which is a critical type of concrete column damage occurs at these columns as illustrfated in figure 1.13.

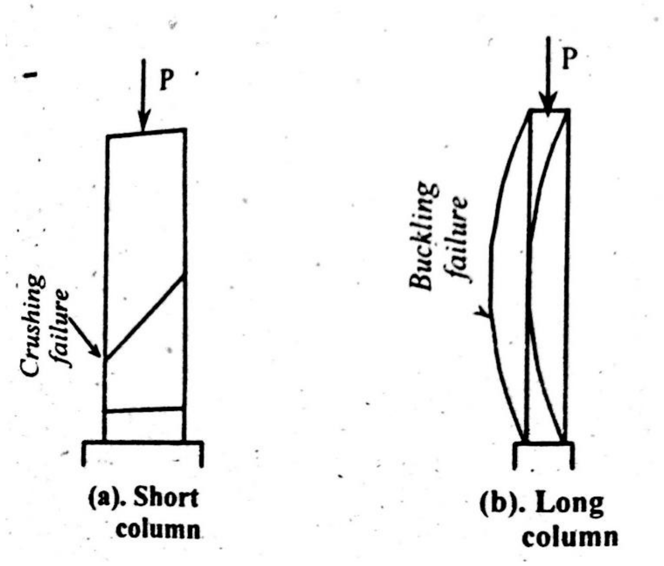


Figure 1.13. Short and long column failure (civiljungle.com)

1.3.4. Inadequate gaps between adjacent buildings

Buildings are sometimes constructed adjacent because of the lack of building lots. In this layout plan, one or two faces of two buildings are in contact to each other. Consequently, the buildings that have not adequate gaps pound to each other during the earthquakeas illustrated in figure 1.14. If the floors of the buildings are not at the same level, pounding effect of the buildings becomes more dangerous.

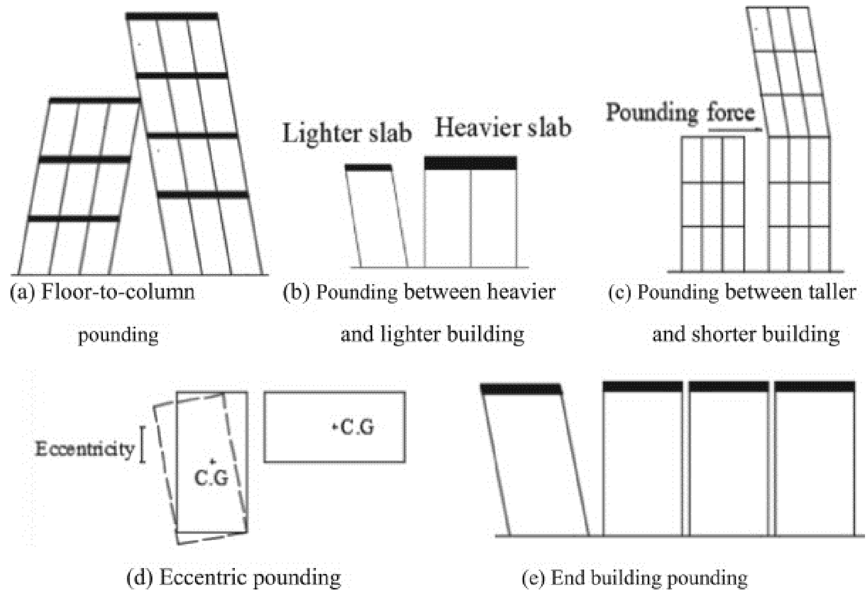


Figure 1.14. Failure due to inadequate gaps between buildings (civiljungle.com).

1.3.5. Strong beam–weak column failure

Deep and rigid beams are used with flexible columns in type of buildings. Therefore, these beams resist more moments, occurred by dynamic loads, than weak columns. In such a design during an earthquake while deep and rigid beams show elastic behaviour, shear failure or compression crushing causes plastic hinges at flexible columnsasillustrated in figure 1.15.

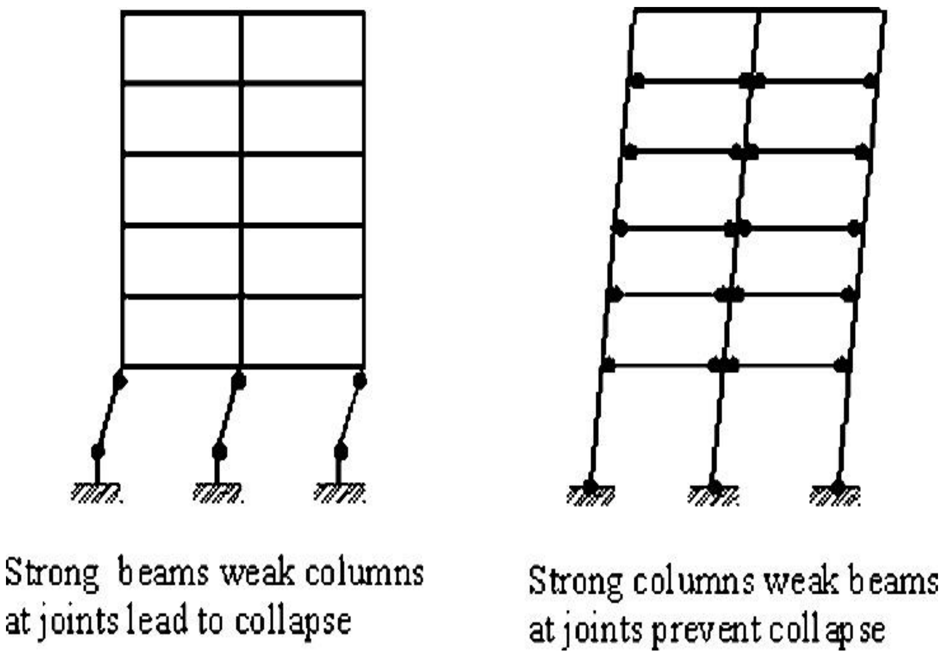


Figure 1.15. Strong beam-weak column and strong column-weak beam failures (Ergin Atimtay,2006)

1.3.6. Failures of gable walls

The most common failure mode at gable walls is out-of-plane collapse in the earthquakes. Although failures of gable walls are not structural damages, these damages may cause loss of lives and properties. Stability problems and large unsupported wall lengths cause damages at these walls. Failure of gabble wall is presented in Figure 1.20.

(a) Gable and partial façade wall collapse (b) Gable and partial façade wall collapse (c) Schematic of gable and partial façade collapse [24]

Figure 1.16. Collapse of gable walls (Dmytro Dizhur et al. , june 2016)

1.3.7. Poor concrete quality and corrosion failure

The other main reasons of damages are low concrete strength which shoul between 2 and 5 MPa and workmanship. Concrete quality is an important factor for building performance against to earthquakes. Handmade concrete is used to without using vibrator in construction of old buildings. Thus, homogeny mixing was not obtained and expected compressive strength was not provided in these buildings. In addition to this, using of aggregates which have improper granulometry, corrosion which decreases reinforcement bar area, and using of smooth steel reinforcement effected strength of concrete.Figure 1.17 illustrates the poor concrete quality failure.

Figure 1.17. Poor concrete quality failure (Gopal Mishra, 2016)

1.3.8. In-plane/out-of-plane effect

One of the most important reasons of life and economic loss during the earthquake is combined effect of in-plane and out-of-plane movement of the wall as illustrated in figure 1.18. In-plane and out-of-plane interaction is very complicated and should be analyzed well for this phenomenon. For low-rise and mid-rise unreinforced masonry (URM) infilled R/C frames, ground story infill walls are expected to be damaged firstly, because they are subjected to the highest in-plane demands. However, under the effect of bidirectional loading, where the two components of a ground motion are equally significant, infill walls of the upper stories may fail under the combination of in-plane and out-of-plane effects. The in-plane demand reduces at the upper stories, while that of out-of-plane forces increases due to the increase of accelerations.

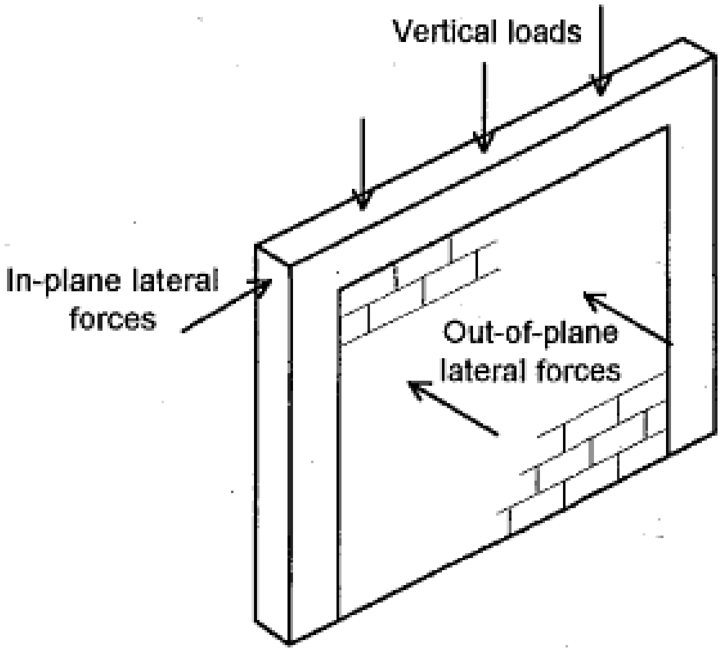


Figure 1.18. In-plane out-plane failure representation.

Conclusion

The aim of the chapter was to present the previous studies on infill walls the basic requirement for structural design of RC buildings and the t different damage due to earthquake. Parameters that defined the response of RC buildings where presented.it has been noticed that infill wall have specific properties from one to another, and their influence on the failure mechanism of buildings have been observe in many cases of buildings subjected to earthquakes. All the parameters presented above are indispensable for the understanding of the influence of infill walls in the response of the building to earthquake. 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

In order to study the contribution of infill wall in the resistance of the building to ruin and its rigidity, a Finite element analysis and a modal analysis are performed on the building. The contribution of infill walls is demonstrate globally by analyzing the whole structure considering them.it is also important to define the contribution and the behavior of each structural element to reach plausible results. In this work, the first step consists in a site recognition through a documentary research followed by the data collection. Then the steps of modelling and design of structural elements of the structural frame will be presented and the analysis will be performed.

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 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 that are the density ρ, the elastic modulus E and the Poisson ratio ν.

The shear modulus can be obtained from the elastic modulus and the Poisson ratio by equation 2.1:

$$G = \frac{E}{2(1+\nu)} \qquad \text{(Eq. 2.1)}$$

Where:

E: is the Young modulus

ν: is the Poisson ratio.

The shear wave velocity Vs is then obtained from the shear modulus by equation 2.2:

$$Vs = \sqrt{\frac{G}{\rho}} \qquad \text{(Eq. 2.2)}$$

Where:

ρ is the density

2.3.2. Design parameters

Structural data are related to the structural plan of each level which include the position of structural elements namely beams, columns and foundation, the concrete core and the characteristics of materials used.

2.3.2.1. Design codes

The norms that will be used for the design of elements are:

- the Eurocode 0 for basis of structural design
- Eurocode 1for actions on structures
- Eurocode 2 for design of concrete structures
- Eurocode 7 geotechnical design
- The FEMA 356 for infill walls modelling

These standards define the loads and the combination of loads for the design.

2.3.2.2. Applied loads

To carry out the analysis, different type of loads applied on the structure should be considered. There are permanent loads and variable loads.

a. Permanent loads

It is the total self-weight of structural and non-structural members. The self-weight of the structural elements is obtained by multiplying the specific weight of concrete γ by the area of the cross section of the elements and the self-weight of the secondary element are given in the Eurocode. The self-weight is taken into account in combinations of actions as a single action.

b. Variable loads

They are loads induced by the occupancy of the building and they can vary from one area to another. If an area is submitted to different imposed load, the most critical should be taken into account for the design and the analysis. The value of the imposed loads depends on the categories of building.

2.3.2.3. Load combination

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). The sign "+" means "combined with"; there are different load combination which are defined as follows, when designing a building.

a. Fundamental load combination

The fundamental load combination at ultimate limit state (ULS) is given by equation 2.3:

$$\sum i\geq 1\ \gamma G, jGk, j + \gamma Q,1Qk,1 + \sum i>1\ \gamma Q, i\Psi 0, iQk, i \qquad \text{(Eq. 2.3)}$$

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 (STR and GEO) verifications are:

γG = 1.35

γG = 1

γQ = 1.50 When unfavorable or 0 when favorable.

b. Rare load combination

The characteristic combination (rare), used for non-reversible serviceability limit states (SLS) to be used in the verifications with the allowable stress method is given by equation 2.4:

$$\sum j \geq 1\ Gk, + Qk,1 + \sum j > 1\ \Psi 0 \qquad \text{(Eq. 2.4)}$$

2.4. Model of the structure

A model is a representation of an idea, an object, a process or a system used to describe and explain phenomena that cannot be experienced directly. The structural model of the buildings is the assembled of the different model of each structural elements.

2.4.1. Beam and column modelling

Beam and column are model as frame elements. A frame element is modelled as a straight line connecting two points. Each element has its own local coordinate system for defining section properties and loads, and for interpreting output.

2.4.2. Foundation modelling

The type of foundation chosen is raft foundation.it will be modelled as thick shell and the soil structure interaction will be take into account into account. Thus spring will be used for the Soil-Structure ineteraction.

2.4.3. Infill wall modelling

There are two methods of modelling infill walls, micro modelling and macro modelling. The main difference between two methods is precision that micro modelling dealing with all individual wall components, block unit, mortar bracing and board partition; whereas macro modelling considers all the infill wall elements as a composite unit.

2.4.3.1. Micro modelling of infill walls

Micro-modelling is used generally to understand the local behavior of an infill wall. Inelastic properties and some mechanical properties as Young's modulus, Poisson's ratio are taken into account in detailed micro-modelling. On the other hand, each joint on infill wall is consisting of mortar and the two interface surfaces for simplified micro-modelling method. Infill consisting of elastic blocks interconnected with fracture tracks at the joints.

2.4.3.2. Macro modelling of infill walls

Macro models are used to investigate the overall response of the infill wall. Modelling of a wall using macro elements can be defined as using different type of springs or strut instead of structural elements. Mortar joints and units are recognized together considering collective mechanical and physical properties to obtain more simplified solution especially for large scaled models. The main difference between macro model and micro model is the local failure mode.

a. Strut characteristics

Infill walls with an opening bigger than 50% of their frame bay dimensions are passed over. Therefore, frame bays with French doors are not taken into account. The coefficient used to compute the strut width is obtained with the relation 2.7.

$$\lambda_1 = [\frac{E_{me}t_{inf}sin2\theta}{4E_{fe}I_{col}h_{inf}}]^{-\frac{1}{4}} \quad \text{(Eq. 2.5)}$$

The infill walls are simulated as simply supported compressed diagonal friction members. These members have thickness «t_{inf}» equal to the thickness of the infill walls and the width is given by equation 2.6.

$$a = 0{,}175(\lambda_1 h_{col})^{-0{,}4} r_{inf} \quad \text{(Eq. 2.6)}$$

Where:

a =Strut width

h_{col} =Column height between centerlines of beams (m);

h_{inf} =Height of infill panel (m);

E_{fe} =Modulus of elasticityof framed material (MPa);

E_{me} =Modulus of allasicity of infill material (MPa);

I_{col} =Moment of inertia of columnm^4;

r_{inf} =Diagonal length of infill panel, (m);

t_{inf} =Thickness of infill panel and equivalent strut (m);

θ =Angle whose tangent is the infill height to length ratio, (rad);

λ_1=coefficient used for the computation of the width a

2.5. Structural design methods

In the structural frame, each element plays a particular role and has different behavior when submitted to loading. The design of structural elements comes down to determine the concrete cross section, the steel reinforcement section and spacing.

2.5.1. Durability and concrete cover

Concrete cover is the least distance between the surface of embedded reinforcement and the outer surface.it is crucial to protect the steel reinforcement from corrosion caused by environmental effect. It is also protect the structural reinforcement from fire. For concrete structures, Eurocode 2 ensured this protection by the definition of a concrete cover taking into account the structural class of the structure and the exposure class. The concrete cover is illustrate in figure 2.1.

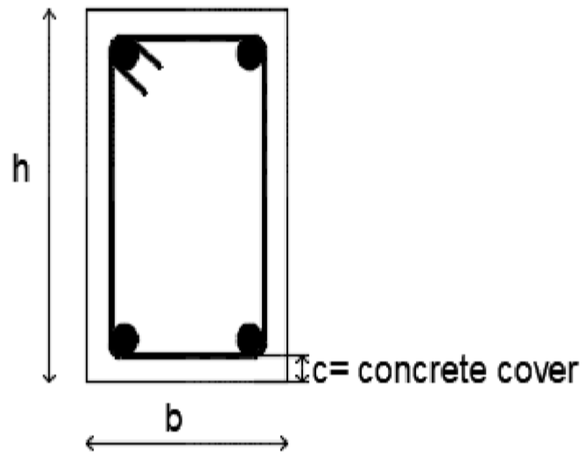


Figure 2.1. Illustration of the concrete cover

The nominal value of the concrete cover is defined as a minimum cover C_{min} plus an allowance in the design for deviation. The minimum cover C_{min} is defined in equation 2.3 as:

$$C_{min} = max\left(C_{min,b}; C_{min,dur} + \Delta C_{dur,\gamma} - \Delta C_{dur,st} - \Delta C_{dur\ add*}; 10mm\right) \quad \text{(Eq. 2.3)}$$

Where:

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

$\Delta_{,\gamma}$: the additive safety element with a recommended value of 0 mm

$\Delta Cdur, st$: reduction of minimum cover for use of stainless steel

$\Delta Cdur\ add$: the add reduction of minimum cover for use of additional protection

$Cmin, dur$: the minimum cover due to environmental conditions

The nominal value of the concrete cover is then expressed by equation 2.4:

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

Where:

$\Delta Cdev$ is the allowance in design for deviation with a recommended value of 10 mm.

