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INFLUENCE OF RIGID AND NON-RIGID SLABS ON THE STRUCTURAL BEHAVIOUR OF TALL BUILDINGS WITH IRREGULAR PLAN: CASE OF A CONCEIVED L-SHAPE BUILDING

Thesis submitted to the partial fulfilment of the requirements for the degree of Masters of Engineering (MEng) in Civil Engineering

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Presented by:
NGAKAM TOUKO ALPHONSINE VANNELLE
Student number: **15TP21070**

Supervised by:
Pr. Carmelo MAJORANA

Co-supervised by:

Eng. Giuseppe CARDILLO

Dr.Eng Guillaume Hervé POH'SIE

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DEDICATION

TO MY MOTHER

DJOUMBISSIE VICTOIRE ALLIANCE

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GLOSSARY

LIST OF ABBREVIATIONS

CM	Centre Of Mass
CR	Centre Of Rigidity
EC	Eurocode
FEMA	Federal Emergency Management Agency
RC	Reinforced Concrete
SAP	Serviceability Limit State
SLS	Structural Analysis Program
SSI	Soil Structure Interaction
ULS	Ultimate Limit State

LIST OF SYMBOLS

$\mathbf{A}$	Area of the cross section
$\mathbf{A_c}$	Area of the concrete cross section
$\mathbf{A_s}$	Area of the lower steel reinforcement section
$\mathbf{A_s'}$	area of upper steel reinforcement section
$\mathbf{A_{s,max}}$	Maximum steel reinforcement section area
$\mathbf{A_{s,min}}$	Minimum steel reinforcement section area
$\mathbf{A_{sw}}$	Cross sectional area of the shear reinforcement
$\mathbf{D}$	Plate rigidity
$\mathbf{E}$	Youngs modulus
$\mathbf{F_{ed,sup}}$	Design support reaction
$\mathbf{G_{1k}}$	Structural load of the building
$\mathbf{G_{2k}}$	Non-structural load apply on the building
$\mathbf{J_{cr}}$	Moment of inertia of the uncracked section
$\mathbf{M_{ED}}$	Soliciting bending moment
$\mathbf{M_{RD}}$	Resisting moment
$\mathbf{Q_k}$	imposed load
$\mathbf{S_e}$	Elastic response spectrum
$\mathbf{T}$	Period of the structure with fixed base

T_B	Lower limit of the period of the constant spectral acceleration branch
T_C	Upper limit of the period of the constant spectral acceleration branch
T_D	Value defining the beginning the constant displacement response range of the spectrum
T_k	period of vibration of mode k
V	Reduction factor
V_{ED}	design shear value
V_{Rd}	resisting shear
V_{rdc}	Shear capacity of concrete
$\Delta C_{dur,add}$ *dd*	Add reduction of minimum cover for use of additional protection
$\Delta C_{dur,st}$	Reduction of minimum cover for use of stainless steel
$\Delta C_{dur,v}$	Additive safety element
α	Angle between shear reinforcement and axis of design element
α_{cw}	coefficient of interaction between compressive stresses
a_g	Design ground acceleration
a_i	Shifting distance
α_x	grashof coefficient in x direction
α_y	grashof coefficient in y direction
b	Width of the element
b_t	Mean width of the tension zone
b_w	Smallest width of the cross section in the tensile area
cc	concrete cover
cc	Concrete cover
c_{min}	Minimum concrete cover
$c_{min,b}$	Minimum cover due to bond requirement
$c_{min,dur}$	Minimum cover due to environmental conditions
d	Effective height of the section
d_r	Design inter-storey drift
f_{cd}	Design resisting strength of the concrete
f_{ctm}	Tensile strength of the concrete
ϕ_{lmin}	Minimum diameter of the longitudinal bars

f_{yd}	Design yielding strength of the steel
f_{yk}	Characteristic yield strength of steel
f_{ywd}	Design yield strength of the shear reinforcement
η	correction factor
h	Structure height or the effective structure height
h	storey height
h_i	Height of the level i
i	Gyration radius of the uncracked concrete section
j	Index denoting the mode of vibration
φ_{ef}	Effective creep ratio
k	Stiffness of the fixed base structure
k	number of modes taken into account
l	is the span length of the beam
l_0	Effective length of the element
λ_{lim}	Limit value of slenderness
m	Mass of the structure
n	relative normal force
n_∞	long-term coefficient of homogenization
n_0	Short-term coefficient of homogenization
q	Behaviour factor
$\rho 1$	Shear reinforcement ratio
r_m	moment ratio
s	stirrup spacing
S	Soil factor
σ_c	stress in concrete
S_{clx}	Maximum longitudinal spacing
σ_{cp}	stress due to axial forces
Sr	recovery area of column
σ_s	stress in steel
t	breath at support
v1	reduction coefficient of shear crack

ω	mechanical reinforcement ratio
x	neutral axis
ξ	viscous damping ratio
z	inner lever arm
γ	Specific weight
γ_c	Partial factor for concrete
γ_s	Partial safety factor for steel
ν	Poisson’s ratio
Ψ_E	Combination coefficient for variable action
θ	Angle of concrete compression struts to the beam axis
λ	Slenderness

ABSTRACT

This thesis is aimed at analysing the influence of rigid and non-rigid slab on the structural behaviour of tall buildings with irregular plan. In order to achieve this objective, the behaviour of the structure was evaluated when modelled with a rigid slab and with a non-rigid slab for fixed base and raft foundation. The literature review exposed gives an overview of slabs type, slab rigidity, different types of irregularities and tall buildings detailing on the specificity of tall buildings. The methodology used consist of defining the procedures, loading and analysis with the European standard. The case study which is an L-shape G+5 residential building was design statically. With the use of the software SAP2000 (Structural Analysis Program), the building was modelled with a rigid slab by applying diaphragms constraints at joints and a non-rigid slab by applying no constraints at joints. All the modelled structures were analyse dynamically via the application of a well-defined response spectrum and compared using their gravity load distribution, periods, lateral displacement inter-storey drift and base shear. Aftereffect, under gravity loads, the raft foundation model showed a reduction in axial force compared to the fixed base model. Moreover, under the aforementioned condition, the flexible slab models distribute axial forces in beams compared the rigid slab model which does not distribute axial forces in beams. The results shown than first three periods for the two models were approximately the same for the first three modes and changes consequently from the forth mode but the mass participating ratio as prescribed by Eurocode (at least 90%) for the rigid slab model is attained in the fourth mode for the fixed the fixed base and tenth mode for the raft foundation model compared to the non-rigid slab model which is attained in the tenth mode for the fixed base model and thirty two mode for the raft foundation model in the x direction. As for the lateral displacement and inter storey drift, the rigid slab models tends to have a high lateral displacement and inter-storey drift compared to the non-rigid (flexible) slab model with higher values in the raft foundation model. Also, a great variation of the base shear way more observed in the y direction with the rigid slab model having a greater base reaction compared to the flexible slab.

Keywords: rigid slab, non-rigid slab, diaphragm, response spectrum, raft foundation, reduction of axial force.

RESUME

Ce mémoire a pour but d'analyser l'influence des dalles rigides et non-rigides sur le comportement structurel des bâtiments de grande hauteur à plan irrégulier. Afin d'atteindre cet objectif, le comportement de la structure a été évalué lorsqu'elle est modélisée avec une dalle rigide et avec une dalle non-rigide pour une base fixe et une fondation sur radier. La revue de la littérature exposée donne un aperçu des types de dalles, la rigidité des dalles, des différents types d'irrégularités et des bâtiments de grande taille en détaillant la spécificité des bâtiments de grande taille. La méthodologie utilisée consiste à définir les procédures, le chargement et l'analyse avec la norme européenne. L'étude de cas, qui est un bâtiment résidentiel R+5 en forme de L, a été conçue de manière statique. Avec l'utilisation du logiciel SAP2000 (Structural Analysis Program), le bâtiment a été modélisé avec une dalle rigide en appliquant des contraintes de diaphragmes aux joints et une dalle non-rigide en n'appliquant aucune contrainte aux joints. Toutes les structures modélisées ont été analysées dynamiquement par l'application d'un spectre de réponse bien défini et comparées en utilisant la distribution des charges de gravité, les périodes, le déplacement latéral, la dérive inter-étage et le cisaillement à la base. En effet, sous les charges de gravité, le modèle de fondation sur radier a montré une réduction de la force axiale par rapport au modèle à base fixe. Les modèles de dalles flexibles distribuent les forces axiales dans les poutres par rapport au modèle de dalles rigides qui ne distribue pas les forces axiales dans les poutres sous charge de gravité. Les résultats montrent que les trois premières périodes pour les deux modèles sont approximativement les mêmes pour les trois premiers modes et changent en conséquence à partir du quatrième mode mais le rapport de participation de la masse tel que prescrit par la norme européenne (au moins 90%) est rapidement atteint dans les modèles a dalle rigide. En ce qui concerne le déplacement latéral et la dérive entre les étages, les modèles de dalles rigides ont tendance à avoir un déplacement latéral et une dérive entre les étages élevés par rapport au modèle de dalles non rigides (flexibles) avec des valeurs plus élevées dans le modèle de fondation sur radier. De même, une grande variation du cisaillement de base est observée dans la direction y, le modèle de dalle rigide ayant une réaction de base plus importante que la dalle flexible.

Mots clés : dalle rigide, dalle non rigide, diaphragme, spectre de réponse, fondation sur radier, réduction de la force axiale.

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GENERAL INTRODUCTION

Increase in urbanisation in recent decades has led to an increase the construction of tall buildings. The key driver for this growth of tall buildings is the lack of space in densely urbanised part of the world. Tall buildings alongside the vertical loads has to resist to the lateral loads (mainly wind or earth quake) which may be static or dynamic. The main resisting elements to lateral forces are the slabs. Slabs are the elements responsible to the distribution of either the vertical loads, or horizontal loads to others structural elements.

Buildings structures are generally designed using the assumption that floor systems are rigid by neglecting its in-plane flexibility. The presence of openings or the thickness of the slab greatly change the in-plane flexibility, thereby affecting the rigidity. Previous studies have been made to illustrate the importance of slab rigidity on buildings. Fouad & Ali (2012) studied the effects of floor slabs flexibility on buildings by subjecting buildings of various storeys to a response spectrum and showed that they responded differently. Prasadgowda C et al. (2017) in the same move studied the evaluation of diaphragm effects on the seismic behaviour of reinforced concrete buildings under lineal dynamic and non-linear static methods. Therefore, slab rigidity is an important factor to be taken into account especially for tall buildings subjected to high wind forces or earth quake forces. Seismic forces are caused by inertia of the structure, which tries to resist ground motions. The point of application of this inertial force is the centre of mass (CM) on each floor of the building which mainly depends on the plan regularity. However, few of the previous studies have been extent ended to irregular buildings

The objective of this work is to determine the influence of rigid and non-rigid slabs on the structural behaviour of tall buildings with irregular plan for a fixed base and for a flexible base. In order to attain the goal, this work is subdivided into three chapters; The first chapter corresponds to the literature review which gives an overview of the related concepts, the second chapter entitled methodology details the steps to be taken to attain the objective and lastly, results' presentation and interpretations of the comparison of rigid and non-rigid slabs under the application of an elastic response spectrum.

CHAPTER 1 LITERATURE REVIEW

Introduction

The design of a structure is a very important task in the sense that every detail in relation to the structure needs to be taken into account in order to assure the structure's safety. These details are mainly influenced by the site of construction, the usage of the structure, the size of the structure and the plan model to be designed. However, with regards to multi-storey structures, the analysis of lateral forces are very important to avoid high storey drift. When a structure is subjected to a lateral force such as wind or seismic force, the resultant of these forces acts on the centre of mass (CM) and the vertical elements of the structure resisting force acts on the centre of rigidity (CR). When the centre of mass does not coincide with the centre of rigidity, an eccentricity is created causing torsion. under this excessive torsion, structural elements may reach to their torsional moment capacity causing failure to the structural system (Uzun et al., 2019). The type of slab, the regularity, type of load considered greatly impacts the analysis of the whole structure. This chapters aims firstly to show that there are different types independent on the facts that different types of slabs exist, they can broadly be classified based on their rigidity and secondly to give an overview of tall buildings particularities and lastly irregularities in structures.

1.1. Slabs

Slabs are flat surfaces, usually horizontal in buildings' floors, roofs or bridges, used to separate one floor level from the other. Slabs are elements responsible for the transmission of vertical and lateral forces acting on a building. For high rise structures, the type of slab is very important because the way forces are been distributed is important for the analysis of the whole structure.

1.1.1. Types of slabs

There exist many types of slabs and can be classified considering the technology, the rigidity material and force distributions.

1.1.1.1. Based on distribution of forces

Depending on how forces are distributed in the slab element, they can either be one way slabs or two ways slabs.

a) One way slab

Forces in one way slab are distributed in one direction and the slab bend in one direction. In conventional slabs, one way slab is supported by beams on the two opposite site to carry the load. Here the ratio of the longer span to the shorter span is greater than or equal to two as illustrated by equation 1.1.

$$\frac{longer\ span}{shorter\ span} \geq 2 \tag{1.1}$$

Generally all cantilevers are one way slabs. The main reinforcement is provided in the shorter span and distribution reinforcement is provide on the longer span.

b) Two way slab

It is a floor type that carries in two directions. Conventional slabs of this type are supported by beam in the four sides. The slab is likely to bend along both direction to the four supporting edges hence distribution reinforcement is provided at both ends of the slab. All flat slabs are two ways slabs. In two way slabs, the ratio of the longer span to that of the shorter span is less than two as written shortly in equation 1.2.

$$\frac{longer\ span}{shorter\ span} < 2 \tag{1.2}$$

To resist to the formation of stress distributions, bars are provided at both ends of the slab. This type of slab are mostly used in multi-storeyed buildings.

1.1.1.2. Based On the Material

With outgrowing technology in different study fields, several types of materials are been used for constructions of floor slabs;

a) Composite slab

These are slabs that made with the combinations of different materials. We can have steel-concrete slabs, steel-timber slabs and timber-concrete slabs. Among the above mentioned, the most used is the timber concrete slab mainly because of its good fire resistance, thermal and acoustic insulation properties compared to reinforced concrete. Timber-concrete slabs though having all these advantages are not widely spread because it is a recent technology and its behaviours haven't yet all been explored. Also the fact that Eurocode contains no particular part on the design of concrete slab.

b) Reinforced concrete slab

Made of concrete (mixture of cement, gravel and sand) and rebar, reinforced concrete slabs are the most used type of slab worldwide mainly due to their availability. Concrete has good compressive strength and very weak tensile strength so makes it difficult to be crushed and much easier to be pulled apart. Due to the concrete property aforementioned, it is necessary to be reinforced to increase its tensile strength since cracks and breaks in slabs are tensile failures. Its main drawbacks are a significant self-weight and the use of non-renewable resources. Moreover, it provides rather low thermal insulation properties, thermal storage capacity and acoustic insulation. Many types of reinforced concrete slabs exist, each with its technology.

i) Flat slabs

A flat slab is a reinforced concrete slab supported directly by columns or caps as seen in figure 1.1. Flat slabs do not have beams so are referred to as beam-less slabs. Here, loads are directly transferred to columns. They are generally used in parking, decks, and commercial buildings. The main advantages of this floor type is the fact that it minimizes floor-to-floor height,

Figure 1.1. Flat slabs (*www.civilread.cm*)

construction time is reduced and it increases the shear strength of the slab. However, they are not suitable for long span.

ii) Conventional slabs

These are slabs supported by on beams and columns, in this type of slab, the thickness is relatively small whereas the load of the beam is large and load is transfered to beams then to columns. Compared to flat slabs, no column caps are required but they use more framework. They are generally square in shape with a length of 4m.conventional slabs can distribute in one way or in two way. A typical two way conventional slab is illustrated in figure1.2.

Figure 1.2. Conventional two-way slab (Nlongkak, Yaounde)

iii) Hollow core slabs

They derive their names from the voids or cores which run through the units. The voids serves as duct or simply to reduce the self-weight of the slab maximising structural efficiency. Hollow slabs has excellent slab capabilities achieving a capacity of 2.5kN/m2 over 16m span and are generally prefabricated. These slabs are not economical for small spans, are difficult to restore and difficult to produce satisfactory connections between the pre cast members. Beside the above flaws, these slabs are not economic for small spans, are difficult to restore, difficult to produce satisfactory connections between the pre cast members. Beside the above flaws, represented in figure 1.3.

Figure 1.3. Hollow core slabs (www.civilread.cm)

iv) Hollow blocks slabs

Mainly used in Cameroon, they are slabs constructed with hollow bricks of general dimensions 50cm×20cm×16cm as displayed in figure 1.4 carried by ribs. This type of slab is the most used type of slab in Cameroon mainly due to the fact that it saves the amount of concrete and hence its own weight is reduced. Facilitate constructions especially when all beams are hidden. Its main disadvantage is the fact that it is not easily repaired and it is not economic for short spans. Figure 1.5 shows the casting of a hollow block slab.

Figure 1.4. Hollow block

Figure 1.5. Hollow block slab

v) Waffle slabs

Generally used at the entrance of hotel, malls, restaurants for their good pictorial view and to ensure artificial lightening. Waffle slabs are reinforced concrete containing square grids with deep sides and are also called grid slabs. They have holes underneath giving an appearance of waffles as showed in figure 1.6. The main purpose of using this technology is for his strong foundation characteristics of cracking and sagging resistance.

Figure 1.6. Waffle slabs (Mount febe hotel, Yaounde)

vi) Pre tensioned and Post tensioned slabs

Ideal for longer spans and suitable for the construction of stronger structure at a suitable price, tensioned slabs is a slab put into tension by tendons or cables before/after its construction. Pre-tensioned is when the slab element is tensioned before casting and it is usually prefabricated. Post tensioned is achieved by placing high-tensile cables in the concrete before casting. The inconveniency of this technology is the fact that it not only need professional skills for its realisation but if proper care is not taken, it can leads to future mishaps.

1.1.1.3. Based on the Rigidity

In addition to resisting gravity loads, slabs are also generally designed to act as diaphragms. In this respect, they are required both to distribute seismic forces to the main elements of horizontal resistance, such as frames and shear walls, and also to tie the structure together so that it acts as a single entity during an earthquake. The robustness and redundancy of a structure is highly dependent on the Performance of the diaphragms.

a) Rigid Slabs

"The diaphragm is considered rigid, if, when it is modelled with its actual in-plane flexibility, its horizontal displacements nowhere exceed those resulting from the rigid diaphragm assumption by more than 10% of the corresponding absolute horizontal displacements in the seismic design situation" Eurocode 8. However, these requirements are much limited compared to other norms because the problem of rigidity is much more complex. Some others set quantitative criteria relating the in-plane deformation of the diaphragm with the average drift of the associated storey. Federal Emergency Management Agency [FEMA 1997] define a slab as "rigid" when this lateral deformation of the diaphragm is less than half the average inter-storey drift of the storey immediately below. Moeini, (2011) during his investigation into the floor's diaphragms flexibility in reinforced concrete structures and code provision concluded that generally, in cases the floor diaphragms may be modelled as fully rigid without in-plane deformability. Uniform Building Code [UBC, 1994] defines rigidly the maximum lateral deformation of the diaphragm (Δflexible) is less than twice the average storey drift of the associated storey (Δstorey). The above definition is summarised in equation 1.3.

$$\beta = \frac{\Delta_{flexible}}{\Delta_{storey}} < 2 \qquad (1.3)$$

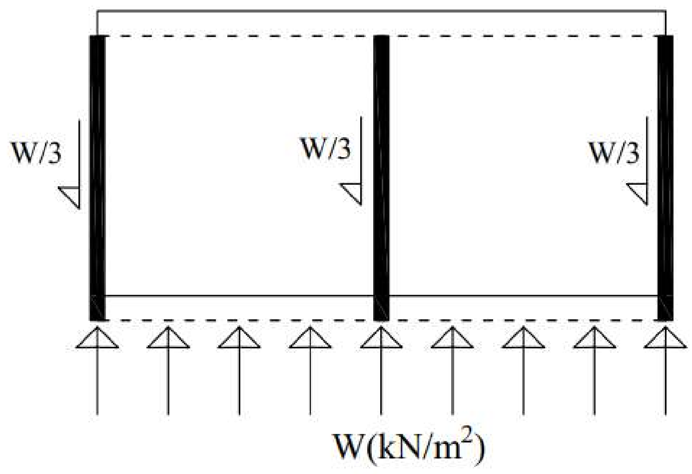


Figure 1.7. Partition of horizontal forces on a rigid slab

The purpose of determining whether a diaphragm is flexible or rigid is to determine whether a diaphragm should have the loads proportioned according to the tributary area or the relative stiffness of the supports. For rigid diaphragms displayed in figure 1.7,

- The load should be distributed according to the stiffness.
- Rotational or torsional behaviour is expected and the action results in a redistribution of shear to the vertical force-resisting elements.

b) Semi-rigid slab or stiff

Moeini, (2011) in his article early mention above draws our attention on the fact that only Federal Emergency Management Agency [FEMA 1997] code among le eight codes studied (Eurocode 8 included) aborts the notion of semi-rigid slabs. It defines a stiff slab as one where the diaphragm is neither flexible nor rigid.

Prasadgowda C et al., (2017) studied the evaluation of roof diaphragm effects on seismic behaviour of reinforced concrete buildings with one of the objectives being to understand the effects of rigid and semi-rigid floor diaphragms on the seismic performance of 5 story reinforced concrete buildings concluded that the diaphragm can significantly alter the time period in case of semi-rigid model. Difference of time periods for the semi-rigid case is small for the first two modes, whereas the difference is large for the mode 3 and onward. This means that for higher modes the structural behaviour becomes critical for models with slabs.

c) Flexible slab

Even though a rigid floor diaphragm is a good assumption for seismic analysis of the most buildings, several building configurations may exhibit significant flexibility in floor diaphragms. Moreover, Eurocode 8 in the stating of criterion for plan regularity mention the fact that for a slab to be rigid, the in plane stiffness has to be sufficiently big compared to the lateral stiffness of vertical elements. According to Eurocode 8, if a floor diaphragm does not respect the definition of rigidity, it is considered flexible. Therefore, and all of the buildings without shear walls are rigid due to the fact that the presence of shear wall increases the vertical stiffness of the building which tends to reduce the rigidity of the slab.

Federal Emergency Management Agency [FEMA 1997] considers a slab as "Flexible" when the maximum lateral deformation of the diaphragm along its length is more than twice the average inter-storey drift of the storey immediately below. The uniform building code defines a flexible slab as in equation 1.4.

$$\beta = \frac{\Delta_{flexible}}{\Delta_{storey}} \geq 2 \qquad (1.4)$$

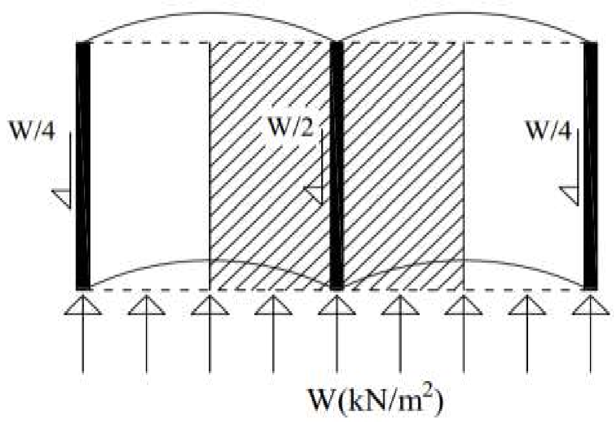


Figure 1.8. Partition of horizontal force on a flexible slab

For flexible diaphragm as illustrated in figure 1.9,

- the loads should be distributed according to the tributary area
- Rotation of the diaphragm may occur because lines of vertical elements have different stiffness, the diaphragm is not considered sufficiently stiff to redistribute the seismic forces through rotation.

1.1.2. Factors affecting the rigidity of slabs

Floor selection in tall building design is one of the important decision structural engineers have to make, since it composes of around 20% of the total structural weight. Lateral load generated from wind or earthquake is transferred to the lateral load resisting system according to respected lateral stiffness at each floor level. The diaphragm behaviour of different types of floor systems usually differs substantially and depends on the details of the floor system, as in some cases the diaphragms behaviour may be unknown. Floor details that affect rigidity are described below.

1.1.2.1. Slab thickness

The variation of floor slab thickness directly changed the ratio between vertical element and horizontal element stiffness. From the connection equation of plates for bending, the rigidity is given by equation 1.5.

$$\mathrm{D} = \frac{Eh^3}{12(1-v^2)} \tag{1.5}$$

Where

E= young modulus of the material

h=slab thickness

ν =poison ratio

From equation 1.5, an increase in the slab thickness increases the rigidity of the slab.

1.1.2.2. Openings in slabs

Due to technical reasons of functionalities, most buildings have openings in their slabs which is either meant for stairs, pipes or an elevator. The presence of openings in floor slabs modifies the floor characteristics itself. The moment capacities of the floor slab are reduced. The reduction values depend on the location of openings and openings' size. Prasadgowda C et al., (2017) studied the evaluation of roof diaphragm effects on seismic behaviour of reinforce concrete buildings with one of the objectives being to understand the effects of floor openings placed in symmetric and asymmetric plan locations with respect to the centreline of the building in order to investigate the influence of floor openings in slender plan buildings. They made the following observations:

- The creation of an opening at a storey level increases the displacement of the particular storey level significantly hence increasing the storey drift.
- The presence of floor opening insignificantly affect the natural period of the overall structure.
- The base shear increases compared to a building with no opening in the slab and the in plane capacities of the slab is reduced

1.1.2.3. Slab type

The structure system of the floor slab some times are an architectural requirement. Where the architectural designer needs the full clear height or no drop beams. The structural engineer must think on the type of slab which can feed these requirements. Elsherbeny, (2016) in his master thesis on the behaviour of reinforced concrete flexible floor diaphragms under seismic load studied the effect of slap type on the rigidity of a slab. He replaced the conventional slab (beam-slab type) by a flat slab and did the following observations:

- The natural period of the structure was increased by about 10 % which reflects the flexibility.
- A reduction of the building monotonic capacity to resist lateral loads of the building weight was observed.
- An increase in base shear was equally observed

With all the above observations, it was concluded that. All other things being equal, the change of slab type from conventional to flab slab tends to increase its flexibility property.

1.1.2.4. Materials used

As seen in equation (1.5), the rigidity is directly link to the intrinsic property (young modulus E and Poisson ratio ν) of a material. The Poisson ratio is the ratio of relative contraction strain normal to the applied load and to the relative extension strain in the direction of load applied. A material with a bigger Poisson Ratio and young modulus will tend to be more rigid comparatively.

1.2. Irregularity of structures

Structural engineers aim at designing the most regular structure possible especially when it has to do with multi-storey buildings since the modelling, analysis, dimensioning, detailing and construction of simple structures are subject to much less uncertainty and thus the prediction of its seismic behaviour is much more reliable . In reality, structures are almost always irregular, as perfect regularity is an idealization that very rarely occurs. Eccentricities may be caused by some irregularities in structures such as mass distribution, stiffness, shape of plan, strength of material used, etc. Nevertheless limits for regularity are set up in Eurocode 8 chapter 4.3. Hence a structure is considered irregular if it exceeds the prescribed limits. Structural irregularity can be broadly classified as vertical irregularity and horizontal irregularity.

1.2.1. Vertical irregularity (irregularity in elevation)

This has to do with variations of structure characteristics with height (elevation). Therefore vertical irregularity are observed in more than one floor building. The different factors affecting regularity in elevation are detailed below.

1.2.1.1. Mass

"The mass of the individual storeys shall remain constant or reduce gradually, without abrupt changes, from the base to the top of a particular building" EC 8 clauses 4.2.3.3. If for one reason or the other the above criterion is not respected, the structure becomes irregular. Researches were done to study the effects of mass and stiffness irregularity on the fundamental period of infilled reinforced concrete frame by Bhatt et al., (2019) and they concluded that the time period of the building is directly affected by the magnitude of the mass irregularity. When irregular mass is in higher elevation the time is increased. Equally under the conduction of the aforementioned, it was observed that the location of mass irregularity for bottom one third parts do not vary the time period significantly but when irregularity is in top one third parts the time period increases significantly.

In the same move, Georgoussis & Mamoua, (2018) with the aim of investigating on the effects of mass eccentricities on the torsional response of medium height multi-storey buildings with in plan and in elevation irregularities and they concluded that mass irregularities (in elevation) tend to shift the optimum torsional axis from the mass axis and a small shift produces a large torsion increasing torsion in the structure. Titiksh, (2018) while studying the effects of irregularities on the seismic response of a medium rise structure came out with the fact that mass irregularities tend to increase greatly the base shear compared to other irregularities.

1.2.1.2. Stiffness

In English, stiffness refers to the inability to move without pain but in civil engineering, stiffness is an intrinsic property of a material related to young modulus. Structural stiffness unlike the above definition refers to the rigidity of a structural element that is the inability to move. Stiffness (rigidity) of vertical element affects directly the centre of rigidity thereby increasing the eccentricities that led to great torsional effect. Unlike mass, In order to limit or reduce these torsional effects, Eurocode 8 advice to avoid abrupt change in stiffness from one level to the other. For more understanding, Bhatt et al., (2019) in his aforementioned study observed that stiffness irregularity equally increase the time period of the structure. Also, the position of the irregularity is relevant because the application of the stiffness irregularity at bottom one third significantly increases the time period compare to when it is the top one third. Titiksh, (2018) in his study of the effects of irregularities on the seismic response of a medium rise structure showed that stiffness irregularity does not greatly affect the base shear. Antony & Pillai, (2016) in their investigations on the effects of vertical irregularities on the seismic performance of RC buildings deduced that stiffness irregularity tends to increase the storey drift (the lateral displacement of one building relative to the other).

1.2.1.3. Setback

This is a type of vertical irregularity in which a certain part of the structure is missed after a certain height (in other words, variation in vertical geometry. For aesthetic considerations and space availability, setback is commonly adopted especially in high-rise buildings. Eurocode 8 prescribes five different criterion for the different setback based on the length variation and total height of building. Setback is indirectly related to above mentioned factors because setback results to a variation in mass and in stiffness.

1.2.2. Horizontal irregularity (irregularity in plan)

As earlier explained, eccentricities are mainly caused by irregularities. Vertical irregularities exclusively affect the centre or rigidity (CR) and plan irregularities tend to change the centre of mass (CM). Below are some properties that can cause a shift of the CM.

1.2.2.1. Irregular distribution of mass, strength, stiffness along plan

Buildings nowadays are built for different purposes. For one such building, following standard of application of loads based on the utility of a structure, it is obvious that different masses are applied at different part of the same plan hence leading to an irregular distribution of mass which changes the position of CM. Moreover, the use of different types of slabs (rigid and flexible slabs) changes the position of the centre of mass. Since design is based on the loads applied, the resultant design will produce different stiffness and strength for the slab. So special care has to be taken when dealing with this type of structures. The eccentricity generated due to this irregular distribution of mass, stiffness or strength produces torsion. Torsion unbalance in a building varies the seismic force reduction factor, the design eccentricity and the natural time period. Varadharajan et al., (2012) in their journal article on the review of different structural irregularities in buildings explained that the irregular distribution of stiffness increases the lateral period.

1.2.2.2. Asymmetrical plan shape

With the on growing development in civil engineering, regular structure are more and more forbidden for structure with L-shape, X-shape, Y-shape (asymmetric plan) mainly for aesthetic reasons. However, it is not without effects because during an earth quake, depending on the general structure of the building which is defined by the plan. Asymmetrical plans tend to shift the centre of mass of a structure (CM) hence creating an eccentricity that produces torsions in the buildings. Lone & Chand, (2019) investigated on a comparative study on seismic and wind performance of multi-storeyed building with plan and vertical irregularities and explained that; as shape changes, displacement increases and lateral load-carrying capacity also changes, so the shape of the building plays an important role in the design of the stable structure. In this same move, İlerisoy, (2019) in his discussion of the structural irregularities in the plan for architectural design within the scope of earthquake codes explained that the structures containing simple geometric dimensional differences in the plane of the plan will cause a change in the structural system calculations and hence the cost values in different orders, even though they do not result in significant architectural differences. Furthermore, the abovementioned researcher proposed some measures to improve the structural behaviour of such a structure by: Using joints of dilatation which divides the structure into more compacts and independent form. Addition of vertical structural system elements (shear walls and columns) that will not adversely affect a building's resistance to torsion, and will exhibit more resistant stiffness to lateral force. Using the more rigid foundation types, the adverse effects can be essentially damped.

1.2.2.3. Diaphragm discontinuity

Even though not emphasised in Eurocode 8, it is an important factor for irregularity as described in other codes such as the china code, the Iran code, Mexico code, etc. The horizontal loads on the buildings are mainly concentrated at the slabs, and the horizontal loads are distributed to the vertical structural elements by the slabs. For this reason, it is necessary to transfer the inertial forces caused by the earthquake effects to the slabs and the structural system elements such as beams, columns and shear walls from the slabs (Terzi and Elçi, 2006). Diaphragm discontinuity irregularity is a plan irregularity type that characterises itself by the presence of openings in slab (floor diaphragm) hence impacting the rigidity of the slab and consequently has large torsional effects. İlerisoy, (2019) in the same study as early mentioned above advice to reduce as much as possible the opening in slab if in a seismic region.

1.2.2.4. Re-entrant corners

"If in plan set-backs (re-entrant corners or edge recesses) exists, regularity in plan may still be considered as being satisfied, provided that these set-backs do not affect the floor in-plan stiffness and that, for each set-back, the area between the outline of the floor and convex polygonal line enveloping the floor does not exceed 5 % of the floor area The in-plan stiffness of the floors shall be sufficiently large in comparison with the lateral stiffness of the vertical structural elements so that the deformation of the floor shall have a small effect on the distribution of the forces among the vertical structural elements. In this respect, the L, C, H, I, and X plan shapes should be carefully examined, notably as concerns the stiffness of the lateral branches, which should be comparable to that of the central part, in order to satisfy the rigid diaphragm condition. The application of this paragraph should be considered for the global behaviour of the building." EN 1998-1:2004 chapter 4.2.3.2. The above are the specifications of the European code.

The flaw of the non-respect of the above specifications create points of high concentration at nodes mainly due to different stiffness at these particular part. Moreover the effect of torsional forces make the analysis difficult. However, studies were conducted in order to determine the degree of complexity of this factor. Titiksh, (2018) Studied the lateral displacement, time period and base shear of re-entrant corners and concluded that structures with re-entrant corner show a maximum lateral displacement and shows on the other hand, a lower time period (high frequency) and base shear compared to stiffness and mass. A brief summary of the relationship between different types of irregularities is displayed in figure 1.9.

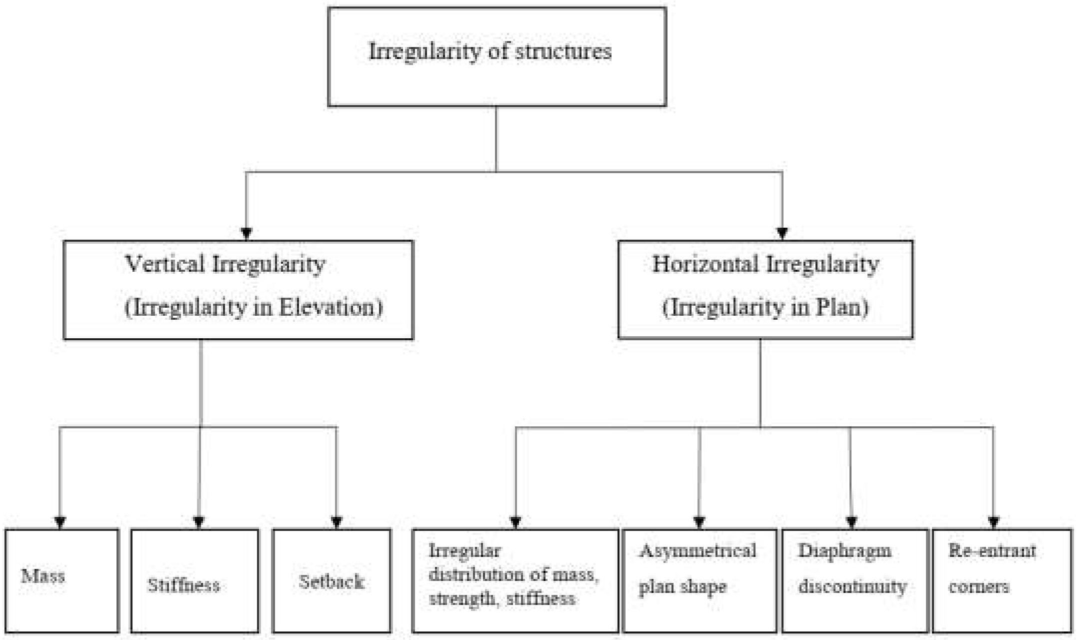


Figure 1.9. Relationship between different types of irregularities

1.3. Tall Buildings

Tall buildings can be seen all over the world, they can be used to show wealth or power, religious beliefs or to push boundaries of engineering. The ancient Egyptians built the pyramid nearly 5000 years ago as tombs of their pharaohs and consult, and to this day they are still standing as some oldest high-rise structure in the world. However, they are not considered as buildings because they are not habitable. There are many different ways to define tall (high-rise) structures. An architect or a town planar will define it as a building that clearly protrudes above the surrounding buildings but for a structural engineer, the definition of a tall building lies with the problems that are associated with the design of the building. It is simpler to consider a building as tall when its structural analyses and design are in some way affected by the lateral loads.

1.3.1. Structural System of Tall Buildings

The engineer's first job is to determine which loads will act on a structure and how strong they might be in extreme cases. Structural engineering would be unnecessary and we would all be out of work if the earth did not pull, the wind did not blow, the earth's surface did not shake or sink, and the air temperature and humidity did not change. But in the real world, we must concern ourselves with all the loads that act unavoidably on buildings.

1.3.1.1. Gravity Loads Systems

a) Gravity loads

Different gravity loads are to be considered when designing a structure. Much care needs to be taken for the design of tall buildings. Vertical loads taken into account are dead load (permanent load), non-permanent loads, construction loads, rain and thermal loads.

A structure consists of elements like columns, beams, floors, arches, or domes that must first of all, support their own weight, the so-called dead load. Dead loads consist of the weight of all materials of construction incorporated into the building including, but not limited to, walls, floors, roofs, ceiling, stairways, building partitions, finishes, cladding, and other similarly incorporated architectural and structural items and fixed service equipment including the weight of cranes.

In addition to its dead load, a structure must support a variety of other weights (people, furniture, equipment, and stored goods) which are not part of the structure and are considered as live loads. Live loads differ from dead loads in their character: they are variable and unpredictable. Changes in live loads occur not only over time but also as a function of location. The change may be a short- or long-term one, thus making it almost impossible to predict live loads in statically terms. Loads caused by the contents of objects within or on a building are called occupancy loads.

Structural members may be subject to loads larger by far than the design loads during construction of a building. These loads, called construction loads, constitute an important consideration in the design of structural elements. The use of heavy equipment such as trucks, truck cane, crawler cane, etc and materials on a small area causes concentrated loads that are much larger than the assumed live loads for which the structure was designed. Structural failures have resulted from such conditions.

b) Gravity resisting loads

The major components associated with the vertical load resisting systems of high rise buildings are the floor system, columns, transfers beams and shear walls. The floor system is part of the gravity load resisting system that remains relatively similar in both high rise and low rise buildings. However, for tall buildings, it is imperative to minimise the weight of the entire structure. Therefore, lightweight floor systems, such as post-tension slabs, are the dominant selection in the design.

