REPUBLIQUE DU CAMEROUN

Paix - Travail – Patrie

Ecole Nationale Supérieure des Travaux Publics

DEPARTMENT OF CIVIL ENGINEERING

DEPARTMENT DE GENIE CIVIL

REPUBLIC OF CAMEROON

Peace – Work – Fatherland

UNIVERSITÀ
DEGLI STUDI
DI PADOVA

DEPARTMENT OF CIVIL ENGINEERING

DEPARTMENT DE GENIE CIVIL

STRUCTURAL ANALYSIS OF A REINFORCED CONCRETE MULTI-STOREY BUILDING: CASE STUDY OF THE NEW ADMINISTRATIVE BLOCK OF THE NATIONAL ADVANCED SCHOOL OF PUBLIC WORKS YAOUNDE

A thesis submitted in fulfilment of the requirement for the degree of Masters in Engineering

(MEng)

Curriculum: Structural engineering

Presented by:

TATTO GAETAN JUNIOR

Matricule Number:

15TP20903

Supervised by:

Prof. Eng. Carmelo MAJORANA

Co-Supervised by:

Eng. Giuseppe CARDILLO
Dr. Eng. Guillaume Hervé POH'SIE

ACADEMIC YEAR: 2019/2020

DEDICATION

I dedicate this work to God

ACKNOWLEDGEMENTS

I give thanks to GOD for covering me with his grace, giving me the patience and the necessary energy to accomplish this work. This document is the fruit of the combined efforts of several individuals that I will like to thank here.

I address my deepest acknowledgments to the President of the jury of this thesis presentation, Prof. Eng. Carmelo MAJORANA for the honour he gave me to preside the jury.

I express my sincere gratitude to the Examiner of this jury, Dr NIMPA Giscard Desting of ENSPY for his criticisms and observations to ameliorate this work.

I would like to express my sincere and intense gratitude to my supervisors Prof. Eng. Carmelo MAJORANA, Eng. Giuseppe CARDILLO and Dr Eng. Guillaume Hervé POH'SIE for the patience, the guidance, continuous encouragement, constructive criticism they gave me during this thesis work.

I also express my deep respect and consideration to the Director of National Advanced School of Public Works, Pr. NKENG George ELAMBO for his great contribution and devotion to our promotion.

I thank Prof. MAJORANA and all the collaborators of University of Padova who are the principal initiators of the Engineering curricula at the NASPW.

I would also thank Prof. MBESSA Michel, head of the department of civil engineering for his encouragement, his tutoring and valuable advices.

I express my sincere gratitude to all the teaching staff of ENSTP and University of Padua for their good quality of teaching and the motivation they developed in us during these years.

I particularly thank all classmates, my friends and colleagues especially CHEFOR Mbafor, CHIA Ghislain Kitu and all members of LA SQUADRA for their assistance and constant encouragement throughout my research work.

Finally, but not the least, I would like to express my deepest gratitude to my father M. TATTO Germain Colince, my mother Mrs Bucho Therese Chengang, my brothers and sisters who made this possible, for their encouragement, support and patience during these years.

ABBREVIATION LIST

CTBUH	*Council of Tall buildings and Urban Habitat*
EC	*Eurocode*
RC	*Reinforced concrete*
SAP	*Structural Analysis Program*
SLS	*Serviceability Limit State*
SSI	*Soil Structure Interaction*
ULS	*Ultimate Limit State*
A	*Area of the cross section*
A_c	*Area of the concrete cross section*
A_s	*cross sectional area of reinforcement*
A_s'	*Addtional cross sectional area of reinforcement*
$A_{s,r}$	*real cross sectional area of reinforcement*
B_x	*width of the footing at the base*
b	*breadth*
C_{min}	*Minimum concrete cover*
$C_{min,b}$	*Minimum cover due to bond requirement*
$C_{min,dur}$	*Minimum cover due to environmental conditions*
d	*Effective depth of a cross-section*
E	*Elastic modulus*
f_{cd}	*cylindrical concrete compressive strength*
f_{yd}	*Design yield strength of reinforcement*
G_1	*Characteristic value of self-weight*
G_2	*Characteristic value of a permanent action*
g_1	*Self-weight of structural element*
g_2	*Self-weight of non-structural elements*
H	*Height of the building*
h	*section depth*
I	*inertia of the element in the buckling direction*
i	*radius of gyration of the uncracked concrete section*
k	*stiffness constant*
M_{ED}	*Bending moment at support*
M_{RD}	*Resisting moment*
N_{Ed}	*Axial force in the cross section due to loading or prestressing*
N_{sd}	*Axial load compute using the recovery area of the column.*
n	*Number of floors in a building*
P	*Axial load at the base of the column*
Q_k	*Characteristic value of the imposed loads*
q	*Distributed load on the building*
S_r	*Recovery area*
V	*shear force*
V_{Ed}	*Acting shear load*

V_{Rd}	*Shear strength*
V_{Rdc}	*design shear resistance of the section without shear reinforcement*
x	*neutral axis*
x_r	*real neutral axis*
α	*angle between the shear reinforcement and the axis of the beam*
α_{cw}	*coefficient for the state of the stress in concrete*
γ	*Weight density*
γ_c	*partial safety factor of concrete*
γ_G	*Partial factor for permanent actions*
γ_P	*Partial factor for prestressing actions*
γ_Q	*Partial factor for variable actions*
γ_{Qi}	*Partial factor for variable actions i*
γ_s	*Partial safety factor for steel*
$\Delta C_{dur\ add}$	*Add reduction of minimum cover for use of additional protection*
$\Delta C_{dur,st}$	*Reduction of minimum cover for use of stainless steel*
$\Delta C_{dur,\gamma}$	*Additive safety element*
θ	*inclination of the cracks or the concrete struts*
λ	*Slenderness*
λ_{lim}	*Limit value of slenderness*
ν	*Poisson's ratio*
σ_{adm}	*Admissible pressure on the soil*
σ_c	*Stress in the concrete*
σ_s	*Stress in the reinforcement*
σ_t	*axial tensile failure stress*

ABSTRACT

The main objective of this work was to analyse and to design a structure ensuring human safety. This project presented a detailed study of an irregular office building with seven floors. The structure is located in the city of Yaoundé, an area of low seismicity. As cities continue to expand with time, to safeguard our rural areas against an eventual breaking point, the multi-storey building as a building type is a possible solution by way of conquering vertical space through agglomeration and densification. The project consisted in modelling the concrete elements using linear elastic analysis without redistribution. The stability of the structure was ensured by the beams, columns and foundations (footings). The study and analysis of this project were established by the software SAP2000. The analysis of the vertical stability consisted in designing the columns. To that effect, an analysis of the loads had been realised. The calculations have been based on the European norms and code of practices. A non-linear static analysis, based on the pushover analysis method, led to the determination of the deformations in the structure. The results of the structural analysis made it possible to determine the modes shapes and modal participation. Finally, the soil- structure interaction will also be discussed in this work. This work permitted one to obtain a section of 20cm by 40cm for the beams, 50cm by 50cm for the columns and 200cm by 200cm for the footings. All these calculations were made taking into account the calculation and verification rules according to Eurocode.

Keywords: Reinforced concrete, modal analysis, pushover analysis, structural joint.

RESUME

L'objectif principal de ce travail était d'analyser et de concevoir une structure assurant la sécurité des personnes. Ce projet a présenté une étude détaillée d'un immeuble de bureaux irrégulier de sept étages. La structure est située dans la ville de Yaoundé, une zone de faible sismicité. Alors que les villes ne cessent de s'étendre au fil du temps, pour sauvegarder nos zones rurales contre un éventuel point de rupture, le bâtiment à étages comme type de construction est une solution possible par la conquête de l'espace vertical par l'agglomération et la densification. Le projet a consisté à modéliser les éléments en béton par une analyse élastique linéaire sans redistribution. La stabilité de la structure était assurée par les poutres, les colonnes et les fondations (semelles). L'étude et l'analyse de ce projet ont été établies par le logiciel SAP2000. L'analyse de la stabilité verticale a consisté à concevoir les colonnes. A cet effet, une analyse des charges a été réalisée. Les calculs ont été basés sur les normes et les codes de pratiques européens. Une analyse statique non linéaire, basée sur la méthode de l'analyse pushover, a permis de déterminer les déformations de la structure. Les résultats de l'analyse structurelle ont permis de déterminer les formes des modes et la participation modale. Enfin, l'interaction sol-structure sera également abordée dans ce travail. Ce travail a permis d'obtenir une section de 20cm par 40cm pour les poutres, 50cm par 50cm pour les colonnes et 200cm par 200cm pour les semelles. Tous ces calculs ont été réalisés en tenant compte des règles de calcul et de vérification selon l'Eurocode.

Mots clés : Béton armé, analyse modale, analyse pushover, joint structurel.

LIST OF FIGURES

LIST OF TABLES

TABLE OF CONTENT

GENERAL INTRODUCTION

Nowadays, construction of reinforced concrete multi-storey building has become more frequent in the world as the cities and the world population continue to expand. The development of the high-rise building has followed the growth of the city closely. The process of urbanisation, which started with the age of industrialisation, is still in progress in developing countries like Cameroon. The role of infrastructure in economic growth and development can be hardly overstated. The hospitality sector in Cameroon plays a pivotal role in enhancing the country's economic growth. According to Eldemery Ibrahim, 'the reasons for adopting high rise building could be solutions for density problems and lack of available land for development.'

As the population rises with the time, there is a need for the high development and construction of multi-storey buildings in the country. The choice of reinforced concrete is chosen due to the high availability of the materials necessary for construction. These reinforced concrete structures contain prismatic sections which are common in developing countries, which resist applied loads without any appreciable deformation of one-part relative to another.

The objective of this work is to design a reinforced concrete building while avoiding the collapse of the structure to ensure the safety of human lives but also to limit damage during an earthquake of low intensity. This work shall be discussed in three main parts.

In the first part this work, a general review shall be done on the different knowledge and concepts based on the action and its typologies. A small study of reinforced concrete, its constituents, and its workability shall also be enumerated as well in the work. In addition to this, some references and discussion shall be implemented on multi-storey buildings, limit state design and structural frame members of the reinforced concrete shall be enumerated in this part of the work. And to conclude this part, a recap of the solicitations and the norms for designing will be mentioned in here.

The second part of this work will bring a general presentation of the building in terms of the site and location, the geometric characteristics and parameter taken for the design of the building. Then the actions and load combination taken for the structural analysis is mentioned here so as to use the solicitations required for the analysis and design of the building. Furthermore, the methodology in presenting the procedure necessary in designing the building is done as well in this second part of the work. The different analysis done on the building such as the modal analysis and pushover analysis shall be as well be taken in the part of this work.

The final part of this work presents all the results obtained from the methodology of the previous chapter necessary for the analysis and design of the building. The different results shall also be accompanied graphically where dimly necessary.

CHAPTER 1. LITERATURE REVIEW

Introduction

Structural analysis is the branch of structural engineering which involves with the behaviour of structures such as buildings, bridges, dams, tower, retaining wall, foundations, and so on. In other words, it is the determination of the effects of load on structures and their components. A structural analysis may have to be performed at three levels using different models which are the global analysis, the member analysis and the local analysis. A structure is a system of interconnected members used to support external loads. One can also define a structure as a complex of columns, beams, girders, spandrels, and trusses connected to one another and to the columns anchored in foundation. The objective of this chapter is to point out the theories and ideas based on actions (load). Emphasis shall be made on reinforced concrete, its constituents and the workability of concrete. Furthermore, a brief detailing of high-rise building based on some references shall be noted in this chapter. In addition to that, a brief study on the different structural members as well as the different limit state design shall be made. Then a look shall be made on the different solicitation required under these limit state design and the different norms to be used in this work.

1.1. Action

Literally, structural actions (or loads) are forces, displacement or acceleration that are applied to components of a structure. These actions could be either direct forces of indirect forces. The direct action includes as earlier mentioned forces and for the indirect actions include temperature changes, moisture variation, uneven settlement or earthquakes. According to Bungale.S (2017), there are two different sources of building loads namely geophysical and human made source.

1.1.1. Geophysical source

The geophysical forces, being the result of continuous changes in nature, may be further subdivided into gravitational, meteorological, and seismological forces. As a result of gravity, the weight of a building itself produces on the structures forces called dead load, and this load remains constant throughout the building's life span. The ever-changing occupancy of a building is also subject to gravitational effects producing a variation of loads over a period of time. Meteorological loads vary with time and location and appear in the form of wind, temperature, humidity, rain, snow, and ice. Seismological forces result from the erratic motion of the ground (Bungale S, 2017).

1.1.2. Human-made source

The human-made sources of loading may be the variations of shocks generated by cars, elevators, machines, and so on, or they may be the movement of people and equipment or the result of blast and impact. Furthermore, forces may be locked into the structures during the manufacturing and construction processes. The stability of the building may require prestressing, which induces forces (Bungale S, 2017).

Knowing of these two sources of building loads, it is of importance to discuss on the different types of actions existing on a structure.

1.1.3. Types of action

There exist so many types of action acting on a structure but for the interest of this work, it is necessary to discuss on 5 main different types of actions.

1.1.3.1. Permanent action (*G*)

Based on EN 1990-E-2002 or commonly known as Eurocode 0, a permanent action is one action that is likely to act throughout a given reference period and for which the variation in magnitude with time is negligible, or for which the variation is always in the same direction (monotonic) until the action attains a certain limit value.

1.1.3.2. Variable action (Q)

it is an action in which the change in magnitude with time is neither negligible nor monotonic (the direction varies with time).

1.1.3.3. Accidental action

Eurocode 0 defines an accidental action as one that occur within a short period of time but the magnitude of this action is significant. This action also tends to affect the structure during the design working life. An accidental action can be expected in many cases to cause severe consequences unless appropriate measures are taken. Another important note to be taken here is the fact that impact, snow, wind and seismic actions may be variable or accidental actions, depending on the available information on statistical distributions.

1.1.3.4. Seismic action (A_E)

According to Eurocode 0, it is an action that arises due to earthquake ground motions

1.1.3.5. Geotechnical action

it is one that is transmitted to the structure by the ground, fill or groundwater

1.2. Reinforced concrete

This is a species of concrete in which steel bars with desirable magnitude (diameter size) are introduced in the casting stage so as to resist the tensile stresses developed by the external loads (action). The reinforcement is usually, though not always the case, steel reinforcing bars (rebar) and is usually embedded passively in the concrete before the concrete sets. Modern reinforced concrete can contain varied reinforcing materials made of steel, polymers or alternate composite material in conjunction with rebar or not. Reinforced concrete may also be permanently stressed (in compression), so as to improve the behaviour of the final structure under working loads.

1.2.1. concrete

Literally, concrete is a multiphase composite construction material, composed primarily of aggregate (generally coarse gravel or crushed rocks such as limestone or granite, and fine aggregate such as sand), cement (Portland and its derivatives) and water.

The main components present in concrete are namely

- Cement (Portland and its derivate);
- Water;
- Inert (sand, gravel);
- Admixtures (accelerating, delaying, fluidifying, aerating, etc.);
- Other (fibers, polymers etc.).

1.2.1.1. Cement

The most diffused category of cement used is the Portland cement. It is a hydraulic material that shall consist in at least 2/3 by mass of calcium silicates, such as $3CaO \cdot SiO_2$ and $2CaO \cdot SiO_2$, and the rest is composed of aluminium- and iron- containing clinker phases and other compounds. Cement is used to create the matrix adhesive to all the components present in the mix. It gives also a certain aliquot of mechanical resistance of the final product.

1.2.1.2. Water

This is one of the most important constituents found in concrete since it takes into account for the *water/cement ratio* (E/C), which is the most important parameter that influences the final mechanical strength of the material, and its workability. The water/cement ratio is defined as the ratio between the weight of the total water (including that contained by the aggregates) and the cement. It usually varies between 0.4 and 0.65 and it can be lower in the case of admixtures use. Water allows the hydration of cement when they come in contact, creating the matrix, which connects all the compounds present. When the water quantity reduces, the concrete strength increases and the workability of the mixture decreases.

1.2.1.3. Aggregates (inert)

They are subdivided into 2 macro-classes namely the fine, which is principally constituted by sand, and the coarse, which is formed by all the grains that pass for at least 95% from a sieve with opening diameter bigger than 4 mm. They can be natural aggregates (sand/gravel) or they can derive from rock demolition. Some of their characteristics influence greatly the resulting concrete, not only for the final mechanic resistance, but also for the workability and durability. In particular shape, porosity and angularity are very important, even for the coarse and fine ones. From the chemical point of view, they must be without impurities, and they must not contain clay or lime. They have also to be graded, so that fine material can fill the free spaces, due to big gravel presence. The diameters vary between 0 and 30 to 35mm (the diameters depend on the dimensions of the structural elements). There are aggregate grading curves, which give a fuse to be used for Reinforced Concrete (RC) structures.

1.2.1.4. Admixtures

These are special constituents added during the concrete mixing so as to improve the properties and performance of fresh concrete. Concrete admixtures are used to enhance the properties of concrete so as to achieve desired workability in case of low water-cement ratio. They also enhance the setting time of concrete for long distance transportation of concrete. There are so many different types of admixtures used in concrete namely the accelerating admixtures, the retarding admixtures, the water-reducing admixtures (plasticizers), air-entraining admixtures, water-proofing, pumping and super-plasticizers just to name a few.

1.2.2. concrete's workability

Relating to workability, it was prevously noted that cement and water content are the most significant parameters that influence this characteristic. It is important to also note that the fresh paste of the material could be at the right level of fluidity to be casted in the formworks, but that the water content has not to exceed a fixed quantity, which signs the required final mechanical strength asked to the building.

Workability is measured by means of slump test, which is performed with a truncated cone with standard dimensions, filled with concrete in three layers, which one is pushed inside with 25 beats in a minute. The concrete drop (slump) is measured when the cone is raised. The concrete consistency is defined according to the slump. Figure 1.1 demonstrates the slump test.

In the situation whereby the slump reaches high values (> 22 cm), a *flow test* is performed instead of the slump test. In this case, a cone is placed in the middle of the flow table and filled with fresh concrete in two layers, each one 25 times tamped with a rod. When the cone is removed, the concrete is allowed to flow, and the diameter of the flow is measured. Figure 1.2 illustrates the flow test

Concrete classes, according to UNI EN-206, are classified and divided as shown on table 1.1

Figure 1.1. slump test (structure courses, 2015)

Figure 1.2. flow test (structure course, 2015)

Table 1.1. Concrete consistency levels according to UNI EN – 206.

Consistency level	Class	slump (mm)
Dry	S1	10-40
Plastic	S2	50-90
Quasi-fluid	S3	100-150
Fluid	S4	160-210
Super-fluid	S5	>220

1.2.3. Compressive strength of concrete

This is the most common factor used to measure concrete quality. It represents the most valued quality for designers and engineers, and it is universally recognized as the key concrete quality control tool, because it is very easy to measure and advantageous. Though, it may not always tell the whole story of concrete. We define the following

$$stress\ (\sigma) = \frac{Force\ (F)}{Area\ (A)} \qquad (1.1)$$

$$strain\ (\varepsilon) = \frac{Change\ in\ length\ (\Delta l)}{Unit\ length\ (l_0)} \qquad (1.2)$$

$$Elastic\ modulus\ (E) = \frac{stress\ (\sigma)}{strain\ (\varepsilon)} \qquad (1.3)$$

Compressive strength is the fundamental parameter, which allows identifying the concrete class of resistance, and it is obtained by dividing the ultimate load applied on the area of the surface normal to load direction. Compressive strength test consists in concrete specimen crashed by a hydraulic press which is a specimen is placed between two slabs, and then the upper is pull down slowly, imprinting a stress on the material.

The strength test results may be affected by variation in: type of test specimen, specimen size, type of mould, curing, preparation of the end surface, rigidity of the testing machine, and rate of application of stress. The standard specimens are tested generally at 28 days and there are two types of specimens used namely cubes (R_{ck} or $f_{ck,\ cube}$) and cylinders (f_{ck}). The cylindrical compressive characteristic resistance is obtained from the uniaxial compression of a cylindrical specimens of diameter 150 mm and height 300 mm. The compressive characteristic resistance of a cube is obtained from the uniaxial compression of a cubic specimens of dimension 150 mm. figure 1.3 illustrate the different specimens used in compressive strength evaluation

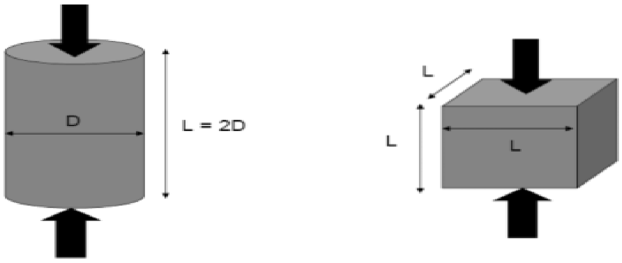


Figure 1.3. Cylinder and cubic specimens in compressive strength evaluation (structure course, 2015)

We consider three classes of concrete strength namely

- Normal strength classes (NR): fck ≤ 50 MPa;
- High performance concrete (HPC): 50 < fck ≤ 70 MPa;
- Ultra-high-performance concrete (UHP): 70 < fck ≤ 90 MPa.

$$f_{ck} = 0.83 \times f_{ck,cube} \qquad (1.4)$$

1.2.4. Tensile strength of concrete

The direct tensile test is difficult to be developed due to problems related to the application of the tensile forces to the ends of the specimen. However, there are other tests which could determine concrete's tensile strength. We have first and foremost the splitting test or the Brazilian test is currently used to determine the tensile strength of the concrete. It describes characteristics of procedures, press machines needed, dimension and shape of necessary specimens. Specimens are cylindrical, and the compression load is applied onto the lateral surface using press slabs. Between concrete surface and slab, a supplementary steel bar and generally a plywood plate is interposed, to distribute homogeneously the load.

Another way to evaluate tensile strength is by means of flexural test. Load configurations could be various, and the most typical are central point loading and third point loading. The first load configuration prescribes that the entire load is applied at the centre of the beam, obtaining higher moment acting on the beam. The maximum stress is reached only in the central section of the beam. The second load configuration prescribes that half the load is applied at each third of the span length, and the maximum stress is present over the centre third portion of the beam.

1.2.5. Reinforcing steel

Concrete is a material that is quite strong in compression, but relatively weak in tension. In order to compensate for this imbalance, a reinforcing steel is cast into it so as to carry the tensile loads. Reinforcing steel, also known as rebar (short for reinforcing bar), is a common steel bar used as a tensioning device in reinforced concrete and in reinforced masonry structures. It is usually formed from carbon steel, and it gives ridges for better mechanical anchoring into the concrete. Generally, there are two kinds of rebar namely smooth bars and ribbed bars.

There are several reasons that explain why steel is so well used in concrete applications. First, as we have seen, steel could imbalance concrete lack of tensile strength. In addition to that, Steel and concrete have similar thermal expansion coefficient therefore experiencing a minimum differential expansion caused by thermal changes, leading to a high interconnection between the two materials.

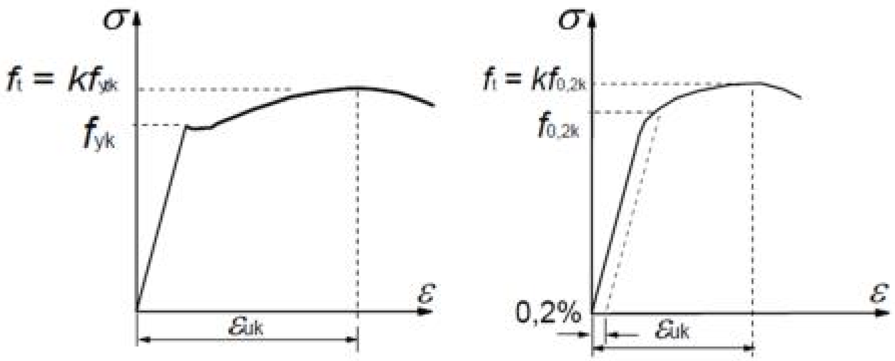
Figure 1.4. reinforcement steel (Structure courses, 2015)

EN 1992-1-1 rules are valid for ribbed bars, de-coiled rods, welded fabrics and lattice girders

1.2.5.1. Strength and ductility characteristics

The application rules for design and detailing in the Eurocode 2 (EN 1992) are valid for a specified yield strength range, f_{yk} = 400 to 600 MPa. The yield strength f_{yk} (or the 0,2% proof stress, $f_{0,2k}$) and the tensile strength f_{tk} are defined respectively as the characteristic value of the yield load, and the characteristic maximum load in direct axial tension, each divided by the nominal cross-sectional area.

The reinforcement shall have adequate ductility as defined by the ratio of tensile strength to the yield stress (f_{tk}/f_{yk}), and the elongation at maximum force ε_{uk}. Figure 1.5 shows stress-strain curves for typical hot rolled and cold worked steel, while Table 1.2 shows the ductility classes for reinforcing bars.

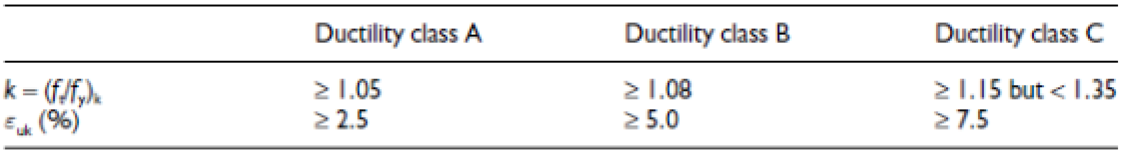


a) Hot rolled steel

b) Cold worked steel

Figure 1.5. Stress-strain diagrams of typical reinforcing steel (EN1992, 2004)

Table 1.2. ductility classes for reinforcing bars.



	Ductility class A	Ductility class B	Ductility class C
$k = (f_t/f_y)_k$	≥ 1.05	≥ 1.08	≥ 1.15 but < 1.35
ε_{uk} (%)	≥ 2.5	≥ 5.0	≥ 7.5

1.2.5.2. Design assumptions

Design should be based on the nominal cross-section area of the reinforcement and the design values derived from the characteristic values. For normal design, either of the following assumptions may be made (see figure 1.6)

- an inclined top branch with a strain limit of ε_{ud} and a maximum stress of k f_{yk}/γ_s at εuk, where k = f_{tk}/f_{yk};
- a horizontal top branch without the need to check the strain limit.

The recommended value of εud is 0,9 ε_{uk}. The mean value of density may be assumed to be 7850 kg/m^3 and the design value of the elasticity modulus Es may be taken as 200 GPa.

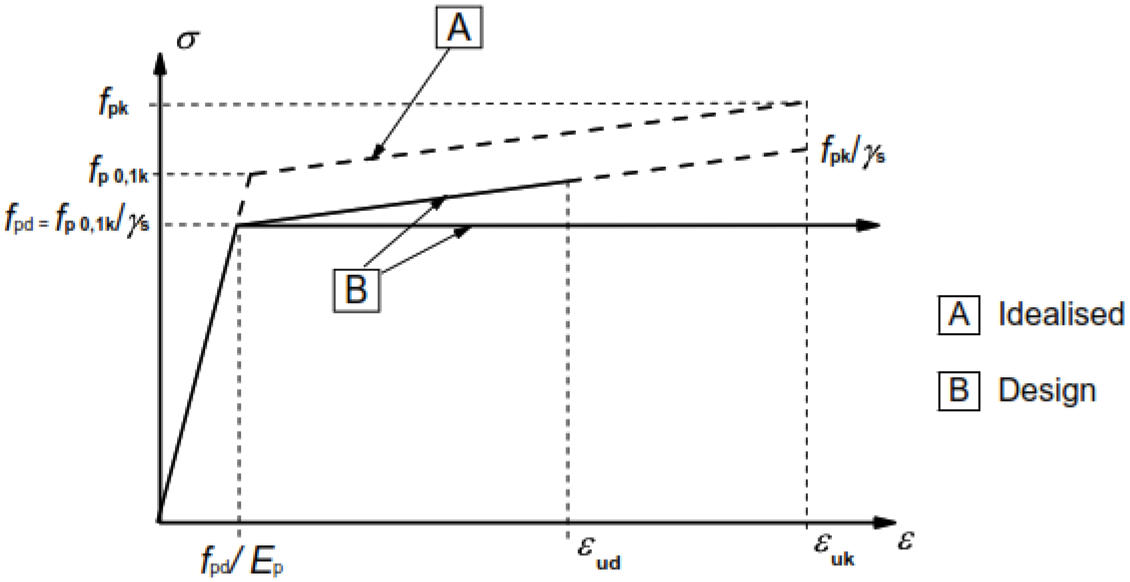
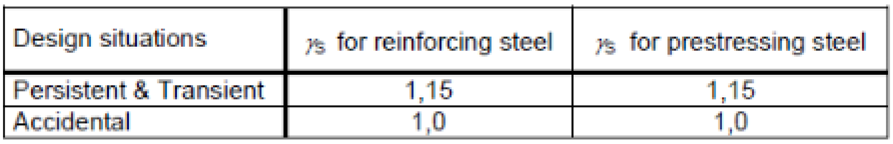


Figure 1.6. Idealised and design stress-strain diagrams for reinforcing steel (EN1992, 2004)

Table 1.3. Partial factors for steel (EN1992, 2004)



Design situations	γ_s for reinforcing steel	γ_s for prestressing steel
Persistent & Transient	1,15	1,15
Accidental	1,0	1,0

1.3. High rise building

There are so many references when talking about high rise building or multi-storey building. A high rise building based on the general opinion of the public could be defined as one having multiple floors above ground in the building. The tallness of a building is relative and cannot be defined in absolute terms either in relation to height or the number of stories. But, from a structural engineer's point of view the high rise building or multi-storey building can be defined as one that, by virtue of its height, is affected by lateral forces due to wind or earthquake or both to an extent that they play an important role in the structural design. “The German regulations define ‘tall or multi-storeyed buildings’ as buildings higher than 22m with rooms for the permanent accommodations of people” (Ross, 2004 in Kheir Al-Kodmany, 2012, pp.131-148). This limit is derived from the length of ladders used by firefighters. The Leicester City Council in the U.K. defines a tall building as a building over 20m in height; and/or a building of any height, which is substantially higher than the predominant height of the buildings in the surrounding area; and/or a building, which would make a significant impact on the skyline of the city.

The growth in modern multi-storied building construction, which began in late nineteenth century, is intended largely for commercial and residential purposes. The development of the high-rise building has followed the growth of the city closely. The process of urbanization that started with the age of industrialization is still in progress in developing countries. Industrialization causes migration of people to urban centers where job opportunities are significant. The land available for buildings to accommodate this migration is becoming scarce, resulting in rapid increase in the cost of land. Therefore, Multi-storey buildings aim to increase the floor area of the building without increasing the area of the land and saving money.

1.4. Structural frame members

Despite of their different load transfer mechanism, all the structural frame members have one mission which is transferring the load to the form top to underneath soil. The definition of the member mostly depends on how the members transfers the load and in what mechanism.

1.4.1. Beam

This is a component that is designed to support transverse loads, that is, loads acting perpendicular to the longitudinal axis of the beam. The beam supports the load by bending only. The bending force induced into the material of the beam as a result of the external loads, own weight, span and external reactions to these loads is called a bending moment.

When a beam is loaded perpendicular to its longitudinal axis, internal forces, shear and moment develop to transmit the applied loads into the supports. If the ends of a beam are restrained longitudinally by its supports, or if a beam is a component of a continuous frame, axial force may also develop. If the axial force is small which is the typical situation for most beams, then it can be neglected when the member is designed. In the case of reinforced concrete beams, small values of axial compression actually produce a modest increase (on the order of 5 to 10 percent) in the flexural strength of the member.

Beams generally carry vertical gravitational forces but can also be used to carry horizontal loads (i.e., loads due to an earthquake or wind). The loads carried by a beam are transferred to columns, walls, or girders, which then transfer the force to adjacent structural compression members. In light frame construction the joists rest on the beam.

This structural member of reinforced concrete is placed horizontally to carry loads over openings. Because both bending and shear in such beams induce tensile stresses, steel reinforcing tremendously increases beam strength. Usually, beams are designed under the assumption that tensile stresses have cracked the concrete and the steel reinforcing is carrying all the tension.

