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INFLUENCE OF OPENINGS ON THE REINFORCED CONCRETE WALLS, ON THE BEHAVIOUR OF TALL STRUCTURES

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

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DEDICATION

To

my parents, Mr and Mrs Besung-Taniform Ignatius.

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LIST OF SYMBOLS AND ABBREVIATIONS

a_g	Design ground acceleration
a_{gR}	Reference peak ground acceleration
a_{vg}	Design vertical ground acceleration
A_c	Cross-sectional area of concrete
A_d	Design value of an accidental action
A_s	Cross-sectional area of reinforcement
$A_{s,\min}$	Minimum reinforcement area
$A_{s\,v,\min}$	Minimum area of the vertical reinforcement
$A_{s\,v,\max}$	Maximum area of the vertical reinforcement
A_{sw}	Cross-sectional area of shear reinforcement
CTBUH	Council of Tall Buildings and Urban Habitat
EC	Eurocode
EN	Europeen norms
FEM	Finite Element Method
f_{cd}	Design concrete compressive strength, N/mm ^{2}
f_{ck}	Concrete cylindrical compressive strength
f_{yd}	Design yield strength of steel reinforcement
f_{yk}	Yield strength of steel reinforcement
F_b	Seismic base shear
$F_{Ed\,Sup,}$	Design support reaction
M_{Ed}	Acting bending moment
M_{Rd}	Resisting bending moment
q_0	Basic value of the behaviour factor, dependent on the type of the structural system and on its regularity in elevation.
q	Behaviour factor
RC	Reinforced concrete
SAP	Structural Analysis Program
SLS	Serviceability limit state
ULS	Ultimate limit state
V_{Ed}	Acting shear load
V_{Rd}	Shear strength

ABSTRACT

The main objective of this work was to study the effect of the size and position of openings on reinforced concrete walls, on the behaviour of tall building structures considering seismic loads. To achieve this goal, an evaluation of the behaviour of a building with various door and window openings; centric, eccentric, staggered and varying orientation, was done. An evaluation was equally done to find the critical storey for positioning large openings. The methodology adopted consisted of a site recognition, definition of materials, loads, geometric and geotechnical data of the building prototype. This was proceeded by conception of the case study which is a regular G+9 storey office building. It was analysed and designed under static and dynamic loads according to European standards. A finite element meshing was done to discretise wall elements. Then, the building was subjected to the Potenza-Italy earthquake response spectrum using SAP 2000 (Structural Analysis Program) version 22. The results were presented and compared in terms of base shear, vibration period, lateral displacement and inter-storey drift for the different building models. It was observed that centric door openings gave lower base shear but greater displacement and inter storey drift than eccentric door openings. In terms of vibration period and lateral displacement, one large regular centric window opening was found most suitable with lower values than many small and staggered window openings. Openings oriented larger in width than in height displayed better behaviour, lower values of results parameters. For the analysis on critical storey, greater displacement was got with opening 3.06 × 2.4(m) placed at first 2 storeys: 1 and 2, and greatest inter-storey drift observed between 3rd and 4th floor slab when opening 3.06 × 2.4(m) is placed at storey 3. These results show that the seismic behaviour of tall building is influenced by these openings and the choice of the position and sizes of openings depends on the parameter the designer wants to control most: seismic force on building, period, displacement or inter storey drift. Therefore, designer should carefully consider this influence in order to ensure a safe and optimum choice.

Keywords: Tall buildings, reinforced concrete walls with openings, inter storey drift, response spectrum analysis, lateral displacement.

RÉSUMÉ

L'objectif principal de ce travail était d'étudier l'effet de la taille et de la position des ouvertures, sur les murs en béton armé et sur le comportement des structures des bâtiments de grande hauteur en considérant les charges sismiques. Pour atteindre cet objectif, une évaluation du comportement d'un bâtiment avec diverses ouvertures de portes et de fenêtres ; centriques, excentriques, décalées et à orientation variable, a été réalisée. Une évaluation a également été faite pour trouver l'étage critique pour le positionnement des grandes ouvertures. La méthodologie adoptée a consisté en une reconnaissance du site, une définition des matériaux, des charges, des données géométriques et géotechniques du prototype de bâtiment. Cette étape a été suivie par la conception de l'étude de cas qui est un immeuble de bureaux ordinaire de G+9 étages. Il a été analysé et conçu sous des charges statiques et dynamiques selon les normes européennes. Un maillage par éléments finis a été effectué pour discrétiser les éléments de paroi. Ensuite, le bâtiment a été soumis au spectre de réponse du séisme de Potenza-Italie en utilisant SAP 2000 (Structural Analysis Program) version 22. Il a été observé que les ouvertures de portes centriques donnent un cisaillement de base plus faibles mais un déplacement et une déviation inter-étage plus grand que les ouvertures de portes excentriques. En termes de période de vibration et de déplacement latéral, une grande ouverture de fenêtre centrique régulière s'est avérée la plus appropriée avec des valeurs inférieures à celles de plusieurs petites ouvertures de fenêtre décalées. Les ouvertures de largeur plus grande que la hauteur ont montré un meilleur comportement. Pour l'analyse de l'étage critique, un déplacement maximal a été obtenu avec de grandes ouvertures placées aux niveaux 1 et 2. La plus grande déviation inter-étage a été observée entre les niveaux 3 et 4 lorsque l'ouverture 3,06 × 2,4(m) est placée à l'étage 3. Ces résultats montrent que le comportement sismique des bâtiments de grande hauteur est influencé par ces ouvertures et que le choix de la position et de la taille des ouvertures dépend du paramètre que le concepteur veut le plus contrôler : force sismique sur le bâtiment, période, déplacement ou déviation inter-étage. Par conséquent, le concepteur doit soigneusement prendre en compte cette influence afin d'assurer un choix sûr et optimal.

Mots clés : Bâtiments de grande hauteur, murs en béton armé avec ouvertures, déviation inter-étage, analyse du spectre de réponse, déplacement latéral.

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GENERAL INTRODUCTION

Rapid urbanisation with a growing population has resulted in scarcity of land thus gearing architectural conception to vertical transformation and tall building structures. With the number of earthquakes recorded yearly, most zones are highly probable to experience an earthquake in the next 50 years. To this effect, reinforced concrete structural walls are widely adopted in many tall reinforced concrete buildings since they provide good resistance to both vertical and lateral loads, contributing to significant lateral stiffness, strength, and overall ductility and energy dissipation capacity of the building. However, for functional and aesthetic requirements, most tall buildings are conceived with openings be it for ventilation, mechanical ducts and facades. The presence of these openings in shear walls influences their behaviour, making them slightly vulnerable under dynamic loading conditions. Their structural properties tend to be altered resulting in greater lateral deflection and inter storey drift. Several research has been carried out on the structural response of shear walls with openings (Muthukumar & Kumar, 2014, Wu & Li, 1996). They recommended the use of many small openings over large openings and observed smaller openings having better performance than walls with large openings. They also found that a door opening of 65% and above converts wall element to frame element behaviour.

Though reasonable studies have been made on the response analysis of shear walls with openings, few literatures exist on the influence of opening locations on reinforced concrete walls on the behaviour of tall structures. To this effect, this work will carry on a parametric study, analysing and evaluating the effect of varying size and position of door and window openings to the behaviour of the structure under seismic loading. An analysis on the effect of opening's orientation and building's behaviour with respect to critical storey for placing openings will equally be done.

To attain this objective, the work is divided in three chapters. The first chapter is about the state of the art permitting to understand the basic concepts related to tall buildings, their behaviours and reinforced concrete walls with openings. The second chapter entitled methodology will present the steps adopted to achieve the objective of this work. Finally, at chapter three the results of the comparison of the different models with varying opening location and size will be presented.

CHAPTER 1. LITERATURE REVIEW

Introduction

In the design of tall buildings, various loads and forces are considered such as gravity loads, self-weight, construction load and the predominant lateral forces like wind action and seismic loads which make its design procedure different from that of average and low-rise buildings. To resist these horizontal forces, several structural systems could be used of which reinforced concrete walls also known as shear walls is one of them. For functionality purpose, these walls often have openings for ventilation, access or facades, which change in size and position depending on its use. In this chapter, seeking to understand what are tall buildings, their structural behaviour under these external loads, the role played by the structural system-reinforced concrete walls and how the change in size and position of their openings influence the structural behaviour of tall buildings as a whole. Springing from past research, the principles and concepts surrounding this topic are here presented.

1.1. Tall buildings

There exist a lot of literature surrounding tall building. Also, many school of thought hold unto various definition of this term. This section aims to bring to understanding what tall buildings are, factors influencing their functioning and design, and the predominant forces acting on them.

1.1.1. Definition of tall buildings

With vertical architecture adopted in recent years, the term tall building is one commonly heard of and designs are taking that trend. The big question is often, how are buildings classified as tall buildings? Is a 15-metre story building tall enough to be considered a tall building? Or is it judged from the slenderness? Or design loads as structural engineers will see it or is it just the height? A lot of controversies and thus no universal definition to tall buildings. Some distinguish tall buildings from high-rise and skyscraper buildings, although determining which is which and defining a building as tall is dependent on the context, surrounding environment, and school of thought. However, here are some of the popular schools of thought of the distinguishing characteristics used in defining tall buildings:

- Any structure where the height can have a serious impact on evacuation (Geoff, 2009).

- According to the Council of Tall Buildings and Urban Habitat (CTBUH), a building is high rise when it is considerably higher than surrounding buildings, or its proportion is slender enough to give it an appearance of a tall building, or embraces certain technologies like damping system, or wind bracing. Figure 1.1 shows their classification according to height (CTBUH, 2020).
  - Alternatively, structural engineers regard a tall building as one that due to its height and aspect ratio, the effect of lateral forces such as wind and seismic actions cannot be simply ignored and need to consider their effect coupled with vertical loads on tall building's response behaviour in the very beginning of the design process. From the structural engineering perspective, a tall building is one whose design and stability is affected by lateral forces(Choong & Tan, 2015).

Then said, it is important to notice that tall buildings have much other specificity than their height only.

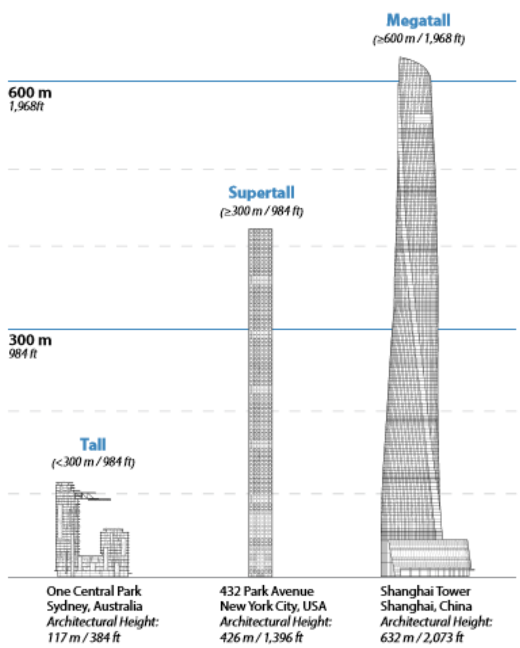


Figure 1.1. Classification of tall buildings according to heights, CTBUH Height Criteria for Measuring & Defining Tall Buildings.

With the world's population and urbanisation increasing at rapid speed, especially in the world's largest cities, architectural conception is geared towards tall structures as they can accommodate many more people on a smaller piece of land than low-rise buildings can on the same piece of land, resulting in a vertical transformation of horizontal expansion. (Beedle et al., 2007). Figure 1.2. shows some of the tall buildings in Cameroon.

a) b)

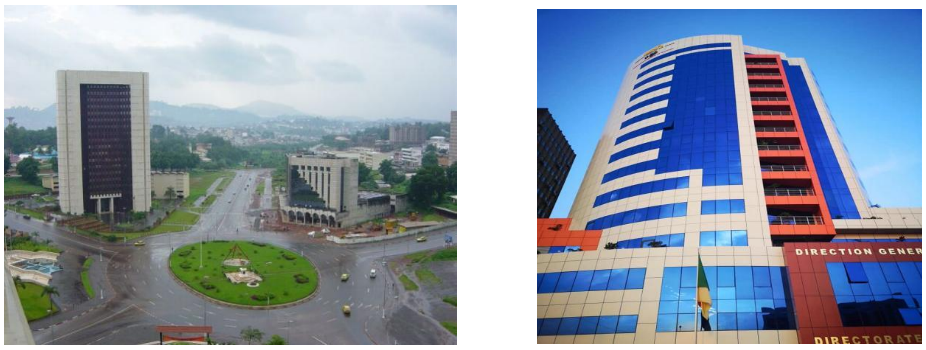


Figure 1.2. Tall buildings in Cameroon, Yaoundé a) Prime Minister's building 89m b) Directorate General of taxation 72m.

This evolution to attain even greater heights extending to kilometres causes engineers to continue working on ensuring the safe usage of tall buildings, designing for the lateral load parameter taking into account the rigidity and damping capacity of these buildings.

1.1.2. Functioning and design of tall buildings

Design criteria can be grouped mainly into force design, displacement-based design and performance-based designs which entails the building's serviceability, strength, and stability. The ultimate limit state, corresponding to loads causing failure, as well as the serviceability limit states involving the criteria governing the building's service life, must be considered. The aim of the structural engineer is to arrive at suitable structural schemes to satisfy these criteria as explained thus.

1.1.2.1. Strength

The most important design requirement for ultimate limit state is that the building structure is strong enough to resist and remain stable under extreme force conditions that may occur during the building's design life. This necessitates a study of the forces and stresses that will be experienced by the members and providing sections and reinforcements to resist these.

1.1.2.2. Stability

Stability is defined as the ability to resist undesired movements or to return to an equilibrium position after a force has been applied(Zeeshan et al., 2018). The stabilisation of a high-rise building is ensured by the different subsystems:

- Floor systems: The primary function of the floor system is to withstand gravitational loads. They may also support in stability, lateral load resistance, by distributing lateral loads to vertical resisting elements and connecting different systems together (Jayachandran, 2009).
- Vertical load resisting systems: Columns, bearing walls, beams, hangers, and cables are used to resist the vertical load in a building. In tall buildings, the vertical load is usually not the most important consideration as it aids in dampening and preventing overturning(Sandelin & Budajev, 2013).
- Damping systems: A dampening system, or damper, may be required to prevent motion in high-rise buildings. Reinforced concrete high-rise buildings often have enough mass to avoid the need for a dampening system(D.K. et al., 2014).
- Lateral load resisting systems: These are structural elements that resist seismic, wind, and eccentric gravity loads. There are many different systems, but they can all be broken down into three basic ones namely: shear walls, moment resisting frames, and braced frames.