1.3.1.2. Lateral Loads Systems

Additional to vertical loads, structures are exposed to lateral forces due to winds and earthquakes. Lateral forces cause very high stresses and deflections.

a) Lateral loads

Earthquakes are catastrophic events that occur mostly at the boundaries of portions of the earth's crust called tectonic plate when movement occurs in these regions, along faults, waves are generated at the Earth's surface that can produce very destructive effects. Earthquake strengths are evaluated on scales like the Richter scale, which measures the magnitude of the energy in the earthquake. For example, an earthquake measuring 4 or 5 on the Richter scale does little damage to well-built buildings, while one measuring 8 or above collapses building and may cause many deaths. Four potential seismic zone are detectable in Cameroun classified into regions based on their highest magnitude on Richter scale. The highest magnitude is 6 on Richter scale which corresponds to Seismic Source Region IV (northern boundary of Congo Craton) (Eloumala et al, 2014). To better understand the action of earthquake load on a building, it is helpful to study strong-motion seismograms (also called time histories). Unlike seismic loads, wind loads need to be considered no matter where the building is Located. Wind is caused by pressure differences in the atmosphere which causes the air to move. The movement of air is fairly undisturbed high off the ground, but close to the ground, the surface of the earth and man-made structures cause turbulence and eddies. Gust, vortex, shedding, buffeting, Galloping and flutter are the main wind induced effects on buildings. Only two of these wind effects (vortex shedding and galloping) must be taken into account according to Eurocode when designing a high-rise building.

There are static, dynamic, and aerodynamic effects of wind on structures. In general, static effects of wind are sufficient in the design of low-rise buildings. In tall buildings, the dynamic and aerodynamic effects along with the static effects are required to be analysed.

b) Lateral resisting systems

Configuration of the lateral loads resisting system within the building is fundamental for a good design, concerning such issues as structural irregularities, torsion, redundancy, and the combination of systems. There exist many lateral loads resisting systems for tall buildings among which we have; the brace frame structure, shear wall structure, rigid frame structure, frame tube structure, outrigger-braced structure. The selection of the lateral load resisting system for specific building is clearly a design decision of fundamental importance, yet there is no system that is best for all buildings. NASSANİ, (2020) in his paper on the lateral load resisting system in high-rise reinforced buildings proposed that, frame tube system is recommended to be used in higher buildings, for example more than 60 stories.

1.3.2. Specificity tall buildings.

When designing high-rise buildings, phenomena arise that could be disregarded when designing lower buildings.

1.3.2.1. Natural frequencies

All structures have specific frequencies for when the structure begins to resonate, these frequencies are called the natural frequencies. The lowest natural frequency is known as the fundamental frequency. The natural frequencies are important when doing the dynamic analysis of a structure. Dynamic loads, for example wind and earthquake loads can cause structures to sway drastically and in worst case cause a collapse. The natural frequencies are described with modes, each mode is described by a natural frequency and a shape. The three first natural frequencies for a building are normally the sway in both directions (x- and y-direction) and the torsional sway (around the z-axis). Some important aspects to take into account when calculating the natural frequency are the mass distribution and the stiffness. The mass and stiffness at each floor are required. Normally, the mass includes all dead loads plus 10-30% of the live load. There is no rule for how much of the live load that should be included and the number is based on what the building is used for and the opinion of the structural engineer. It is important to include all mass since it will have a great effect on the natural frequency. The moment of inertia is taken around an axis in the centre of gravity of the building. The mass distribution along the building height is needed to determine the natural frequencies of the building. Furthermore, the displacements and natural frequencies can be used to calculate the acceleration at the top of the building according to Eurocode.

1.3.2.2. Accidental loads and progressive collapse

Progressive collapse is defined as a collapse of a large part of a structure initiated by a smaller failure of a load bearing element. According to Eurocode 1990, buildings are to be designed and executed to not take a disproportionate amount of damage from explosions, impacts or the consequences of human error, with regard to the severity of the load. What a building is supposed to withstand is decided for each individual project with the client and the authorities. Furthermore, a building should be able to withstand limited damage without the entire building collapsing. To take progressive collapse into consideration, buildings are put into three different consequence classes based on the type of the building. Class one is a low risk group which includes buildings where people rarely are located and class three is a high risk group. All buildings above 15 floors and buildings where a large amount of people are located are placed into consequence class three. If a building is in class three, a risk assessment should be done for the building.

1.3.2.3. Wind on pedestrians

When wind hits a building it will take the easiest way to get past, going around rather than over is the easiest for any relatively slender building. Wind going around buildings can give rise to strong vortexes causing discomfort for pedestrians near the base of the building. Analytically it is almost impossible to estimate the effects the wind will have on pedestrians due to the large amount of factors involved. Wind tunnel tests can be used to obtain reliable estimates of wind conditions around the base of a building.

1.3.2.4. Comfort requirement

One of the most important aspects to consider during the design process of high-rise buildings is the serviceability limit state. The design for comfort is performed with two main factors in mind, the horizontal deflection and the motion of the building. The lateral movements of the building includes the maximum deflection of the building and the story drift. The story drift is the difference in deflection between consecutive floors and is of interest due to the possibility of damages to non-structural elements such as cladding. The horizontal deflection is calculated with equivalent static loads and the limit for horizontal deflection is generally set to H=450-H=500. When looking at the motions of the building the acceleration is the factor that is evaluated. Comfort due to motion will be discussed further due to its complicated nature (Ferrareto et al.2015) as sighted by Viktor & Stefan, (2016). Careful study of the response to dynamic loads such as wind load sand seismic loads must hence be performed.

1.3.2.5. P-delta effect

In high-rise buildings, the P-delta effect can have a big impact on the overturning moment. The P-delta effect is a second order effect and is caused by the axial force and the displacement of the structure. When designing high-rise buildings in seismic zones this effect is of even greater importance due to the swaying of the buildings. If the swaying of the building is large and therefor creates a large displacement, the overturning moment could cause damage to the building. To avoid damage or collapse due to the P-delta effect, the lateral stiffness or the strength of the building must be increased. Since this phenomenon is most common in seismic zones. Figure 1.10 illustrates the phenomena.

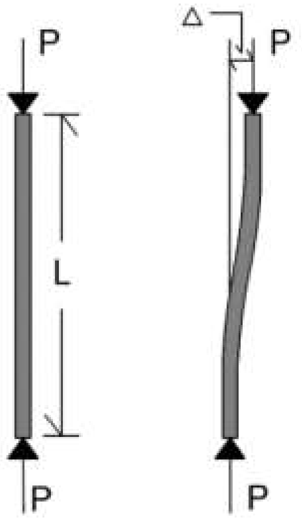


Figure 1.10: illustration of p-delta effects

Conclusion

Present part was aim at given an over view on slabs, tall buildings and irregularities. From journal articles and books, researchers explain the broadness and importance of each element mentioned above. There exist different types of slabs, based on the distribution of force, based on the materials and based on the rigidity. As for irregularity, it can be in plan or in elevation. Tall buildings specificities were equally detailed. Hence for a proper designs and analysis of a tall building, the type of slab has to be meaningfully chosen, the slab analysis has to be chosen based on its rigidity. In the same move the presence of an irregularity type needs to be analysed to see its impact on the structure and proper care in the choice of a convenient resisting system. The methods adopted for the study are explained in the next chapter.

CHAPTER 2 METHODOLOGY

Introduction

This part is aimed at showing a clear method and tools used to achieve the objective of the work as early mentioned in the general introduction. The first part of this work will explain the conception through the codes used, actions applied and combinations, followed by a static design of our case study, thirdly a seismic design, the modelling on a fixed base and equally taking into consideration soil structure interaction and lastly, the analysis criteria to bring out the main differences between rigid and non-rigid slab laying more emphasis on the superstructure behaviour.

2.1. Conception

The title of this thesis clearly gives some characteristics that the building needs to have among which, the plan irregularity and the tall buildings. For analysis required in this thesis, an L-shape building will be conceived for residential purposes and the norms applied is the British standard of European code of construction. The structural conception is detailed in the oncoming section via the application of specific norms and loads.

2.1.1. Codes

A good construction project respect a specific norm depending on where the construction is done. Worldwide, we have so many types of norms among which the china code, the turkey code, the American code, Eurocode, etc. Eurocode is the standardised code recommended by the European Committee for Standardization. Depending on the site of construction, on the material used and the type of structure to be done, different parts of Eurocodes are used. For this case study the parts used are:

- Eurocode 0: Basis of structural design
- Eurocode 1: Actions on the structures, part 1:general actions
- Eurocode 2: Design of concrete structure, part 1: general rules and rules for buildings
- Eurocode 8: Design of structures of earthquake resistance, part 1: general rules, seismic action rules for building.

2.1.2. Actions and combinations of actions

2.1.2.1. Actions

There exist different types of loads but for this case study, the loads applied are permanent or variable

a) Permanent actions

They are actions which have negligible variation with time although the structure lifespan. Permanent loads can either be structural or non-structural.

- Structural loads (G1): self-weight of slabs, beams, pillars
- Non-structural loads (G2): weight of wall, Ceramic stone ware, sand layers, etc.

b) Variable actions

They are actions for which the variation in magnitude with time is neither negligible nor monotonic. They are constituted of imposed loads and seismic actions.

i) Imposed loads (Q_k)

Imposed loads are those arising from occupancy. It includes the normal use by persons, the furniture and moveable objects, vehicles and other. The value of the imposed loads is defined in the Eurocode according to the category of use of the building.

ii) Seismic actions

Seismic load result from earthquake and the earthquake motion at a given point on the earth surface is represented by an elastic acceleration response spectrum which is defined as the relationship a relation between the ground motion and the period of vibration during an earthquake. The horizontal component of the seismic action, the elastic response spectrum $S_e(T)$ is given by:

$$0 \leq T \leq T_B: S_e(T) = a_g S\left[1 + \frac{T}{T_b}(2.5\eta - 1)\right] \tag{2.1}$$

$$T_B \leq T \leq T_C: S_e(T) = 2.5 a_g S\eta \tag{2.2}$$

$$T_C \leq T \leq T_D: S_e(T) = 2.5 a_g S\eta(\frac{T_C}{T}) \tag{2.3}$$

$$T_D \leq T \leq 4s : \; S_e(T) = 2.5a_g S\eta(\frac{T_C T_D}{T^2}) \tag{2.4}$$

Where :

S_e(T)	is the elastic response spectrum
T:	is the vibration period of a linear single degree of freedom system
a_g:	is the design ground acceleration on type A ground ($a_g = \gamma_1 a_g R$)
T_B:	is the lower limit of period of the constant spectral acceleration branch
T_C:	is the upper limit of the period of the constant spectral acceleration branch
T_D:	is the value defining the beginning the constant displacement response range of the spectrum
S:	is the soil factor that depends on the ground type
η:	is the correction factor given by $\eta = \sqrt{10/(5 + \xi)}$
ξ:	is the viscous damping ratio

The values of T_B, T_C, T_D for each ground type and shape of spectrum used depends on the country in concern. However, for type I spectrum (high and moderate seismic regions) the design parameters recommended by Eurocode 8 is display in table 2.1.

Table 2.1. Elastic response spectrum parameters for the design

Ground type	Soil factor (s)	T_B (s)	T_C (s)	T_D (s)
A	1.0	0.15	0.4	2.0
B	1.2	0.15	0.5	2.0
C	1.15	0.20	0.6	2.0
D	1.35	0.20	0.8	2.0
E	1.4	0.15	0.5	2.0

2.1.2.2. Combination of Actions

Combination of actions is a set of design values used for the verification of the structural reliability for a limit state under the simultaneous influence of different actions. The combinations used for the design are the fundamental, the rare and the seismic combination.

Fundamental combination used for Ultimate Limit State (ULS)

$$\sum_{j\geq 1} \gamma_{G,j} G_{k,j} + \gamma_{Q,1} Q_{k,1} + \sum_{i>1} \gamma_{Q,i} \psi_{0,i} Q_{k,i} \tag{2.5}$$

Where the coefficients $\gamma_{G,j}$ and $\gamma_{Q,i}$ are partials factors which minimize the action which tends to reduce the solicitations and maximize the one which tends to increase it. The choice Table A1.2 (B) preconized by the Eurocode 0 for the structural and Geotechnical (STR and GEO) gives more information on the usage of theses coefficients as displayed in the figure Characteristic combination (rare), usually used for non-reversible serviceability limit states (SLS). To be used in the verifications with the allowable stress method.

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

(*) 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. I permanent actions only.

NOTE 2 The γ and ξ values may be set by the National annex. The following values for γ a
$\gamma_{Gj,sup} = 1,35$
$\gamma_{Gj,inf} = 1,00$
$\gamma_{Q,1} = 1,50$ 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.

Figure 2.1. Partial factor values

$$\sum_{j\geq 1} G_{k,j} + Q_{k,1} + \sum_{i>1} \psi_{0,i} Q_{k,i} \tag{2.6}$$

Seismic combination, used for the ultimate and serviceability limit states related to the seismic action E

$$\sum_{j\geq 1} G_{k,j} + E + \sum_{i>1} \psi_{2,i} Q_{k,i} \tag{2.7}$$

Note:

$G_{k,j}$ is the characteristic value of the permanent action j

$Q_{k,i}$ is the characteristic value of the accompanying variable action i

E is combination of the effects of the horizontal component of the seismic action.

2.2. Static design methodology

Static design consist of determining the action of static loads on different part of the structure. The static analysis of a building start from the durability verifications to the design of slabs, beams and columns, the analysis can equally be extended to the interaction between the soil and the footings through footings design. Only the first three aforementioned will be done in this work.

2.2.1. Durability and concrete cover

Durability can be defined as the conservation of the physical and mechanical characteristics of the structure and the materials with which the structures are constructed all through the design life. It can be verified by considering a good concrete cover which guarantees the protection of steel against corrosion, the safe transmission of bond forces and an adequate fire resistance. Concrete cover is define by Eurocode as the distance between the surface of the reinforcement closest to the nearest concrete surface (including links and stirrups and surface reinforcement where relevant) and the nearest concrete surface. The nominal concrete cover C_{nom} by Eurocode is defined in equation 2.8 and illustrated in figure 2.2.

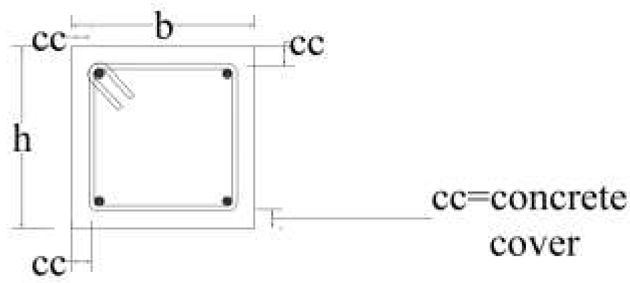


Figure 2.2. Illustration of concrete cover

$$C_{nom} = C_{min} + \Delta C_{dev} \tag{2.8}$$

$$C_{min} = max\left(C_{min,b}; C_{min,dur} + \Delta C_{dur,\gamma} - \Delta C_{dur,st} - \Delta C_{dur,add}; 10mm\right) \tag{2.9}$$

Where:

C_{min} is the minimum concrete cover

ΔC_{dev} is the allowance in design for deviation

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

$C_{min,dur}$ minimum cover due to environmental conditions

$\Delta C_{dur,\gamma}$ additive safety element

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

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

2.2.2. Slab design

In this work, the two most used type of slab used in Cameroon will be design i.e. the conventional two way slabs and the hollow block slab. A strand of 1m (for the conventional two-way slab) and a rib (for the hollow block slab) will be modelled on the software SAP2000 V21 to obtain the solicitations after application of loads in different arrangements and designed for ultimate limit state (ULS) and serviceability limit state (SLS).

2.2.2.1. Analysis of a hollow block slab

As clearly detailed in chapter one, hollow block slabs are carried ribs so the main element to be design here is the rib. A rib is designed as an inverted T-shape structure with the total height corresponding to the height of the slab.

2.2.2.2. Analysis of a conventional two way slab

A two way slab is designed using grashof plate theory. Therefore the dimensions of the slab panel has to be far greater than the thickness of the slab e. if a uniform surface load **q** acts on the slab , the analysis is done as below.

a) Grashof principle of analysis for one slab panel

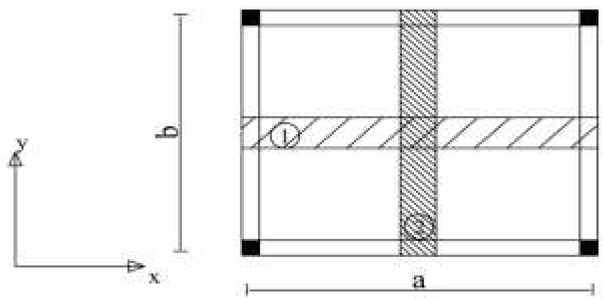


Figure 2.3. Grashof model for one panel

If $a \gg e$ and $b \gg e$, the load carried in x and y directions of a slab as illustrated in figure 2.3 are respectively defined by equations 2.10 and 2.11.

$$q_x = (\frac{b^4}{a^4 + b^4})q \tag{2.10}$$

$$q_y = (\frac{a^4}{a^4 + b^4})q \tag{2.11}$$

$\alpha_x = \frac{b^4}{a^4+b^4}$ and $\alpha_y = \frac{a^4}{a^4+b^4}$ are called Granshof coefficients in x and y directions respectively. These coefficient represent the fraction of the applied charge to each direction as represented by figure 2.4.

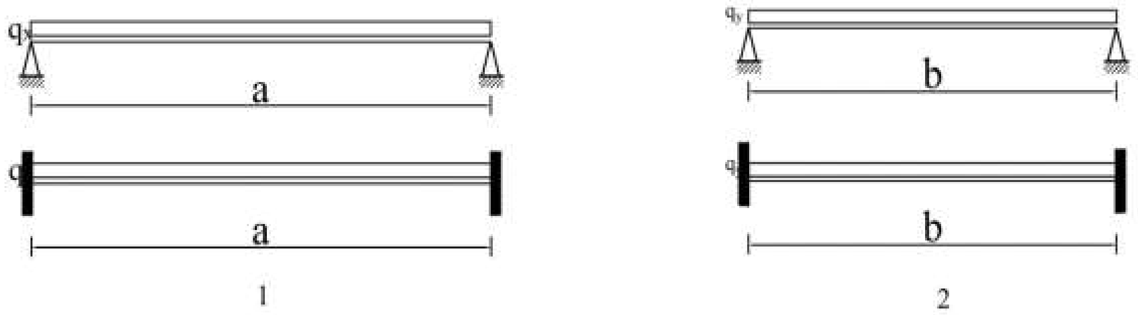


Figure 2.4. Mechanical scheme of band in x direction 1 and mechanical scheme of band in y direction 2 for one panel

a) Grashof principle of analysis for several slab panel

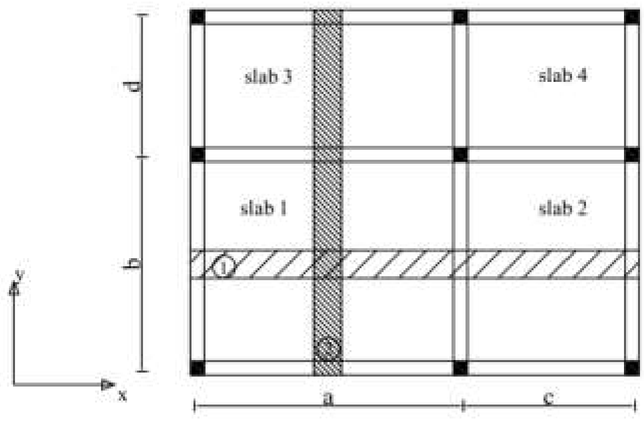


Figure 2.5. Grashof model for multiple panel

For a conventional two-way slab with several slab panel as in figure 2.5, with all the lengths a,b,c,d>>e, the value of each coefficient of the different slab panel is calculated and the average is used for the design. For stand one in the x direction, grashof coefficient as displayed in table 2.2.

Table 2.2. Computing grashof in the x direction for multiple panel

panel	Length(x) /m	Length(y) /m	$\alpha_x = \frac{y^4}{x^4 + y^4}$	$\alpha_y = \frac{x^4}{x^4 + y^4}$
Slab 1	a	b	$\alpha_{x1} = \frac{b^4}{a^4 + b^4}$	$\alpha_{y1} = \frac{a^4}{a^4 + b^4}$
Slab 3	a	d	$\alpha_{x1} = \frac{d^4}{a^4 + d^4}$	$\alpha_{y2} = \frac{a^4}{a^4 + d^4}$
average			$\alpha_x = \frac{\alpha_{x1} + \alpha_{x2}}{2}$	$\boldsymbol{\alpha_y = \frac{\alpha_{y1} + \alpha_{y2}}{2}}$

For strand two in the y-direction, grashof coefficient are computed as explained in table 2.3.

Table 2.3. Computing grashof coefficient in the y direction for multiple panels

panel	Length(x) /m	Length(y) /m	$\alpha_x = \frac{y^4}{x^4 + y^4}$	$\alpha_y = \frac{x^4}{x^4 + y^4}$
Slab 1	a	b	$\alpha_{x1} = \frac{b^4}{a^4 + b^4}$	$\alpha_{y1} = \frac{a^4}{a^4 + b^4}$
Slab 2	c	b	$\alpha_{x1} = \frac{b^4}{c^4 + b^4}$	$\alpha_{y2} = \frac{c^4}{c^4 + b^4}$
average			$\boldsymbol{\alpha_x = \frac{\alpha_{x1} + \alpha_{x2}}{2}}$	$\alpha_y = \frac{\alpha_{y1} + \alpha_{y2}}{2}$

The above highlighted values are the values used in the x and y directions respectively and the static schemes that complies with the above, used for the design is given by figure 2.6.

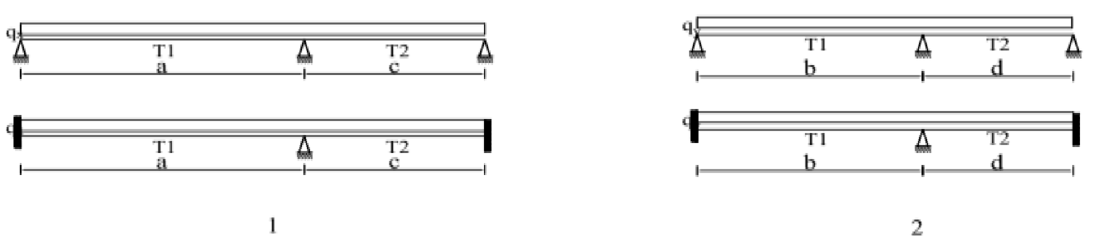


Figure 2.6. Mechanical scheme of band in x direction 1 and mechanical scheme of band in y direction 2 for multiple panels

2.2.2.3. Ultimate Limit State design

Under ULS design, the design is done for bending moment, shear force and axial force but since there is no axial force present, verifications will be done for bending moment and shear for the hollow block slab and only bending moment for the conventional two way slab.

a) Bending moment design

Eurocode 2 prescribe a reduction of the bending moment at support for the envelope curve solicitations obtained from solicitations curve as represented by figure 2.7. The value of the reduction is function on the connection between the elements.

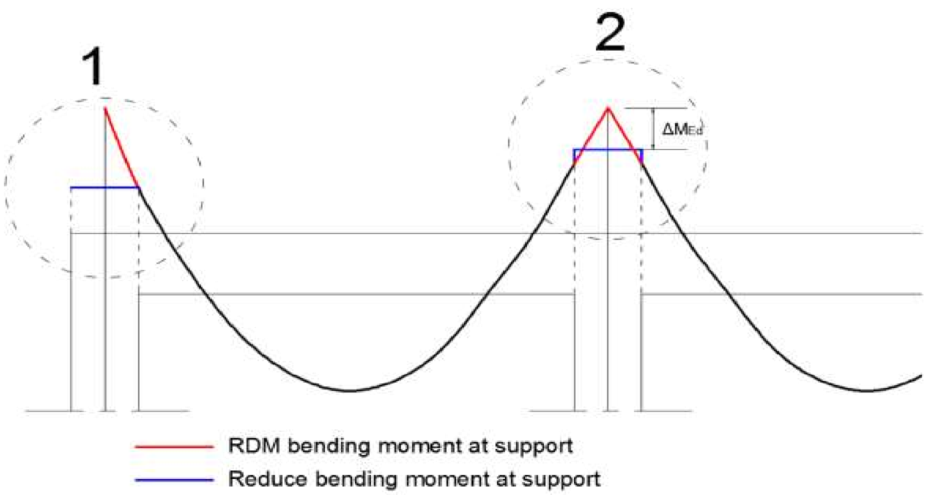


Figure 2.7. Reduction of the bending moment at support (Djeukoua, 2019)

(1) Where the element which is monolithic with its supports, the critical design moment at the support should be taken as that at the face of the support.

(2) Where the element is continuous over a support, the analysis is done considering that the support do not provide no rotational restraint. The amount of the reduction s given by:

$$\Delta M_{ED} = \frac{F_{ED,sup} t}{8} \quad (2.12)$$

Where

t: is the breadth of the support

$F_{ED,sup}$: is the design support reaction

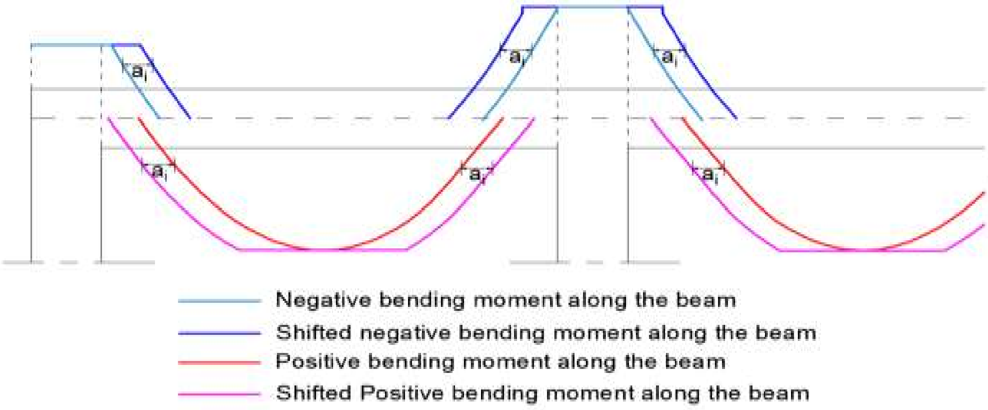


Figure 2.8. Illustration of the shifting rule (Djeukoua, 2019)

The final curve is obtained by shifting the moment curve of distance ai in the worst direction as presented in the figure to take into account the additional tensile force.

The shifting distance is defined by equation 2.13.

$$a_i = d \tag{2.13}$$

Where

d is the effective depth

The bending moment solicitations obtained, the determination and the verification of the steel reinforcement can be done.

i) Longitudinal steel reinforcement of the slabs

Steel reinforcement is computed for the section of rib (for a hollow block slab) and a rectangular section (1m strand for a conventional two way slab as display in figure 2.9.

The longitudinal reinforcement at each point is computed using equation 2.14.

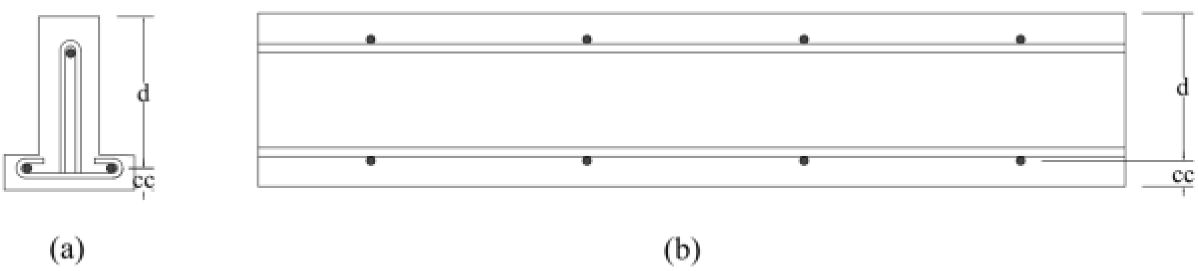


Figure 2.9. Example of reinforcement in (a) rib slab and (b) two way slab

$$A_S = \frac{M_{ed}}{0.9 \times d \times f_{yd}} \tag{2.14}$$

The section of steel obtained has to verify the provision in Eurocode 2 on the maximum and minimum longitudinal reinforcement described by equation 2.15 and 2.16 respectively.

$$A_{s,min} = max\left(0.26\frac{f_{ctm}}{f_{yk}}b_t d; 0.0013 b_t d\right) \tag{2.15}$$

$$A_{s,max} = 0.04A_c \tag{2.16}$$

Where:

b_t: is the mean width of the tension zone

d: is the effective depth of the section

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

f_{yk}: is the characteristic strength of steel

f_{yd}; is the design yielding strength of steel

ii) Verification of the steel reinforcement

The verification of the section is done by calculating the resisting bending moment of the section using the position of the neutral axis inside the section.The position of the neutral axis is computed using the equation 2.17.

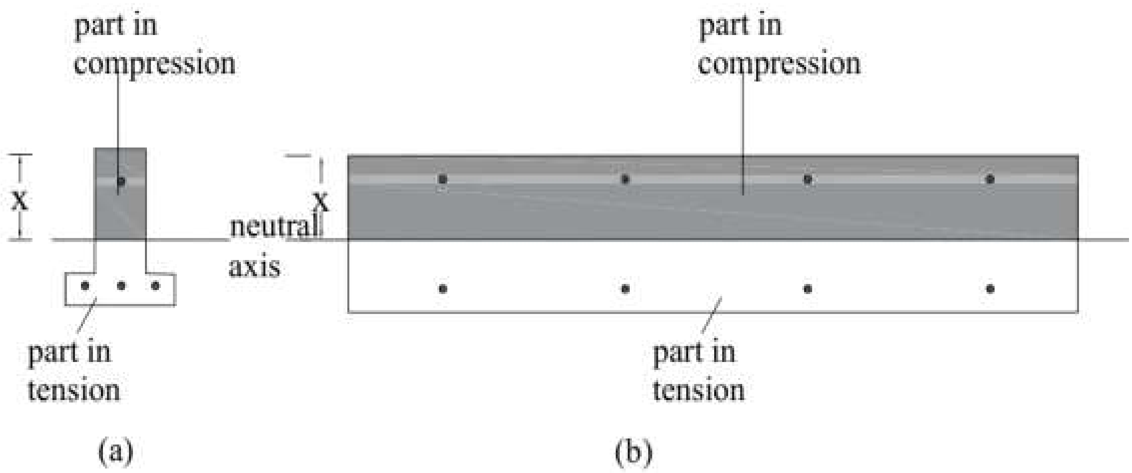


Figure 2.10. Compression and tension section in (a) rib slab and (b) two way slab

$$x = \frac{d}{2\beta_2} - \sqrt{\left(\frac{d}{2\beta_2}\right)^2 - \frac{M_{ED}}{\beta_1\beta_2 b f_{cd}}} \tag{2.17}$$

Where

d: is the effective depth of the section

b: is the width of the section

f_{cd}: is the design compressive strength of the concrete

β_1: is a correction factor equal to 0.81

β_2: is a correction factor equal to 0.41

The resisting moment is computed using the equation 2.18:

$$M_{RD} = A_{s,provided} f_{yd}(d - \beta_2 x) \tag{2.18}$$

Where

$A_{s,provided}$: is the effective area of the steel reinforcement

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

b) Shear design

Transversal reinforcement needs to be inserted in the rib to overtake shear forces present as illustrated in figure 2.11.

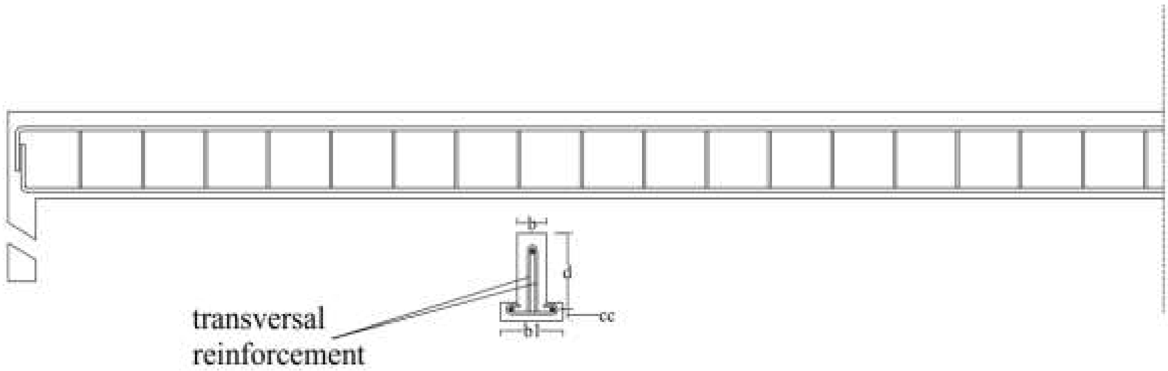


Figure 2.11. Illustration of transversal reinforcement in a rib

From the envelope curve of shear solicitations, the design shear value V_{ED} is obtained and used for different calculations. The necessity of shear reinforcement is computed by comparing the design shear value V_{ED} with the normalised shear capacity of concreteV_{Rdc}. If no shear reinforcement necessary, the conditions described in equation 2.19 should be satisfied.

$$V_{ed} < V_{Rdc} \tag{2.19}$$

With

$$V_{\mathrm{Rdc}} = \max\begin{cases}[\mathrm{C_{Rdc}} \mathrm{k}(100\rho_1 \mathrm{f_{ck}})^{1/3} + k_1\sigma_{\mathrm{cp}}]b_w d \\ [0.035k^{3/2}\mathrm{f_{ck}}^{1/2} + k_1\sigma_{\mathrm{cp}}]b_w d\end{cases} \tag{2.20}$$

Where:

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

$k_1 = 0.15$

$\sigma_{\mathrm{cp}} = \frac{N_{ED}}{A_C} < 0.2f_{cd}$ Which represent the stress due to axial force

d: is the effective depth of the section

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

$\rho_1 = \frac{A_{provided}}{b_w d} \leq 0.02$: reinforcement ratio

$\mathrm{C_{Rdc}} = \dfrac{0.18}{\gamma_c}$

When equation 2.19 is satisfied, the minimum reinforcement is used. Otherwise, the resisting strength is computed using equation 2.23.

$$\theta = \frac{1}{2}\arcsin\left(\frac{2V_{ED}}{0.9\alpha_{cw}f_{cd}v_1 d b_w}\right) \tag{2.21}$$

$$\frac{A_{sw}}{s} = \frac{V_{ED}}{0.9 d f_{ywd} ctg\theta^2} \tag{2.22}$$

$$V_{\mathrm{Rd}} = \min(0.9 d b_w \alpha_{cw} f_{cd} v_1 \frac{(ctg\alpha + ctg\theta)}{1 + ctg^2\theta}; \frac{A_{sw}}{s} z f_{ywd} cot\theta \tag{2.23}$$

Where

V_{Rd}: Resisting shear

A_{sw}: Area of the web reinforcement

f_{ywd}: design yield stress of the shear reinforcement

θ: Inclination of the cracks or the concrete struts

α: Angle between the shear reinforcement and the axis of the design element (α = 90° for stirrups)

f_{cd}: design value concrete compressive strength

s: stirrup spacing

α_{cw}: Coefficient of interaction between compressive stresses which can be assumed = 1 for non pre stressed structures.

ν_1: Reduction coefficient for shear cracked concrete ($\nu_1 = 0.6$ for $f_{ck} \leq 60\ N/mm^2$).

2.2.2.4. Serviceability Limit State Verification

SLS correspond to conditions beyond which specified service requirements for a structure or structural member are no longer met. The main SLS verifications for buildings are stress limitation, the crack and the deflection control.

Only the stress limitation will be described in this work. The verification of the allowable stress on the beam is done at the characteristic (rare) combination and permits to avoid inelastic deformation of the reinforcement and longitudinal cracks in concrete. The stress value is function of the coefficient of homogenisation in short terms and long terms as expressed in equations 2.24 and 2.25 respectively.

$$n_0 = \frac{E_s}{E_c} \tag{2.24}$$

$$n_\infty = n_0(1 + \varphi_L \times \rho_\infty) \tag{2.25}$$

Where $\rho_\infty = 2 \div 2.5$ and $\varphi_L = 0.55$ for shrinkage of concrete. Then the neutral axis for an uncracked section is computed as presented in equation 2.26:

$$x = \frac{-n(A_S' + A_s) + \sqrt{[n(A_S' + A_s]^2 + 2bn(A_s'd' + A_sd)}}{b} \tag{2.26}$$

With A's and As are the upper and lower steel reinforcement inside the section respectively. b, d' and d, are the geometrical characteristics of the section presented in the figure 2.12. For scheme (b), the value of b is 1m.

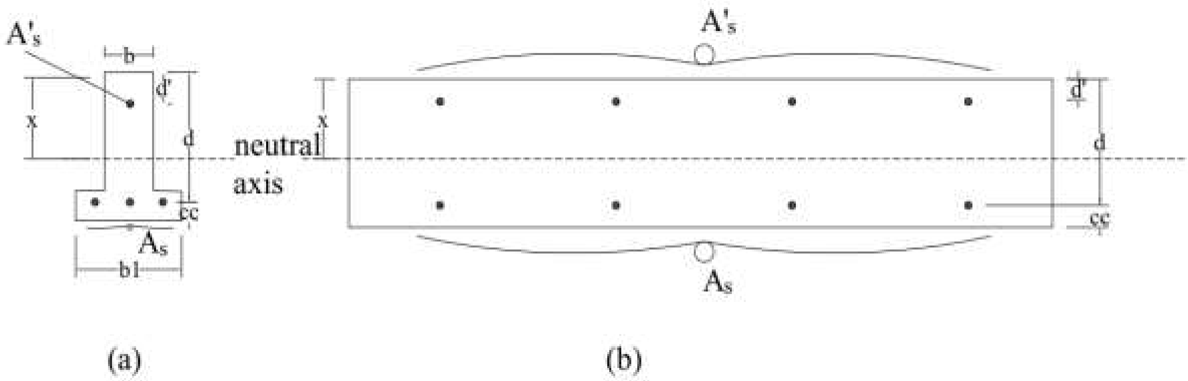


Figure 2.12.Transversal section of a reinforced (a) ribbed slab and (b) two way slab

The moment of inertia of the uncracked section is defined by equation 2.27.

$$J_{cr} = \frac{bx^3}{3} + nA_s(d-x)^2 + nA_s'(x-c)^2 \quad (2.27)$$

The values of the stresses in concrete and steel are respectively computed via equations 2.28 and 2.29.

$$\sigma_c = \frac{M_{ED}}{J_{cr}} \quad (2.28)$$

$$\sigma_s = \frac{M_{ED}}{J_{cr}} n(d-x) \quad (2.29)$$

The above stresses in stress and in concrete are verified following Eurocode 2 prescription defined by equation 2.30 and 2.31.

$$\sigma_c \leq k_1 f_{ck} \quad (2.30)$$

$$\sigma_s \leq k_1 f_{yk} \quad (2.31)$$

With $k_1 = 0.6$ and $k_2 = 0.8$

2.2.3. Beam design

Unlike slab design, beam design is composed of an Ultimate Limit State (ULS) design and a Serviceability Limit State verification (SLS).