2.5.2. The beams

Reinforced concrete beams are structural elements that are designed to carry transverse external loads . The loads cause bending moment, shear forces and in some cases torsion across their length. The most used concrete crossed section is the rectangular cross section. Prior to the design of reinforced concrete beam begin, there are certain assumption that need to be made. 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. This calculation is made with the working width corresponding to the section that it is calculate.

2.5.2.1. Ultimate Limit State design

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

a. Bending moment design

Design for bending moment is done with the envelope curve of the bending moment solicitation parameter. .

i. Longitudinal steel reinforcement

The section of the beam is a rectangular one. The longitudinal steel reinforcement is computed using eqution 2.5.

$$A_s = \frac{M_{Ed}}{0.9d.f_{yd}} \quad \text{(Eq. 2.5)}$$

Where:

A_s is the area of the steel reinforcement section;

M_{Ed} is the beding moment;

d is the effective depth;

f_{yd} the design yiedng strength of the steel.

According to the Eurocode 2, the maximum and the minimum steel reinforcement are given by equation 2.6 and 2.7.

$$A_{s,min} = \max(0.26\frac{f_{ctm}}{f_{yk}}b_t d; 0.0013 b_t d) \quad \text{(Eq. 2.6)}$$

$$A_{s,} = 0.004 A_c \quad \text{(Eq. 2.7)}$$

Where:

b_t is the mean width of the tension zone;

d is the is the effective depth of the section;

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

ii. Verification of the steel reinforcement

The number of reinforcement bars needed and the corresponding area of reinforcement are computed. The section is verified, using the position of the neutral axis inside the section, by calculating the resisting bending moment of the section.

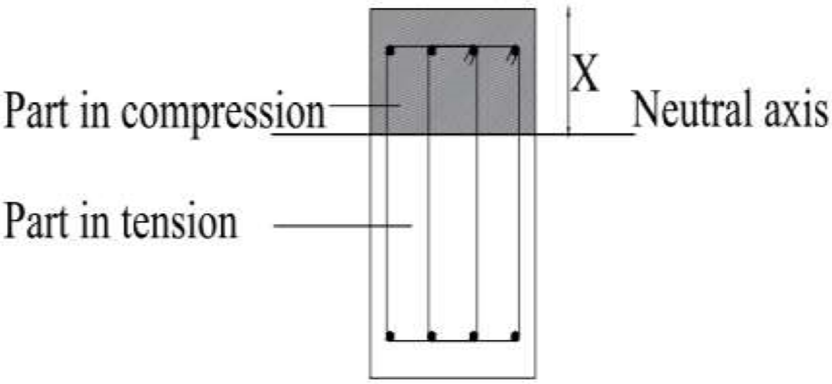


Figure 2.2. Neutral axis position in the beam section

The computation of the neutral axis is given by equation 2.8.

$$x = \left(\frac{d}{2\beta_2}\right) - \sqrt{\left(\frac{d}{2\beta_2}\right)^2 - \frac{M_{Ed}}{\beta_2 \beta_1 . b . f_{cd}}} \quad \text{(Eq. 2.8)}$$

Where:

d: Effective depth of the section;

b: Width of the section ;

f_{cd}: Design concrete compressive strength

$\beta_1 and\ \beta_2$: Correction factor equal to 0.81and 0.41 respectively

The resisting moment is given by equation 2.9 .

$$M_{Rd} = A_{s,}.f_{yd}.(d - \beta_2.x) \quad \text{(Eq. 2.9)}$$

Where:

$A_{S,real}$: is the effective area of the steel section,

f_{yd}: is the design yielding strength of the steel .

b. Shear verification

To resist shear, the beam needs shear reinforcement. The transversal reinforcement require depend on the value of the maximum shear V_{ed}. V_{ed} should be greater than the design resistance without shear reinforcement given by equation 2.10.

$$V_{Rd,c} = \{[C_{Rd,c}k(100\rho_l f_{ck})^3 + k_1\sigma_{cp}]b_w d; (V_{min} + k_1\sigma_{cp})b_w d\} \quad \text{(Eq. 2.10)}$$

With;

f_{ck}: 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

$\sigma_{cp} = \frac{N_{Ed}}{b_w d} < 0.2 f_{cd}\ [N/mm^2]$

N_{Ed}: Axial force of the cross section due to loading or prestressing

$b_w d$: Concrete cross-sectional area

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

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

Minimum shear reinforcement is provided where, according to the provision above, no shear reinforcement is required.

For members where the design shear reinforcement is required, the shear resistance is the minimum between $Vrds$ and $Vrdmax$ defined by equation 2.11 and 2.12:

$$V_{Rd,} = \alpha_{cw} b_w z \nu_1 f_{cd} / (\cot\vartheta + \tan\vartheta) \quad \text{(Eq. 2.11)}$$

$$V_{Rd,S} = \frac{A_{sw}}{S} z f_{ywd} \cot \vartheta \quad \text{(Eq. 2.12)}$$

Where:

f_{ywd} is the design yielding strength of the shear reinforcement ;

$\nu 1$ is a reduction factor for concrete cracked in shear ($\nu 1$ = 0.6 for $f_{ck} \leq 60\ N/mm^2$) ;

α_{cw} is a coefficient taking account of the state of stress in the compression cord αcw = 1 for non-prestressed structures.

The cross sectional area of the shear reinforcement with a maximum value given by the relation 2.13 as

$$\frac{A_{sw,max} f_{ywd}}{b_w S} \leq \frac{1}{2} \alpha_{cw} b_w v_1 f_{cd} \quad \text{(Eq. 2.13)}$$

Where:

Asw =Cross sectional area of the shear reinforcement;

$fywd$ =Design yield strength of the shear reinforcement;

bw =Smallest width of the cross section in the tensile area ;

S = the spacing of the stirrups.

Figure 2.3 shows the maximum longitudinal spacing and transversal spacing of a span.

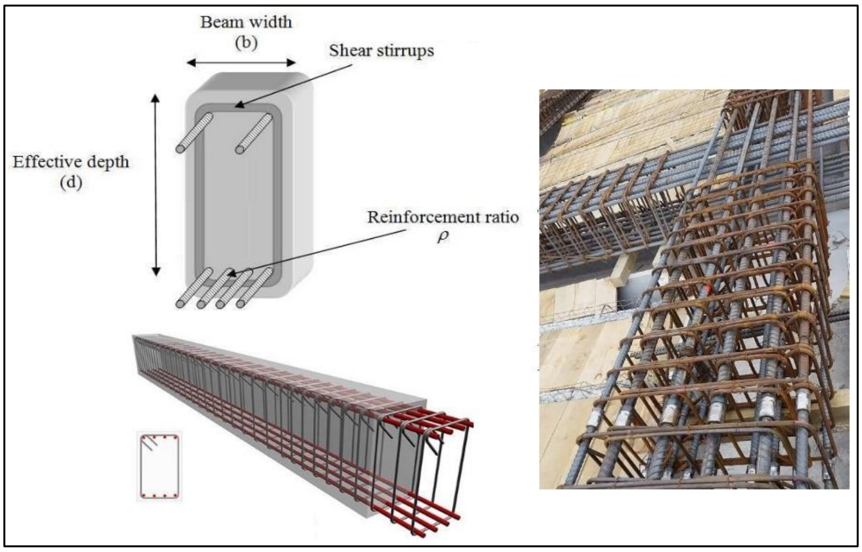


Figure 2.3. Maximum longitudinal spacing and transversal spacing of the legs

The design shear reinforcement obtained has to verify the detailing of members. In the case of the beam, it defines the maximum longitudinal spacing of the shear assembly, the maximum transversal spacing of the legs in a series of shear link and the minimum shear reinforcement ratio . The spacing is given by equation 2.14 and 2.15:

$$S_{l,} = 0.75(1+\cot\alpha) \quad \text{(Eq. 2.14)}$$

$$S_{t,} = 0.75d \leq 600mm \quad \text{(Eq. 2.15)}$$

$$\rho_{w,min} = \left(0.08\sqrt{f_{ck}}\right)/f_{yk} \quad \text{(Eq. 2.16)}$$

Where reinforcement ratio is given by equation 2.17.

$$\rho_w = A_{sw}/(s.b_w.\sin\alpha) \quad \text{(Eq. 2.17)}$$

2.5.2.2. 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.

a. Stress verification

The stress value is function of the modular ratio in short terms and long terms expressed in equation 2.18 and 2.19 respectively:

$$n_0 = \frac{E_s}{E_c} \quad \text{(Eq. 2.18)}$$

$$n_\infty = n_0(1+\varphi_L \times \rho_\infty) \quad \text{(Eq. 2.19)}$$

Where $\varphi_L = 0.55$ for shrinkage of concrete and the parameter $\rho_\infty = 2 \div 2.5$ for an uncracked concrete section, the neutral axis is computed thus:

$$x = \frac{-n(A_S + A'_S) + \sqrt{[n(A_S + A'_S)^2] + 2bn(A_S d + A'_S d')}}{b} \quad \text{(Eq. 2.20)}$$

Where A'_s and A_s are the upper and lower steel reinforcement inside the section respectively. b, d' and d, are the geometrical characteristics of the section presented in the Figure 2.4.

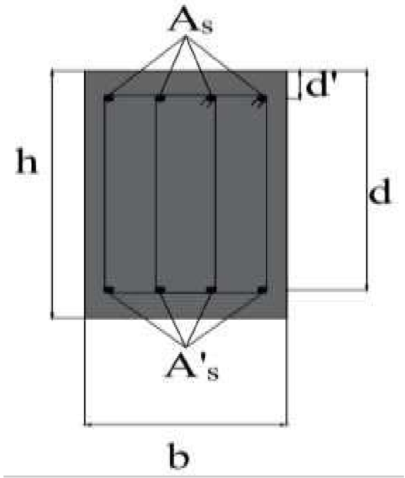


Figure 2.4. Geometric characteristics of a transversal beam section

The moment of inertia of the uncracked beam section is given by the relation 2.21.

$$J_{cr} = \frac{bx^3}{3} + nA_s(d-x)^2 + nA'_s(x-c)^2 \qquad \text{(Eq. 2.21)}$$

The stresses in the concrete and steel can be computed using Eq 2.22 and Eq2.23.

$$\sigma_s = \frac{M_{Ed}(d-x)}{J_{cr}} \times n_\infty \qquad \text{(Eq. 2.22)}$$

$$\sigma_c = \frac{M_{Ed}.x}{J_{cr}} \qquad \text{(Eq. 2.23)}$$

The verification of these stresses for the obtained cross section as provided by Eurocode 2, are given by equations 2.24 and 2.25:

$$\sigma_c \leq k_1 * f_{ck} \qquad \text{(Eq. 2.24)}$$

$$\sigma_s \leq k_3 * f_{yk} \qquad \text{(Eq. 2.25)}$$

With $k_1 = 0.6$ and $k_3 = 0.8$.

b. Deflection verification

The deflection is the vertical displacement of a point on a loaded beam. According to the Eurocode 2, the deformation of member or structure shall not be such that it adversely affects its proper functioning or appearance. Eurocode 2 allows to avoid calculus if equation 2.26 or equation2.27 are satisfied.

$$l/d \le K\left[11+1.5\cdot\sqrt{f_{ck}}\,\frac{\rho_0}{\rho}+3.2\cdot\sqrt{f_{ck}}\left(\frac{\rho_0}{\rho}-1\right)^{3/2}\right] \qquad \text{if } \rho<\rho_0 \qquad \text{(Eq. 2.26)}$$

$$l/d \le K\left[11+1.5\cdot\sqrt{f_{ck}}\,\frac{\rho_0}{\rho-\rho'}+1/12\cdot\sqrt{f_{ck}}\sqrt{\frac{\rho'}{\rho_0}}\right] \qquad \text{if } \rho>\rho_0 \qquad \text{(Eq. 2.27)}$$

where:

l/d is the limit span/depth

K is the factor to take into account the different structural systems (see Annex 3)

ρ_0 is the reference reinforcement ratio = $\sqrt{f_{ck}}\ 10^{-3}$

ρ is the required tension reinforcement ratio at mid-span to resist the moment due to the design loads (at support for cantilevers)

ρ' is the required compression reinforcement ratio at mid-span to resist the moment due

to design loads (at support for cantilevers)

f_{ck} is in MPa units

The deflection will be evaluated in order to define the vertical deformation of the building's frame.

2.5.3. Column design

A column or pillar is a structural element that transmits, through compression, the weight of the structure above to other structural elements below. Reinforced concrete columns are the most widely used columns for framed structure. They are composed of concrete as a matrix. The steel frame is embedded in concrete. Concrete carries the compressive load and reinforcement resists tensile load. The reinforcing materials can be made of steel, polymers, or alternate composite materials.ULS is considered in the case of the column and the verifications done for axial force, bending moment and shear force. The procedure for shear force verification for the column is the same as that of the beam.

2.5.3.1. Axial load resistance of the section

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 the minimum area section of the column is estimate using equation 2.28.

$$N_{Rd} = 0.6 \times f_{cd} \times A_c \ge N_{sd} \qquad \text{(Eq. 2.28)}$$

Where:

N_R is the design axial compression force

A_c is the concrete section area;

N_{sd} is the axial load computed using the recovery area of the column the axial load is computed

Using the relation 2.29.

$$N_{sd} = q \times S_r \times n \qquad \text{(Eq. 2.29)}$$

Where:

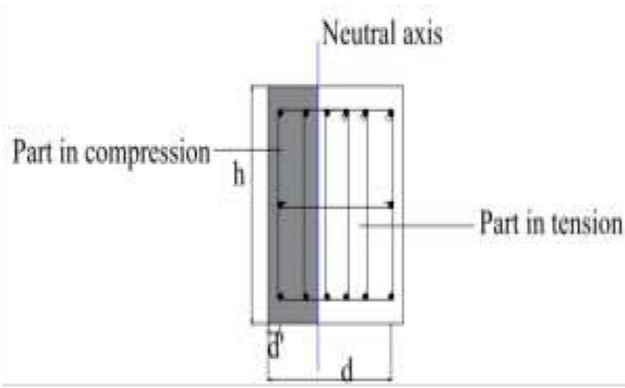
q is the uniform distributed loads on each floor computed at ULS;

S_r is the recovery area of the column;

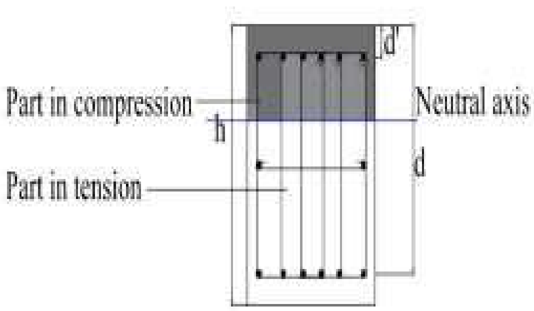
n is the number of stories above the considered column.

2.5.3.2. 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 Points lying within the diagram are respectful of the design criteria otherwise failure occurs. When considering a rectangular section as shown in Figure 2.5, the computation of the points is done as presented in the subsequent sections.



(a) x-direction



(b) y -direction

Figure 2.5. M-N diagram computation of a rectangular column section in both loading directions

a. First point

At this point, the section is completely subjected to tension, thus, the concrete is not reacting (concrete does not work under tension). It imposed $\varepsilon_s = \varepsilon_{su}$, $\varepsilon_s{}' = \varepsilon_{syd}$. The stress inside the element corresponds to the design yielding strength of the steel reinforcement and the limit axial force and bending moment are obtained from equations 2.30 and 2.31.

$$N_{Rd} = f_{yd}.A_s + f_{yd}.A'_s \quad \text{(Eq. 2.30)}$$

$$M_{Rd} = f_{yd}.A_s.\left(\frac{h}{2} - d'\right) - f_{yd}.A_s.\left(\frac{h}{2} - d'\right) \quad \text{(Eq. 2.31)}$$

Where:

N_{rd} is the resisting compression force;

h is the height of the column section;

d' is the relative effective depth.

b. Second point

At the second point, the section is completely subjected to tension. We impose that; the strains, $\varepsilon_s = \varepsilon_{su}$, $\varepsilon_c = 0$. The upper steel yielding condition is verified. If the steel has not yielded, then $\varepsilon_s{}'$ is determined. equations 2.30 and equation 2.31 are used for the computation of the axial force and the bending moment.

c. Third point

Here failure is due to concrete and the lower reinforcements have yielded. It's assumed that $\varepsilon_s \geq \varepsilon_{syd}$, $\varepsilon_c = \varepsilon_{cu2}$ and the position of the neutral axis is determined. Yielding condition of the upper steel reinforcement is verified. If the steel is yielded or not is determined by determining $\varepsilon_s{}'$ in order to determine the corresponding stress. The axial force and bending moment corresponding to the third point are computed thus:

$$N_{Rd} = -\beta_1.b.x.f_{cd} + f_{yd}.A_s - f_{yd}.A'_s \quad \text{(Eq. 2.32)}$$

$$M_{Rd} = f_{yd}.A'_s.\left(\frac{h}{2} - d'\right) + f_{yd}.A_s\left(\frac{h}{2} - d'\right) + \beta_1.b.x.f_{cd}\left(\frac{h}{2} - \beta_2.x\right) \quad \text{(Eq. 2.33)}$$

d. Fourth point

To compute the coordinate of this point, the failure is due to concrete and the lower reinforcement reaches exactly $\varepsilon_s = \varepsilon_{syd}$. Likewise, we determine the neutral axis position and the strain $\varepsilon_s{}'$. equations 3.32 and 3.33 are used to compute the fourth point.

e. Fifth point

At this state, $\varepsilon_s = 0$ and failure is due to concrete. The lower reinforcement is considered to have yielded and the position of the neutral axis is the same as that of the effective depth. Axial force and bending moment are computed using equation 2.3.4 and 235 respectively

$$N_{Rd} = -\beta_1 . b . x . f_{cd} - f_{yd} . A'_s \quad \text{(Eq. 2.34)}$$

$$M_{Rd} = f_{yd} . A'_s . \left(\frac{h}{2} - d'\right) + \beta_1 . b . d . f_{cd} \left(\frac{h}{2} - \beta_2 . x\right) \quad \text{(Eq. 2.35)}$$

f. Sixth point

It imposes that concrete is uniformly compressed and assume the strains $\varepsilon_s = \varepsilon_c \geq \varepsilon_{c2}$.