1.4.2. Column

They are the vertical members carrying mainly axial loads (interior columns) but sometimes they carry axial loads and moments in the case of exterior beams. In other words, they are the members that carry loads primarily in compression. Usually columns carry bending moments as well, about one or both axes of the cross-section, and the bending action may produce tensile forces over apart of cross-section. Even in each case, columns are generally referred to as compression members, because the compression forces dominate their behaviour.

There are three types of reinforced concrete compression members are in use. These are

- Members reinforced with longitudinal bars and lateral ties.
- Members reinforced with longitudinal bars and continuous spiral.
- Composite compression members reinforced longitudinally with structural steel shapes, pipe, or tubing, with or without additional longitudinal bars, and various types of lateral reinforcement.

1.4.3. Slabs

Floors slabs are the horizontal elements used to support permanent and live loads present at the generic level of a building, in addition to their own dead load. They transfer these loads to the other substructures (beams, walls, pillars) that support them.

The most general way to define a slab sub-system refers to the mechanical concept of thin plate subjected to transverse loads and to loads acting in the plane of the slab. According to this definition, a slab can be described as a two-dimensional structural system, variously restrained, able to serve as a plate, subjected to flexural and shear stresses, when equilibrating the external transverse loads, or when able to serve as a diaphragm, subjected to the plane stresses, when redistributing the horizontal loads acting on the building such as wind and seismic loads, among the various vertical members.

1.4.3.1. Different forms of slabs

There are a number of standard floor/roof structural systems used in cast-in-place reinforced concrete constructions namely

a. One-way ribbed (joist) slab

In this case, floor loads are transferred to the beams through the joists. It provides the depth required for stiffness and readily accommodates floor penetrations.

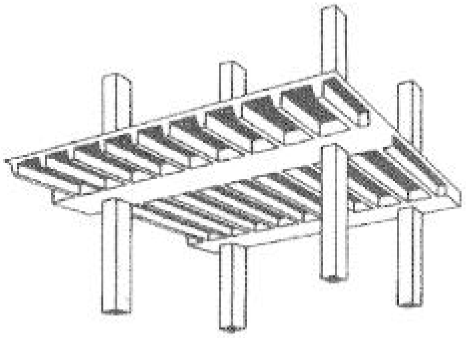


Figure 1.7. One-way ribbed slab (concrete centre, 2006)

b. Waffle slab (two-way joist slab)

There are joists in both directions, this floor system is the strongest and will have the least deflection. It's typically used when stiffness is important or if there are heavy loads.

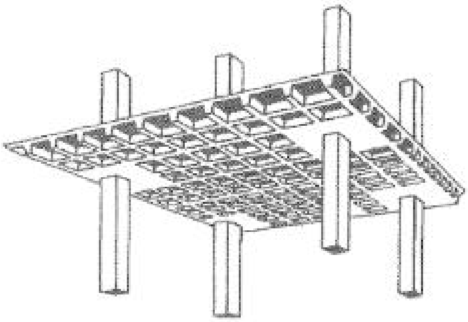


Figure 1.8. waffle slab (concrete centre, 2006)

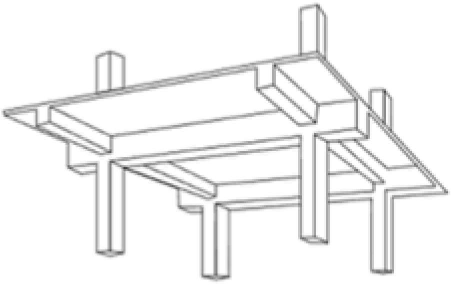


Figure 1.9. slab on beam (concrete centre, 2006)

c. Slab on beams

Monolithic RC slab, in which floor loads are transferred to the beams, which are then transferred to the columns thus making it ideal for heavy load areas. It is a common system for parking structures, elevator and stair areas.

d. Flat slab

Monolithic RC slab with no beams between the columns. Instead, the floor is heavily reinforced in both directions. In addition, there is reinforcing steel in the floor at the columns to transfer the loads (for punching effects).

Figure 1.10. Slab on beam (concrete centre, 2006)

e. Flat slab with drop panel

This system is not only economical but can use flying form systems. There is a drop panel to provide extra thickness around the columns to limit punching effect. Provides uniform clear space below slab as well as providing flexible layout of columns partitions.

Figure 1.11. Flat slab with drop panel (concrete centre, 2006)

f. voided slab

Concrete mass is removed from the areas in the slab where it is not needed to resist load and as a result of the reduced dead load, voided slabs can span further.

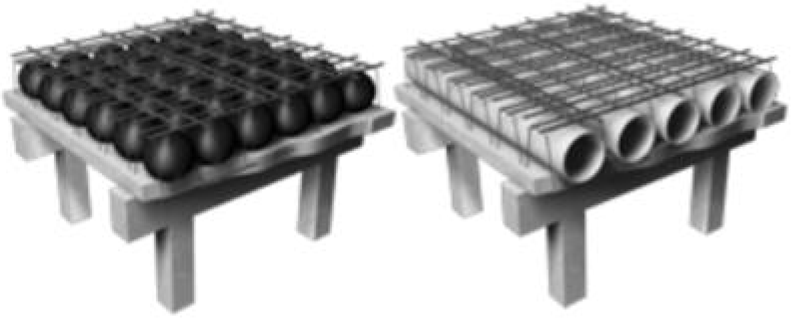


Figure 1.12. Voided slab; mono (left) or bi-directional (right)

1.4.4. Foundation

A foundation is a lower portion of building structure that transfers its gravity loads to the earth. Generally, we distinguish two categories of foundations namely the shallow foundation and the deep foundation.

1.4.4.1. shallow foundation

A shallow foundation is a foundation that transmit structural loads to the near-surface soils. As types, there is the Spread footing foundation and the Mat (raft) foundation.

a. Spread footing foundation

The spread footing foundation is an enlargement at the bottom of a column or bearing wall that spreads the applied structural loads over a sufficiently large soil area. One can distinguish

i. Square spread footings

Supporting a single centrally-supported column

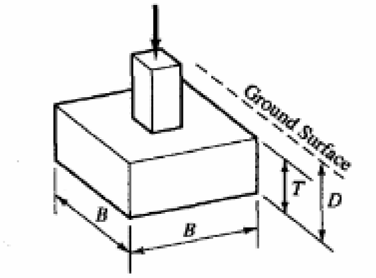


Figure 1.13. Square spread footings (SNU geotechnical engineering lab, 2018)

ii. Rectangular spread footings

In cases that obstructions prevent construction of a square footing with a sufficiently large base area and large moment loads are present.

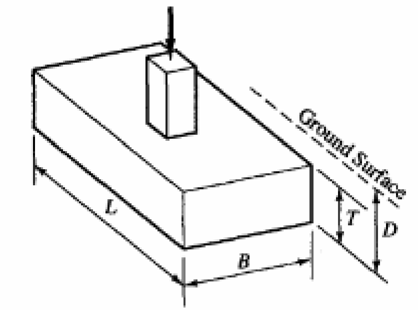


Figure 1.14. Rectangular spread footings (SNU geotechnical engineering lab, 2018)

iii. Circular spread footings

Supporting a single centrally-supported column, but less common than square footing. (flagpoles).

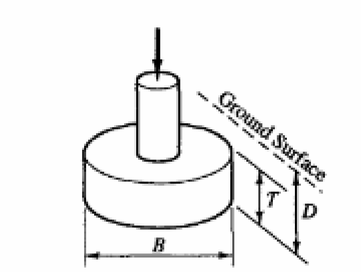


Figure 1.15. Circular spread footings (SNU geotechnical engineering lab, 2018)

iv. Continuous spread footings

Used to support bearing walls.

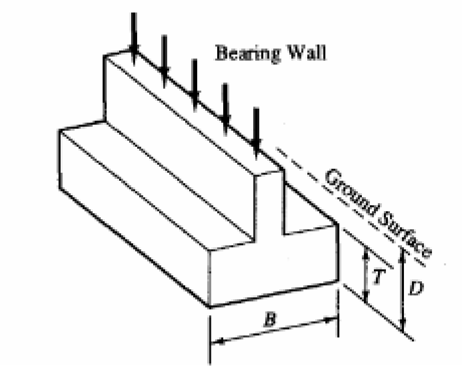


Figure 1.16. Continuous spread footings (SNU geotechnical engineering lab, 2018)

v. Combined footing

When columns are located too close together for each to give its own footing.

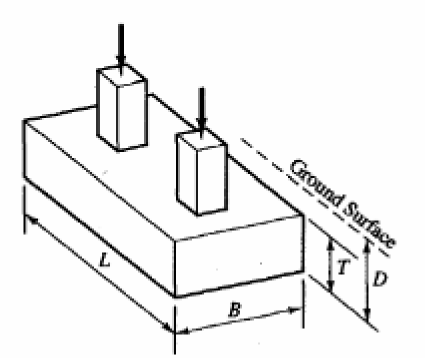


Figure 1.17. Combined footing (SNU geotechnical engineering lab, 2018)

vi. Strap footing with a grade beam

Provides the necessary moment resistance in the exterior footing with eccentric load and a more rigid foundation system.

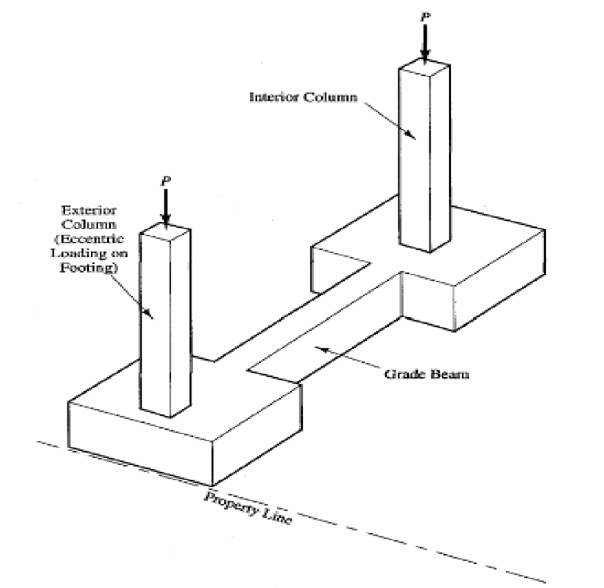


Figure 1.18. Strap footing (SNU geotechnical engineering lab, 2018)

b. Raft foundation

A raft foundation is a very large spread footing that usually encompasses the entire footprint of the structure. In a few words, it can also be defined as a large slab supporting the structure as a whole. As advantage over the spread footing foundation, it spreads the structure load over a larger area, thus reduces bearing pressure. Also, it provides much more structural rigidity hence reduces the potential for excessive differential settlements. Raft foundation has an easier water proofing mechanism compared to the spread footing foundation, it also has a greater weight and thus is able to resist greater uplift pressure.

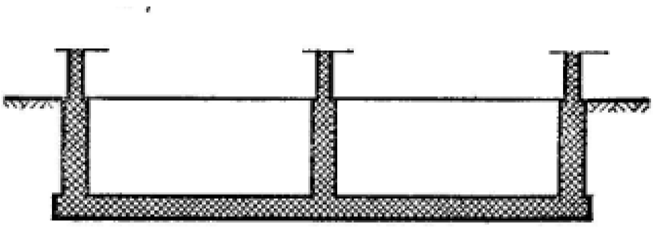


Figure 1.19. Raft foundation (SNU geotechnical engineering lab, 2018)

1.4.4.2. Deep foundation

A deep foundation is a type of foundation whereby the embedment is larger than its maximum plane dimension. This type of foundation is designed to be supported on deeper geologic materials because either the soil or rock near the ground is not competent enough to take the design loads or it is more economical to do so.

The advantages that deep foundation has over shallow foundation is cumbersome. To begin with, taking into account the deeper geologic materials, a deep foundation occupies a relatively smaller area of the ground surface. Secondly, deep foundations can usually take larger loads than shallow foundations that occupy the same area of the ground surface. Furthermore, deep foundations can reach deeper competent layers of bearing soil or rock, whereas shallow foundations cannot. Deep foundations can also take large uplift and lateral loads, whereas shallow foundations usually cannot. The term deep foundation includes piles, piers, or caissons.

a. Pile

A pile is a slender structural element installed in the ground to transfer significant structural loads to the deep ground. These load is transferred by the base resistance below the pile tip and along the lateral surface of the pile.

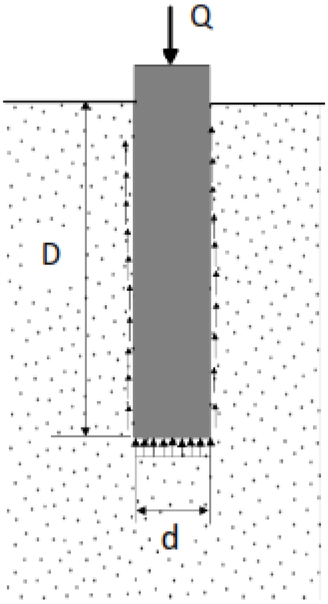


Figure 1.20. Pile foundation

1.5. Limit state design

A limit state is one beyond which a structure can no longer fulfil its design criteria that is the structure will become unfit for use. The aim of limit state design is to achieve acceptable probabilities that a structure will not become unfit for its intended use during its design life, that is, the structure will not reach a limit state. Limit state design of an engineering structure must ensure that under the worst loadings the structure is safe, and during normal working conditions the deformation of the members does not detract from the appearance, durability or performance of the structure. Three basic methods using factors of safety to achieve safe, workable structures have been developed over many years; they include the following

- The permissible stress method in which ultimate strengths of the materials are divided by a factor of safety to provide design stresses which are usually within the elastic range.
- The load factor method in which the working loads are multiplied by a factor of safety.

- The limit state method which multiplies the working loads by partial factors of safety and also divides the materials' ultimate strengths by further partial factors of safety.

The permissible stress method has proved to be a simple and useful method but it does have some serious inconsistencies and is generally no longer in use. Because it is based on an elastic stress distribution, it is not really applicable to a semi-plastic material such as concrete, nor is it suitable when the deformations are not proportional to the load, as in slender columns.

In the load factor method the ultimate strength of the materials should be used in the calculations. As this method does not apply factors of safety to the material stresses, it cannot directly take account of the variability of the materials, and also it cannot be used to calculate the deflections or cracking at working loads.

The limit state method of design overcomes many of the disadvantages of the previous two methods. This is done by applying partial factors of safety, both to the loads and to the material strengths, and the magnitude of the factors may be varied so that they may be used either with the plastic conditions in the ultimate state or with the more elastic stress range at working loads.

As earlier mentioned, the objective of this design is to ensure that the structure does not attain its limit state. With this in mind, there are principally 2 limit state design namely the ultimate limit state (ULS) design and the serviceability limit state (SLS) design.

1.5.1. Ultimate limit state

It is that limit state associated to the failure (collapse) of the structure. In other words, it is that limit state for which the structure must be able to withstand, with an adequate factor of safety against collapse, the loads for which it is designed to ensure the safety of the building occupants and/or the safety of the structure itself. The ultimate limit states enable the designer to calculate the strength of the structure

1.5.2. Serviceability limit state

This is the design to ensure a structure that the structure is comfortable and usable. In other word, this is the state beyond which requirements for the correct exercise and use of the structure are not more satisfied. These states include

1.5.2.1. Deflection

This check is to verify that the appearance or efficiency of any part of the structure must not be adversely affected by deflections nor should the comfort of the building users be adversely affected.

1.5.2.2. Cracking

This is to ensure that local damage due to cracking and spalling must not affect the appearance, efficiency or durability of the structure.

1.5.2.3. Durability

This must be considered in terms of the proposed life of the structure and its conditions of exposure.

Other serviceability limit state designs include excessive vibration, fatigue and fire resistance.

1.6. Solicitations

The solicitations are based on the combination of action. These combinations are hence dependent on the limit state verification.

1.6.1. ULS combination

1.6.1.1. Fundamental combination

$$\gamma_G\, G_{max}\text{“} + \text{”}\, \gamma_G\, G_{min}\text{“} + \text{”}\gamma_P\ P\text{“} + \text{”}\gamma_{Q1}\, Q_{k1}\text{“} + \gamma_{Q2}\, \Psi_{02}\, Q_{k2}\text{“} + \text{”}\gamma_{Q3}\, \Psi_{03}\, Q_{k3}\text{“} + \text{”} \quad (1.5)$$

1.6.2. SLS combination

These include

1.6.2.1. Characteristic combination (rare)

$$G_{max}\text{“} + \text{”}\ \ G_{min}\text{“} + \text{”}\ P\text{“} + \text{”}Q_{k1}\text{“} + \text{”}\Psi_{02}\, Q_{k2}\text{“} + \text{”}\Psi_{03}\, Q_{k3}\text{“} + \text{”}\ \ldots \quad (1.6)$$

1.6.2.2. Frequent combination

$$G_{max}\text{“} + \text{”}\ \ G_{min}\text{“} + \text{”}\ P\text{“} + \text{”}\Psi_{11}\, Q_{k1}\text{“} + \text{”}\, \Psi_{22}\, Q_{k2}\text{“} + \text{”}\Psi_{23}\, Q_{k3}\text{“} + \text{”}\ \ldots \quad (1.7)$$

1.6.2.3. Quasi permanent combination

This combination is generally used for long-term effects

$$G_{max}\text{“} + \text{”}\ \ G_{min}\text{“} + \text{”}\ P\text{“} + \text{”}\Psi_{21}\, Q_{k1}\text{“} + \text{”}\Psi_{22}\, Q_{k2}\text{“} + \text{”}\Psi_{23}\ \ Q_{k3}\text{“} + \text{”}\ \ldots \quad (1.8)$$

1.6.2.4. Seismic combination

Used for the ultimate and serviceability limit states related to the seismic action E

$$E\text{``} + \text{''}\,G_{max}\text{``} + \text{''}\ G_{min}\text{``} + \text{''}\ P\text{``} + \text{''}\Psi_{21}\,Q_{k1}\text{``} + \text{''}\,\Psi_{22}\,Q_{k2}\text{``} + \ldots \quad (1.9)$$

1.6.2.5. Accidental combination

Used for the ultimate limit states related to the design accidental actions A

$$A_d\text{``} + \text{''}\,G_{max}\text{``} + \text{''}\ G_{min}\text{``} + \text{''}\ P\text{``} + \text{''}\Psi_{21}\,Q_{k1}\text{``} + \text{''}\,\Psi_{22}\,Q_{k2}\text{``} + \ldots \qquad (1.10)$$

1.7. Code for designing

Codes used in the design and verification of the structural elements are principally:

- Eurocode 0: Basis of structural design
- Eurocode 1: Actions on structures Part 1-1: General Actions Densities, self-weight, imposed loads for buildings. (EC1-1)
- Eurocode 2: Design of concrete structures Part 1-1: general rules and rules for buildings. (EC2-1)
- Eurocode 7: Geotechnical design.

Conclusion

The objective of the chapter was to bring about the main knowledge about structural engineering and structural analysis. Also, discussion on the different forms of action existing on the structure. Then, further studies were made based on reinforced concrete and its constituents. A brief detailing was done on the different members of the structural frame. A study was also made on different limit state design and their application in the use of solicitations and on the standard code of practices for the design of the building. With these knowledges, it is then important for one to know how a structural analysis shall be made on the structure.

CHAPTER 2. PRESENTATION OF PROJECT AND METHODOLOGY

Introduction

This chapter presents the detailed procedure in which the structure is analysed and designed using the case study of this work. The presentation of the project as well shall be done briefly in form of the location, plans and the geometrical characteristics of the building. There exist so many types of structural analysis but for the interest of this work it will be necessary to know the following; analysis with respect to serviceability limit state and analysis with respect to the ultimate limit state. The analysis with respect to the ultimate limit state could be as well subdivided into analysis with limited redistribution (the moments in the most solicitated sections that is, at the supports are multiplied by a reduction coefficient and the moments at the other sections are consequently multiplied to ensure equilibrium) and without redistribution, plastic analysis (used only at the ULS) and non-linear analysis (used both at ULS and SLS admitting a non-linear behaviour adapted for the materials). In this work, the analysis with respect to ULS without redistribution shall be used. For the linear elastic analysis, the calculation of elements at both serviceability limit states and ultimate limit states can be performed using a linear analysis based on elasticity theory. The determination of the different elements at ultimate limit state can be carried out according to a linear analysis based on the theory of elasticity. Linear analysis can be used to determine the stresses, under the following assumptions namely un-cracked section, stress - linear strain relationships and mean values of elastic modulus.

2.1. Presentation of project

The area of studies considered for this portion of work is located in the centre region of Cameroon in Yaoundé situated specifically at Elig-Effa. The building is classified as a category B so reserved for office use. The surface area occupied by the building is 859.29 square metres. The following figures fig 2.1 and fig 2.2 represent the elevation and the floor plans respectively

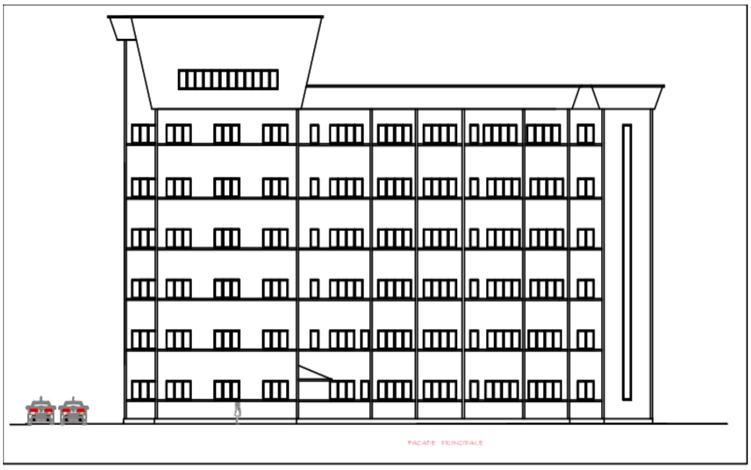


Figure 2.1. Elevation plan

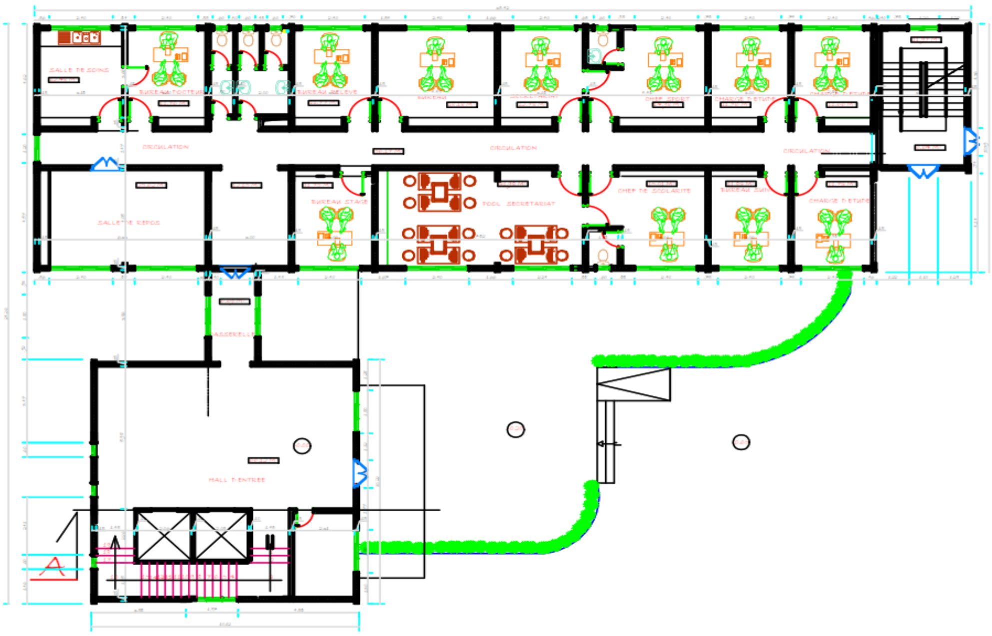


Figure 2.2. Floor plan

2.2. Geometric characteristics and parameters consideration for the building

2.2.1. Geometric characteristics for the building

The building has a dimension of length 35.42 m, width 24.26 m and height of 3 m each level. It is a 7-storey building with one elevator and stair cases each. It is an irregular building comprised of 3 different sections which are linked by structural joints. It is comprised of offices and conference halls meant for meetings and conferences.

2.2.2. Parameters considered for the building

The parameters taken into account for the structural analysis of the building are summarized below.

For the earthquake zone it is noticed that according to the Eurocode 8 the building is constructed in an area with negligible seismic activity (Zone I) hence the seismic analysis will not be studied in this work. The building class is of group B with the office building height of 21 m. the admissible bearing capacity of the soil shall be considered as = 0.25 N/mm^2. The Eurocode 7 is intended to be applied to the geotechnical aspects of the design of buildings and civil engineering works. EN 1997 is concerned with the requirements for strength, stability, serviceability and durability of structures. It is also intended to be used in conjunction with EN 1990-E-2002, which establishes the principles and requirements for safety and serviceability, describes the basis of design and verification and gives guidelines for related aspects of structural reliability.

2.3. Actions for analysis

As earlier mentioned in the previous chapter, an action is a force, a displacement or an acceleration that acts on a structure's component. The actions are divided mainly into two parts namely the permanent actions and the variable actions. Permanent actions are those which are normally constant during the structure's life. Variable actions, on the other hand, are transient and not constant in magnitude, as for example those due to wind or to human occupants (Mosley, 2007).

2.3.1. Permanent actions

These include the weight of the structure itself and all architectural components such as exterior cladding, partitions and ceilings. Equipment and static machinery, when permanent fixtures, are also often considered as part of the permanent action. The preliminary design calculations are generally required to estimate the probable sizes and self-weights of the structural concrete elements. According to Bill Mosley, Permanent actions are generally calculated on a slightly conservative basis, so that a member will not need redesigning because of a small change in its dimensions. Overestimation, however, should be done with care, since the permanent action can often actually reduce some of the forces in parts of the structure

2.3.2. Variable actions

They are more difficult to determine accurately. For most of them, it is only possible to make conservative estimates based on standard codes of practice or past experience. Examples include the weights of its occupants, furniture, or machinery; the pressures of wind, the weight of snow, and of retained earth or water; and the forces caused by thermal expansion or shrinkage of the concrete.

2.4. Load combinations for the analysis

Permanent and variable actions will occur in different combinations, all of which must be taken into account in determining the most critical design situation for any structure. For example, the self-weight of the structure may be considered in combination with the weight of furnishings and people, with or without the effect of wind acting on the building (which may also act in more than one direction). In cases where actions are to be combined it is recommended that, in determining suitable design values, each characteristic action is not only multiplied by the partial factors of safety but also by a further factor given the symbol Ψ (see annex).

As earlier seen in the first chapter, there are three combinations of actions to be applied in this case study namely the fundamental combination for the verification of the building under ULS, the characteristic combination and the quasi-permanent combination for the verification of the structure under SLS. The combination of effects of actions to be considered should be based on the design value of the leading variable action and on the design combination values of accompanying variable actions.

2.4.1. Fundamental combination

$$\gamma_G\, G_{max}\text{“} + \text{”}\gamma_G\, G_{min}\text{“} + \text{”}\gamma_P\ P\text{“} + \text{”}\gamma_{Q1}\, Q_{k1}\text{“} + \gamma_{Q2}\,\Psi_{02}\, Q_{k2}\text{“} + \text{”}\gamma_{Q3}\,\Psi_{03}\, Q_{k3}\text{“} + \text{”}(2.1)$$

2.4.2. Characteristic combination

$$G_{max}\text{“} + \text{”}\ G_{min}\text{“} + \text{”}\ P\text{“} + \text{”}Q_{k1}\text{“} + \text{”}\Psi_{02}\, Q_{k2}\text{“} + \text{”}\Psi_{03}\, Q_{k3}\text{“} + \text{”} \ldots \quad (2.2)$$

2.4.3. Quasi permanent combination

$$G_{max}\text{“} + \text{”}\ G_{min}\text{“} + \text{”}\ P\text{“} + \text{”}\Psi_{21}\, Q_{k1}\text{“} + \text{”}\Psi_{22}\, Q_{k2}\text{“} + \text{”}\Psi_{23}\ Q_{k3}\text{“} + \text{”} \ldots \quad (2.3)$$

2.5. Analysis of the structure

It is important to know the bending moment, torsional moment, axial forces and shear forces when designing a structure. In most cases, the elastic analysis is used to determine the distribution of these forces within the structure; but because to some extent reinforced concrete is a plastic material, a limited redistribution of the elastic moments is sometimes allowed. A plastic yield-line theory may be used to calculate the moments in concrete slabs. The properties of the materials, such as the elastic modulus, which are used in the structural analysis should be those associated with their characteristic strengths.

A structure should be analysed for each of the critical loading conditions which produce the maximum stresses at any particular section. For these structures it is conventional to draw the bending-moment diagram on the tension side of the members.

In this study, the beams are all continuous beams and hence application of the software SAP2000 will be required in the determination of the bending moment, shear forces and axial forces applied on the structure.

2.6. Modelling of concrete elements using SAP2000

SAP2000 is a general-purpose civil engineering software ideal for the analysis and design of any type of structural system. Basic and advanced systems, ranging from 2D to 3D, of simple geometry to complex, may be modelled, analysed, designed and optimized using a practical intuitive object-based modelling environment that simplifies and streamlines the engineering process. It is also a powerful design to design structures following AASHTO specifications, ACI and AISC building codes. These features, and many more make SAP2000 the state-of-the-art in structural analysis program.

The SAP2000 graphic user interface is used to model, analyse, design, and display the structure geometry, properties and analysis results. The analysis procedure can be divided into three parts namely the pre-processing, solving and post processing.

2.6.1. Pre-processing

In pre-processing, the following information is needed by SAP2000.

- Choosing the units for this project.
- Setting up geometry.
- Defining material and member section properties.
- Assigning member section properties and element releases.
- Defining load cases, Maximum output station.
- Assigning load magnitudes.
- Assigning restraints.

2.6.2. Solving.

In this part SAP2000 will assemble and solve the global matrix. The following steps are needed:

- Select model
- Click ‘do not run’ for modal analysis from the analysis menu
- From the analysis menu, select Run now.

2.6.3. Post processing.

The main options in post processing are:

- Displaying the deformed shape.
- Displaying the member forces.
- Joints, this gives joint reaction.
- Show table.
- Element output and select the load combination to obtain results.
- File, export to excel.
- Designing the structural members and checking the safety of a design.
- Modifying the structure.

2.7. Design of concrete elements

The procedure in designing the concrete elements is as shown in the following sections:

2.7.1. Materials

In the design and verification of the reinforced concrete elements the mechanical properties considered for concrete and steel are reported in the following paragraphs.

2.7.1.1. Concrete

The following properties shall be used in this study for the analysis of the building:

In the case of common building sites, for CPA- based concretes, EMC I 42.5 or CPJ- CEM II/A 42.5, dosed at 350 kg/m^3, it is possible to take the following:

Grade	C25/30
Characteristic cylinder strength (f_{ck})	25 MPa
Characteristic cube strength (R_{ck})	30 MPa
Mean compressive strength ($f_{cm}= f_{ck}+8$)	33 MPa
Mean tensile strength ($f_{ctm}=0.3f_{ck}{}^{(2/3)} \leq$ C50/60)	2.6 MPa
Secant modulus of elasticity ($E_{cm}= 22(f_{cm}/10)^{0,3}$)	31.5 GPa

For the verification for ULS, we have the following;

partial factor for ULS	$\gamma_c= 1.5$
design compressive strength ($f_{cd}=\alpha_{cc}\cdot f_{ck}/\gamma_c$)	25/1.5 = 16.67 MPa

where $\alpha_{cc} = 1$ is the coefficient taking account of long-term effects on the compressive strength and of the unfavourable effects resulting from the way the load is applied.

For the design of cross-sections, the stress-strain relationship illustrated in figure 2.3 and figure 2.4 may be used.

- For C25/30, $\varepsilon_{c1} = 2.1$ ‰
- If the concrete grade is < C50/60 then; $\varepsilon_{c2} = 2.0$ ‰, $\varepsilon_{c3}= 1.75$ ‰ and $\varepsilon_{cu} = 3.5$ ‰.