2.2.3.1. Ultimate limit state

ULS design of the beam is done for the bending moment and the shear force since no axial force will applied in our case study

a) Bending moment design

The approach is the same as the early described. From the envelope curves for different solicitations, the value of the bending moment at different points of the beam are obtained and reduction is done with a value described by equation 2.32. But here, the shift of the envelope is through a distance a_i defined by the equation

$$a_i = \frac{z(cot\theta - cot\alpha)}{2} \quad (2.32)$$

Where:

z: is the inner lever arm

θ: is the is the angle of concrete compression struts to the beam axis

α: is the angle between shear reinforcement and the beam axis perpendicular to the shear force

i) Longitudinal steel reinforcement of the beams

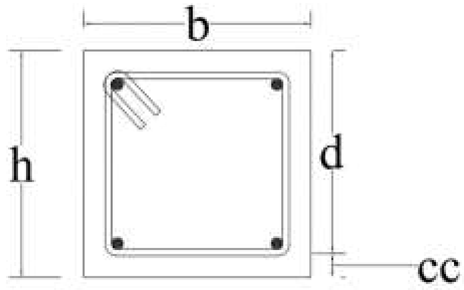


Figure 2.13. Example of reinforced section of beam

Knowing the geomentrical characteristics of the beam as displayed in figure, the quantity of reinforcement at each point can be calculated from envelope curve of bending moment using equation 2.14. The section of reinforcement obtained is verified to not be beneath or exceed respectively the minimum and maximum area of reinforcement as prescribed by Eurocode 2 using the equation 2.15 and 2.16 respectively.

i) Verification of the steel reinforcement

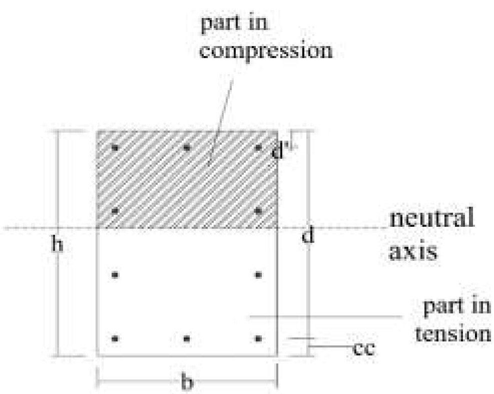


Figure 2.14. Illustration of tension and compression part in a reinforced beam section

The design section has to be verified for the longitudinal reinforcement by calculating the value of the resisting moment and comparing with the design moment value to assure that the section can resist to the oncoming bending moment solicitations. The neutral axis and the resisting moment are computed using equations 2.17 and 2.18 respectively.

b) Shear design

Shear solicitation are equally obtained from the envelop curve solicitation. Shear if designed by providing transversal reinforcement (stirrups) to counteract this effect. Figure shows the disposition of transversal reinforcement in a beam section.

Shear verifications done following the steps in equations 2.19, 2.20 and 2.23

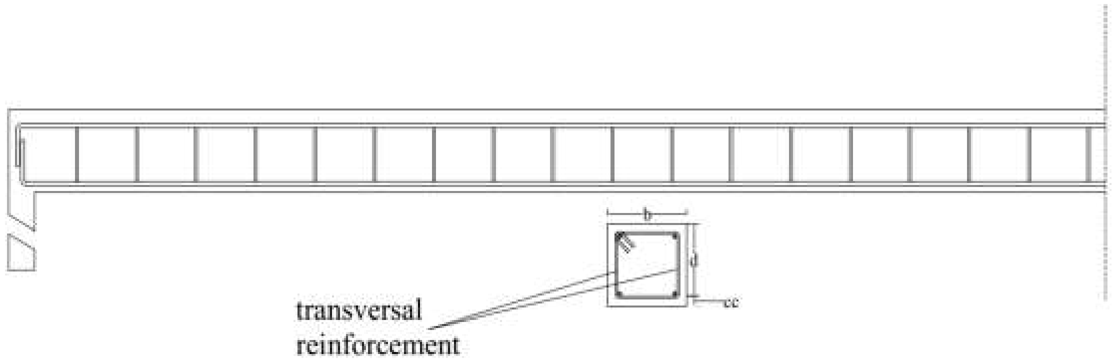


Figure 2.15. Example of transverse reinforcement in a beam

2.2.3.2. Serviceability Limit State Verification

Beam differs from slab as early explained above just from its cross section. So the stress verification here is computed in the same manner as detailed by equations 2.24, 2.25, 2.26, 2.27, 2.28 and 2.29 through the computation of short-term , long-term coefficient of homogenization, long-term neutral axis, moment of inertia of uncracked section, stress in concrete and stress in steel respectively. They are computed using the geometrical characteristics in figure 2.16. the stress in steel and concrete is verified using equation 2.30 and 2.31.

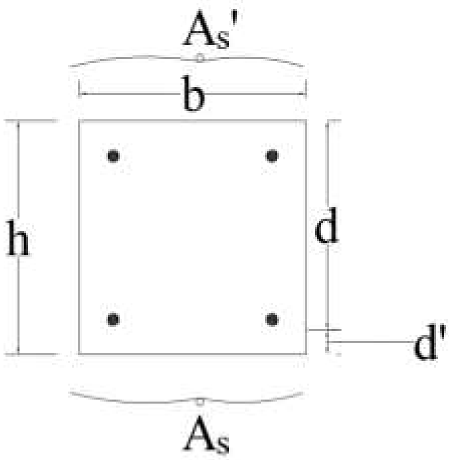


Figure 2.16. Beam section with different characteristics

2.2.4. Column design

Column design is done at ULS for the axial force, the bending and the shear force and the verification is done for the slenderness. The solicitations required for the design are obtained from the 3D model. In this work, the building will be modelled using the software SAP2000 V22.

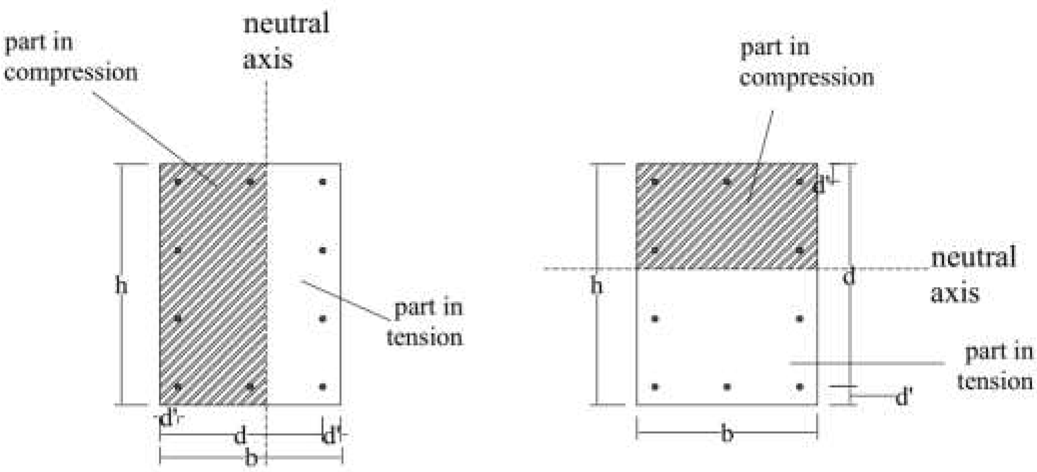


Figure 2.17. Rectangular section to illustrate the computation of the M-N diagram for different

2.2.4.1. Bending moment-axial force verification

The envelope of the bending moment and the axial force solicitations obtained, the design is done through the M-N interaction diagram. The column strength interaction diagram is a curve of plot points in which each point has two ordinates. The first ordinate is bending moment strength and the second is the corresponding axial force. For each level, we have to ensure that the maximum M-N solicitation belong to the M-N interaction diagram of the section considered. The section used to compute the m-n interaction diagram at different point are as follows.

a) First point

The section is completely subjected to tension; hence, the concrete is not reacting. We impose $\varepsilon_s = \varepsilon_{su}, \varepsilon'_s = \varepsilon_{syd}$ then $\sigma_s = \sigma'_s = f_{yd}$. The limit axial force and bending moment are obtained from the equations 2.33 and 2.34 respectively.

$$N_{Rd} = f_{yd}A_s + f_{yd}A'_s \tag{2.33}$$

$$M_{Rd} = f_{yd}A_s\left(\frac{h}{2} - d'\right) - f_{yd}A'_s(\frac{h}{2} - d') \tag{2.34}$$

b) Second point

The section is completely subjected to tension; hence, the concrete is not reacting. We impose $\varepsilon_s = \varepsilon_{su},\ \varepsilon_c = 0$. The upper steel has to be verified if yielded or not. The value of the limit axial force and bending moment at this point are computed using equation 2.33 and 2.34.

c) Third point

We impose now failure due to concrete and the lower reinforcement is yielded. $\varepsilon_s \geq \varepsilon_{syd},\ \varepsilon_c = \varepsilon_{cu}$ Then, computing the neutral axis, N_{Rd} and M_{Rd} can be computed using respectively equations 2.35 and 2.36.

$$N_{Rd} = -\beta_1 bx f_{cd} + f_{yd}A_s - f_{yd}A'_s \tag{2.35}$$

$$M_{Rd} = f_{yd}A_s\left(\frac{h}{2} - d'\right) + f_{yd}A'_s\left(\frac{h}{2} - d'\right) + \beta_1 bx f_{cd}(\frac{h}{2} - \beta_2 x) \tag{2.36}$$

d) Fourth point

We impose that the failure is due to concrete and the lower reinforcement reaches exactly the yielding point, $\varepsilon_s = \varepsilon_{syd},$. As for the previous point, we determine the neutral axis position, N_{Rd} and M_{Rd} are determined using the equations 2.35 and 2.36 respectively.

e) Fifth point

We impose that the failure is due to concrete and the lower reinforcement reaches exactly $\varepsilon_s = 0$ then the neutral axis position is equal to the effective depth of the section. The limit axial force and bending moment id obtained from the equations 2.37 and 2.38.

$$N_{Rd} = -\beta_1 bx f_{cd} - f_{yd} A'_s \tag{2.37}$$

$$M_{Rd} = f_{yd} A'_s \left(\frac{h}{2} - d'\right) + \beta_1 bx f_{cd} \left(\frac{h}{2} - \beta_2 x\right) \tag{2.38}$$

f) Sixth point

We impose that concrete is uniformly compressed and assume the strains $\varepsilon_s = \varepsilon_c \geq \varepsilon_{c2}$. Axial force and bending moment are computed as below.

$$N_{Rd} = -bh f_{cd} - f_{yd} A'_s - f_{yd} A_s \tag{2.39}$$

$$M_{Rd} = f_{yd} A'_s \left(\frac{h}{2} - d'\right) - f_{yd} A_s \left(\frac{h}{2} - d'\right) \tag{2.40}$$

Eurocode 2 provide some limitation in the steel reinforcement as describe by equations 2.41 and 2.42

$$A_{s,min} = \max\left(\frac{0.10 N_{ED}}{f_{yd}}; 0.002 A_c\right) \tag{2.41}$$

$$A_{s,max} = 0.04 A_c \tag{2.42}$$

With

N_{ED}: design axial compressive force

f_{yd}: design yield strength

2.2.4.2. Shear design

Unlike hollow blocks slab and beam as early explained above, shear design of column follows the same procedures however, the detailing of members prescribed by Eurocode 2 imposed a minimum diameter of 6 mm or one quarter the maximum diameter of the longitudinal bars. Equally, the maximum spacing has to be reduced by a factor 0.6 in sections within a distance equal to the larger dimension of the column cross-section above or below the beam. This maximum spacing between the transversal reinforcement is defined by equation 2.43.

$$S_{cl,max} = \min(20\phi_{l,min}; b; 400mm) \tag{2.43}$$

Where

b: is the lesser dimension of the column

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

2.2.4.3. Slenderness verification

Structural deformations causes additional action (second order effects) to some element. Elements like column are more likely to be affected by second order effects hence the need of slenderness verification. It consists in verifying if the slenderness of the element is below a limit value, defined by the Eurocode 2 as expressed in equation 2.44.

$$\lambda_{lim} = \frac{20ABC}{\sqrt{n}} \tag{2.44}$$

Where

$A = \frac{1}{1+0.2\varphi_{ef}}$ (φ_{ef} is the effective creep ratio; A=0.7 if φ_{ef} is unknown)

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

$C = 1 - r_m$ (r_m is the moment ratio, $r_m = \frac{M_{01}}{M_{02}}$)

$n = \frac{N_{ED}}{A_c f_{cd}}$ (relative normal force)

The slenderness of an element is computed using equation 2.45.

$$\lambda = \frac{l_0}{i} \tag{2.45}$$

Where

l_0: is the effective length of the element

$i = \frac{I}{A}$ (i, I, and A represent the gyration radius of the uncracked section, the moment of inertia and the area of the section respectively.

2.3. Numerical modelling

The program SAP2000 V22 which is a structural design software through the finite elements method specially dedicated to the analysis of the stability and the resistance of structure is used to model this work. The static analysis to perform the static design of the structure and the modal analysis to extract the dynamic characteristics of the structure. Considering the fact that it is a comparative study, two models with a fixed base will be drawn all having the same parameters except the slab rigidity factor and two others (rigid and non-rigid slab) models will be drawn with a flexible base using soil structure interaction which is a more realistic base. The model is a frame resisting structure with one way slab. Beams and columns are modelled as frame structures, the slab charges are distributed uniformly on beams and the flexible base is modelled using thick shell. The connection between these elements are done using joints.

The rigid slab is modelled by applying diaphragm constraint in the z-direction at each storey level hence reducing the degree of freedom of a joint to three degrees of freedom (translation in the x-direction, translation in the y direction and rotation about the z-axis). On the other hand, the non-rigid slab model is modelled without diaphragm constraints at joints hence each joint has six degree of freedom (three translations and 3rotations).

The flexible base (foundation) is modelled by applying the notion of soil-structure interaction (SSI). SSI is taken into account via the application of the sub-structural approach where springs are inserted in the foundation with spring elasticity value computed from the modulus elasticity of the chosen soil as displayed in equation 2.46. This approach assumes linear soil and structural behaviour.

$$k = C \times A \qquad 2.46$$

Where

k Spring stiffness

C Modulus of subgrade reaction

A Area of meshed element

2.4. Seismic analysis methodology

As early mention in the literature review, slabs are the element that overtake and redistribute horizontal forces to the others structural element hence the necessity to do a seismic design. Beams and/or columns may be considered as secondary seismic members because they are not part of the seismic action resisting system.

2.4.1. Methods used

According to EC8 section 4.2.3.1, with regard to the implication on the structural regularity, different considerations are given to the regularity characteristics of building in plan and in elevation. The study of the seismic behaviour of an irregular building is performed better with the modal analysis in case of linear elastic analysis as shown in table 2.4.

Table 2.4. Requirement about structural regularity on seismic analysis and design

Regularity		Allowed simplifications		Behaviour factor(q)
Plan	Elevation	Model	Linear elastic analysis	For linear analysis
Yes	Yes	Planar (2D)	Lateral force	Reference value
Yes	No		modal	Decreased value
No	Yes	Spatial (3D)	Lateral force	Reference value
Yes	No		modal	Decreased value

Type of analysis used for seismic design can be a linear-elastic analysis (lateral force method analysis and modal response spectrum analysis) or non-linear static (non-linear static analysis or non-linear dynamic analysis). For this work, the modal response spectrum analysis will be performed for the rigid slab and the non-rigid slab.

2.4.2. Modal response spectrum analysis (linear dynamic analysis)

A linear dynamic analysis is used for evaluating irregular or dynamically complex buildings. Dynamic complexity is common in flexible structural systems. Flexibility is greatly influenced by the selection of structural system and building height. Linear dynamic analysis is used to determine the degree of influence each mode shape will have on a structure's performance (Taranath, 2013). The importance of higher modes depends on the relationship between the fundamental mode of the structure and the dynamic ground-shaking characteristics of the site the structure–ground shaking. Interaction is usually modelled using a response spectrum. Eurocode 8 section 4.3.3.3 includes a procedure for developing a designed response spectrum as well as this work.

2.4.2.1. Modal analysis

The structural response is the combination of many modes. A modal analysis is being carried out on a spatial model (3D) of the structures (one with rigid slab and the other with flexible slab) to determine the frequency and modes of vibration of the structure. Given that there exist as many modes of vibrations, the selection of mode of vibration to deal with is important has to satisfy two conditions as stated by Eurocode 8 section 4.3.3.3.1.

- The sum of the effective modal masses for the selected modes should be at least 90% of the total mass of the structure.
- All modes with effective modal masses greater than 5% of the total mass are takin into account.

Otherwise, the minimum number of modes to be considered in spatial analysis should satisfy the conditions defined in equation 2.47

$$\begin{cases} k \geq 3\sqrt{n} \\ T_k \leq 0.2s \end{cases} \tag{2.47}$$

Where

k is the number of modes taken into account

n is the number of storeys above the foundation or the top of a rigid basement

T_k Is the period of vibration of mode k

2.4.2.2. Modal combination of modal responses

Many methods exist in order to obtain a peak response quantity (displacements, velocities, acceleration or internal forces) for a multi degree of freedom system, among which the Square Root of the Sum of Squares (SRSS) and the Complete Quadratic Combination (CQC) which are the most common. The CQC method is more effective in evaluating the maximum response when the modal frequencies are close to each other. If two vibration modes have close frequencies, their contribution to the global response is not independent and is achieved by applying equation 2.48.

$$E_E = \sqrt{\sum_{n=1}^{m}\sum_{i=1}^{m}\rho_{in}E_{Ei}E_{En}} \tag{2.48}$$

However, Eurocode 8 part 1 set rules for the combination of modal responses. It states that the response in two modes i and j (including both translational and torsional modes) may be taken as independent to each other if their periods T_i and T_j satisfies the condition described by equation 2.49.

$$T_j \leq 0.9T_i \tag{2.49}$$

Whenever all relevant modal responses are independent, the maximum value of a seismic action effect is computed using the SRSS combination defined as in equation 2.50.

$$E_E = \sqrt{\sum E_{Ei}^2} \tag{2.50}$$

2.5. Analysis criteria

This study will be analysed using five important parameters, among which the gravity load distribution natural period of vibration, the lateral displacement, the inter story drift and the base shear. Hence for each case, model with rigid slab and model with non-rigid slab, all the aforementioned criterion will be analysed and compared.

2.5.1. Gravity load distribution

The two models will be modelled on SAP2000 for both fixed based and flexible base through soil structure interaction. The distribution of gravity forces will be studied for the four models and results compared. The bending moment, the shear, the axial force distribution will be analysed when equal gravity loads are been applied. As early mentioned, the loads applied for this case study will be applied considering one way distribution at slabs.

2.5.2. Period

The frequency of period of vibration of a structure is mainly affected by the mass, the stiffness, the height and the column orientation. The period of vibration is a native property of a building which is computed via the modal analysis. Modal analysis permits us to have the period and frequency for different modes and also give us a preview for different possible deformation. It is the first step for the dynamic studying of a structure. It is used to determine the structural ability of the building to resist to seismic load. For this work, the modes, the mass participating ratios will be computed directly from the structural analysis program SAP2000 for both rigid and non-rigid models under fixed base and modelled foundation.

2.5.3. Lateral displacement

Resulting from horizontal forces (seismic load in our case) and p-delta effects for high building. It permits also a proper estimation of the separation distance between buildings. In the case of this study, the p-delta (second order) effects will be neglected. Figure 2.14 shows the displacement of the last storey of a building. This effect has to be mitigated in structures to avoid damage.

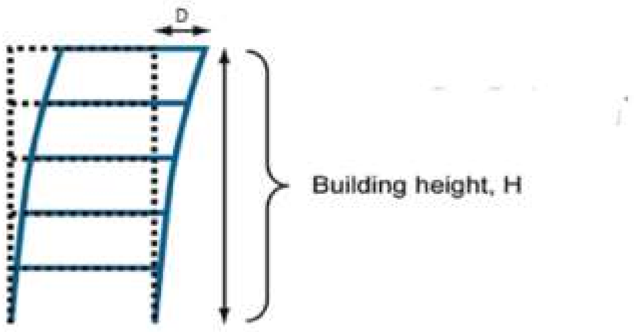


Figure 2.18. Illustration of storey displacement

2.5.4. Inter-storey drift

It represents the relative displacement of top floor compared to the bottom floor. Inter-storey drift represents the most important parameter to be analysed as it is strictly connected to the damage suffered by both structural and non-structural elements. The inter-story drift has been employed as an index to evaluate the deformation capacity of a building and to further determine its performance. For a proper design, Eurocode 8 sets some limitations according to this parameter as define in equations 2.51 and 2.52.

$$d_r V \leq 0.005h \tag{2.51}$$

$$d_r = d_{i+1} - d_i \tag{2.52}$$

Where:

d_r	is the inter-storey drift at (i+1) level
d_{i+1}	displacement at (i+1) level
d_i	displacement at (i) level
V	reduction factor
h	Storey height

2.5.5. Base shear

Base shear is an estimate of the maximum expected lateral force on the base of the structure due to seismic activity. It is an output of overall behaviour of the structure. The base shear is further distributed to each storey as storey shear. The value of the storey shear depends on the slab rigidity hence, the value of the base shear will be evaluated and compared in the case of model with rigid slab or the model with non-rigid slab for both foundations models.

Conclusion

In drawing to a close for this chapter which had as main objective the detail presentation of norms and a descriptive analysis of how the work will be carried out for both static analysis and dynamic analysis through the application of an elastic response spectrum. The case study will be modelled using SAP2000 V22 and all relative calculations will be performed on Microsoft excel while applying the European standard clearly detailed in this part. The results will be presented in the oncoming chapter.

CHAPTER 3 RESULTS PRESENTATION AND INTERPRETATION

Introduction

The above chapter clearly explained step by step the methods used. This present part is aimed at present the results obtained following the defined method. The presentation of the case study, the material used, loads applied and combinations used are detailed in the first part of this section. Moreover, the second part of this section presents the static design of the case study via design of two types of slabs, design of beam for a one-way slab and design of column. Lastly, after applying the response, the results for different case study will be explained in the third part.

3.1. Presentation of case study

Here, the case study is described, the properties of material used, loads and load combinations are presented.

3.1.1. Description of case study

The buildings conceived to be meet the necessary requirement is a G+5 residential building hence category A from Eurocode classifications. The structure is irregularity in plan is a geometrical irregularity with an L-shape floor plan with a degree of protrusion of 35%. The building is regular is elevation with no setbacks or irregular distribution of mass in elevation. The total height of the structure is 18m with a constant inter-story height of 3m and a total surface area of 370m^2 in plan.

3.1.2. Material properties

The super structure been made of reinforced concrete, the concrete class used is C25/30 and for the reinforcement, Fe450B is used. The others concrete and steel properties are detailed in table 3.1 and table 3.2 respectively.

Table 3.1. Concrete characteristics

Definition	Property	Value	Unit
Concrete class	class	C25/30	
Characteristic compressive strength at 28 days	f_{ck}	25	N/mm^2
Partial safety factor	γ_c	1.5	

Definition	Property	Value	Unit
Design value of compressive strength	$f_{cd} = \frac{\alpha_{cc} f_{ck}}{\gamma_c}$	14.6	N/mm^2
Mean value of axial tensile strength of concrete	$f_{ctm} = 0.3(f_{ck})^{2/3}$	0.87	

Table 3.2. Steel reinforcement characteristics

Definition	Property	Value	Unit
Steel class	class	450B	
Characteristic yield strength	f_{yk}	450	N/mm^2
Partial safety factor for steel	γ_s	1.15	
Design value of tensile strength	$f_{yd} = \frac{f_{yk}}{\gamma_s}$	391.30	N/mm^2

3.1.3. Loads on the building and load combination

3.1.3.1. Loads

The vertical loads acting on the building are presented as follows.

a) Permanent loads

The loads applied on the building are structural and non-structural. The self-weight of the slab is a structural permanent load (G_1) and others are non-structural loads (G_2).

Table 3.3. Permanent loads applied

Designation	Symbol	Value (kN/m^2)
Hollow-block slab (16+4)	G_1	2.85
Reinforced concrete slab (x cm thickness)	G_1	x*0.25
Cement mortar screed (2cm)	G_{2k1}	2*0.2
Sand layer (2cm)	G_{2k2}	2*0.18
Ceramic stone ware(1cm)	G_{2k3}	0.22
Walls on slab	G_{2k4}	1.00
Walls of 15*20*40 with cm mortar	G_{wall}	2.5

b) Variable loads

i) Imposed load

The building been meant for residential purposes (category B), the value of the imposed loads Q_k is chosen to be 1.5 kN/m^2. Floor 1 to 5 have 1.5 kN/m^2 as imposed load. The last floor is not accessible except for maintenance (use category H) so the imposed load can be taken as 1.0 kN/m^2.

ii) Seismic load

The seismic load applied in this work will be applied as a ground acceleration corresponding to the horizontal component of the elastic response spectrum. The properties of the spectrum is defined in table 3.4 and illustrated by figure 3.1.

Table 3.4. Seismic parameters

Ground	Type C
Horizontal response spectrum	Type 1
Building class	Residential building
Soil factor	1.2
Peak ground acceleration (a_{gR})	0.25g
Importance factor	1.0
Damping ratio	5%

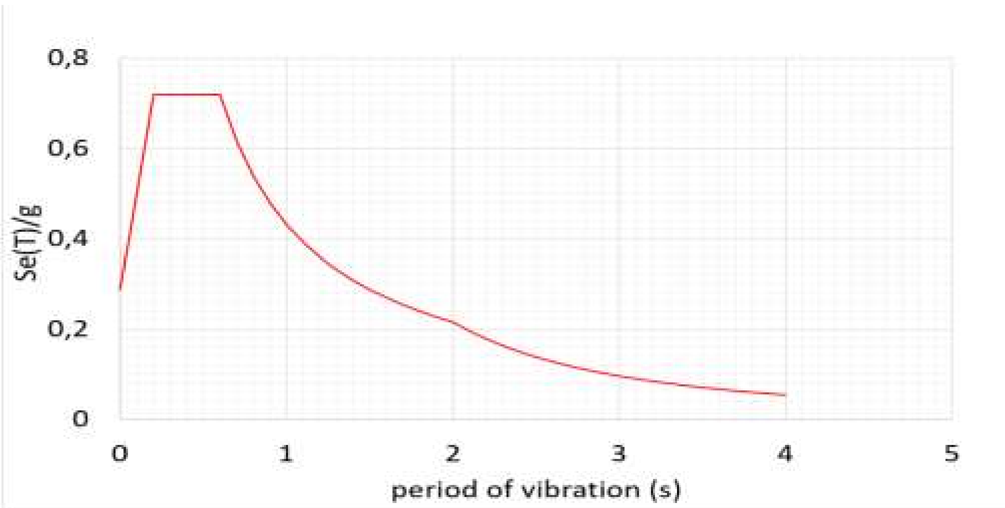


Figure 3.1. Horizontal component of elastic response spectrum

3.1.3.2. Loads combinations

The fundamental load combination used for the design at ULS is define by equation

$$1.35G_1 + 1.35G_2 + 1.35G_{wall} + 1.35G_{roof} + 1.5Q_{roof} + 1.5Q_K \quad (3.1)$$

$$G_2 = G_{k1} + G_{k2} + G_{k3} + G_{k4} \quad (3.2)$$

As for irreversible serviceability limit state, the characteristic combination (RARE) is used.

$$G_1 + G_2 + G_{wall} + G_{roof} + Q_{roof} + Q_K \quad (3.3)$$

The combination used for the seismic analysis is:

$$G_1 + G_2 + G_{roof} + E + 0.3Q_k + 0.3Q_{roof} \quad (3.4)$$

Where E is action effect due to the combination of the effects of the horizontal component of the seismic action. This action is computed using the combination. The first two combination are displayed by the equations 3.5 and 3.6. Ex and Ey represent the seismic load in the x and y direction respectively.

$$E_x + 0.3E_y \quad (3.5)$$

$$E_y + 0.3E_x \quad (3.5)$$

1.1. Static design of the case study

This section present the design of both horizontal components (slab and beam) and vertical components (column).

3.1.4. Durability and concrete cover

For a structural class S4 and exposure class XC1, following the procedure described in section 2.2.1, the nominal concrete cover following Eurocode 2 prescription is given by:

$$C_{min} = \max(20{,}15{,}10) = 20mm$$

$$C_{nom} = 20mm + 10mm = 30mm$$

So, the concrete cover will be considered 30mm for the designs.

3.1.5. Slab design

For this building, two types of slab will be design; a hollow block slab and a conventional two way slab.

3.1.5.1. Design of hollow block slab

a) Preliminary design of slab

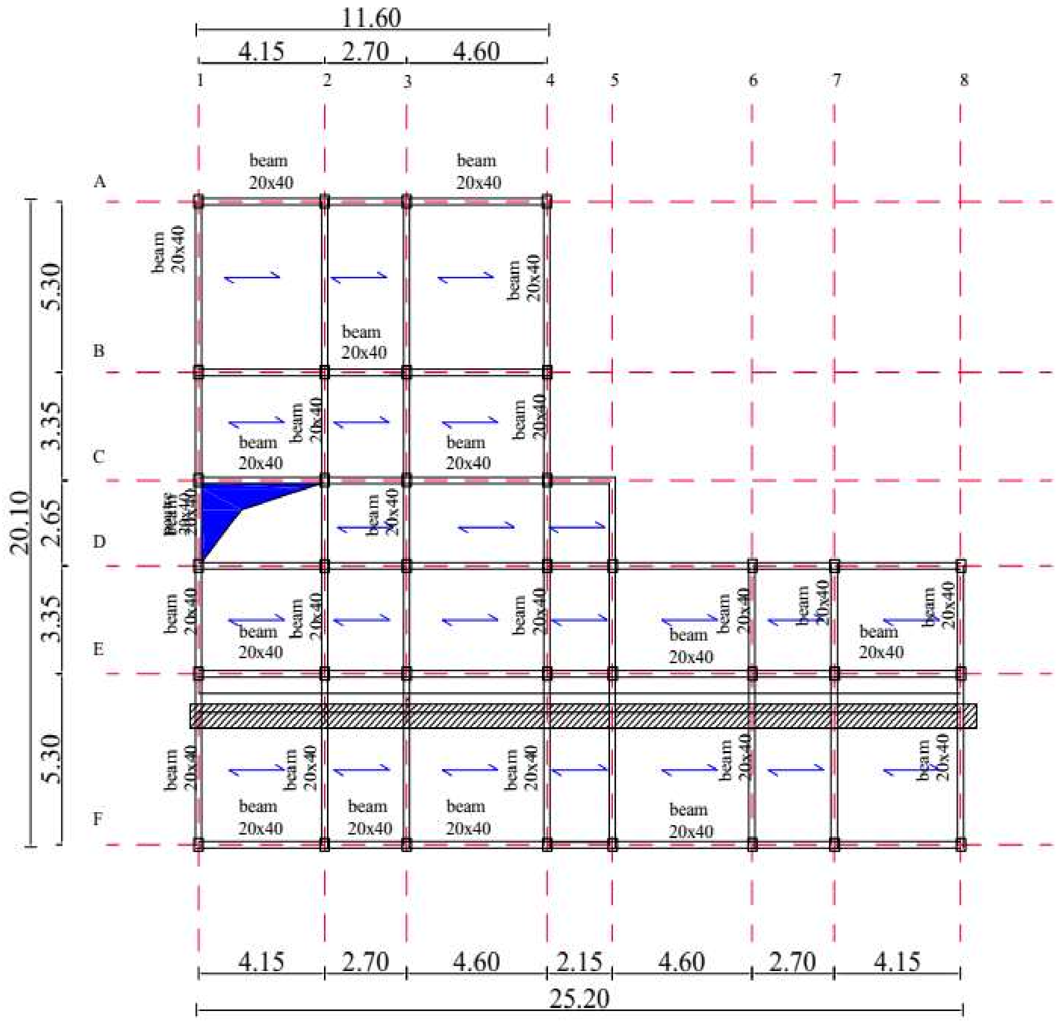


Figure 3.2. 2D plan illustrating the rib to design

$$L = min\begin{cases} Lmax\ x = 4.60 \\ Lmax\ y = 5.30 \end{cases} \quad L = 4.60m$$

The slab to be designed is illustrated in figure 3.2. The thickness of slab $e > \frac{L}{25} = 18.4cm$ so, 20 cm (16+4) slab will be used.

b) Load definition and load combinations

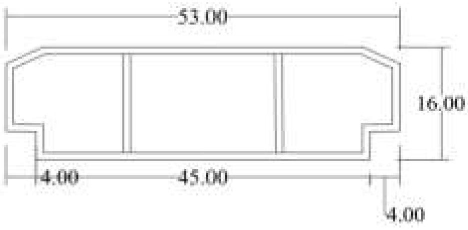


Figure 3.3. Transversal section of hollow blocks

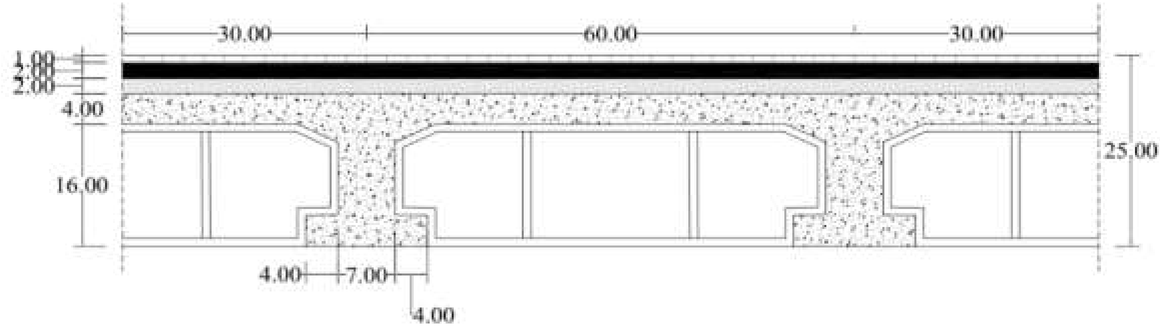


Figure 3.4. Disposition of different elements on slab

Taking into consideration the disposition of different element as explained in figure 3.3 and the transversal section in figure 3.4, the influence area of a rib is 0.6m and the different linear loads applied to the ribs are as defined in table 3.5.

Table 3.5. Uniform distributed loads on rib

Designation	Value (kN/m)
G_1	1.71
G_2	1.19
Q_k	0.90

Based on the notion of influence area, the designed rib is illustrated in the plan model. The rib can be design span by span using the mechanical scheme illustrated in figure 3.5 or as a continuous rib over 7 spans as in figure 3.6. The design will be done using the mechanical scheme in figure 3.5.

Figure 3.6. Mechanical scheme one span

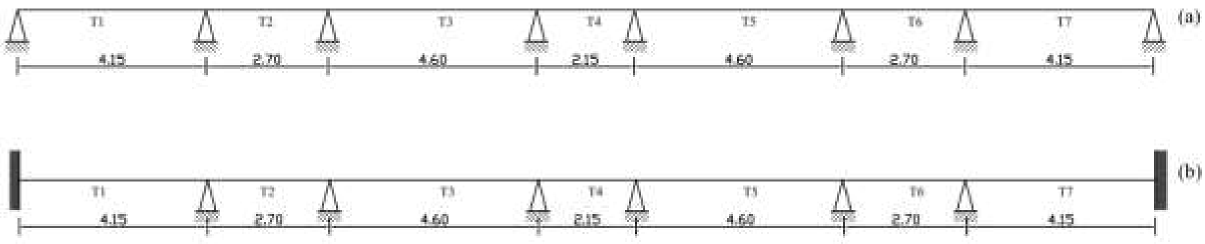


Figure 3.5. Mechanical scheme used to design the ribs

Taking into considerations the span length, the five possible combinations used to maximise moment and shear are defined from figure 3.7 to 3.11.

- Combination 1

$1.35\ (G_1T_1 + G_1T_2 + G_1T_3 + G_1T_4 + G_1T_5 + G_1T_6 + G_1T_7) + 1.35\ (G_2T_1 + G_2T_2 + G_2T_3 + G_2T_4 + G_2T_5 + G_2T_6 + G_2T_7) + 1.5\ (Q_kT_1)$

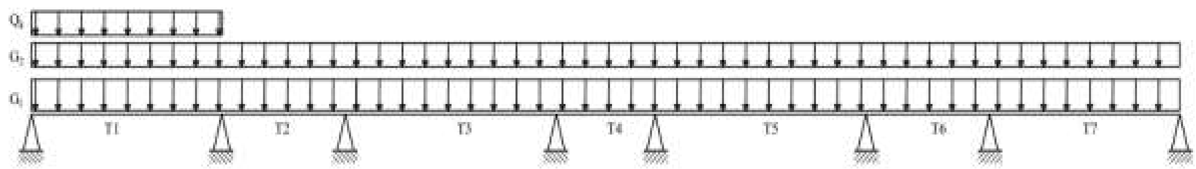


Figure 3.7. First combination ULS in rib

- Combination 2

$1.35\ (G_1T_1 + G_1T_2 + G_1T_3 + G_1T_4 + G_1T_5 + G_1T_6 + G_1T_7) + 1.35\ (G_2T_1 + G_2T_2 + G_2T_3 + G_2T_4 + G_2T_5 + G_2T_6 + G_2T_7) + 1.5\ (Q_kT_3)$

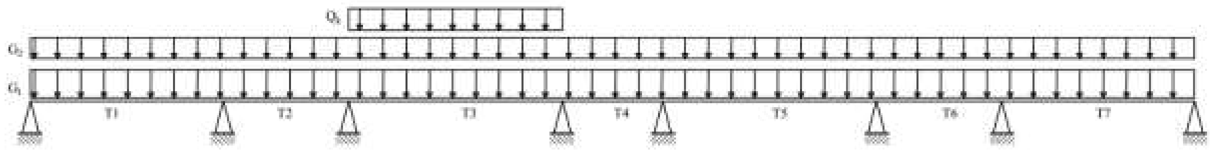


Figure 3.8. Second combination ULS in rib

- Combination 3

$1.35\ (G_1T_1 + G_1T_2 + G_1T_3 + G_1T_4 + G_1T_5 + G_1T_6 + G_1T_7) + 1.35\ (G_2T_1 + G_2T_2 + G_2T_3 + G_2T_4 + G_2T_5 + G_2T_6 + G_2T_7) + 1.5\ (Q_kT_1 + Q_kT_2 + Q_kT_3 + Q_kT_4 + Q_kT_5 + Q_kT_6 + Q_kT_7)$

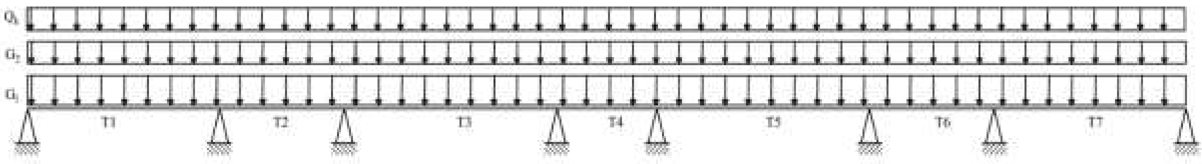


Figure 3.9. Third combination ULS in rib

- Combination 4

$1.35\ (G_1T_1 + G_1T_2 + G_1T_3 + G_1T_4 + G_1T_5 + G_1T_6 + G_1T_7) + 1.35\ (G_2T_1 + G_2T_2 + G_2T_3 + G_2T_4 + G_2T_5 + G_2T_6 + G_2T_7) + 1.5\ (Q_kT_1 + Q_kT_2 + Q_kT_3 + Q_kT_4 + Q_kT_5 + Q_kT_6 + Q_kT_7)$

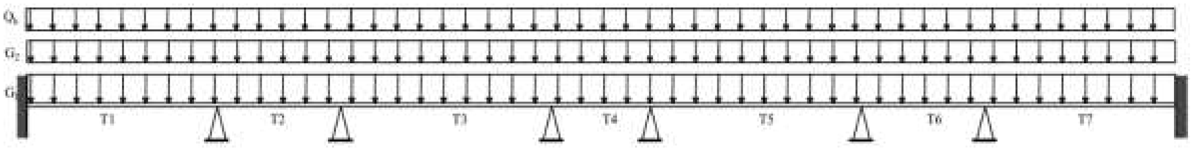


Figure 3.10. Forth combination ULS in rib

- Combination RARE

$(G_1T_1 + G_1T_2 + G_1T_3 + G_1T_4 + G_1T_5 + G_1T_6 + G_1T_7) + (G_2T_1 + G_2T_2 + G_2T_3 + G_2T_4 + G_2T_5 + G_2T_6 + G_2T_7) + (Q_kT_1 + Q_kT_2 + Q_kT_3 + Q_kT_4 + Q_kT_5 + Q_kT_6 + Q_kT_7)$

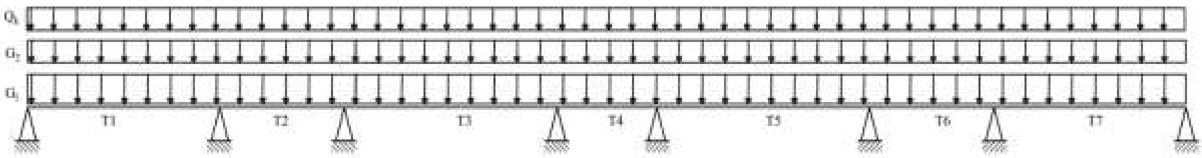


Figure 3.11. Combination SLS in rib

c) Solicitations

Solicitations are given in terms of moment and shear. The shear diagrams and bending moment diagrams for different combinations are displayed in figure 3.12 and figure 3.13 respectively.