Axial force and bending moment are computed using equation 2.3.6 and 237 respectively.

$$N_{Rd} = -b . h . f_{cd} - f_{ywd} . A'_s - f_{yd} . A_s \quad \text{(Eq. 2.36)}$$

$$M_{Rd} = f_{yd} . A'_s . \left(\frac{h}{2} - d'\right) - f_{yd} . A_s \left(\frac{h}{2} - d'\right) \quad \text{(Eq. 2.37)}$$

An example of an M-N interaction diagram is shown in Figure 2.6. The figure shows the M-N diagram in red for positive and negative loading directions with the couple of points (solicitation M_{Ed} and N_{Ed}), in blue, which lie within it, thereby respecting the design criteria. Provisions of Eurocode with regards to the steel reinforcement of the column.

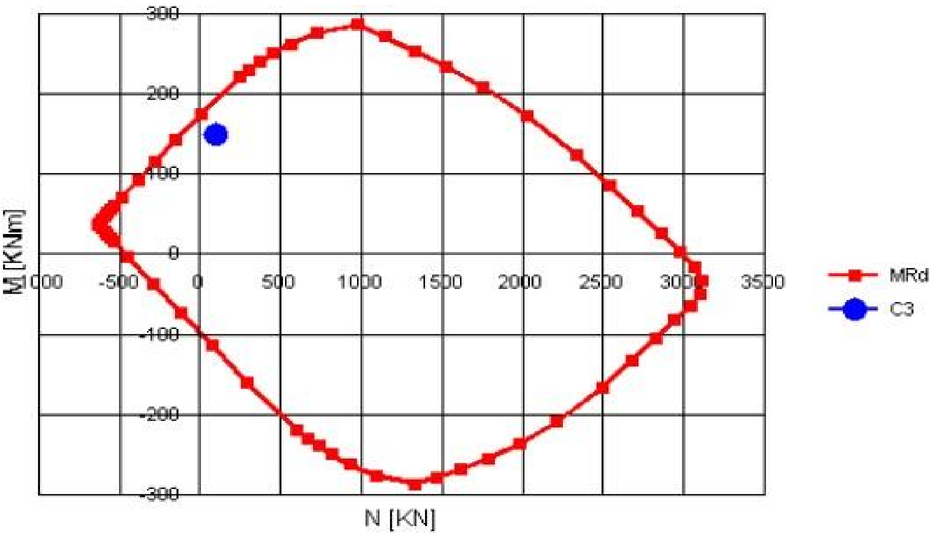


Figure 2.6. Example of M-N diagram (D'Antino et al., 2016)

The steel reinforcement of the column is considered taking into account the limitations of the Eurocode 2 defined by equation 2.38 and 2.39.

$$A_{s,min} = max\left(\frac{0.10N_{Ed}}{f_{yd}}; 0.002A_c\right) \quad \text{(Eq. 2.38)}$$

$$A_{s,} = 0.04A_c \quad \text{(Eq. 2.39)}$$

Where,

N_{Ed} is design axial compression force ;

f_{yd} is design yield strength of the longitudinal reinforcement.

2.5.3.3. Shear verification

The Eurocode 2 requires a minimum diameter of 6mm or one quarter of the maximum diameter of the longitudinal bars. The maximum spacing of the transverse reinforcement is given by the equation 2.40.

$$S_{cl,} = \min(20\emptyset_{l,}; b; 400mm) \quad \text{(Eq. 2.40)}$$

Where:

$\emptyset_{,}$ is the minimum diameter of the longitudinal bars

b is the lesser dimension of the column

The factor of 0.6 is used to reduce the maximum spacing in sections within a distance equal to the larger dimension of the column bars.

2.5.3.4. Slenderness verification

The need for slenderness verification arises from whether or not second order effects are to be accounted for. Eurocode 2 recommendations are outlined in equation 2.41.

$$\lambda_{lim} = 20.A.B.C/\sqrt{n} \qquad \text{(Eq. 2.41)}$$

With:

$A = \frac{1}{1+0.2\varphi_{ef}}$ (φ_{ef}: is the effective creep ratio; If not known, A=0.7)

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

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

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

The expression in equation 2.42 is the one used for the estimation of slenderness λ.

$$\lambda = l_0/i \qquad \text{(Eq. 2.42)}$$

Where:

I: The gyration radius of the uncracked concrete

l_0: Effective length of the element (l_0 =0.7l)

$$i = \sqrt{\frac{I}{A}} \qquad \text{(Eq. 2.43)}$$

I: Moment of inertia and A is the area of the section.

2.5.4. The foundations

The type of foundation (see figure 2.7)depends on the type of soil and the wingspan of the structure. A raft foundation is a continuous slab which covers the whole plan area of the structure. It is used when the supporting soil has a low bearing capacity and compressible. The choice of the mat dimension depends on geotechnical conditions and the slab floor and strengthening beams are designed as inverted elements as explained below.

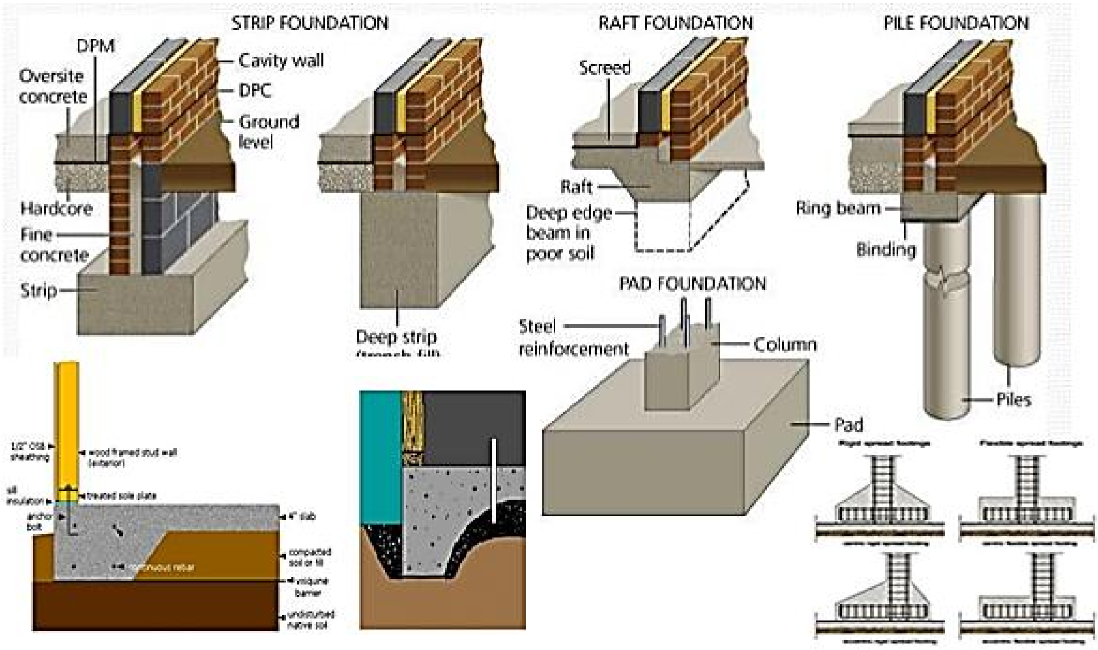


Figure 2.7. Type of foundations

2.5.4.1. Geotechnical design of the raft

The slab is designed by the conventional rigid method. The pressure developed by the total vertical load of applied on the raft should less than the allowable pressure of the soil as expressed in relation 2.44 as:

$$\sigma = \frac{Q}{A} \leq \sigma_{adm} \qquad \text{(Eq. 2.44)}$$

Where:

σ Is the contact pressure

σ_{adm} is the admissible pressure on the soil

Q Is the total vertical arriving at the foundations

A is the surface of the mat

The position of the resulting vertical load is computed along x and y axis as shown in relation 2.45.

$$X = \frac{\sum_i P_i x_i}{\sum_i P_i} \; and \; Y = \frac{\sum_i P_i y_i}{\sum_i P_i} \qquad \text{(Eq. 2.45)}$$

Where:

P_i is the load at the base of the column i

x_i is the position of the column i along x-direction

y_i is the position of the column i along y-direction

If there is an eccentricity between the resultant vertical load and the center of gravity of the raft the contact pressure distribution is obtained as expressed in equation 2.46.

$$\sigma_{max/min} = \frac{Q}{A} \pm \frac{Q_{ex}}{I_{yy}} y \pm \frac{Q_{ey}}{I_{xx}} x \qquad \text{(Eq. 2.46)}$$

Where:

e_x is the eccentricity along x axis

e_y is the eccentricity along y

I_{xx} is the moment of inertia of the section along x axis

I_{yy} is the moment of inertia of the section along y axis

x and y are the position of a given point from x and y axis respectively.

2.5.4.2. Structural design of the raft

The raft is made of panels and strengthening beams. The panel are designed as a plate thick element. The panels will be modelled in SAP2000 as a plate resting on Winkler's springs.The stiffness of the springs is obtained from the modulus of subgrade reaction of the soil times the area of the meshing square using relation 2.47.

$$k = C \times A \qquad \text{(Eq. 2.47)}$$

Where:

C is the modulus of subgrade reaction of the soil

A is the mesh area

The depth of the slab can be taken at first approximation with respect to relation 2.48.

$$d \geq L/20 \qquad \text{(Eq. 2.48)}$$

Where L is the distance between axes of columns.

The effective depth along direction x and direction y is given according to equation 2.49 and 2.50 respectively expressed as:

$$d_x = d - c_{nom} - 3\emptyset/2 \qquad \text{(Eq. 2.49)}$$

$$d_y = d - c_{nom} - \emptyset/2 \qquad \text{(Eq. 2.50)}$$

Where:

d is the total depth of the slab ;

c_{nom} is the concrete cover ;

$\emptyset$ is the diameter of the bars used as reinforcement .

The computation of the reinforcement area is done for one-meter length using equation 2.13 with the mean value of the effective depth along x and y. The strengthening beams are designed as inverted beams. The depth of the beam H follows the relation 2.51.

$$H \geq L/10 \qquad \text{(Eq. 2.51)}$$

The moment at mid span, the moment at a support and the shear can be after running the analysis in the software.

2.6. Analysis of methods

The analysis to be performed are the finite element analysis and the pushover analysiSAP 2000 v22 finite element software. Modal analysis will eventually be performed with the aim of take notice the building vibration modes. In order to evaluate the contribution of infill wall on the mechanic behavior of the structure, different analysis will be first performed on a reinforced concrete 2 dimensions frame with and without infill walls using micro modelling. Secondly the same analysis will be performed on the whole structural system of the building with and without infill wall using macro modelling.

2.6.1. Modal analysis

2.6.1.1. Description

Modal analysis is used to determine the vibration modes of a structure. These modes are useful to understand the behavior of the structure. They can also be used as the basis for modal superposition in response-spectrum and modal time-history Load Cases.

2.6.1.2. Periods and frequencies

The following time- properties are printed for each Mode:

- Period, T, in units of time
- Cyclic frequency, f, in units of cycles per time; this is the inverse of T
- Circular frequency, ω, in units of radians per time; ω = 2πf
- Eigen value, ω2, in units of radians- per- time squared

2.6.2. Lateral force analysis

The analysis consist on apply a horizontal force equal to 10 percent of the self weight of the frame structure at each level of the building as illustrated in figure 2.7. The force will be apply along the principal beams direction to evaluate the deformability along this direction.

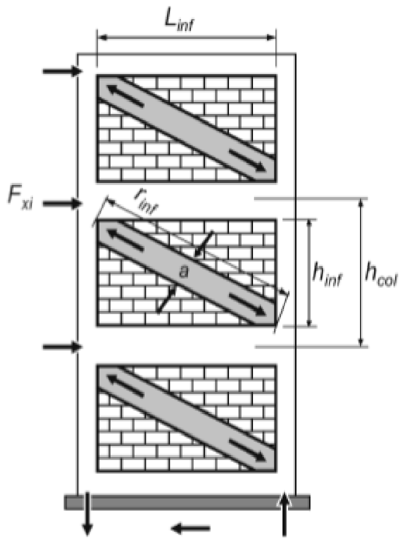


Figure 2.8. Distribution of the lateral force (FEMA356)

The obtained displacement x, will be used to compute the stiffness K of the frame structure for each case. Stiffness is a measure of how much force is require to displace a building by a certain amount.the stinesse formula is given by the relation 2.52.

$$k = \frac{F}{x} \quad \text{(Eq. 2.52)}$$

Where

k is the stiffness

F is the horizontal force

x is the horizontal displacement.

2.6.3. Vertical deformation analysis

The aim of this analysis is to determine the deflection of the beams under the effect of the infill wall self weight.The analysis consist on variying the value of the infill wall selfweight according to the type of wall and determine the variation of the deflection when the infill walls are considered. The equation of deflection is given by equation.

Conclusion

The aim of this chapter was to present in details the methodology of the different analysis that will be perform on the building. The site recognition and the different data collection procedure were first presented, secondly the design steps of different structural elements, then the modelling specification and parameters were defined for the conception of the structural frame of the buildings. Analysis procedure, description and searched parameters were state at last according to the Eurocode and FEMA 356. The result of these analysis will be displayed and discussed in chapter 3 for the understanding of the influence of infill walls in the design of RC buildings.

CHAPTER 3: RESULTS PRESENTATION AND INTERPRETATION

Introduction

This chapter presents the different answers to the methodology studied in chapter 2, with the intention of leading our research on the demonstration of the role of infill wall in the structural behavior of an RC building.in parallel, a comparative studied between brick wall, drywall and masonry wall will be perform to show which infill wall have a greater impact on the structure. All the analysis will be performed on the software SAP2000 and the results presentation will be done using Excel.

3.1. General presentation of the site

The documentary research permit to obtained the necessary data to describe the site.the case study is a conceived reinforced concrete building for residential purpose.

3.1.1. Presentation of the case study

The building that has to be studied is a residential reinforced concrete building located at Douala in the Akwa-nord district. The total height of the building is equal to 12.8 meters and each storey measures 3.20 m of height.it is regular in plane and elevation. The slab is assumed to be a reinforced concrete slab with hollow blocks of thickness 20cm. The structure has identical floor plan as from level one to level three as shown in the elevation view in figure 3.1.

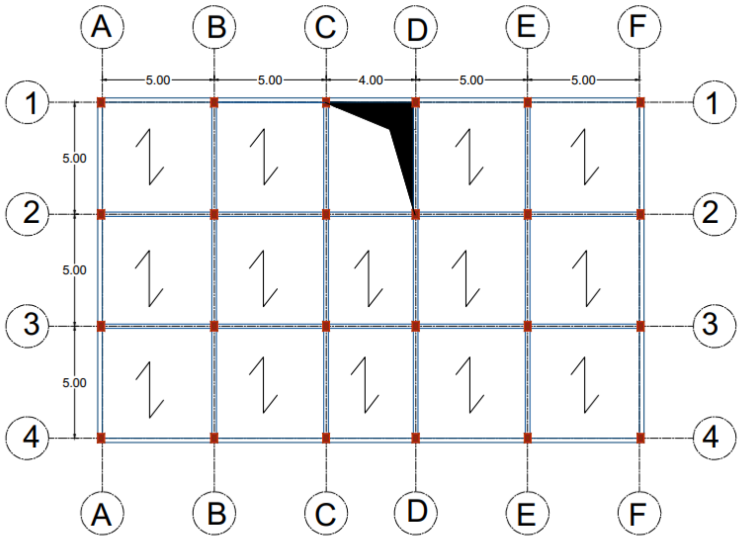


Figure 3.1. Shuttering plan of the four floor

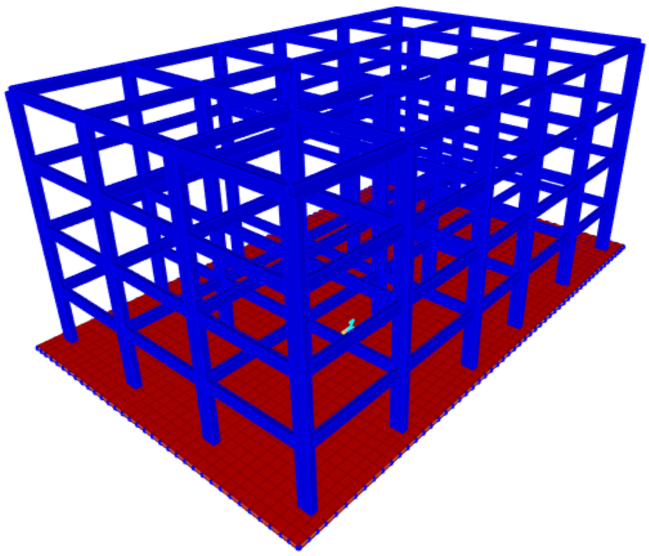


Figure 3.2. 3D model of the frame

3.1.2. Geologic and topographic data

The current topography of the site shows that building is located on a field with a negligible slope.The site belongs to the sedimentary basin of the Douala area and rest on a plinth of crystalline. The basin is made up of thick continental and luvio marine formation separated by some fossiliferous marine intercalation.