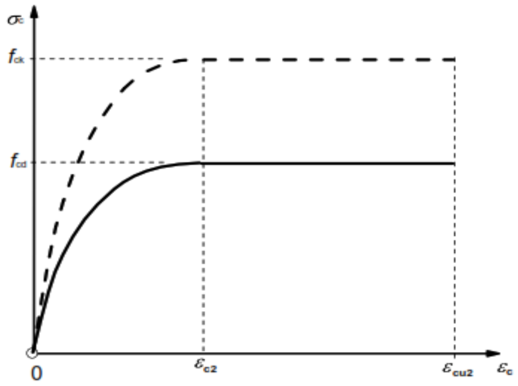


Figure 2.3. Parabola-rectangle diagram for concrete under compression (EN 1992 1-1, 2004)

In the verification for SLS,

- partial factor for S.L.S $\gamma_c = 1.5$
- design compressive strength ($f_{cd} = \alpha_{cc} \cdot f_{ck}/\gamma_c$) 25/1.5 = 16.67 MPa

2.7.1.2. Reinforcing steel

The requirement needed for the reinforcing steel in the design of this structure is detailed in the following points:

- Ribbed bars type: B450C

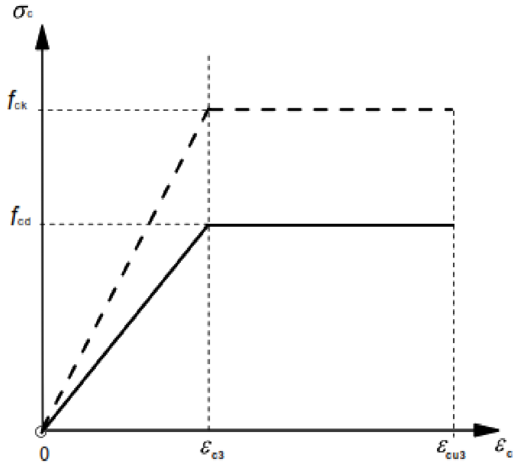


Figure 2.4. Bi-linear stress-strain relation (EN 1992 1-1, 2004)

- Yield strength (f_{yk}): 450 MPa
- Tensile strength (f_{tk}): 540 MPa
- Characteristic strain at max force (ε_{uk} (%)) ≥ 5.0%
- Modulus of elasticity (Es): 200 GPa

For the verification for ULS:

- partial factor for ULS $\gamma_s = 1.15$
- design yield strength ($f_{yd}=f_{yk}/\gamma_s$) 450/1.15 = 391 MPa
- deformation at design yield strength ($\varepsilon_{yd} = f_{yd}$ / Es) 1.96 ‰
- elastic moduli ratio n_0 = Es/Ec = 6.35

for the verification of SLS:

- partial factor for S.L.S $\gamma s = 1.15$
- design yield strength ($f_{yd}=f_{yk}/\gamma_s$) 450/1.15 = 391 MPa

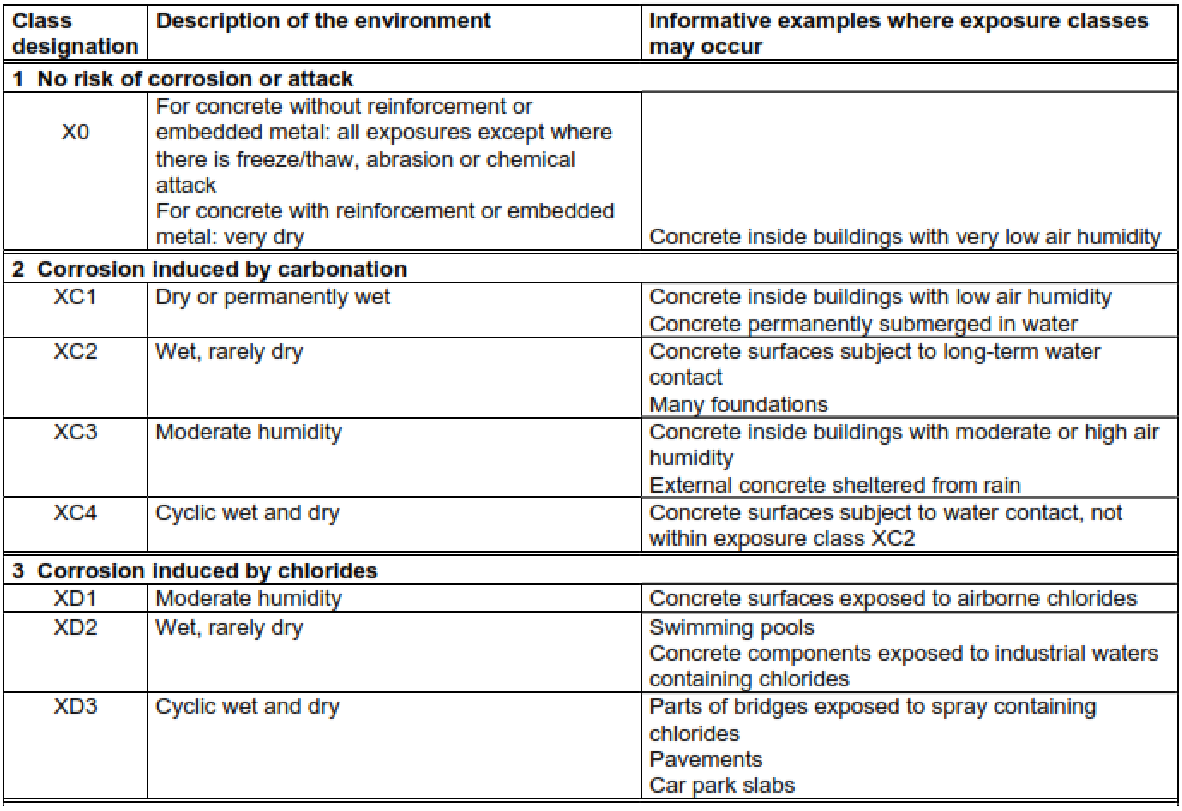
2.7.2. Durability and cover to reinforcement

A durable structure shall meet the requirements of serviceability, strength and stability throughout its design working life, without significant loss of utility or excessive unforeseen maintenance. The required protection of the structure shall be established by considering its intended use, design working life, maintenance program and actions. The possible significance of direct and indirect actions, environmental conditions and consequential effects shall be considered. In particular, corrosion protection of steel reinforcement depends on thickness of concrete cover (as well as cracking).

2.7.2.1. Environmental condition

The environmental conditions should be taken into account in order to design the building. The exposure classes are classified on three main categories of increasing aggressively attack. Based on Eurocode 2, our building is in an ordinary environmental condition, Table 2.1 below gives the exposure classes related to environmental conditions.

Table 2.1. Exposure classes related to environmental conditions (EN 1992 1-1, 2004)



Class designation	**Description of the environment**	**Informative examples where exposure classes may occur**
1 No risk of corrosion or attack		
X0	For concrete without reinforcement or embedded metal: all exposures except where there is freeze/thaw, abrasion or chemical attack For concrete with reinforcement or embedded metal: very dry	Concrete inside buildings with very low air humidity
2 Corrosion induced by carbonation		
XC1	Dry or permanently wet	Concrete inside buildings with low air humidity Concrete permanently submerged in water
XC2	Wet, rarely dry	Concrete surfaces subject to long-term water contact Many foundations
XC3	Moderate humidity	Concrete inside buildings with moderate or high air humidity External concrete sheltered from rain
XC4	Cyclic wet and dry	Concrete surfaces subject to water contact, not within exposure class XC2
3 Corrosion induced by chlorides		
XD1	Moderate humidity	Concrete surfaces exposed to airborne chlorides
XD2	Wet, rarely dry	Swimming pools Concrete components exposed to industrial waters containing chlorides
XD3	Cyclic wet and dry	Parts of bridges exposed to spray containing chlorides Pavements Car park slabs

2.7.2.2. Concrete cover

This is the distance between the surface of the reinforcement closest to the nearest concrete surface (which includes links and stirrups and surface reinforcement where relevant) and the nearest concrete surface.

According to Eurocode 2, the nominal cover shall be specified on the drawings. It is defined as a minimum cover, c_{min}, plus an allowance in design for deviation, Δc_{dev}.

$$c_{nom} = c_{min} + \Delta c_{dev} \tag{2.4}$$

2.7.2.3. Minimum cover

Eurocode 2 stipulates that the minimum concrete cover, c_{min}, has to be provided in order to ensure first and foremost the safe transmission of bond forces. In addition to that, it ensures the protection of the steel against corrosion (durability) and also ensures an adequate fire resistance.

The greater value for c_{min} satisfying the requirements for both bond and environmental conditions shall be used.

$$c_{min} = max\{c_{min,b};\ c_{min,dur} + \Delta c_{dur,\gamma} - \Delta c_{dur,st} - \Delta c_{dur,add};\ 10\ mm\} \qquad (2.5)$$

where:

$c_{min,\ b}$ is the minimum cover due to bond requirement,

$c_{min,\ dur}$ is the minimum cover due to environmental conditions,

$\Delta c_{dur,\ \gamma}$ is the additive safety element,

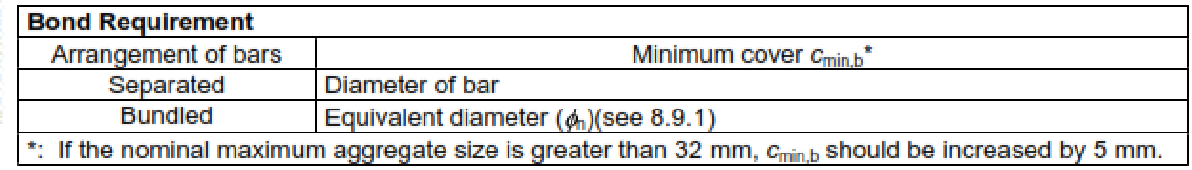
$\Delta c_{dur,\ st}$ is the reduction of minimum cover for use of stainless steel,

$\Delta c_{dur,\ add}$ is the reduction of minimum cover for use of additional protection.

i. Minimum cover due to bond requirement, $c_{min,\ b}$.

For the transmission of bond forces to be safe and in order to ensure adequate compaction of the concrete, the minimum cover should not be less than $c_{min,\ b}$ given in Table 2.2

Table 2.2. Minimum cover, $c_{min,b}$, requirements with regard to bond

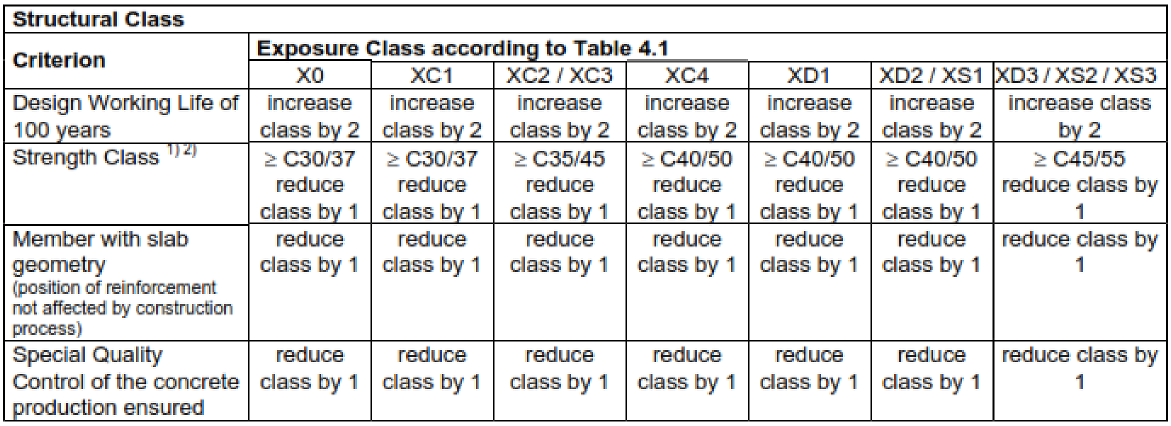


Bond Requirement	
Arrangement of bars	Minimum cover $c_{min,b}$*
Separated	Diameter of bar
Bundled	Equivalent diameter (ϕ_n)(see 8.9.1)
*: If the nominal maximum aggregate size is greater than 32 mm, $c_{min,b}$ should be increased by 5 mm.	

ii. Minimum cover due to environmental conditions, $c_{min,\ dur}$.

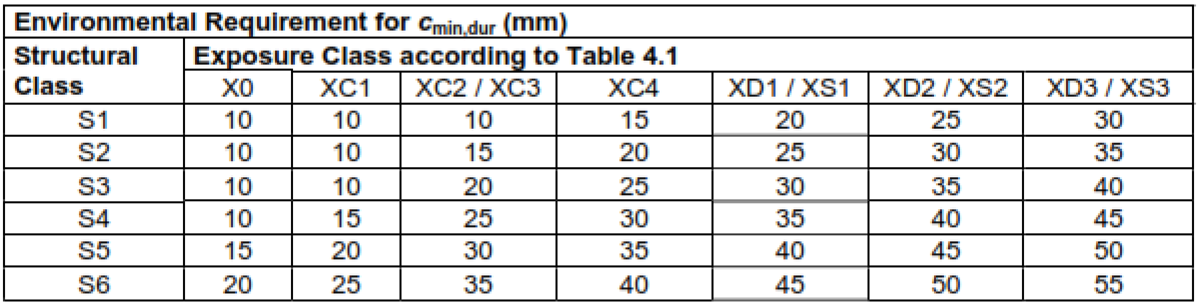
The minimum cover values for reinforcement and prestressing tendons in normal weight concrete taking account of the exposure classes and the structural classes is given by $c_{min,\ dur}$. It is to be noted that the structural classification and values of $c_{min,\ dur}$ for use in a country may be found in its National Annex. The recommended structural class (design working life of 50 years) is S4 and the recommended modifications to the structural class is given in Table 2.3. The recommended minimum structural class is S1. The recommended values of $c_{min,\ dur}$ are given in Table 2.4 for reinforcing steel.

Table 2.3. Recommended structural classification



Structural Class							
Criterion	**Exposure Class according to Table 4.1**						
	X0	XC1	XC2 / XC3	XC4	XD1	XD2 / XS1	XD3 / XS2 / XS3
Design Working Life of 100 years	increase class by 2	increase class by 2	increase class by 2	increase class by 2	increase class by 2	increase class by 2	increase class by 2
Strength Class $^{1) 2)}$	≥ C30/37 reduce class by 1	≥ C30/37 reduce class by 1	≥ C35/45 reduce class by 1	≥ C40/50 reduce class by 1	≥ C40/50 reduce class by 1	≥ C40/50 reduce class by 1	≥ C45/55 reduce class by 1
Member with slab geometry (position of reinforcement not affected by construction process)	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1
Special Quality Control of the concrete production ensured	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1

Table 2.4. Values of minimum cover, $c_{min,dur}$, requirements with regard to durability for reinforcement steel



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

iii. Additive safety element, $\Delta c_{dur,\gamma}$.

The concrete cover should be increased by the additive safety element $\Delta c_{dur,\gamma}$. It is to be noted that the recommended value is 0 mm.

iv. Reduction of minimum cover for use of stainless steel, $\Delta c_{dur,st}$

Where stainless steel is used or where other special measures have been taken, the minimum cover may be reduced by $\Delta c_{dur,st}$. For such situations the effects on all relevant material properties should be considered, including bond. The recommended value, without further specification, is 0 mm.

v. Reduction of minimum cover for use of additional protection, $\Delta c_{dur, add}$

For concrete with additional protection (e.g. coating) the minimum cover may be reduced by $\Delta c_{dur, add}$. The recommended value, without further specification, is 0 mm.

2.7.2.4. Allowance in design for deviation, Δc_{dev}

According to Eurocode 2, in order to calculate the nominal cover, c_{nom}, an addition to the minimum cover shall be made in design to allow for the deviation (Δc_{dev}). The required minimum cover shall be increased by the absolute value of the accepted negative deviation. The recommended value is 10 mm.

2.7.3. Design procedure of structural elements

2.7.3.1. Reinforced concrete beam design

Reinforced concrete beam design consists primarily of producing member details which will adequately resist the ultimate bending moments, shear forces and torsional moments. At the same time serviceability requirements must be considered to ensure that the member will behave satisfactorily under working loads. It is difficult to separate these two criteria, hence the design procedure consists of a series of interrelated steps and checks. These steps could be condensed into three basic design stages namely

- preliminary analysis and member sizing;
- detailed analysis and design of reinforcement;
- serviceability calculations.

Using SAP2000, the bending moment solicitations is obtained from the different load combinations.

a. preliminary analysis and member sizing

The preliminary analysis need only provide the maximum moments and shears in order to ascertain reasonable dimensions. Beam dimensions required are

- cover to the reinforcement
- breadth (b)
- effective depth (d)

- overall depth (h)

Suitable dimensions for b and d (see Figure 2.5) can be decided by a few trial calculations as follows:

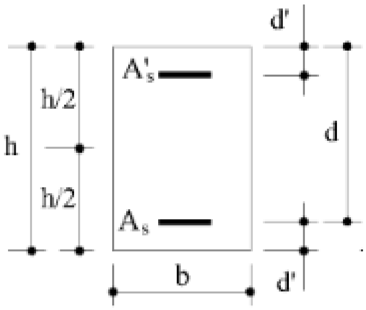


Figure 2.5. Example of transversal beam section with longitudinal reinforcement (Bill M, 2007)

With compression reinforcement it can be shown that

$$\frac{M_{ED}}{bd^2 f_{ck}} \leq \frac{8}{f_{ck}} \qquad (2.6)$$

approximately, if the area of bending reinforcement is not to be excessive.

For no compression reinforcement, it can be shown that

$$K = \frac{M_{ED}}{bd^2 f_{ck}} \leq K_{bal} \qquad (2.7)$$

Where $K_{bal} = 0.167$ for $f_{ck} \leq C50$.

For the maximum design shear force,

$$V_{Ed,max} \leq V_{Rd,max} = 0{,}18 b_w d(1 - f_{ck}/250) f_{ck} \qquad (2.8)$$

To avoid congested shear reinforcement, $V_{Ed,\,max}$ should preferably be somewhat closer to half (or less) of the maximum allowed.

b. detailed analysis and design of reinforcement

i. Design for bending without moment redistribution

The calculation required for the main bending reinforcement is done using the series of equations below.

The resisting bending moment, M_{RD}, is:

$$M_{RD} = A_{s,r}\, f_{yd}\,(d - \beta_2\, x_r) \qquad (2.9)$$

Where $\beta_2 = 0.42$ and $\beta_1 = 0.81$.

Equation shows the formula to determine the neutral axis (see Figure 2.6)

$$x = \frac{d}{2\beta_2} - \sqrt{\left(\frac{d}{2\beta_2}\right)^2 - \frac{M_{ED}}{\beta_1\beta_2 b f_{cd}}} \qquad (2.10)$$

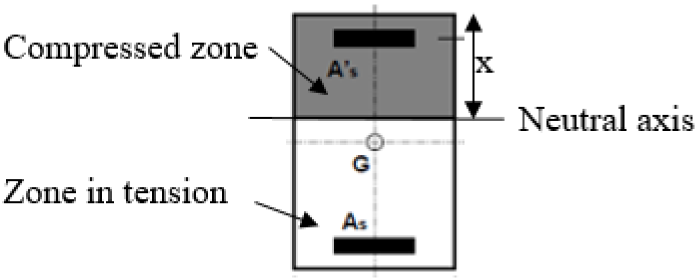


Figure 2.6. Illustration of the neutral axis position in a section (Wandji,2019)

The steel reinforcement formula is calculated as:

$$A_s = \frac{M_{ED}}{0.9\, d\, f_{yd}} \qquad (2.11)$$

1. Additional reinforcement

If $M_{ED} < M_{RD}$, then the additional reinforcement is not needed. Otherwise, the new additional steel reinforcement is given by:

$$A_s' = \frac{M_{ED} - M_{RD}}{f_{yd}(d - d')} \qquad (2.12)$$

The minimum area for longitudinal tension reinforcement is given by:

$$A_{s,min} = \max\left(0.0013bh;\ 0.26\frac{f_{ctm}}{f_{yk}}bd\right) \qquad (2.13)$$

The maximum area for longitudinal tension or compression reinforcement is:

$$A_{s,max} = 0.04bh \qquad (2.14)$$

2. Number of steel bars

The number of steel bars, n_b, is given by:

$$n_b = \frac{A_s}{\pi D^2/_4} + 1 \qquad (2.15)$$

The space between the reinforcements is determined using equation

$$s = \frac{b - 2d'}{n_b - 1} \qquad (2.16)$$

with the minimum space of

$$s_{b,min} = max\left(k_1 d_b\,; k_2 + d_{ag}; 20\ mm\right) \qquad (2.17)$$

Where $k_1 = 1.0, k_2 = 5.0\ mm,$ d_b is the maximum bar size and d_{ag} is the maximum aggregate size

3. Real neutral axis

The real neutral axis, x_r, is given by:

$$x_r = \frac{A_{s,r} f_{yd}}{\beta_1 b f_{cd}} \qquad (2.18)$$

ii. Shear design

When shear forces are small, the concrete section on its own may have sufficient shear capacity (V_{Rdc}) to resist the ultimate shear force (V_{ED}) resulting from the worst combination of actions on the structure, although in most cases a minimum amount of shear reinforcement will usually be provided.

In those sections where $V_{ED} \leq V_{Rdc}$, then no calculated shear reinforcement is required. The shear capacity of the concrete in such situations is given by an empirical expression:

$$V_{Rdc} = \left[C_{Rd,c} k (100\, \rho_l\, f_{ck})^{1/3} + k_1 \sigma_{cp}\right] b_w d \qquad (2.19)$$

with a minimum value of

$$V_{Rdc,min} = \left[v_{min} + k_1 \sigma_{cp}\right] b_w d \qquad (2.20)$$

where:

$C_{Rd,\,c} = 0.18/\gamma_c$, γ_c *assumed* 1.5

$k = 1 + (200/d)^{1/2} \leq 2.0$, d in mm

$\rho_l = A_{sl}/(b_w d) \leq 0.02$

$k_1 = 0.15$

$\sigma_{cp} = N_{ED}/A_c < 0.2\ f_{cd}$

$$v_{min} = 0.035 k^{3/2} f_{ck}^{1/2} \qquad (2.21)$$

1. **Inclined shear reinforcement**

According to Eurocode 2, the angle between the cracks (that is the angle of the concrete strut) and the longitudinal axis of the beam can be determined according to the following limit:

$$21.8° \leq \theta \leq 45°$$

The shear resistance for the steel tie strength is given by:

$$V_{RD} = \frac{A_{sw}}{s} z f_{ywd} (\cot\theta + \cot\alpha) \sin\alpha \qquad (2.22)$$

Where $z = 0.9d$

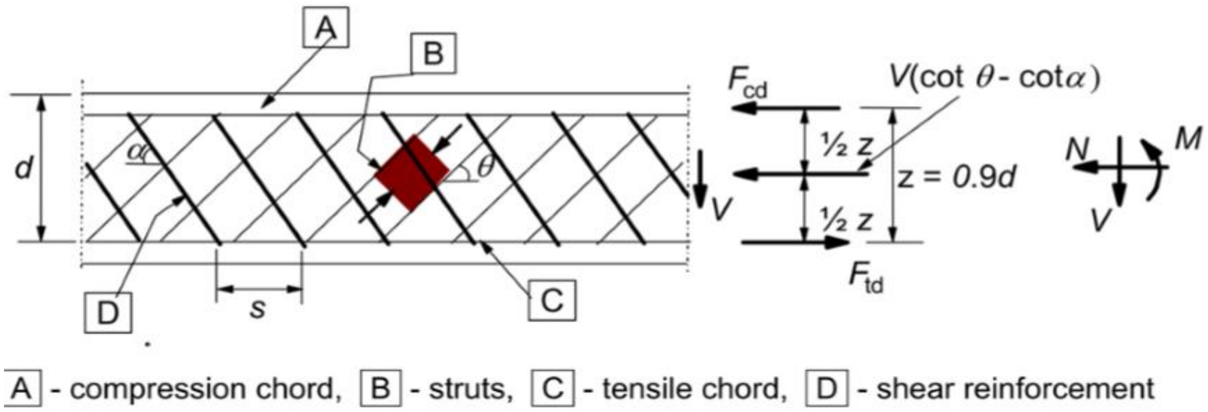


Figure 2.7. Truss model (EN 1992 1-1,2004)

2. Vertical shear reinforcement

All shear will be resisted by provision of links with no direct contribution from the shear capacity of the concrete itself. The amount and spacing of the shear links are determined using equation and will depend on the value of θ used in the design.

$$\frac{A_{sw}}{s} = \frac{V_{ED}}{z f_{ywd} \cot \theta} \qquad (2.23)$$

The ratio for shear reinforcement, ρ_w, is given by:

$$\rho_w = \frac{A_{sw}}{bs} \qquad (2.24)$$

with a minimum value of

$$\rho_{w,min} = 0.08 \frac{\sqrt{f_{ck}}}{f_{yk}} \qquad (2.25)$$

c. Serviceability calculation (SLS design)

The objective of this calculation is to verify that the structure is comfortable and useable, by showing that the maximum stress in steel and concrete in the beam is respectively less than the stress in steel and concrete provided by a formula given according to Eurocode 2. In addition to the stress limitation verification, it will be necessary to verify the cracking as well as the deflection of the beam.

i. Stress limitation verification

The maximum stresses in both the concrete and reinforcement are computed and it must be verified that these stresses are less than the maximum permitted values reported in EC2. The conditions to be satisfied in this case are

$\sigma_c < k_1 f_{ck}$ *and* $\sigma_s < k_3 f_{yk}$ under characteristic combination and as well $\sigma_c < k_2 f_{ck}$ under quasi-permanent combination.

Where: σ_c is the compressive stress in the concrete, σ_s is the tensile stress in the reinforcement, k_1 is a coefficient = 0.6, k_2 is a coefficient = 0.45 and k_3 is a coefficient = 0.8. The assessment of stresses at the SLS can be performed considering linear elastic materials, neglecting concrete tensile strength. The modulus of elasticity of a concrete is controlled by the moduli of elasticity of its components. For loads with a duration causing creep, the total deformation including creep may be calculated by using an effective modulus of elasticity for concrete which considers the creep coefficient φ relevant for the load and time interval. When the assessment can be performed neglecting time factor, the coefficient $n = E_s/E_c$ can be assumed equal to 15. The neutral axis position is determined using the formula

$$y = \frac{n.A_s}{b}\left[-1 + \sqrt{1 + \frac{2.b.d}{n.A_s}}\right] \qquad (2.26)$$

The moment of inertia is calculated by the expression given in equation below

$$J = \frac{by^3}{3} + n.A_s.(d-y)^2 \qquad (2.27)$$

The compressive strength in concrete and the tensile stress in steel are both determined using the expressions given below

$$\sigma_c = \frac{M}{J}y \qquad (2.28)$$

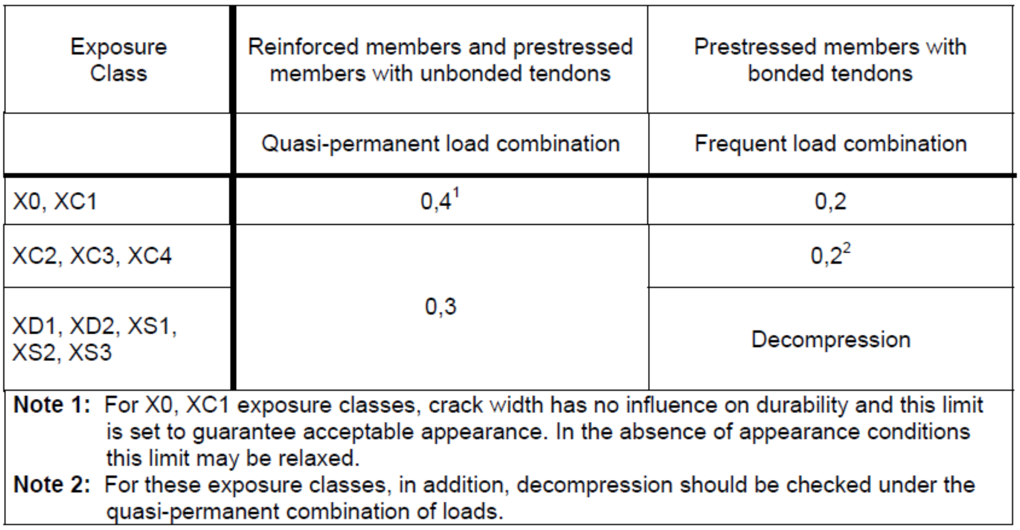
$$\sigma_s = \frac{n.M.(d-y)}{J} \qquad (2.29)$$

ii. Crack control

Cracking shall be limited to an extent that will not impair the proper functioning or durability of the structure or cause its appearance to be unacceptable. Cracking is normal in reinforced concrete structures subject to bending, shear, torsion or tension resulting from either direct loading or restraint or imposed deformations. Cracks may be permitted to form without any attempt to control their width, provided they do not impair the functioning of the structure. In this case as well, it is possible to conduct the verification with two methods: direct and indirect calculation. But for the interest of this work it is best probable to use the indirect method only.

A limiting calculated crack width, w_{max}, taking into account the proposed function and nature of the structure and the costs of limiting cracking, should be established. The recommended values for relevant exposure classes are given in table 2.5.

Table 2.5. Recommended values of w_{max} (mm) (EN 1992 1-1, 2004)



Exposure Class	Reinforced members and prestressed members with unbonded tendons	Prestressed members with bonded tendons
	Quasi-permanent load combination	Frequent load combination
X0, XC1	0,4[1]	0,2
XC2, XC3, XC4	0,3	0,2[2]
XD1, XD2, XS1, XS2, XS3		Decompression

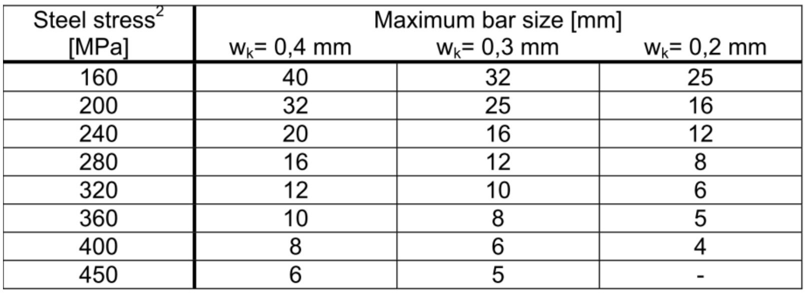
Note 1: For X0, XC1 exposure classes, crack width has no influence on durability and this limit is set to guarantee acceptable appearance. In the absence of appearance conditions this limit may be relaxed.

Note 2: For these exposure classes, in addition, decompression should be checked under the quasi-permanent combination of loads.

1. Indirect calculation method of crack verification

The rules for the calculation of crack widths may be presented in a tabular form by restricting the bar diameter or spacing as a simplification (tables 2.6 and 2.7 respectively)

Table 2.7. Maximum bar diameters for crack control



Steel stress[2] [MPa]	Maximum bar size [mm] w_k= 0,4 mm	w_k= 0,3 mm	w_k= 0,2 mm
160	40	32	25
200	32	25	16
240	20	16	12
280	16	12	8
320	12	10	6
360	10	8	5
400	8	6	4
450	6	5	-

Table 2.6. Maximum bar spacing for crack control

Steel stress[2] [MPa]	Maximum bar spacing [mm] w_k=0,4 mm	w_k=0,3 mm	w_k=0,2 mm
160	300	300	200
200	300	250	150
240	250	200	100
280	200	150	50
320	150	100	-
360	100	50	-

the determination of the steel stress under cracking conditions for both long term and short term is given by the expression

$$\sigma_s = \frac{M}{A_s\left(d - \frac{y}{3}\right)} \qquad (2.30)$$

2. Crack width calculation

In case of a direct calculation of the opening of the cracks, the crack width, w_k, may be calculated from Equation 2. below

$$w_k = \frac{\sigma_s}{E_s}\left[1 - \frac{\sigma_{s,cr}}{\sigma_s}\right]\left[k_3 c + k_1 k_2 k_4 \frac{\Phi}{\rho_s}\lambda\right] \qquad (2.31)$$

Where

iii. Deflection control

The deformation of a member or structure shall not be such that it adversely affects its proper functioning or appearance.

Appropriate limiting values of deflection taking into account the nature of the structure, of the finishes, partitions and fixings and upon the function of the structure should be established. Deformations should not exceed those that can be accommodated by other connected elements such as partitions, glazing, cladding, services or finishes. In some cases, limitation may be required to ensure the proper functioning of machinery or apparatus supported by the structure, or to avoid ponding on flat roofs. Care should be taken to ensure that the limits are appropriate for the particular structure considered and that that there are no special requirements.