1.1.2.3. Serviceability

The cumulative vertical movements caused by creep and shrinkage in tall buildings can cause distress in non-structural elements and significant structural actions in horizontal elements. Any cumulative differential movement will have an impact on the lateral drift, acceleration, and torsional velocity criteria which are usually used to govern tall buildings(Kokai, 2002). Table 1.1 gives the limiting values for lateral displacement per Longarini.

Table 1.1. Resistant displacements (δ_{Sd} is top displacements, δ_{Sid} is inter-story displacements, H is building height, and hi is inter-story height) (Longarini et al., 2017).

δ_{Rd} values	δ_{Rid} values
$H/600 \leq \delta_{\mathrm{Sd}} \leq H/500$	$\delta_{\mathrm{Sid}} \leq hi/600$

1.1.3. Predominant forces acting on tall buildings

The loading on tall buildings is quite different from that on low - rise buildings. The collection of gravity loading over a large number of stories, more prone to lateral drift with low damping, and wind-induced building sway, one of the most important problems encountered (MEHMET & HUSEYIN, 2014). In earthquake regions, factors that need to be considered are deflections and accelerations from horizontal loading. For seismic loads, ground movement causes the structure to move at the base, creating a lateral force in the form of shear. The inertial forces try to prevent the building from moving with a force given by equation 1.1.

$$F = M.a \qquad 1.1$$

Where:

F is an inertial force

M is the mass (equal to building weight divided by acceleration due to gravity, g)

a is the acceleration.

The shear force is distributed from top to bottom of the structure with its maximum at the structure's highest point (Sandelin & Budajev, 2013) (see figure 1.3). The horizontal components of ground motion require greater consideration for seismic design since adequate resistance to vertical seismic loads is usually provided by vertical load resisting elements (Bungale, 2010). These seismic forces have a period, proportional to the height of the building and characteristics of the wave. When the period of the wave turns to be equal to the natural period of the building, resonance occurs with great amplitude, acceleration increases, which is fatal to an already weakened structure.

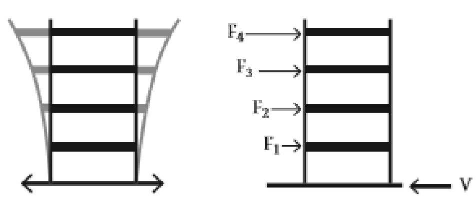


Figure 1.3. Lateral loads due to earthquake (Bungale, 2017)

1.2. Tall building design under seismic conditions

When earthquake occurs, tall buildings are more vulnerable to structural collapse in various ways. To this effect, their design and construction process should take into consideration their behaviour under these seismic loads, the load path of the forces and the structural systems put in place to resist this seismic force. This section aims to bring into understanding these aspects.

1.2.1. Seismic behaviour of tall buildings

In response to seismic forces, tall buildings have a static and dynamic response. Its behaviour in terms of the lateral resistance depends upon the stiffness contributions of its members and differs from one system to others based on parameters like the structural system, the plan arrangement of the bracing members (symmetrical or unsymmetrical), the relation to the position of the gravity centre of the building and the height of the building. (AL BALHAWI et al., 2016). Figure 1.4. shows a simplified response of a building during an earthquake.

Earthquakes can cause structural collapse by causing failure at joints by pulling apart, overstressing gravity load-bearing elements or by sufficient lateral displacement. This behaviour is controlled by configuration, stiffness, strength and ductility. Configuration has to deal with overall geometry, structural system and load patterns(Murty et al., 2012). The tall building's performance is assessed with lateral deformation, natural period and mode of oscillation.

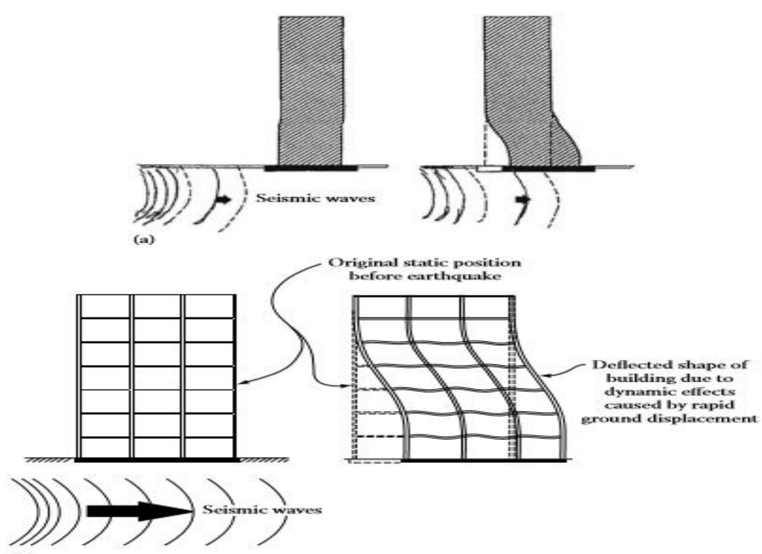


Figure 1.4. Earthquake response of tall buildings (Bungale, 2010)

1.2.2. Load path

Buildings typically consist of vertical and horizontal structural elements. The vertical elements that transfer lateral and gravity loads are the shear walls and columns. The horizontal elements such as floor and roof slabs distribute lateral forces to the vertical elements acting as horizontal diaphragms. A complete load path is a basic requirement. The general load path is as follows: seismic forces originating throughout the building are delivered through connections to horizontal diaphragms; the diaphragms distribute these forces to lateral force–resisting elements such as shear walls and frames; the vertical elements transfer the forces into the foundation; the foundation transfers the forces into the supporting soil. If there is a discontinuity in the load path (figure 1.5), the building is unable to resist seismic forces regardless of the strength of the elements. Examples of gaps in the load path would include a shear wall that does not extend to the foundation, a missing shear transfer connection between a diaphragm and vertical elements, a discontinuous chord at a diaphragm's notch, or a missing collector(Bungale, 2010).

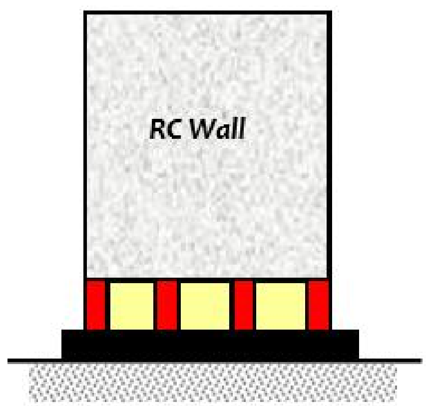


Figure 1.5. Structural wall discontinued in ground storey, the worst choice, interruption in load path(Murty et al., 2012).

1.2.3. Lateral load resisting structures

Lateral load resisting systems are structural elements which resist horizontal loads like seismic, wind and eccentric gravity loads. For lateral load resisting systems, the most commonly used structural systems have been classified and are broadly defined as shown in figure 1.6.

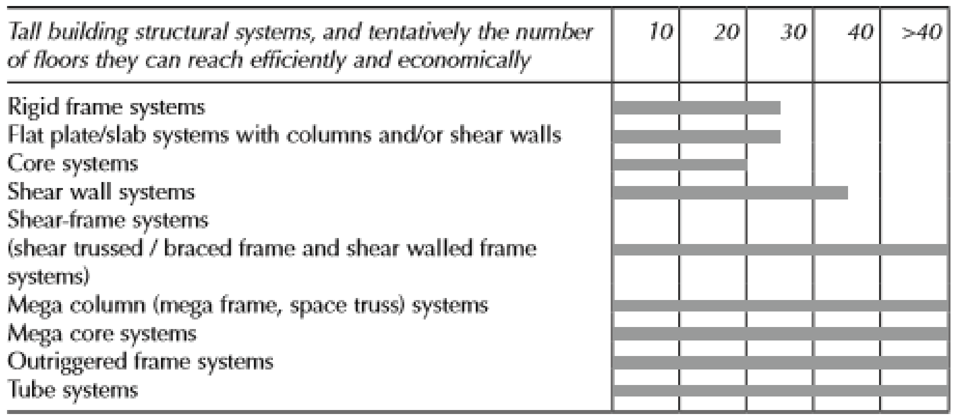


Figure 1.6. Tall building structural systems and the number of floors they can reach(MEHMET & HUSEYIN, 2014).

The selection of a structural system for buildings is influenced primarily by the intended function, architectural considerations, internal traffic flow, height, and aspect ratio, and to a lesser extent, the intensity of loading. (Bungale, 2010). The drift of the tower should be kept within limits, such as H/500 (Jayachandran, 2009). These systems can be broken down to three fundamental ones which all other systems are a combination of as shown in figure 1.7 (Sandelin & Budajev, 2013).

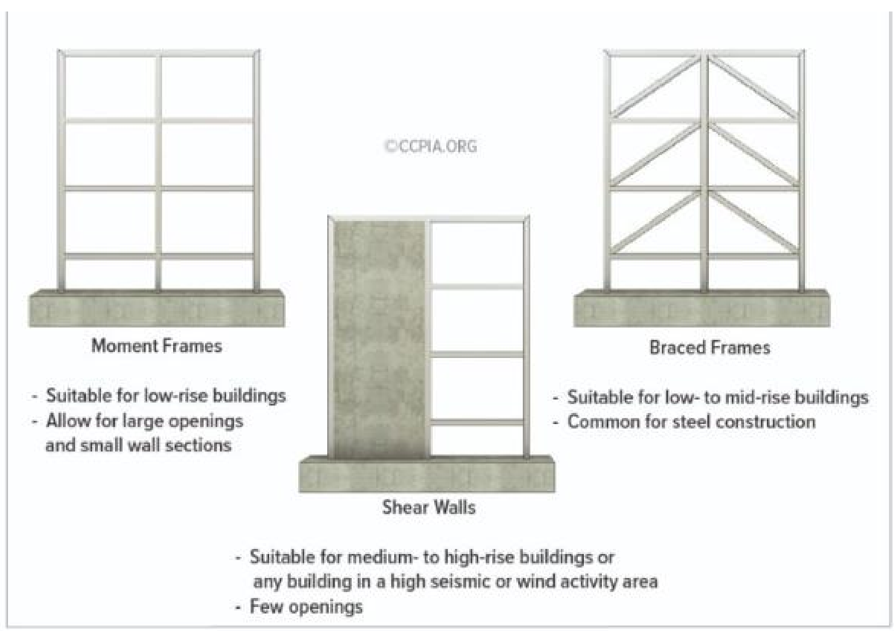


Figure 1.7. Types of lateral load resisting systems (Certified Commercial Property Inspectors, 2022)

1.3. Reinforced concrete walls

Reinforced concrete walls are the most commonly used lateral load resisting systems as they provide sufficient stiffness. In this section, their definition, main role, structural properties and design will be discussed. In addition, the effect of openings on RC walls and past research on this is presented.

1.3.1. Definition and role of reinforced concrete wall

Reinforced concrete (RC) walls are vertical plate-like structural walls (figure 1.8), often referred to as shear walls, which are being employed in mid to high-rise buildings because of their effectiveness in resisting vertical and lateral loads, principally earthquake and wind forces.

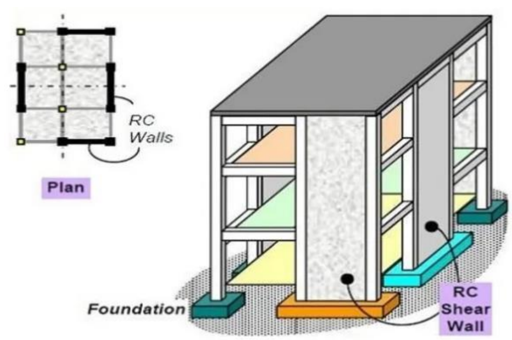


Figure 1.8. Shear walls (Gopal, 2009)

For structural requirements, they reduce overall displacement of building especially when provided all through the height of the buildings. Their position governs the earthquake performance and response like mode of oscillation. The extent to which a wall will contribute to resistance depends on its geometric configuration, orientation, and location within the building. The placing of the rotational centre at the centre of the building is advised because it exceedingly reduces the building's susceptibility to twisting due to evenly distributed horizontal loads (Bungale, 2010) as shown in figure 1.9. Placing RC walls around elevators, stairs and utility shafts is common as they do not interfere with architectural layout which may require access openings, ventilations or facade designs which create a discontinuity in load paths and alter structural response of the building, the basis on which this thesis is founded.

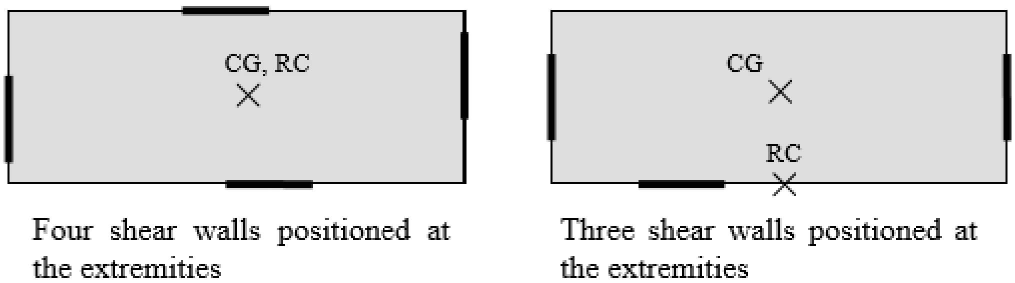


Figure 1.9. Examples of locations of stabilising units(Gustafsson & Hehir, 2005)

1.3.2. Structural properties and design of reinforced concrete walls

Reinforced concrete wall is designed as a compression member. Concrete walls are classified as reinforced concrete wall when reinforcement> 0.4%. Shear walls are generally classified on the basis of aspect ratio (height/width ratio). In general, the structural response of shear walls depends strongly on the type of loading, aspect ratio of shear wall, and size and location of the openings in the shear walls. (Muthukumar & Kumar, 2014). Shear walls could be solid or perforated with openings (couple/pierced shear walls, see figure 1.10) (Gopal, 2009).