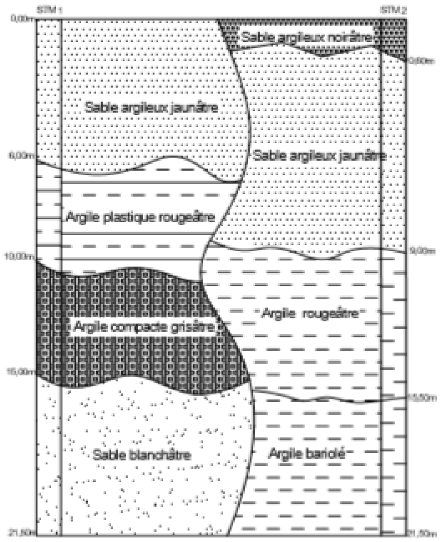


Figure 3.3. Longitudinal geotechnical profile of the site

3.1.3. Geographic and Climatic data

Douala city is located on the borderline of the Atlantic Ocean, at the bottom of the Guinea's gulf, at the opening of the Wouri River. The project is located at a place called "Aiwa-Nord" .the zone feature a tropical monsoon climate with relatively consistent temperature during the years. Nevertheless, cooler temperature are observed in July and august. The average of annual temperature is 27°c and the average of humidity is 83%.

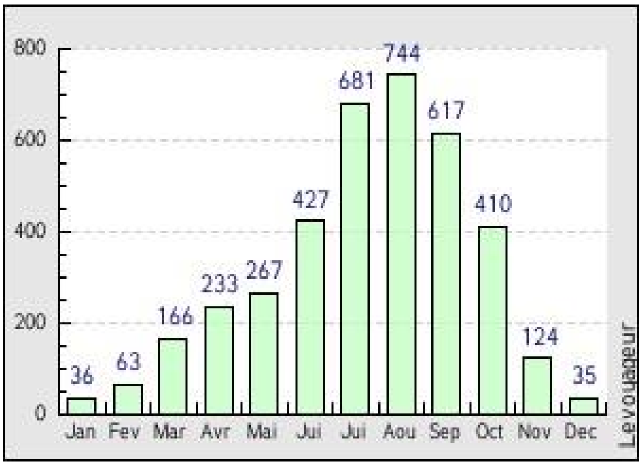


Figure 3.4.precipitation of the Douala region

3.1.4. Durability and concrete cover determination

Considering a concrete structural class S4 and the exposure class XC1 together with the provision of the Eurocode 2 outlined in section 2.51, the concrete cover obtained is:

$$C_{min} = \max(14{,}15{,}10) = 15\ mm$$

Applying the equation of nominal cover, $C_{nom} = 15 + 10 = 25\ mm$

Thus, the concrete cover that will be considered for the design is c=30 mm for the upper structural elements, and c=50 mm for the foundation.

3.2. Design parameters

3.2.1. Loads

The case study is a residential building. According to the Eurocode, variable load for residential building is from 1.5KN/m² to 2.0KN/m² thus the chosen value is 1.5KN/m².the different applied load are presented in table 3.1.

Table 3.1. Applied loads

Nature	Description	Value	Unit
G_{1K}	Hollow body slab	3	KN/m²
G_{2k}	Secondary element load	1.2	KN/m²
Q_k	Variable load	1.5	KN/m²

3.2.2. Load combinations

The load combination in equation 3.1 and 3.2 provide for the verification of the structure at Ultimate Limit State.

$$1.35G_k + 1.5Q_k \qquad \text{(Eq. 3.1)}$$

$$G_k = G_{1k} + G_{2k} \qquad \text{(Eq. 3.2)}$$

For non-reversible Serviceability Limit State (SLS), the verification is done using equation3.3.

$$G_k + Q_K \qquad \text{(Eq. 3.3)}$$

3.2.3. Material properties

The concrete class chosen is C25/30 and the longitudinal steel reinforcement is B450 . A characteristic yield strength of 235 MPa for the transversal reinforcement is considered. Table 3.2 shows the main characteristics of concrete and Table 3.3 that of steel used as reinforcement for the design o f the structural elements.

Table 3.2. Concrete characteristics

Property	Value	Unit	Definition
Class	C25/30	-	Concrete class
Rck	30	MPa	Characteristic cubic compressive strength
fck	25	MPa	Characteristic compressive strength of concrete at 28 days
$fcm = fck + 8$	33	MPa	Mean value of concrete cylinder compressive strength
γc	1.5	-	Partial factor for concrete
$f_{ctm} = 0.3 \times (f_{ck})^{\frac{2}{3}}$	2.56	MPa	Mean value of axial tensile strength of concrete
$f_{ctd} = 0.7 \times \frac{f_{ctm}}{\gamma_c}$	1.2	MPa	Design resistance in traction
$Ecm = 22000 \times \left(\frac{fcm}{10}\right)^{0.3}$	31476	MPa	Secant modulus of elasticity
ν	0.2	-	Poisson's ratio
G	13115	MPa	Shear modulus
γ	25	KN/m^3	Specific weight of the concrete

Table 3.3. Reinforcement characteristics

Property	Value	Unit	Definition
Class	B450C	-	Steel class
f_{yk}	450	MPa	Characteristic yield strength
γ_s	1.15	-	Partial safety factor for steel

γ	78.5	MPa	Specific weight of the steel
ν	0.3	-	Poisson's ratio

3.3. Structural element design

3.3.1. Beam design results

The horizontal structural elements of the considered building are composed of the beams which support the slab. The principal beam chosen for the design is highlighted in figure 3.5.It is the most loaded beam and its section will be adopt for the other beams.

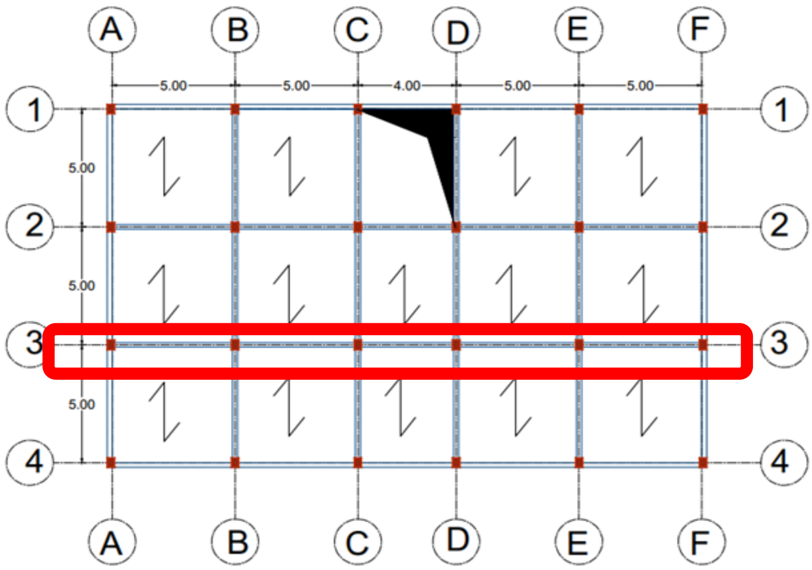


Figure 3.5. Most solicited beam to design.

The predesign of the cross section is obtained by assuming the value of the section height and the section depth. The section height is obtained from the maximum span as:

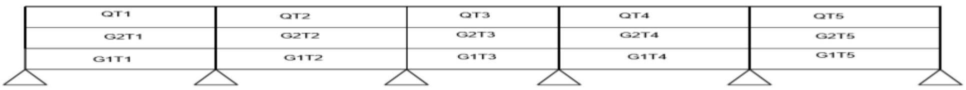
$h = \frac{5}{12} = 0.41cm$ And b = 20~30 cm. An initial value of section characteristic is taken $h = 40\ cm$ and b = 20 cm.

This section, is modelled in the software as frame element with the different restraints as supports as shown in the figure 3.6.

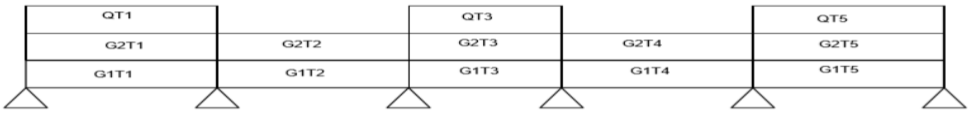
Figure 3.6. Simple support model

From this model, loads combinations for maximum moment at mid span and maximum shear are presented on figure 3.7. Also, the load combination ULS 7, ULS 8 and ULS 9 are respectively symmetric to the load combination ULS 6, ULS 5 and ULS 4.

Combination ULS 1

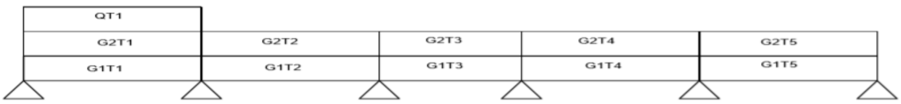


Combination ULS 2

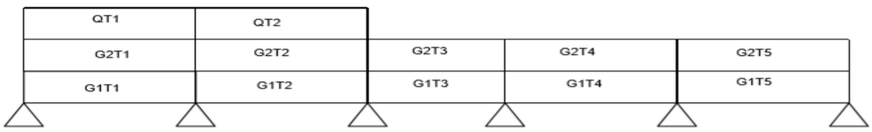


Combination ULS 3

Combination ULS 4



Combination ULS 5



Combination ULS 6

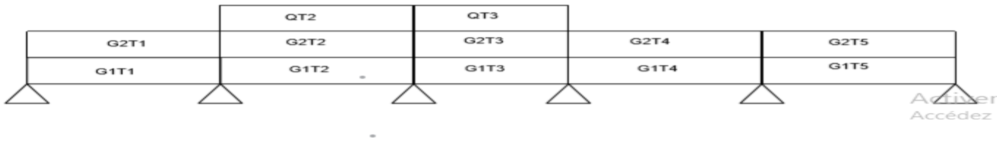


Figure 3.7. Applied load combinations

3.3.1.1. Ultimate limit state design

The nine loads arrangements inserted in the software SAP 2000 permit to obtain solicitations curves along the beam for the bending moment and the shear force represented in figure 3.8 and 3.9.

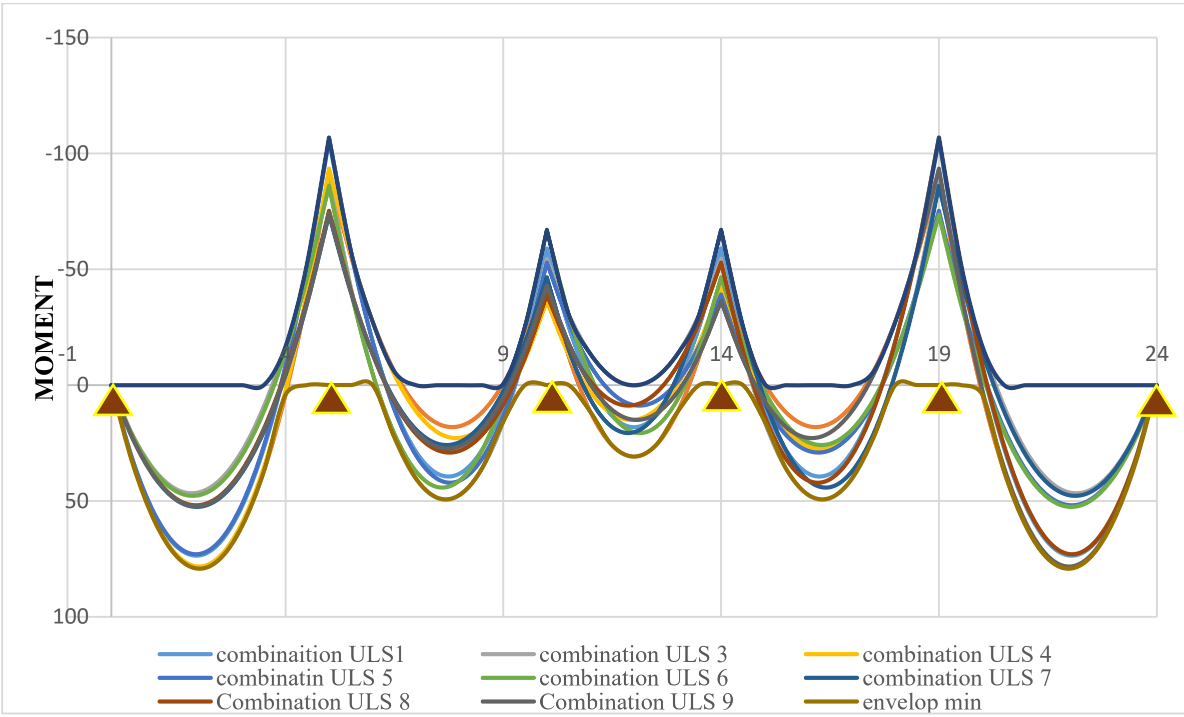


Figure 3.8. Moment diagrams of the beam

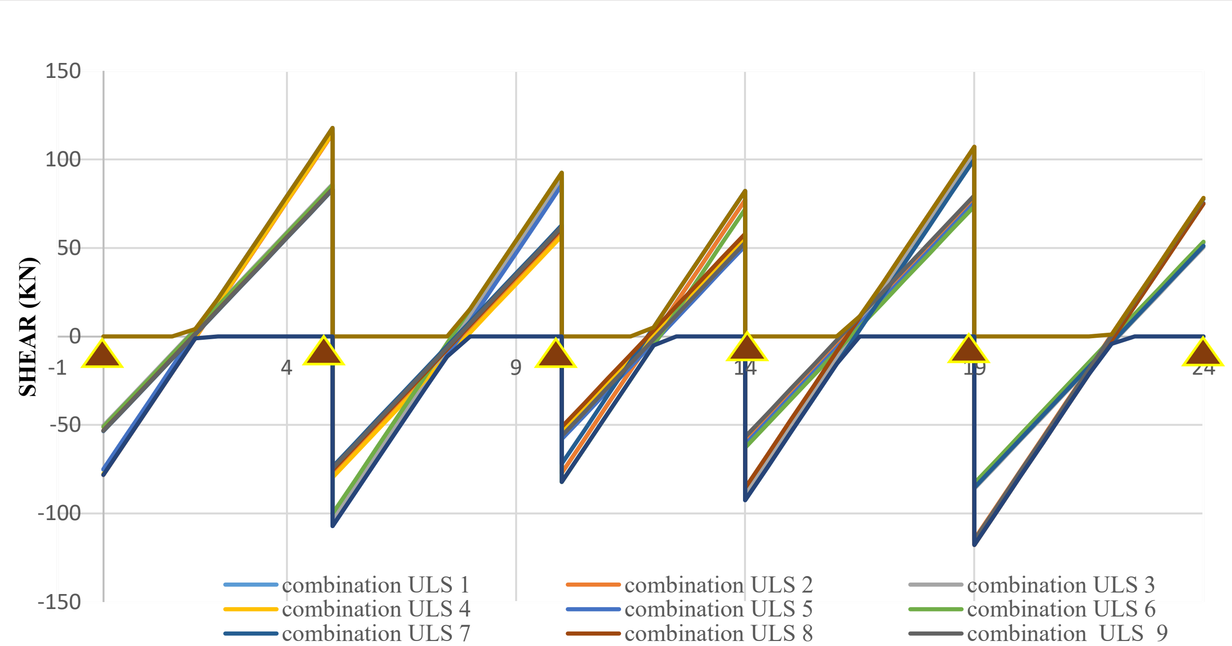


Figure 3.9. Shear diagrams of the beam

The obtained envelop curve are presented in figure 3.10 and figure 3.11.