The appearance and general utility of the structure could be impaired when the calculated sag of a beam, slab or cantilever subjected to quasi-permanent loads exceeds span/250. Deflections that could damage adjacent parts of the structure should be limited. For the deflection after construction, span/500 is normally an appropriate limit for quasi-permanent loads. Other limits may be considered, depending on the sensitivity of adjacent parts.

The limit state of deformation may be checked by either by limiting the span/depth ratio or by comparing a calculated deflection with a limit value.

1. Case where calculations may be omitted

Generally, it is not necessary to calculate the deflections explicitly as simple rules, for example limits to span/depth ratio may be formulated, which will be adequate for avoiding deflection problems in normal circumstances. More rigorous checks are necessary for members which lie outside such limits, or where deflection limits other than those implicit in simplified methods are appropriate.

The limiting span/depth ratio may be estimated using Equations (2.32) and (2.33) and multiplying this by correction factors to allow for the type of reinforcement used and other variables.

$$\frac{l}{d} = K\left[11 + 1.5\sqrt{f_{ck}}\frac{\rho_0}{\rho} + 3.2\sqrt{f_{ck}}\left(\frac{\rho_0}{\rho} - 1\right)^{3/2}\right] \qquad if\ \rho \leq \rho_0 \qquad (2.32)$$

$$\frac{l}{d} = K\left[11 + 1.5\sqrt{f_{ck}}\frac{\rho_0}{\rho - \rho'} + \frac{1}{12}\sqrt{f_{ck}}\sqrt{\frac{\rho'}{\rho_0}}\right] \qquad if\ \rho > \rho_0 \qquad (2.33)$$

Where

- $\frac{l}{d}$ is the limit span depth;
- K is the factor to take into account for the different structural systems (see table 2.8);
- ρ_0 is the reference reinforcement ratio;

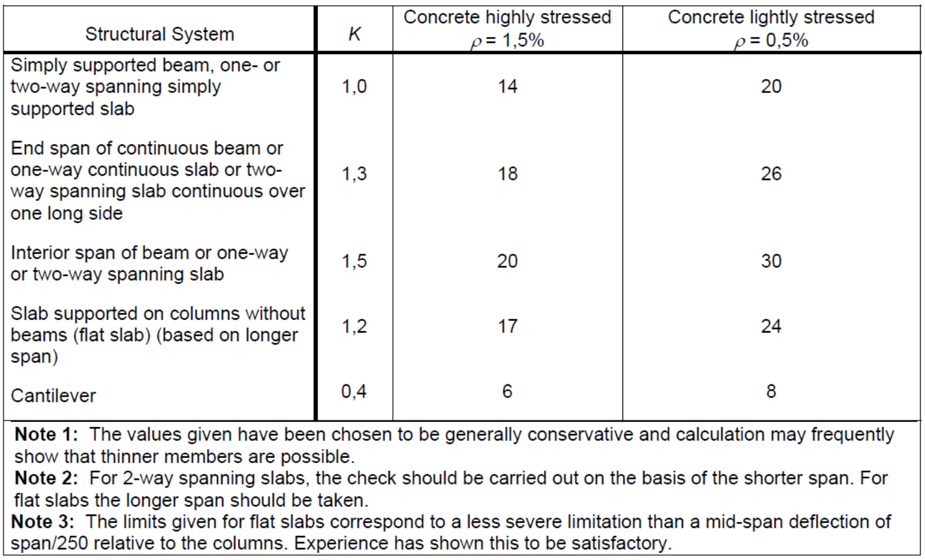
$$\rho_0 = \sqrt{f_{ck}}.10^{-3} \qquad (2.34)$$

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

$$\rho = \frac{A_s}{A_c} \qquad (2.35)$$

- 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.

Table 2.8. Basic ratios of span/effective depth for reinforced concrete members without axial compression



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.

According to Eurocode 2, equations (2.37) and (2.38) have been derived on the assumption that the steel stress, under the appropriate design load at SLS at a cracked section at the mid-span of a beam or slab or at the support of a cantilever, is 310 MPa, (corresponding roughly to f_{yk} =

500 MPa). Where other stress levels are used, the values obtained using Equations (2.37) and (2.38) should be multiplied by 310/σ_s. It will normally be conservative to assume that:

$$\frac{310}{\sigma_s} = \frac{500}{f_{yk}A_{s,req} / A_{s,prov}} \qquad (2.36)$$

where:

- σ_s is the tensile steel stress at mid-span (at support for cantilevers) under the design load at SLS
- $A_{s,\,prov}$ is the area of steel provided at this section
- $A_{s,\,req}$ is the area of steel required at this section for ultimate limit state

2.7.3.2. Column design

Columns assessment is performed considering maximum reactions from RC beams, and the self-weight of columns. In case of only axially loaded elements, reinforcement bars parallel to the axis should have a diameter of not less than $Ø_{min}$. The recommended value is 8 mm.

The total amount of longitudinal reinforcement should not be less than $A_{s,\,min.}$ The recommended value is given by the following expression

$$A_{s,min} = \frac{0.10N_{Ed}}{f_{yd}} \ or \ A_{s,min} = 0.002A_c \qquad (2.37)$$

Whichever of them is greater. The area of longitudinal reinforcement should not exceed $A_{s,\,max.}$ The recommended value is

$$A_{s,max} = 0.04A_c \qquad (2.38)$$

outside lap locations unless it can be shown that the integrity of concrete is not affected, and that the full strength is achieved at ULS. This limit should be increased to 0.08 Ac at laps. For columns having a polygonal cross-section, at least one bar should be placed at each corner.

For the determination of the minimum section of the column, the formula below is given

$$A_c = \frac{N_{Ed}}{0.6 f_{cd}} \qquad (2.39)$$

For cross-sections with symmetrical reinforcement loaded by the compression force it is necessary to assume the minimum eccentricity, $e_0 = h/30$ but not less than 20 mm where h is the depth of the section.

a. M-N interaction diagram

Column assessment can be performed calculating moment-axial load interaction diagram (also known as the M-N interaction diagram). It is a diagram that shows all the limit situation that can determine the failure of our section. The points which are lying onto the diagram represent the limit configuration: beyond them, failure occurs. The internal points represent the safe combinations of solicitations that the section could be subjected without determining failure. Tension is assumed negative, whereas compression is considered positive. Figure 2.5 shows the different point of analysis of the M-N interaction to be studied.

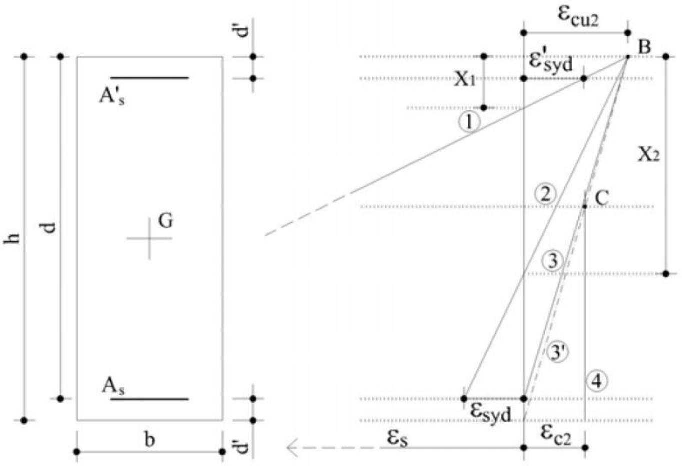


Figure 2.8. Possible strain distribution at ULS

1. First point

The section is completely subjected to tension; hence concrete is not reacting (ε_c = 0%). Therefore, it is calculated as thus:

$$\sigma' = \sigma_s = f_{yd} \qquad (2.40)$$

$$N_{Rd} = -A_s f_{yd} - A'_s f_{yd} \qquad (2.41)$$

$$M_{Rd} = A_s f_{yd}\left(\frac{h}{2} - d'\right) - A'_s f_{yd}\left(\frac{h}{2} - d'\right) \qquad (2.42)$$

2. Second point

The section is completely subjected to tension; hence concrete is not reacting. It is of important in this point to verify if the upper steel is yielded or not, that is $\varepsilon'_s > \varepsilon_{yd}$ if yielded. If the upper steel reinforcement is yielded, $\sigma'_s = f_{yd}$, if not $\sigma'_s = \varepsilon'_s E_s$ and $\sigma_s = f_{yd}$.

$$N_{Rd} = -A_s f_{yd} - A'_s f_{yd} \qquad (2.43)$$

$$M_{Rd} = A_s f_{yd}\left(\frac{h}{2} - d'\right) - A'_s f_{yd}\left(\frac{h}{2} - d'\right) \qquad (2.44)$$

3. Third point

In this case, the failure is due to concrete ($\varepsilon_c = 0.35\%$), and the lower reinforcement is yielded: therefore, it is taken as a hypothesis that there is a balanced failure. This means that

$$\varepsilon_s \geq \varepsilon_{yd}$$

It is important to verify if the upper steel (which is compressed, because it is placed upper than the neutral axis position) is yielded or not

$$\varepsilon'_s = \left[(d - d')\left(\frac{\varepsilon_s + \varepsilon_c}{d}\right)\right] - \varepsilon_s \qquad (2.45)$$

The position of the neutral axis is determined by the formula

$$x = \frac{\varepsilon_c . d'}{\varepsilon_c - \varepsilon_s} \qquad (2.46)$$

$$N_{Rd} = -A_s f_{yd} + A'_s f_{yd} + \beta_1 . b . x . f_{cd} \qquad (2.47)$$

$$M_{Rd} = A_s f_{yd}\left(\frac{h}{2} - d'\right) + A'_s f_{yd}\left(\frac{h}{2} - d'\right) + \beta_1 . b . x . f_{cd}\left(\frac{h}{2} - \beta_2 . x\right) \qquad (2.48)$$

4. Fourth point

Here, the failure is due to concrete ($\varepsilon_c = 0.35\%$), and the lower reinforcement reaches exactly ε_{yd}. As hypothesis, there is a balanced failure. Therefore,

$$\varepsilon_s = \varepsilon_{yd} \qquad (2.49)$$

$$\varepsilon_s' = \left[(d - d')\left(\frac{\varepsilon_s + \varepsilon_c}{d}\right)\right] - \varepsilon_{yd} \qquad (2.50)$$

The neutral axis position is:

$$x = \frac{\varepsilon_c . d'}{\varepsilon_c - \varepsilon_s} \qquad (2.51)$$

$$N_{Rd} = -A_s f_{yd} + A_s' f_{yd} + \beta_1 . b . x . f_{cd} \qquad (2.52)$$

$$M_{Rd} = A_s f_{yd}\left(\frac{h}{2} - d'\right) + A_s' f_{yd}\left(\frac{h}{2} - d'\right) + \beta_1 . b . x . f_{cd}\left(\frac{h}{2} - \beta_2 . x\right) \qquad (2.53)$$

5. Fifth point

The failure in this case is due to concrete (ε_c = 0.35%), and the lower reinforcement reaches exactly ε_{yd} = ε_s = 0%. Hence, it means that the neutral axis is just equal to the effective depth of the section:

$$x = d \qquad (2.54)$$

$$N_{Rd} = -A_s f_{yd} + A_s' f_{yd} + \beta_1 . b . x . f_{cd} \qquad (2.55)$$

$$M_{Rd} = A_s f_{yd}\left(\frac{h}{2} - d'\right) + A_s' f_{yd}\left(\frac{h}{2} - d'\right) + \beta_1 . b . x . f_{cd}\left(\frac{h}{2} - \beta_2 . x\right) \qquad (2.56)$$

6. Sixth point

The section now is uniformly compressed, hence:

$$\varepsilon_s = \varepsilon_c = 0.2\% \qquad (2.57)$$

$$N_{Rd} = b . h . f_{cd} + A_s f_{yd} + A_s' f_{yd} \qquad (2.58)$$

$$M_{Rd} = A_s f_{yd}\left(\frac{h}{2} - d'\right) - A_s' f_{yd}\left(\frac{h}{2} - d'\right) \qquad (2.59)$$

b. Column transverse reinforcement

The diameter of the transverse reinforcement (links, loops or helical spiral reinforcement) should not be less than 6 mm or one quarter of the maximum diameter of the longitudinal bars, whichever is the greater. The diameter of the wires of welded mesh fabric for transverse reinforcement should not be less than 5 mm.

The transverse reinforcement should be anchored adequately. The design of transverse reinforcement for a column is similar to the shear reinforcement of a beam.

$$\sigma_{cp} = \frac{N_{Ed}}{A_c} < 0.2 f_{cd} \qquad (2.60)$$

The spacing of the transverse reinforcement along the column should not exceed $s_{cl,tmax}$. The recommended value is:

$$s_{cl,tmax} = min\,(0.6\ \times 20\ \times diameter\ of\ longitudinal\ bar\ ;\ 0.6\ \times b\ ;\ 0.6\ \times 400\ ) \quad (2.61)$$

The web reinforcement against stirrup spacing is given by:

$$\frac{A_{sw}}{s} = 2\,\frac{area\ of\ stirrup}{stirrup\ spacing} \qquad (2.62)$$

c. Slenderness and effective length of columns

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

$$\lambda_{lim} = \frac{20.A.B.C}{\sqrt{n}} \qquad (2.63)$$

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

$B = \sqrt{1+2\omega}$, ($\omega = {A_s f_{yd}}/{A_c f_{yd}}$ is the mechanical reinforcement ratio, if ω is not known, B = 1.1 may be used);

$C = 1.7 - r_m$, ($r_m = {M_{01}}/{M_{02}}$ is the moment ratio; if r_m is not known C =0,7 may be used);

$n = \frac{N_{Ed}}{A_c f_{cd}}$ being the relative normal force.

The slenderness of a column or wall is given by

$$\lambda = \frac{l_0}{i} \qquad (2.64)$$

With $i = \sqrt{\frac{I}{A}}$ being the radius of gyration for the uncracked concrete section where I is the moment of inertia and A is the area of the section.

l_0 is the effective length of the element ($l_0 = 0.7.\,l$).

2.7.3.3. Foundation design

Foundations are composed by spread footing (isolated). The method of design chosen for the foundation is the axially loaded pad base design method.

a. Design of isolated pad bases

Isolated pad bases are square or rectangular slabs provided under individual columns. They spread the concentrated column load safely to the ground and maybe axially or eccentrically loaded. Mass concrete can be used for lighter foundations if the underside of the base lies inside a dispersal angle of 45°. Otherwise, a reinforced concrete pad is required. There is little specific guidance given in Eurocode 2 for the design of footings. The following procedure is normally used.

- When the base is axially loaded the load may be assumed to be uniformly distributed. The actual pressure distribution depends on the soil type; refer to soil mechanics textbooks.
- When the base is eccentrically loaded, the reactions may be assumed to vary linearly across the base.

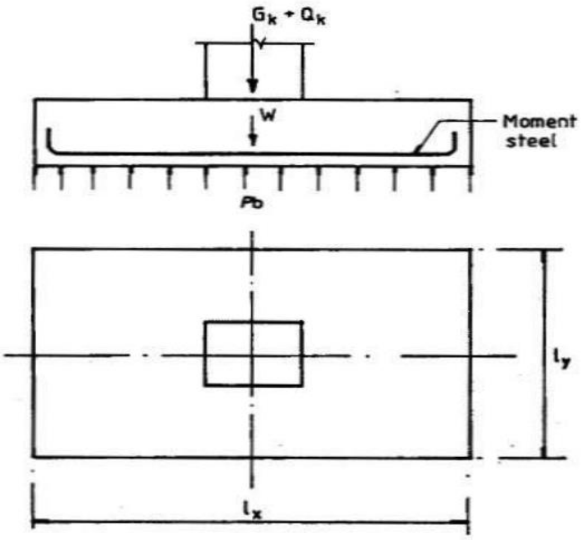


Figure 2.9. Reinforced Concrete Pad Foundation

i. Axially Loaded Pad Bases

Refer to the axially loaded pad footing shown above where the following symbols are used:

G_k = characteristic dead load from the column (kN);

Q_k = characteristic imposed load from the column (kN);

W = weight of the base (KN);

L, B = base length and breadth (m);

P = safe bearing pressure (kN/m^2 or kPa).

The required area is found from the characteristic loads including the weight of the base:

$$\text{Base area} = \frac{(G_k + Q_k + W)}{P} = B \times L \;\; [m^2] \qquad (2.65)$$

The design of the base is made for the ultimate load delivered to the base by the column shaft, i.e., the design load is $G_k + Q_k$.

ii. Preliminary Design

1. Loads at the base of the column

To give dimensions to the footing it is necessary to know to what loads it will be subject. So, it is necessary to calculate all the weights of the elements that will be supported by the foundation under study and the foundation is the one below the column chosen. N_{Ed} is the load taken from the column at SLS.

As the results of the ground investigation provide estimates of the soil strength, the value will be used to pre-design the footing. Using EC2 will be presented next to the equations required for the preliminary design of footings with centred square columns. So, to determine the area of the foundation:

$$\sigma_{sd} \leq \sigma_{Rd} \qquad (2.66)$$

$$\frac{N_{Ed}}{B_0^2} \leq \sigma_{Rd} \qquad (2.67)$$

where $B_0 = \sqrt{\frac{N_{Ed}}{\sigma_{Rd}}}$

$$B_x = B_0 + 2e_x \qquad (2.68)$$

$$B_y = B_0 + 2e_y \qquad (2.69)$$

$$e_{x,y} = \frac{M_{x,y}}{V} \qquad (2.70)$$

Where B_x and B_y are the dimensions of the footing, N_{Ed} is the vertical load applied to the base of the column plus the weight of the footing. Since one doesn't has yet the dimensions of the footing it will be assume that the weight of the footing is 10% of the vertical load it's carrying.

$\sigma_{Rd} = 0.25MPa$ is the resistance of the soil;

$e_{x,y}$ is the eccentricity of the load in the directions x and y.

Since there are no moments, eccentricity does not exist and so the dimensions of the footing, B_x and B_y will be equal ($B_x = B_y = B_0$).

Now it is necessary to dimension the height of the footing. This height has to be sufficiently large to resist shear but not exceedingly large that might implicate a waste of money. Thus, some equations will be used to define this dimension and then, with all the dimensions already set, the structure stability against shear and punching will be tested. With this, we can guarantee that this element will not need extra steel to prevent those two phenomena.

$$h \geq \frac{B-b}{4} + 0.05 \quad [m] \qquad (2.71)$$

Where,

h is the height of the footing, B is the length of footing and b is the length of column.

iii. Design of reinforcement

The area of steel is gotten from the following equation given below

$$A_s = \frac{N\,(B-b)}{8.\,d\,f_{yd}^{*}} \qquad (2.72)$$

with $f_{yd}^{*} = 0.85.\,f_{yd}$.

The area of steel is determined for each direction *x* and *y*.

iv. Anchorage of column starter bars

Apart from the reinforcement in the base, column bars extend at least an anchorage length to which column reinforcement is attached. It is common practice to cast along with the base a short length of the column. This is called a kicker. This facilitates the positioning of the formwork for the column. The required compression lap length is determined by

$$l_0 = l_{b,requuired} \geq max\{15 \times bar\ diameter; 200mm\} \qquad (2.73)$$

2.8. Period of vibration

For the determination of the fundamental period of vibration period T₁ of the building, expressions based on methods of structural dynamics (for example the Rayleigh method) may be used.

For buildings with heights of up to 40 m the value of T_1 (in seconds) may be approximated by the following equation:

$$T_1 = C_t H^{3/4} \qquad (2.74)$$

Where C_t is 0.075 for moment resistant space concrete frames. The value of T_1 calculated from Rayleigh's formula or from numerical methods must not exceed those estimated from appropriated empirical formulae of more than 20%. With a fundamental period of T = 0s, we must have the first two modes which are the translational mode and the third mode that is the torsional mode.

2.9. Soil-structure interaction (foundation study)

Interaction with the soil can be represented by springs acting on the translation or rotation connected to a mobile base moving according to the imposed seismic motion. The movement of the mobile base is the one defined in free field by an accelerogram, while the movement of the building foundation depends on the mass of the structure and the stiffness of the springs that represent the ground. These two movements are therefore different, the more deformable the ground, the greater the gap.

In general, in studies of common structures, soil properties are not taken into account. The most commonly used model has a perfect embedding at the base at which to directly imposes movement. This obviously leads to an approximation in the calculation of the answer. The errors thus introduced can be significant in the case of a highly deformable soil: the period of the first mode is underestimated and results in an error by excess or by default on the acceleration value according to the zone where we are located on the calculation spectrum. It is therefore necessary to design models in which the soil is represented either by springs either by finite elements or by other types of modelling allowing to reproduce his behavior.

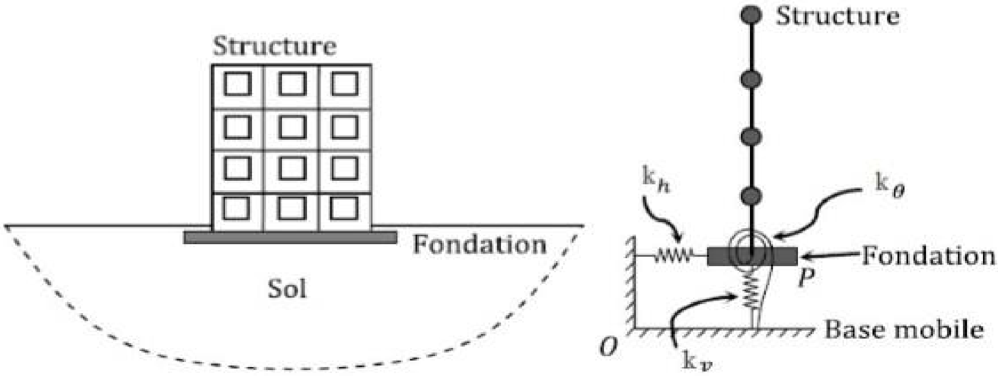


Figure 2.10. Diagram of the soil-structure interaction (A. Seghir, 2010)

The preliminary design to obtain the foundation section is obtained as shown below:

$$S \geq \frac{P}{\sigma_{soil}} \qquad (2.75)$$

Two types of stiffness are defined in this case; the vertical and horizontal stiffness for a rectangular foundation (the length of the foundation, L exceeds its width, B)

2.9.1. Vertical stiffness

It is calculated using the formula below

$$k_v = \frac{E}{2(1-v^2)}\beta_v\sqrt{B.L} \qquad (2.76)$$

With $\beta_v = 1.55\left(\frac{L}{B}\right)^{0.25} + 0.80\left(\frac{B}{L}\right)^{0.5}$ (2.77)

2.9.2. Horizontal stiffness

The horizontal stiffness shall be calculated in two directions.

2.9.2.1. Horizontal stiffness with respect to the foundation width

It is determined as thus

$$k_B = \frac{E}{2(2-v)(1-v)}\beta_B\sqrt{B.L} \qquad (2.78)$$

With $\beta_B = 3.4\left(\frac{L}{B}\right)^{0.15} + 1.2\left(\frac{B}{L}\right)^{0.5}$ (2.79)

2.9.2.2. Horizontal stiffness with respect to the foundation length

It is determined as follows

$$k_L = \frac{E}{2(2-v)(1-v)}\beta_L\sqrt{B.L} \qquad (2.80)$$

With $\beta_B = 3.4\left(\frac{L}{B}\right)^{0.15} + 0.4\left(\frac{L}{B}\right)^{0.5} + 0.8\left(\frac{B}{L}\right)^{0.5}$ (2.81)

2.10. Seismic analysis

Seismic analysis is a major tool in earthquake engineering which is used to understand the response of buildings due to seismic excitations in a simpler manner. In the past the buildings were designed just for gravity loads and seismic analysis is a recent development. It is a part of structural analysis and a part of structural design where earthquake is prevalent. One of the challenges for building design on the basis of their seismic performance has been the development of simple and effective methods for analyzing, designing and verifying the effects of earthquakes on structures. Methods of analysis must be able to predict the demand for forces and displacements imposed by earthquakes realistically. In response to these requirements, some regulations have incorporated methods to determine the travel demand imposed on a building likely to have inelastic behaviour during an earthquake. Analysis methods are broadly classified as linear static, linear dynamic, nonlinear static and nonlinear dynamic methods. This classification is illustrated in figure 2.8.

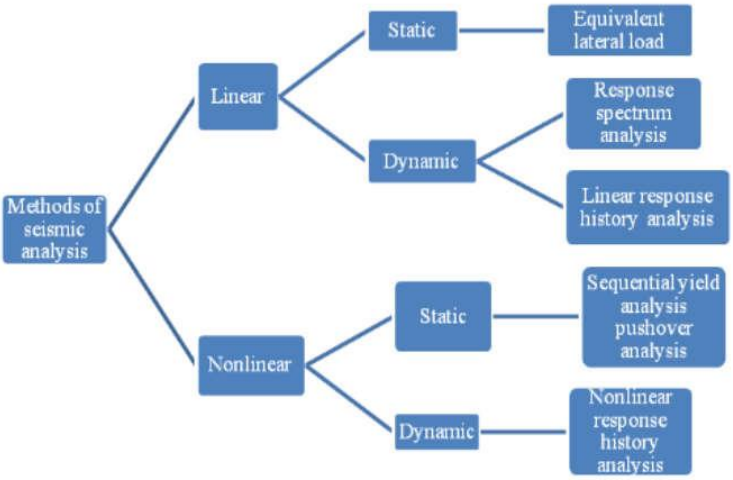


Figure 2.11. Classification of different methods of seismic analysis

In this study, the type of seismic analysis to be done here is the nonlinear static analysis that is the pushover analysis.

2.10.1. Pushover analysis

Pushover analysis is a static procedure that uses a simplified nonlinear technique to estimate seismic structural deformations. Structures redesign themselves during earthquakes. As individual components of a structure yield or fail, the dynamic forces on the building are shifted to other components. A pushover analysis simulates this phenomenon by applying loads until the weak link in the structure is found and then revising the model to incorporate the changes in the structure caused by the weak link. A second iteration indicates how the loads are redistributed. The structure is "pushed" again until the second weak link is discovered. This process continues until a yield pattern for the whole structure under seismic loading is identified.

Pushover analysis is commonly used to evaluate the seismic capacity of existing structures and appears in several recent guidelines for retrofit seismic design. It can also be useful for performance-based design of new buildings that rely on ductility or redundancies to resist earthquake forces.

Conclusion

The chapter's objective was to bring out the main general presentation of the building by its plans, its geometric characteristic and parameters taken for the consideration of the building plan. The procedure for designing the structure under ULS and SLS was also mentioned in order to analyze the structure with respect to modal analysis. In addition, the pushover analysis was explained, which is quite accurate for structural design applications.

CHAPTER 3. ANALYSIS AND DESIGN OF THE BUILDING

Introduction

This chapter brings out the detailed calculation and result of the analysis and design of the building. The elements of a structure are classified, by consideration of their nature and function, as beams, columns, slabs, walls, plates, arches, shells etc. Rules are provided for the analysis of the commoner of these elements and of structures consisting of combinations of these elements. The loads applied on the building where calculated according to Eurocode 1 'Actions on structures. These loads include the following: self-weight of the structural element, non-structural permanent (dead) loads and variable (live) loads.

3.1. Hypothesis

The class of concrete used for this study is C25/30 and the characteristic yield strengths (f_{yk}) of steel is B450C. The elastic modulus for steel is given as $E_s = 200\ 000\ N/mm^2$. The different variable and permanent loads used are illustrated below under action of structures. The snow load is not taken into account since Yaoundé has no snow falls. The earthquake intensity is $0.7 m/s^2$, hence considered negligible

3.2. Predesign for the constructive elements

There are so many notes when considering the predesign of a building but the ones to be considered here are the following:

3.2.1. Beam

For the beam section;

$$\frac{l}{15} \leq h \leq \frac{l}{10} \qquad (3.1)$$

$$0.3h \leq b \leq 0.6h \qquad (3.2)$$

Where l is the length of the longest span of the beam, b is the breadth (width) of the beam's section and h is the height (depth) of the beam section.

By this, the model is as shown below in fig 3.1.

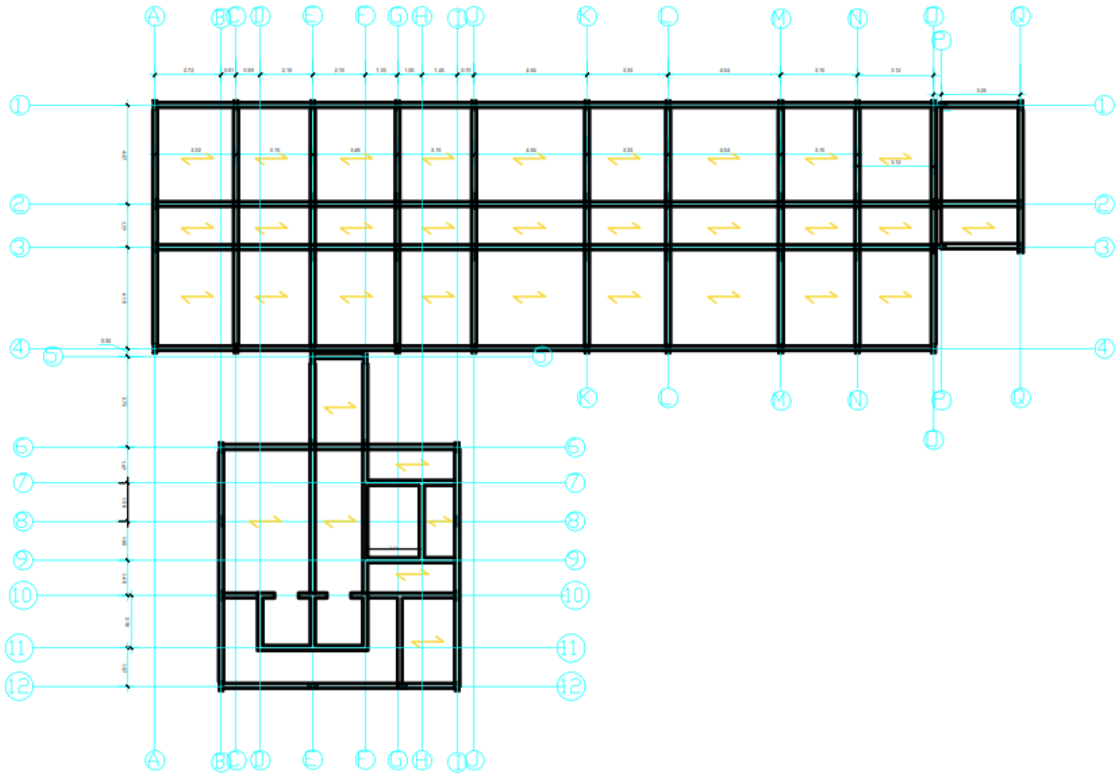


Figure 3.1. Formwork model

As earlier said in the previous chapter, the building is comprised of three different sections all connected by structural joints as shown in figure 3.2 where:

- The section in Blue is block A (section A of the building)
- The section in Red is block B (section B of the building)
- The section in green is block C (section C of the building)

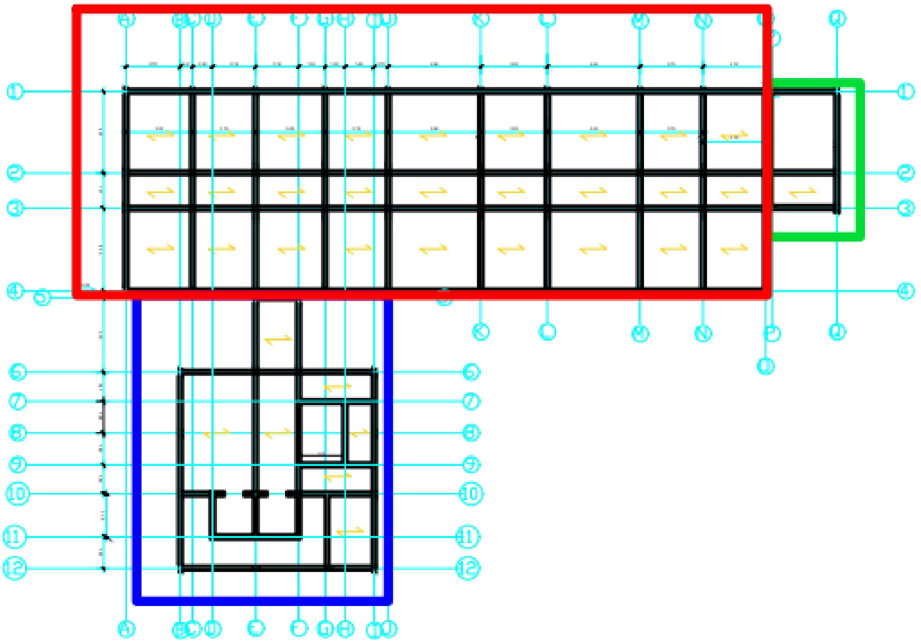


Figure 3.2. the different sections of the irregular building

The sections are linked by separation joints hence makes the building to be irregular. The model shows that the longest influence area is $l_{f,max} = 3.985m$ and the considering the principal beams, the longest span is $l = 4.64m$.