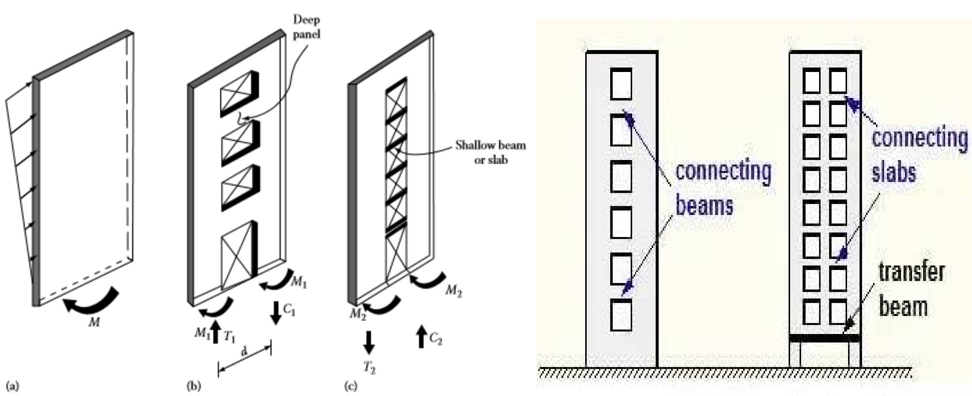


Figure 1.10. The load-resisting mechanisms in solid and coupled shear walls. (Gopal, 2009).

In most buildings, there can be a combination of the shear wall and rigid frame systems which interact horizontally to increase stiffness and stability at the top as shown in figure 1.11 (Badami & Suresh, 2014).

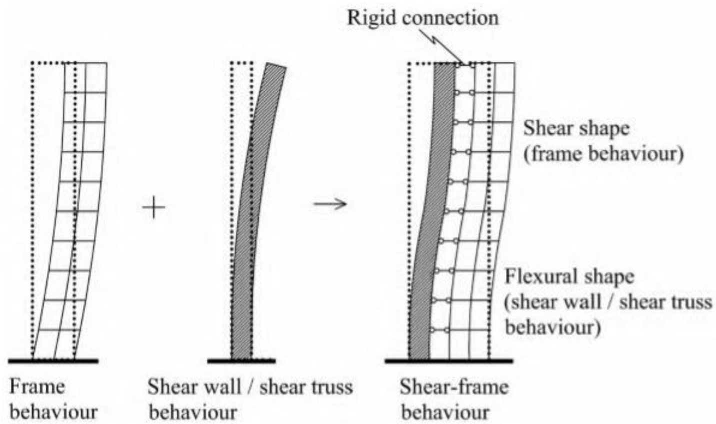


Figure 1.11. The behaviour of the shear-frame system under lateral loads(MEHMET & HUSEYIN, 2014)

Reinforced concrete (RC) walls are composed of a mixture of concrete and steel reinforcement, thus inherit the properties of reinforced concrete like compressive strength, durability, fire resistance just to name a few. Concrete has good mechanical properties in terms of resistance, rigidity (particularly compression) and durability. Rigid frames resist lateral loads by dissipating energy through their ductility and may undergo excessive lateral deformations. Shear trusses and walls, despite being less ductile than the frame; dissipate energy while staying within elastic limits because of the larger size of the shear area subjected to the shear force caused by lateral loads, and exhibit smaller lateral deformations(MEHMET & HUSEYIN, 2014).

1.3.3. Openings on reinforced concrete walls

The most efficient shear walls are the solid shear walls, and they are highly desirable. However, to meet the current functional, architectural, and mechanical requirements of buildings (typically include provision of doors, windows, or the services like the air-conditioning and ventilation ducts) openings are often required in the shear walls. In engineering terms, these reinforced concrete walls with openings are generally termed as pierced walls or coupled walls (Jaseela & Pillai, 2017). These openings vary in shape (circular, rectangular, triangular), size (standard door and window sizes, commercial space entrances or even facade aesthetics) and position too. Openings on RC walls are a source of weakness and depending on their size and position, they influence the stiffness and load-bearing capacity of the pierced reinforced concrete walls compared to a solid wall. These openings in RC walls lead to interruption in structures load path especially when found at base level.

1.3.4. Recent studies on reinforced concrete walls with openings

Many researchers have conducted experimental and finite element studies for assessing the effects of openings in reinforced concrete shear walls. However, there is a lack of comprehensive comparisons between different studies. Some most recent experimental and finite element studies available in the literature and a review of the major findings are presented here (Borbory, 2020).

- Ashok Kankunt la et al. (2016) through a finite element simulation, studied the behaviour of shear walls, including an opening under seismic load on member forces by varying the position and size and concluded that changing the position of the shear wall of reinforced concrete structures with various opening sizes in buildings openings are economical(Ashok et al., 2016).
- Anas M. Fares in his study of impact of shear wall openings on lateral stiffness found that, the lateral stiffness is affected by the size of the openings, the window's opening has a small effect on the lateral stiffness and may be neglected if the window's opening area ratio to the total wall side area is up to 3%. Also, when the wall height to length ratio increases, the effect of opening decreases and the minimum door opening ratio that converts the solid wall to a frame will be equal to 65% from the total wall area(Fares, 2021a).
- G. Muthukumar and Manoj Kumar in their work, Influence of Opening Locations on the Structural Response of Shear Wall recommended the use of a large number of small openings than few large openings and walls with staggered windows having a better displacement performance than those with regular windows(Muthukumar & Kumar, 2014).
- Hui Wu & Bing Li in their Parametric study of reinforced concrete walls with irregular openings concluded that, walls with small sized openings can perform better than walls with larger openings, comparing their lateral displacements. Also, due to the influence of axial loading, the opening should not be arranged too close to the boundary to provide large column zones; strengthening walls along the load paths to improve their performances(Wu & Li, 1996).
- H.-S. Kim, D.-G Lee in their study of analysis of shear walls with openings using super elements had as one of their observations, that the effect of window openings on the behaviour of the structure gave a noticeable displacement when a/b=0.5, that is its effect becomes more

significant. With a=width and height of opening, b= width and height of shear wall. He also observed that, for a fixed width, an increase in height results in increasing displacement and for a fixed height, an increase in width equally results to an increased displacement.

Conclusion

This first chapter aimed at finding the basic concepts on tall building, their behaviour to lateral loads, role played by RC walls and the effect openings have on their structural performance. It was found that tall buildings are affected by lateral loads like seismic loads which in response are reflected in their vibration period and lateral displacements. Also, lateral load resisting systems like RC walls increase the buildings rigidity to these deflections. From past studies, the wall's behaviour was compared using lateral displacement and found that various opening sizes and locations do affect building's response. In the next chapter, the methodology adopted in carrying out this analysis and verification is presented.

CHAPTER 2. RESEARCH METHODOLOGY

Introduction

With the aim of analysing and verifying the varying structural behaviour of a reinforced concrete wall with openings, using FEM numerical calculation: a linear analysis is carried out on the structure using the SAP2000 software. A parametric study on the influence of changes of openings' sizes and position on the influence of tall building's behaviour under seismic loading. In order to carry these, a systematic set of procedures is used. This chapter explains how these are carried out in order to obtain results of the analysis.

2.1. General site recognition

The site recognition is done through documentary research aimed at gathering information about the site. The information obtained include the general physical parameters of the site like its geographical location, its climate, relief, geology and its socio-economic parameters; its demography and some economic activities.

2.2. Site visit

The purpose of this activity is the site description resulting from its observation and that of its surrounding in order to collect necessary data for conception and design.

2.3. Definition of the case study parameters

The structure to be used for modelling is conceived, first by defining the purpose that is use category of the building. Following this is defining the details of the building, that is, building's dimensions, length and width, number and dimensions of its different rooms which then results to the distribution plans and structural plans from structural analysis. Other structural data like the material properties, permanent and variable loads used are defined equally.

2.4. Codes definition

The following codes are used in the design and verification of structural elements:

- Eurocode 0: Basis of structural design
- Eurocode 1: Actions on structures Part 1-1: General actions. Densities, self-weight, imposed loads for buildings. (BS EN 1991-1-1-)

- Eurocode 2: Design of concrete structures Part 1-1: general rules and rules for buildings. (BS EN 1992-1).
- Eurocode 8: Design of structures for earthquake resistance Part 1: General rules, seismic actions and rules for buildings (BS EN 1998-1).

2.5. Loads definition

Different types of loads can be applied on a structure. This analysis is focused on a building structure and EN 1990 classifies the different kinds of actions by the variation in time: the permanent loads and the variable loads.

2.5.1. Permanent loads

This kind of load is constituted by the self-weight of structural and non-structural elements. The weight of the structural elements is obtained by multiplying the specific weight of concrete by the section of the elements. The self-weight of the non-structural elements are extracted form Eurocode 1.

2.5.2. Variable loads

In this study, variable loads are of two types namely the imposed loads and the seismic loads, effect of wind is neglected.

2.5.2.1. Imposed loads

Imposed loads are those arising from occupancy. It includes the normal use by people, the furniture and moveable objects and others. According to the Eurocode 1, different use categories of areas exist.

2.5.2.2. Seismic action

This action is due to earthquake ground motions. It is accounted in the analysis by the definition of the elastic response spectrum as described detailly in EN 1998.

2.6. Combination of loads

In the EN 1990 code, the following fundamental combination of actions are given for the ultimate limit states (ULS) or the serviceability limit states (SLS).

2.6.1. ULS design

The combination of actions for ULS design is given by the relation:

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

Where:

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

$Q_{k,1}$ is the characteristic value of the leading variable action 1

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

Ψ is the combination factor that is function of the use category of the building

The coefficients $\gamma_{G,j}$ and $\gamma_{Q,i}$ are partial factors which minimise and maximise the action which tends to reduce and increase the solicitations respectively. EN 1990, recommends:

$\gamma_{G,j}$ (sup) = 1.35 and $\gamma_{G,j}$ (inf) = 1

$\gamma_{Q,I}$ = 1.5 for unfavourable condition and 0 for favourable

$\gamma_{Q,I}$ =1.5 for unfavourable condition and 0 for favourable

2.6.2. SLS verification

The combinations of actions for serviceability limit states (SLS) are defined symbolically by the expressions 2.2 to 2.5.

- Characteristic combination:

$$G_1 + G_2 + P + Q_{k1} + \psi_{02} \cdot Q_{k2} + \psi_{03} \cdot Q_{k3} + \ldots \quad (2.2)$$

- Frequent combination:

$$G_1 + G_2 + P + \psi_{11} \cdot Q_{k1} + \psi_{22} \cdot Q_{k2} + \psi_{23} \cdot Q_{k3} + \ldots \quad (2.3)$$

- Quasi-permanent combination:

$$G_1 + G_2 + P + \psi_{21} \cdot Q_{k1} + \psi_{22} \cdot Q_{k2} + \psi_{23} \cdot Q_{k3} + .. \quad (2.4)$$

- Combination of actions for accidental design situations:

$$G_1 + G_2 + P + A_d + \psi_{21} \cdot Q_{k1} + \psi_{22} \cdot Q_{k2} + \ldots \quad (2.5)$$

Where:

G_1 is the characteristic value of self-weight

G_2 is the characteristic value of a permanent action

Q_{k1} is the characteristic value of the leading variable action 1

Q_{ki} is the characteristic value of the accompanying variable action i

A_d is the design value of an accidental action

P_k is the characteristic value of a prestressing action γ_{G1} partial factor for permanent structural actions: 1.0 – 1.3

γ_{G2} is the partial factor for permanent no structural actions: 0.0 – 1.5 γ_{Qj} partial factor for variable action i: 0.0 – 1.5

ψ_0 is the factor for characteristic value of variable action i

ψ_1 is the factor for frequent value of variable action ψ_2 factors for quasi-permanent value of variable action

2.6.3. Seismic combination

The combination of seismic actions with other loads is:

$$\sum_{j\geq 1} G_{k,j} + \mathrm{E} + \sum_{i>1} \Psi_{0,i} Q_{k,i} \quad 2.6$$

A seismic combination of the seismic actions in the 2 main directions x and y is carried out to take into account the propagation of the seismic action in these directions. Ex, Ey the seismic forces in x and y directions respectively neglecting z direction, are combined as:

$$\pm E_x \pm 0.3E_y \quad 2.7$$

$$\pm 0.3E_x \pm E_y \quad 2.8$$

2.7. Structural design procedure

For the design procedure, using the material properties, environmental conditions, loads stated above, the detailed procedure for design of elements is elaborated in this section according to the codes listed in Section 2.4.

2.7.1. Material characteristics

The class and type of concrete and steel reinforcement used are defined in order to know their structural properties.

2.7.1.1. Concrete properties

The class of concrete used is defined and other properties like its characteristic compressive strength (f_{ck}) determined.

2.7.1.2. Steel reinforcement properties

The class of steel used is defined and other properties like its characteristic yield strength (f_{yk}) determined.

2.7.2. Durability and concrete cover

A durable structure must meet the requirements of serviceability, strength, and stability for the duration of its design working life, without significant loss of utility or excessive unforeseen maintenance. Adequate concrete cover is needed to ensure adequate bond and corrosion protection of steel reinforcement and is given by:

$$c_{nom} = c_{min} + \Delta c_{dev} \tag{2.9}$$

The first step in determining durability is to select exposure conditions from the Table 2.1 from Eurocode 2.

Table 2.1. Values of minimum cover, $c_{min,\,dur}$ requirements with regard to durability for reinforcement steel in accordance with EN 10,080 (Table 4.4N_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

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

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

The greater value for c_{min} satisfying the requirements for both bond and environmental conditions shall be used and is given by:

$$c_{min}=\max\{c_{min,\,b};\ c_{min,\,dur}+\Delta c_{dur,\gamma}-\Delta c_{dur,\,st}-\Delta c_{dur,\,add};10\text{ mm}\} \qquad 2.10$$

Where:

$c_{min,\,dur}$ is the minimum cover due to environmental conditions. (Table 2.1)

$c_{min,\,b}$ is the minimum cover due to bond requirement (Table 2.2)

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

$\Delta c_{dur,\,st}$ is the reduction of minimum cover for use of stainless steel, recommended value is 0

$\Delta c_{dur,\,add}$ is the reduction of minimum cover for use of additional protection, without specifications, recommended value is 0

Δc_{dev} is the allowance in design for deviation with recommended value of 10 mm

2.7.3. Beam element design

Beams receive loads from the slabs, walls and columns and transfer to supporting columns. They could be simply supported, continuous or cantilevered and have rectangular, square, T or L-shape sections. The beam designed is that with largest influence area and designed according to these steps.

2.7.3.1. Preliminary design of beams

The beam dimensions b and h are determined in the preliminary design and is given by:

Simply supported beams, $h \geq \frac{L}{14}$ $and\ b \cong 0.5h$ 2.11

Cantilevered beam, $h \geq \frac{L}{18}$ $and\ b \cong 0.5h$ 2.12

Embedded beam, $h \geq \frac{L}{20}$ $and\ b \cong 0.5h$ 2.13

The various load combinations are applied on the beam and their solicitations, bending moment and shear are obtained and plotted on excel. The maximum values are then used to design longitudinal and transverse reinforcement.