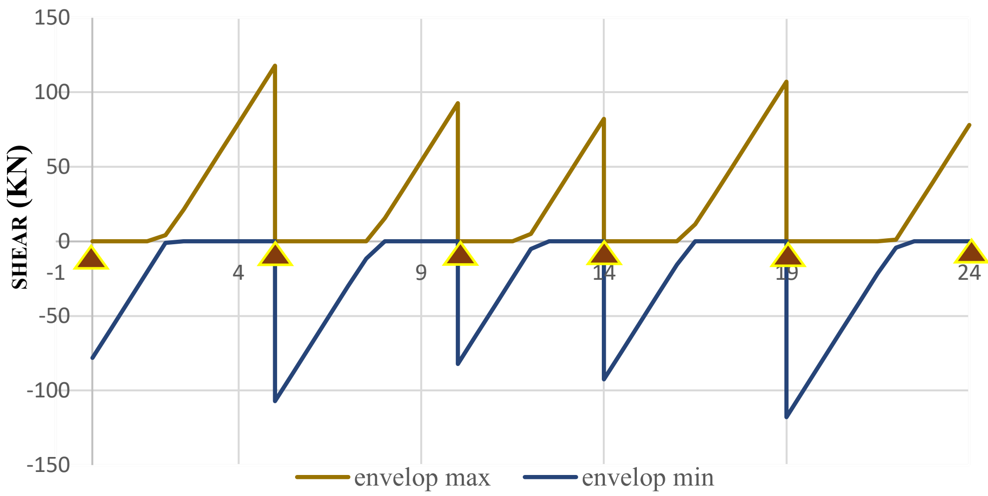


Figure 3.10. Envelop curve of the shear

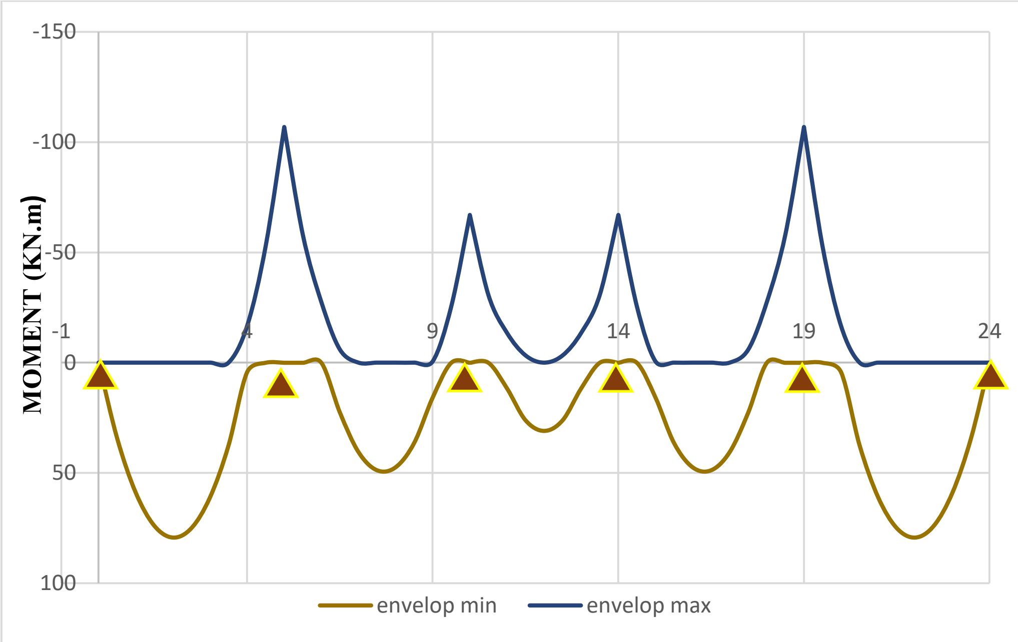


Figure 3.11. Envelop curve of the bending moment of the curve

From this graph and the equation 3.10, the steel reinforcement is computed and presented in table 3.4**.**

Table 3.4. Longitudinal steel reinforcement

steel reinforcement longitudinal					
	midspan	support 2	support 3	support 4	support 5
X (neutral axis)	83,95	116,93	70,02	70,02	116,93
As cal	486,30	677,33	405,62	405,62	677,33
As adpt	615,44	803,84	615,44	615,44	803,84
Med	79,2411	106,872	66,9791	66,9791	106,872
Mrd	100,284	126,834	101,626	101,626	126,834
n	4ϕ14	2ϕ14 2ϕ16	4ϕ14	4ϕ14	2ϕ14 2ϕ16

For the transversal reinforcement, the designed shear value is Ved=117.749kN. The application of the criteria presented in section 2.5.2.1.b,permit to calculate the design shear resistance without reinforcement. The value obtained is $V_{Rd,c}$ =1900, 8kN ≥Ved.

Thus, considering a diameter of 8 mm the design procedure presented on the section permits to obtain the spacing of the stirrups necessary to resist to the envelope of the shear solicitations which is 200 mm at mid span and 90 mm at the support. The distribution of the shear reinforcement is illustrated in Annexe 6.

3.3.1.2. Serviceability limit state design

The nine load arrangements inserted in SAP 2000 at the characteristic rare combination permit to obtain the solicitation curves presented in the figure 3.12.

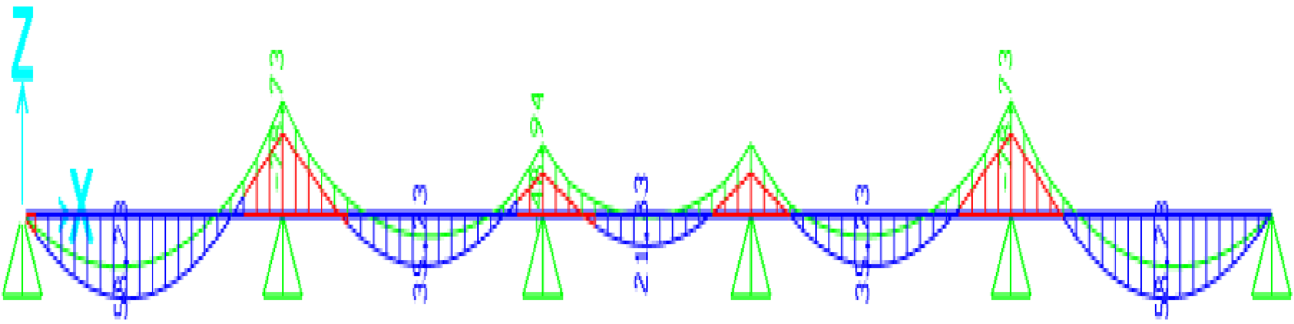


Figure 3.12. Bending moment curve at SLS

The stresses in concrete and in the reinforcing steel are respectively obtained as expressed in equations 2.22 and 2.23 .

$$\sigma_s = \frac{M_{Ed}(d-x)}{J_{cr}} \times n_\infty \qquad \text{(Eq. 3.4)}$$

$$\sigma_c = \frac{M_{Ed} \cdot x}{J_{cr}} \qquad \text{(Eq. 3.5)}$$

From these equation the values obtained are:

σ_c =8.04 N/mm²<0.8 F_{ck} and σ_s =164.97 N/mm² <0.4$f_{yd.}$ the stress is verified.

3.3.2. Column design results

The Colum designed is showed in figure 3.13.

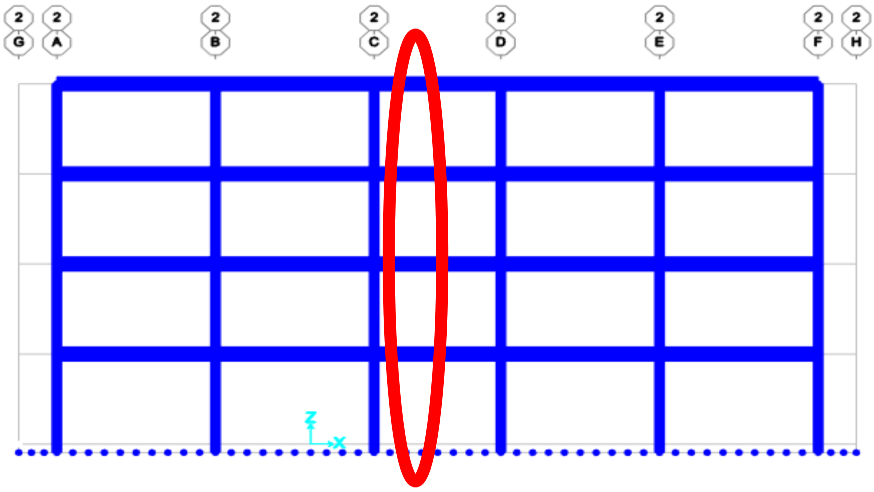


Figure 3.13 Most solicited column

According to equation 2.35 and 2.34, the value of Ac is given by the relation 3.6.

$$Ac \geq q \times Sr \times n / 0{,}6fcd = 93176.47 \text{ mm}^2 \quad \text{(Eq. 3.6)}$$

With $q = 1.35G_k + 1.5Q = 7.92kN/m^2$

The assumed crossed section of the column is 30x40 thus the area of the cross section is Ac=1200cm² and according to equation 2.6 and equation 2.16, the values of Asmin and Asmax is given in table 3.5. The maximum axial force of the envelop min obtained shown that the maximum axial force is Ned=-788.413KN.

Table 3.5.Transversal steel reinforcement section area

Transversal steel reinforcement		
Asmin	202.42	mm²
As max	3727.05	mm²
As assumed	461,58	mm²

The computed steel reinforcement value is As =461,58m² which correspond to 3Φ14.the spacing of the bars is thus $s = 86mm\ along\ x\ and\ s = 61mm\ along\ y$ which are less than the maximum spacing $s_{cl} = 280\ mm$; Φ8 will be use transversal reinforcement with a spacing of 20 at midspan and 15mm at the boundaries.

The designed column section is verified for each level because each couple of points M-N (moment-axial force) belongs to the M-N interaction diagram domain as shown in figure 3.14.

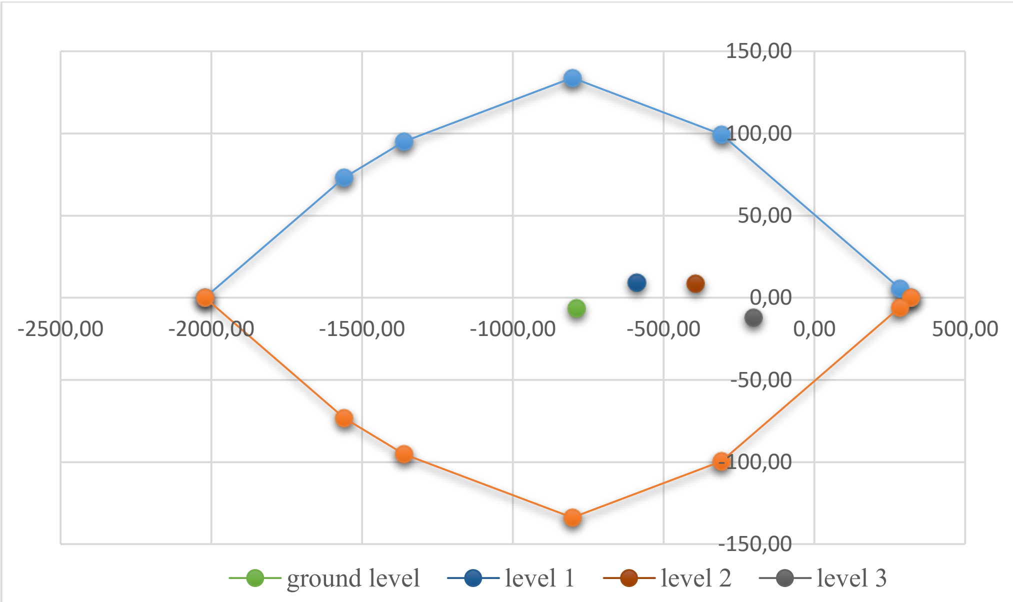


Figure 3.14. M-N interaction diagram along X direction

3.3.3. Foundation design result

The foundation chosen in a raft foundation. The assumed depth of the raft was given by the equation 2.54 is $d \geq \frac{l}{20}$ $thus\ d \geq 0.25$ we choosed d=30cm. According to the geotechnical report, admissible soil pressure is 2.5MPa. The solicitations along X and Y axis are obtained after modelling the panel as a thick plate resting on flexible support in SAP2000 .The influence area for each spring chosen for the analysis is 0.25 m^2. The vertical stiffness of the individual springs obtained using equation 2.53 is K=17500kN/m.The soil pressure due to the axial force is presented on figure 3.15 the graduation units is KN/m^2.

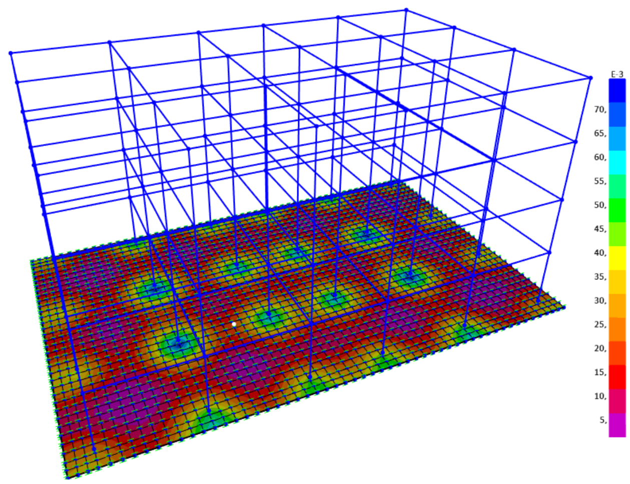


Figure 3.15. Stress distribution in the raft foundation

The design moments used at edge and mid span and the corresponding steel are presented in Table 3.6.

Table 3.6. Moment inside the raft

	X-direction	Y-direction	units
Positive moment at midspan	133.705	133.436	KN.m
Negative moment at edges	-16.452	-17.345	KN.m

Assuming a 16 mm diameter bars to be used and a concrete cover of 40 mm, the effective depth of the outer layer to be used in the design for moments in the short span direction is:

$$d_y = 300 - 40 - \frac{16}{2} = 352\ mm$$

The effective depth of the inner layer to be used in the design for moments in the long span direction is:

$$d_x = 352 - \frac{16}{2} = 344\ mm$$

From the effective depth the steel reinforcement obtained are presented in table 3.7.

Table 3.7. Steel reinforcement of the raft

Steel reinforcement area	X	Y	Units
As midspan	993,30	968,77	mm²/m
As edges	122,22	119,44	mm²/m

The adopted steel reinforcement is Φ16 for a spacing of 40mm at mid span and 50 mm at edges.

3.4. Analysis results and interpretation

The design of RC building only consider infill walls as distributed loads on the beams. The analysis performed aims to determine the resisting capacity of the infill wall. The comparative studies will be made between the simple frame, the frame with masonry infill, frame with earth brick infill, and frame with plasterboard infill. The materials characteristics is presented in table 3.8.

Table 3.8. Infill wall material properties

Material	Weight per unit volume (KN/m^3)	Elastic modulus E (MPa)	Poisson ratio υ	Thickness t (cm)
Masonry	19	2650	0,22	15
Plasterboard	8.32	4350	0.25	
Earth bricks	21	1500	0.21	

The following results have been obtained after performing the different analysis describe in the methodology.

3.4.1. Local behavior of infill walls

In order to show the influence of the infill walls in the global behavior of the building, it is important to evaluate the interaction between the infill wall and the frame structure. To do so, we consider an infill frame in which the infill wall material will vary. The characteristics of the frame are the following:

- Beam_ section: b=0.2 m; h= 0.4 m; length=5m;
- Column _Sections: b=0.3 m; h= 0.4 m ; height=3,2m

Figure 3.16 displays the model of the infill frame with and without meshed infill panel.

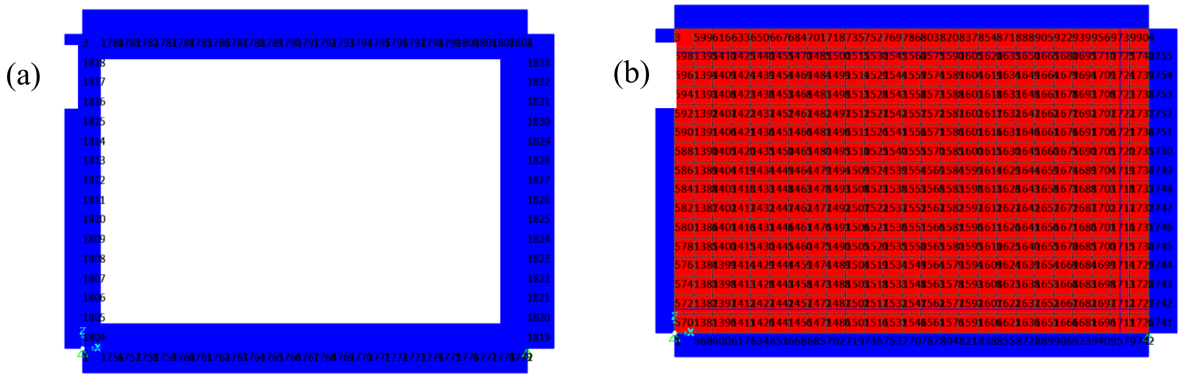


Figure 3.16.model of the frame (a) without an infill panel (b) with an infill pannel

The micro model discretization of the elements is perform in a way that the boundary nodes of infill frame correspond to the frame nodes. Afterward, a horizontal force F=5,14KN corresponding to 10% of the self-weight of the structure, is applied on one upper node. Figure 3.17 illustrate the deformed shape of the infill frame with and without infill walls obtained.