By conclusion, the beam's sections were obtained as $h = 40cm$ and $b = 20cm$.

3.2.2. Column

For the column section;

$$N_{Rd} = 0.6. f_{cd} A_c \geq N_{sd} \qquad (3.3)$$

where A_c is the area of the concrete section and N_{sd} is the axial load computed using the recovery area of the column.

The axial load N_{sd} is determined using the expression:

$$N_{sd} = q. S_r. n \qquad (3.4)$$

where q is the uniform distributed loads on each floor compute at ULS combination, S_r is the recovery area of the column section and n is the number of floors of the building.

3.3. Action on structures

The different action taken into consideration are the permanent action and the variable action.

3.3.1. Permanent actions (loads)

Permanent actions include structural loads and non-structural loads applied on the building. These loads are considered in the design of the beams and columns. (see table 3.1 and table 3.2)

Table 3.1. self-weight action

actions	symbol	value	unit
self-weight			
breadth	b	200	mm
height	h	400	mm
concrete unit weight density	γ_c	25	kN/m^3
self-weight	G_1	2	kN/m

Table 3.2. permanent actions

permanent load	symbol	value	unit
floor			
floor compression table (16+4)		2.8	kN/m^2
mortar		0.2	kN/m^2
ceramic tiles		0.45	kN/m^2
permanent load total	G_2	3.45	kN/m^2

3.3.2. Variable actions

The use of the building enables its classification in category B (for office use). Table 3.3 details the different imposed loads.

Table 3.3. Imposed loads

imposed load	symbol	value	unit
office	Q_k	2.5	kN/m^2

3.4. Materials

For the analysis, the class of concrete adopted is C25/30 and the steel reinforcement is B450C. The main characteristics of these materials for linear analysis and design of the structure are given in section 2.7.1.1 (of previous chapter) for concrete and section 2.7.1.2 (of previous chapter) for the steel reinforcement.

3.5. Static Analysis and design of The Building

This part of the work focuses on the design of reinforced concrete resisting elements such as columns, beams and footings, based on prescriptions given by Eurocodes 1 and 2. It represents the bases of justification to the resistance, stability and durability of the building to solicitations induced by vertical loads.

3.5.1. Presentation of the structural model

A simplified analytical model was proposed for modelling the reinforced concrete building using standard structural analysis software. The program SAP2000 was used to implement the proposed model for evaluating structural response by means of linear static and linear dynamic analysis. The chosen three-dimensional (spatial) structural model used is shown in fig 3.3. The basic characteristics of the model are as follows:

- All elements (columns and beams) are modelled as line elements.
- All elements are fully fixed in foundation (fixed joint support).
- Frames are connected together by means of rigid diaphragms (in horizontal plane) at each floor level. The slabs are not modelled.
- The accidental torsional effects are taken into account by means of torsional moments about the vertical axis.
- The rupture links are considered in this structural model.

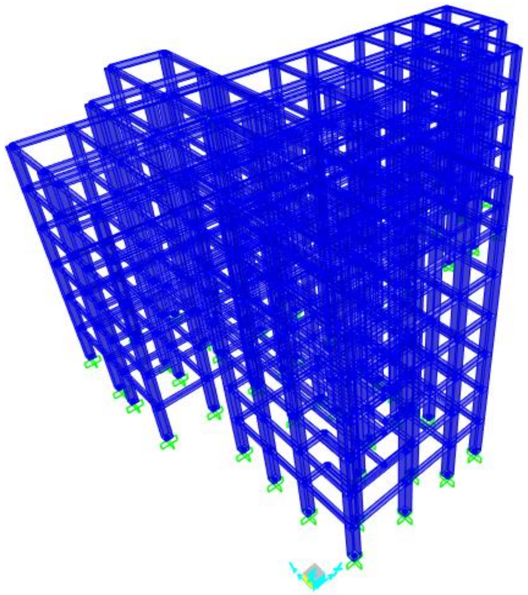


Figure 3.3. RC building modelled with SAP2000

3.5.2. Beam design

The selected beam shown in fig 3.4 was chosen for the design of the element as principal beam

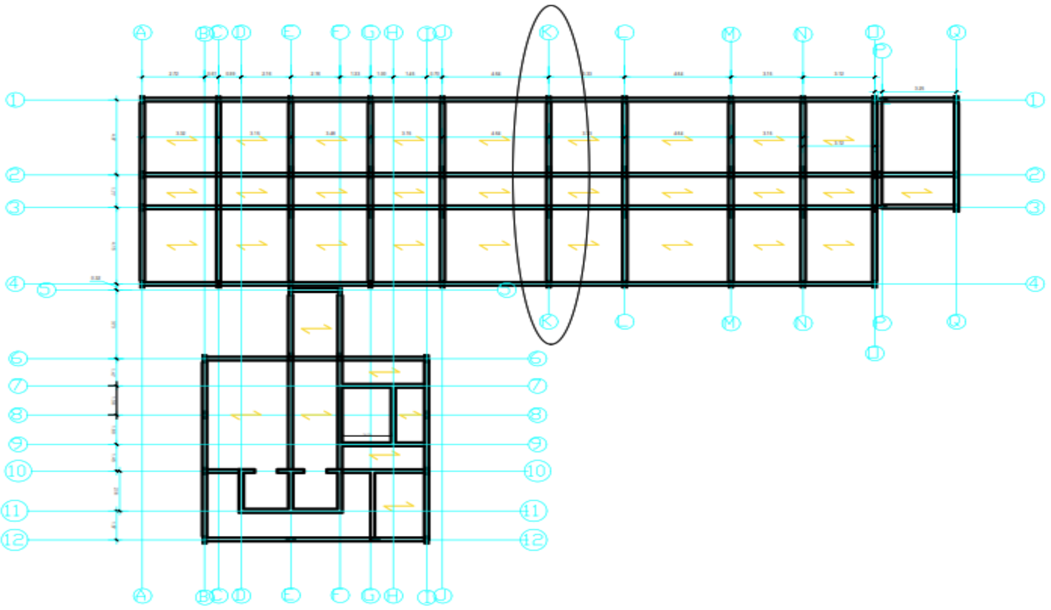


Figure 3.4. Selected beam for design

The following different load combinations were also taken for the design of the building as shown in figures 3.5, 3.6, 3.7 and 3.8 respectively.

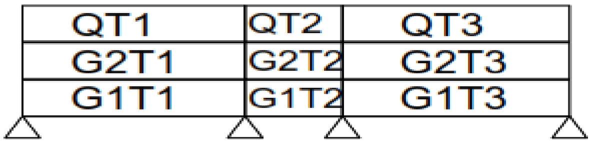


Figure 3.5. Load combination 1

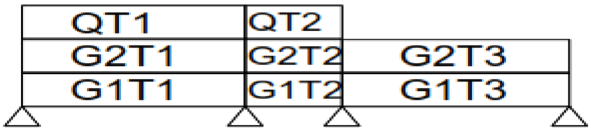


Figure 3.6. Load combination 2

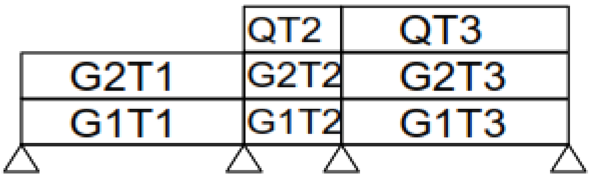


Figure 3.7. Load combination 3

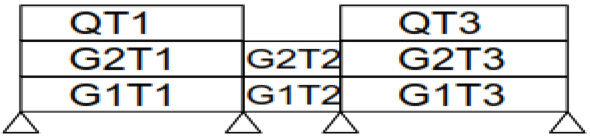


Figure 3.8. Load combination 4

3.5.2.1. Bending moment verification

From the above 4 load combinations, 4 solicitation curves (bending moment) were obtained as shown in fig 3.9.

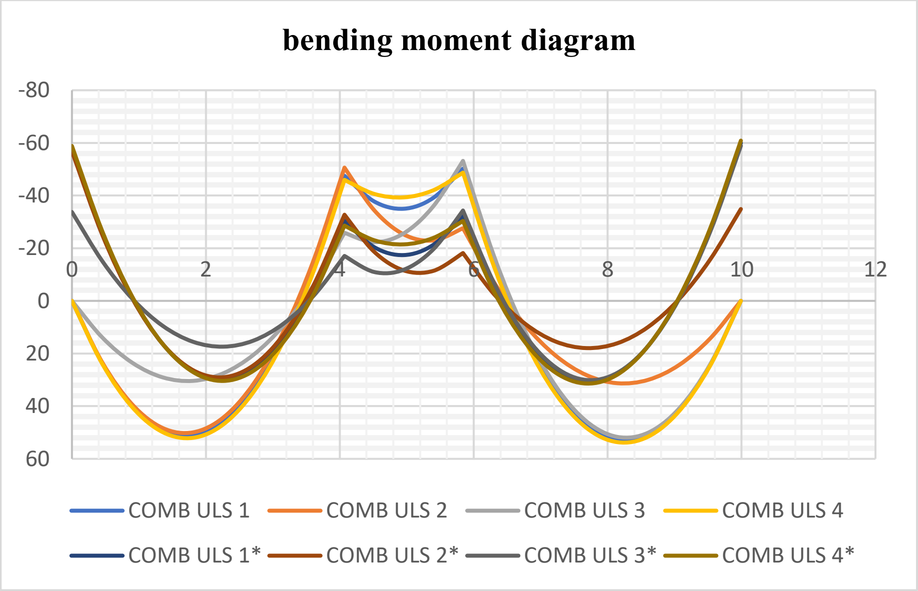


Figure 3.9. bending moment diagram for the different load combinations

The following envelope curve of bending moment was obtained (see fig 3.10)

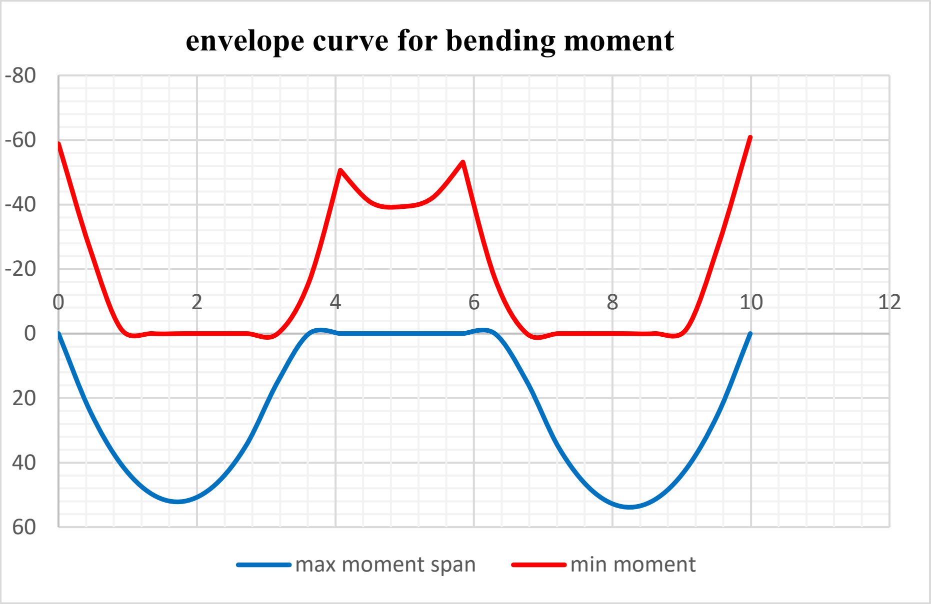


Figure 3.10. envelope curve for bending moment

The steel section was evaluated and verified for a beam section of 20x40 as shown in figure 3.11.

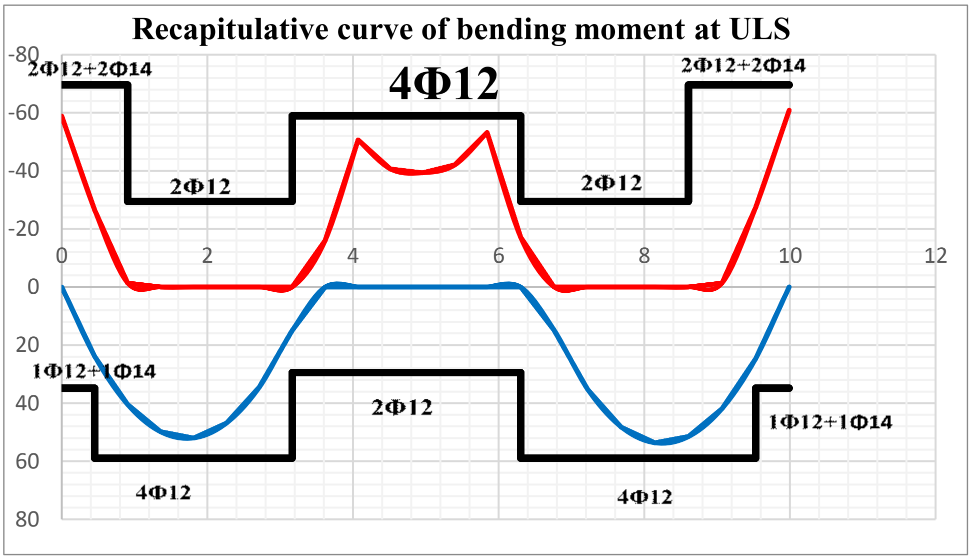


Figure 3.11. Recapitulative curve of bending moment at ULS

3.5.2.1. Shear verification

The solicitations curves obtained from 4 load arrangements are presented in fig 3.12.

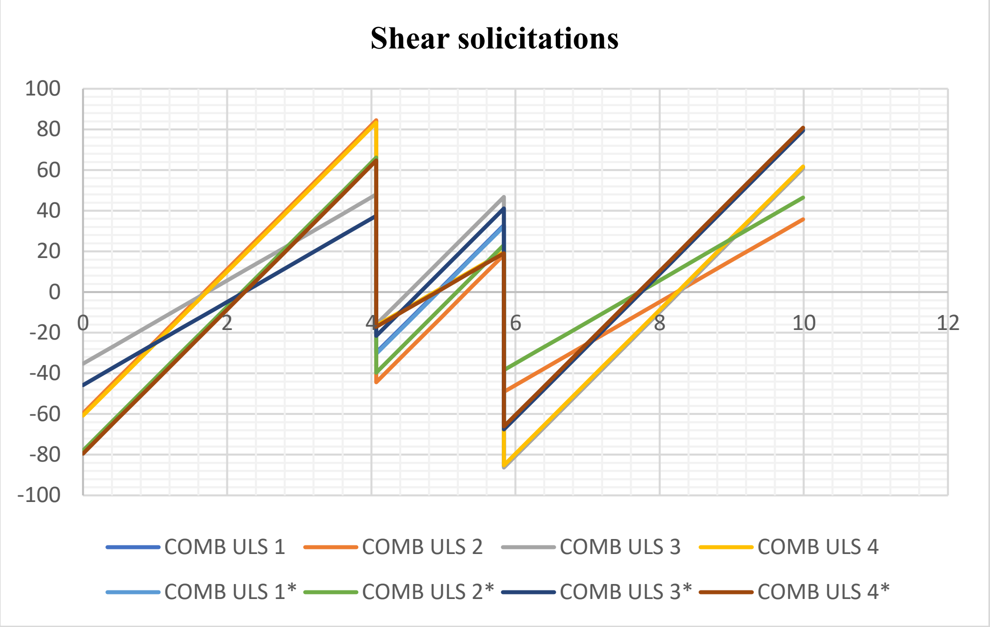


Figure 3.12. Shear solicitations obtained from 4 load combinations at ULS

The envelope curve diagram is as shown below in figure 3.13.

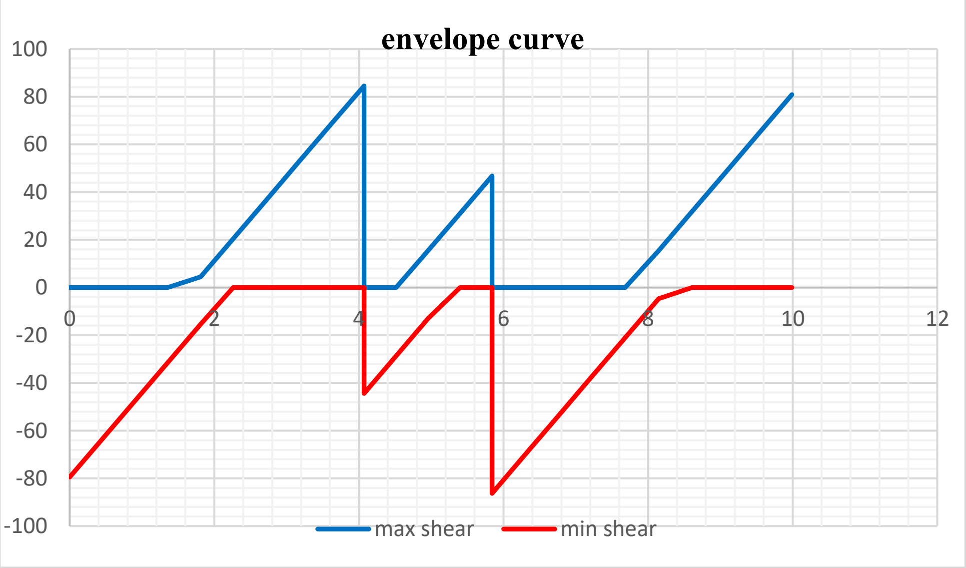


Figure 3.13. Envelope curve for shear at ULS

According to Eurocode 2, it was shown and observed that the acting shear V_{Ed} is less than the designed value V_{Rdc} defined in the previous chapter. The reinforcement for the stirrups were gotten for 2 legs of ϕ6 after 75mm and 85mm where most required.

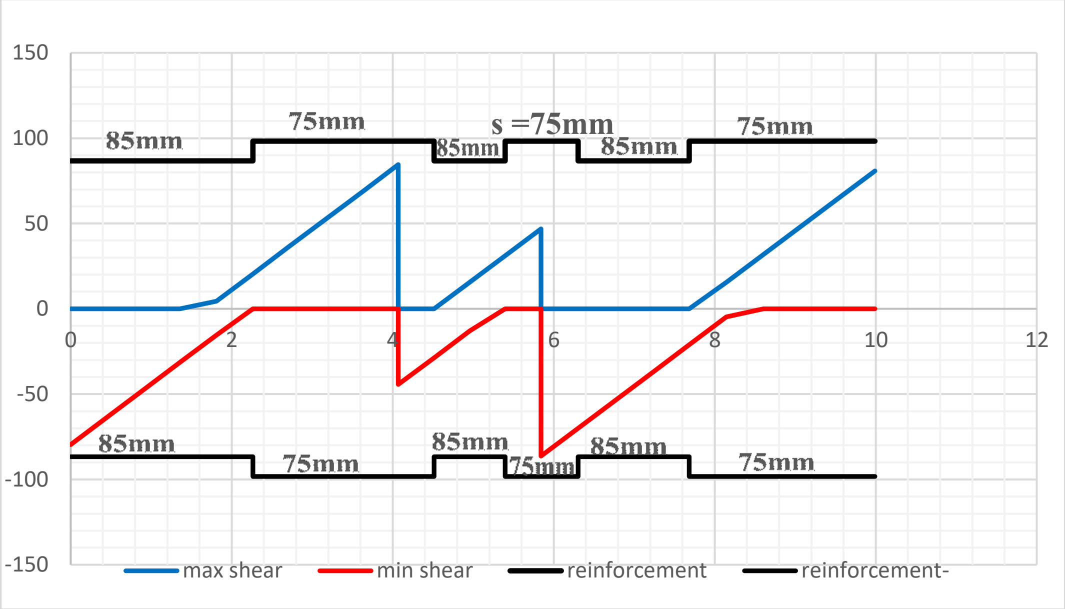


Figure 3.14. Recapitulative shear verification curve of the beam

3.5.2.2. SLS verification

a. Stress limitation

The verification of the allowable stress on the beam was done at the characteristic combination. The 4 load combinations inserted in SAP 2000 at the characteristic combination permit to obtain the solicitation curves presented in figure 3.15.

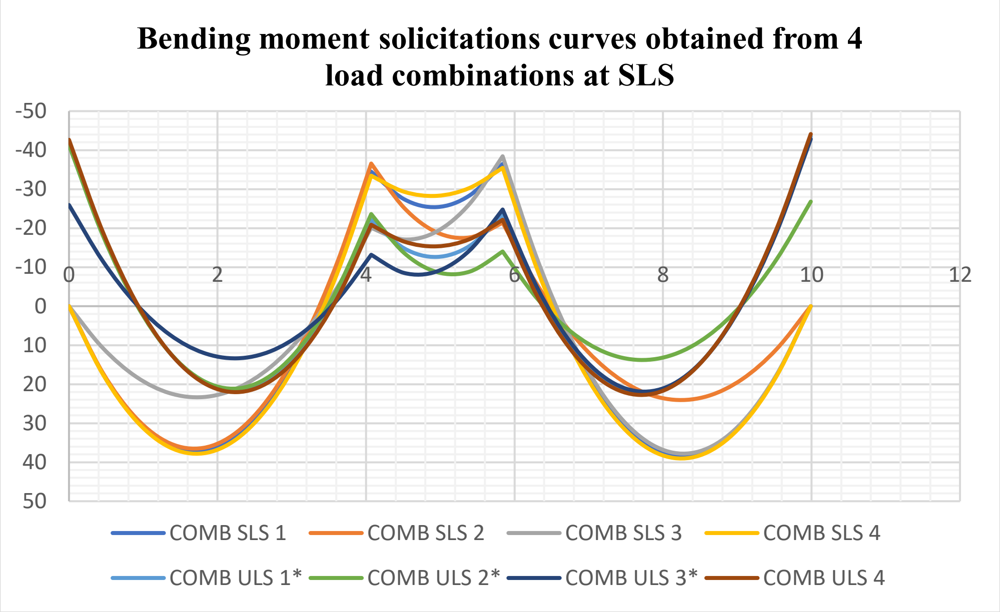


Figure 3.15. Bending moment solicitations curves obtained from 4 load combinations at SLS

These solicitation curves permit to obtain the envelope curve presented on the figure 3.16.

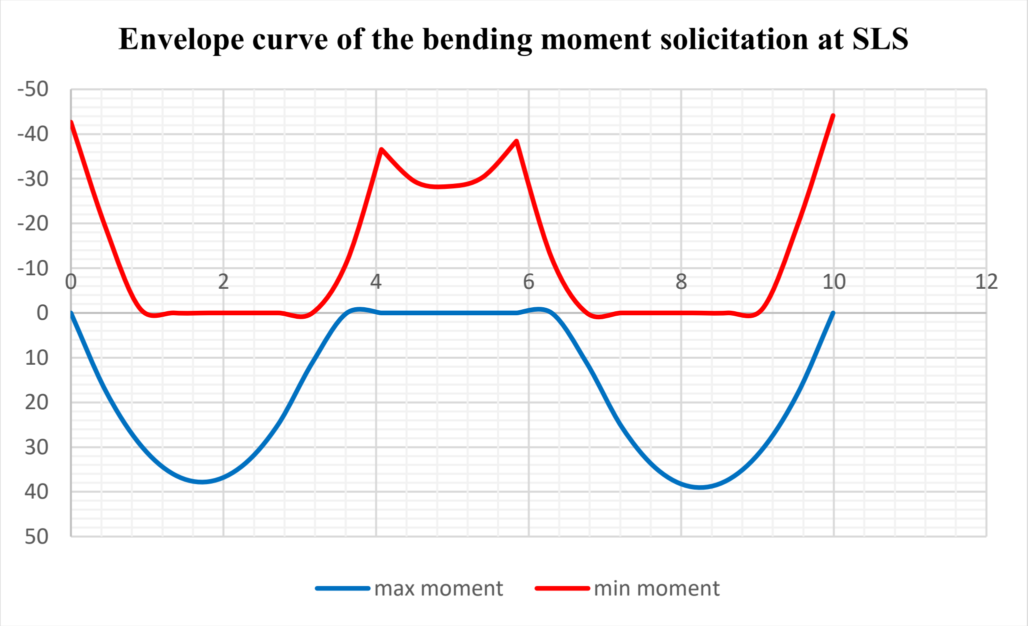


Figure 3.16. Envelope curve of the bending moment solicitation at serviceability limit state

It is then possible to draw the stress curves inside the concrete and inside the steel reinforcement. The figure 3.17 and 3.18 show a comparison of the stress inside the concrete and the steel reinforcement to the admissible stress respectively.

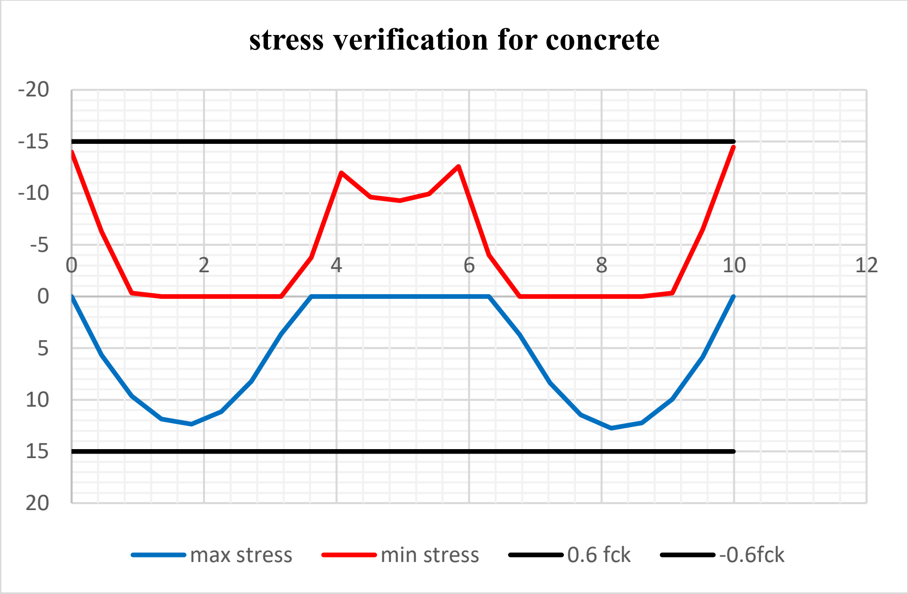


Figure 3.17. Recapitulative curve of the stress verification of the concrete

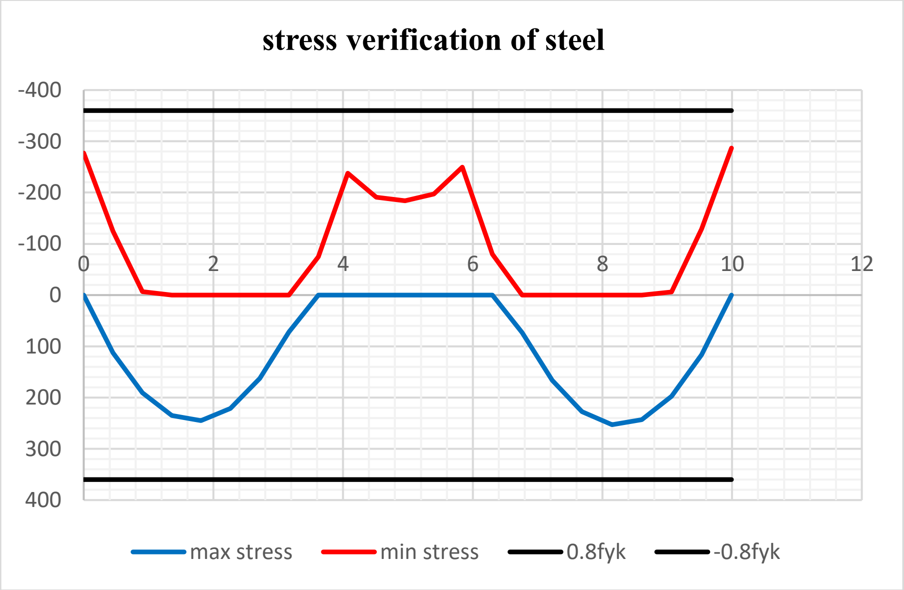


Figure 3.18. Recapitulative curve of the stress verification of the reinforcement

b. Crack control

The crack control verification was performed on the beam by using the quasi permanent combinations.

Figure 3.19 represents the bending moment solicitation diagram from the 4 different load arrangements.

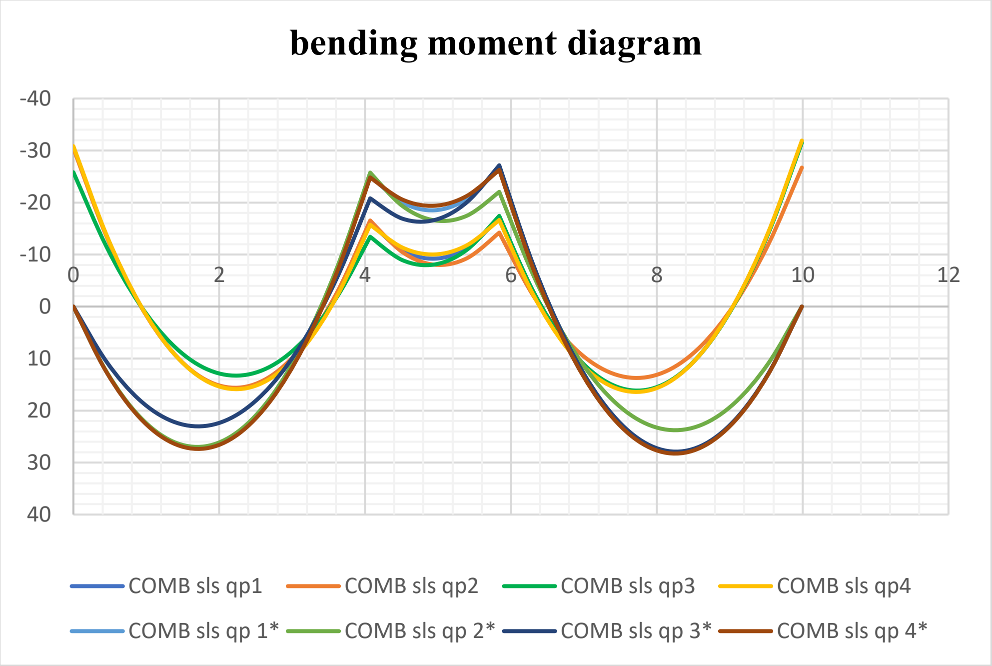


Figure 3.19. Bending moment solicitations curves obtained from 4 load combinations at SLS quasi-permanent

From the bending moment diagram above, the envelope curve is obtained as shown in figure 3.20.

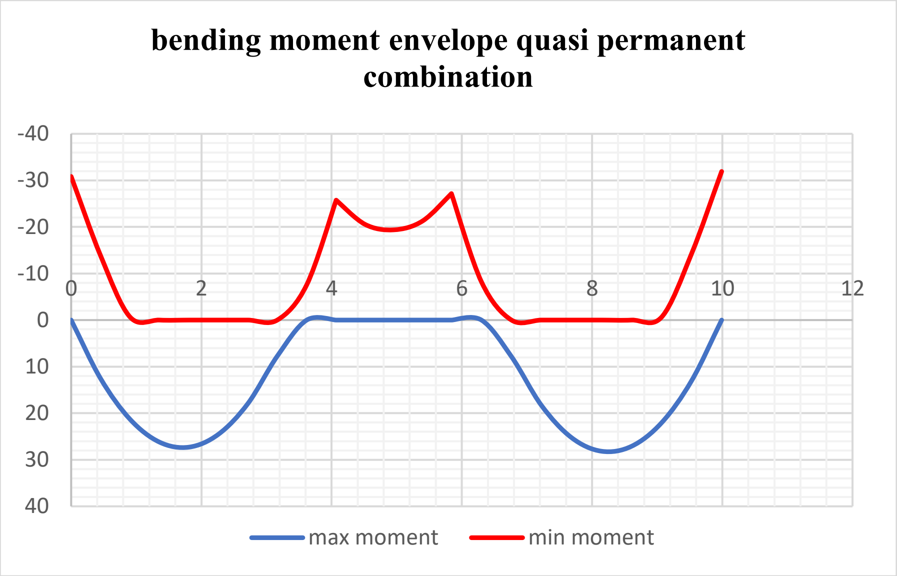


Figure 3.20. Envelope curve of the bending moment solicitation at serviceability limit state. quasi-permanent

From the above envelope, the moment required for the verification of crack control was used in the determination of the maximum crack width and the crack width.