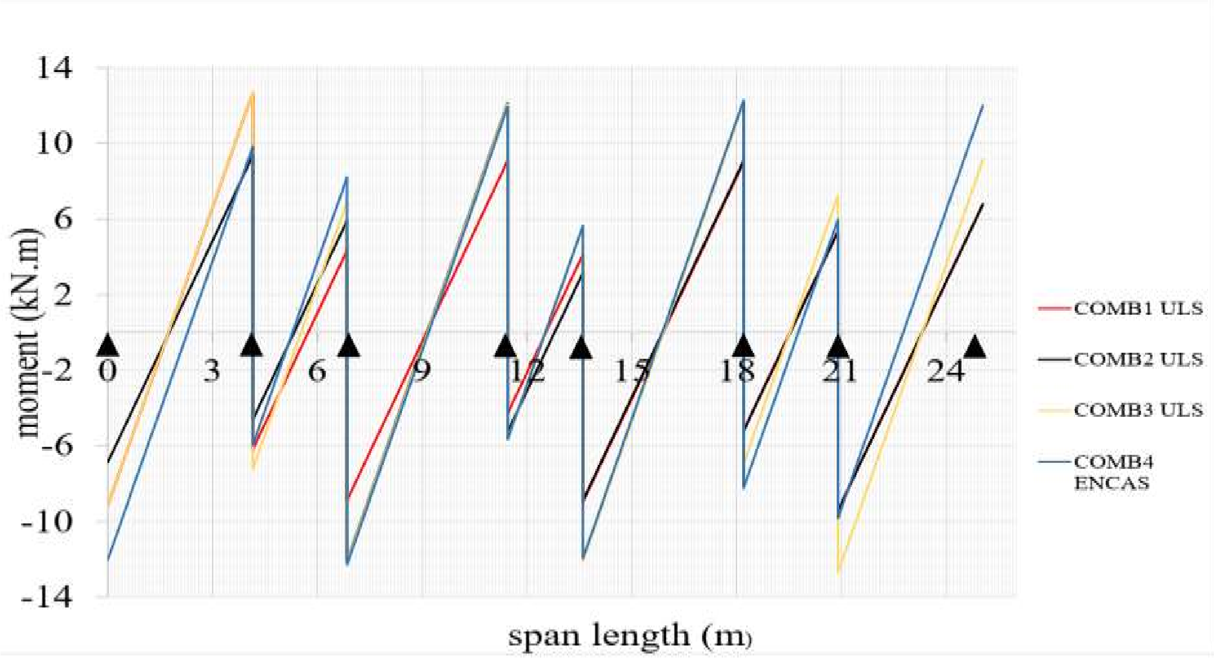


Figure 3.12. Shear diagrams in rib

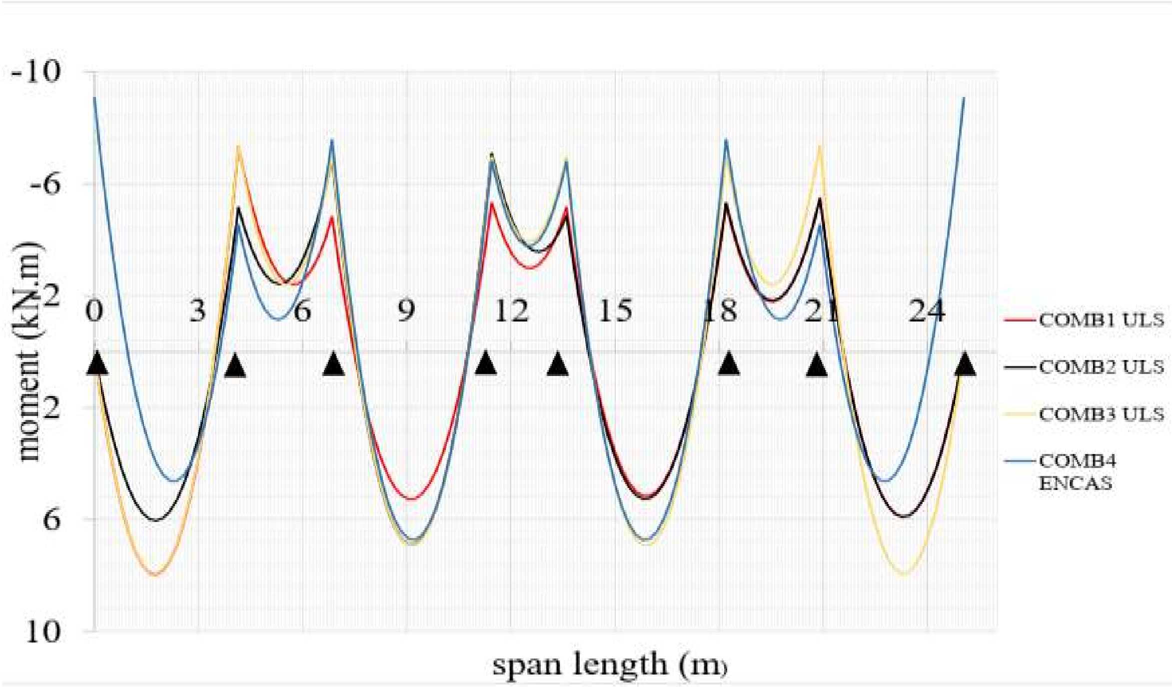


Figure 3.13. Moment diagrams in rib

The design value of sollicitations of bending moment and shear are obtained from the envelope curve which is display in figure 3.14 and 3.15.

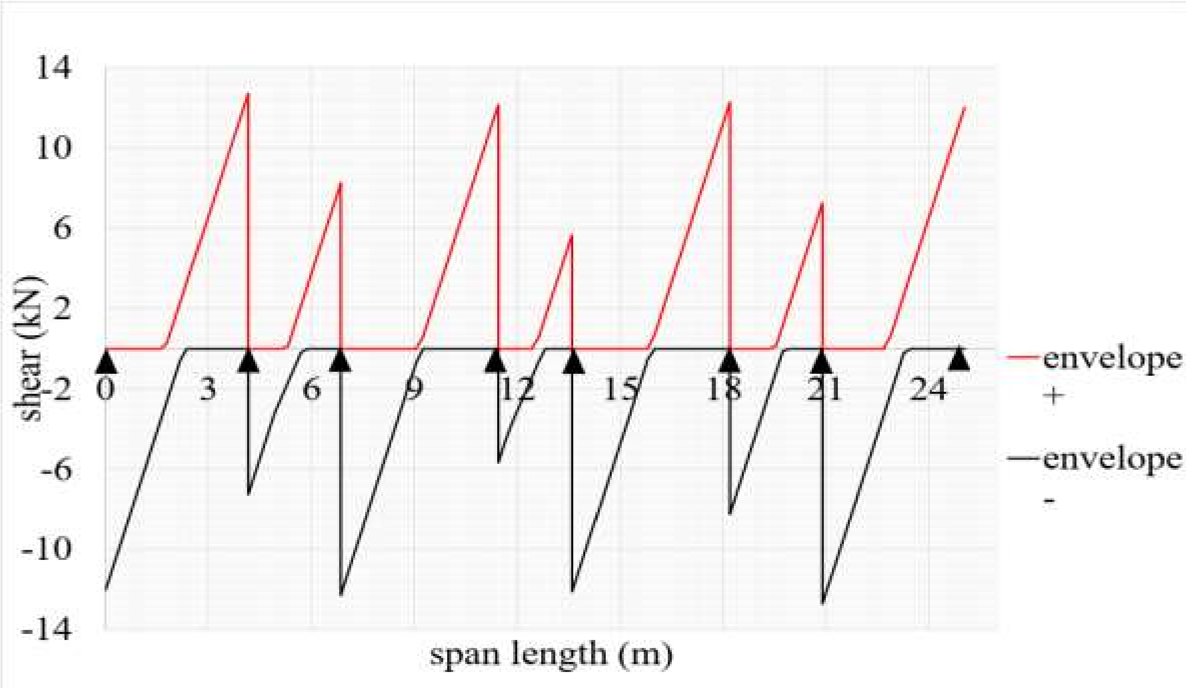


Figure 3.15. Envelope curve of shear in rib

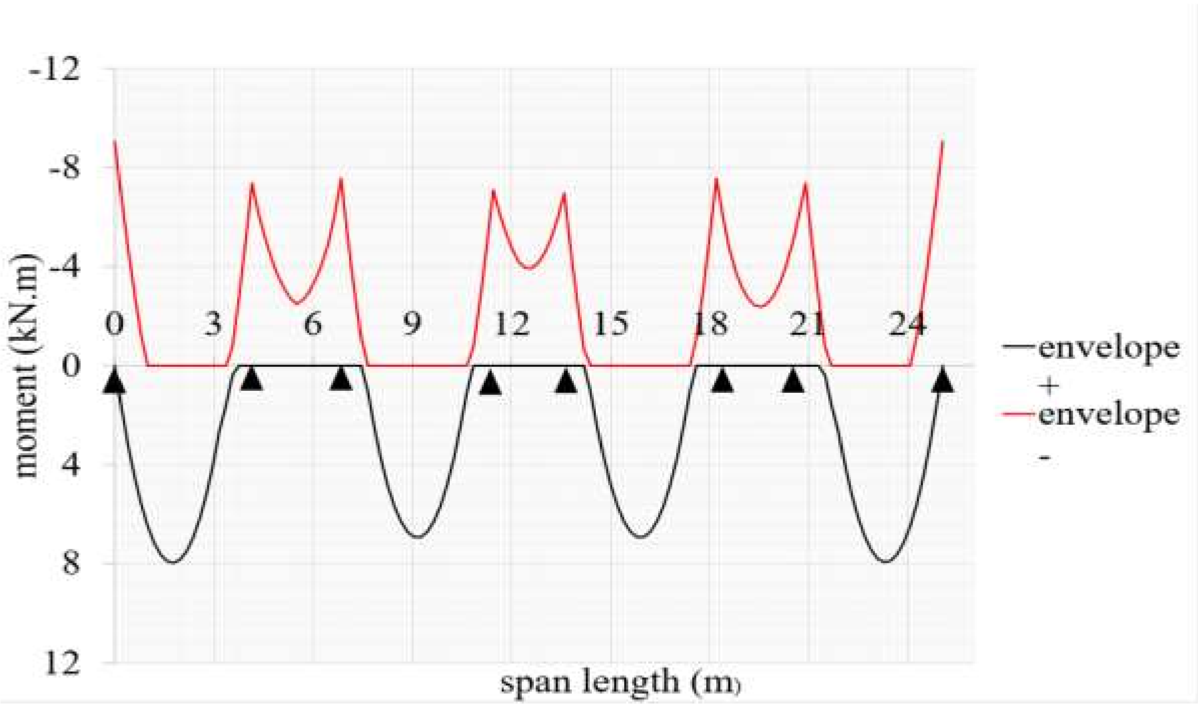


Figure 3.14. Envelope curve of bending moment in rib

The maximum values for moment deduced from the shifted envelope of bending moment figure 3.16 and shear for different parts of the rib are shown in table 3.6.

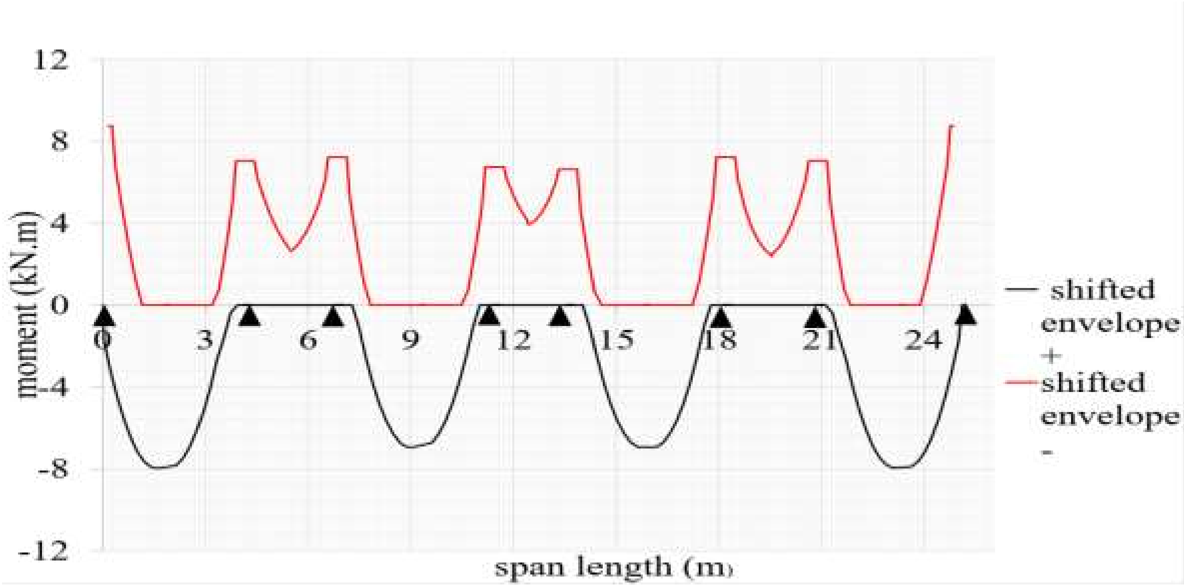


Figure 3.16. Shifted envelope curve for bending moment in rib

Table 3.6. Maximum solicitations at span and support for the rib

Location	Moment (kN.m)	Shear (kN)
Span	7.98	/
support	8.70	12.70
Span(SLS rare)	5.73	/

d) Calculations of steel bar reinforcement

i) Longitudinal reinforcement

The section considered is figure3.17 and concrete and steel properties used for the design are shown in table 3.7

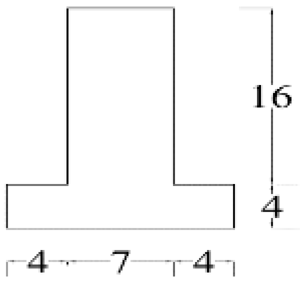


Figure 3.17. Section to design (cm)

Table 3.7. Required diameter of reinforcement for rib

location	**M_{ed} (kN.m)**	**$A_{s; required}$ (mm^2)**	**Diameter of reinforcement used**	**$A_{s; provided}$ (mm2)**
Span	7.98	129.81	2ϕ10	158
support	8.75	131.54	2ϕ10	158

ii) Transverse reinforcement

The designed shear value at support is given by V_{ed}=12.70kN. The normalised shear capacity of concrete after calculation is given by $V_{\text{Rdc}} = 8.67kN$. Therefore, shear reinforcement required because $V_{\text{ed}} > V_{\text{Rdc}}$. Hence, V_{Rd} needs to be computed. Applying equation while assuming shear reinforcement ϕ6 used with spacing of 15cm, V_{Rd}=17.43kN > V_{ed} hand hence verified.

e) Verifications and detailing checks

Ultimate limit state (ULS) and Serviceability limit state (SLS) conditions will be verified as detailed in table 3.8.

Table 3.8. Verifications required for rib

		Conditions	**verifications**
ULS verifications	Neutral axis (x)	0.167d<x<0.618d	x=77.93mm 23.38<x<86.52 **OK**
	Moment (M_{RD})	M_{ED}< M_{RD}	M_{ED}=7.98 kN.m M_{RD}=8.89 kN.m **OK**
SLS verifications	Deflection (l/d)	$(l/d)_{allowable} > (l/d)_{actual}$	$(l/d)_{allowable}$ =27.00mm $(l/d)_{actual}$ =26.286mm **OK**
	Stress (σ)	σ_c<0.65f_{ck} σ_s<0.8f_{yk}	σ_c=11.33<16.25(N/mm^2) σ_S=201.09<360 (N/mm^2)**OK**
	Cracking	Read from table with a crack width of 0.3:	Max bar size=32mm Max bar spacing=289mm **OK**

Joint to the above verification, the minimum area of longitudinal reinforcement A_{smin} is computed. A_{smin}=24.6mm^2.

f) Disposition

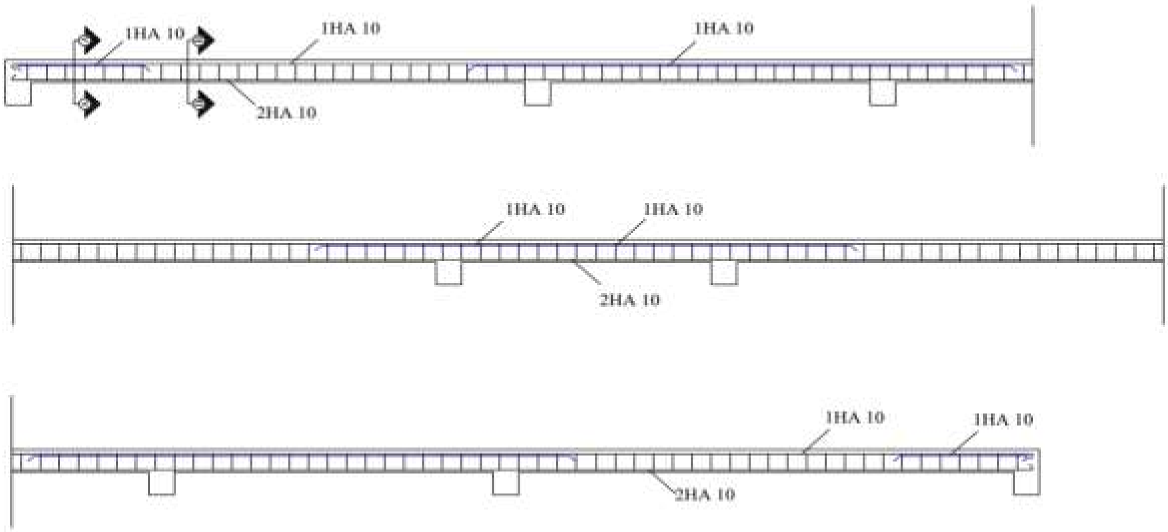


Figure 3.18. Longitudinal reinforcement of designed rib

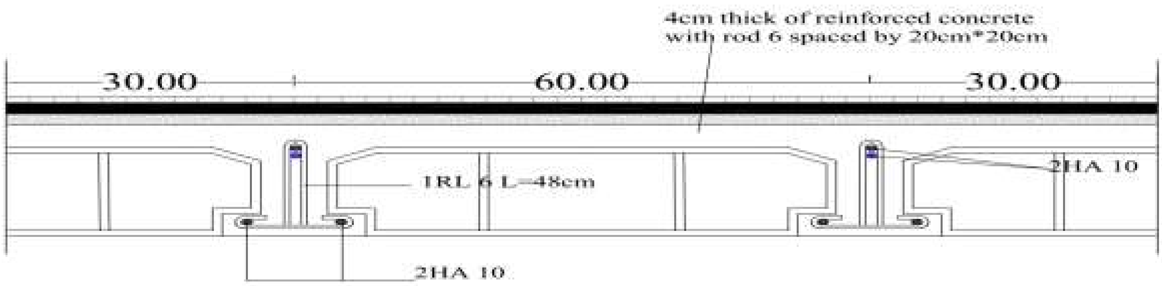


Figure 3.19. Section AA (transverse view at support)

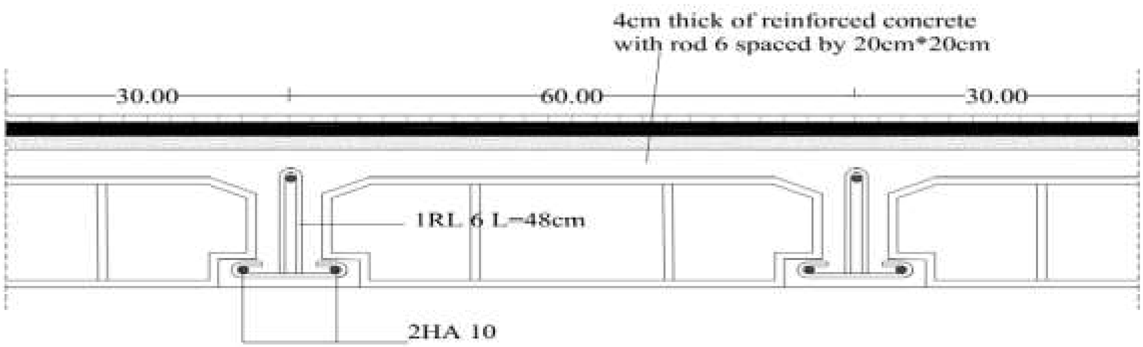


Figure 3.20. Section BB (transverse view at mid span)

3.1.5.2. Design of a conventional two way slab

a) Preliminary design of slab

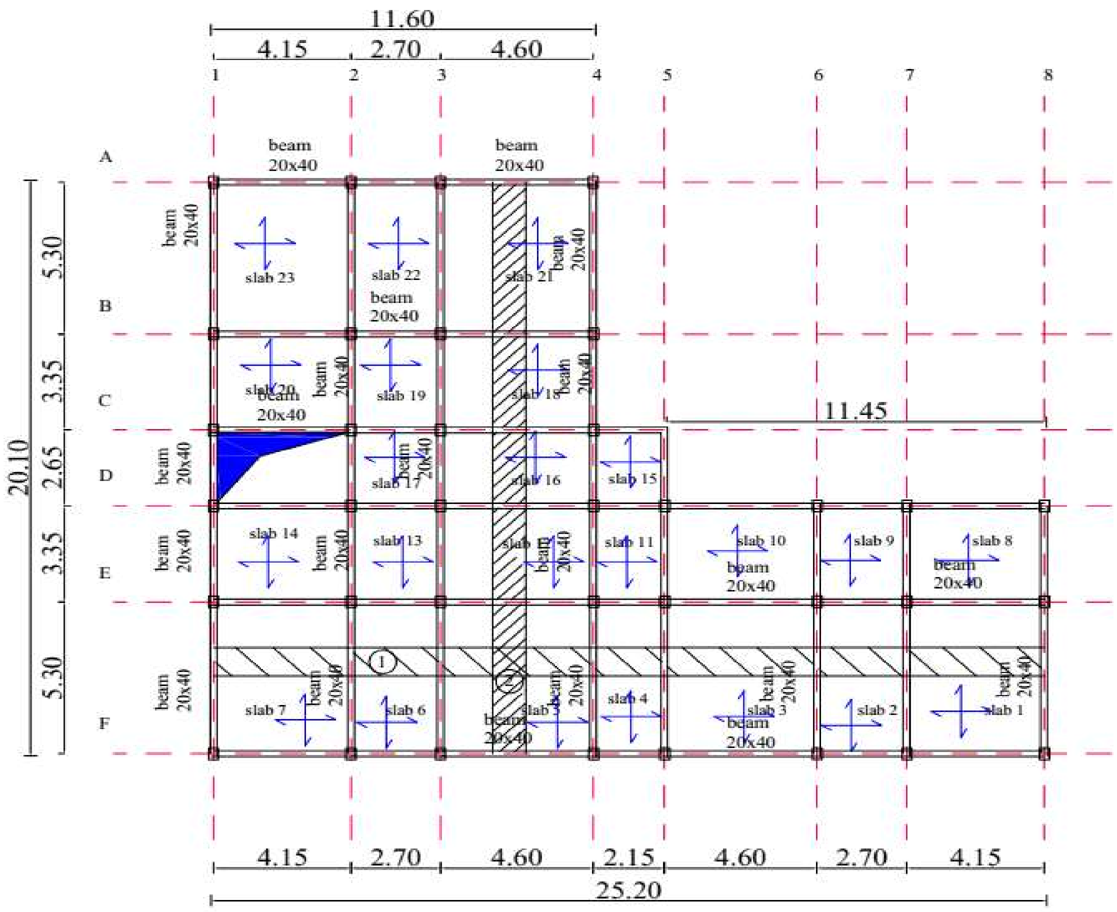


Figure 3.19. Plan model illustrating the two stands to design

Thickness of slab $\frac{L}{25} \le e \le \frac{L}{15} = 18.4 \le e \le 30.6$ so, 20 cm slab will be taken for the design.

b) Load definition

Determining Grashof coefficients using band 1 as represented in figure 3.21 the values are displayed in table 3.9 and table 3.10.

Table 3.9. Computing grashof coefficients in the x direction

panel	Length(x) /m	Length(y) /m	$\alpha_x = \frac{b^4}{a^4 + b^4}$	$\alpha_y = \frac{a^4}{a^4 + b^4}$
Slab 1	4.15	5.30	0.73	0.27

panel	Length(x) /m	Length(y) /m	$\alpha_x = \frac{b^4}{a^4+b^4}$	$\alpha_y = \frac{a^4}{a^4+b^4}$
Slab 2	2.70	5.30	0.94	0.06
Slab 3	4.60	5.30	0.64	0.36
Slab 4	2.15	3.07	0.81	0.90
Slab 5	4.60	5.30	0.64	0.36
Slab 6	2.70	5.30	0.94	0.06
Slab 7	4.15	5.30	0.73	0.27
Average			**0.77**	0.23

Table 3.10. Computing Grashof coefficients in the y direction

panel	Length(a) /m	Length(b) /m	$\alpha_x = \frac{b^4}{a^4+b^4}$	$\alpha_y = \frac{a^4}{a^4+b^4}$
Slab 5	4.60	5.30	0.64	0.36
Slab 12	4.60	3.35	0.22	0.78
Slab 16	4.60	2.65	0.01	0.90
Slab 18	4.60	3.35	0.22	0.78
Slab 21	4.60	5.30	0.64	0.36
average			0.36	**0.64**

Therefore, the applied load in each direction for a band of 1m for the computed grashof load is given by table 3.11.

Table 3.11. Linear loads applied in each direction

Designation	Value(kN/m)	α_a	α_b	q_a (kN/m)	q_b (kN/m)
G_1	5.0	0.77	0.64	3.85	3.2
G_2	2.0	0.77	0.64	1.54	0.28
Q_k	1.5	0.77	0.64	1.16	0.96

c) **Design in the x direction**

i) Load combination

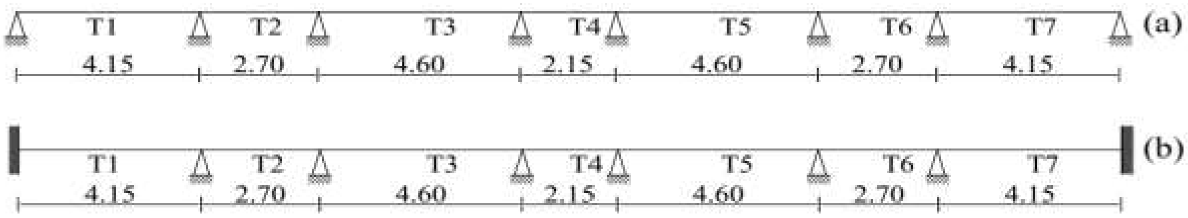


Figure 3.20. Mechanical schemes; (a) simple supported and (b) built-in`

The mechanical scheme used here is as shown in figure 3.22 and the resultant load combinations are illustrated from figure 2.23 to figure 3.27

- Combination 1

$1.35\ (G_1T_1+ G_1T_2+ G_1T_3+ G_1T_4 + G_1T_5 + G_1T_6 + G_1T_{7)}) + 1.35\ (G_2T_1+ G_2T_2+ G_2T_3+ G_2T_4 + G_2T_5 + G_2T_6 + G_2T_7) +1.5\ (Q_kT_1)$

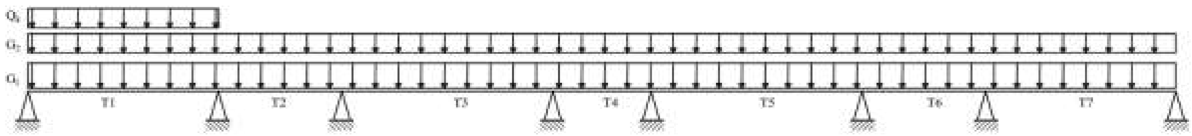


Figure 3.21. First combination ULS for the two way slab in the x -direction

- Combination 2

$1.35\ (G_1T_1+ G_1T_2+ G_1T_3+ G_1T_4 + G_1T_5 + G_1T_6 + G_1T_{7)}) + 1.35\ (G_2T_1+ G_2T_2+ G_2T_3+ G_2T_4 + G_2T_5 + G_2T_6 + G_2T_7) +1.5\ (Q_kT_3)$

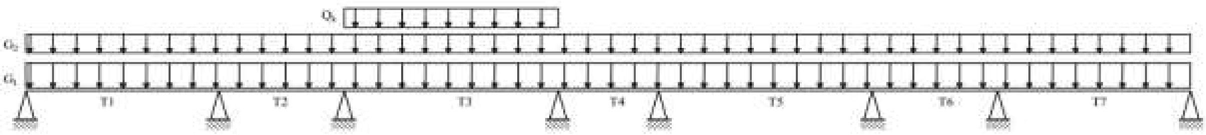


Figure 3.22. Second combination ULS of the two way slab in the x direction

- Combination 3

$1.35\ (G_1T_1+ G_1T_2+ G_1T_3+ G_1T_4 + G_1T_5 + G_1T_6 + G_1T_{7)}) + 1.35\ (G_2T_1+ G_2T_2+ G_2T_3+ G_2T_4 + G_2T_5 + G_2T_6 + G_2T_7) +1.5\ (Q_kT_1+ Q_kT_2+ Q_kT_3 +Q_kT_4 +Q_kT_5 +Q_kT_{6+} Q_kT_7)$

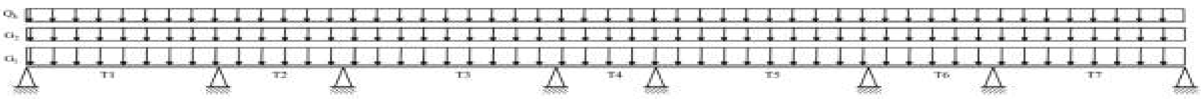


Figure 3.23. Third combination ULS of the two way slab in the x direction

- Combination 4

$1.35\ (G_1T_1 + G_1T_2 + G_1T_3 + G_1T_4 + G_1T_5 + G_1T_6 + G_1T_7) + 1.35\ (G_2T_1 + G_2T_2 + G_2T_3 + G_2T_4 + G_2T_5 + G_2T_6 + G_2T_7) + 1.5\ (Q_kT_1 + Q_kT_2 + Q_kT_3 + Q_kT_4 + Q_kT_5 + Q_kT_6 + Q_kT_7)$

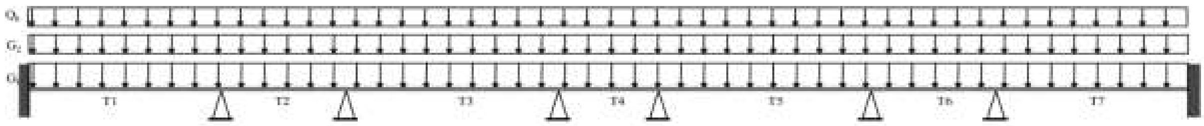


Figure 3.24. Forth combination ULS of the two way slab in the x direction

- Combination RARE

$(G_1T_1 + G_1T_2 + G_1T_3 + G_1T_4 + G_1T_5 + G_1T_6 + G_1T_7) + (G_2T_1 + G_2T_2 + G_2T_3 + G_2T_4 + G_2T_5 + G_2T_6 + G_2T_7) + (Q_kT_1 + Q_kT_2 + Q_kT_3 + Q_kT_4 + Q_kT_5 + Q_kT_6 + Q_kT_7)$

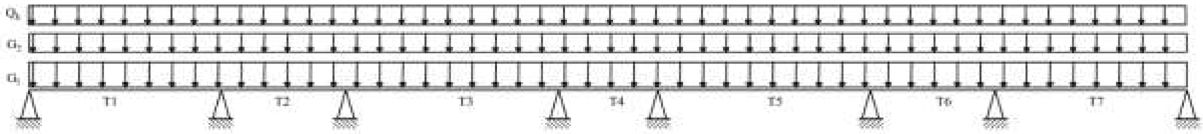


Figure 3.25. Combination SLS of the two way slab in the x direction

ii) Solicitations

The solicitations in terms of moment are shown in figure 3.28 and the maximum solicitations for all the combination are illustrated in figure 2.29.

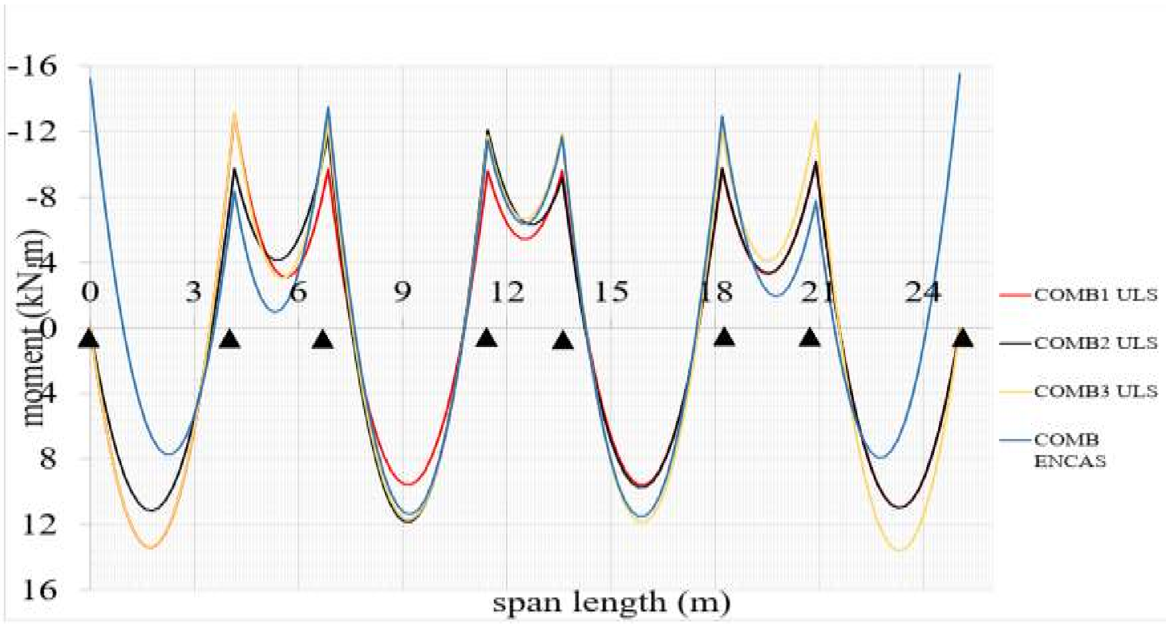


Figure 3.26. Bending moment curves of different combinations in the x-direction

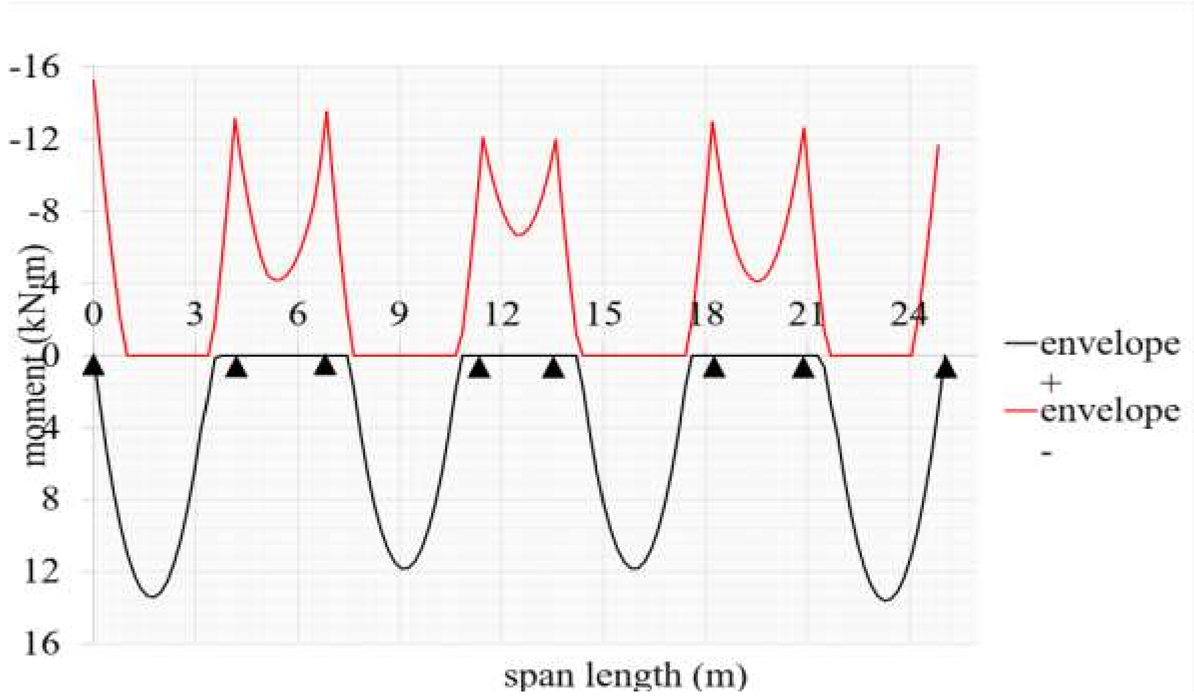


Figure 3.28. Envelope curve of bending moment in the x direction

The solicitations used for the design are chosen from the shifted envelope and the maximum solicitations and are summarised in table 3.12 obtained from the shifted envelope curve illustrated in figure 3.30.