2.7.3.2. Bending moment design

According to Eurocode 2, the bending moment at support obtained from load combinations should be reduced by ΔM_{Ed}, which is defined as:

$\Delta M_{Ed} = F_{Ed}\ Sup.\ t/\ 8$ 2.14

Where:

t is the breadth of the support

$F_{Ed\ Sup,}$ is the design support reaction

In the preliminary design according to ULS criteria, the area of longitudinal tension reinforcement should not be taken as less than $A_{s,\ min}$ and should not be taken as more than $A_{s,\ max}$, which is given by:

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

$A_{s,max} = 0.004 \cdot Ac$ 2.16

Where:

bt 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

The steel reinforcement section defined, the effective area of the steel reinforcement is obtained by computing the necessary area and the corresponding number of bars. 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.as presented in the figure 2.1.

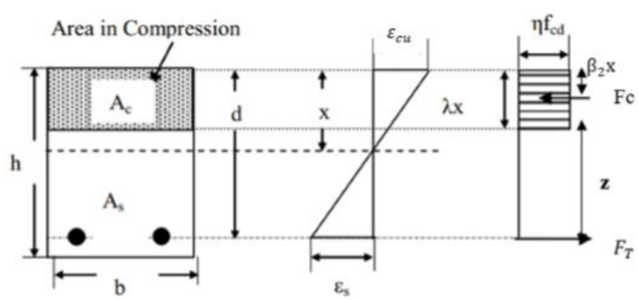


Figure 2.1 Stress-Strain distribution in beam

This neutral axis is obtained from the equation 2.17.

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

2.17

Where:

d is the is the effective depth of the section

b is the width of the section fcd: is the design compressive strength of the concrete

$\beta 1$ is a correction factor equal to 0.81

$\beta 2$ is a correction factor equal 0.41

This resisting moment is then given by the relation 2.18.

$$M_{RD} = A_{s.real}.f_{yd}.(d - \beta_2 x) \qquad 2.18$$

Where:

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

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

2.7.3.3. Shear verification

From the envelope curve of the shear solicitation, the necessity of the shear reinforcement is verified by comparing the acting shear V_{Ed} to the design shear resistance of the member without shear reinforcement $V_{Rd,C}$ which is defined by:

$$V_{Rd,C} = max\left\{\left[C_{Rd,c}\, k(100\rho_1\, f_{ck})^{\frac{1}{3}} + k_1\sigma_{cp}\right)\right] b_{wd}; \left(V_{\min} + k_1\sigma_{cp}\right) b_w\, d\} \qquad 2.19$$

Where:

f_{ck} is the characteristic strength of the reinforcement

$C_{Rd,c} = 0.18/\gamma_c$

γ_c is the partial safety factor for the concrete (assumed = 1.5)

d is the effective depth of the section

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

σ_{cp} is the average stress in the concrete due to the axial compressive force N_{Ed}.

$$\sigma_{cp} = N_{Ed}/A_c\ <0.2 f_{cd}\ [N/(mm^{\wedge}2\ )] \qquad 2.20$$

N_{Ed} is the axial force in the cross section due to loading or prestressing (in N)

A_c is the area of the concrete cross section

$$k = 1 + \sqrt{(200/d)} \leq 2.0 \quad \text{With d in millimetres} \qquad 2.21$$

$\rho_1 = A_{st}/(b_w\ d) \leq 0.02$ 2.22

If no design shear reinforcement is required, the minimum shear reinforcement is applied according to the detailing of that member. For members where the design shear reinforcement is required, the shear resistance is the minimum between $Vrds$ and $Vrdmax$ defined by the equations 2.23 and 2.24 respectively.

$V_{rd,ax} = \alpha_{cw} b_w z v_1 f cd/(cot\theta + tan\theta)$ 2.23

$V_{Rd,} = A_{sw} \cdot z f_{ywd} cot\theta$ /S 2.24

Where:

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

v_1 is a reduction factor for concrete cracked in shear ($v1$ = 0.6 for $fck \leq 60\ N/mm2$)

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

A_{sw} is the cross-sectional area of the shear reinforcement with a maximum value given by the relation 2.25 as:

$A_{sw,ax} f_{ywd} /(b_w S) \leq 1\ 2\ \alpha_{cw} b_w v_1 f cd$ 2.25

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

These limitations are given respectively in the equations 2.26 to 2.28.

$Sl_{,ax} = 0.75d(1 + cot\alpha)$ 2.26

$S_{l,ax} = 0.75d \leq 600\ mm$ 2.27

$\rho_{w,\ min} = (0.08\sqrt{f_{ck}})/f_{yk}$ 2.28

With the shear reinforcement ratio computed as shown in equation 2.29 as:

$\rho_w = A_{sw}/(s.\ b_w.\ sin\alpha)$ 2.29

2.7.3.4. Stress verification.

The stress in the concrete and the reinforcement are obtained using the relations 2.30 to 2.32, with the envelope curve of bending moment at serviceability limit state.

$$\sigma_c = (M_{Ed}.x)/I \qquad 2.30$$

$$\sigma_s = (n.M_{Ed}.(d-x))/I \qquad 2.31$$

$$\sigma_s' = (n.M_{Ed}.(x-d'))/I \qquad 2.32$$

Where:

x is the neutral axis position

I is the inertia moment of the section

M_{Ed} is the maximum moment at SLS

In order to limit longitudinal cracks within the concrete and inelastic strain in the reinforcement, Eurocode 2 requires a stress limitation as shown in the expressions 2.33 and 2.34.

$$\sigma_c \leq k_1.f_{ck} \qquad 2.33$$

$$\sigma_s \leq k_3.f_{yk} \qquad 2.34$$

With k_1=0.6 and k_3=0.8.

2.7.3.5. Crack control

EN 1992 defines for the control of cracking that, firstly the limited calculated crack width, w_{max}, which depends on exposure class of the building. Proceeding to looking up the maximum diameter of longitudinal bars and spacing the longitudinal bars have to have, to avoid cracking.

2.7.3.6. Deflection control

It is verified here that neither the efficiency nor appearance of the building should be affected by deflections. The limiting span/depth ratio may be estimated using the expressions 2.35 and 2.36.

$$l/d \leq K[11 + \frac{1.5(f_{ck})^{0.5}\rho_0}{\rho} + 3.2(f_{ck})^{0.5}\left(\frac{\rho_0}{\rho} - 1\right)^{\frac{3}{2}}] \quad \text{if } \rho<\rho0 \qquad 2.35$$

$$l/d \leq K[11 + \frac{1.5(f_{ck})^{0.5}\rho_0}{\rho-\rho'} + \frac{1}{12}(f_{ck})^{0.5}\left(\frac{\rho'}{\rho_0} - \right)^{\frac{1}{2}}] \quad \text{if } \rho<\rho0 \qquad 2.36$$

Where:

l/d is the limit span/depth

K is the factor to take into account the different structural systems.

ρ_0 is the reference reinforcement ratio= $(f_{ck})^{0.5}10^{-3}$

ρ is the required tension reinforcement ratio at mid span to resist the moment due to design loads

ρ' is the required compression reinforcement ratio at mid span to resist the moment due to design loads.

2.7.4. Column design

The column chosen to be designed is that with greatest solicitations axial, force and moment, and also that with greatest influence area.

2.7.4.1. Preliminary design of columns

The column, in its preliminary design, its section is got by, $b_{column}=b_{beam}$

$$b \leq h \leq 4b \qquad 2.37$$

Where:

min {b, h} = 20 cm

Also, in seismic zones, the preliminary design of columns considers that the axial force is taken by 60% of the resistance force provided by the concrete section.

$$N_{RD} = 0.6 \cdot f_{cd} \cdot A_c \geq N_{ED} \qquad 2.38$$

Where:

Ac is the area of the column section

Ned is the axial load

The column section can then be determined using this minimum section. The solicitations on the columns will then be got using the SAP 2000 software where a modal analysis will also be carried out to validate the columns sections and check for errors.

2.7.4.2. Longitudinal reinforcement

According to Eurocode 2, reinforcement bars parallel to the axis of axial loading should have a diameter of not less than $Ø_{min}$. The recommended value is 8mm. The total amount of longitudinal reinforcement should not be less than $A_{s,\,min}$. The recommended value is given by:

$$A_{s,min} = max\{0.1\ N_{Ed}/f_{yd}\ ; 0.002A_c\ \} \qquad 2.39$$

Where:

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

N_{Ed} is the design axial compression force.

The area of longitudinal reinforcement should not exceed $A_{s,\,max}$. The recommended value is $0.04Ac$ outside lap locations unless it can be shown that the integrity of concrete is not affected, and that the full strength is achieved at ULS. (§ 9.5.2-EC2).

Column assessment can be carried out by calculating a moment-axial load interaction diagram, which depicts the entire limit situation that can determine the failure of our section. The points on the diagram represent the limit configuration: anything beyond them results in failure. The internal points represent the safe combinations of solicitations to which the section could be subjected without failing. Some important points should be determined before drawing this diagram as shown in figure 2.2.

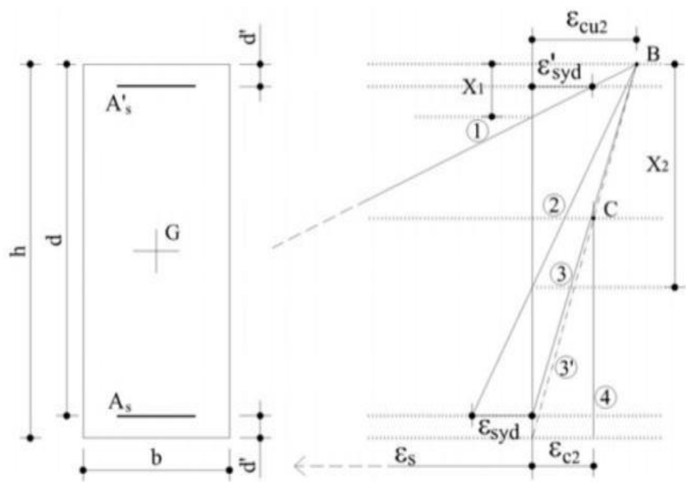


Figure 2.2. Possible strain distribution at ULS

2.7.4.3. Column transverse reinforcement

The transverse reinforcement (links, loops, or helical spiral reinforcement) should have a diameter of at least 6 mm or one quarter the maximum diameter of the longitudinal bars, whichever is greater. The diameter of welded mesh fabric wires for transverse reinforcement should not be less than 5 mm. The transverse reinforcement should be well anchored. The transverse reinforcement spacing along the column should not be greater than $S_{cl, tmax}$. The recommended value is the smallest of the three distances:

$$spacing = min\ [20\emptyset_{min}, lesser\ dimnesionof\ column, 400mm] \tag{2.40}$$

2.7.4.4. Slenderness verification

The slenderness verification determines whether to consider the second-order effect. It consists in determining whether the element's slenderness is less than a limit value defined by Eurocode 2 as in the relation 2.41.

$$\lambda_{\text{lim}} = \frac{20 \cdot A \cdot B \cdot C}{\sqrt{n}} \tag{2.41}$$

Where:

$A = 1/(1 + 0.2\ \varphi\_ef\ )$ ($-_{ef}$ is the effective creep ratio; A= 0.7 if φ_{ef} is not known)

$B = \sqrt{(1 + 2w)}$ ($w=A_sf_{yd}/A_cf_{cd}$ is the mechanical reinforcement ratio)

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

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

The slenderness of an element is evaluated by:

$$\lambda = \frac{l_0}{i} \tag{2.42}$$

Where:

i is the gyration radius of the uncracked concrete section

l_o is the effective length of the element (l_0=0.7l)

The gyration radius of the uncracked section is given by:

$$i = \sqrt{(I/A)} \tag{2.43}$$

Where:

I is the moment of inertia

A is the area of the section.

2.7.5. Shear wall design

The shear wall's reinforcement is designed in the vertical, horizontal and transverse directions. The resisting moment provided by this reinforcement, M_{RD} is verified to be greater than the acting moment, M_{ED}.

2.7.5.1. Vertical reinforcement

The vertical reinforcement area should be between $A_{s, vmin}$ and $A_{s, vmax}$. Outside lap locations, the recommended values for $A_{s, vmin}$ is 0.002 Ac and for $A_{s, vmax}$ is 0.04 Ac, unless it can be demonstrated that the concrete integrity is not compromised and that full strength is achieved at ULS. At laps, this limit can be doubled. In designs where the minimum area of reinforcement, $A_{s, vmin}$, controls, half of this area should be located at each face. The distance between two adjacent vertical bars must not be greater than three times the wall thickness or 400 mm, whichever is less.

2.7.5.2. Horizontal reinforcement

Each surface should have horizontal reinforcement running parallel to the faces of the wall (and to the free edges). It should not be less than $A_{s,\,hmin}$. The recommended value of $A_{s,\,hmin}$ is 25% of the vertical reinforcement or 0.001 Ac, whichever is greater. The distance between two adjacent horizontal bars should not exceed 400 mm.

2.7.5.3. Transverse reinforcement

In any part of a wall where the total area of the vertical reinforcement in the two faces exceeds 0.02 Ac, transverse reinforcement in the form of links should be provided in accordance with the requirements for columns. Where the main reinforcement is placed nearest to the wall faces, transverse reinforcement should also be provided in the form of links with at least of 4 per m^2 of wall area.

With the calculated As, the resisting moment is calculated by equation 2.44.

$$M_{Rd} = A_s.f_{yd}.(d-0.4x) \quad ; M_{RD} > M_{ED} \qquad 2.44$$

2.7.6. Foundation design

Soil's bearing capacity is defined as σ_s, the area of footing is gotten by:

$$A = \frac{N_{Ed}}{\sigma_s} \qquad 2.45$$

Self-weight of footing is got as:

$$G_f = a\ x\ b\ x\ h\ x\ \gamma_c \qquad 2.46$$

The maximum stress on soil is computed as in equation 2.47 and should be less than σ_s.

$$\sigma_{max} = \frac{N_{tot}}{A} + \frac{M_{Ed}}{\frac{BA}{6}} \qquad 2.47$$

Using the solicitation, bending moment, got from the SAP2000 results, the longitudinal and transverse reinforcement is calculated per metre following the steps in section 2.5.3.

Soil-structure interaction is taken into account by the use of springs whose characteristic depend on foundation dimension, the soil properties, the amplitude of the input motion and the fundamental vibrations periods. The spring stiffness k has x, y and z components with kx=ky=0.5kz.