The crack width limitation was verified and as conclusion it was shown that using an exposure class of XC2, the maximum bar sizing and the maximum bar spacing satisfy the bar sizing and bar spacing obtained from table 2.5 of previous chapter (see table 3.4 below)

Table 3.4. results obtained for the maximum crack width and bar spacing

Exposure class XC2	$w_k = 0.3$
Steel stress [MPa]	190.34
Maximum bar size, ϕ [MPa]	25
Maximum bar spacing [mm]	250

The calculation of the crack width was also performed and as result, it is shown that the beam section fulfills the crack control limitation. Table 3.5 represents the values of crack width within short term and long term.

Table 3.5. crack width values

Term of usage	Crack width [mm]
Short term	0.106
Long term	0.112

c. Deflection control

The deflection check was performed using the solicitation obtained from the quasi-permanent combination. Using the equations (2.37), an estimation of the ratio l/d was calculated and obtained.

It was checked that the beam section satisfied the deflection control using the span depth ratio. Table 3.6 illustrates the span depth ratio limitation and the control verification

Table 3.6. results obtained from the span to depth ratio

Span depth ratio [l/d] (from equation 2.37)	24.17
Short term stress [n = 6.3]	1.69
Long term stress [n = 15]	1.62
Span depth ratio calculated [l/d]	12.54
Check for short term	40.84
Check for long term	39.16

3.5.3. Column design

In the design of the column, the maximum solicitation will be chosen for the design of the structure. The encircled column in fig 3.21 was chosen for the design.

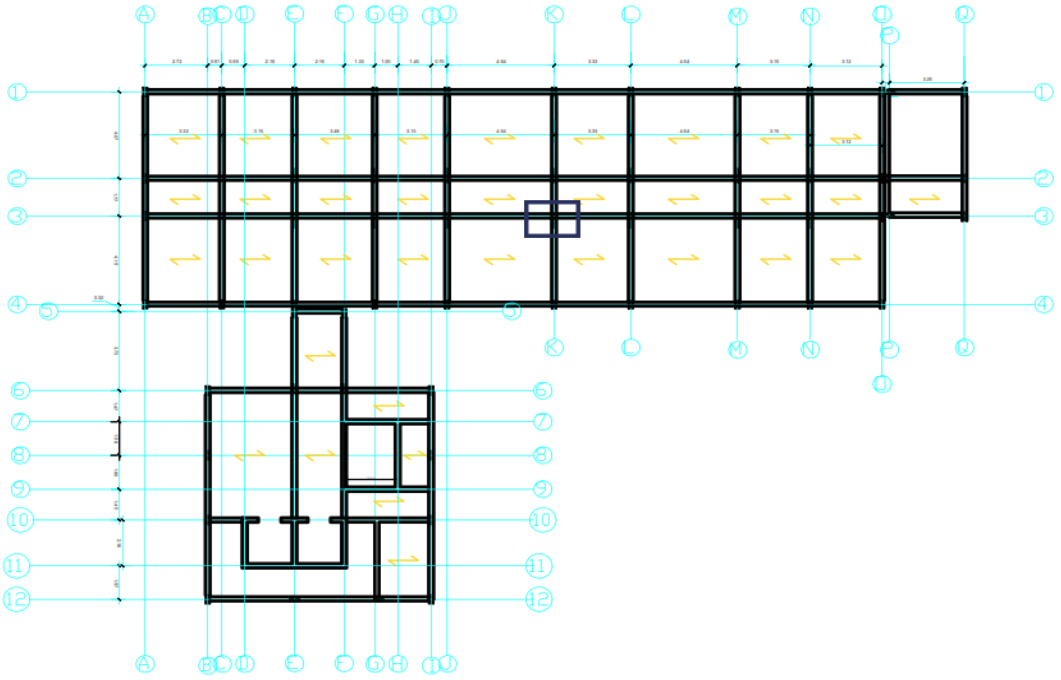


Figure 3.21. selected column for design

3.5.3.1. Preliminary design of column

The column is considered using 60% of the concrete resistance. The minimum section of the column is therefore estimated by the following expression:

$$N_{Rd} = 0.6. f_{cd} A_c \geq N_{sd}$$

With the area of the concrete section for column being calculated as:

$$A_c = \frac{N_{sd}}{0.6 f_{cd}} = \frac{q. S_r. n}{0.6 f_{cd}} = 232223.66 mm^2$$

Considering a square section, a, then:

$$a = \sqrt{A_c} = 481.89594 mm$$

So therefore, the column section is designed as a (50 x 50) cm^2 column.

Therefore:

$$N_{Rd} = 0.6 \times 16.67 \times 500 \times 500 = 2500500N = 2500.5kN > N_{Ed} = 832.677kN$$

with $A_{s,min} = \max\left\{\frac{0.01N_{Ed}}{f_{yd}}; 0.002A_c\right\} = \max\{21.3mm^2; 500mm^2\} = 500mm^2$

and $A_{s,max} = 0.04A_c = 10000mm^2$.

Therefore, choosing bars of 6ϕ14 and a cover of 30mm will suffice for the column design.

3.5.3.2. Modal analysis response of the building

The modal analysis of the structure permits an estimation of the section of the vertical element through the verification of the vibration modes of the structure and the period of the first vibration. The fundamental period of the building is given by:

$$T_1 = C_t H^{3/4} = 0.075 \times 21^{3/4} = 0.7357s$$

The 3D modelling of the building in SAP 2000 with a fixed base and a percentage of participation of the imposed loads of 30% permit to have the two first modes translational and the torsional third mode with the period for the first vibration mode T_1 = 0,63091 s.

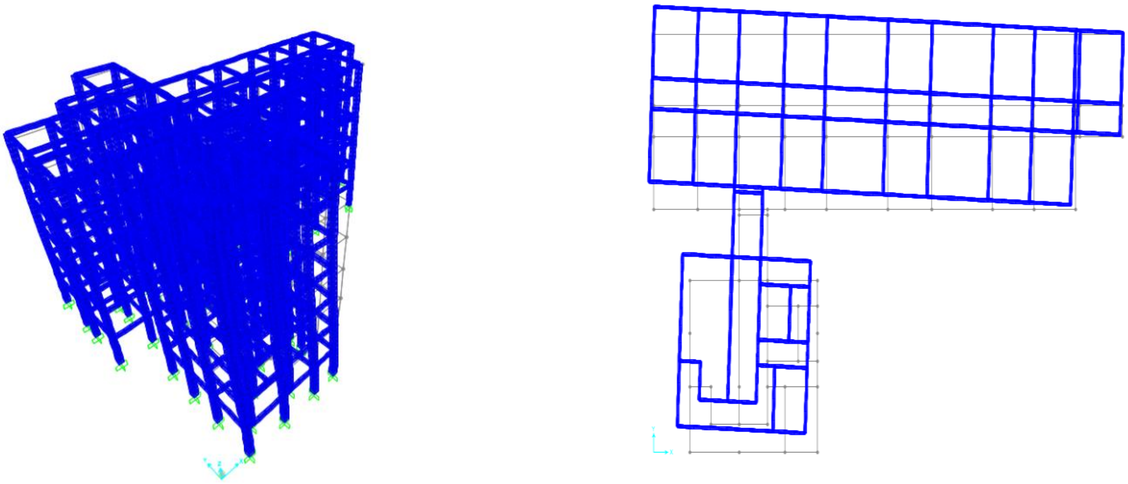


Figure 3.22. First vibration mode of the structure: Translational in the y direction

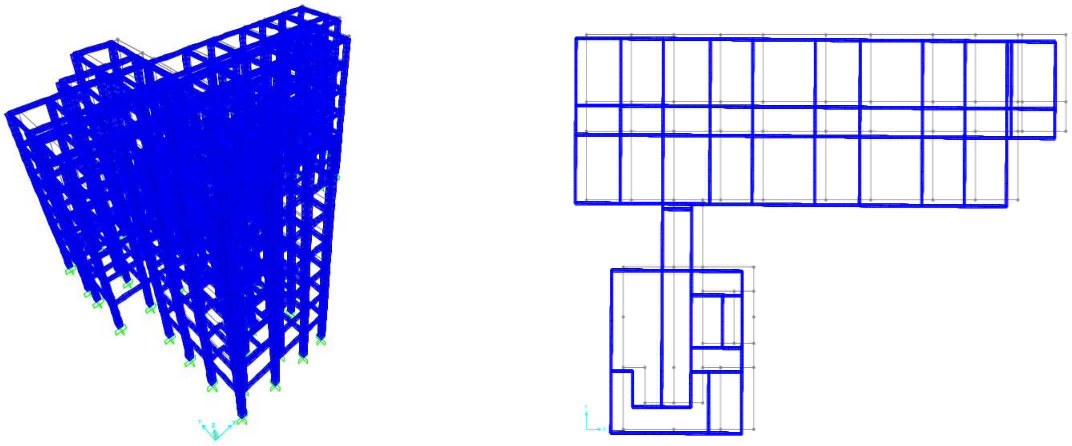


Figure 3.23. Second vibration modes of the structure: translational in the x direction

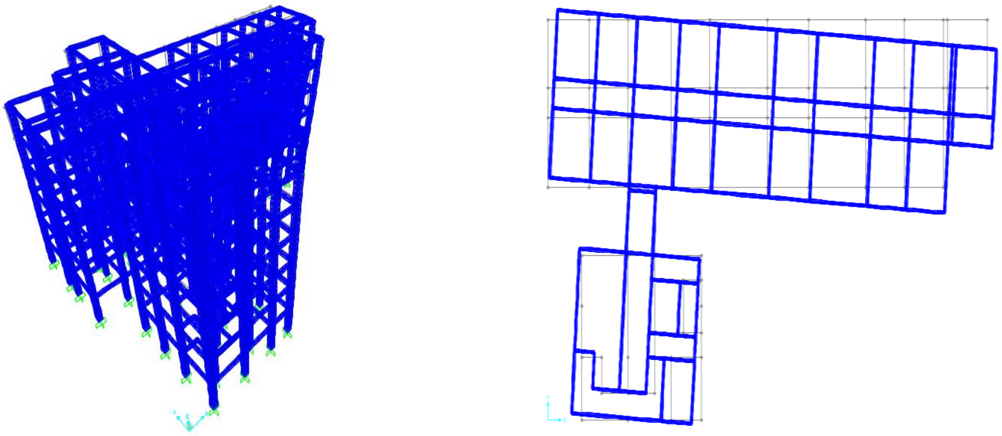


Figure 3.24. Third vibration modes of the structure: Torsional

3.5.3.3. Bending moment, shear and axial forces verifications

The 4 load arrangements (combinations) obtained from beams permit to obtain different solicitation curves for the bending moment both in the x-direction and the y-direction and the axial load as presented in the figure 3.25, 3.26 and 3.27 respectively.

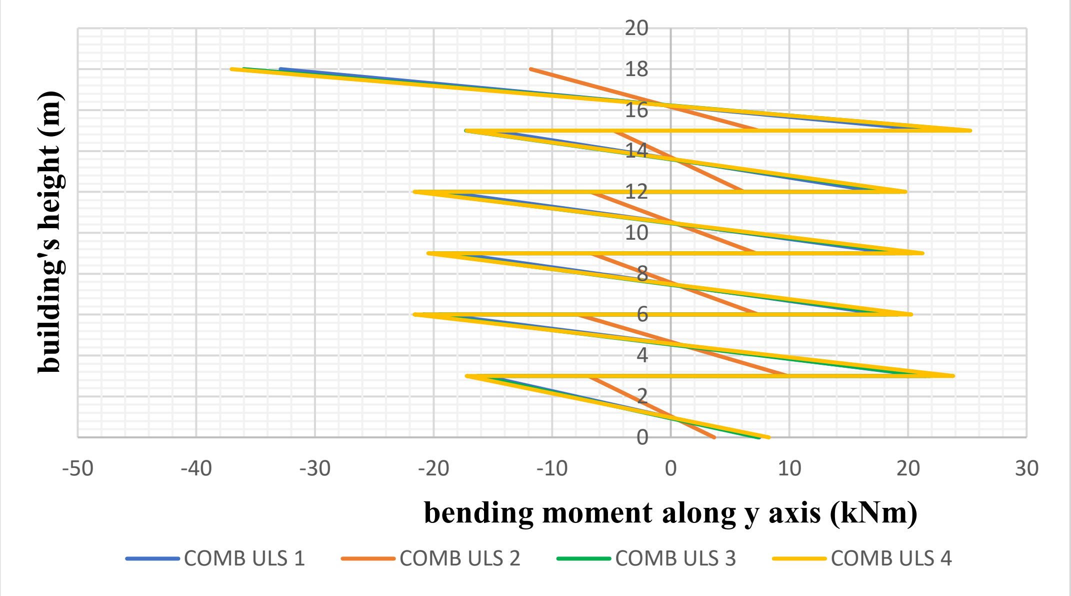


Figure 3.25. Bending moment solicitation curves on the column in the y-direction

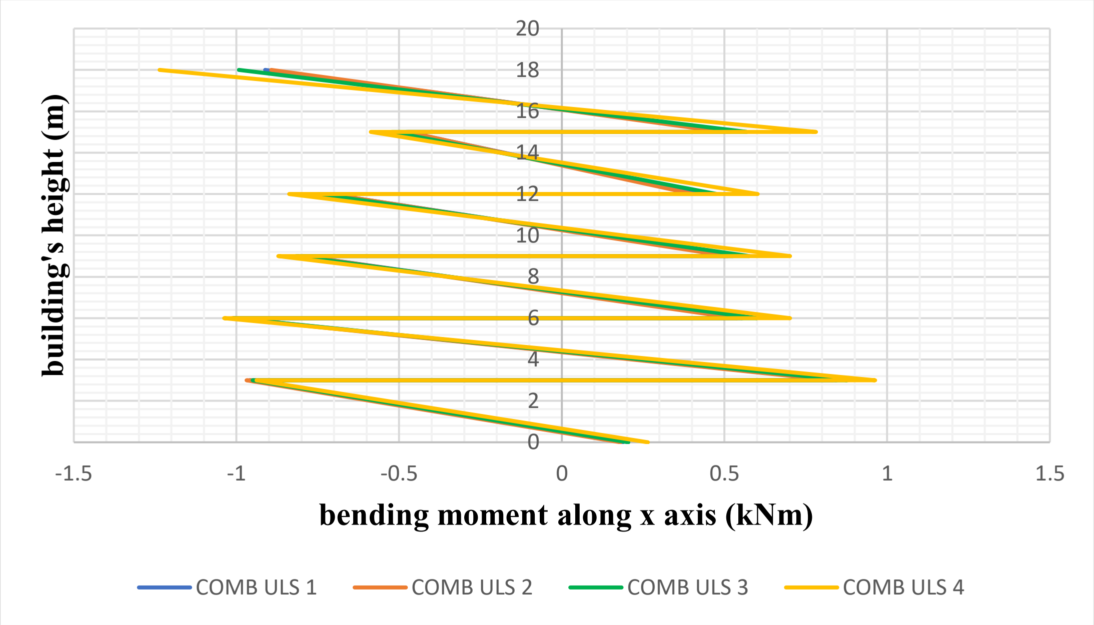


Figure 3.26. Bending moment solicitation curves on the column in x-direction

The axial load is as shown in fig 3.27 below

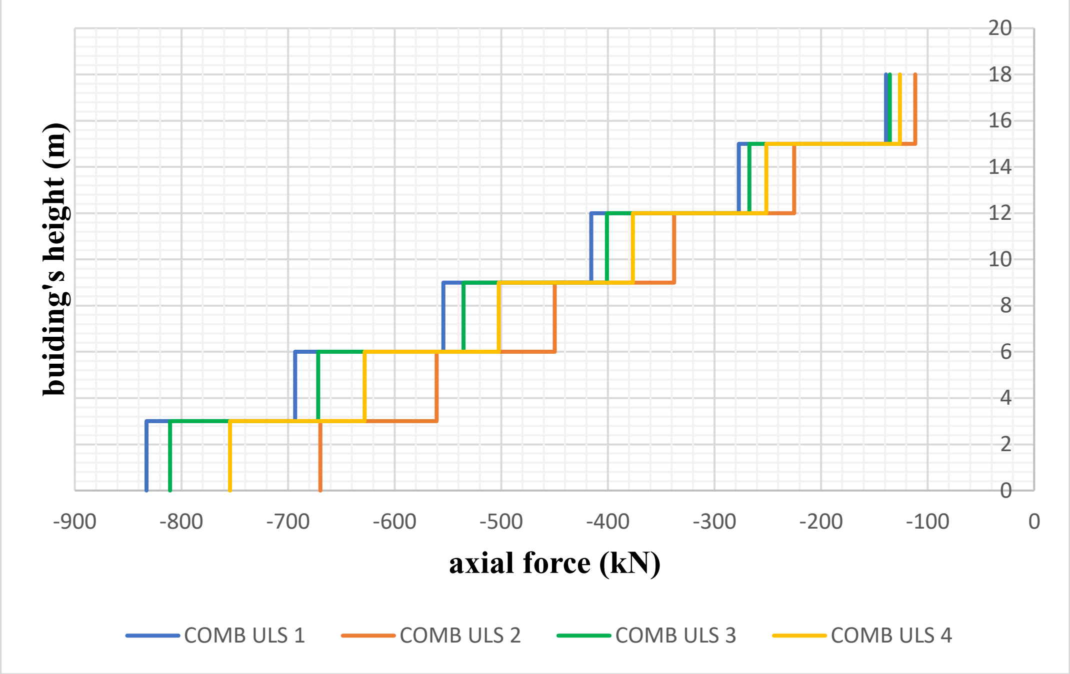


Figure 3.27. Axial load solicitation curve on the column

From these solicitations, the envelope curve of the bending moment in both x and y direction and the axial load solicitation are obtained as presented in figures 3.28, 3.29 and 3.30 below respectively.

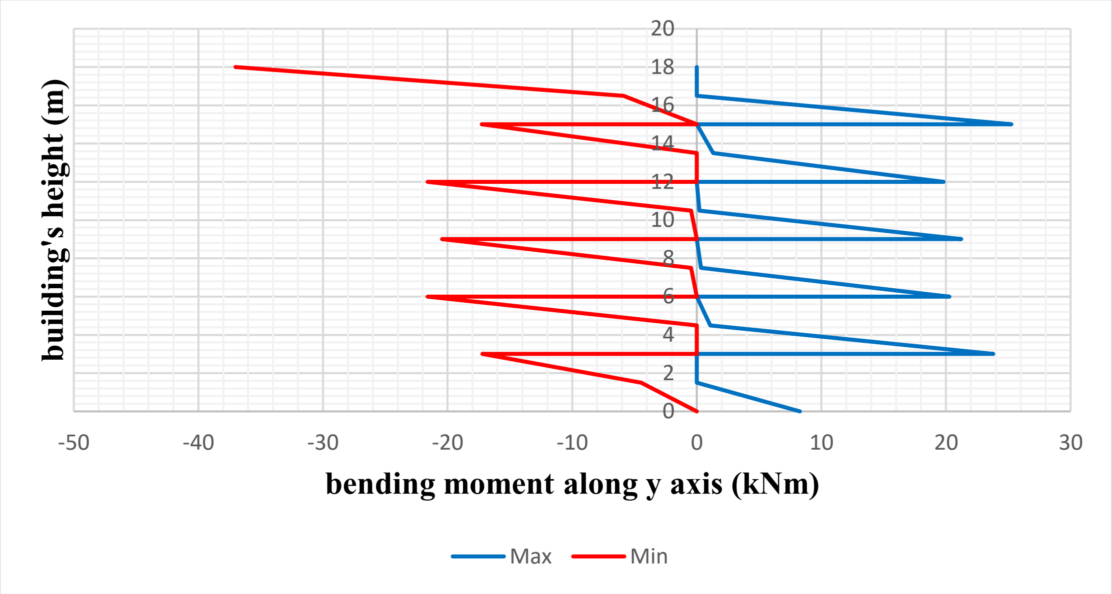


Figure 3.28. Bending moment envelope curve on the y-axis

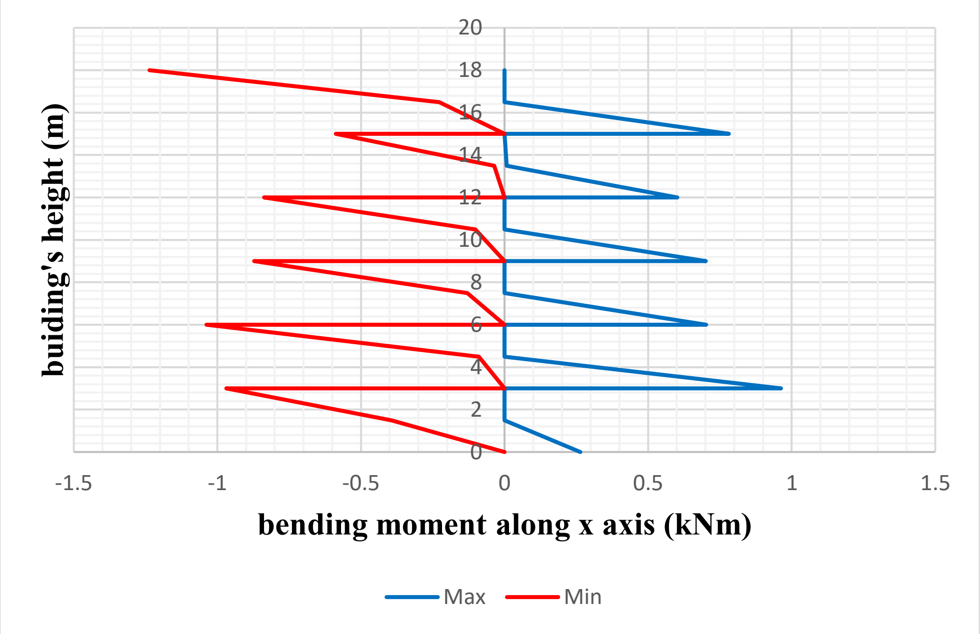


Figure 3.29. Bending moment envelope curve on the x-axis

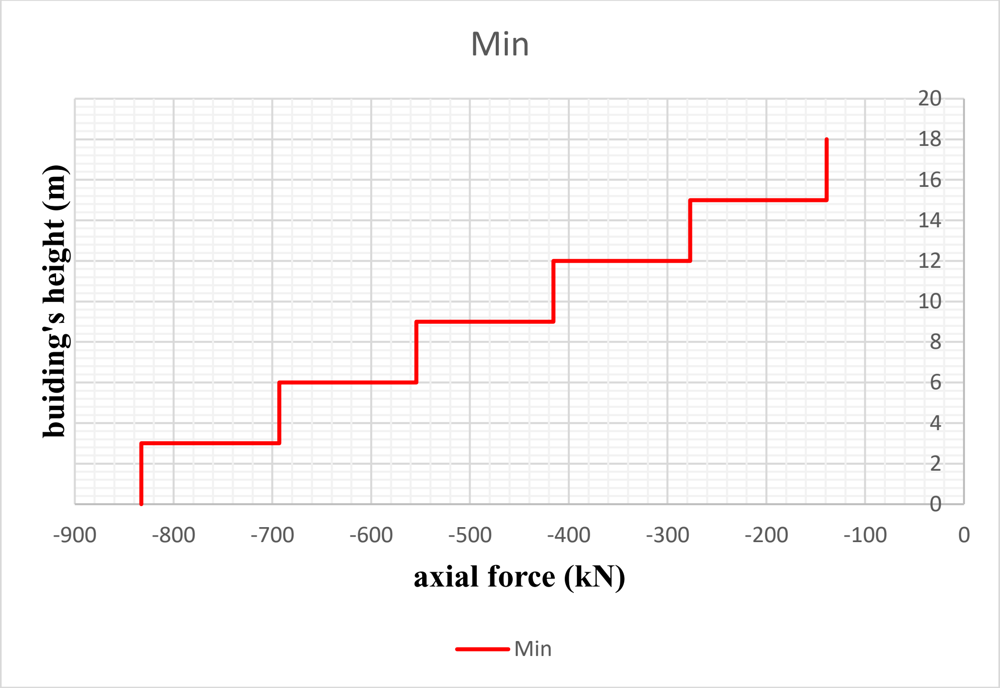


Figure 3.30. Axial load envelope curve

3.5.3.4. M-N interaction diagram

The interaction diagram of the column, considering a concrete section of 50cm x 50cm is presented in figure 3.31.

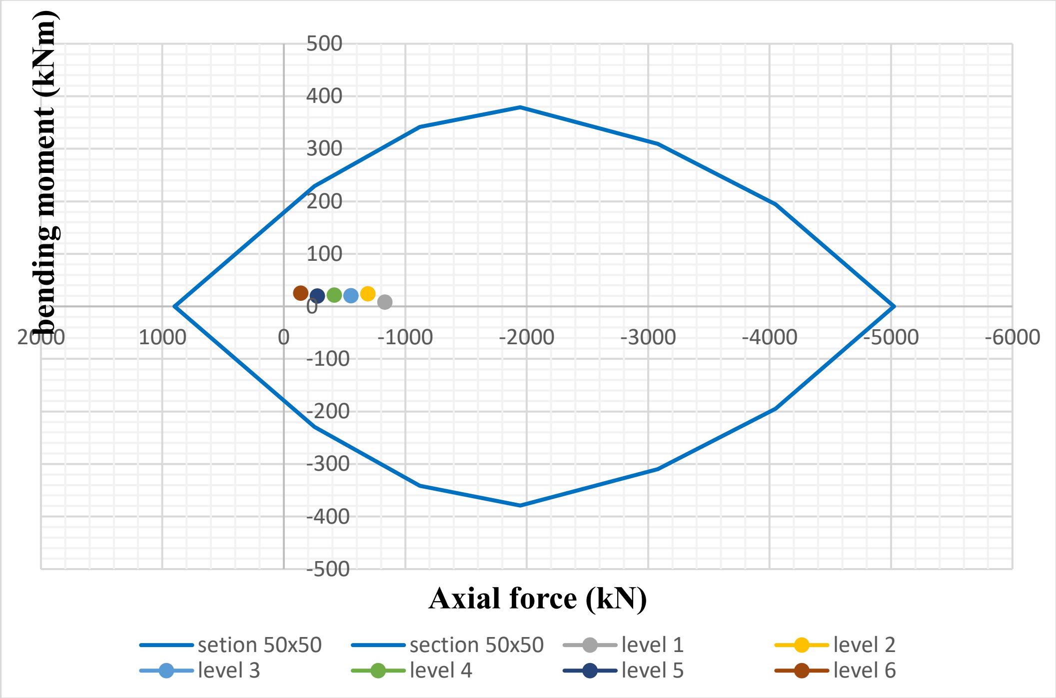


Figure 3.31. Moment-axial load interaction diagram

Each dot represents the maximum acting axial force-bending moment for each floor and decreases as we move to the top of the building. It is noticed from the graph above that the no point lies out of the diagram with the column section 50cm x 50cm and is therefore acceptable, hence no failure occurs within this section.

3.5.3.5. Slenderness verification

Following the procedure in section c of section 2.7.3.2 on the slenderness and effective length of a column, the parameters λ *and* λ_{lim} were determined as shown in tables 3.7 and 3.8 below:

Table 3.7. Parameter for slenderness verification

b[mm]	h[mm]	l[mm]	l_0[mm]	I[mm^4]	A[mm^2]	i	λ
500	500	1800	1260	5208333333	250000	144.338	8.72954

Table 3.8. Parameter for the limit slenderness verification

A	B	C	n	λ_{lim}
0.7	1.08	1	0.1998	33.826

It is shown from the above tables 3.7 and 3.8 that $\lambda < \lambda_{lim}$ hence the designed column is not a slender column.

3.5.3.6. Shear verification

The different load arrangements permit to obtain curves in the x and y-direction as presented in the figure 3.32 and figure 3.33 respectively

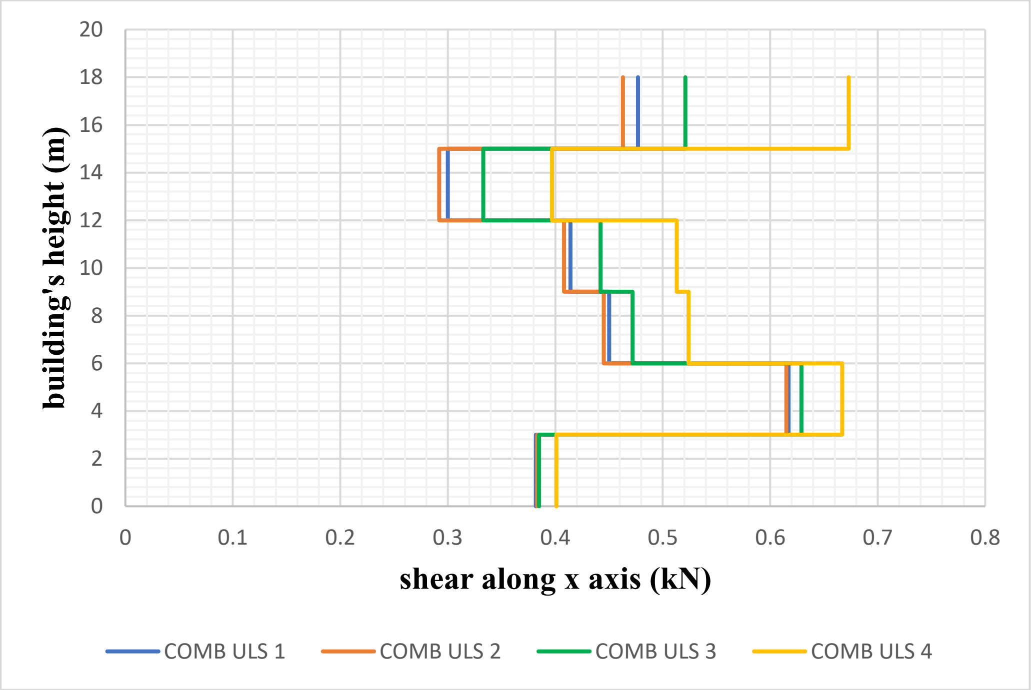


Figure 3.32. Shear force along the x-axis

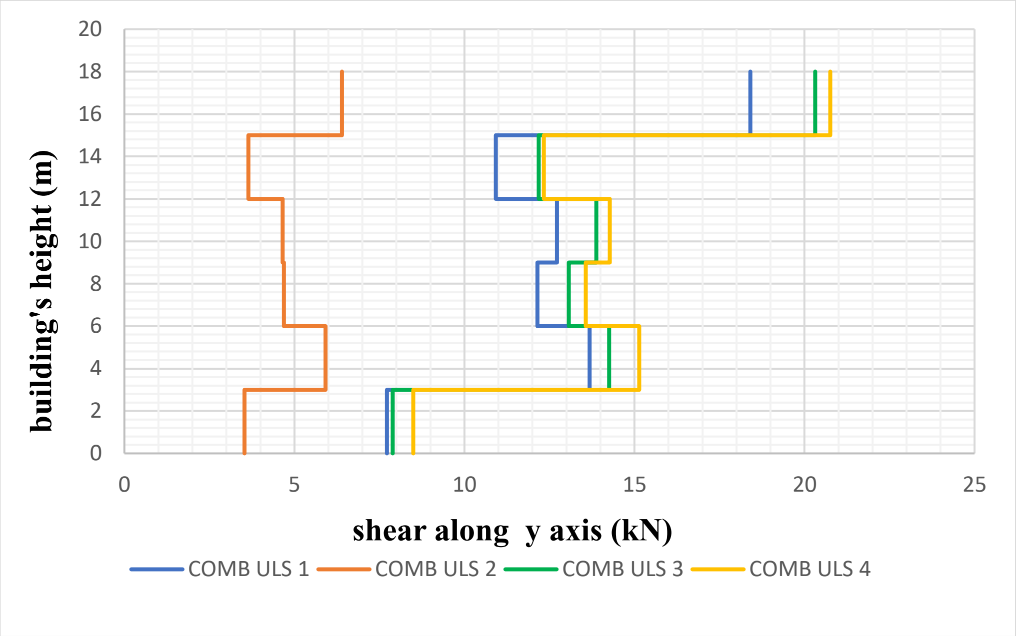


Figure 3.33. Shear force along the y-axis

The envelope curves of these solicitations are obtained and presented in the figure 3.34 and figure 3.35 respectively.