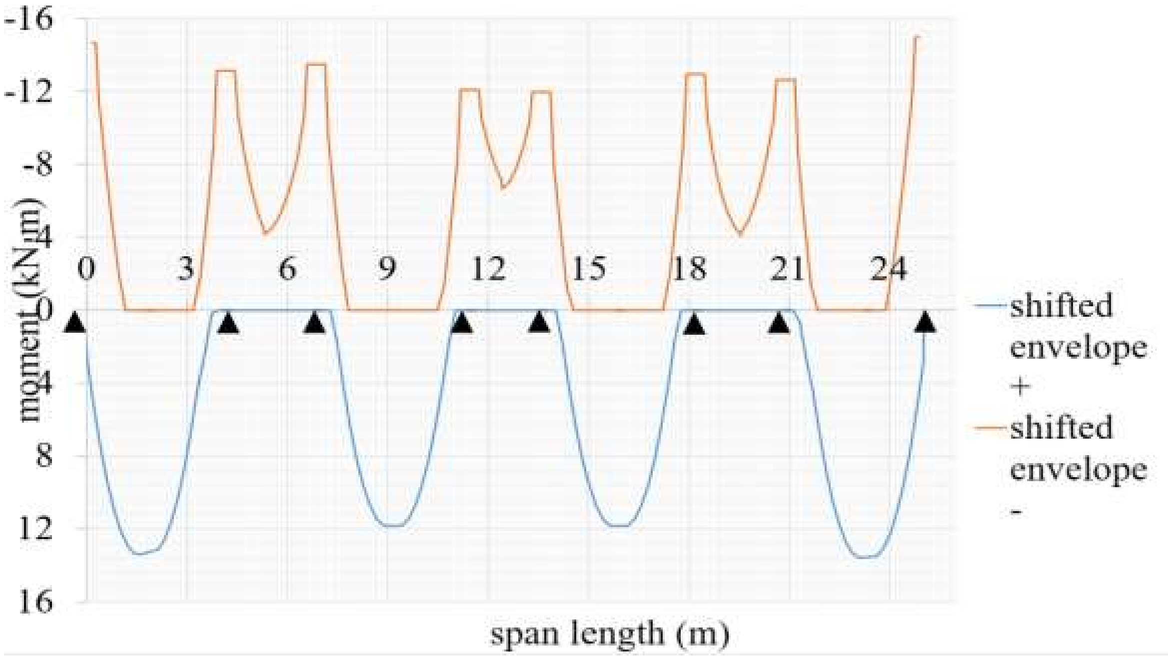


Figure 3.27. Shifted curve of bending moment in the x-direction

Table 3.12. Solicitations used for design in the x-direction

Location	Moment (kN.m)
span	13.60
support	14.67
Span (rare)	9.73

iii) Calculations of steel reinforcement

For a strip of 1m and of slab thickness 20cm, the designed section is as presented by figure 3.31. The results are detailed in table 3.13.

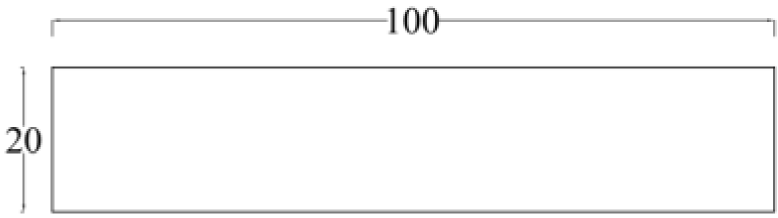


Figure 3.29. Design section (cm)

Table 3.13. Required diameter of reinforcement in the x-direction

location	M_{ed} (kN.m)	$A_{s;\ required}$ (mm^2)	Diameter of reinforcement used	$A_{s;\ provided}$ (mm2)
support	14.67	220.53	4ϕ10	316
span	13.60	204.44	4ϕ10	316

iv) Verifications

ULS and SLS verification will be carry out as clearly elaborated in table 3.14.

Table 3.14. Verifications required for the two-way slab in the x-direction

		Conditions	verifications
ULS verifications	Moment(M_{RD})	$M_{ED} < M_{RD}$	M_{ED}=13.41 kN.m M_{RD}=20.48 kN.m **OK**

		Conditions	verifications
SLS verifications	Deflection(l/d)	$(l/d)_{allowable} > (l/d)_{actual}$	$(l/d)_{allowable} = 99.48$mm $(l/d)_{actual}$ =31.18mm **OK**
	Stress (σ)	$\sigma_c<0.65f_{ck}$ $\sigma_s<0.8f_{yk}$	σ_c=3.44<16.25(N/mm^2) σ_S=194.45<360 (N/mm^2) **OK**
	Cracking	Read from table with a crack width of 0.3: Max bar size Max bar spacing	Max bar size=16mm **OK**

c) Design in the y-direction

i) Load combination

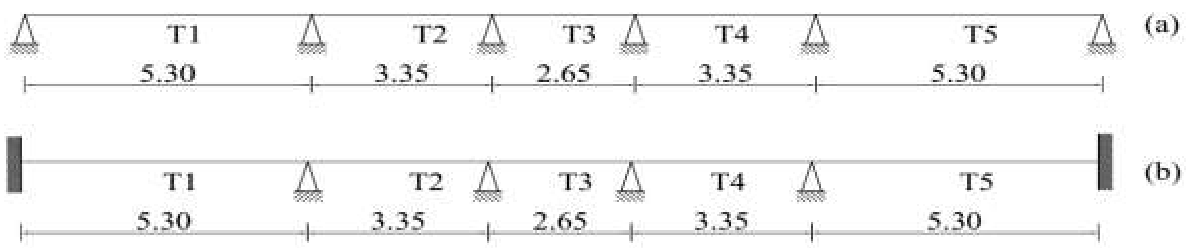


Figure 3.30. Mechanical schemes (a) simple supported and (b) built-in

The mechanical scheme used here is as shown in figure 3.32 and resultant load cases for these different static schemes are detailed in figures 3.33 to 3.38.

- Combination 1

$1.35\ (G_1T_1+ G_1T_2+ G_1T_3+ G_1T_4 + G_1T_5) + 1.35\ (G_2T_1+ G_2T_2+ G_2T_3+ G_2T_4 + G_2T_5) +1.5\ (Q_kT_1)$

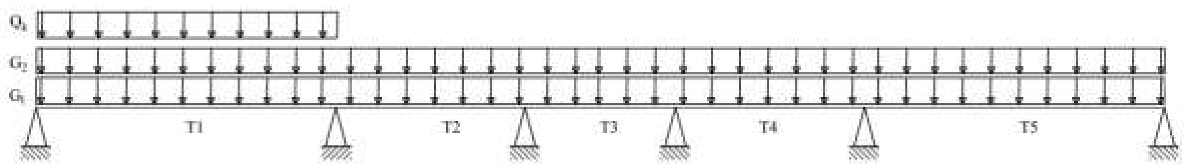


Figure 3.31. First load combination ULS for two way slab in the y-direction

- Combination 2

1.35 (G_1T_1+ G_1T_2+ G_1T_3+ G_1T_4 + G_1T_5) + 1.35 (G_2T_1+ G_2T_2+ G_2T_3+ G_2T_4 + G_2T_5) +1.5 (Q_kT_2)

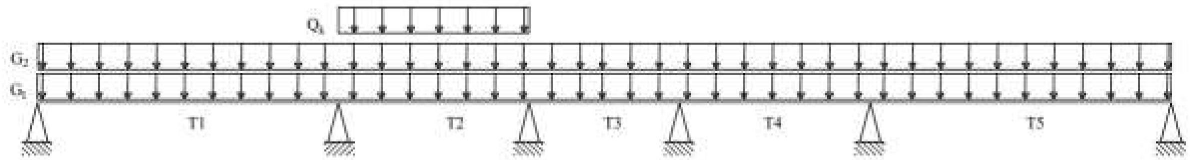


Figure 3.32. Second load combination ULS for two way slab in the y-direction

- Combination 3

1.35 (G_1T_1+ G_1T_2+ G_1T_3+ G_1T_4 + G_1T_5) + 1.35 (G_2T_1+ G_2T_2+ G_2T_3+ G_2T_4 + G_2T_5) +1.5 (Q_kT_3)

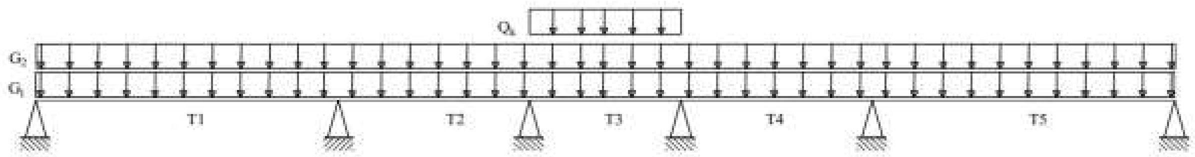


Figure 3.33. Third load combination ULS for two way slab in the y-direction

- Combination 4

1.35 (G_1T_1+ G_1T_2+ G_1T_3+ G_1T_4 + G_1T_5) + 1.35 (G_2T_1+ G_2T_2+ G_2T_3+ G_2T_4 + G_2T_5) +1.5 (Q_kT_1+ Q_kT_2+ Q_kT_3 +Q_kT_4 +Q_kT_5)

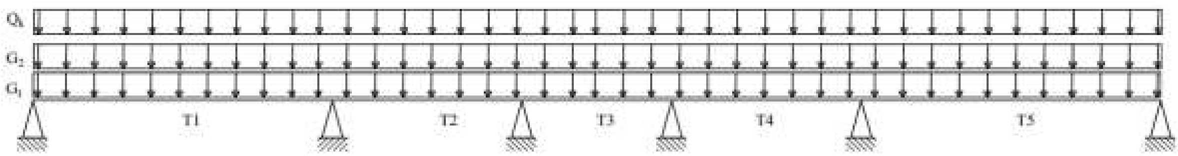


Figure 3.34. Forth load combination ULS for two way slab in the y-direction

- Combination 5

1.35 (G_1T_1+ G_1T_2+ G_1T_3+ G_1T_4 + G_1T_5) + 1.35 (G_2T_1+ G_2T_2+ G_2T_3+ G_2T_4 + G_2T_5) +1.5 (Q_kT_1+ Q_kT_2+ Q_kT_3 +Q_kT_4 +Q_kT_5

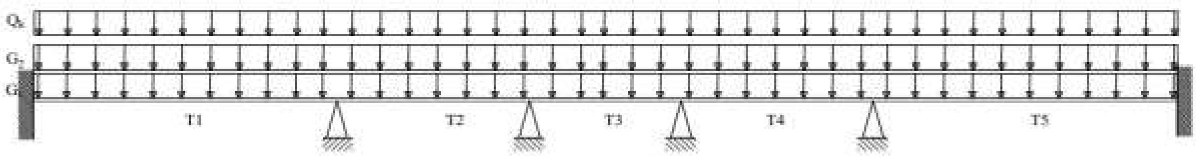


Figure 3.35. Fifth load combination ULS for two way slab in the y-direction

- Combination RARE

$(G_1T_1+ G_1T_2+ G_1T_3+ G_1T_4 + G_1T_5) + (G_2T_1+ G_2T_2+ G_2T_3+ G_2T_4 + G_2T_5) + (Q_kT_1+ Q_kT_2+ Q_kT_3 +Q_kT_4 +Q_kT_5)$

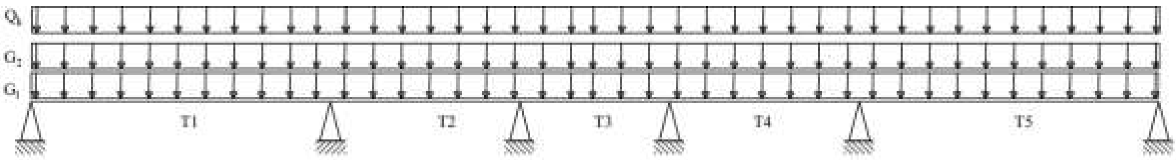


Figure 3.36.Load combination SLS for two way slab in the y-direction

ii) Solicitations

Expressed in terms of moment and not shear because shear is not an important solicitation in the design of two way slab the solicitations are displayed in figure 3.39.

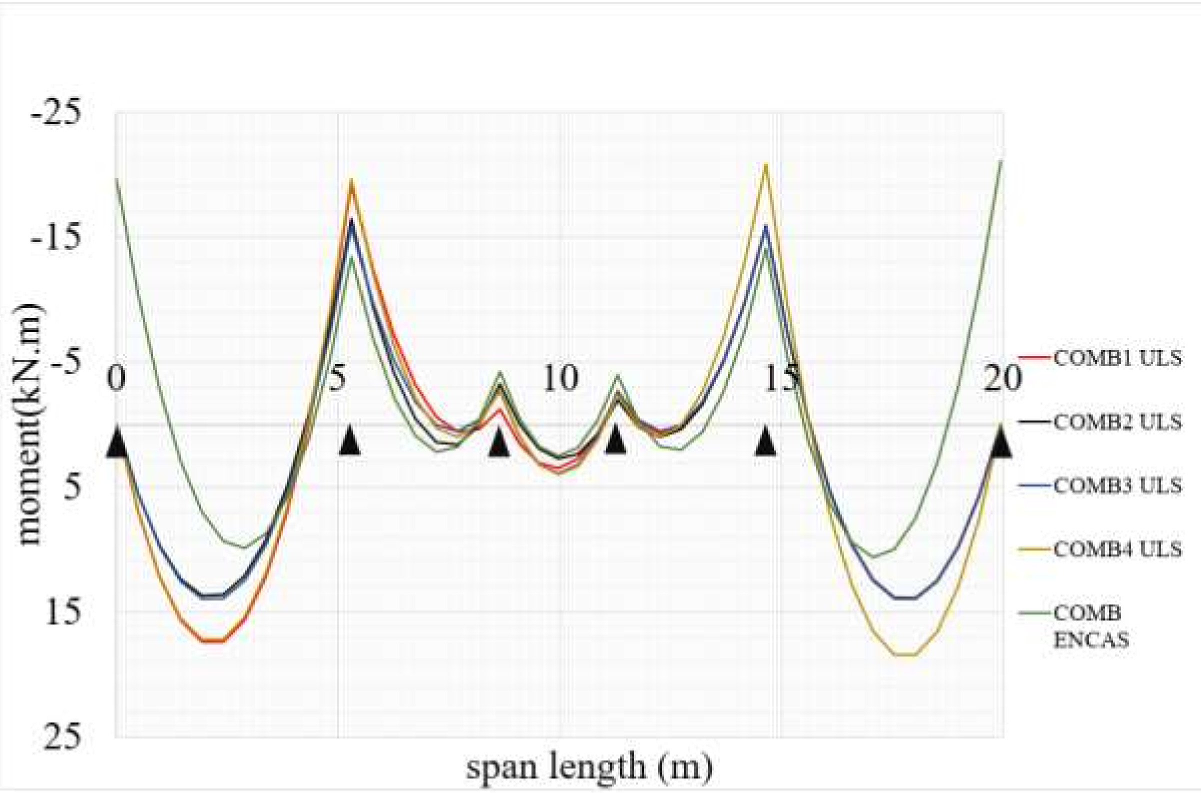


Figure 3.37. Moment diagrams in the y-direction

The envelope of the different combinations is illustrated in figure 3.40 and applying EC2 restriction and shifting, the shifting envelope is displayed in figure 3.41 and the values of bending moment extracted used for the design.

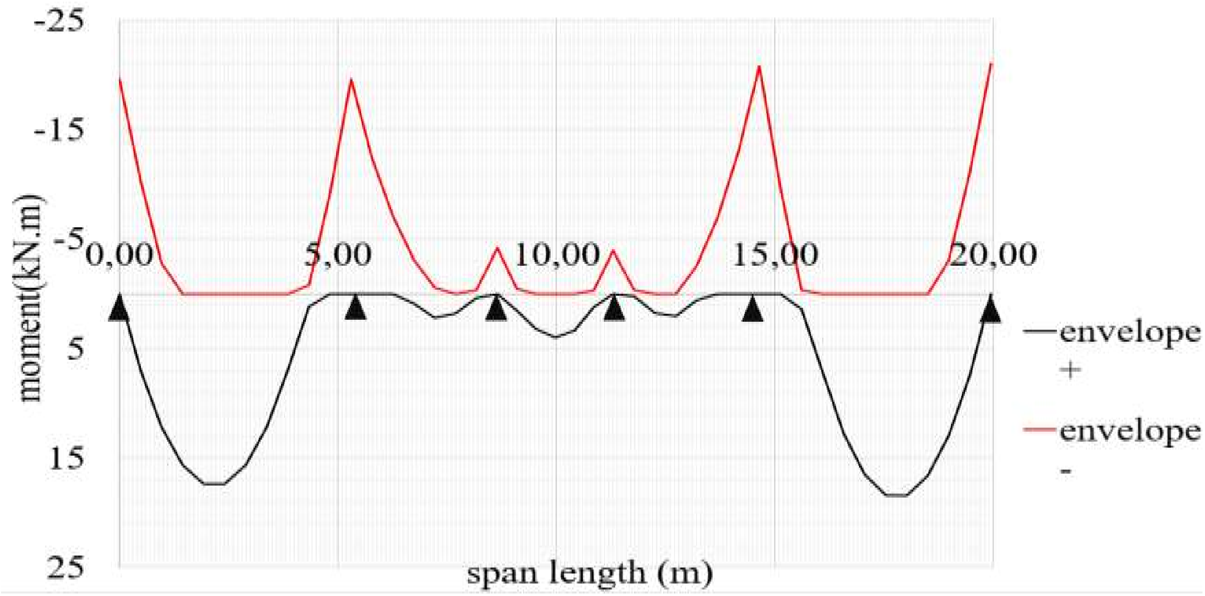


Figure 3.38. Envelope curve of bending moments in the y-direction

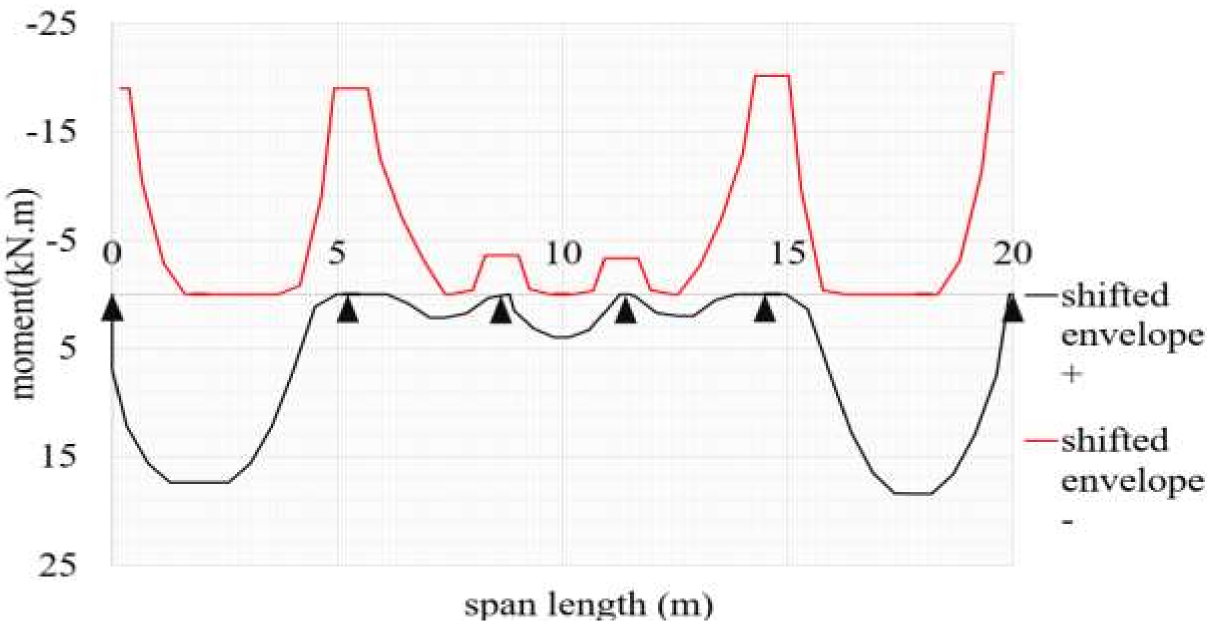


Figure 3.39. Shifted envelope for bending moment in the y-direction

Table 3.15. Solicitations used for design in the y-direction

Location	Moment (kN.m)
Span	18.44
Support	20.46
Span (rare)	12.52

iii) Calculations of steel reinforcement

The design section is the same section as early defined above in the design in the x direction. Likewise, the concrete and steel characteristic remains unchanged. Using equations in section 2.2.2, the results are displayed in table 3.16.

Table 3.16: Required diameter of reinforcement in the y-direction

location	M_{ed} (kN.m)	$A_{s;\,required}$ (mm^2)	Diameter of reinforcement used	$A_{s;\,provided}$ (mm2)
support	20.46	341.75	2ϕ10 & 2ϕ12	384
span	18.44	308.00	4ϕ10	316

iv) Verifications

ULS and SLS verification will be carry out as clearly elaborated in table 3.17.

Table 3.17. Verifications on the two-way slab in the y-direction

		Conditions	verifications
ULS verifications	Moment(M_{RD})	$M_{ED} < M_{RD}$	M_{ED}=18.43 kN.m M_{RD}=20.48 kN.m **OK**
SLS verifications	Deflection(l/d)	$(l/d)_{allowable} > (l/d)_{actual}$	$(l/d)_{alowable}$ =85.24mm $(l/d)_{actual}$ =31.18mm **OK**
	Stress (σ)	$\sigma_c < 0.65f_{ck}$ $\sigma_s < 0.8f_{yk}$	σ_c=4.27<16.25(N/mm^2) σ_S=241.187<360 (N/mm^2) **OK**

		Conditions	verifications
	Cracking	Read from table with a crack width of 0.3:Max bar size	Max bar size=16mm **OK**

d) Detailing checks and disposition

As for the detailing checks, the minimum reinforcement required for a strip of 1m,

- The minimum longitudinal reinforcement, A_{smin}=251.4mm^2 corresponding to 4ϕ10.
- At support, the reinforcement should extend at least 0.2 times the length of the adjacent span, measured from the face of the support.
- The maximum spacing for the principal reinforcement Smax,slab=min(3h,400mm) = 400mm. use 300mm.
- The maximum spacing for the secondary transverse reinforcement $S_{max,slab}$=min(3.5h,450mm)=450mm. use 300mm.
- Transveral reinforcement are included in order to counteract the effects of torsional moment mxy. ϕ10 is used with a spacing of 40cm.
- Eurocode 2 prescribes that half of the amount of steel put in span should be extended to the support.

In areas with concentrated loads or areas of maximum moment those provisions become respectively:

- for the principal reinforcement, S_{max}= min(2h,250 mm) = 250mm. use 150mm
- The secondary reinforcement, Smax=min (3h,400)=400mm. use 150mm

From plate theory, there exist a torsional moment mxy in slabs which is null in the middle slab but maximum in support. In order to overtake this, surplus reinforcement disposed diagonally will be done at support as displayed in figure 3.43.

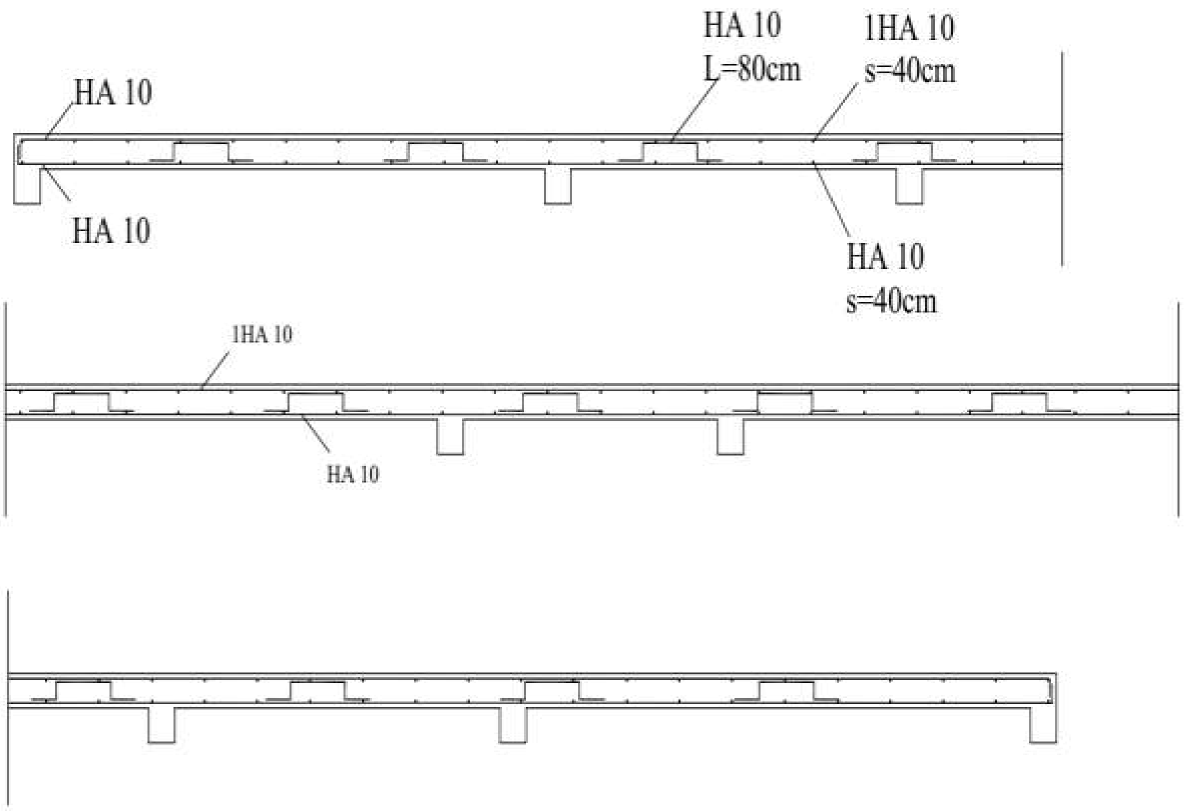


Figure 3.40. Longitudinal reinforcement in the x-direction

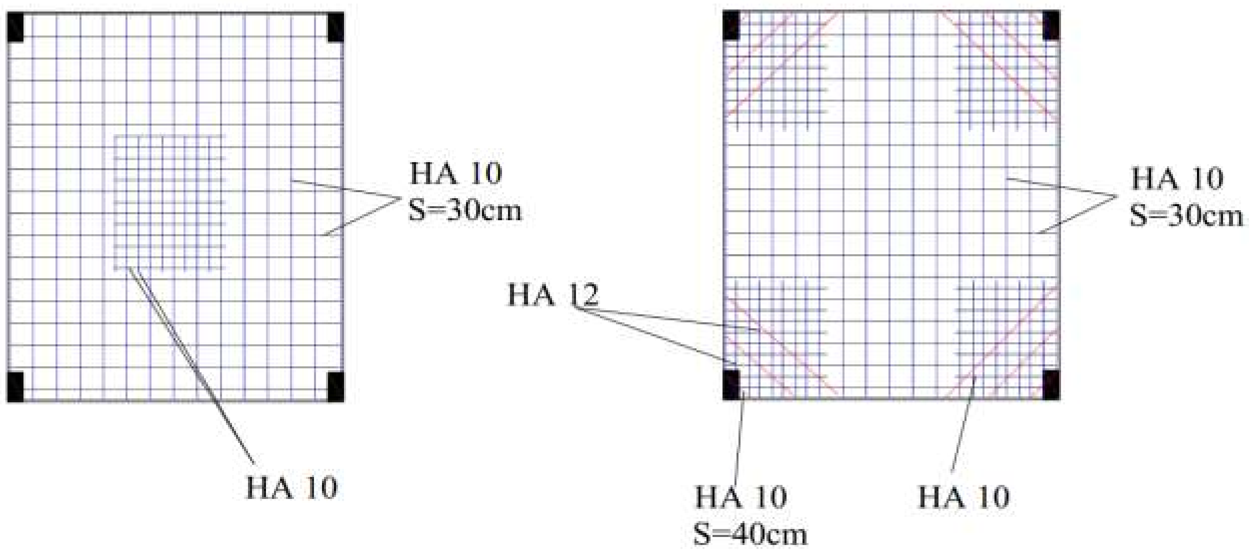


Figure 3.41. Lower layer (left) and upper layer (right) reinforcement of panel "slab 1"

3.1.6. Beam design

The model to design is this work is considered to be a one way slab made with hollow blocks as early detailed.

a) Prilimimary design

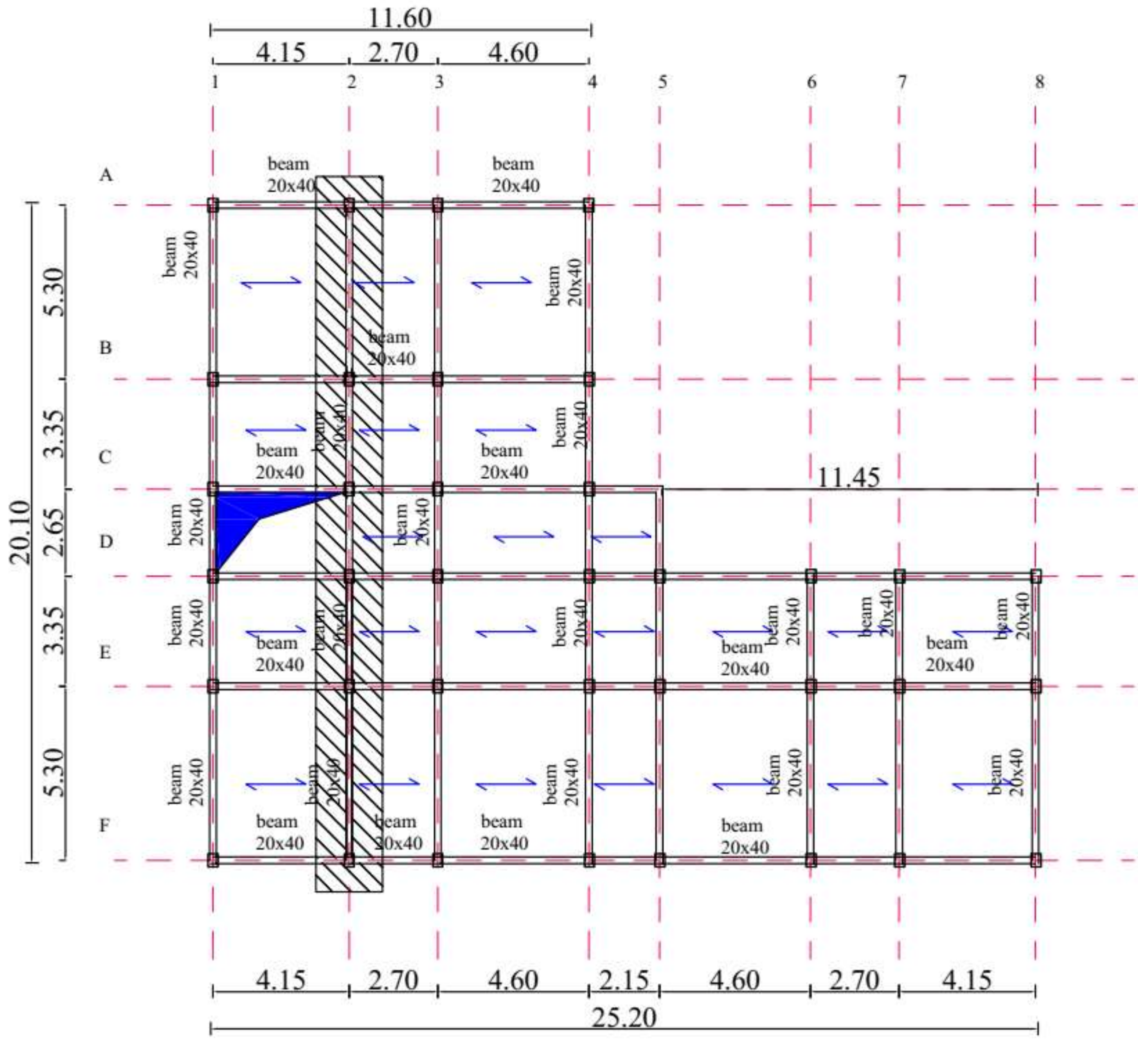


Figure 3.42. Plan illustrating beam to design

The beam to design is the principal beam that lies in grid 2 illustrated in figure 3.44. The beam is a rectangular beam and the preliminary dimensions for the design are obtained to satisfy the conditions

$h \geq \frac{L}{14} = \frac{530}{14} = 37.85$ and $0.3h \leq b \leq 0.5h$ hence chose h=50cm and b=30cm for the design.

b) Load definition and load combination

Referring to the design beam as illustrated in figure 3.44, the influence area of the apply load can easily be computed. So the linear loads applied are as displayed according to table 3.18.

Table 3.18. Uniform loads for beam design

Designation	Value (kN/m²)	Influence area	Value (kN/m)
G_1	2.85	3.43	9.76
G_2	2.0	3.43	6.85
G_{wall}	2.5	3.00	7.50
Q_k	1.5	3.43	5.14

For these loads, different arrangement can be done in order to obtain the greatest solicitation. The different combinations are displayed in figures 3.45 to 3.50.

- Combination 1

$1.35\ (G_1T_1+ G_1T_2+ G_1T_3+ G_1T_4 + G_1T_5) + 1.35\ (G_2T_1+ G_2T_2+ G_2T_3+ G_2T_4 + G_2T_5) +1.35\ (G_{wall}T_1+ G_{wall}T_2+ G_{wall}T_3+ G_{wall}T_4 + G_{wall}T_5) +1.5\ (Q_kT_1)$

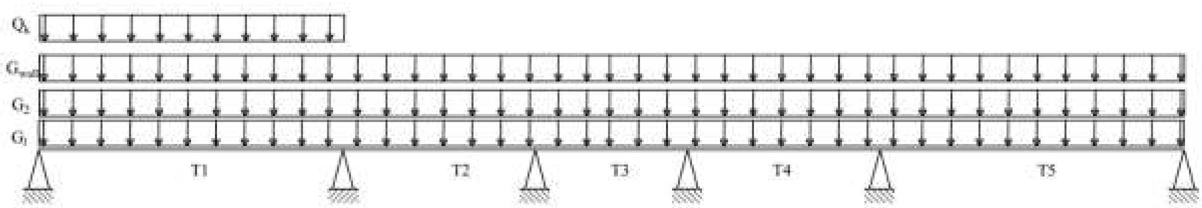


Figure 3.43: First load combination ULS for beam

- Combination 2

$1.35\ (G_1T_1+ G_1T_2+ G_1T_3+ G_1T_4 + G_1T_5) + 1.35\ (G_2T_1+ G_2T_2+ G_2T_3+ G_2T_4 + G_2T_5) +1.35\ (G_{wall}T_1+ G_{wall}T_2+ G_{wall}T_3+ G_{wall}T_4 + G_{wall}T_5) +1.5\ (Q_kT_2)$

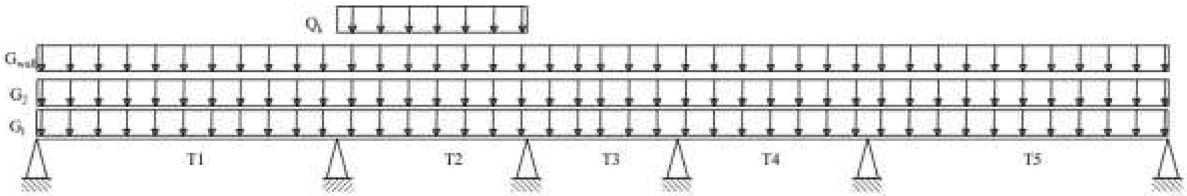


Figure 3.44: Second load combination ULS for beam

- Combination 3

$1.35\ (G_1T_1 + G_1T_2 + G_1T_3 + G_1T_4 + G_1T_5) + 1.35\ (G_2T_1 + G_2T_2 + G_2T_3 + G_2T_4 + G_2T_5) + 1.35\ (G_{wall}T_1 + G_{wall}T_2 + G_{wall}T_3 + G_{wall}T_4 + G_{wall}T_5) + 1.5\ (Q_kT_3)$

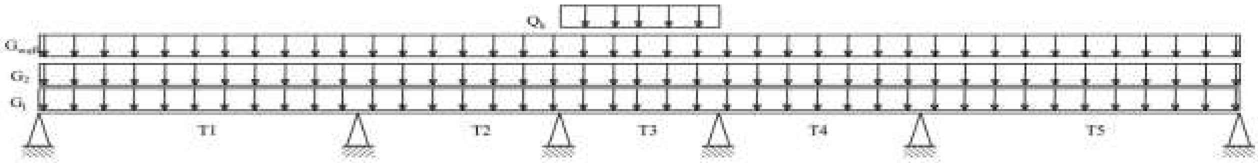


Figure 3.45. Third load combination ULS for beam

- Combination 4

$1.35\ (G_1T_1 + G_1T_2 + G_1T_3 + G_1T_4 + G_1T_5) + 1.35\ (G_2T_1 + G_2T_2 + G_2T_3 + G_2T_4 + G_2T_5) + 1.35\ (G_{wall}T_1 + G_{wall}T_2 + G_{wall}T_3 + G_{wall}T_4 + G_{wall}T_5) + 1.5\ (Q_kT_1 + Q_kT_2 + Q_kT_3 + Q_kT_4 + Q_kT_5)$

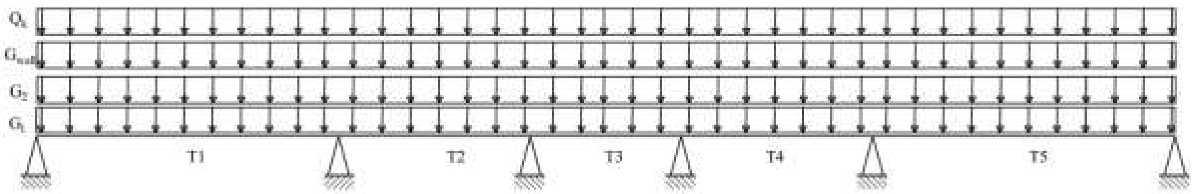


Figure 3.46: Forth load combination ULS for beam

- Combination 5

$1.35\ (G_1T_1 + G_1T_2 + G_1T_3 + G_1T_4 + G_1T_5) + 1.35\ (G_2T_1 + G_2T_2 + G_2T_3 + G_2T_4 + G_2T_5) + 1.35\ (G_{wall}T_1 + G_{wall}T_2 + G_{wall}T_3 + G_{wall}T_4 + G_{wall}T_5) + 1.5\ (Q_kT_1 + Q_kT_2 + Q_kT_3 + Q_kT_4 + Q_kT_5)$

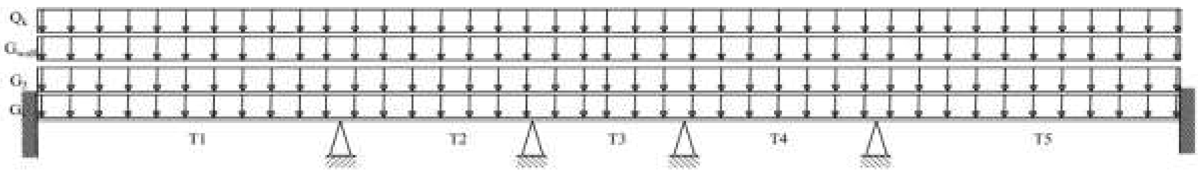


Figure 3.47: Fifth load combination ULS for beam

- Combination RARE

$(G_1T_1 + G_1T_2 + G_1T_3 + G_1T_4 + G_1T_5) + (G_2T_1 + G_2T_2 + G_2T_3 + G_2T_4 + G_2T_5) + (G_{wall}T_1 + G_{wall}T_2 + G_{wall}T_3 + G_{wall}T_4 + G_{wall}T_5) + (Q_kT_1 + Q_kT_2 + Q_kT_3 + Q_kT_4 + Q_kT_5)$

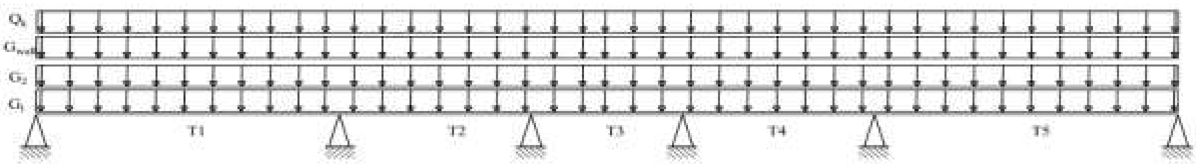


Figure 3.48. Sixth load combination ULS for beam

c) Solicitations

Solicitations are mainly in terms of bending moment and shear as displayed in figures 3.51 and 3.52 respectively for the different combinations.