2.8. Seismic analysis

Seismic analysis is used to understand the response of buildings due to seismic excitations in a simpler manner. For this analysis, the various models with varying size and position of window openings will be used, having dimensions of traditional door and window openings. Springing from Anas Fares study (Fares, 2021b), one set has opening area percentage less than 65% and the other greater than 65%, to analyse and understand its influence. In this study, upon application of the seismic action, the modal response spectrum analysis will be used to analyse and verify the structure's behaviour.

2.8.1. Seismic action

Within the scope of EN 1998, the earth quake hazard is described in terms of a single parameter, i.e., the value of the reference peak ground acceleration on type A ground, $a_{gR.}$ The earthquake motion at a given point on the surface is represented by an elastic ground acceleration response spectrum, henceforth called an "elastic response spectrum". It has both horizontal and vertical components; various shapes and the selection of the shapes depends on the part of the country.

2.8.1.1. Horizontal elastic response spectrum (Se)

The elastic response spectrum is simply defined as a relation between the ground motion and the period of vibration during an earthquake. Figure 2.3 shows its shape.

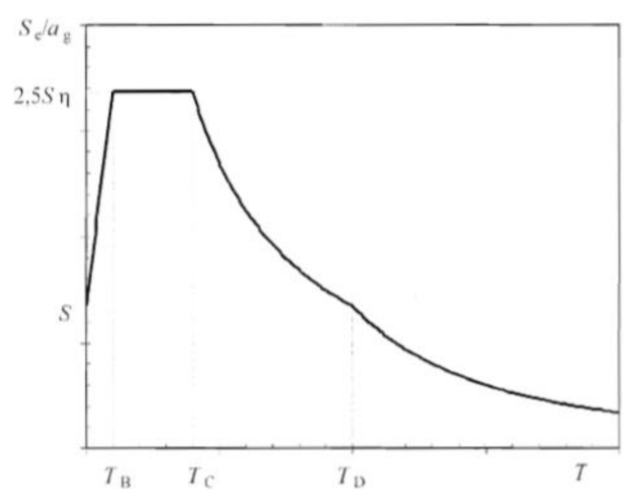


Figure 2.3. Shape of the elastic response spectrum

For the horizontal components of the seismic action, the elastic response spectrum S_e (T) is defined by the equations 2.48 to 2.51.

$$0 \leq T \leq T_B : S_e\ (T) = a_g \cdot S \cdot [1 + T/T_B \ \cdot (\eta \cdot 2.5 - 1)] \qquad 2.48$$

$$T_B \leq T \leq T_c : S_e\ (T) = a_g \cdot S \cdot \eta \cdot 2.5 \qquad 2.49$$

$$T_C \leq T \leq T_D : S_e\ (T) = a_g \cdot S \cdot \eta \cdot 2.5[T_C/T] \qquad 2.50$$

$$T_{CD} \leq T \leq 4s : S_e\ (T) = a_g \cdot S \cdot \eta \cdot 2.5[(T_C\ T_D)/T\hat{}2\ ] \qquad 2.51$$

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 (ag = γI. agR)

T_B is the lower limit of the 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 of the constant displacement response range of the spectrum

S is the soil factor

η is the damping correction factor with a reference value of η=1 for 5% viscous damping.

2.8.1.2. Ground type

Eurocode 8 considers seven different ground types, depending on their mechanical properties. This will depend on the geology of the area.

2.8.1.3. Peak ground acceleration values (a_{gR})

These values are available for different hazard zones in the National Annexe, usually through a hazard map. The chosen value has to be corrected by the importance factor γ_I (which depends on the building class) to obtain the design ground peak acceleration a_g ($a_g = \gamma_I . a_{gR}$). See Appendix 2.

2.8.1.4. Damping ratio

Damping depends on the material and structural type. Eurocode 8 uses a default damping ratio ζ of 5% so that the corresponding correction factor η is 1.

2.8.1.5. Spectra type

According to the Eurocode 8, there are two spectra types:

- Type 1 for high and moderate seismicity region (magnitude Ms greater than 5.5)
- Type 2 for low seismicity regions

2.8.2. Design response spectrum (Sd)

To avoid explicit inelastic structural analysis in design, the capacity of the structure to dissipate energy through mainly ductile behaviour is accounted for by performing elastic analysis using a reduced response spectrum, introducing q which is called behaviour factor. q is an approximation of the ratio of seismic forces that the structure would experience if its response were completely elastic with 5% damping to the seismic forces used for the design. The behaviour factor is then given by:

$$q = q_0 \, k_w \geq 1.5 \qquad 2.52$$

Where:

q_0 is the basic value of the behaviour factor dependent on the type of the structural system and on its regularity in elevation.

k_w is the factor reflecting the prevailing failure mode in structural systems with walls. For buildings that are regular in elevation, the basic values of q_0 for the various structural types are given in EN 1998 Section 5.2.2.2.

For the horizontal components of the seismic action, the design spectrum, S_d (T), is defined by:

$$0 \leq T \leq T_B: S_d\,(T) = a_g \cdot S \cdot [2/3 + T/T_B \;\cdot ((2.5)/q - 2/3)] \qquad 2.53$$

$$T_B \leq T \leq T_c: S_d\,(T) = a_g \cdot S \cdot (2.5)/q \qquad 2.54$$

$$T_C \leq T \leq T_D: S_d\,(T) = a_g \cdot S \cdot (2.5)/q \cdot [T_C/T] \geq \beta \cdot a_g \qquad 2.55$$

$T_D \leq T : S_d\ (T) = a_g \cdot S \cdot (2.5)/q \cdot [(T_C\ T_D)/T^2\ ]$ 2.56

Where:

$S_d\ (T)$ is the design spectrum;

β is the lower bound factor for the horizontal design spectrum.

q is the behaviour factor.

2.8.3. Linear dynamic analysis

Linear analysis covers a broad range of methods, which use linear elastic material behaviour (stiffness matrix remains constant) to determine structural dynamic properties and responses. Linear analysis can be conducted using force-based approaches, such as linear static analysis (equivalent lateral force analysis) and linear dynamic analysis (modal response spectrum analysis) or it can be conducted using a linear time-history approach. Linear static analysis is typically restricted to use in regular structures, where dynamic behaviour is dominated by the fundamental mode of vibration, without significant higher modes and torsion effects and in regions of low seismic forces. Since tall buildings often exhibit significant higher mode effects and the effects of torsion are important, a linear dynamic analysis will be performed.

2.8.3.1. The modal analysis

Before seismic forces are applied, an understanding of the natural vibration of the structure is needed and modal analysis helps understand the buildings dynamic behaviour. For this analysis, the software SAP 2000 is used. According to Eurocode 8, the response of all modes of vibration contributing significantly to the global response are taken into account if the sum of the effective modal masses for the modes taken into account amounts to at least 90% of the total mass of the structure.

a. Determination of total mass

The inertial effect of the design seismic action is evaluated by taking into account the presence of the masses associated with all gravity loads and is calculated as:

$\sum G_{k,\,j}$ "+" $\sum \psi_{E,\,i}\,.Q_{k,\,i}$ 2.57

Where:

$\psi_{E\,i}$, is the combination coefficient for variable action i, which takes into account the likelihood of the loads $Q_{k,\,i}$ not being present over the entire structure during the earthquake.

These coefficients may also account for a reduced participation of masses in the motion of the structure due to the non-rigid connection between them. The recommended values for ψ are given in EN 1990:2002.

b. Distribution of the horizontal seismic forces

The seismic action affects shall be determined by applying, to the two planar models, horizontal forces F_i to all storeys as shown in figure 2.4.

$$F_i = F_b x \frac{s_i\, m_i}{\sum_j s_j m_j} \qquad 2.58$$

Where:

F_i is the horizontal force acting on storey i.

F_b is the seismic base shear.

m_i , m_j are the storey masses.

s_i, s_j are the displacements of masses m_i, m_j in the fundamental mode shape.

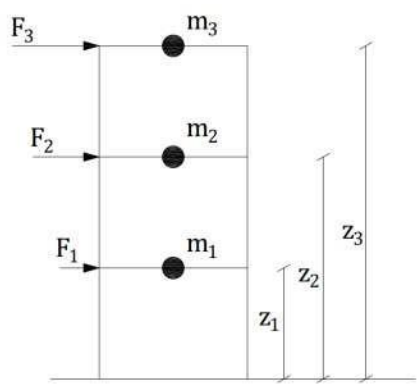


Figure 2.4. Lateral distribution of seismic forces

The mass is computed in accordance with EC8:3.2.4 (2):

$$m_i = W_i / g \qquad 2.59$$

Where:

$$W_i = \sum G_{k,j} + \sum \psi_{E,i} . Q_{k,i} \quad 2.60$$

2.8.3.2. Modal response spectrum analysis

The first objective of a modal response spectrum analysis is to determine the peak value of any seismic action effect of interest (be it a global effect, such as the base shear, or local ones, such as member internal forces, or even intermediate ones, such as inter-storey drifts) in every one of the n modes considered due to the seismic action component in direction X, Y or Z. The steps followed are:

- Selection of useful modes
- Ground-level Acceleration of the Structure
- Response of the Structure

In this case, the analysis criteria will be the base shear, displacement and inter-storey drifts.

a. Base shear force

The seismic base shear force F_b, for each horizontal direction in which the building is analysed, is determined by:

$$F_b = S_d\ (T_1) . m\ . \lambda \quad 2.61$$

Where:

F_b is the shear base in the direction of analysis

$S_d\ (T_1)$ is the ordinate of the design spectrum at period T_1

T_1 is the fundamental period of vibration of the building for lateral motion in the direction considered

m is the total mass of the building, above the foundation or above the top of a rigid basement;

λ is the correction factor, the value of which is equal to $\lambda = 0.85$ if $T_1 < 2\ T_C$ and the building has more than two storeys, or $\lambda = 1.0$ otherwise.

b. Displacement and inter-story drift

The displacement of a building can be represented by EN 1990 (Annexe-1) as in figure 2.5.

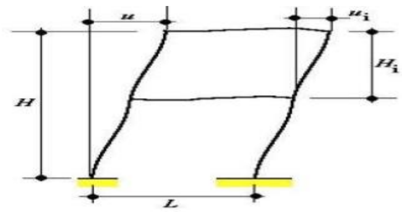


Figure 2.5. Lateral displacement (EN 1990-1)

Where:

u_i is the horizontal displacement over a storey height H_i

u is the overall horizontal displacement over the building height H.

According to EN 1998-1, the displacements induced by the design seismic action, is calculated on the basis of the elastic deformations of the structural system, non-dissipative. The behaviour coefficient is 1 and determined at SLS with a design spectrum where q=1.

Inter storey drift is defined as the difference in the displacement values of adjacent storey divided by the storey height. Drift control is necessary to limit damage to interior partitions, stairs and glass and cladding systems. Eurocode 8 gives a damage limitation in terms of inter-storey drift;

$$d_r \cdot v \leq 0.005h \qquad 2.62$$

Where:

d_r is the design inter storey drift,

h is the story height.

v is the reduction factor which takes into account the lower return period of the seismic action associated with the damage limitation requirement. It depends on the importance class of the building. The recommended values of v are 0.4 for importance classes III and IV and v = 0.5 for importance classes I and II.

c. Combination of modal responses

To minimise the disadvantages, it is necessary to combine the modal responses. The design code EN 1998 suggests two methods, the Square Root of Sum of Squares and Complete Quadratic Combination, depending on if the response of two vibration modes dependent of each other or not.

i. SRSS (Square Root of Sum of Squares)

Whenever all relevant modal responses may be regarded as independent of each other, the maximum value of a seismic action effect may be taken as:

$$E_E = \sqrt{\sum_{i=1}^{n} E_{Ei}^2} \qquad 2.63$$

Where:

E_E is the seismic action effects under consideration, force or displacement

E_{Ei} is the value of the seismic action effects due to the vibration mode i.

In case the two vibration modes i and j are not independent, a more accurate procedure for the combination of modal maxima, such as the 'Complete Quadratic Combination' shall be applied.

ii. CQC (Complete Quadratic Combination)

This method considers the correlation between modal responses. The global response is achieved applying the equation 2.64.

$$E_E = \sqrt{\sum_{i=1}^{n} \sum_{j=1}^{n} e_{ij} E_i E_j} \qquad 2.64$$

Conclusion

This chapter focused on describing the method for analysis, design and verification of the different models of the structure. The method used to design structural elements such as beam, column, foundation and shear walls of the structure were discussed in static design. For the dynamic analysis, the modal analysis and response spectrum seismic analysis procedures have been explained. Also, the structural response parameters used for the comparison of the different reinforced concrete wall models have been defined and explained. The analysis will be carried out with the SAP 2000 v22 software and the comparative results better represented with the Microsoft Excel software as presented in the next chapter.

CHAPTER 3. PRESENTATION OF CASE STUDY AND RESULTS

Introduction

The methodology presented in the previous chapter is applied to a case study and the results are presented here. This chapter will consist in a preliminary part the presentation of the case study and the different load and material properties considered for its analysis. This will be followed by a static and dynamic analysis in order to determine the different sections of the structural elements of the superstructure and the substructure. Then, the definition of different models with varying size and position of openings. The results of the comparison of the building's seismic behaviour in terms of vibration periods, lateral displacement and inter-storey drift for these models are also presented and interpreted.

3.1. General presentation of the site

The building is conceived to be located in the town of Buea, capital of the Southwest Region of Cameroon. Some physical parameters and socio-economic parameters of the area shall be presented in this section (Angelica, 2020).

3.1.1. Geographical location

Buea belongs to the Fako division of the South-West region and measures a total surface area of 870km$^{2.}$ Situated at latitude 4.1560° North and longitude 9.2632° East at an elevation of 900 meters above sea level southeast slope of Mount Cameroon. Figure 3.1 shows the map of Buea.

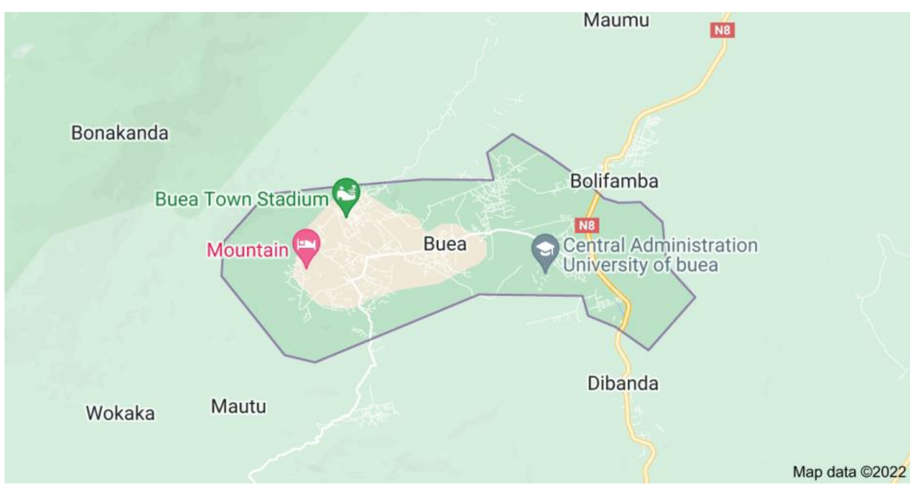


Figure 3.1. Map of Buea (Google maps)

3.1.2. Geology and relief

The area is composed of undulating high and low lands with many rocks and gravels due to volcanic eruptions. The area is well drained due to the generally hilly nature of the terrain and the fact that they are free-draining. The soil type consists of basalts and is because of the first volcanic activity in the Fako Mountain area, which occurred in the cretaceous system. These soils have been weathered and partly covered by more recent deposits; thus, the soils are black.