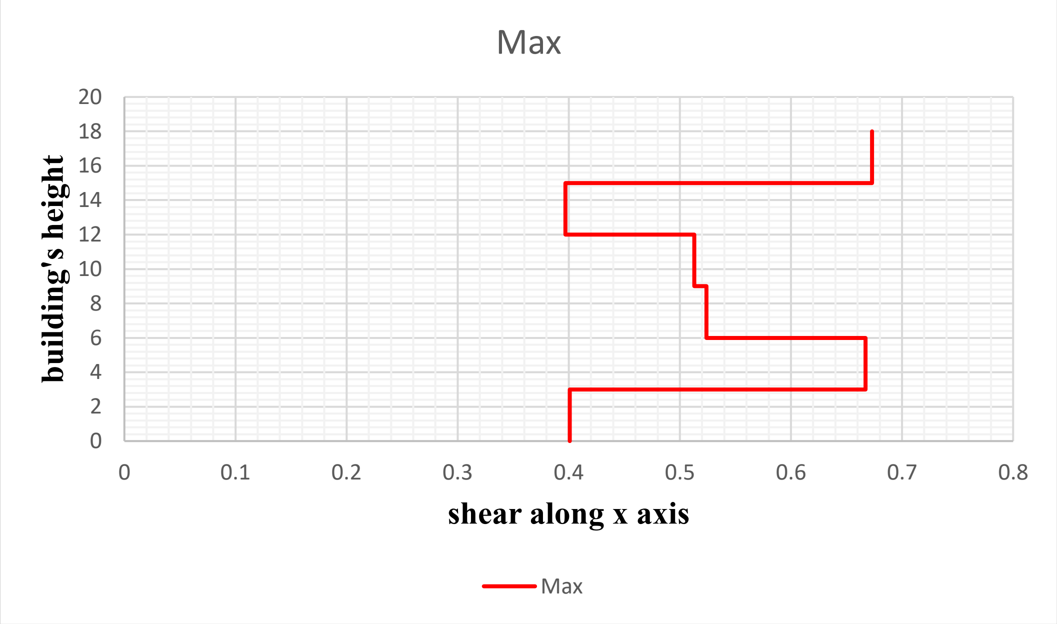


Figure 3.34. Shear force envelope curve on the column in the x-direction

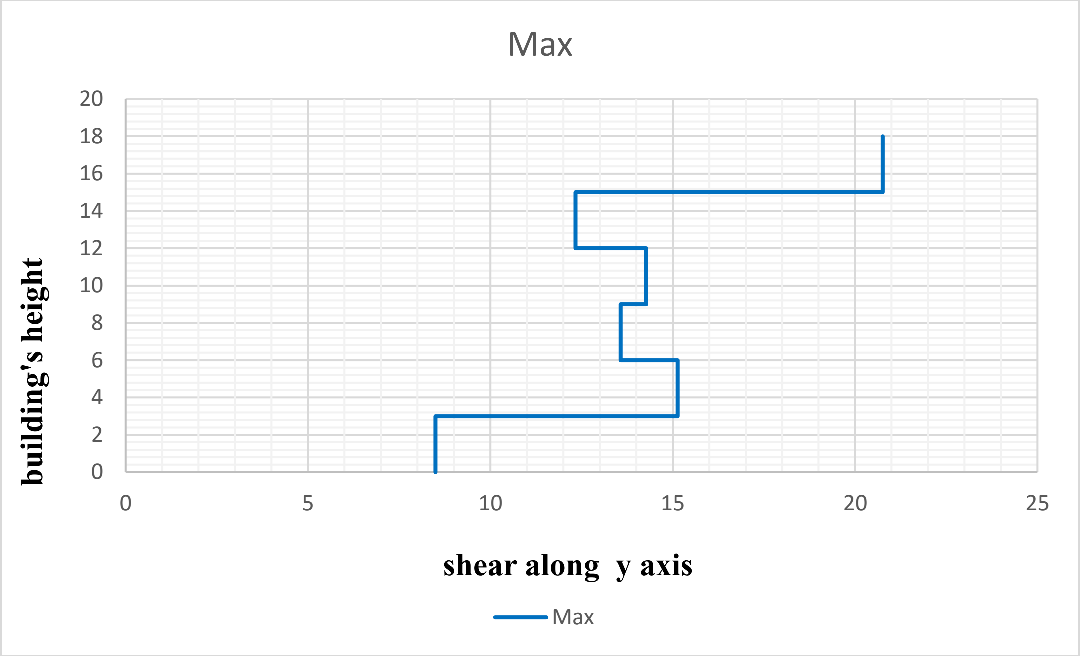


Figure 3.35. Shear force envelope curve on the column in the y- direction

Applying the procedure presented in the section 2.7.3.2.b, it is noticed that the shear resistance of the section without shear reinforcement is greater than the maximum shear solicitation on the column so the detailing of members has to be applied to have the spacing. In this case, it is considered a diameter of 8 mm and the maximum spacing of the transverse reinforcement is given by:

$$s_{cl,tmax} = \min(0.6 \times 20 \times 28\,;\ 0.6 \times 500\,;\ 0.6 \times 400\,)$$
$$= \min(336mm; 300mm; 240mm)$$

$$s_{cl,tmax} = 240mm$$

So, applying the prescriptions of the section 2.7.3.2.b, one can obtain a spacing of the shear reinforcement of: 20 cm within 0.55 cm above and below the beams and 40 cm for the rest of the column.

3.5.4. Foundation design

As earlier mention in the previous chapter in section 2.7.3.3, the result of the design of the foundation was done assuming the fact that the load was submitted only to the axial load transmitted from the column to the spread footing. By this assumption, the following section details the design of the footing.

3.5.4.1. Footing's size

The loads are gotten directly from the column that is:

$$N_{Ed} = 832.677kN$$

Since the dimensions of the footing are unknown the assumed footing weight is 10% of the load from the column so therefore:

$$W = 0.1 * 832.677 = 83.2677kN$$

Therefore, the total load subjected to the footing is

$$N_{Ed,footing} = N_{Ed} + W = 915.9447kN$$

From this load, it is then possible to determine the section of the footing following the procedure from section 2.7.3.3 as follows:

$$B_0 = \sqrt{\frac{N_{Ed}}{\sigma_{Rd}}} = 1914.1mm$$

So therefore, the section of the footing to be taken into consideration (taking a square footing) is: $\boldsymbol{B}_0 = B_x = B_y = 2.0m$

3.5.4.2. Footing's depth

As earlier seen in section 2.7.3.3, the depth of the footing is gotten as follows:

$$h = \frac{B - b}{4} + 0.05m = \frac{2.0 - 0.5}{4} + 0.05 = 0.425m$$

so therefore, the depth taken for the design of the footing is 50cm. the footing is rigid therefore no moment acts on the footing.

3.5.4.3. Reinforcement for the footing

The area required for the reinforcement is gotten as follows:

$$A_s = \frac{N_{Ed,footing}\,(B-b)}{8.\,d\,f_{yd}^{*}} = \frac{915.9447 * 10^3 * (2000-500)}{8 * 450 * 0.85 * 391} = 1148.32mm^2$$

Considering 6 bars of diameter 16mm that is 6ϕ16, the new area provided is $A_{s,provided} = 1206.372mm^2$.

3.5.4.4. Bar spacing

According to Eurocode 2,

$$s_{b,min} = \max\left(k_1 d_b\,; k_2 + d_{ag}; 20\,mm\right) = \max(16; 30; 20) = 30mm$$

Where $k_1 = 1$, $k_2 = 5$mm, d_b is the bar's diameter and the d_{ag} maximum aggregate's size

$$s_{max} = min(20d_b; B_{small}; 450)$$

Where B_{small} is the shorter side of column

So $s_{max} = 320$mm

The spacing to be considered is therefore:

$$s = \frac{B_x - 2d'}{n_b - 1} = \frac{2000 - 2 * 50}{6-1} = 380mm \geq s_{max} = 320mm$$

So therefore, the spacing considered is s = 320mm

3.5.4.5. Anchorage length

From section 2.7.3.3, the anchorage length is determined by:

$$l_0 = l_{b,requuired} \geq max\{15 \times bar\ diameter; 200mm\} = \max\{240mm; 200mm\}$$

$$l_0 = 240mm$$

Therefore, an anchorage length of 240mm is taken into consideration.

3.5.5. Modal analysis

The modal analysis is a method that is able to determine the dynamic characteristics of a structure. The structure vibrates with his high magnitude at its resonance frequency. It is therefore important to determine the modal parameters that are the natural frequencies and the vibration mode. Indeed, from the modal analysis, it’s possible to know the deformation trends of a structure under a certain natural frequency in order to strengthen the weaker part of the structure firstly if the structure need to be strengthened.

The modal analysis of the structure permits to obtain the first twelve vibration modes with the periods and the frequencies presented in table 3.9.

Table 3.9. modal participating mass ratio and period of vibration

TABLE: Modal Participating Mass Ratios						
OutputCase	**StepType**	**StepNum**	**StepNum**	**SumUX**	**SumUY**	**SumUZ**
Text	Text	Unitless	Sec	Unitless	Unitless	Unitless
MODAL	Mode	1	0,63091	0,21104	0,42042	1,734E-08
MODAL	Mode	2	0,62492	0,76732	0,6443	1,963E-07
MODAL	Mode	3	0,56544	0,78928	0,78722	2,275E-07
MODAL	Mode	4	0,19483	0,88162	0,79498	1,262E-06
MODAL	Mode	5	0,19449	0,89621	0,87544	1,867E-06
MODAL	Mode	6	0,17255	0,89849	0,90211	1,903E-06
MODAL	Mode	7	0,10772	0,93476	0,90617	2,671E-06
MODAL	Mode	8	0,10356	0,93957	0,93515	2,675E-06
MODAL	Mode	9	0,09449	0,9398	0,94437	5,876E-06
MODAL	Mode	10	0,07778	0,95257	0,9463	9,903E-06
MODAL	Mode	11	0,06974	0,95966	0,96234	0,00002215
MODAL	Mode	12	0,06524	0,96636	0,96719	0,00003203

3.5.6. Soil structure interaction

The soil-structure interaction analysis (figure 3.36) is the method of evaluating the collective response of the three linked systems mentioned below for a specified ground motion.

- The structure
- The Foundation
- The underlying soil

It can therefore be defined as the process in which the response of the soil influences the motion of the structure and the motion of the structure influences the response of the soil.

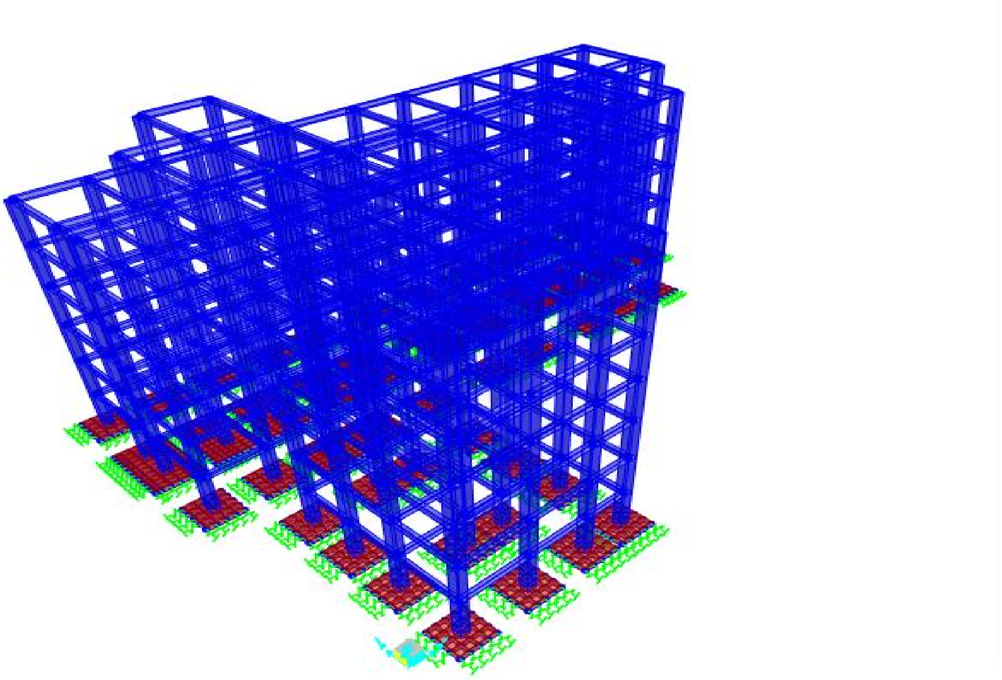


Figure 3.36. Soil structure interaction of the building

From the results obtained by the isolated footing, the structure is designed using distributed springs as shown in figure 3.36. The parameters taken into consideration is as shown in table 3.10

Table 3.10. Parameters for the soil structure interaction

Soil type	Elastic modulus of soil
B	110000 kN/m^2

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 3.5. The modulus of subgrade reaction is obtained using the figure presented in annex A table A5.

$$k = C \times A \qquad (3.5)$$

Where:

C: is the modulus of subgrade reaction of the soil

A: is the mesh area

The modulus of subgrade reaction representing the spring stiffness along Z is obtained taken the meshing area equal to 0.5 m x 0.5 m as

$$k = C \times A = 27500 kN/m$$

The soil pressure's result as shown in figure 3.37 below explains the fact that the maximum soil stress(pressure) having a value of 247 kN/m^2 is less than that of the admissible soil pressure of 0.25 MPa (250 kN/m^2) which means that the footing design for this building is accepted for the soil structure interaction.

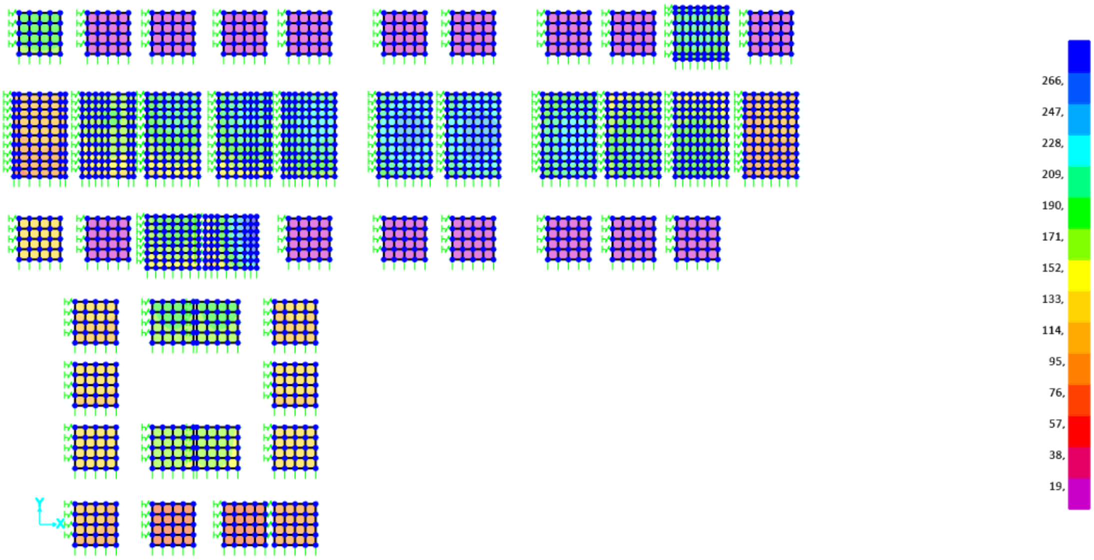


Figure 3.37. Soil pressure diagram

After placing the distributed springs, the first three modes of vibrations obtained with their periods were

- First mode of vibration translation in the y direction with a period, T = 0.705415 s
- Second mode of vibration translation in the x direction with a period, T = 0.631725 s
- The third mode is torsional with a period, T = 0.612676 s.

It is noticed that the natural periods of the structure have increased compared to the corresponding rigidly supported structure.

3.5.7. Non-linear static analysis (pushover analysis)

The objective of this analysis is to verify the deformation of the structure when subjected to a force. It could be seismic or any lateral force applied to the structure. A comparative study of the pushover analysis is performed by using the rigid fixed support at the foundation and using the distributed spring support that is using soil structure interaction on the building. These comparative studies are done under two main conditions

- The structural joints (links) in-between the three sections of the building are conserved and studied
- The structural joints do not exist on the three section of the building and still studied

3.5.7.1. Case where the structural joints are conserved

The results of the pushover analysis are then represented by the diagrams as shown in figures 3.38 and 3.39 for the pushover analysis for both the x-axis and y-axis obtained from SAP2000

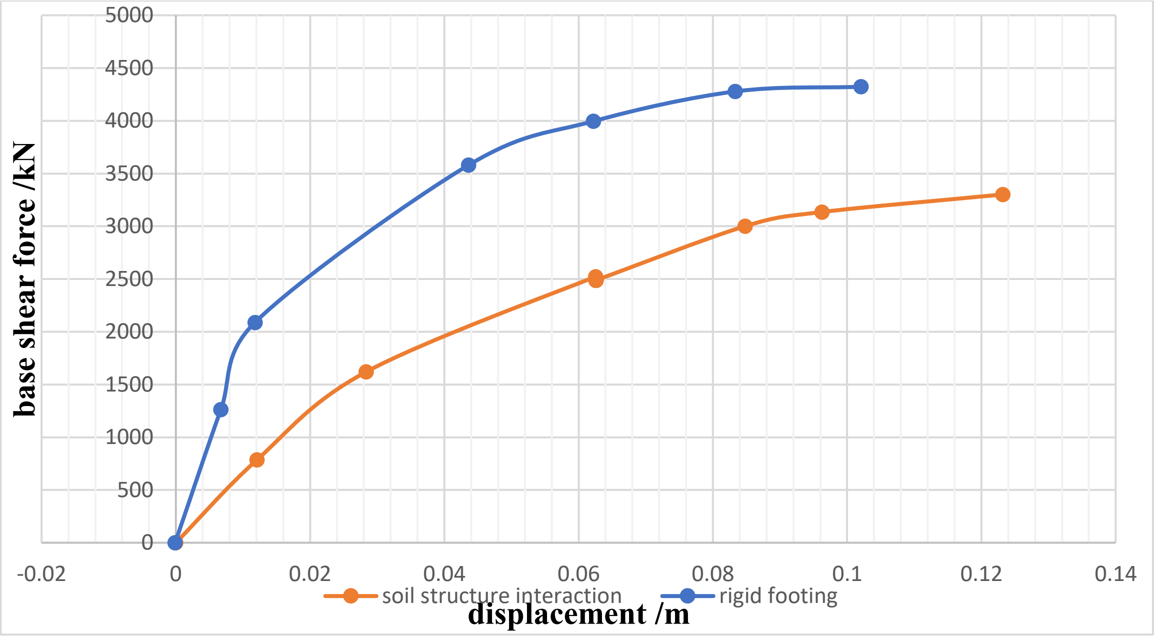


Figure 3.38. Pushover curves for the structure with structural joints in the x- direction

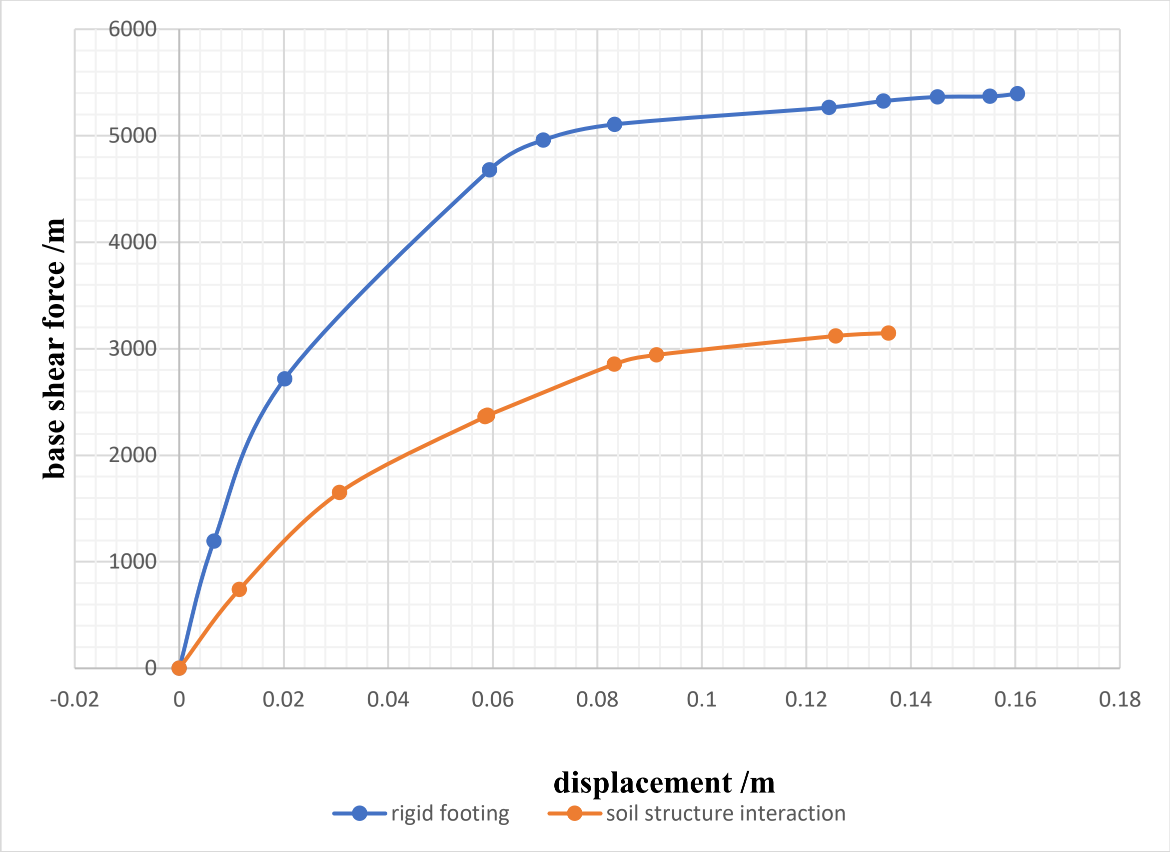


Figure 3.39. Pushover curve for the structure with structural joints in the y-direction

It is noticed that the pushover curve of the fixed support is greater than that of the soil structure interaction. These results prove that using distributed spring foundation is a better technique to keep the structure from falling down.

3.5.7.2. Case where there are no structural joints between the sections of the structures

Following the similar proceedings as done for the case with the building with rupture joints, the following curves in figures 3.39 and 3.40 details the explanation done with the pushover analysis.

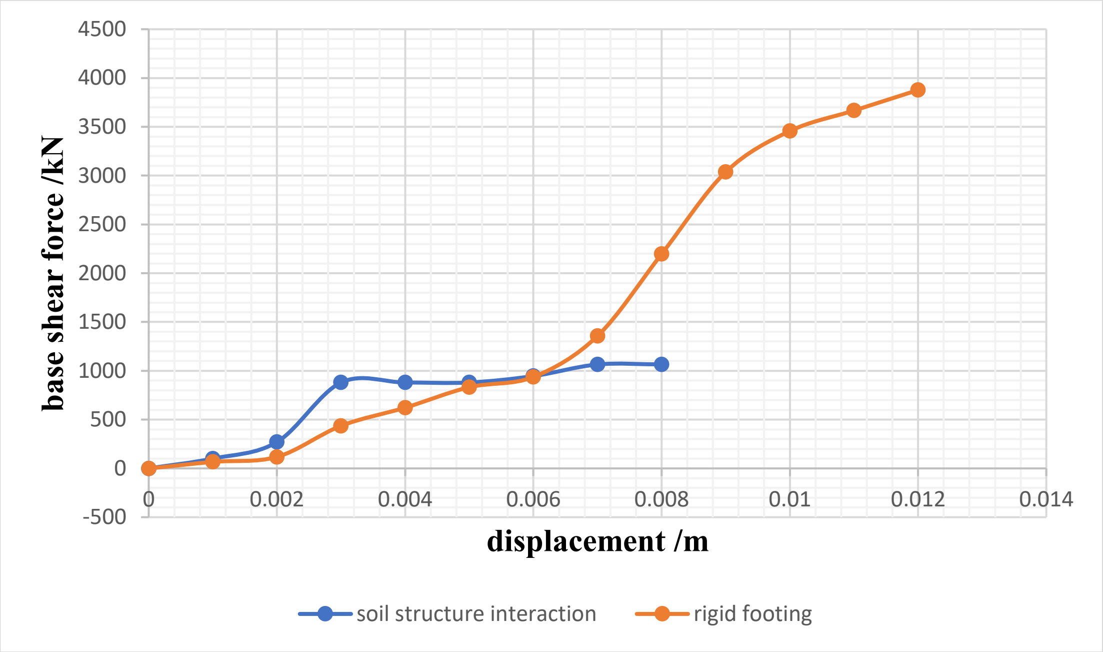


Figure 3.40. Pushover curve for the structure without structural joints in the x-direction

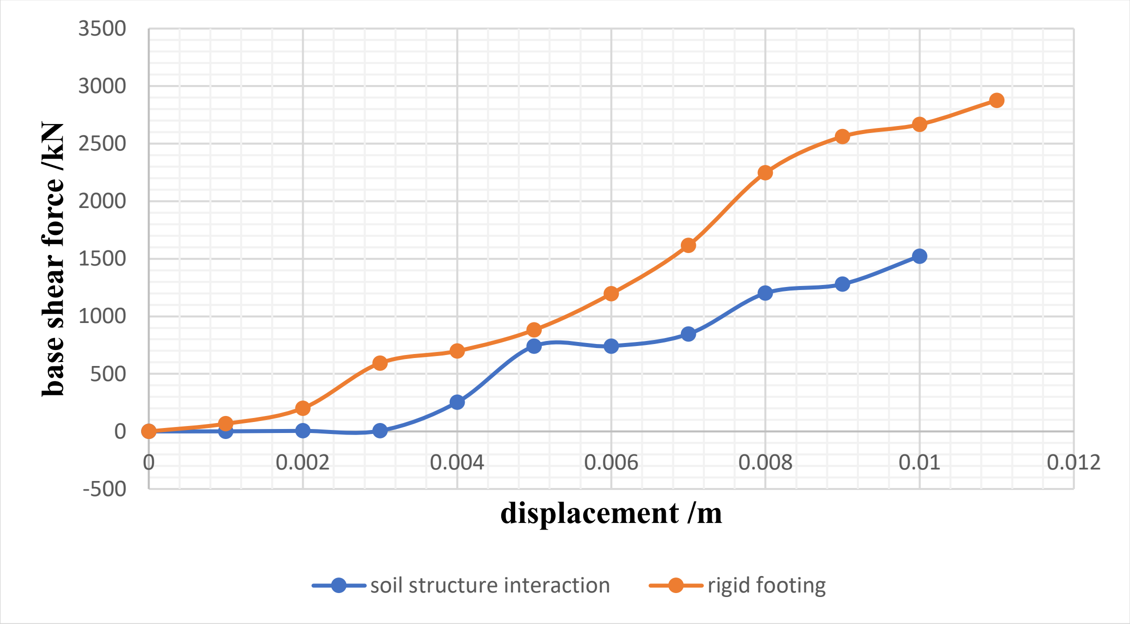


Figure 3.41. Pushover curve for the structure without structural joints in the y-direction

The above results show that there is no connection and no cohesion between the three parts of the building section. This leads to the conclusion that with structural joints, the irregular structure undergoes a uniform deformation.

Conclusion

The chapter's objective was to show the engineering design of the reinforced concrete building frame member such as the beam, the column and the footing of the foundation using the norms provided for this work that is the Eurocode norms. The result permitted one to obtain the beam section of 20cm by 40cm for the beam, 50cm by 50cm for the column and 200cm by 200cm for the footing. The modal analysis that was performed permitted one to obtain the modal participating mass ratio for the building. In addition to the modal analysis, a soil structure interaction was done on the building in order to obtain the soil pressure of the isolated rigid footings of the building. And finally, the pushover analysis was performed in order to check for the deformation on the building using the structural joint connections all over the three sections of the structure and using no joint connection on the building both under soil structure interaction.

GENERAL CONCLUSION

The main objective of this work was to design a reinforced concrete multi-storey building comprised of seven floors in such a way that human safety is ensured (under ULS). The building is under a zone of low seismicity. A brief knowledge of the literature review based on reinforced concrete, its constituent and its workability were made in the first part of the work.

The building was analysed and designed using a three-dimensional finite element model using SAP2000. This study also identified the most unfavourable combinations of actions so as to reduce the actions' calculation time. In parallel with the modal analysis study, a static study was carried out. This made it possible to analyse the distribution of charges and compare it with the results of the finite element method. A structure should be designed for load combination as the code provision for they have significant effect on the design of the structure.

At the end of this work, the section obtained for the different structural frames were the following: 20cm by 40cm for the beam which resists to both applied bending moment and shear force, 50cm by 50cm for the column that is the applied axial force-bending moment for each floor is less than the resisting axial force-bending moment hence no failure occurs. And for the isolated footing, a section of 200cm by 200cm was used for the building. For the modal analysis observed three modes of vibration with the first two modes translation along the y and x axis and the third mode torsion. The soil structure interaction permitted one to express the fact that using spring foundation, the building is under more safety than using the rigid foundation. The pushover analysis result also drove to the conclusion that the deformation obtained using spring foundation (soil structure interaction) is less than that of the rigid footing as represented on the base shear force-displacement diagram. It also emphasizes on the fact that structural joints are suitable and most recommended in irregular structures.

As difficulties, it took a longer period of time for the pushover analysis results to be displayed especially with the modelling of the hinges to be applied at the point of rupture.

REFERENCES

Books and Articles

A.J Bond, O. Brooker , A.J Harris, T. Harrison, R.M Moss, R.S Narayanan & R. Webster (2006). *How to Design Concrete Structures using Eurocode 2. The concrete centre*

Akash Malik (2019). Analysis and Design of Multistorey Building*, International Journal for Research in Science and Technology (IJIRST) Volume 5/ Issue 12 / May 2019 pp 81-86*

Al-Kodmanys., M. A. (2012, September). Tall Buildings and Urban Habitat of the 21st Century. A Global Perspective*, Open access Buildings pp.* 384-423.

Bill Mosley, J. B. (2007). *Reinforced concrete design to Eurocode 2. palgrave macmillan*

Bungale S. Taranath (2017) *Tall Building Design: Steel, Concrete, and Composite Systems*

Chanakya Arya (2009). *Design of Structural Elements. Spon Press*

Molem C., A. F. (2016). *Implications of Infrastructural Development on Cameroon's Economic Emergence, 7(4), Journal of Economics and Sustainable Development., pg. 102.*

Rangachari narayanan, K. V. (2011). *Multi-storey buildings. Institute for steel development $ growth* (INSDAG).

SEGHIR, A. (2010, Novembre 22). *Contribution à la modélisation numérique de la réponse sismique des ouvrages avec interaction sol-structure et interaction fluide-structure. Application à l'étude des barrages poids en béton.* Marne-la-Vallée, Est, Paris

SNU Geotechnical Engineering Lab (2018). *Geotechnical Engineering*.