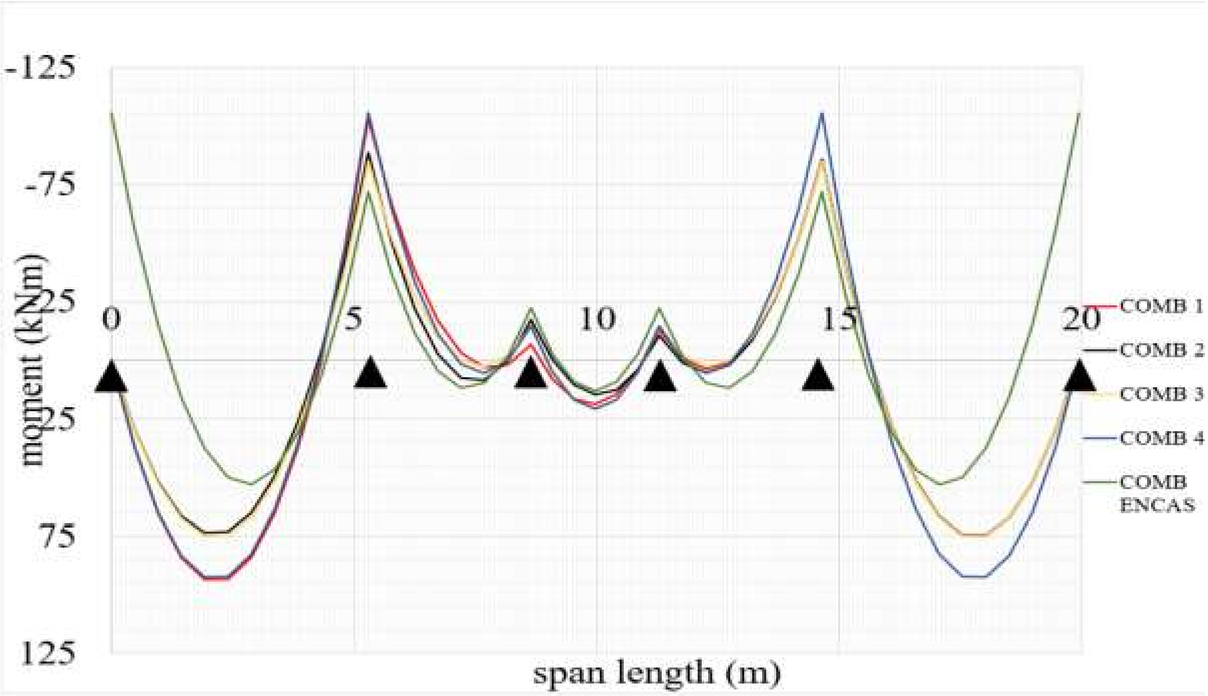


Figure 3.50. Bending moments diagrams

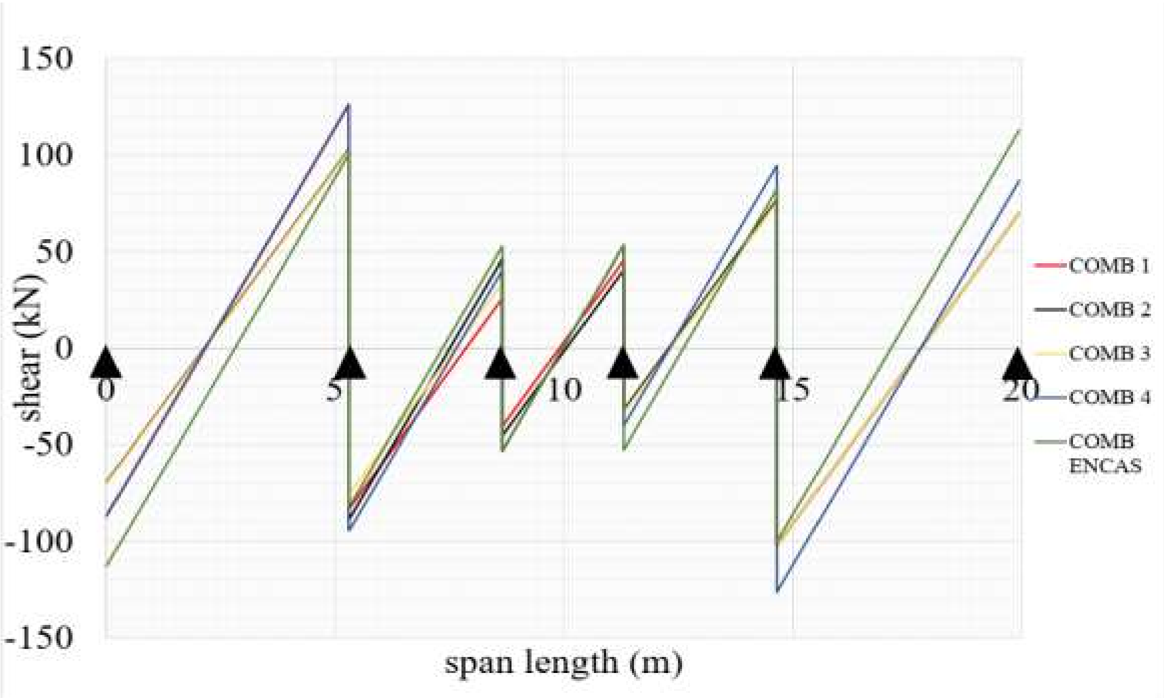


Figure 3.49. Shear diagrams

From the above diagrams, the envelope of each is done in order to obtain the maximum and minimum solicitations at each point and the obtained values are used for the design. The envelopes are represented by figure 3.53 and figure 3.54.

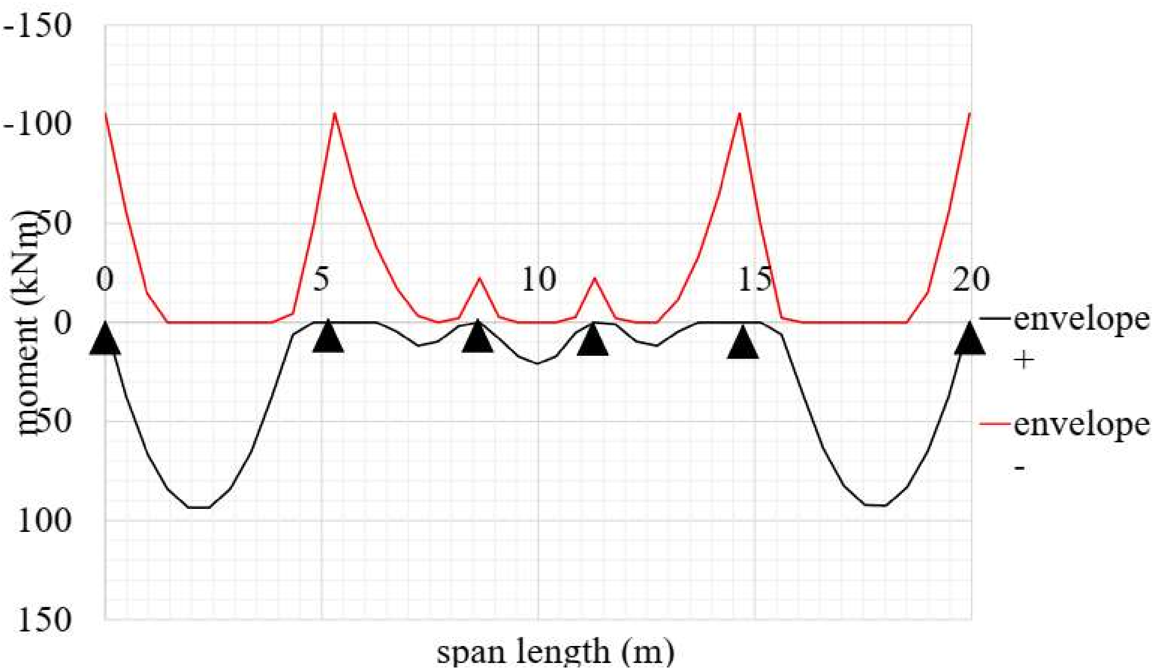


Figure 3.52. Envelope curve of bending moment

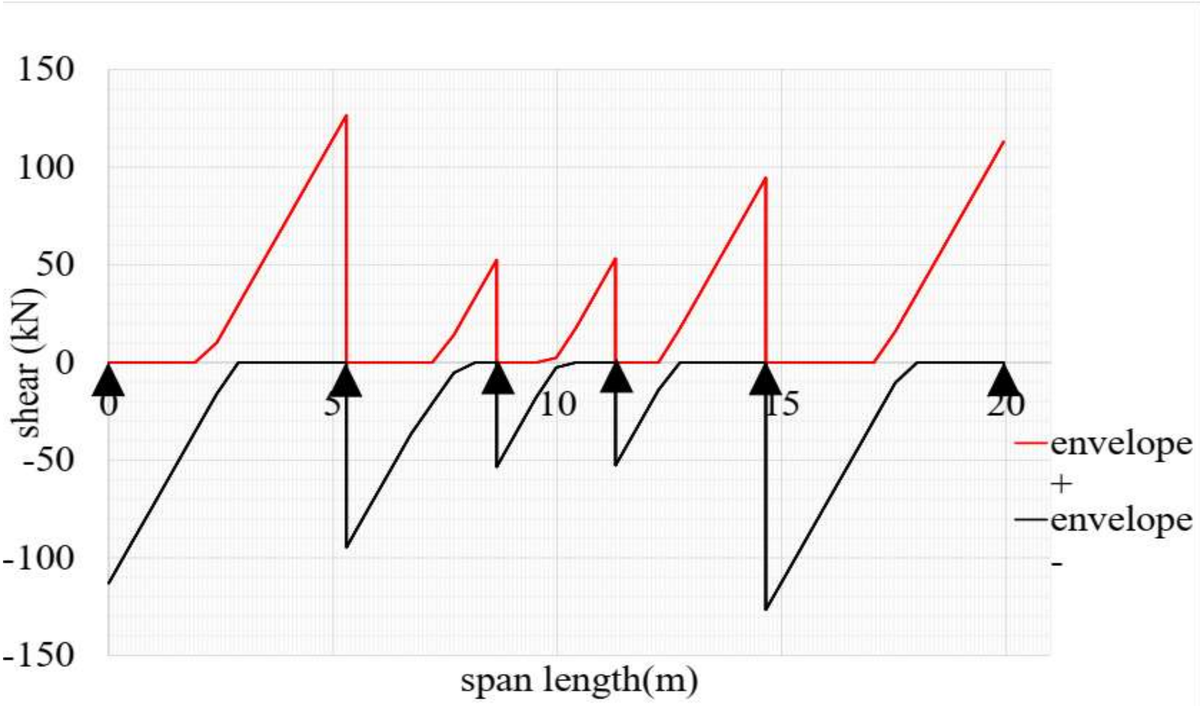


Figure 3.51. Envelope curves for shear

From the envelope curves, the shifted envelope curve of bending moment is drawn as displayed in figure 3.55 and the value used to design at critical points are displayed in table 3.19.

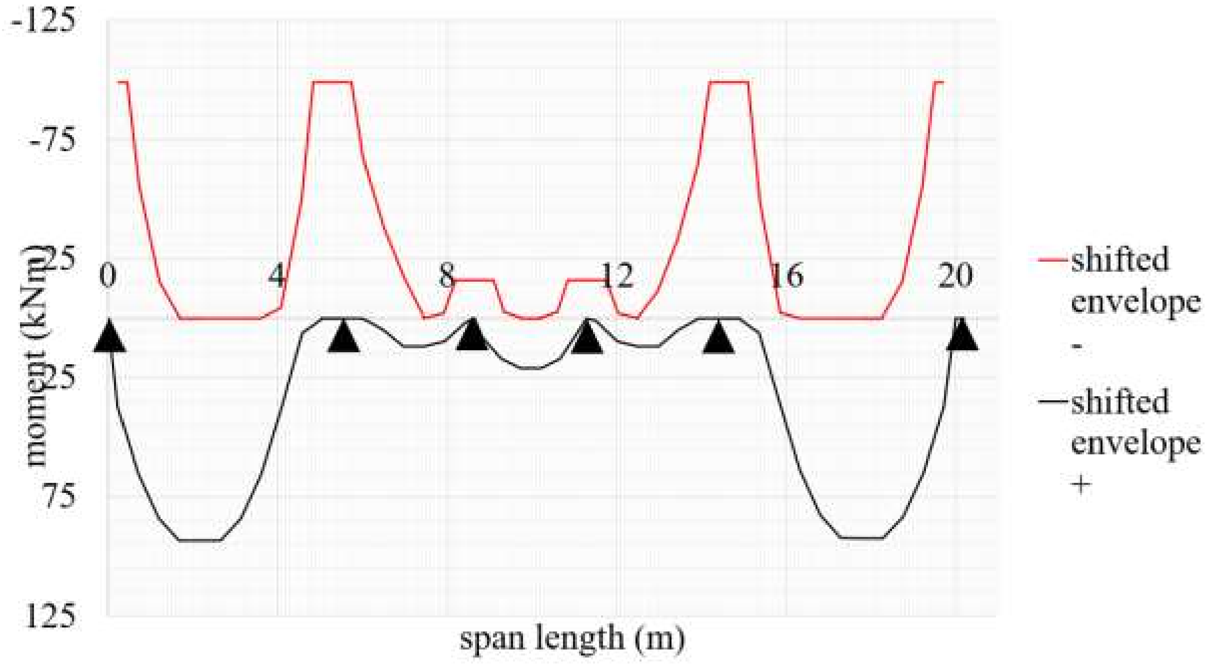


Figure 3.53. Shifted envelope curve of bending moment

Table 3.19. Solicitations used for design in the beam

Location	Moment (kN.m)	Shear (kN)
Span	93.50	/
support	105.5	126.6
Span(SLS rare)	50.08	/

d) Calculation of steel bars reinforcement

Using the procedures and formulae elaborated in section 2.2.3, the longitudinal and transversal reinforcement can be calculated.

i) Longitudinal reinforcement

Taking into considerations the dimensions of figure 3.56, the results are summarised in table 3.20.

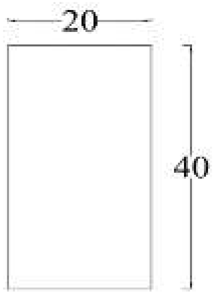


Figure 3.54. Beam section to design

Table 3.20. Required diameter of reinforcement in beam element

location	**M_{ed} (kN.m)**	**$A_{s; required}$ (mm^2)**	**Diameter of reinforcement used**	**$A_{s; provided}$ (mm2)**
support	99.17	761.06	2ϕ20 & 1ϕ16	829
span	108.95	836.12	3ϕ20	762

ii) Transverse reinforcement

The designed shear value at support from the envelope shear diagram is V_{ed}=12.70kN. The normalised shear capacity of concrete after calculation is $V_{\text{Rdc}} = 46.6kN$. Therefore, shear reinforcement required because $V_{\text{ed}} > V_{\text{Rdc}}$. Hence, V_{Rd} needs to be computed using the equations abovementioned in equations 2.19, 2.20 and 2.21 in section 2.2.2. for ϕ8 with a spacing of 20cm, the resisting shear is computed and the value obtained is $V_{\text{Rd}} = 184kN > V_{ED}$.

e) Verifications and detailing checks

The design beam is verified at ULS and SLS through the calculation of resisting bending moment, stress and crack calculations at critical points and result displayed in table 3.21

Table 3.21. Verifications on beam element

		Conditions	**verifications**
ULS verifications	Neutral axis (x)	0.167d<x<0.618d	x=131.5mm 61.8 < x <2 90.5 **OK**
	Moment(M_{RD})	M_{ED}< M_{RD}	M_{ED}=93.5 kN.m

		Conditions	verifications
			M_{RD}= 94.6 kN.m**OK**
SLS verifications	Deflection(l/d)	$(l/d)_{allowable} > (l/d)_{actual}$	$(l/d)_{allowable}$ =27.00mm $(l/d)_{actual}$ =26.286mm **OK**
	Stress (σ)	$\sigma_c<0.65f_{ck}$ $\sigma_s<0.8f_{yk}$	σ_c=10.09<16.25(N/mm^2) σ_S=206.74<360 (N/mm^2) **OK**
	Cracking	Read from table with a crack width of 0.3: Max bar size Max bar spacing	Max bar size=25mm Max bar spacing=250mm **OK**

Regarding to detailing of beam element, Eurocode 2-1 section 9.2 clearly explain the requirements, some of which are

- Minimum longitudinal reinforcement $A_{s,min} = 96.2\ mm^2$
- Maximum longitudinal reinforcement $A_{s,max} = 2960mm^2$

f) Disposition

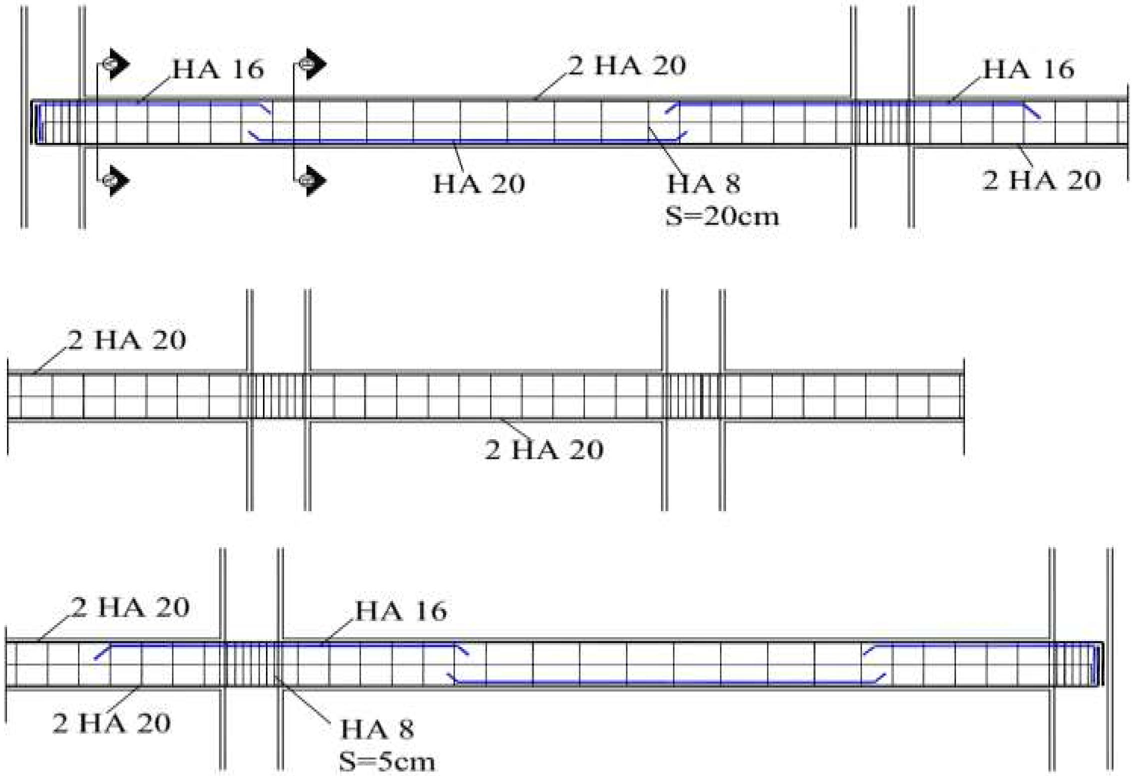


Figure 3.55. Longitudinal view of reinforced beam

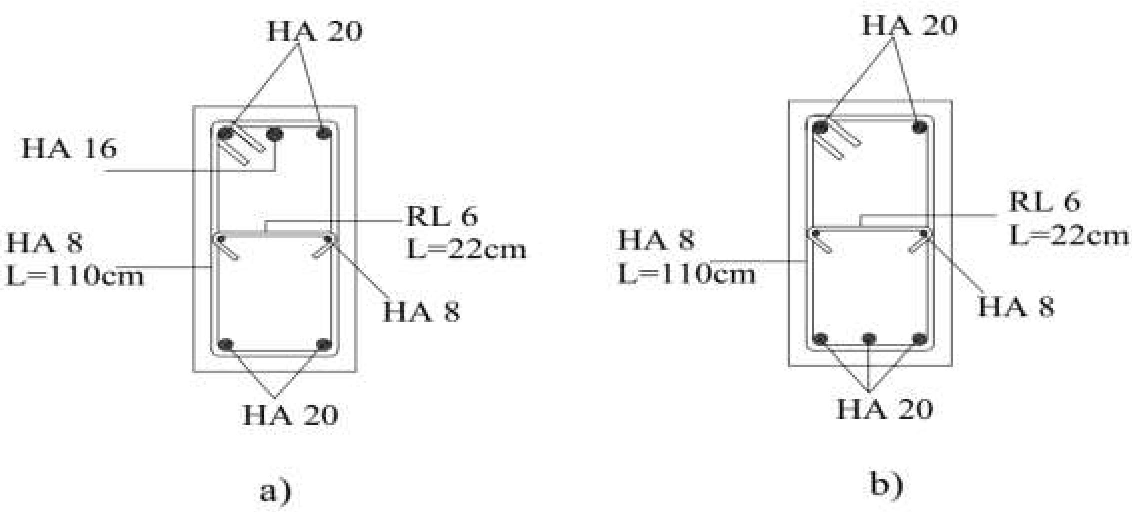


Figure 3.56. a) Section AA and b) section BB (transverse view at support and span respectively)

3.1.7. Column design

The preliminary design, M-N design, shear verifications and slenderness are presented. The column is design for a fixed based model.

3.1.7.1. Preliminary design

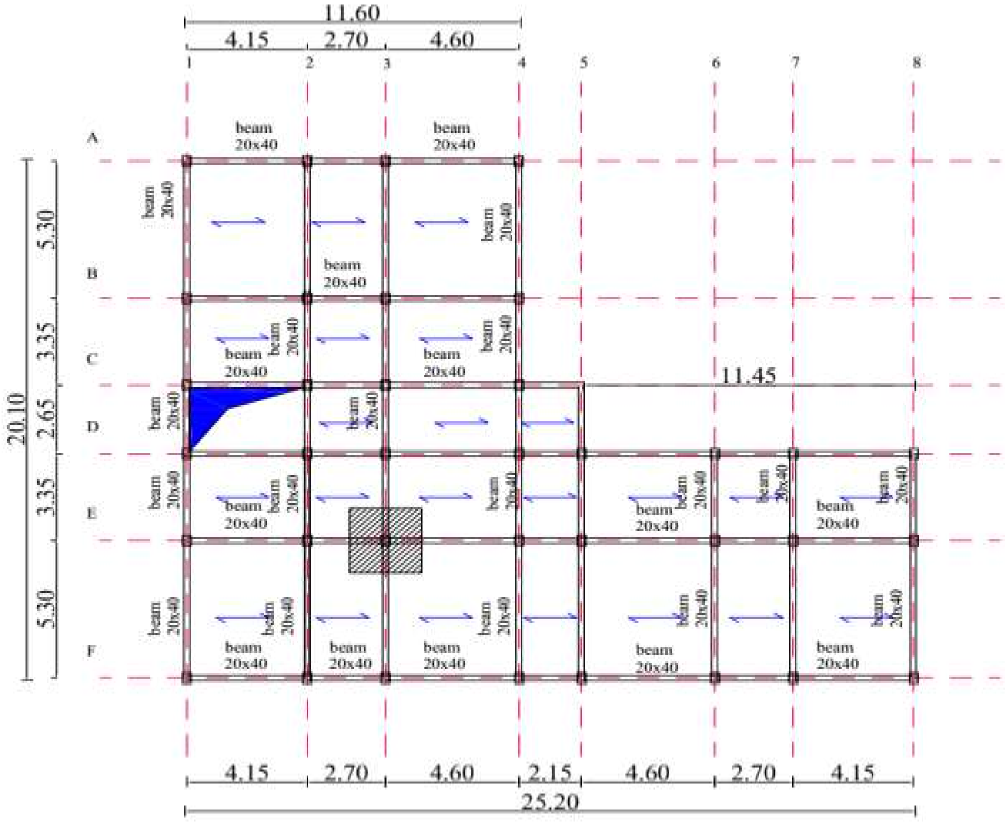


Figure 3.57. Plan view indicating the column to design

In a seismic area, the preliminary design of the column considers that 60% of the concrete resistance is used to take over the axial force. The column to design is illustrated in figure 3.59 and the formulation used for the preliminary design is represented by equation 3.6.

$$N_{sd} = 0.6 f_{cd} A_c \geq q S_r n \quad (3.6)$$

With

n: number of stories above the considered column

Ac: concrete section area

q: uniform distributed load at each floor at ULS

Sr: recovery area of a column

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

Hence, $A_c \geq 105397.26 mm^2$; hence choose column of dimensions $25cm \times 50cm$.

3.1.7.2. Solicitations

The solicitations on column is obtain by modelling on sap 2000 and applying different combinations with charges early described in section 3.2.3. Then the axial force diagram is plotted, the shear and moment in both x and y direction is also plotted as displayed by figures 3.60, 3.61, 3.62, 3.63 and 3.64 respectively.

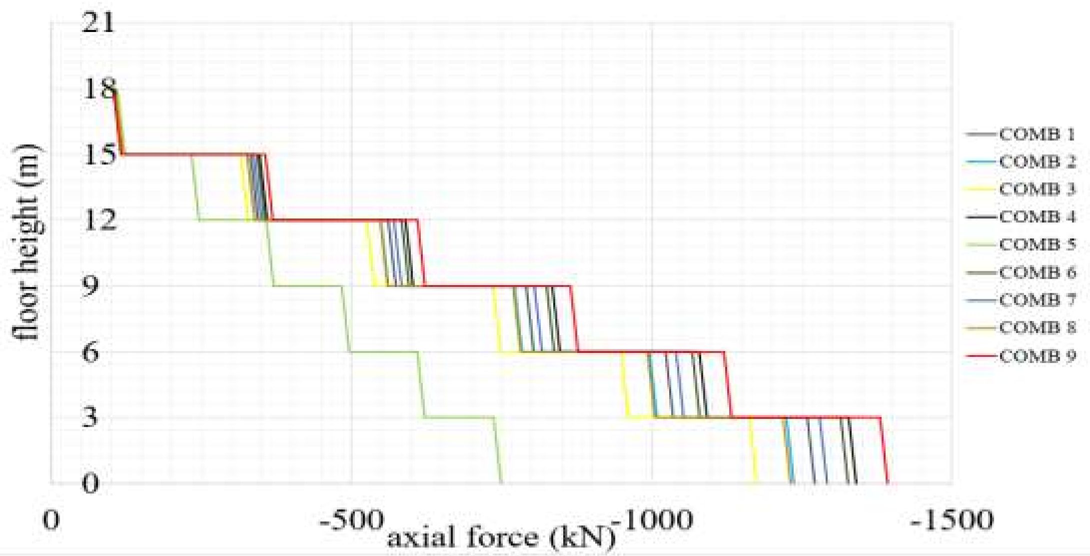


Figure 3.58. Axial forces diagrams

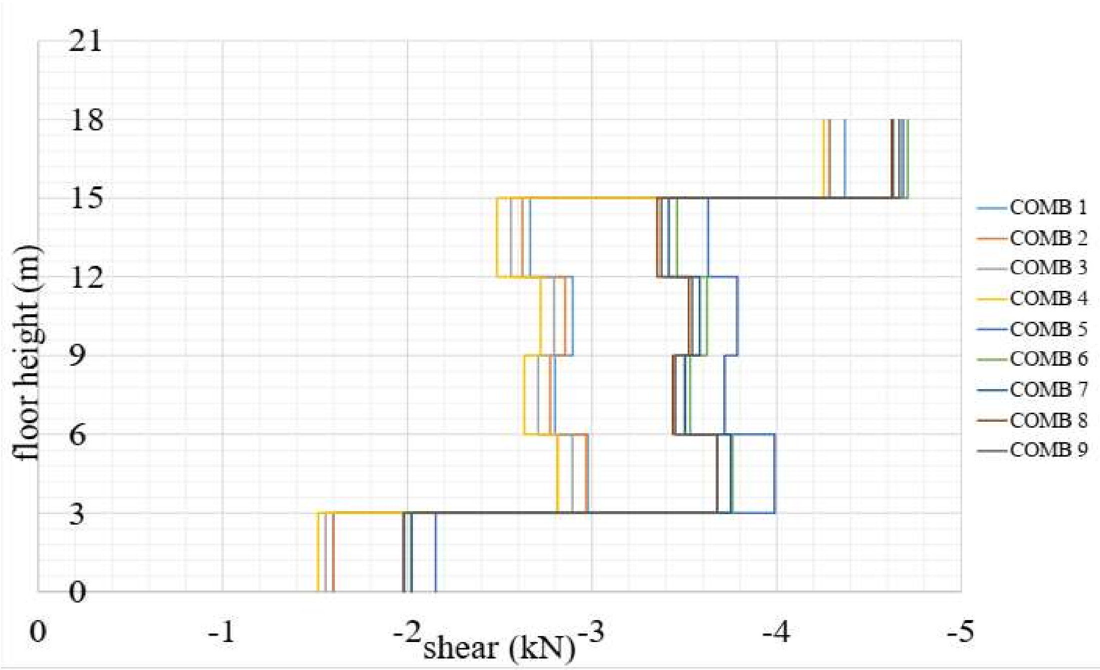


Figure 3.59. Shear diagrams in the y direction

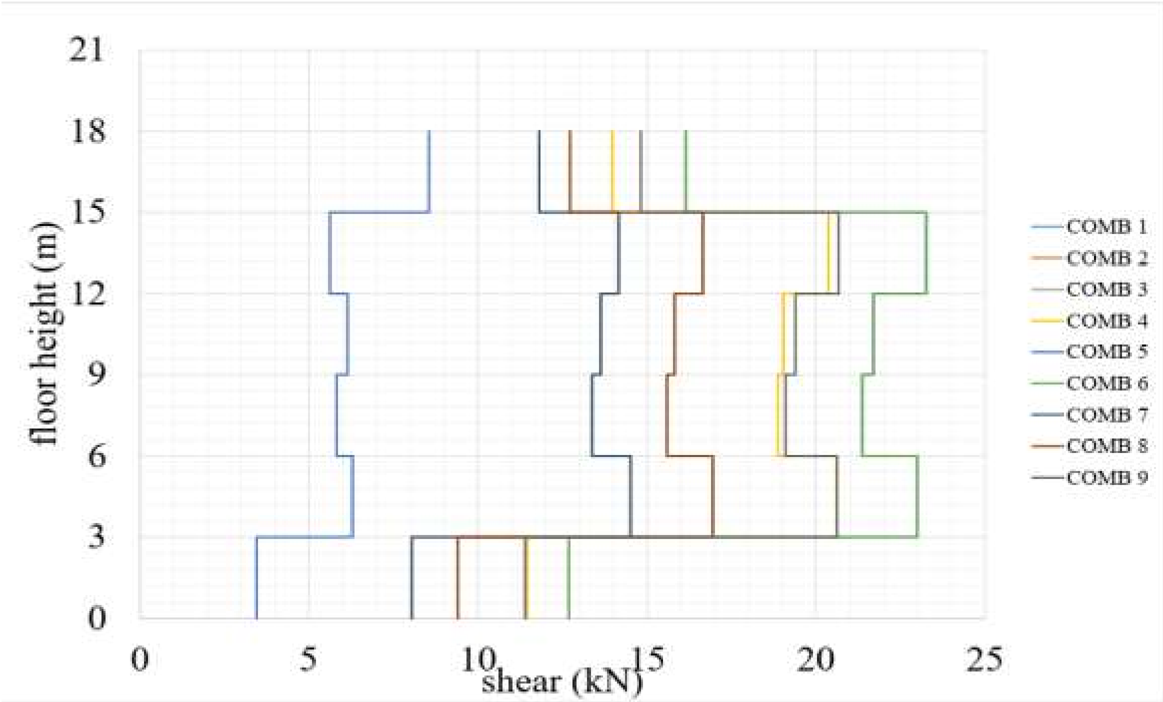


Figure 3.61. Shear diagrams in the x-direction

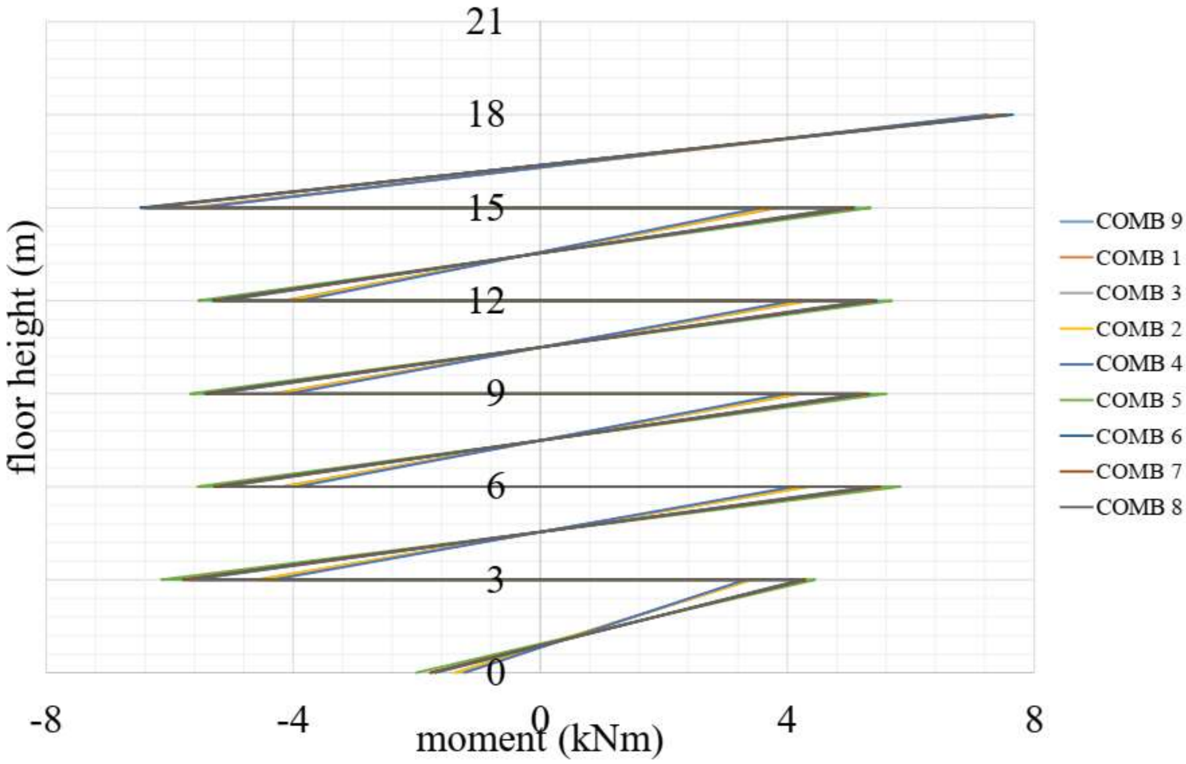


Figure 3.60. Moments diagrams in the x-direction

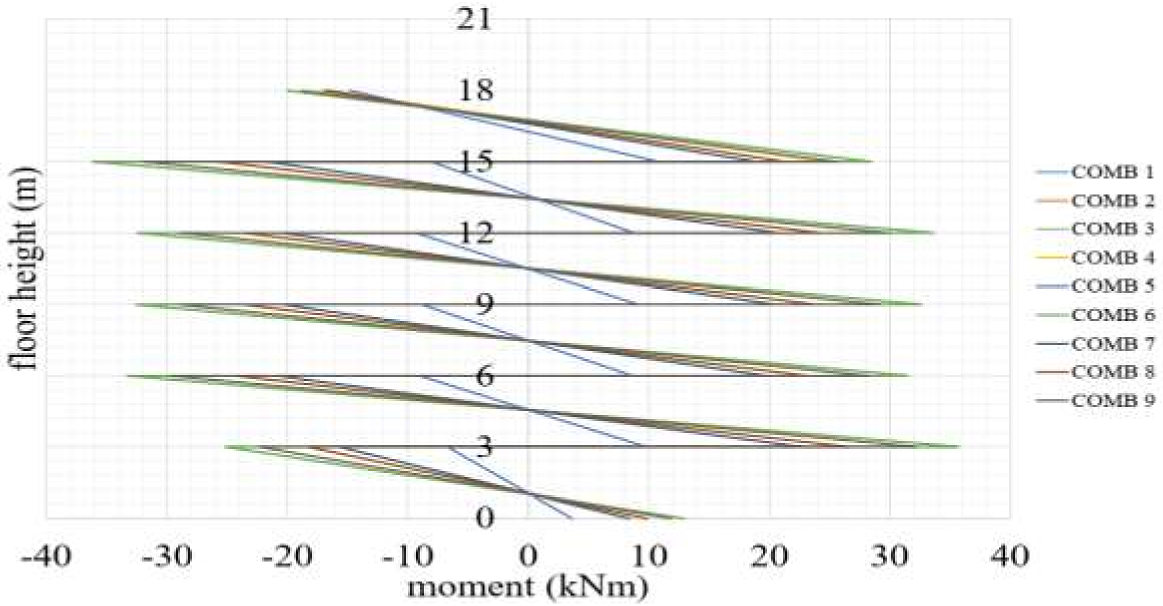


Figure 3.62. Moment diagrams in the y-direction

In order to design and carry out verifications, the envelope of the above solicitations is done for the ninth combinations. The envelope of axial force, the shear and bending moment in the x and y direction are displayed in figures 3.65, 3.66, 3.67 3.68 and 3.69.