3.1.3. Climate and hydrology

Buea GMT time is +1 hour and is mostly cloudy. It has an equatorial climate with 2 major seasons. Rainy season which runs from March to October and Dry season, from November to May. Temperature ranges between 20ºC to 28ºC while, annual rainfall ranges between 3000mm to 5000mm. The conditions here are generally the tropical rainforest climate with rainfall almost during the entire year. However, average monthly High/Low Temperature for these urban spaces ranges from 23 low to 32 high.

3.1.4. Demography and economic activities

Buea has an estimated population of about 200.000 inhabitants constituting essentially of the Bakweri (the indigenes). Buea's main economic activities are education, trade, public service, and diplomatic services, so its buildings consist of residential homes, schools, hospitals, commercial, and multipurpose buildings.

3.2. Presentation of the case study

Data of the conceived case study; geometric details of the model, material properties and actions on the building are presented in this section.

3.2.1. Geometry of building

The building which has been adopted for our study is a nine-storey building, ground floor + 8, of 3.4 m height per story, regular in plan and in elevation. The total height of the building is 30.6 m, 19 m long and 17 m wide. The building has mainly offices and meeting rooms thus category B and class II buildings according to code EN 1991-1 and EN 1998 respectively. The structural elements consist of frame elements as beams and columns, a thick-ribbed slab of total thickness 20 cm and an RC shear wall of thickness of 30 cm, around the stair case area, and constant all over the height of the building. Concrete cover adopted is 4 cm for all levels. The detailed architectural and structural plans are shown in appendix 3 & 4.

3.2.2. Materials properties

For the analysis, the class of concrete and steel adopted is defined for the whole structure.

3.2.2.1. Concrete properties

The class of concrete adopted is C25/30. The main properties used for the linear analysis and design are given in table 3.1.

Table 3.1. Concrete characteristics

Property	Value	Unit	Definition
Class	C25/30	-	Concrete class
R_{ck}	30	N/mm^2	Characteristic cubic compressive strength
f_{ck}	25	N/mm^2	Characteristic compressive strength of concrete at 28 days
f_{cm}=fck+8	33	N/mm^2	Mean value of concrete cylinder compressive strength
α_{cc}	0.85		The coefficient of long-term effects on compressive strength and unfavourable effects from the way the load is applied
γ_c	1.5	-	Partial safety factor for concrete
$f\_cd = (\propto_{cc} f_{ck})/\gamma_c$	14.17	N/mm^2	Design value of concrete compressive strength
$f\_ctm = 0.3\ x\ (f_{ck})^{\frac{2}{3}}$	2.56	N/mm^2	Mean value of axial tensile strength of concrete
$f\_ctd = 0.7\ x\ \frac{f_{ctm}}{\gamma_c}$	1.2	N/mm^2	Design resistance in traction
$E\_cm = 22.\left(\frac{f_{cm}}{10}\right)^{0.3}$	31	GPa	Secant modulus of elasticity
γ	25	KN/m^3	Specific weight of concrete
ε_{c2}	2	‰	The strain at reaching the maximum strength

ε_{cu2}	3.5	‰	The ultimate strain

3.2.2.2. Steel reinforcement properties

The steel reinforcement is FeB500C. The main properties used for the linear analysis and design are given in table 3.2.

Table 3.2. Steel reinforcement characteristics

Property	Value	Unit	Definition
Class	B500C	-	Longitudinal reinforcement class
γ_s	1.15	-	Partial safety factor of steel
f_{yk}	500	N/mm ^{2}	Characteristic yield strength of longitudinal reinforcement
f_u	600	N/mm ^{2}	Characteristic ultimate strength of longitudinal reinforcement
f_{yd}	434.78	N/mm ^{2}	Design yield strength of longitudinal reinforcement
$\varepsilon_s=\varepsilon_{yd}$	0.196	%	Yield strain
ε_{ud}	6.75	%	Ultimate strain
f_{ywk}	235	N/mm ^{2}	Characteristic yield strength of transversal reinforcement
f_{ywd}	204.35	N/mm ^{2}	Design yield strength of transversal reinforcement
Es	210,000		Modulus of elasticity
γ	78.5	KN/m^3	Specific weight of steel
ν	0.3		Poisson ratio

3.2.3. Loads used

The loads considered in the design of the structure are permanent loads and variable loads.

3.2.3.1. Permanent loads

Permanent loads include structural loads and non-structural loads applied on the building. The structural load is got from the self-weight of the structural elements. The self-weight of the beams and columns are calculated by the software automatically and an equivalence of the dead load of the ribbed slab is applied on the primary beams. Table 3.3. shows the list of permanent loads.

Table 3.3. Permanent loads

Nature	Designation	Value	Unit
G1k	self-weight of slab	2.8	KN/m^2
G2k	nonstructural	1.8	KN/m^2
G2k	Partition wall	3.5	KN/m^2
G2k	External walls	6.9	KN/m

3.2.3.2. Variable loads

These include the imposed live loads of the building and the seismic load.

a. Live loads

According to the EN 1991-1-1 code, building classified as category B, the imposed live load is between 2 and 3 $KN/m^{2.}$ For this case, 2.5 KN/m^2 was considered. With the roof being inaccessible, except for maintenance (category H), imposed load on roof is $1KN/m^2$.

b. Seismic loads

Seismic loads used are described with its modal spectrum presented in section 3.3.3.

3.2.4. Durability and concrete cover

Design working life of 50 years, structural class S4, exposure class XC1 for superstructure and XC2 for foundations. According to the procedure mentioned in section 2.7.2 of methodology:

$C_{min} = max\ (16;15+0\text{-}0\text{-}0;10\ mm) = 16\ mm$

$C_{nom}=16+10=26\ mm$

Nominal concrete cover of 26 mm, taking a value greater than or equal to that, *Cnom* used is 40 mm for security.

3.3. Linear analysis

The building is modelled with SAP 2000 v22, beams, columns and shear walls. Loads acting are applied and combined and the solicitations got are then used to design and verify the structural elements. Upon application of seismic load, a response analysis is done and results presented in this section.

3.3.1. Static analysis

In the static analysis, the forces and solicitations on the various elements were got and then designed according to the norms, verifying them in SLS too.

3.3.1.1. Design of beam

The beam selected for design is the beam with largest influence area and largest solicitations in SAP 2000 as identified in figure 3.2.

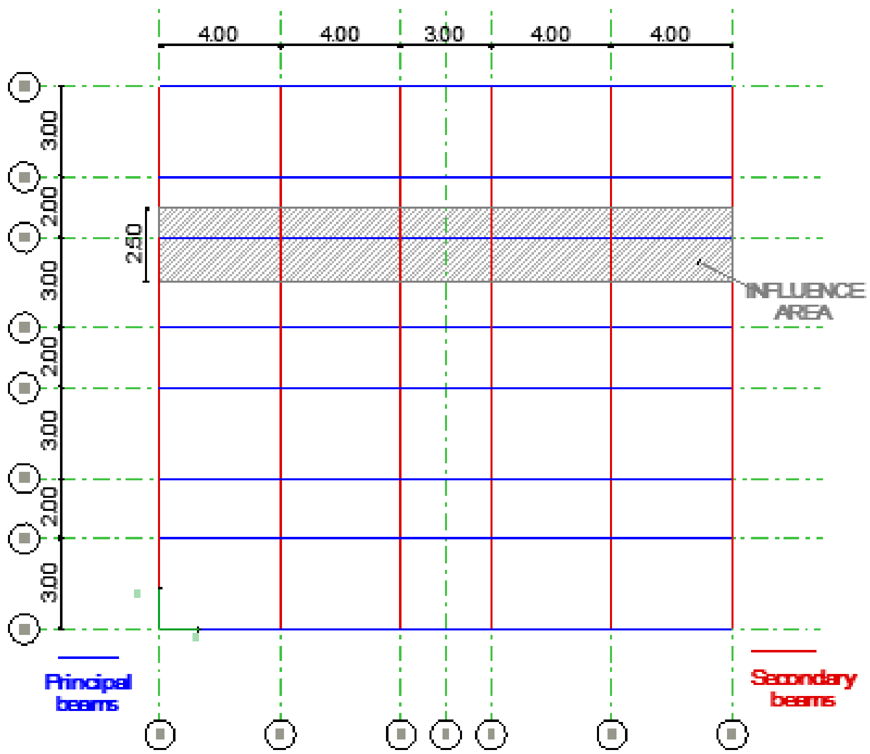


Figure 3.2. Principal and secondary beams

a. Preliminary design

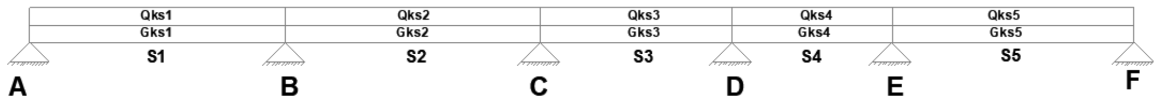
According to the procedure mentioned in section 2.7.3, the longest span of beam is 4.0 m and beam's depth is calculated as:

$$h \geq 4.0/14 \;\; and \; b \cong 0.5\, h, hence\ consider\ h = 0.6m\ and\ b = 0.3m$$

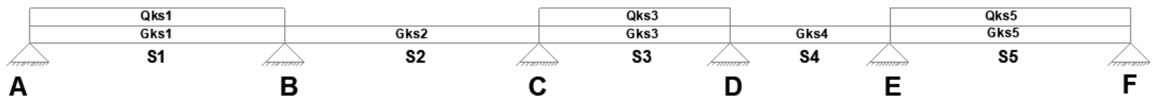
b. Load combinations

The loads along each span were combined so as to have various cases of maximum moment and shear at spans and at supports. The 13 combinations are shown in figure 3.3.

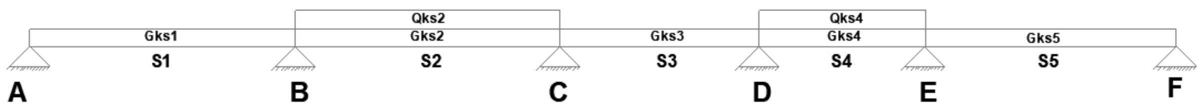
- Combination 1



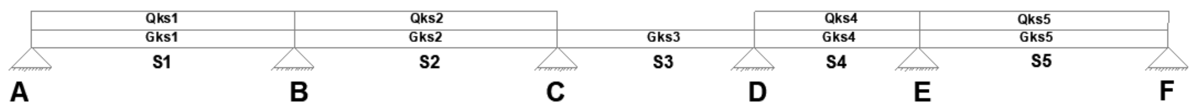
- Combination 2



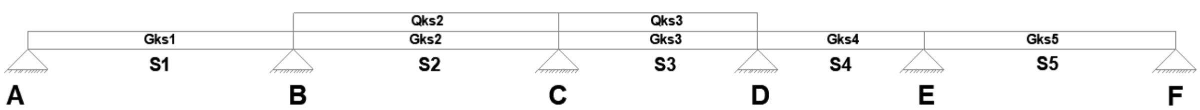
- Combination 3



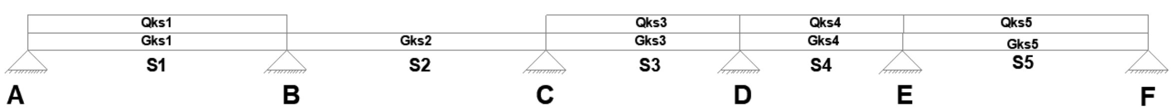
- Combination 4



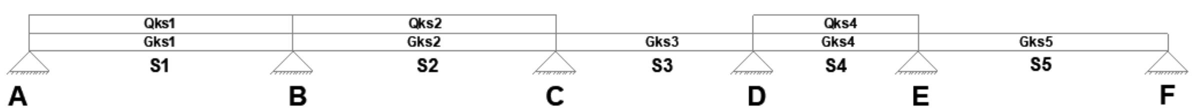
- Combination 5



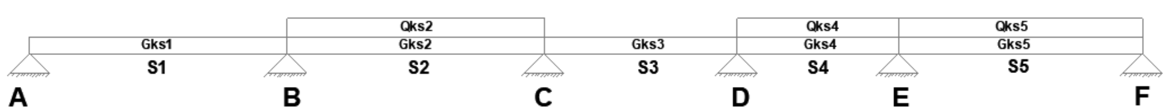
- Combination 6



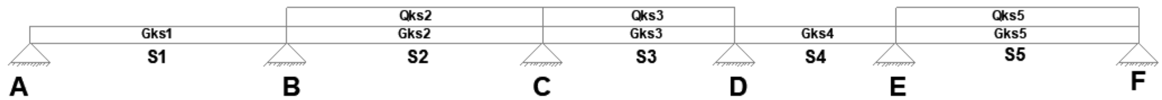
- Combination 7



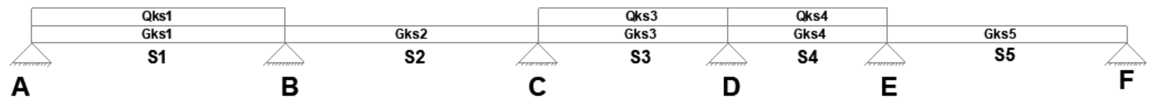
- Combination 8



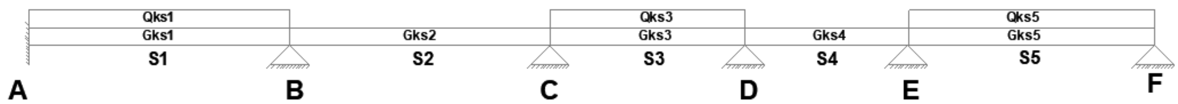
- Combination 9



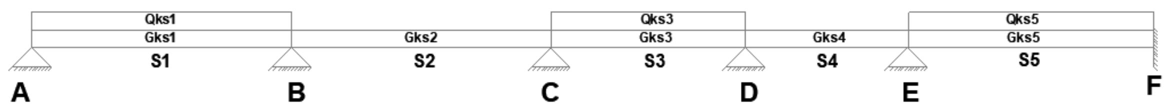
- Combination 10



- Combination 11



- Combination 12



- Combination 13

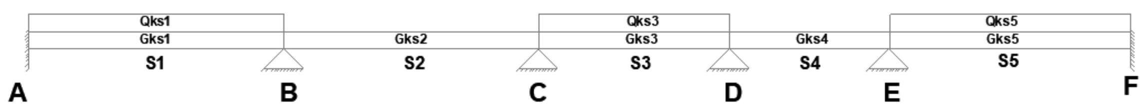


Figure 3.3. Load combination for beam design.