Structure course (2015). *Design of structures.*

Standard norms

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

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

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

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

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

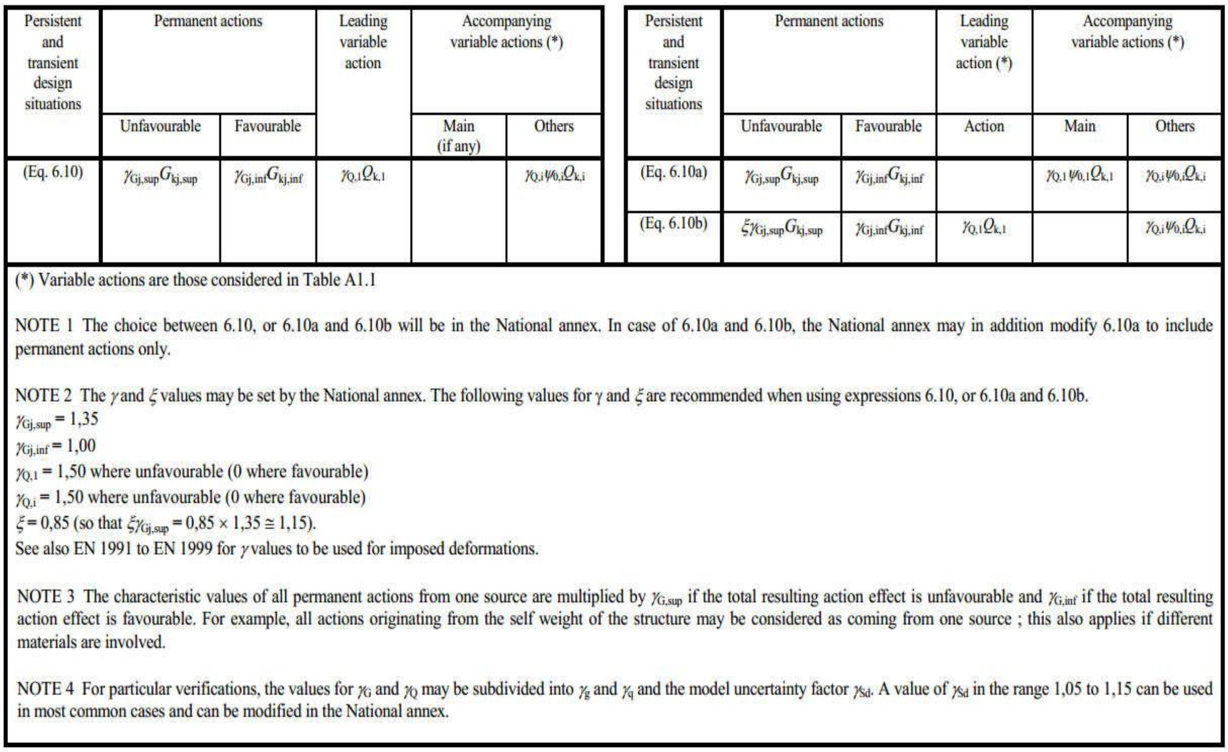
Other thesis

Wandji Nana, T. L. (2019). *Structural analysis and design of a G+10 reinforced concrete building. (Master thesis).* National Advanced School of Public Works Yaoundé, Cameroon.

ANNEXES

Annex A. Tables provided for the methodology

Table A1. Design values of actions (EC 0 Part 1)



Persistent and transient design situations	Permanent actions		Leading variable action	Accompanying variable actions (*)	
	Unfavourable	Favourable		Main (if any)	Others
(Eq. 6.10)	$\gamma_{Gj,sup}G_{kj,sup}$	$\gamma_{Gj,inf}G_{kj,inf}$	$\gamma_{Q,1}Q_{k,1}$		$\gamma_{Q,i}\psi_{0,i}Q_{k,i}$

Persistent and transient design situations	Permanent actions		Leading variable action (*)	Accompanying variable actions (*)	
	Unfavourable	Favourable	Action	Main	Others
(Eq. 6.10a)	$\gamma_{Gj,sup}G_{kj,sup}$	$\gamma_{Gj,inf}G_{kj,inf}$		$\gamma_{Q,1}\psi_{0,1}Q_{k,1}$	$\gamma_{Q,i}\psi_{0,i}Q_{k,i}$
(Eq. 6.10b)	$\xi\gamma_{Gj,sup}G_{kj,sup}$	$\gamma_{Gj,inf}G_{kj,inf}$	$\gamma_{Q,1}Q_{k,1}$		$\gamma_{Q,i}\psi_{0,i}Q_{k,i}$

(*) Variable actions are those considered in Table A1.1

NOTE 1 The choice between 6.10, or 6.10a and 6.10b will be in the National annex. In case of 6.10a and 6.10b, the National annex may in addition modify 6.10a to include permanent actions only.

NOTE 2 The γ and ξ values may be set by the National annex. The following values for γ and ξ are recommended when using expressions 6.10, or 6.10a and 6.10b.

$\gamma_{Gj,sup} = 1{,}35$

$\gamma_{Gj,inf} = 1{,}00$

$\gamma_{Q,1} = 1{,}50$ where unfavourable (0 where favourable)

$\gamma_{Q,i} = 1{,}50$ where unfavourable (0 where favourable)

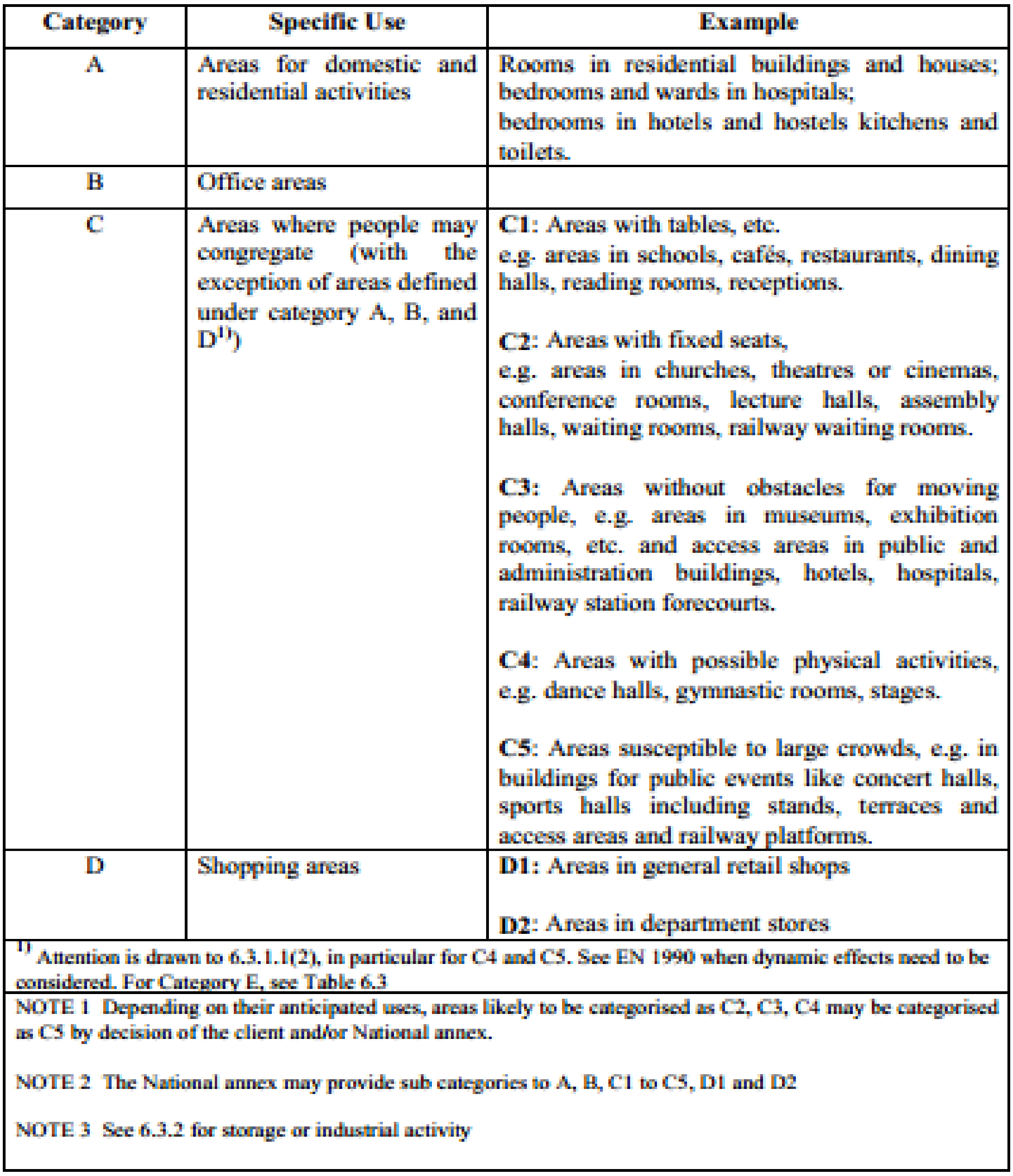
$\xi = 0{,}85$ (so that $\xi\gamma_{Gj,sup} = 0{,}85 \times 1{,}35 \cong 1{,}15$).

See also EN 1991 to EN 1999 for γ values to be used for imposed deformations.

NOTE 3 The characteristic values of all permanent actions from one source are multiplied by $\gamma_{G,sup}$ if the total resulting action effect is unfavourable and $\gamma_{G,inf}$ if the total resulting action effect is favourable. For example, all actions originating from the self weight of the structure may be considered as coming from one source ; this also applies if different materials are involved.

NOTE 4 For particular verifications, the values for γ_G and γ_Q may be subdivided into γ_g and γ_q and the model uncertainty factor γ_{Sd}. A value of γ_{Sd} in the range 1,05 to 1,15 can be used in most common cases and can be modified in the National annex.

Table A2. Categories of use of the building (EC 1 Part 1)



Category	Specific Use	Example
A	Areas for domestic and residential activities	Rooms in residential buildings and houses; bedrooms and wards in hospitals; bedrooms in hotels and hostels kitchens and toilets.
B	Office areas	
C	Areas where people may congregate (with the exception of areas defined under category A, B, and D[1])	C1: Areas with tables, etc. e.g. areas in schools, cafés, restaurants, dining halls, reading rooms, receptions. C2: Areas with fixed seats, e.g. areas in churches, theatres or cinemas, conference rooms, lecture halls, assembly halls, waiting rooms, railway waiting rooms. C3: Areas without obstacles for moving people, e.g. areas in museums, exhibition rooms, etc. and access areas in public and administration buildings, hotels, hospitals, railway station forecourts. C4: Areas with possible physical activities, e.g. dance halls, gymnastic rooms, stages. C5: Areas susceptible to large crowds, e.g. in buildings for public events like concert halls, sports halls including stands, terraces and access areas and railway platforms.
D	Shopping areas	D1: Areas in general retail shops D2: Areas in department stores

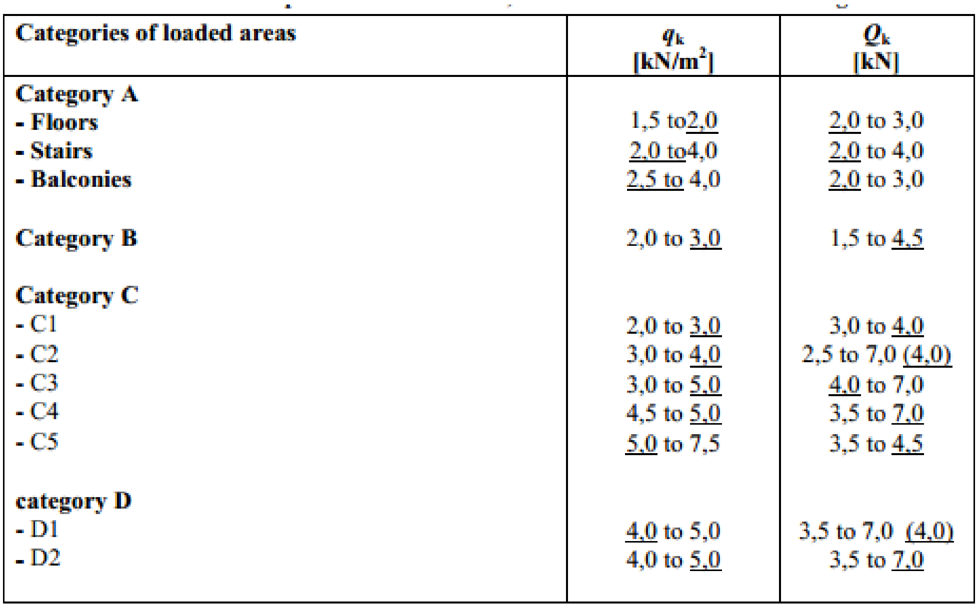
[1] Attention is drawn to 6.3.1.1(2), in particular for C4 and C5. See EN 1990 when dynamic effects need to be considered. For Category E, see Table 6.3

NOTE 1 Depending on their anticipated uses, areas likely to be categorised as C2, C3, C4 may be categorised as C5 by decision of the client and/or National annex.

NOTE 2 The National annex may provide sub categories to A, B, C1 to C5, D1 and D2

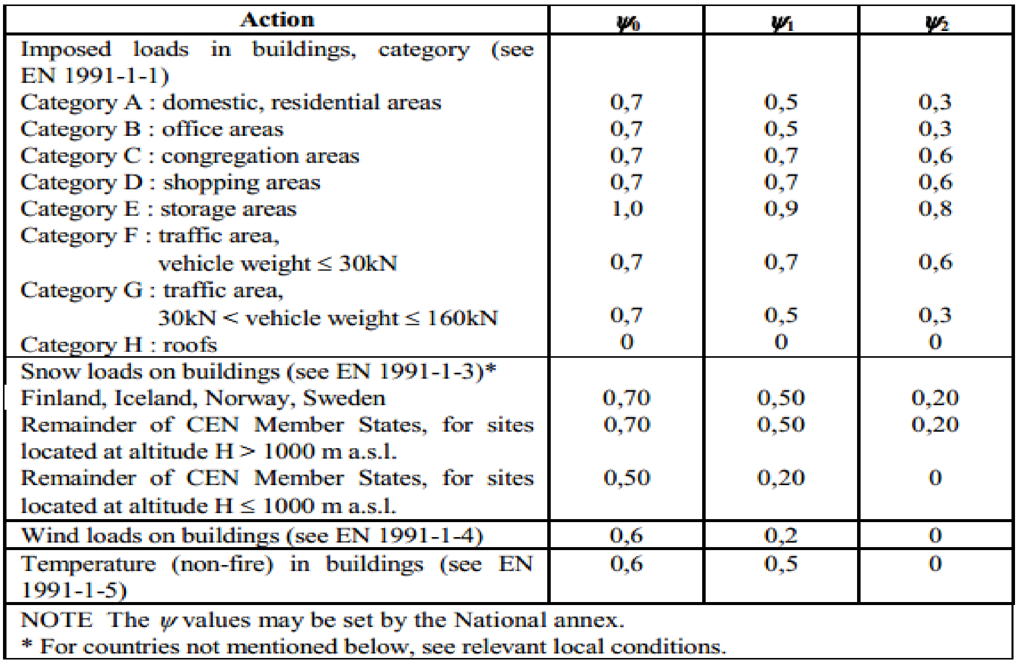
NOTE 3 See 6.3.2 for storage or industrial activity

Table A3. Imposed loads on floors, balconies and stairs in buildings (EC 1 Part 1)



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

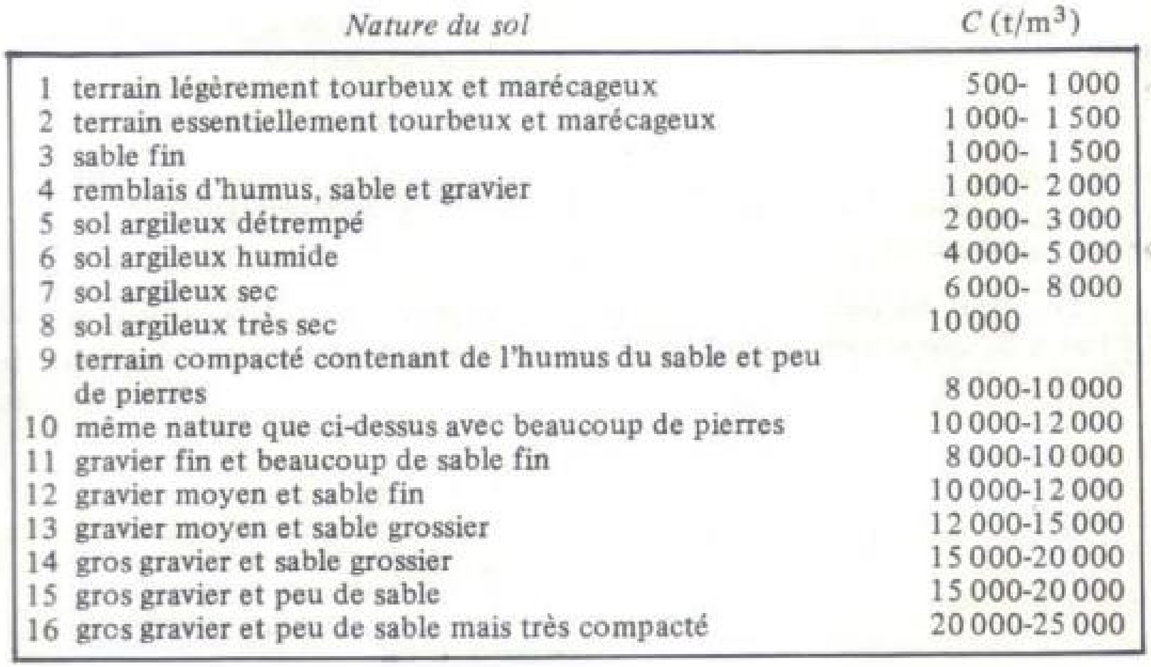
Table A4. Recommended values of Ψ factors for buildings (EC 0 Part 1)



Action	ψ_0	ψ_1	ψ_2
Imposed loads in buildings, category (see EN 1991-1-1)			
Category A : domestic, residential areas	0,7	0,5	0,3
Category B : office areas	0,7	0,5	0,3
Category C : congregation areas	0,7	0,7	0,6
Category D : shopping areas	0,7	0,7	0,6
Category E : storage areas	1,0	0,9	0,8
Category F : traffic area, vehicle weight ≤ 30kN	0,7	0,7	0,6
Category G : traffic area, 30kN < vehicle weight ≤ 160kN	0,7	0,5	0,3
Category H : roofs	0	0	0
Snow loads on buildings (see EN 1991-1-3)*			
Finland, Iceland, Norway, Sweden	0,70	0,50	0,20
Remainder of CEN Member States, for sites located at altitude H > 1000 m a.s.l.	0,70	0,50	0,20
Remainder of CEN Member States, for sites located at altitude H ≤ 1000 m a.s.l.	0,50	0,20	0
Wind loads on buildings (see EN 1991-1-4)	0,6	0,2	0
Temperature (non-fire) in buildings (see EN 1991-1-5)	0,6	0,5	0

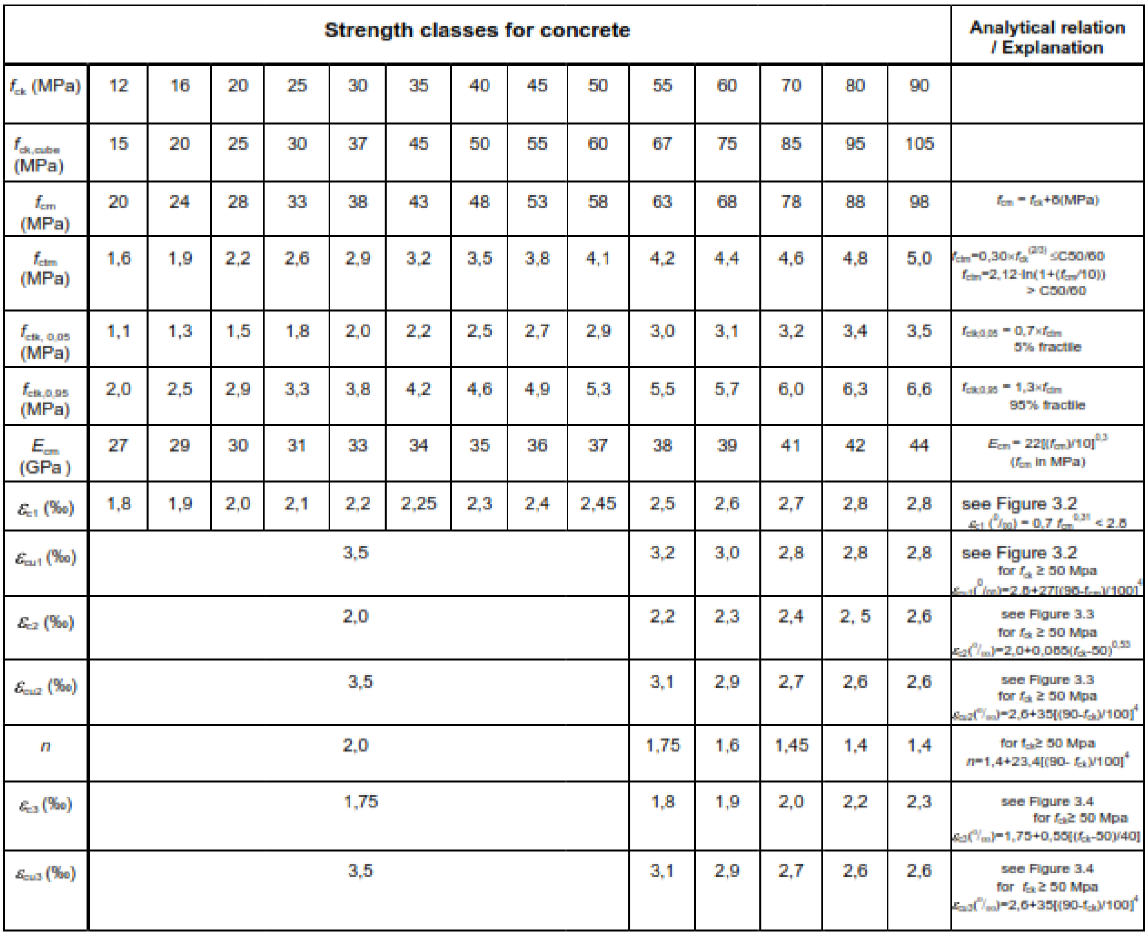
NOTE The ψ values may be set by the National annex.
* For countries not mentioned below, see relevant local conditions.

Table A5. Values of subgrade modulus for different soil types (Forni, sd)



	Nature du sol	C (t/m³)
1	terrain légèrement tourbeux et marécageux	500- 1 000
2	terrain essentiellement tourbeux et marécageux	1 000- 1 500
3	sable fin	1 000- 1 500
4	remblais d'humus, sable et gravier	1 000- 2 000
5	sol argileux détrempé	2 000- 3 000
6	sol argileux humide	4 000- 5 000
7	sol argileux sec	6 000- 8 000
8	sol argileux très sec	10 000
9	terrain compacté contenant de l'humus du sable et peu de pierres	8 000-10 000
10	même nature que ci-dessus avec beaucoup de pierres	10 000-12 000
11	gravier fin et beaucoup de sable fin	8 000-10 000
12	gravier moyen et sable fin	10 000-12 000
13	gravier moyen et sable grossier	12 000-15 000
14	gros gravier et sable grossier	15 000-20 000
15	gros gravier et peu de sable	15 000-20 000
16	gros gravier et peu de sable mais très compacté	20 000-25 000

Table A6. Strength classes for concrete (EC2 part 1)



Strength classes for concrete															Analytical relation / Explanation
f_{ck} (MPa)	12	16	20	25	30	35	40	45	50	55	60	70	80	90	
$f_{ck,cube}$ (MPa)	15	20	25	30	37	45	50	55	60	67	75	85	95	105	
f_{cm} (MPa)	20	24	28	33	38	43	48	53	58	63	68	78	88	98	$f_{cm} = f_{ck}+8$(MPa)
f_{ctm} (MPa)	1,6	1,9	2,2	2,6	2,9	3,2	3,5	3,8	4,1	4,2	4,4	4,6	4,8	5,0	$f_{ctm}=0,30\times f_{ck}^{(2/3)}$ ≤C50/60 $f_{ctm}=2,12\cdot\ln(1+(f_{cm}/10))$ > C50/60
$f_{ctk,\ 0,05}$ (MPa)	1,1	1,3	1,5	1,8	2,0	2,2	2,5	2,7	2,9	3,0	3,1	3,2	3,4	3,5	$f_{ctk;0,05} = 0,7\times f_{ctm}$ 5% fractile
$f_{ctk,0,95}$ (MPa)	2,0	2,5	2,9	3,3	3,8	4,2	4,6	4,9	5,3	5,5	5,7	6,0	6,3	6,6	$f_{ctk;0,95} = 1,3\times f_{ctm}$ 95% fractile
E_{cm} (GPa)	27	29	30	31	33	34	35	36	37	38	39	41	42	44	$E_{cm} = 22[(f_{cm})/10]^{0,3}$ (f_{cm} in MPa)
ε_{c1} (‰)	1,8	1,9	2,0	2,1	2,2	2,25	2,3	2,4	2,45	2,5	2,6	2,7	2,8	2,8	see Figure 3.2 $\varepsilon_{c1}(^0/_{00}) = 0,7\ f_{cm}^{0,31} < 2.8$
ε_{cu1} (‰)	3,5									3,2	3,0	2,8	2,8	2,8	see Figure 3.2 for $f_{ck} \geq 50$ Mpa $\varepsilon_{cu1}(^0/_{00})=2,8+27[(98-f_{cm})/100]^4$
ε_{c2} (‰)	2,0									2,2	2,3	2,4	2, 5	2,6	see Figure 3.3 for $f_{ck} \geq 50$ Mpa $\varepsilon_{c2}(^0/_{00})=2,0+0,085(f_{ck}-50)^{0,53}$
ε_{cu2} (‰)	3,5									3,1	2,9	2,7	2,6	2,6	see Figure 3.3 for $f_{ck} \geq 50$ Mpa $\varepsilon_{cu2}(^0/_{00})=2,6+35[(90-f_{ck})/100]^4$
n	2,0									1,75	1,6	1,45	1,4	1,4	for $f_{ck} \geq 50$ Mpa $n=1,4+23,4[(90-f_{ck})/100]^4$
ε_{c3} (‰)	1,75									1,8	1,9	2,0	2,2	2,3	see Figure 3.4 for $f_{ck} \geq 50$ Mpa $\varepsilon_{c3}(^0/_{00})=1,75+0,55[(f_{ck}-50)/40]$
ε_{cu3} (‰)	3,5									3,1	2,9	2,7	2,6	2,6	see Figure 3.4 for $f_{ck} \geq 50$ Mpa $\varepsilon_{cu3}(^0/_{00})=2,6+35[(90-f_{ck})/100]^4$

Table A7. ground types (EC8 part 1)

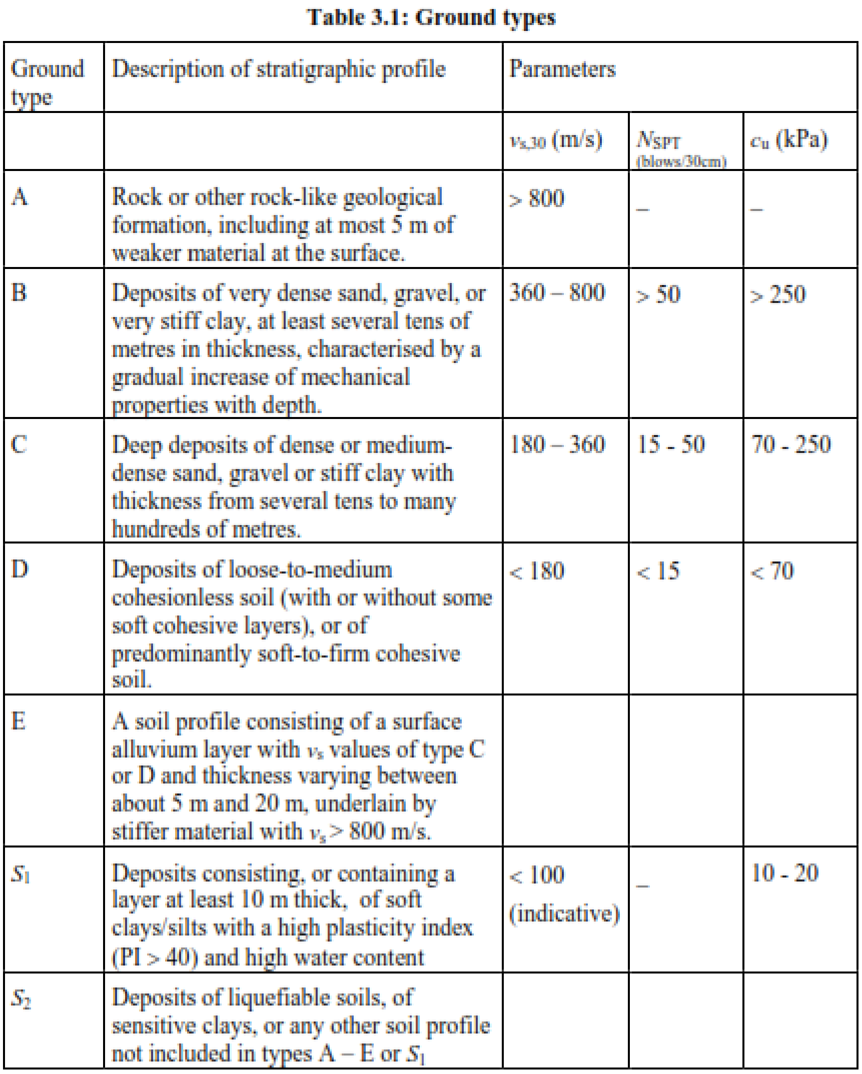


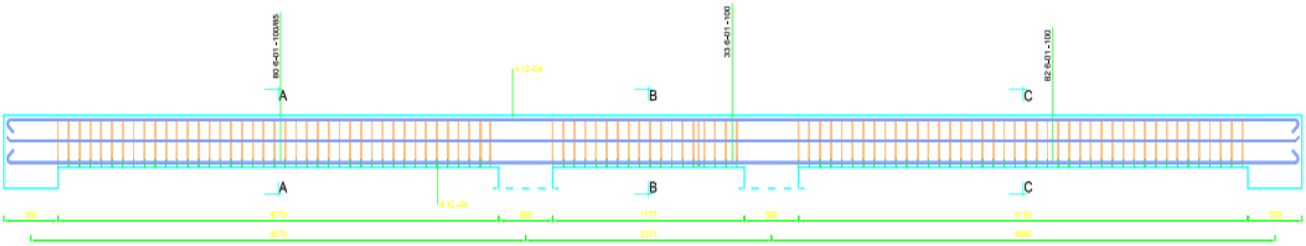
Table 3.1: Ground types

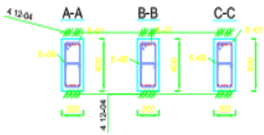
Ground type	Description of stratigraphic profile	Parameters		
		$v_{s,30}$ (m/s)	N_{SPT} (blows/30cm)	c_u (kPa)
A	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.	> 800	_	_
B	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of metres in thickness, characterised by a gradual increase of mechanical properties with depth.	360 – 800	> 50	> 250
C	Deep deposits of dense or medium-dense sand, gravel or stiff clay with thickness from several tens to many hundreds of metres.	180 – 360	15 - 50	70 - 250
D	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.	< 180	< 15	< 70
E	A soil profile consisting of a surface alluvium layer with v_s values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with v_s > 800 m/s.			
S_1	Deposits consisting, or containing a layer at least 10 m thick, of soft clays/silts with a high plasticity index (PI > 40) and high water content	< 100 (indicative)	_	10 - 20
S_2	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types A – E or S_1			

.

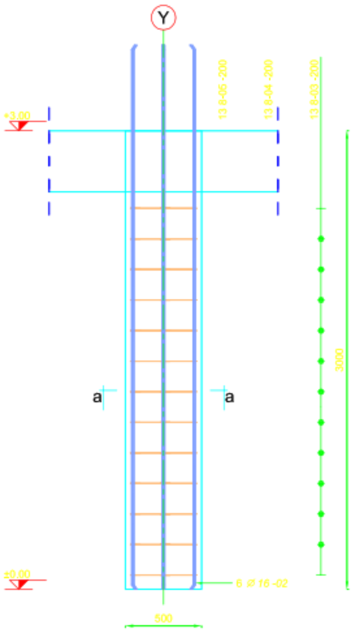
Annex B. Results implementations

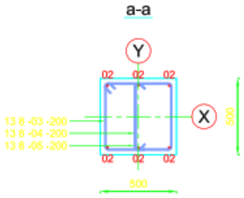
Annex B1. Detailing for the beam





Annex B2. Detailing for the column





Annex B3. Detailing for the footing