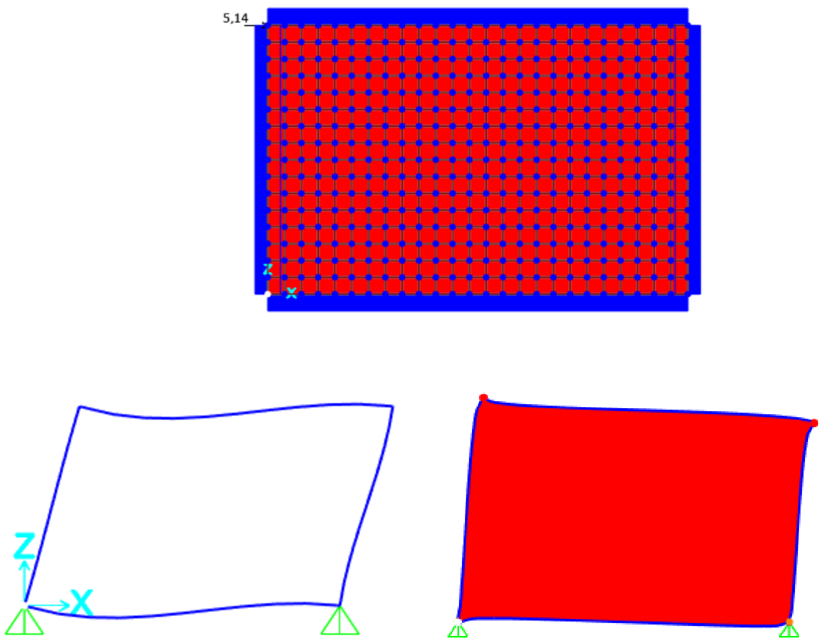
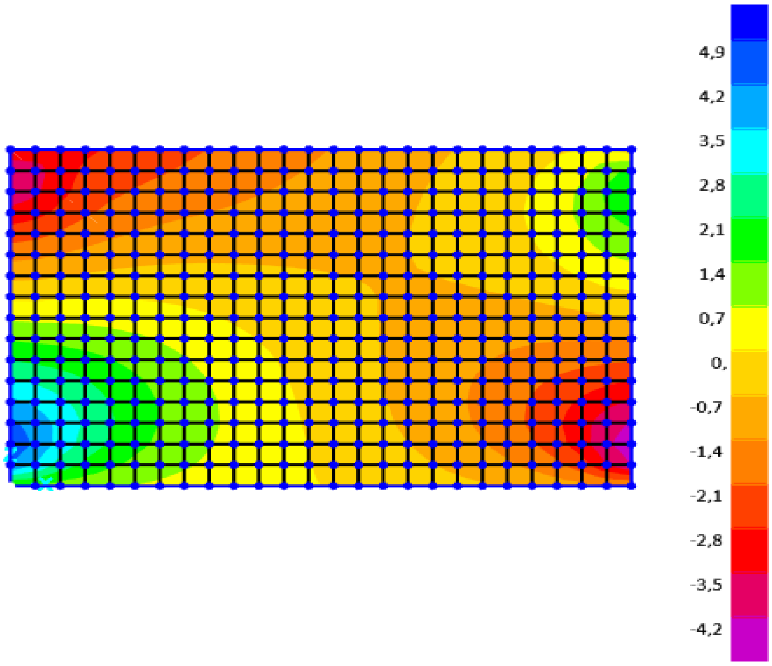


Figure 3.17. Deformed shapes under horizontal force

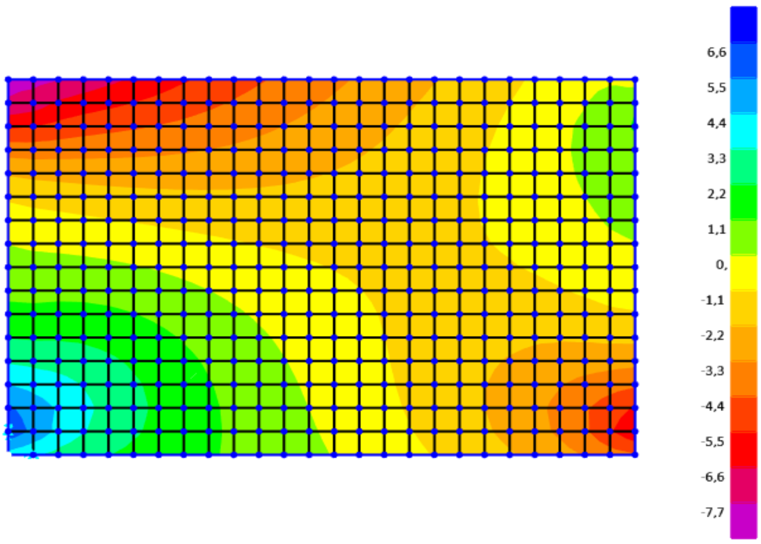
3.4.1.1. Stress state in the infill panel

As a result to the horizontal loading, in addition to horizontal displacement a stress variation is notice in the infill panel, and highest stress is noticed in the diagonal range of the panel. The stress state along the X-direction in drywall, masonry wall and brick wall is illustrated in figure 3.18.

(a)



(b)



(c)

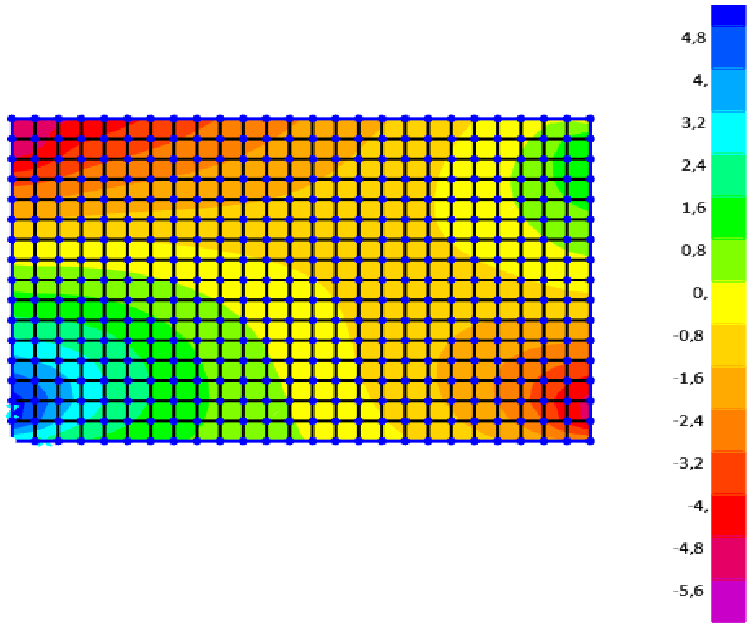


Figure 3.18. (a) Stress state in drywall (b) stress state in brick wall (c) stress state in masonry wall

The stress into the infill walls witness that they are in compression, thus they behave like compressive strut when a horizontal force is applied.

3.4.1.2. Horizontal frame deformation

It also noticed that, the presence of the infill wall contribute to the rigidity of the frame the obtained node displacement obtained after the application the horizontal force along X- axis equal to 5,14kN showed that the simple frame deformation decreased when the infill panel is considered in the analysis. The results are displayed in Table 3.9.

Table 3.9. Joint displacement along X-axis

Joint	joint displacement along X-axis			
	simple frame	frame with brick infll	frame with masonry infill	frame with drywall infill
1	0	0	0	0
2	0	0	0	0
3	0,000352	5,74E-07	4,74E-07	3,13E-07
4	0,000319	8,16E-08	1,81E-07	0,000000182

The observation made is that the horizontal deformation is less significant when the plasterboard infill wall is considered in the analysis.

3.4.1.3. Vertical frame deformation

The vertical deformation of the frame have been obtain by following the analysis steps presented in section 2.6.3.Only vertical distributed force is considered and the combination of vertical force applied is $1.35G_1$+1.35 G_2. Table 3.10 shows the equivalent distributed load corresponding to each type of wall.

Table 3.10. Equivalent distributed load of infill walls

Type of wall	γ (kN/m^3)	Height of the wall (m)	Thickness(cm)	Load (kN/ml)
brick wall	18	3.2	15	8.64
drywall	8.32			4.00
masonry	19			9.12

The obtained deformed shapes obtained are illustrated in figure 3.19

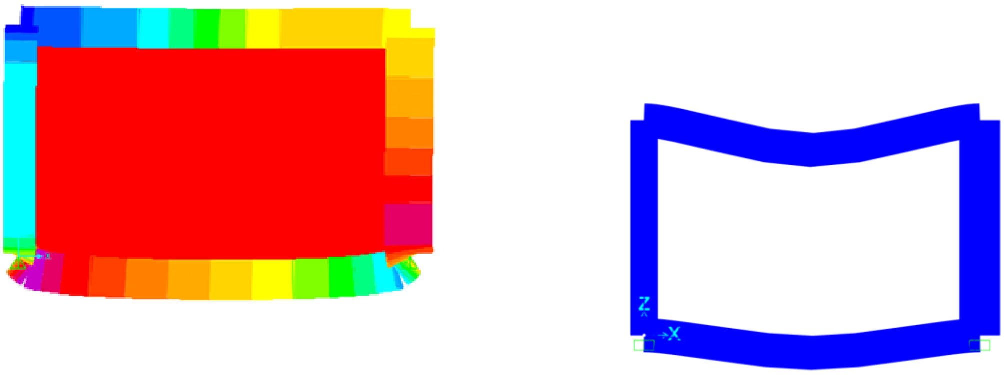


Figure 3.19. deformed shape of the frame under vertical distributed load

The first observation made is that the apparent deformation of the frame is low when infill walls are considered. This behavior is explain by the fact that infill wall can go into compression when the load is applied. The finite element analysis of the infill, show the presence of the stress all over the infill wall as illustrated in figure 3.20.

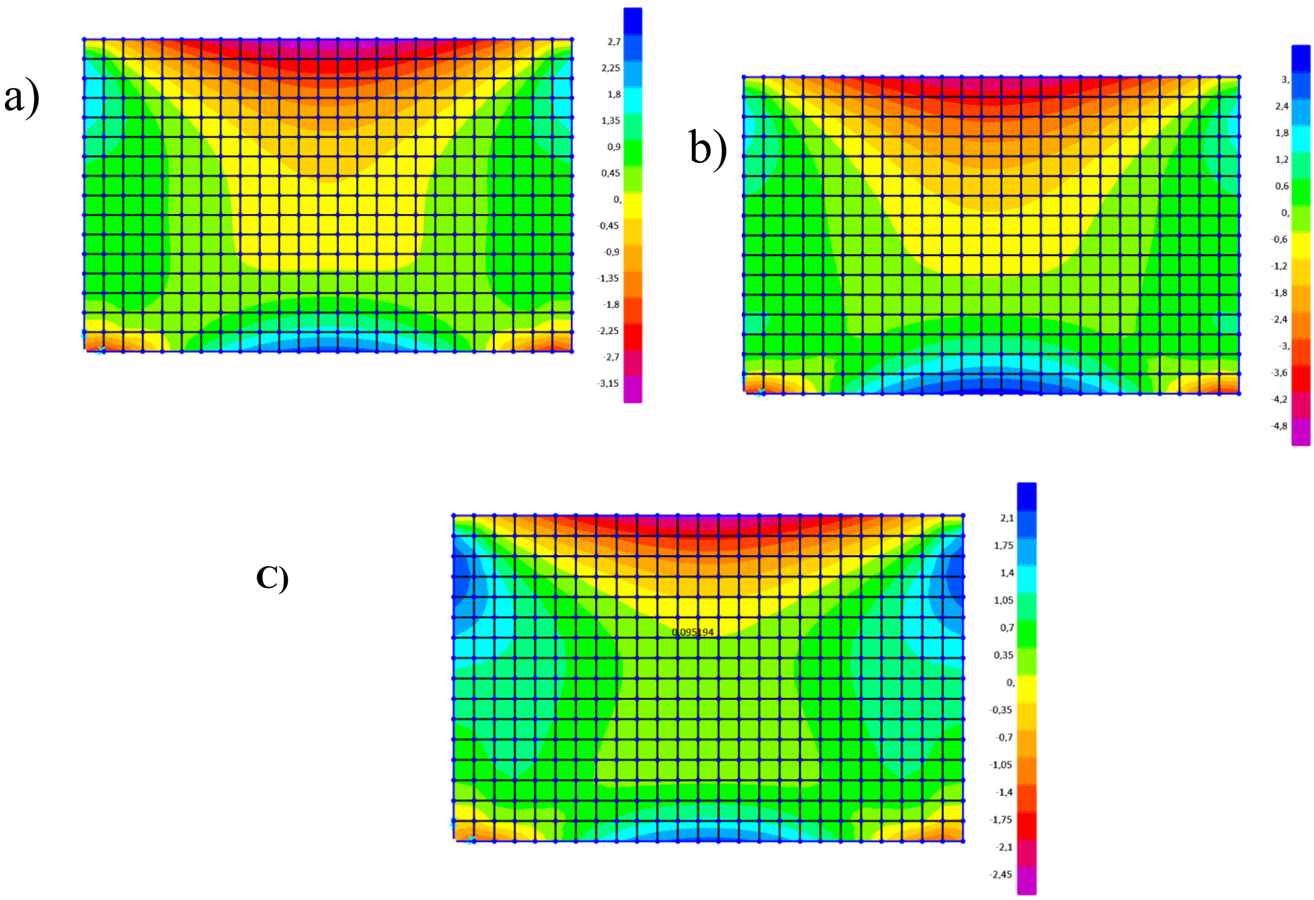


Figure 3.20. In infill wall behavior under distributed load a) masonry b) drywall c) brick wall

The deflection is firstly evaluated without the presence of infill walls and secondly considering infill wall self-weight. For the first model we consider a simple supported beam 5m with a section of 20x40cm.

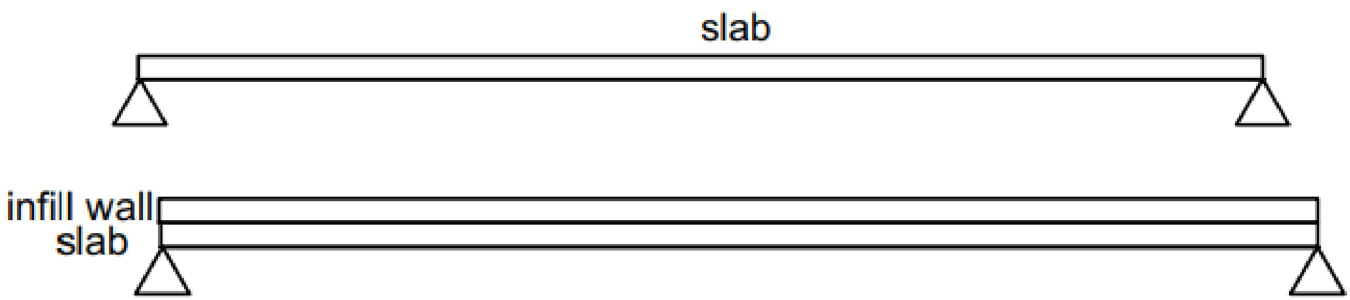


Figure 3.21.loads distribution

The deflection of the beam when we vary the infill wall is presented in table 3.11

Table 3.11. Beam deflection

Load case	Beam	Beam +slab	Beam slab+ brick	Beam+slab+ drywall	Beam+slab+ masonry
1 span	0	0.004761	0.007503	0.00603	0.07655

For the second model, we consider a 3 span simple supported beam. The length of each span is respectively 5m, 4m and 5m. Considering the possible configuration of one infill walls opening greater than 50% of the wall area. The distribution is presented in figure 3.22.