Using these combinations, the solicitations at ULS and SLS were got for design and verification.

c. Ultimate limit state design

In the software SAP 2000, the load combinations were inputted and the internal forces (moment and shear) all along the beam were got. Exported to excel, the solicitations and envelope curve, showing maximum and minimum values, are represented on figures 3.4-3.7 for better analysis.

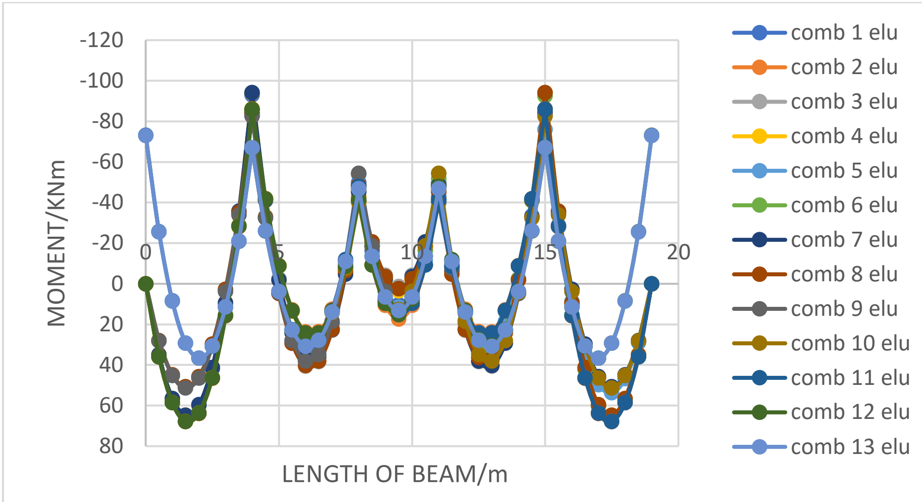


Figure 3.4. Bending moment diagram for load combinations on the beam

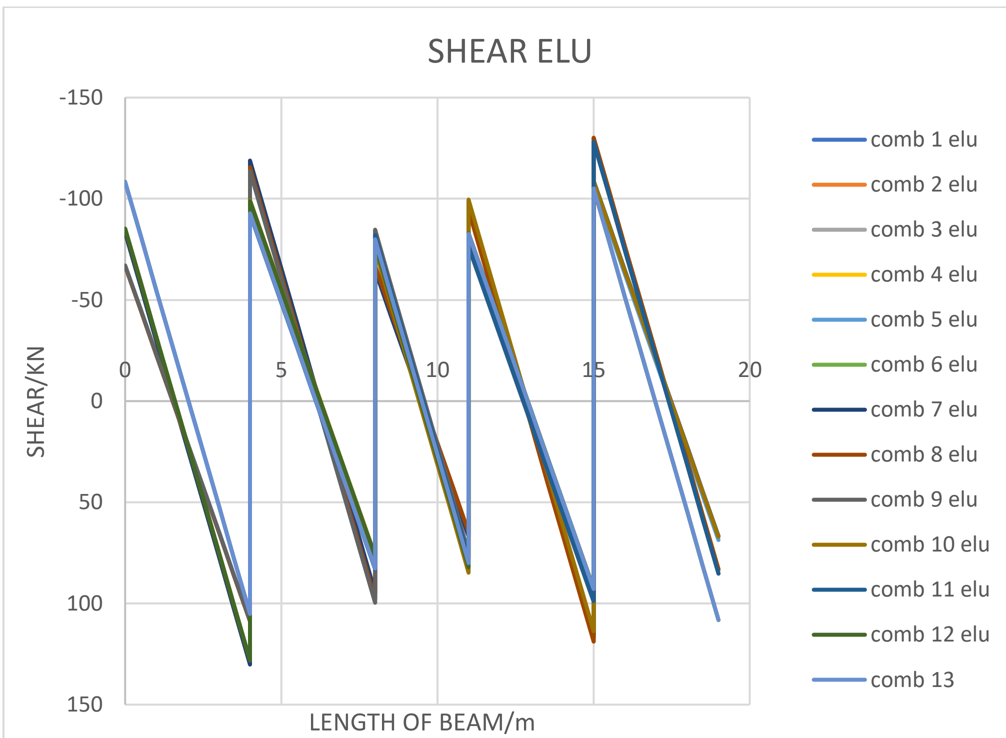


Figure 3.5. Shear force diagrams for load combinations on the beam

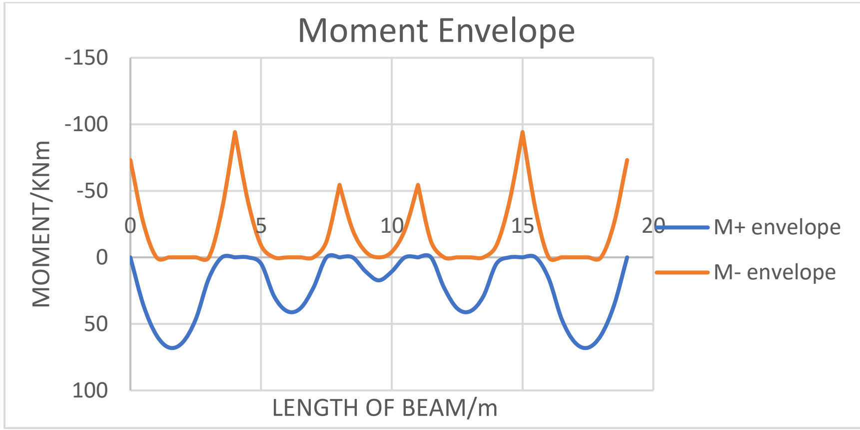


Figure 3.6. Envelope curve of the bending moment on the beam

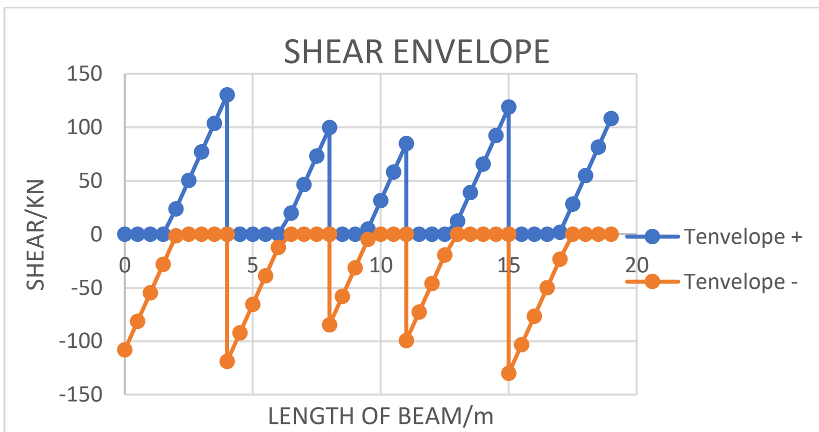


Figure 3.7. Envelope curve of Shear force on the beam

Following the procedure in section 2.7.3, a 30 × 60 (cm) section was adopted and the longitudinal and transverse reinforcement obtained.

i. Longitudinal reinforcement

Table 3.4 below shows the longitudinal reinforcement for the beam.

Table 3.4. Longitudinal reinforcement for beam

Location	M_{Ed} (KNm)	$A_{s,required}$ (mm^2)	Reinforcement adopted	$A_{s,provided}$ (mm^2)	$M_{Rd,provided}$ (KNm)
Support	70	365.4	3ϕ16	603.2	121.8
Span	100	802.5	4ϕ16	804.2	165.4

ii. Transverse reinforcement

Designed with V_{Ed}=151KN, the reinforcement provided were rods of 8 at a spacing of 10 cm at the supports and 15 cm at mid span sections.

d. Serviceability limit state design

In the software SAP 2000, the load combinations were inputted and the internal forces (moment and shear) all along the beam were got. Exported to excel, these solicitations and the envelope curve, showing maximum and minimum values, are represented on figures 3.8 and 3.9 for better analysis.

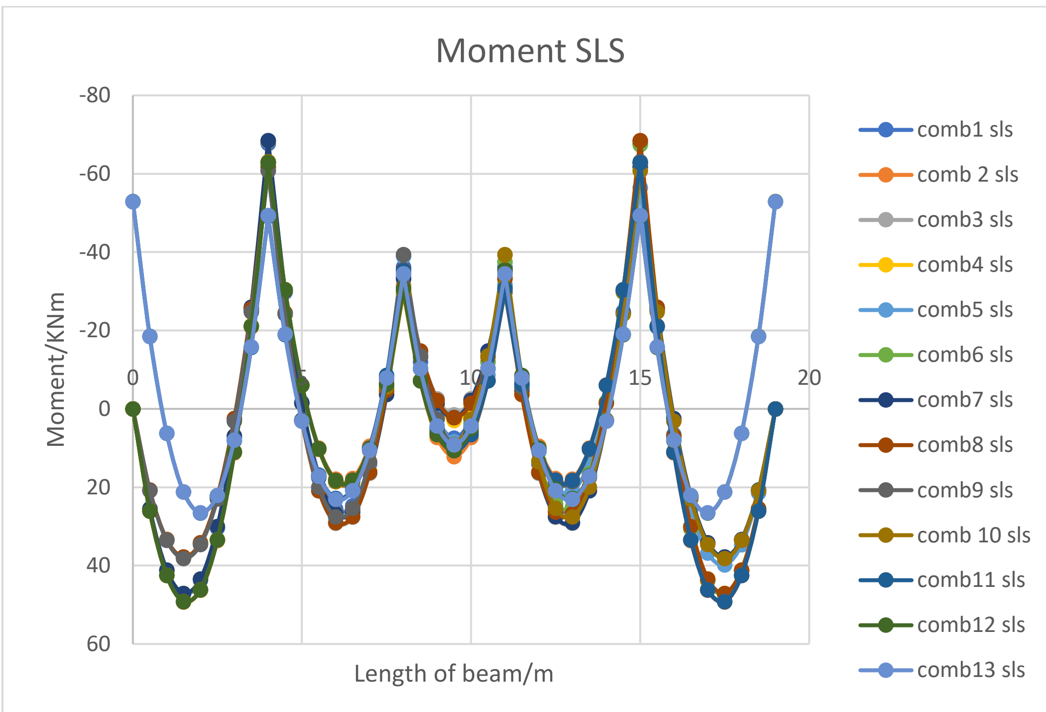


Figure 3.8. Bending moment diagram for load combinations on the beam

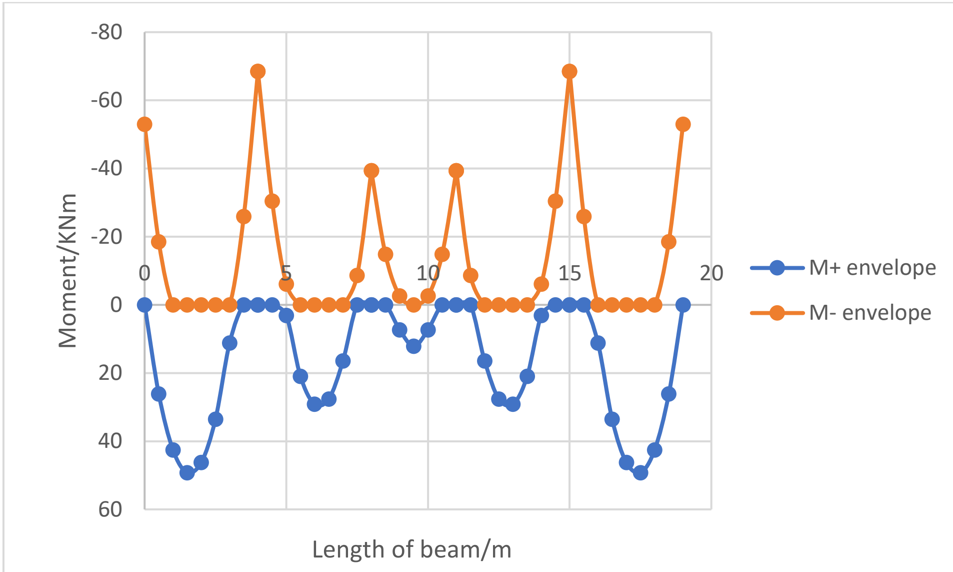


Figure 3.9. Envelope curve of SLS bending moment

i. Stress limitation

Following the procedure described in section 2.7.3.4, the stress distribution in concrete and longitudinal reinforcement was verified with:

σ_c=11.7 <15 and

σ_s=204 <360 (N/mm) verified respectively.

ii. Deflection

The deflection control was calculated as in section 2.7.3.6. resulting in:

l/d=7.35 <117 (mm) hence verified.

iii. Crack control

For crack control to be verified, a maximum reinforcement diameter of 20mm and a maximum reinforcement spacing of 250 mm were obtained at a limiting crack width of 0.4 mm as detailed in section 2.7.3.5.

3.3.1.2. Design of column

For the design, the columns with greatest axial forces were considered that is, column on grid line B-6 as shown in figure 3.10. The preliminary design, solicitations, longitudinal reinforcement, M-N design, shear verification and slenderness are presented.