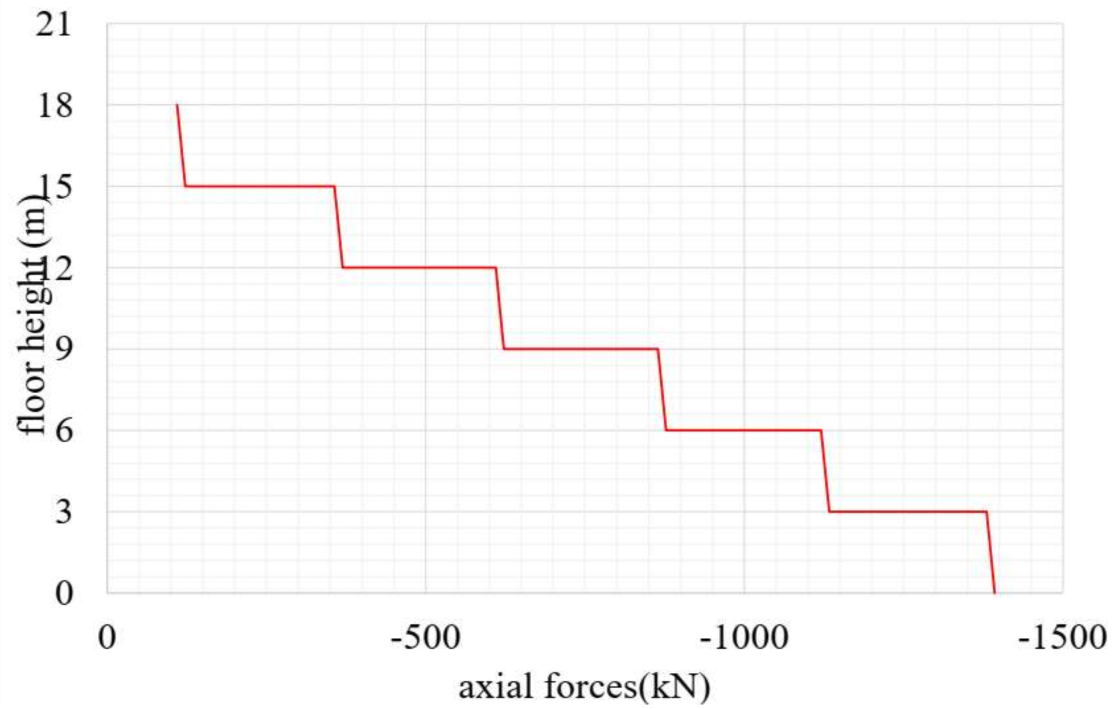


Figure 3.63. Axial force envelope

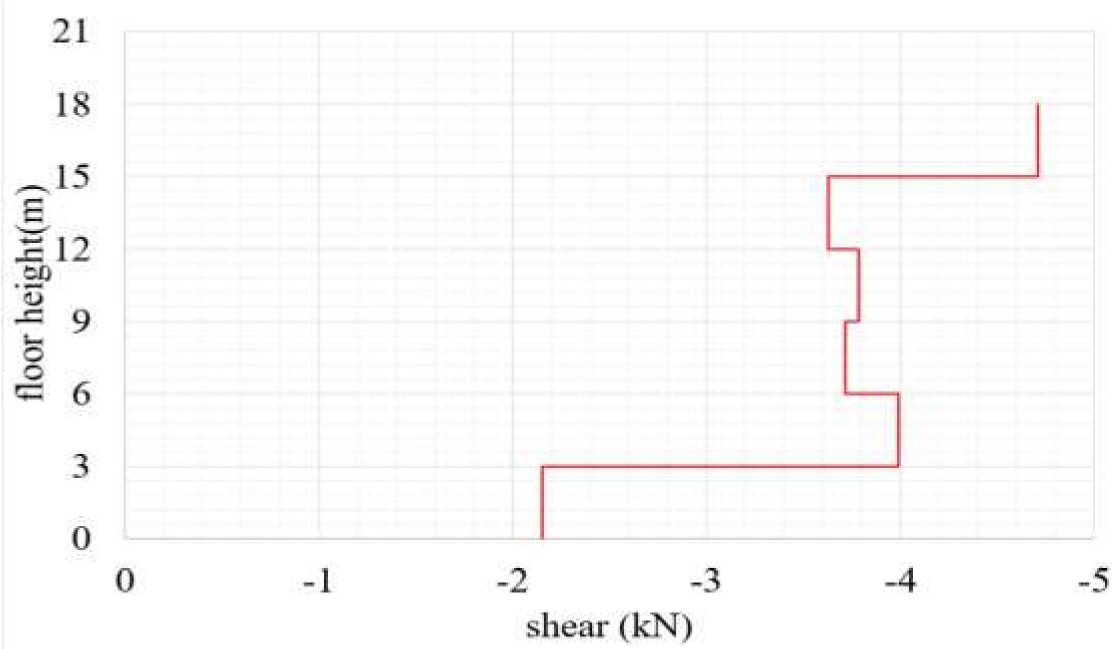


Figure 3.65. Shear envelope curve in the x direction

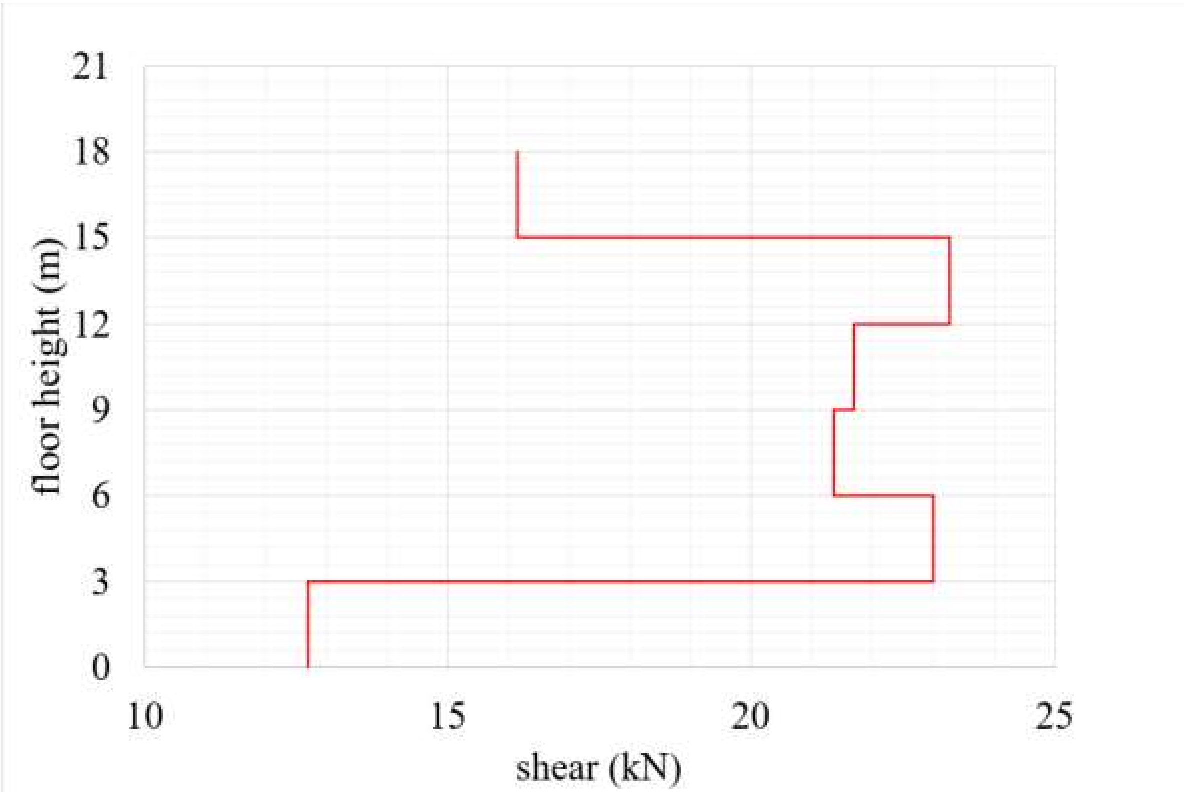


Figure 3.64. Shear envelope curve in the y direction

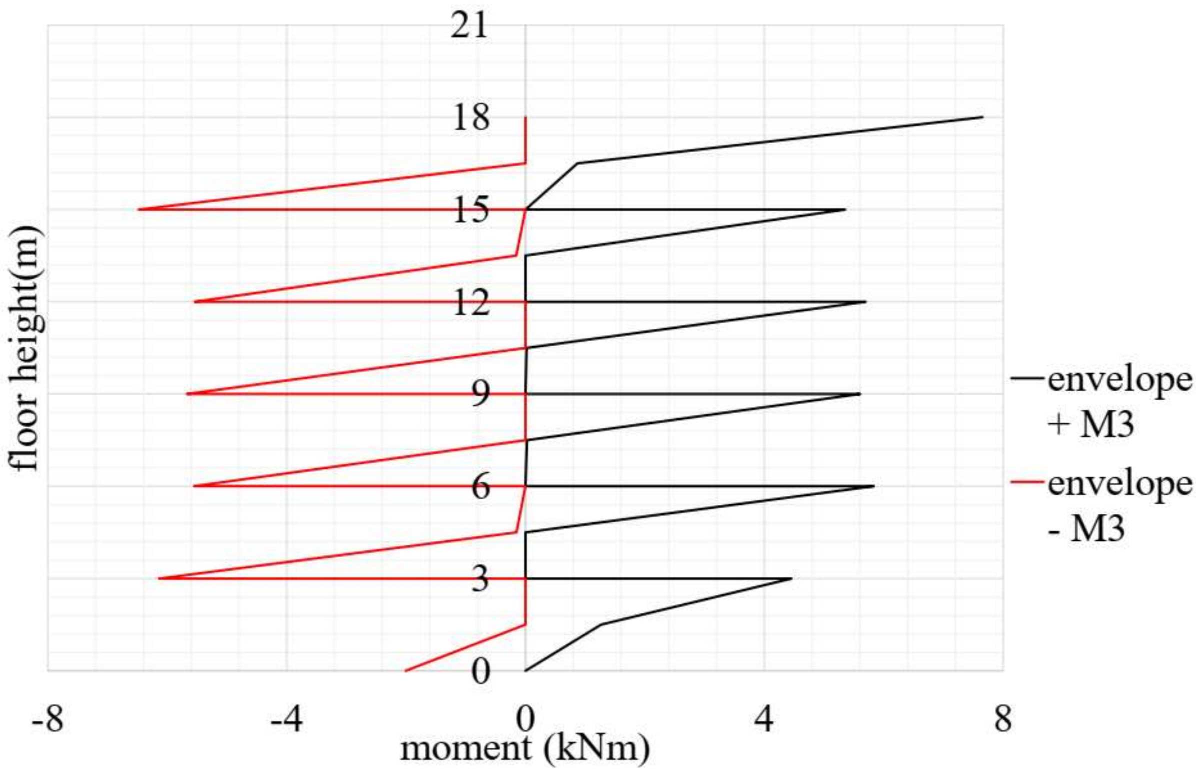


Figure 3.66. Bending moment envelope curve in x direction

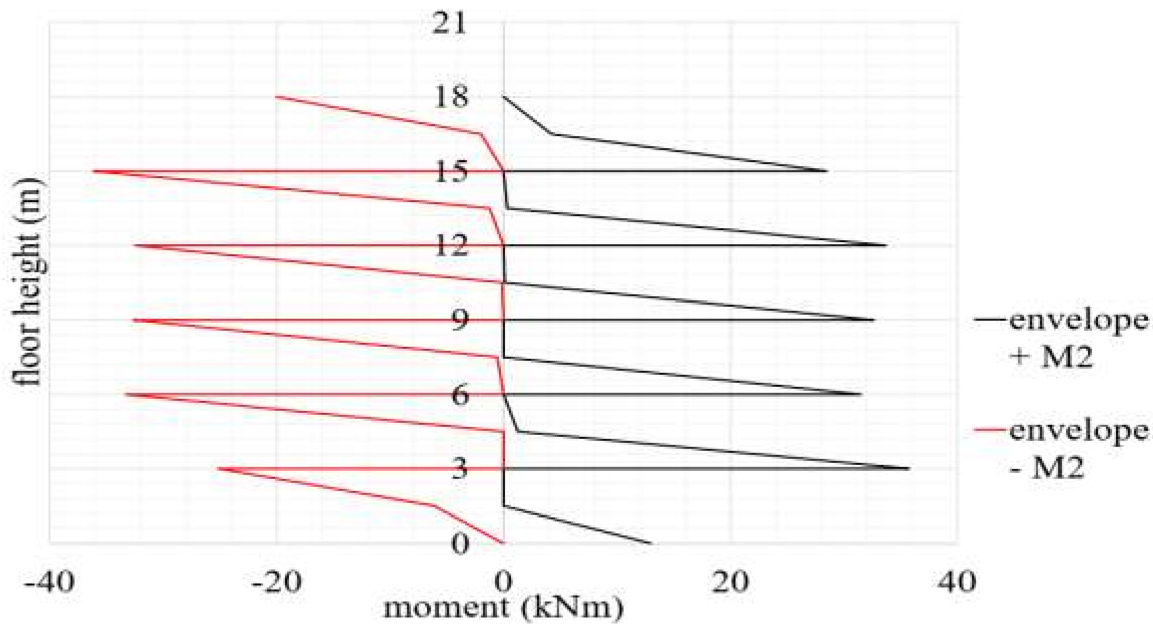


Figure 3.67. Bending moment envelope in the y direction

3.1.7.3. Bending moment and axial force verification

The verification of the axial loads and the bending moment is done through the interaction diagram as presented in section 2.2.4. Complying with Eurocode 2 as described in the same section, $355.94mm^2 \leq A_s \leq 5000mm^2$. The column will be provided with 4ϕ20 and 6ϕ16. In this, the section will be verified at each level as represented in figure 3.70 and 3.71.

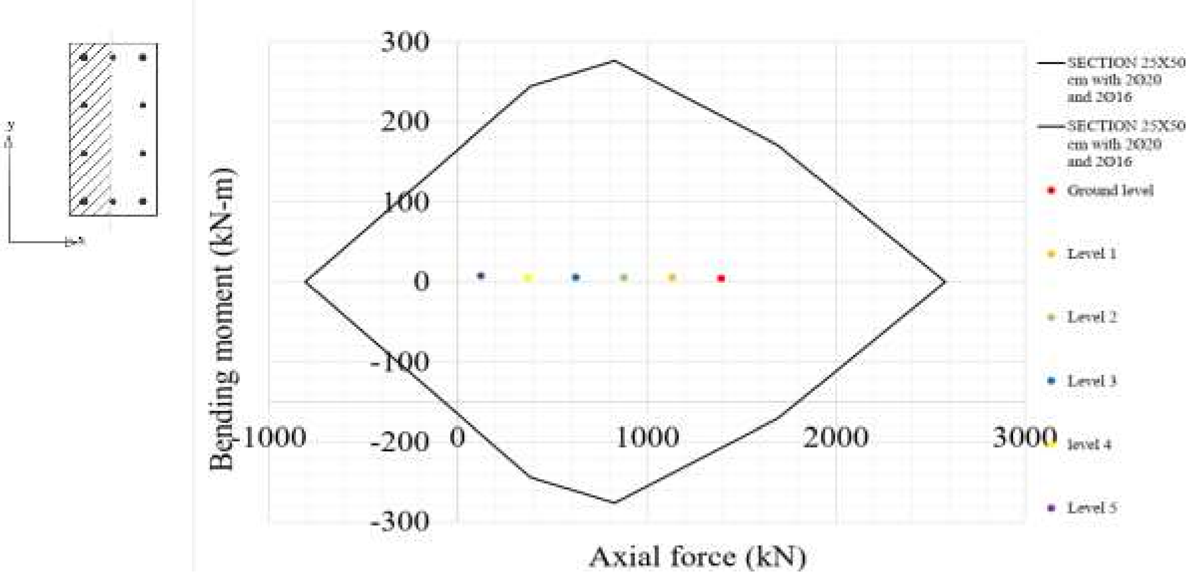


Figure 3.68. M-N interaction diagram in the x-direction

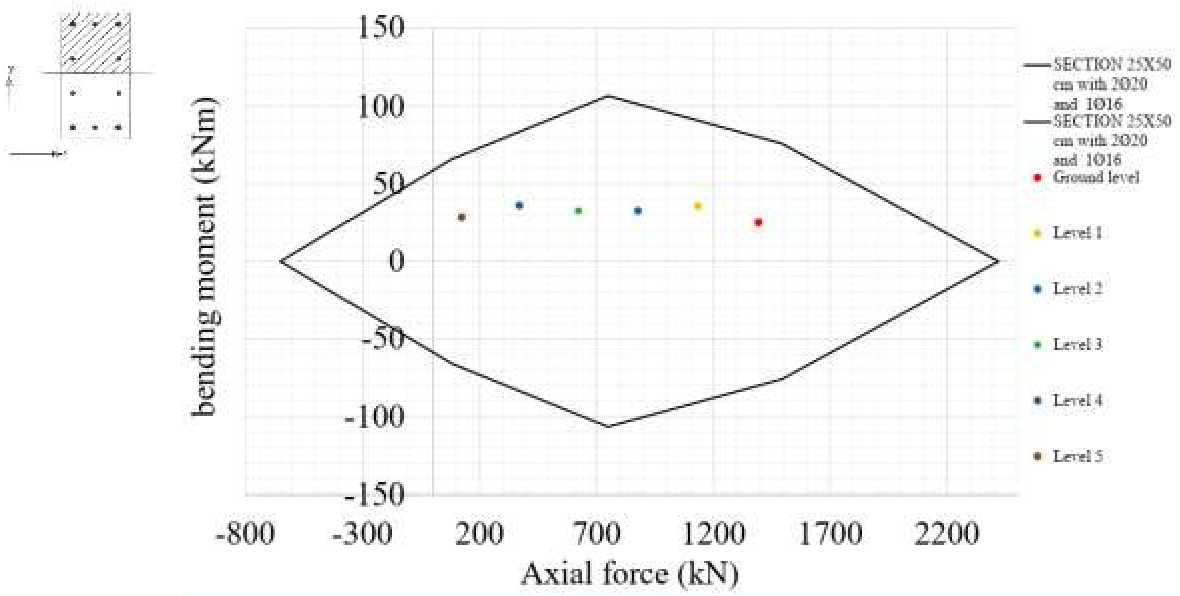


Figure 3.69. M-N interaction diagram in the y-direction

For all columns on the line E3, the M-N (bending moment-axial force) values lies within the interaction diagrams in both direction. Hence verified.

3.1.7.4. Shear verification

From the shear solicitations displayed in the previous part and following the procedure explained in section 2.2.4.2, it is observed that the shear resistance of the section without shear reinforcement is greater than the maximum shear solicitation on the column. Hence for $\phi 8$ used, the maximum spacing of the transverse reinforcement is:

$$S_{cl,max} = \min(320; 250; 400) = 250mm$$

Hence, a spacing of 20cm will be applied.

3.1.7.5. Slenderness verification

Following the steps in 2.2.4, the parameters to compute are displayed in table 3.22

Table 3.22. Parameters for slenderness verification

n	ω	A	B	C	λ_{lim}	λ
0.79	0.45	0.70	1.45	0.7	15.97	14.549

From the table 3.22 we have $\lambda < \lambda_{lim}$, so the slenderness of the column is verified.

3.1.7.6. Dispositions

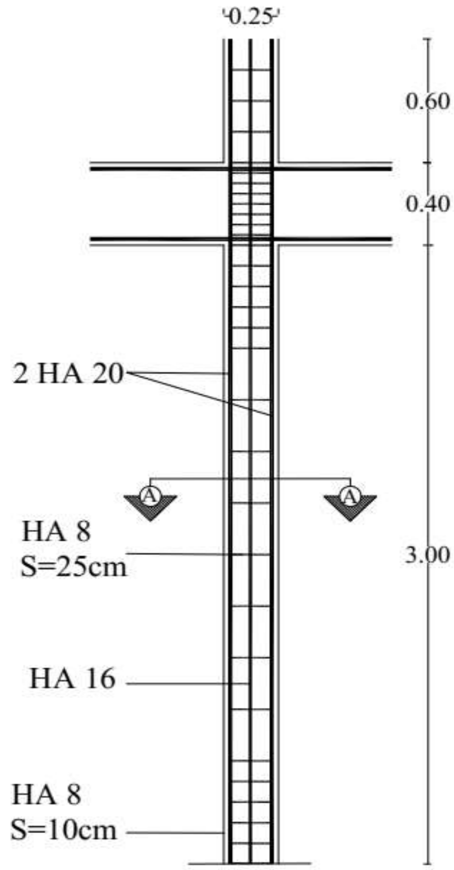


Figure 3.71. Reinforcement of column E3

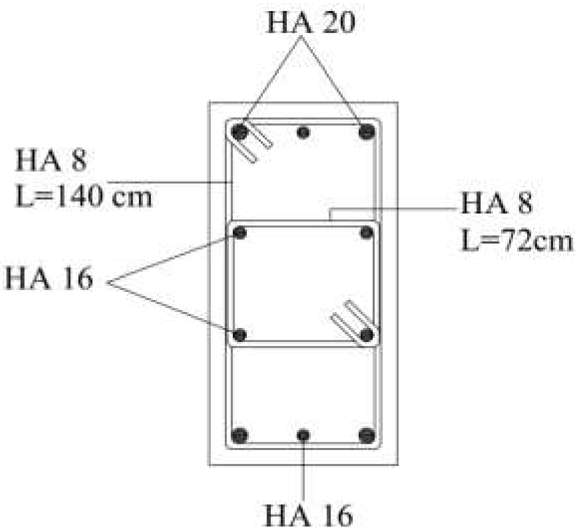


Figure 3.70. Section AA (transversal view of column reinforcement)

3.1.7.7. Axial force for the soil structure interaction model

Modelling using soil-structure interaction is more realistic compared to fixed base model. Table 3. Gives the variation in axial force for column E3.

Table 3.23. Axial force variation for fixed base model and raft foundation model

Storey level	Fixed base (kN)	Raft foundation(kN)
Ground level	1385.27	1387.05
Storey 1	1127.22	1128.68
Storey 2	872.40	873.47
Storey 3	619.07	619.84
Storey 4	367.07	367.58
Storey 5	121.8	122.31

From table 3.23, there is no great difference between axial force in the fixed base and in raft foundation. This is due to the fact that medium dense soil have been used with a high elastic modulus. Hence the designed column can equally be applied for the raft foundation model for the static design.

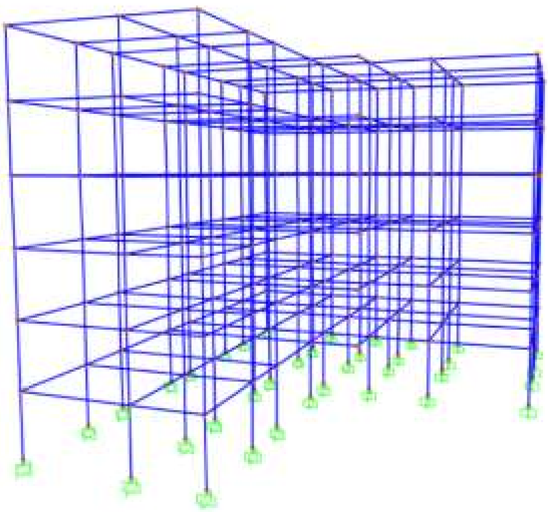
3.2. Analysis response

Deals with both (rigid slab and non-rigid slab) models for a fixed base foundation and raft foundation loaded under both static and variable loads.

3.2.1. Structural models

The structure modelled on sap2000 software has the beam section in reinforced concrete of section 20cm×40cm; column section is 25cm×50cm and one-way reinforced concrete Slab 20cm thick. The parametric study is carried out two models of a fixed base and on a raft foundation; one with a rigid slab, and the other with a non-rigid slab. The slab is made rigid by applying diaphragm constraint at each floor. The elastic response early describe in section 3.1.3 in applied in each case and different results are analysed using the period, the lateral displacement, the inter-storey drift, the axial force, the base shear. Figure 3.74 and 3.75 display the rigid slab model and the non-rigid slab model for the fixed base and raft foundation respectively. The raft foundation is modelled with shell element is meshed for an element of 0.5 by 0.5 and a thickness of 0.5m with a spring constant in the x and y direction obtained from expression 2.46 to be:

$$k_{middle} = 27500kN/m \text{ and } \; k_{edge} = 13750kN/m$$



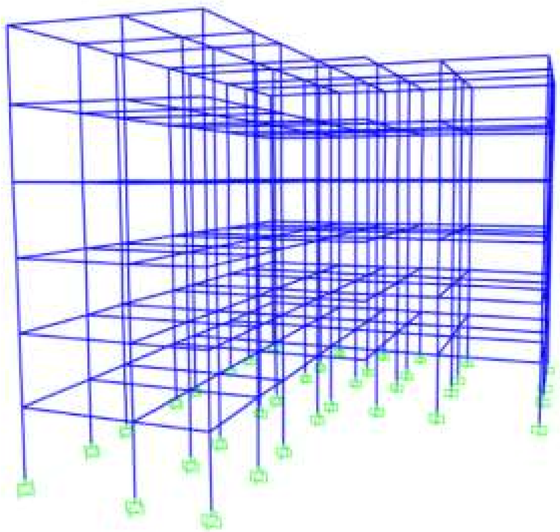


Figure 3.72. Rigid slab model (left) and non-rigid slab model (right) with fixed base

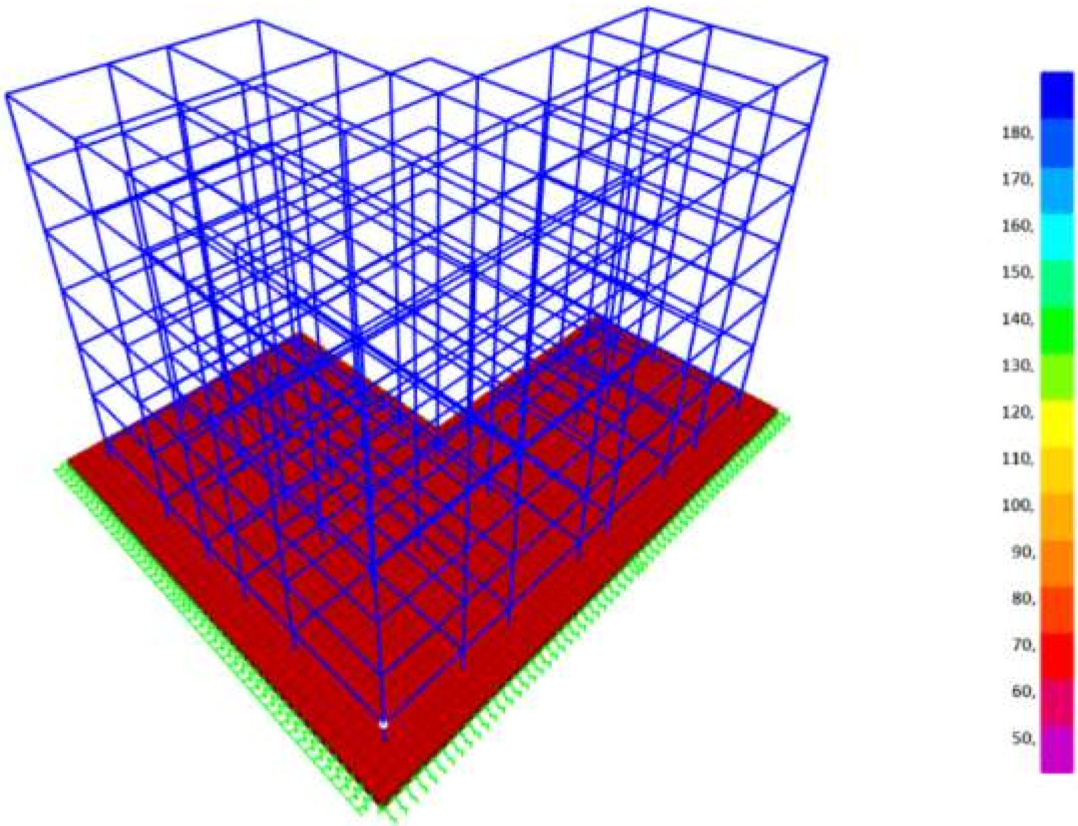
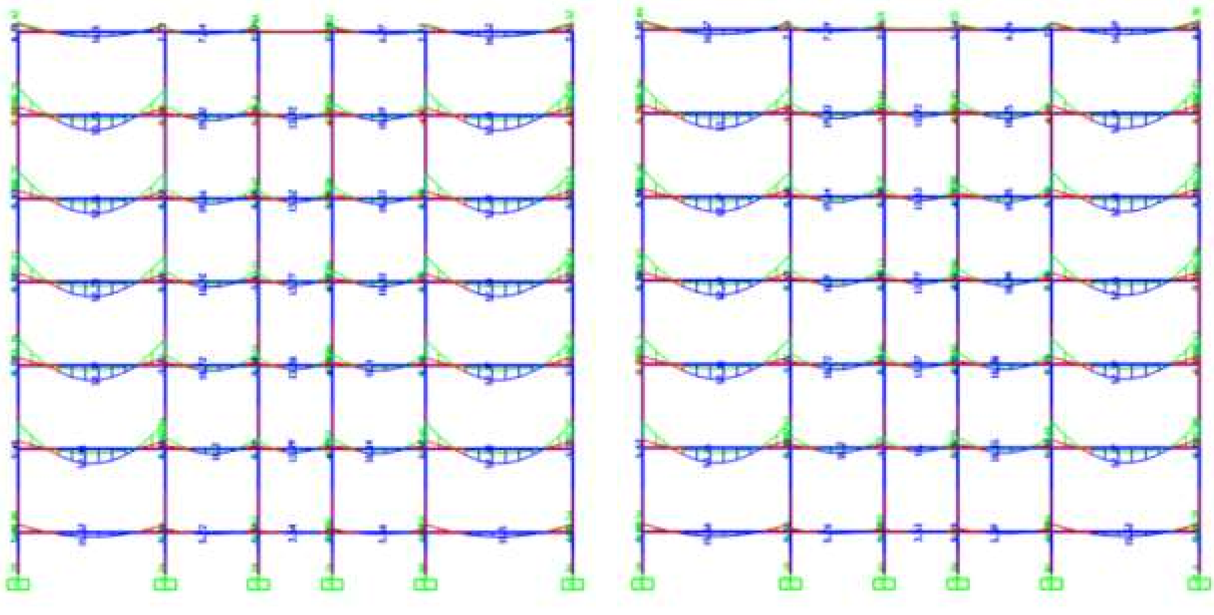


Figure 3.73. Soil pressure solicitation Model with raft foundation

3.2.2. Gravity load distribution

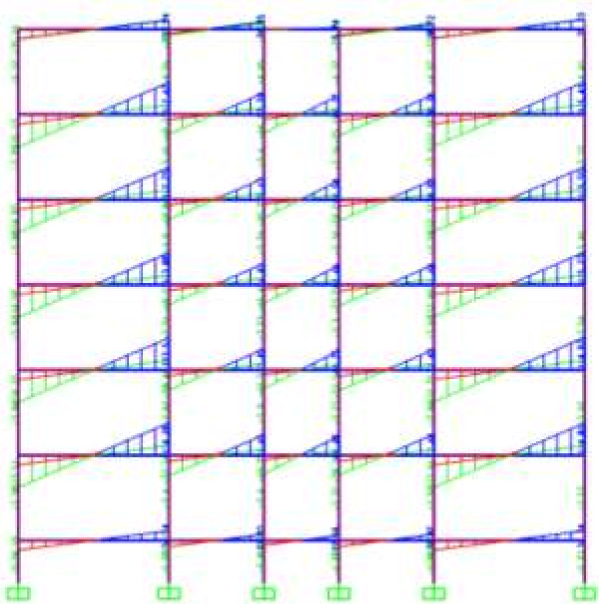
Under gravity loads, the solicitations (moment shear and axial force) for each structure are displayed from the envelope combinations. Following details observations and as displayed by figure 3.76 and 3.77, the moment solicitation and shear solicitation does not vary under the application of gravity loads for both models of the fixed base. The rigid slab model and the non-rigid slab model has approximately the same value of moment and shear point by point. However, as displayed by figure 3.78, there is a difference in the axial force solicitation due to gravity loads. Clearly, in image "a" of the above mentioned figure, the rigid slab model has no axial force present in the horizontal element (beam) and in image "b" of the same figure, there is an axial force present in the horizontal element which goes to a maximum value of 13.83 kN (not negligible) for the fixed base. Hence showing that flexible slabs have to be avoided because a beam is design in flexion (without axial forces).

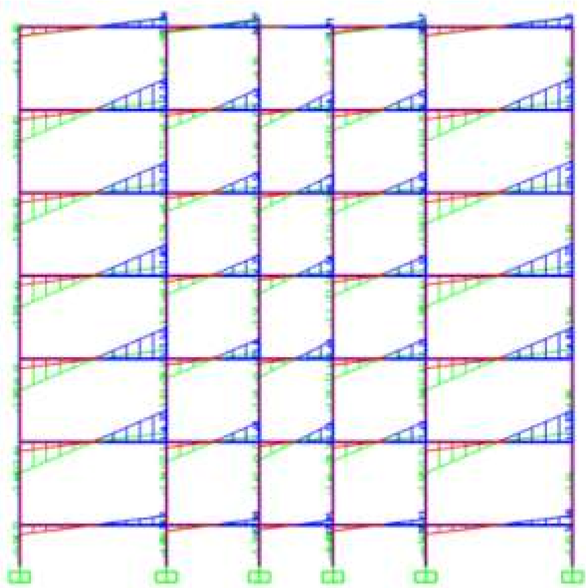
Likely, in the raft foundation models, equal observations are made where the moment and shear distribution is the same for the rigid and for the non-rigid slab model and a value of axial force in horizontal elements (beams). The maximum axial force has a value of 13.47kN.



a b

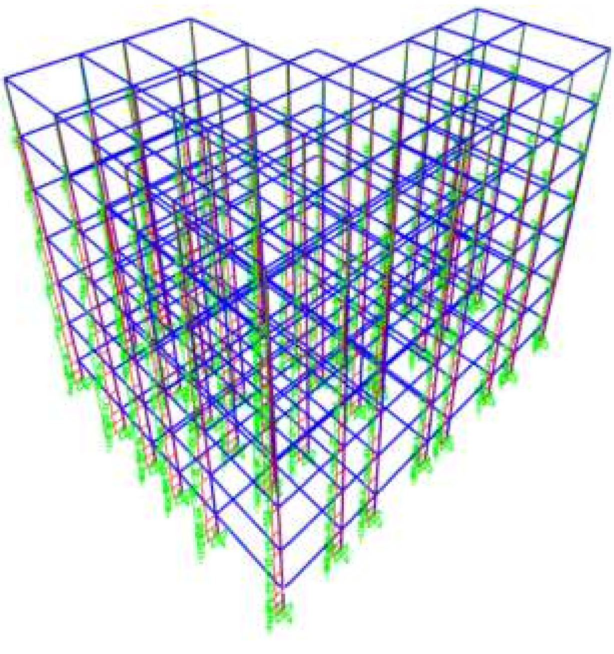
Figure 3.74. Moment solicitations (3-3) for fixed base a) rigid slab model and b) non-rigid slab model

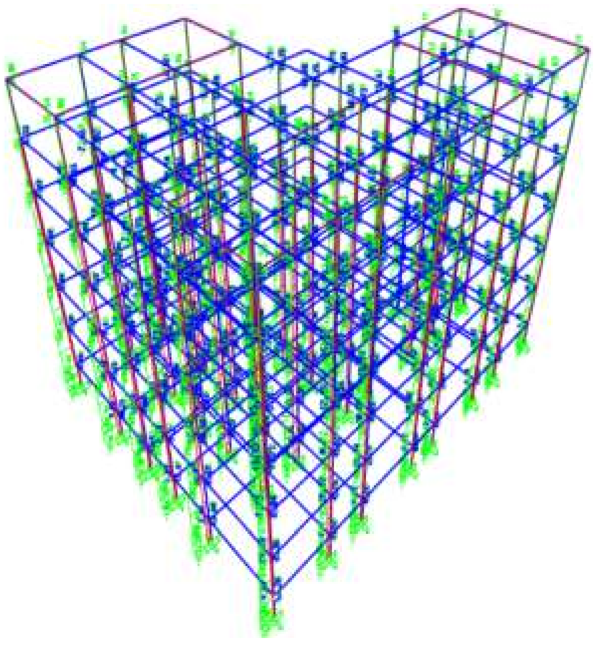




a b

Figure 3.75. Shear solicitations (2-2) for a) rigid slab model and b) non-rigid slab model



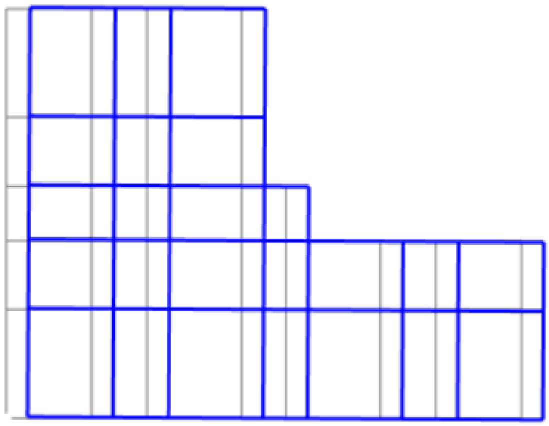


a b

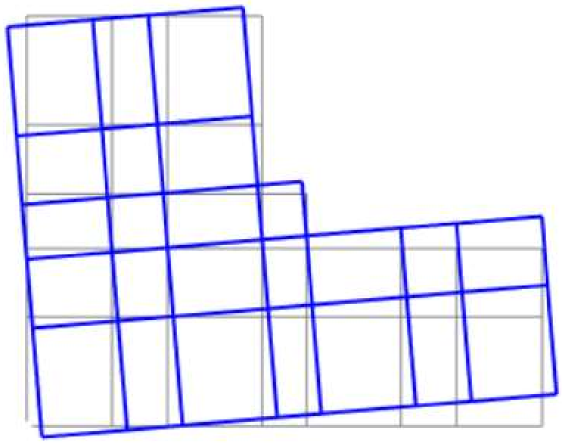
Figure 3.76. Axial force solicitations for fixed base a) rigid slab model and b) non-rigid slab model

3.2.3. Period of vibration

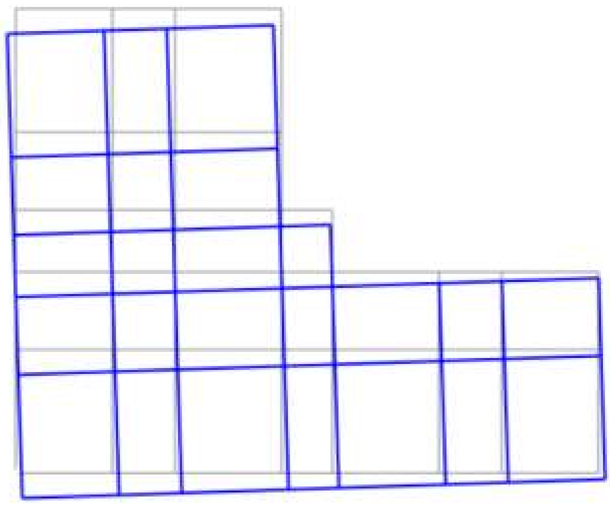
Resulting from the modal analysis as described in section 2.4, figure 3.79 shows the different deformations pattern for both rigid and non-rigid model for the fixed base and raft foundation.