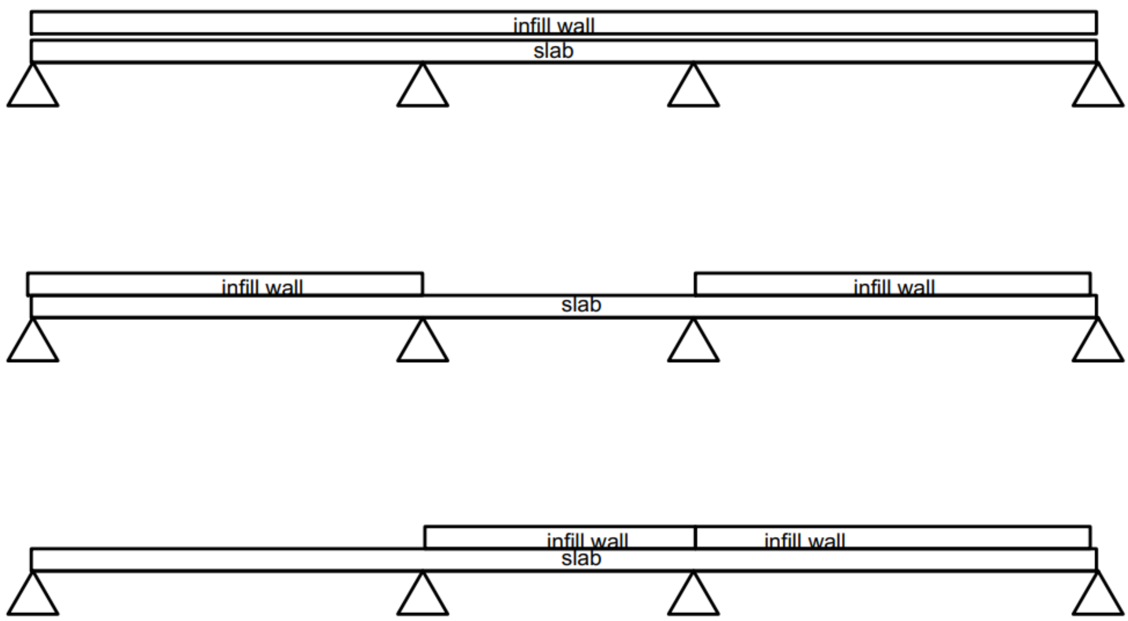


Figure 3.22. Distribution of the loads considering openings

The deflection obtained after the analysis are presented in table 3.12

Table 3.12. Deflection of each configuration

Load case	beam	Beam +slab	Beam+slab+ brick	Beam+slab+ drywall	Beam+slab+ masonry
1st configuration	1 span	0.002842	0.004479	0.0036	0.00457
	2 span	-0.000568	-0.000869	-0.00056	-0.000914
	3 span	0.002842	0.004479	0.0036	0.00457
2nd configuration	1 span	0.002842	0.004848	0.003771	0.004959
	2 span	-0.000568	0.001531	-0.001014	-0.001585
	3 span	0.002842	0.004848	0.003771	0.004959
3rd configuration	1 span	0.002842	0.002674	0.002764	0.002655
	2 span	-0.000568	0.000309	-0.000524	-0.000522
	3 span	0.002842	0.004279	0.003507	0.004359

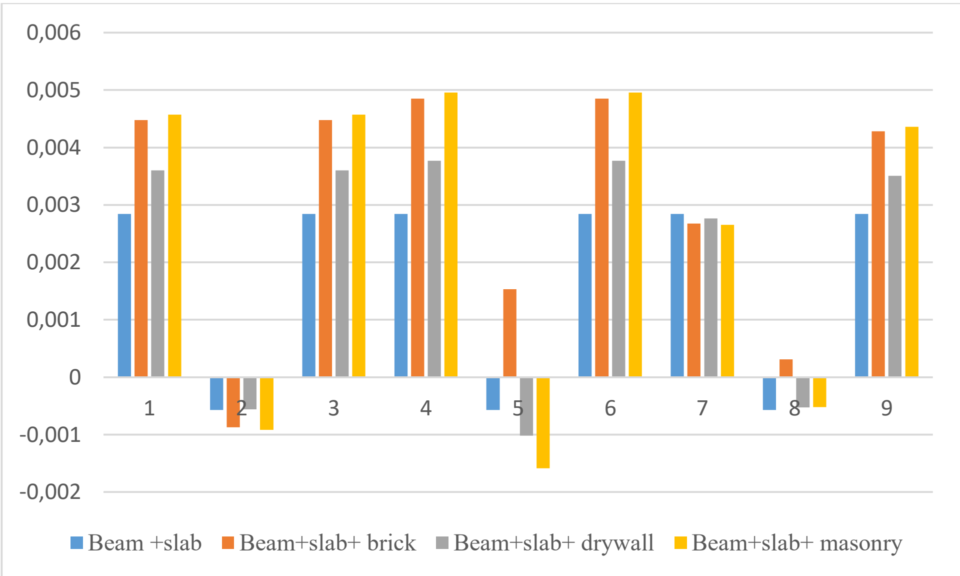


Figure 3.23. Variation of the deflection

From figure 3.23 the following observations are made:

- The deflection is more pronounced when the infill walls are considered, and the drywall offers a lower deflection.
- The variation of the deflection is lower when the drywall is used for infill

3.4.2. Structural influence of infill wall

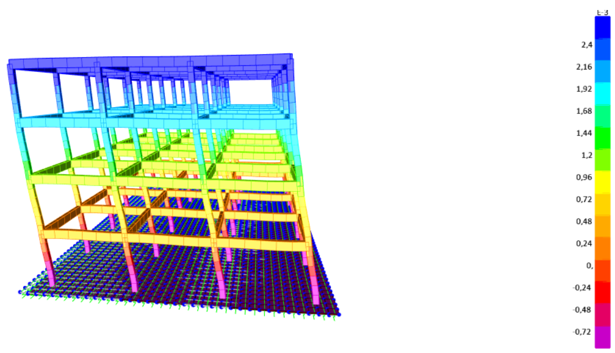
The section 3.3.2. Showed that the infill wall has a local influence on the frame resistance.in this section we will show the influence of infill walls on the whole structure .Remember that the conceived case study is a four story reinforced concrete residential building. The infill walls are modelled as strut for the analysis as illustrated in figure 3.20.the modeling of the infill wall depends on the architectural distribution of the building. The strut dimension (the width in particular), varying according to the type of material .The calculated strut widths are given in table 3.13.

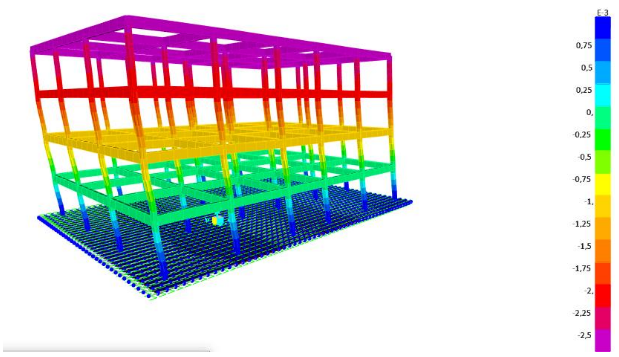
Table 3.13. Strut dimensions

strut dimension				
span length	marerial	a (m)	hcol	lamda
4m	masonry	0,35	3,2	2,4
	plasterboard	0,37	3,2	2,13
	earthbricks	0,33	3,2	2,77
5m	masonry	0,41	3,2	2,45
	plasterboard	0,43	3,2	2,17
	earthbricks	0,39	3,2	2,83

3.4.2.1. Modal analysis results

The modal analysis gives the modal period and natural frequencies of the building. The three fist modes are translational along the X-axis, translational along the Y-axis and torsional around the Z-axis. The different deformed shape for each modes is illustrated in fig 3.24.





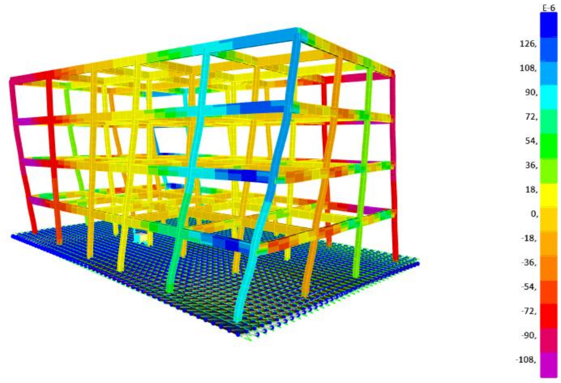


Figure 3.24 Deformed shapes for the three first modes

When infill walls are considered in the modal analysis, the modal periods vary according to the type of wall. Figuree3.17 shows the different values of the modals periods according to each case. The first observation made is that the values of the modal periods decrease when the infill walls are considered. According to the Eurocode 8 the right value of the period of vibration is between 0 and 2s the lower it is, the better it is. According to Figure 3.25, the presence drywall infill reduces the most the period of vibration of the structure compare to masonry and brick walls.

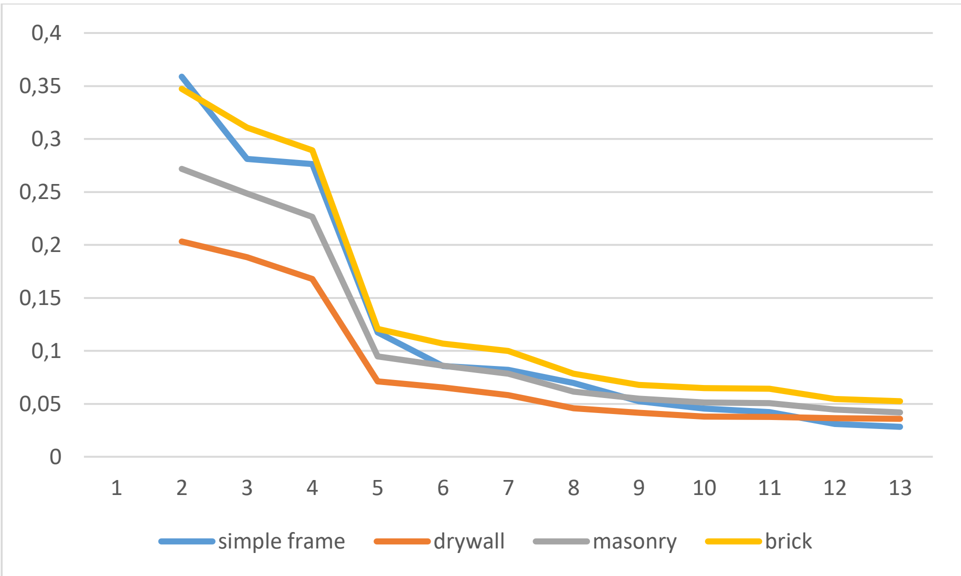


Figure 3.25. Modal period's variation

3.4.2.2. Influence of the infill walls on the lateral deformability of the building

The deformability of the building can be torsional or translational. The translation deformation can be evaluated by the nodes displacement displacement along the direction of the applied force. The force applied is 10% of the self-weight of the building. The self-weight calculated is P= 1708, 8KN. The lateral force is thus F=170.8KN. The figure 3.26 illustrate the deformation of the building with and without infill walls. For this analysis, the foundation are considers as fixed base.

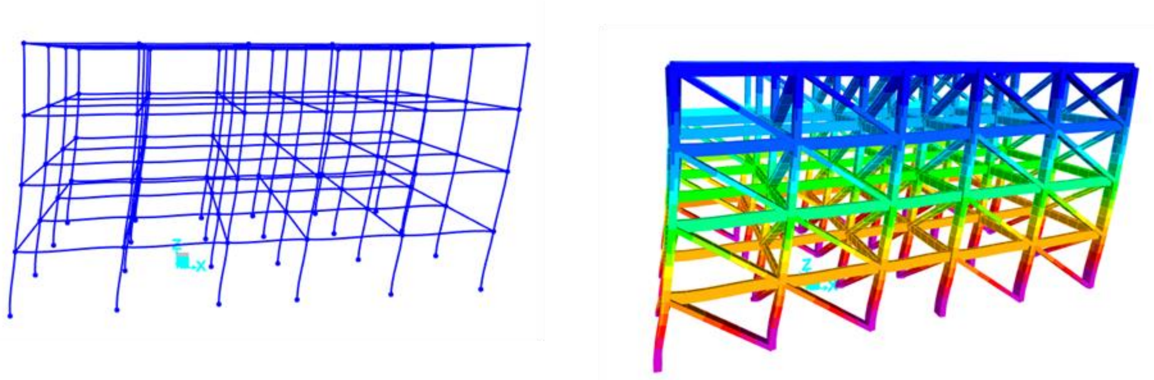


Figure 3.26. Deformed shapes of the frame after the analysis

The figure 3.27 represents the nodes displacement along the x-axis when the structure is submitted to the horizontal force F.the first observation is that the presence of infill walls reduce considerably the nodes displacement. It also noticed that the masonry infill wall offer a less performance than both brick wall and drywall.

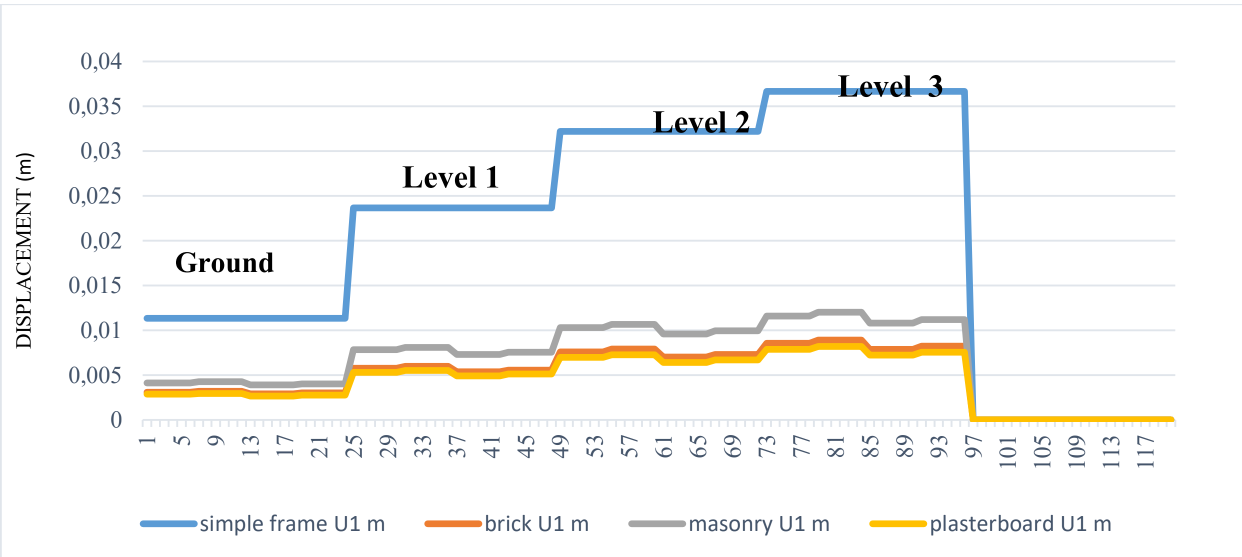


Figure 3.27. Nodes displacements

From the lateral displacement of the levels, the global stiffness of the building as the figure 3.28 illustrate. The stiffness is a measure of how much force is require to displace a building by a certain amount .the stiffer is the building, the greater is the force needed. The plaster infill wall increase the rigidity of the building especially the ground level .This characteristic increase the stability and the rigidity of the building.

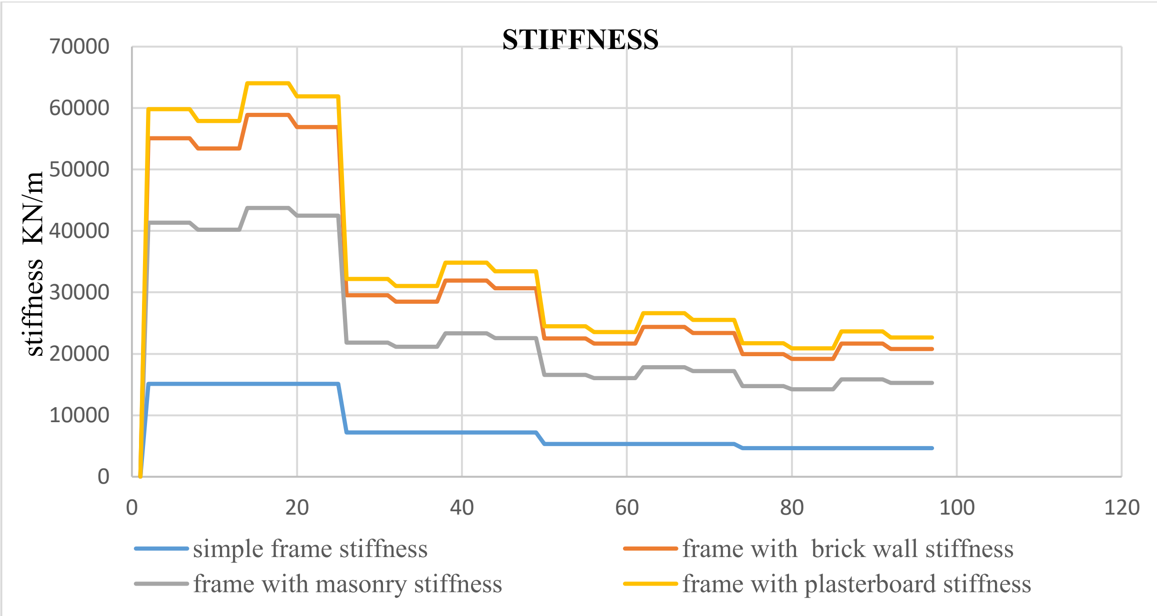


Figure 3.28. Stiffness at each level

Figure 3.28 shows an increase of the building stiffness of a column row from the top to the bottom. The rigidity of the building is more significant with the presence of infill walls.

Conclusion

The main objective of this chapter was the presentation and the interpretation of the result obtain from the analysis performed on the case study, by following the methodology described in chapter 2. The case study was first presented, and the design of the structural elements permit to obtain their respective cross sections and steel reinforcement. Afterwards, the analysis performed on the case study showed that the presence of infill wall reduce the lateral and the vertical deformability of the buiding.it was also noticed that the drywall offers a better performance in term of stiffness and deflection than masonry and brick infill wall.

GENERAL CONCLUSION

This study had for main objective to show how the consideration of infill walls influences the design of a multistory reinforced concrete building.to lead the research, most used type of infill walls were presented with their properties (thermal, wind resistence, acoustic and failure mechanism), then different type of multi-story building and the various failure mechanism than can occur in a RC building.

Reinforced concrete building design and modelling steps were presented in the methodology, followed by the description of the different analysis that had to be performed. The proposed case study was a seventh story building with one basement and we considered different types of infill for the analysis especially masonry, compressed earth brick (CEB) and plasterboard. Infill wall were modelled as strut for the macro model and as thin shell for the micro model. Infill walls with opening greater than 50% the infill wall area where not took into account for the modelling. Soil structure interactions were also considered. The micro modelling permitted to study the local behavior of an infill frame with and without an infill wall meanwhile the macro modelling showed the global behavior of the structure. The modal analysis enquired us about the period of vibration of the building.

It has been noticed that the presence of infill wall reduce the period of vibration of the building thus, make it more stable. Meanwhile, the Finite Element Analysis showed an increase of the rigidity along the X-axis and the Y-axis and a reduction of the beam deflection when infill walls are considered in the analysis. The comparative study between drywall, masonry wall and brick wall showed that drywall has a better resisting performance than the masonry wall and brick wall. It also showed that because of its high module of elasticity and its lightness, the drywall offer a better resistance to deflection.

The analysis result recommend the consideration of infill wall in the design of multistory building in seismic zone in particular the use of drywall for infill to procure a better resistance to e wind. Depending on the accessibility to plasterboard as infill material, CEB can also be used as a palliatives more ecologic solution.

Further experimental investigation can be lead to determine which material offer a better thermal and acoustic performance or a better resistance to Earthquakes. The influence of the configuration of infill wall on the performance of the structure can also be studied.

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ANNEXES

ANNEXE 1. Values of minimum cover, $c_{min,dur}$, requirements with regard to durability for reinforcement (Eurocode 2)

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

ANNEXE 2. Building category (Eurocode 2)

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		

ANNEXE 3. Imposed loads of concrete building (Eurocode 2)

Table 6.2 - Imposed loads on floors, balconies and stairs in buildings

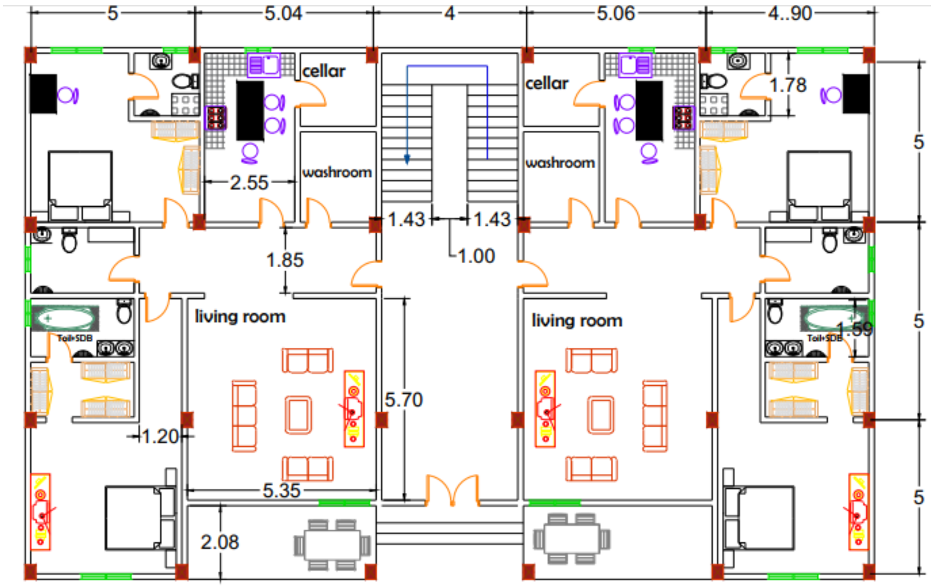
Categories of loaded areas	q_k [kN/m^2]	Q_k [kN]
Category A		
- Floors	1,5 to2,0	2,0 to 3,0
- Stairs	2,0 to4,0	2,0 to 4,0
- Balconies	2,5 to 4,0	2,0 to 3,0
Category B	2,0 to 3,0	1,5 to 4,5
Category C		
- C1	2,0 to 3,0	3,0 to 4,0
- C2	3,0 to 4,0	2,5 to 7,0 (4,0)
- C3	3,0 to 5,0	4,0 to 7,0
- C4	4,5 to 5,0	3,5 to 7,0
- C5	5,0 to 7,5	3,5 to 4,5
category D		
- D1	4,0 to 5,0	3,5 to 7,0 (4,0)
- D2	4,0 to 5,0	3,5 to 7,0

ANNEXE 4. Basic ratios of span/effective depth for reinforced concrete members without axial
compression (Eurocode 2)

Structural System	K	Concrete highly stressed ρ = 1,5%	Concrete lightly stressed ρ = 0,5%
Simply supported beam, one- or two-way spanning simply supported slab	1,0	14	20
End span of continuous beam or one-way continuous slab or two-way spanning slab continuous over one long side	1,3	18	26
Interior span of beam or one-way or two-way spanning slab	1,5	20	30
Slab supported on columns without beams (flat slab) (based on longer span)	1,2	17	24
Cantilever	0,4	6	8

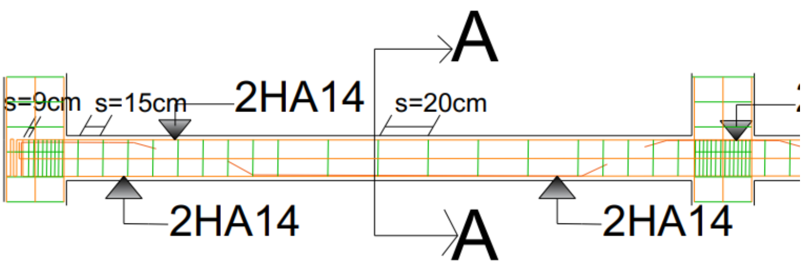
Note 1: The values given have been chosen to be generally conservative and calculation may frequently show that thinner members are possible.
Note 2: For 2-way spanning slabs, the check should be carried out on the basis of the shorter span. For flat slabs the longer span should be taken.
Note 3: The limits given for flat slabs correspond to a less severe limitation than a mid-span deflection of span/250 relative to the columns. Experience has shown this to be satisfactory.

ANNEX 5. Architechtural plan of the levels

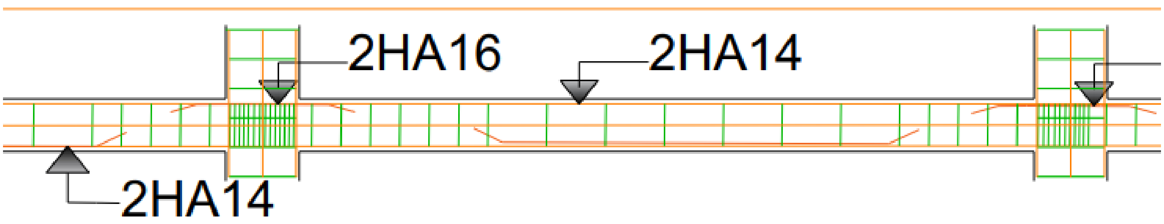


ANNEXE 6. Beams steel reinforcement

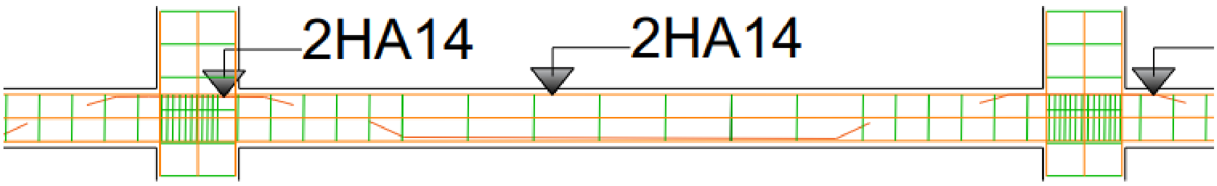
T1



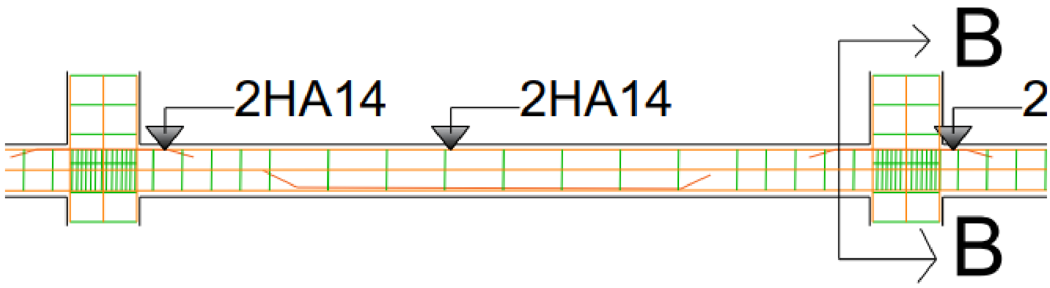
T2



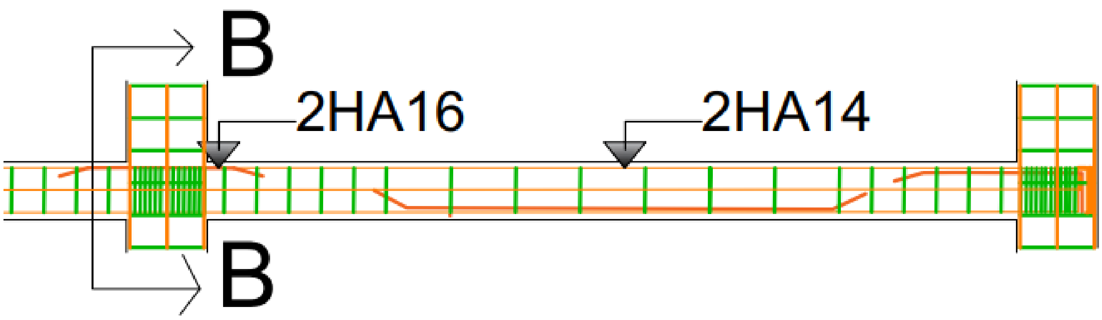
T3



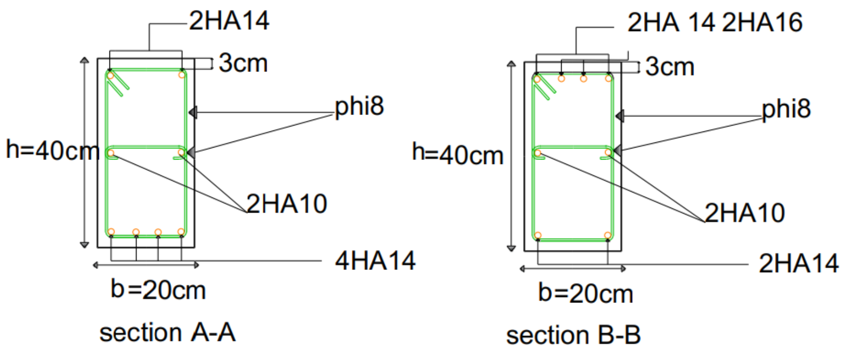
T4



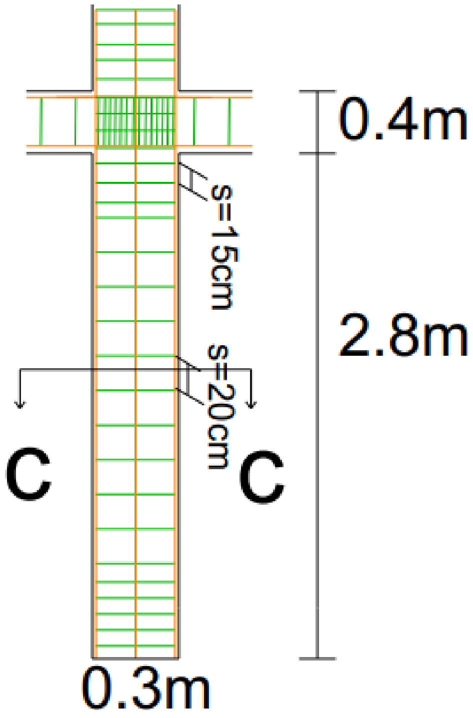
T5



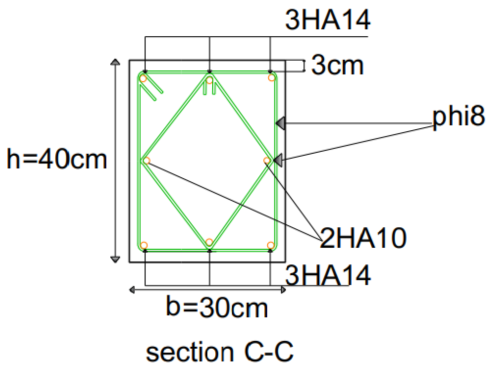
Beam cross sections



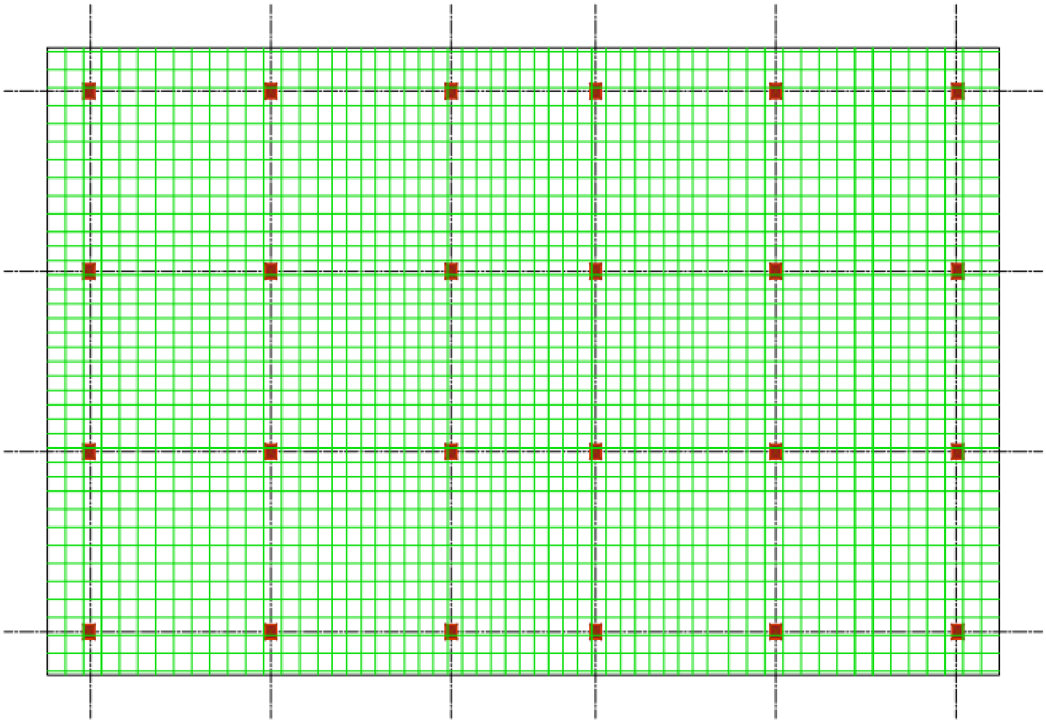
ANNEXE 6. Columns steel reinforcement



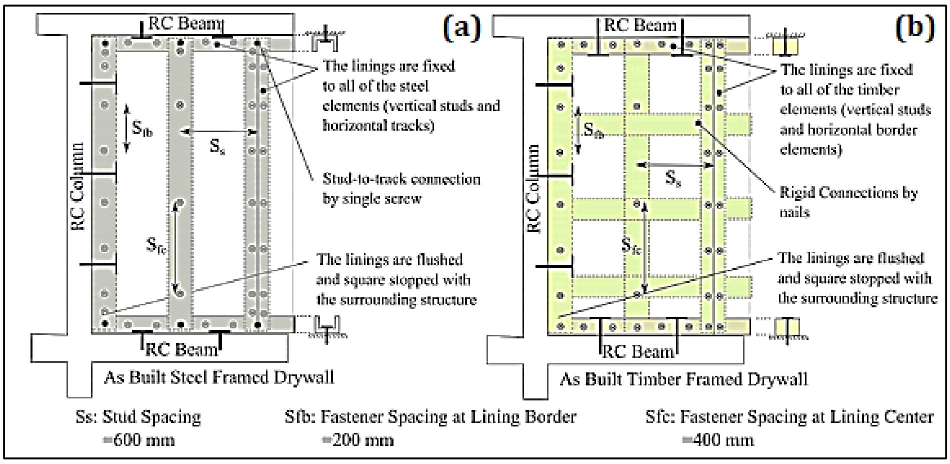
columns cross section



ANNEXE 7:raft foundation steel reinforcement configuration

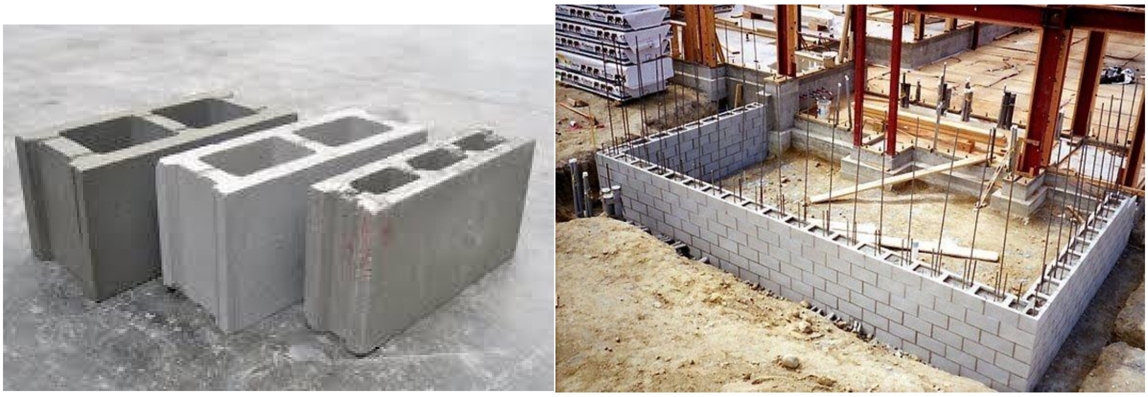


ANNEX 8: Typologiy of infill walls



Cross section of a steel stud drywall (a) and a timber stud drywall (b) (Ali Sahin Tasligedik et al.2014)

Glass partition wall



Representation of a reinforced masonry wall (the Constructor .org)

wood walls (JaeChoi 2022)