Mode 1 (translation in x direction)



Mode 2 (rotation about z)



mode 3 (translation in y direction and rotation)

Figure 3.77. First three modes patterns of the modal analysis

For the first three modes, as represented by figure 3.79, the rotation (mode 2 and mode 3) overtakes on translational motion (mode 3) due to the high degree of irregularity present in the structure. This situation creates consequent torsion in the structure which if not controlled can easily leads to damage. A possible solution to reduce torsion will be to regularise the structure by applying rupture joint which is a gap of distance h(cm)/100 corresponding to 18cm gap in the appropriate section.The periods obtained for the first twelve modes are representated in table 3.23 and illusrated in figure 3.78.

Table 3.24. Periods of rigid and non-rigid slab for fixed base and raft foundation.

modes	Fixed base		Raft foundation	
	Rigid period (s)	**Non-rigid period (s)**	**Rigid period (s)**	**Non-rigid period (s)**
1	1.23638	1.23932	1.35606	1.35871
2	1.01915	1.05029	1.14504	1.16280
3	0.93915	0.94672	1.07231	1.07841
4	0.40689	0.83040	0.44397	0.88521
5	0.31857	0.70359	0.35932	0.73136
6	0.29229	0.61842	0.33901	0.64173
7	0.24003	0.50016	0.26177	0.51064
8	0.17485	0.48469	0.20521	0.49706
9	0.17243	0.45697	0.19247	0.47010
10	0.15901	0.41355	0.18862	0.44941
11	0.14008	0.38338	0.16087	0.40984
12	0.12744	0.34870	0.14778	0.36681

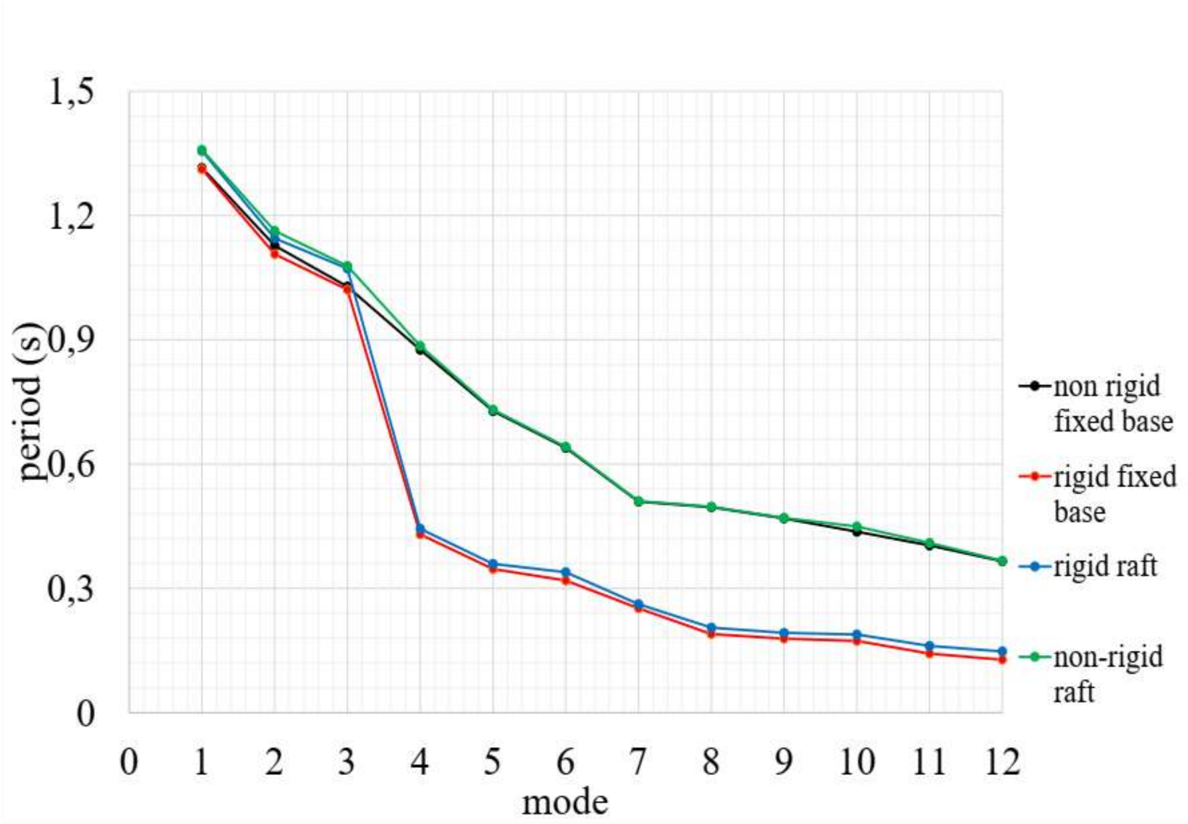


Figure 3.78. Period variation for rigid and non-rigid model

From figure 3.79, for the fixed base model, it is observed little variation in the period for the first three modes but from mode 4, there is a large difference but the rigid model having a relatively high period. This high period denotes that a rigid slab increases the rigidity of the global structure and a non-rigid slab decreases the rigidity of the structure. Furthermore, the modal analysis being an important parameter for a proper seismic design, it is observed that in the x direction for the fixed base model, the modal mass participation ratio as prescribed by Eurocode (at least 90%) for the rigid slab model is attained in the fourth mode compared to the non-rigid slab model which is attained in the thirteen mode. In the same move, in the y direction, the mass participation ratio is attained in the eleventh mode compared to the non-rigid slab model which is attained at the thirty-fifth mode. This results denote the fact that more modes will be taken into consideration for the seismic design and more torsion in the non-rigid slab model.

The model with the raft foundation follows the same pattern as the fixed base model but with relatively higher periods. For the rigid slab model, the mass participating ratio is attained at the eleventh and tenth mode respectively in the x and y directions whereas, the non-rigid slab model attain the mass participating ratio at the forty-fourth and forty-fifth mode respectively in the x and y direction. These result illustrate the fact that the fixed base model provides more stiffness to the structure compare to raft foundation model which is relatively more realistic.

3.2.4. Lateral displacement

Resulting from the application of seismic load Ex and Ey, the envelope effects from the seismic combination result to the deformation pattern displayed in figure 3.81 and 3.82 for the rigid and non-rigid model respectively for the raft foundation.

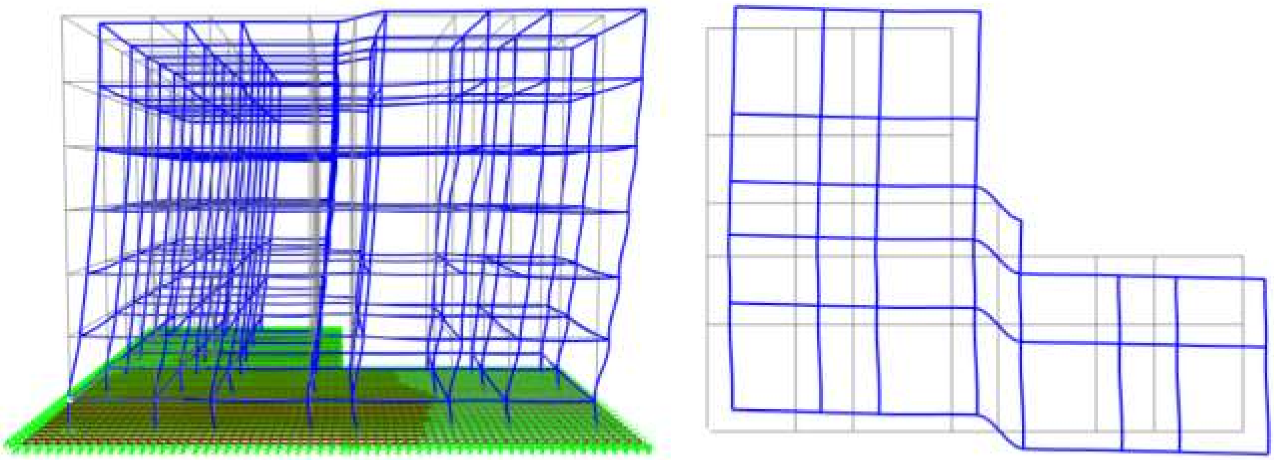


Figure 3.79. Deformation due to seismic load in the rigid slab model with raft foundation

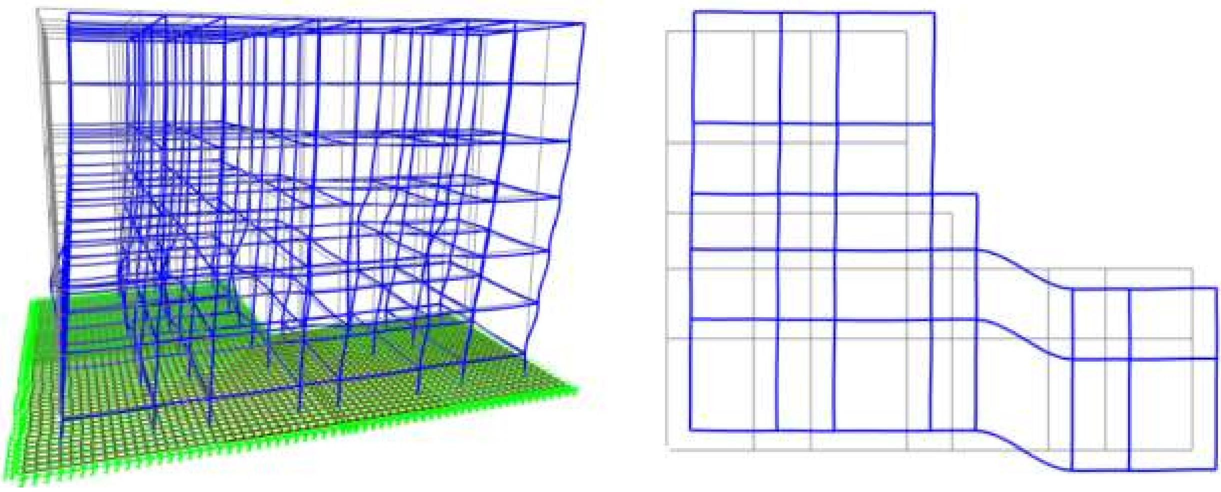


Figure 3.80. Deformation due to seismic load in the non-rigid slab model with raft foundation

The haphazard in-plane deformation of a non-rigid slab model makes it difficult to evaluate the lateral displacement. However, in other to evaluate the storey displacement in each floor, a fix point is consider both in the rigid slab model and in the non-rigid slab model. The results are presented in table 3.25 and 3.26, illustrated by figure 3.83 and figure 3.84.

Table 3.25. Lateral displacement of in both x and y direction for the fixed base models

Storey level	Rigid slab model		Non-rigid slab model	
	U1(x-direction)	U2(y direction)	U1(x-direction)	U2(y direction)
0	0.54	0.49	0.46	0.37
1	3.8	3.20	3.21	2.44
2	7.06	6.23	5.96	4.80
3	9.82	8.89	8.32	6.90
4	11.94	10.94	10.13	8.53
5	13.30	12.26	11.30	9.58
6	13.85	12.87	11.79	10.08

Table 3.26. Lateral displacement of in both x and y direction for raft foundation models.

storey level	Rigid slab model		Non-rigid slab model	
	U1(x-direction)	U2 (y direction)	U1(x-direction)	U2 (y direction)
0	1.21	1.27	1.02	1.00
1	4.54	3.97	3.81	3.03
2	7.84	6.96	6.57	5.32
3	10.63	9.55	8.92	7.35
4	12.77	11.56	10.75	8.92
5	14.16	12.86	11.97	9.95
6	14.75	13.48	12.44	10.45

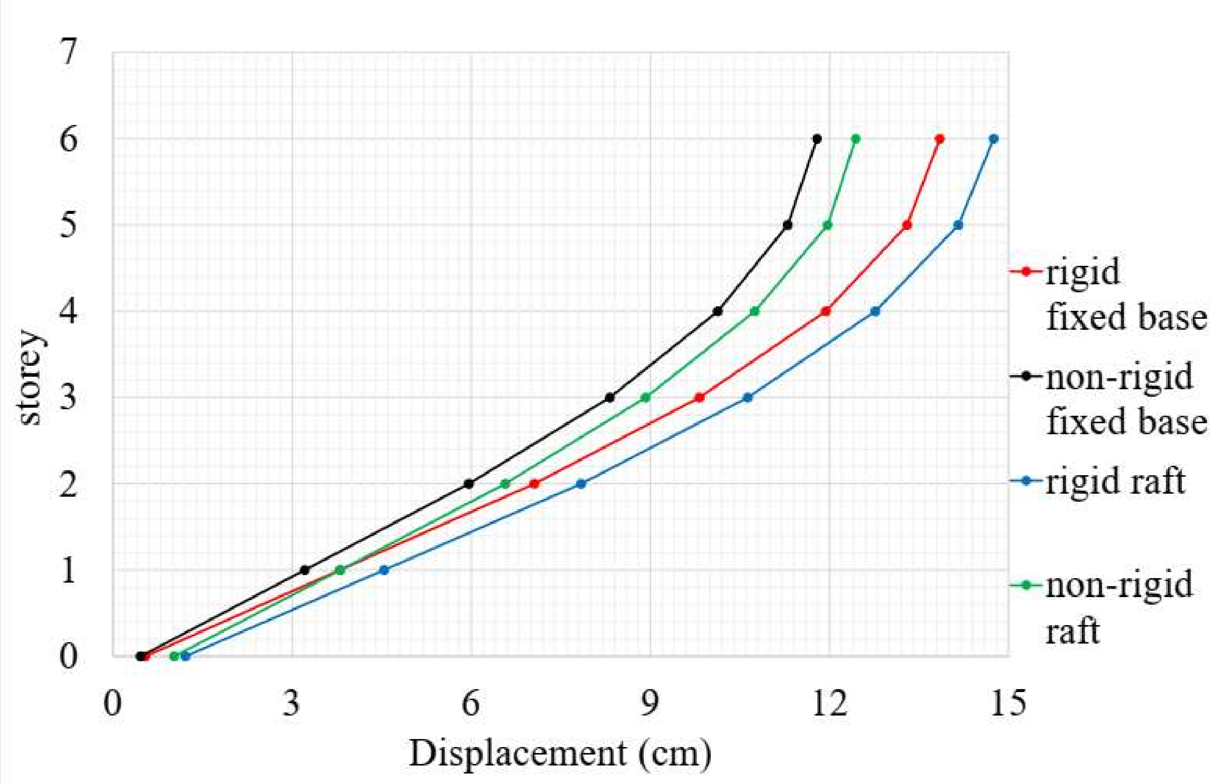


Figure 3.81. Lateral storey displacement in the x direction

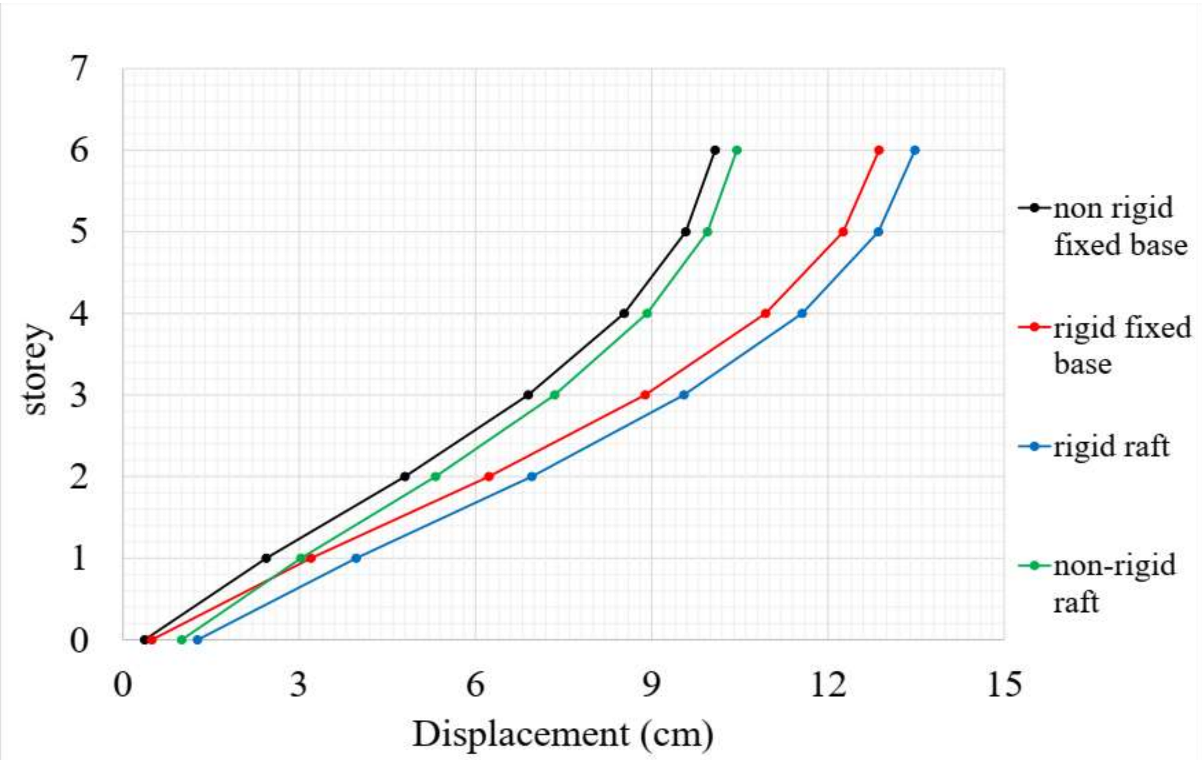


Figure 3.82. Lateral storey displacement in the y-direction

Indeed, as expected from previous studies, from the fixed base model, the rigid slab model is observed to have a relatively higher lateral storey displacement along the x direction. The maximum lateral displacement in the rigid slab is 13.85 cm and in the non-rigid slab is 11.79 cm hence a difference of 2.06 cm. Likewise, in the y direction, the same observations are made. The maximum lateral displacement in the studied direction is higher in the rigid slab model. The difference in the lateral displacement is 2.79cm. This value is higher in the y direction because of the L-shape of the structure which is more accentuated in the y direction.

Higher displacements values are observed in the raft foundation model, with the rigid model having a maximum displacement of 14.79 cm in the x-direction and the non-rigid model shows a maximum displacement of 12.44 cm in the same direction hence a difference of 2.31 cm. the difference in lateral displacement in the y-direction is 3.03cm. From these result, the fixed base method under estimate lateral deformations.

3.2.5. Inter-storey drift

Used as an index to evaluate the deformation capacity of a building and to further determine its performance, the limit set by Eurocode 1.5%. From the importance class (class II) of the model, the recommended reduction factor V = 0.5.Computed from the lateral displacement at each floor the variation of the inter-storey drift is summarised as displayed in table 3.27 and 3.28 for the fixed base and raft foundation respectively.

Table 3.27. Inter-storey drift in x and y direction for fixed base

Floor level	Rigid slab model		Non-rigid slab model	
	x-direction	y-direction	x-direction	y-direction
0	0.00	0.00	0.00	0.00
1	1.63	1.36	1.38	1.04
2	1.63	1.52	1.38	1.18
3	1.38	1.33	1.18	1.05
4	1.06	1.03	0.91	0.82
5	0.68	0.66	0.52	0.53
6	0.28	0.31	0.25	0.25

Table 3.28. Inter-storey drift in x and y direction for raft foundation

Floor level	Rigid slab model		Non-rigid slab model	
	x-direction	y-direction)	x-direction	y-direction
0	0.00	0.00	0.00	0.00
1	1.67	1.35	1.40	1.02
2	1.65	1.50	1.38	1.15
3	1.40	1.30	1.18	1.02
4	1.07	0.01	0.92	0.79
5	0.69	0.65	0.61	0.52
6	0.30	0.31	0.23	0.25

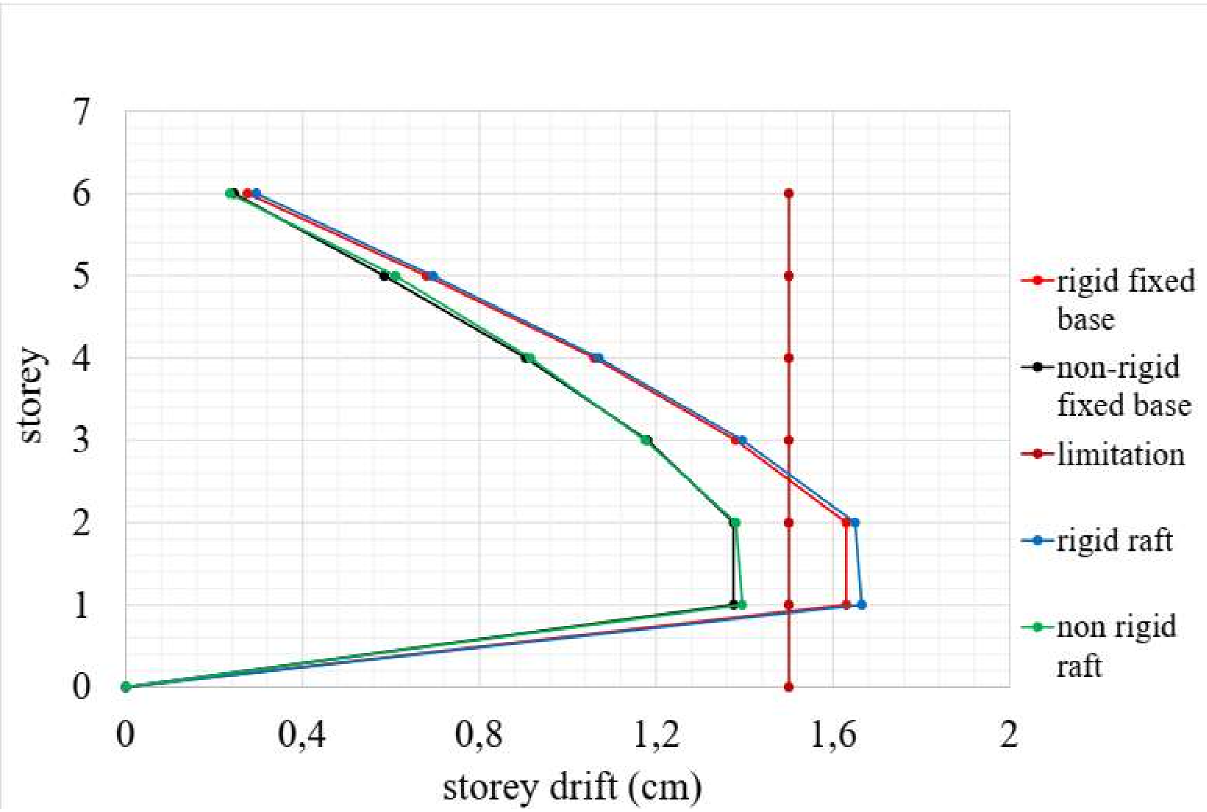


Figure 3.83. Inter-storey drift in the x-direction

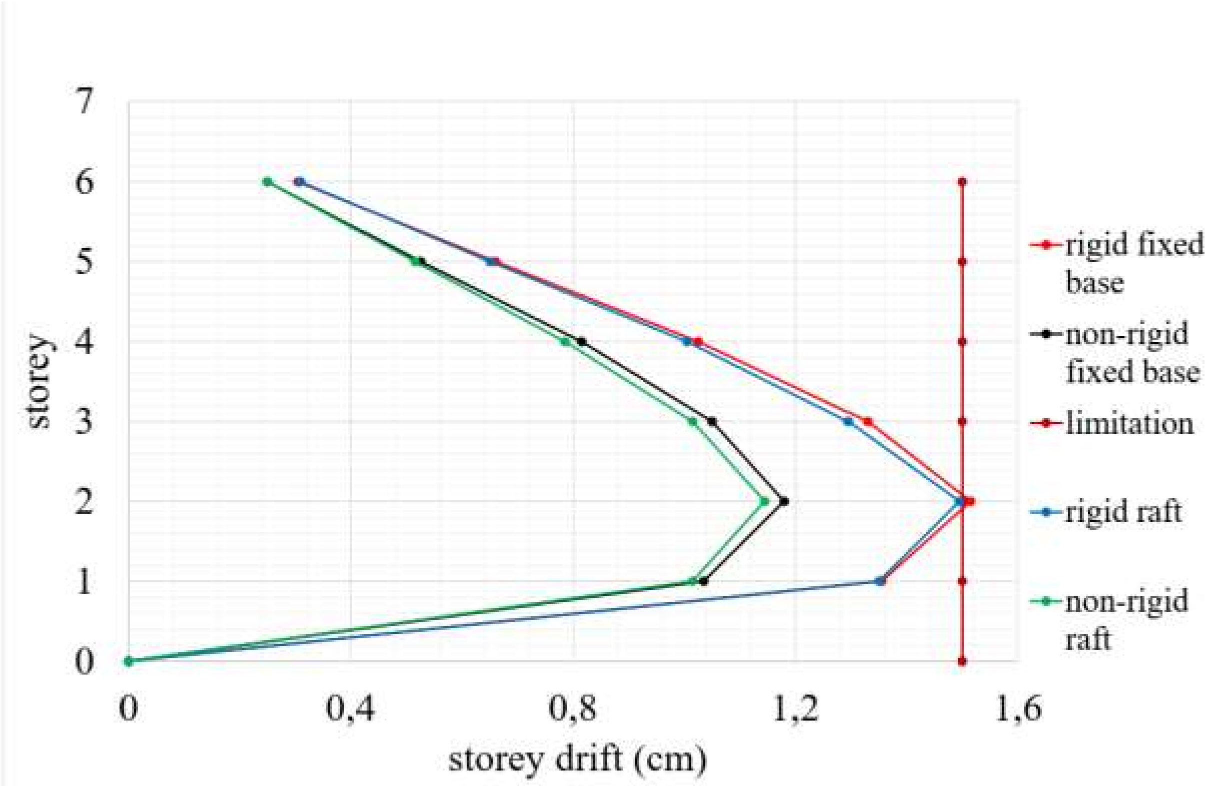


Figure 3.86. Interstorey drift in the y-direction

The inter-storey drift affect the seismic performance of the structural elements such as beams, columns and non-structural elements like the interior partitioning, glass windows, shaft. Referring to figures 3.85 displaying the inter storey drift in the x-direction, taking into account slab rigidity and the inter storey drift limitations is not respected by the rigid slab model either in fixed base or in raft foundation. However, the limitations are not attain in the flexible slabs models for both cases hence no possible damage. The maximum inter-storey drift for the fixed base model is observed in storey 2 corresponding to a height of 6 m to be 1.67 cm corresponding to the rigid model. As so the raft foundation, maximum inter-storey drift is observed in storey 1 (3m height) with an equal value of 1.67 cm in the x direction.

The inter-storey drift is relatively reduced in the y direction as illustrated by figure 3.86. Following the same pattern as in the x direction, the maximum values are observed in the rigid models of either the fixed base or the raft foundation. Its value is reduced in the rigid slab model of the raft foundation. As for the rigid slab model of the fixed base, the limitation is attainted with a value of 1.52.

3.2.6. Base shear

The base shear resulting at the ground level due to the application of a ground acceleration (response spectrum) is evaluated and presented in table 3.29.

Table 3.29. Base shear computed from response spectrum

Base shear (kN)	Fixed base models		Raft foundation model	
	Rigid slab model	Non-rigid slab model	Rigid slab model	Non-rigid slab model
X direction	5244.19	5214.68	5769.38	5691.46
Y direction	6288.74	6272.84	6779.20	6711.12

Higher base reaction is observed in the rigid slab models than in the non-rigid slab models in both directions. The rigid slab model tends to affect the overall rigidity of the structure and hence increasing the robustness of the structure to motion. The differences between the base shears for the rigid slab model and non-rigid slab model in the x-direction are 29.51kN and 77.92kN corresponding to a percentage decrease of 0.6% and 1.3% respectively for the fixed base and raft foundation model.

Conclusion

In drawing to a close, this section was aimed at presenting the case study, static designing, applying the elastic response spectrum and analysing the response of the rigid slab model and the non-rigid slab model under fixed base and raft foundation. For the defined case study, a hollow block slab of thickness 20cm (16+4) is designed, a conventional two-way slab of thickness 20cm is equally designed. The beam section obtained is a 20cm×40cm. as for the vertical element, the section obtained is 25×50cm. Moreover, the analysis of the global solicitations of both models under gravity load only showed the presence of an axial force in horizontal structural elements (beam) in the case of a non-rigid slab model for both the fixed base and the raft foundation. Equally, the seismic analysis through the application of the elastic response spectrum revealed that the rigid slab models have a relatively higher period, inter-storey drift and base shear compare to the non-rigid slab models.

GENERAL CONCLUSION

The end goal of this work was to determine how slab rigidity influences the structural behaviour of tall buildings with irregular plan. In order to better understand the core of the work, the literature review was presented in the first chapter which highlighted the notion of slab rigidity, the concept of irregularity then ending with the concept of tall buildings. The methodology adopted was a static design and dynamic analysis through the application of an elastic spectrum of type I. The static design of an L-shape G+5 residential storey building was done manually on an excel sheet and the solicitations used for the design were obtained from SAP2000 V22. The elastic response spectrum was applied for two foundations systems (fixed base and raft foundation) of the same building, but one modelled with a rigid slab and the other with a non-rigid slab.

The results of the analysis reveals that: **(1)** Flexible slabs on buildings with irregular plan does not greatly affects the first three periods compared to rigid slab on the same building but changes greatly as from mode four where, the rigid model has a relatively high period. Also, the modal mass participating ratio of 90% is attained faster in the rigid slab model than in the flexible slab model. **(2)** The lateral displacement was higher in the rigid slab model than in the flexible slab model. The difference is higher in the y-direction due to the comparative high degree of protrusion in the y direction. **(3)** The inter-storey drift follows the same pattern as for the lateral displacement giving a bigger value in the rigid slab models. Eurocode limitations are not respected for these model in the x-direction. **(4)** The base shear was higher in the rigid model as well. Furthermore, the flexible slab model create an axial force in the horizontal elements (beams) when loaded by gravity loads only. Therefore, in order to have a proper control on displacement modes principally, the rigid slab model is prevail over the flexible slab model in tall building with irregular plan.

The shortcomings related to this work is the non-consideration of the actual in plane rigidity of the slabs modelled and the elastic assumptions of the buildings behaviour. So, as perspectives, a local analysis and slab analysis for both rigid and non-rigid slabs in tall buildings with irregular plan can be studied in order to evaluate cracks development and rupture. Moreover the study can be extended by evaluating the influence of slab rigidity on the structural behaviour of tall buildings in the non-linear field.

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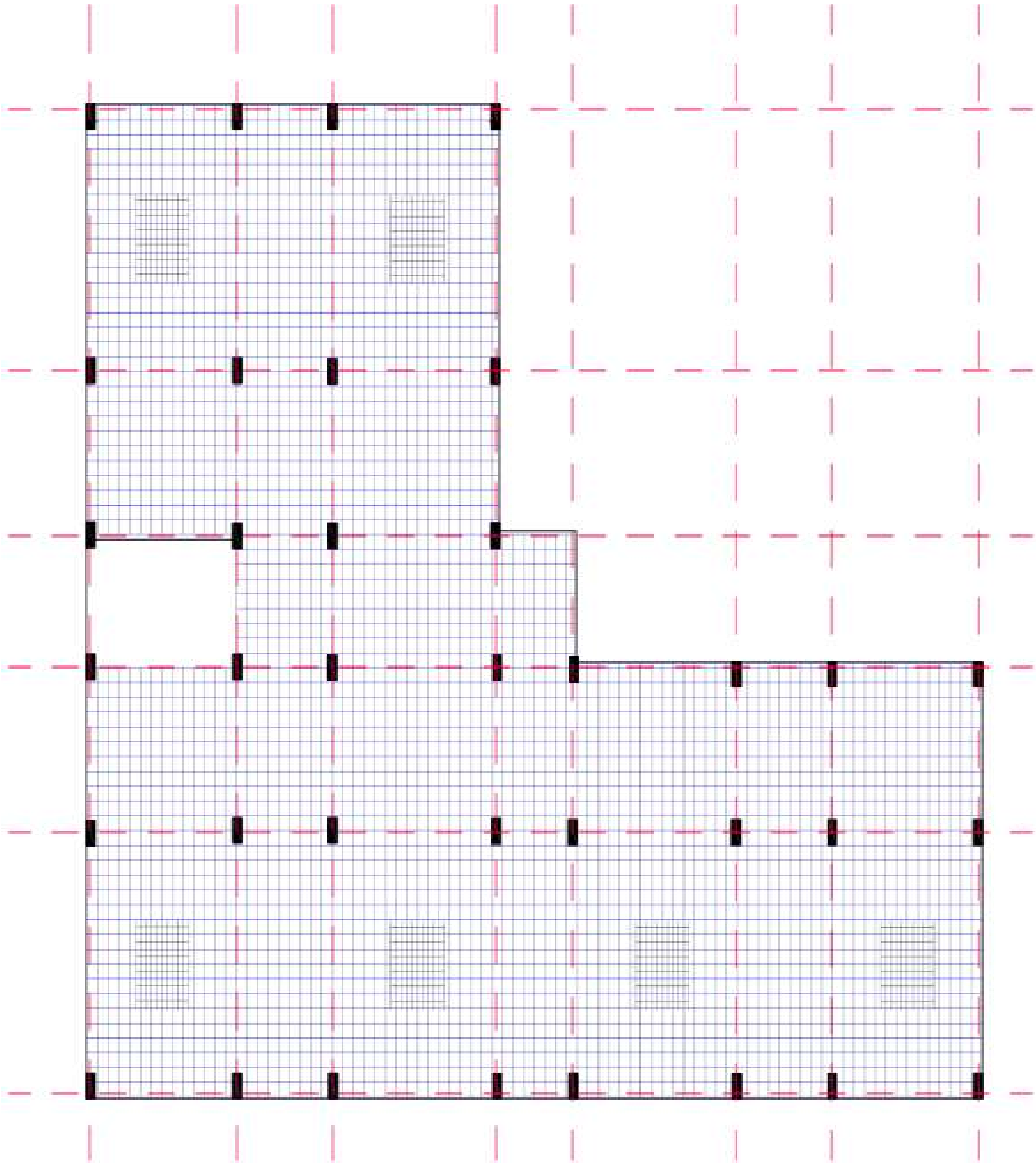
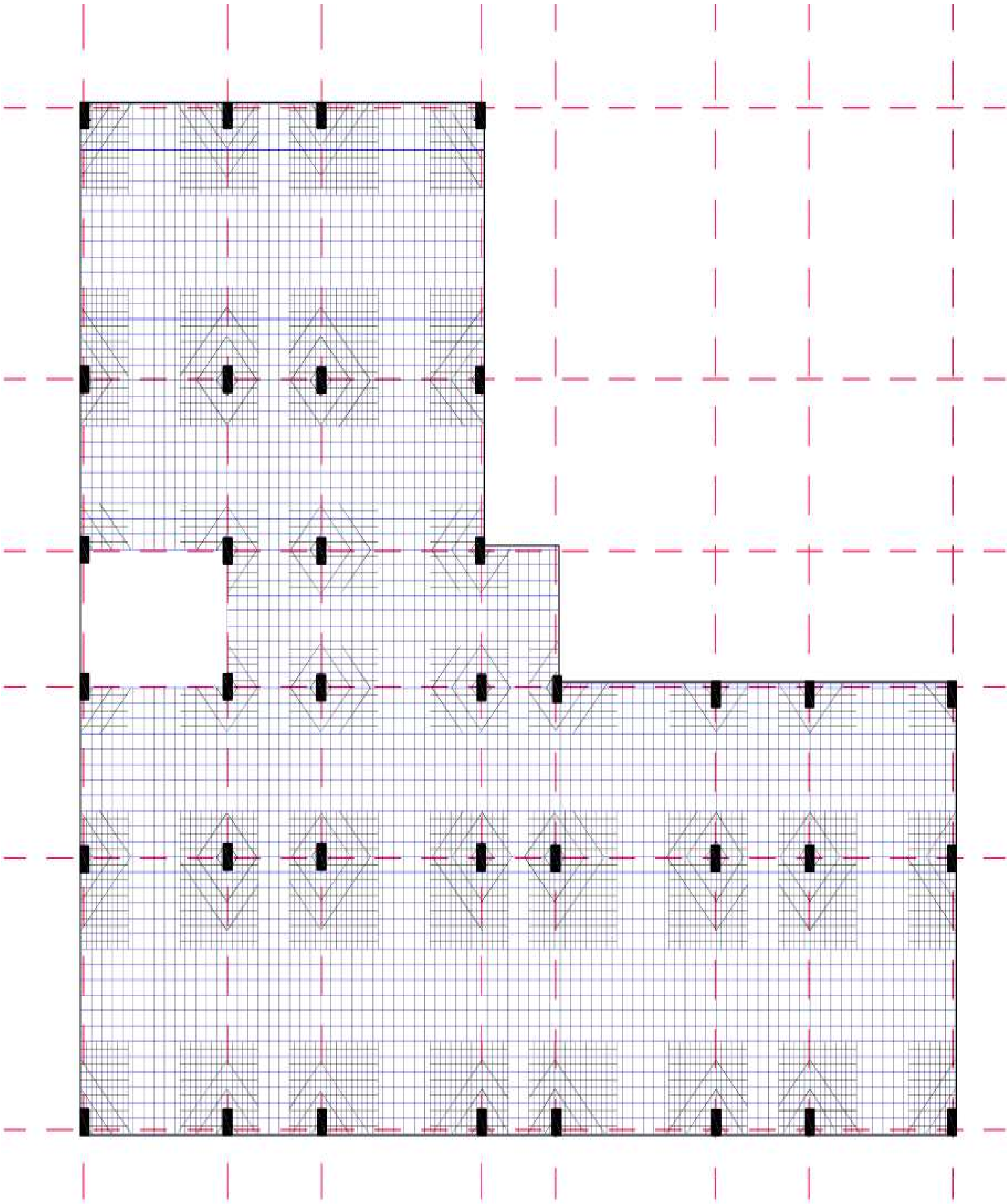
ANNEXES

ANNEX A: Plans

Annex A 1 . Lower nappe reinforcement of the conventional two-way slab



Annex A 2. Upper nappe reinforcement of the conventional two-way slab



ANNEX B: Tables for methodology

Annex B 1. 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

Annex B 2. Values of Minimum cover, $Cmin$,, requirements with regard to durability for reinforcement steel (EC2)

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

Annex B 3. Values of subgrade modulus for different soil type.

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