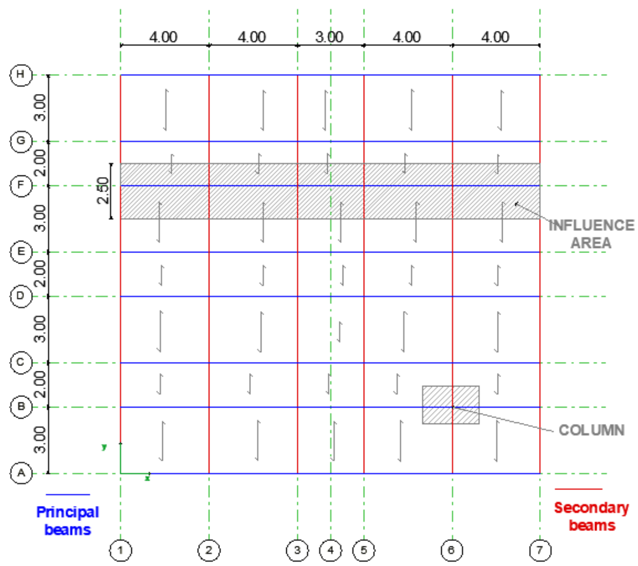


Figure 3.10. Illustrating column to be designed.

a. Preliminary design

According to section 2.7.4, $A_c \geq 183{,}015\ mm^2$. Hence a section of 600 × 600(mm) is chosen and adapted to all storeys.

b. Solicitations

In the software SAP 2000, the load combinations were inputted and the internal forces, moment and axial force, all along the columns were got. Exported to excel, these solicitations and the envelope curve, showing maximum and minimum values, are represented on figures 3.11 to 3.13 for better analysis and design.

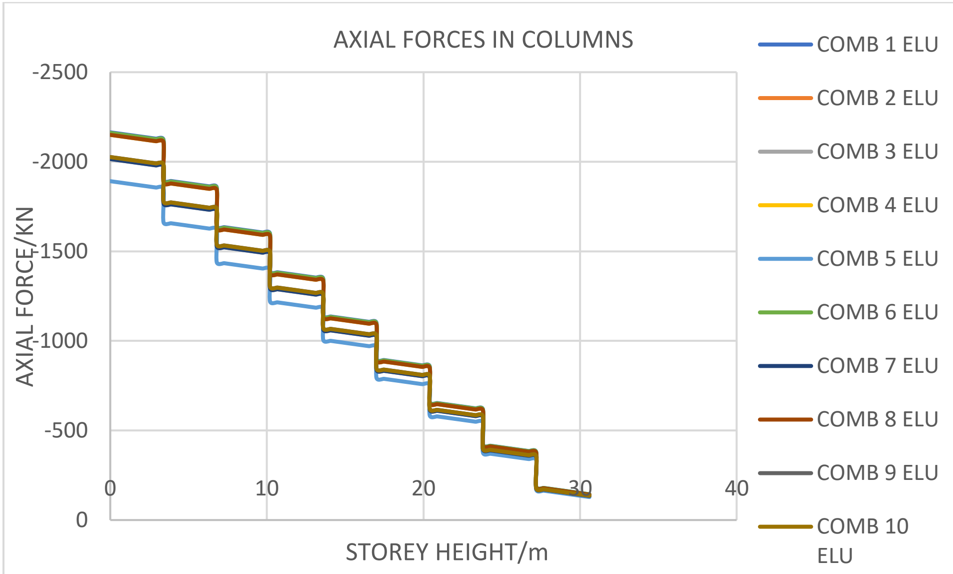


Figure 3.11. Axial forces diagram

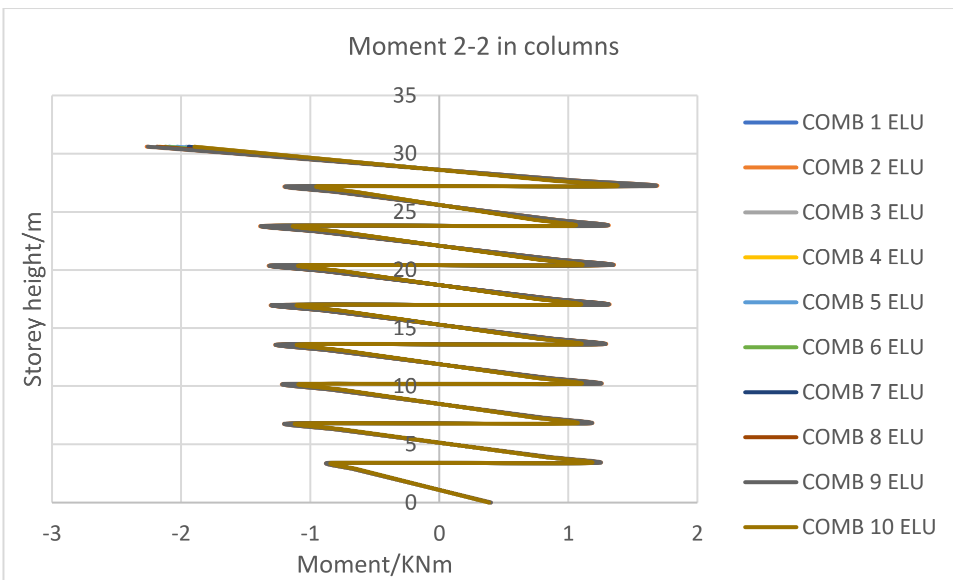


Figure 3.12. Moment diagrams in the x-direction

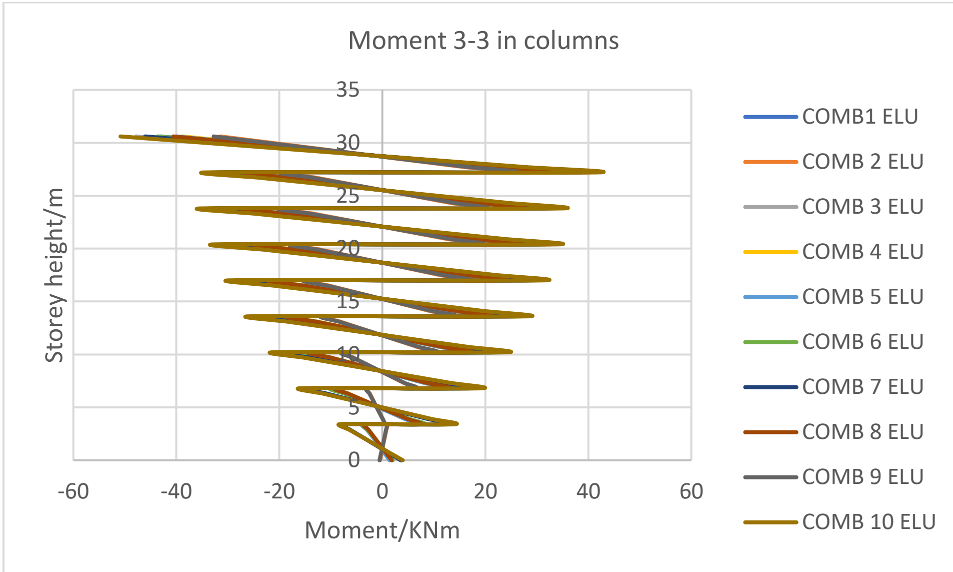


Figure 3.13. Moment diagram in the y-direction

c. Longitudinal reinforcement

According to the procedure in section 2.5.4.2 of methodology, the longitudinal reinforcement of the columns was determined as in table 3.5.

Table 3.5. Longitudinal reinforcement for columns

Ned (KN)	As, min (mm ^{2})	As, max (mm ^{2})	Reinforcement provided	As, provided (mm ^{2})
2165	357.9	14,400	12ϕ16	2412.7

d. M-N interaction diagram

It is necessary to check that the designed column can withstand the interaction between the bending moment and the axial force without failure and this was done with an M-N interaction diagram, figure 3.14.

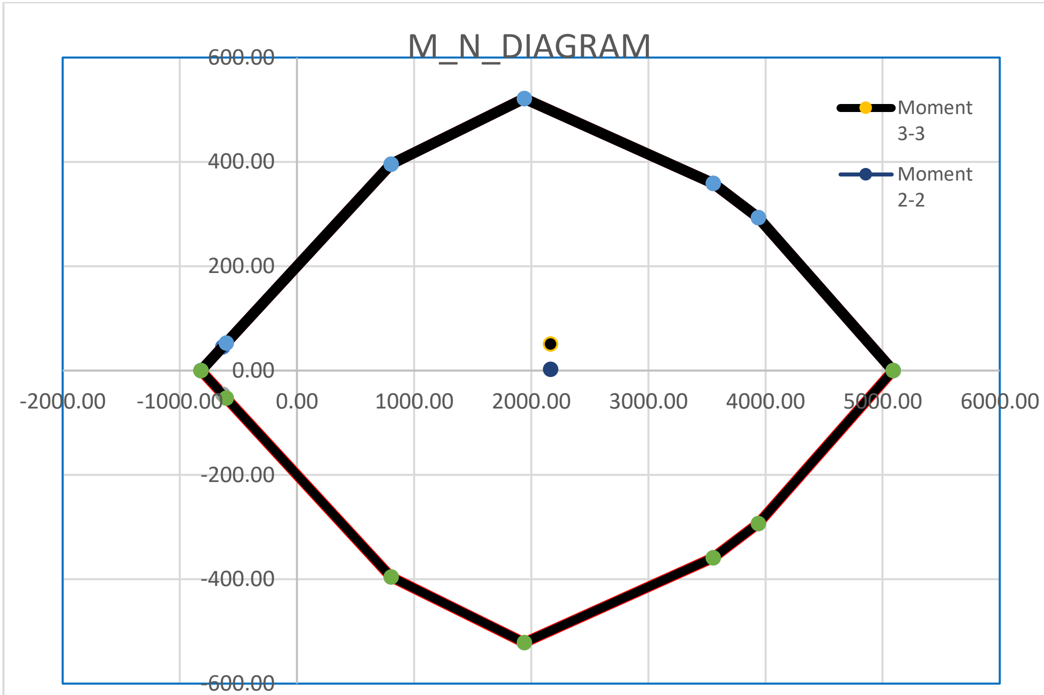


Figure 3.14. M-N interaction diagrams along x and y direction

e. Design for Shear

The concrete area is capable of withstanding the shear force but the minimum reinforcement and spacing of the transverse rods must be provided which is bars of 6 with spacing of 240 mm. Thus, a spacing of 200 mm was adopted.

f. Slenderness verification

According to section 2.7.4, buckling verification in columns, gave the values:

λ_{lim} =21.12

λ= 13.74

$\lambda < \lambda_{lim}$, thus column verifies slenderness.

3.3.1.3. Design of foundation

According to section 2.7.6, the foundation was designed for its reinforcement.

a. Preliminary design

The bearing capacity of soil σ_s is 0.35 MPa and Ned= 2165 KN. According to equation 2.44, A=4.44m². The dimension for footing is taken as 2.4x2.4x0.8 (m).

b. Longitudinal reinforcement

According to section 2.7.6, G_f= 115.2 KN.

σ_{max} = 0.32 MPa.

With the distance between neighbouring footings, being less than 1 m and the total surface area of footing, 276 m², greater than 50% of area of building, Foundation adopted was a raft foundation with depth of 1 m.

Taking into account soil-structure interaction, the nature of the soil is moist clay soil, subgrade modulus 83,333 t/m^3 with springs of dynamic stiffness values (k_x, k_y, k_z) = (5000,5000,10,000) KN/m were assigned.

– The maximum acting bending moment in the raft foundation got from SAP2000 in table 3.6.

Table 3.6. Solicitations in foundation

	X direction	Y direction	units
Positive bending moment	690	523	KN. m
Negative bending moment	568	490	KN. m

– The reinforcement adopted for both directions per metre was calculated, giving a resisting M_{RD}> M_{ED}, hence, reinforcement provided resist bending moment in foundation as shown in table 3.7.

Table 3.7. Foundation reinforcement

MEd/KN. m	As calculated	As provided	MRd/KN. m	Spacing (mm)
690	1699	(6ϕ20) 1885	763	176
568	1392	(6ϕ20) 1885	763	176

– The beams on foundation with maximum moment of 440KN.m adopt the same reinforcement as the raft foundation with intermediate reinforcement.

3.3.2. Modal analysis

A modal analysis was performed to understand the natural vibration of the building. Modelling the building with SAP 2000 software, the different modal properties were got and expressed for better analysis of its deformation trends.

3.3.2.1. Building model

The 3D model of the building is shown in figure 3.15, made up of horizontal and vertical elements representing the beams and columns respectively, shell area element representing the reinforced concrete walls and foundation. The slab elements of the building were not represented as one-way slabs were considered with their loads distributed to principal beams as shown in figure 3.2. Diaphragms were applied to each floor to ensure a rigid slab system.

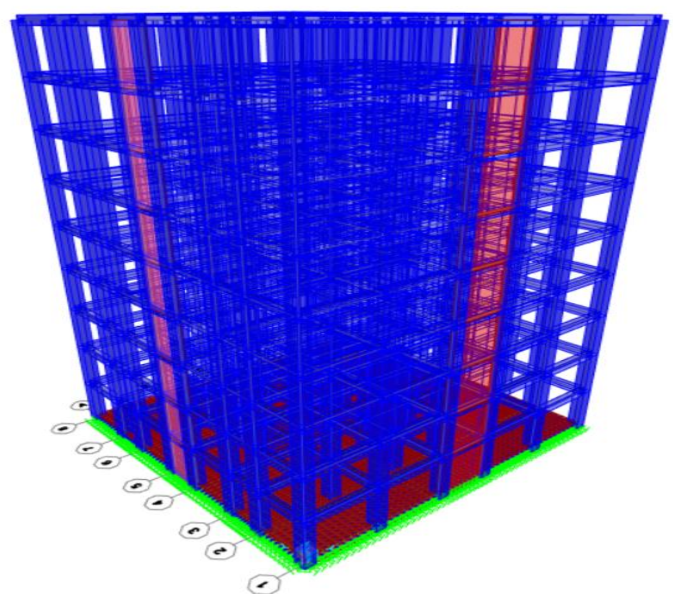


Figure 3.15. 3D model of building

3.3.2.2. Modal properties

In dynamic analysis, the higher frequency modes can be neglected. As a result, this analysis considers the first 26 modes whose sum of effective modal masses amounts to at least 90% of the total mass of the structure in the principal x, y and z directions. Table 3.8 shows the first 26 modes, their respective periods, frequencies and effective modal masses graphically represented in figure 3.16.

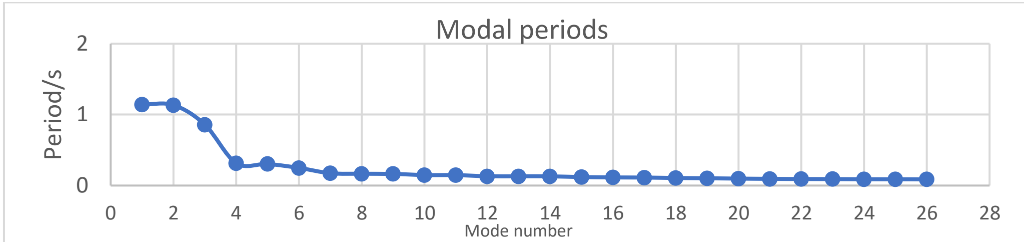
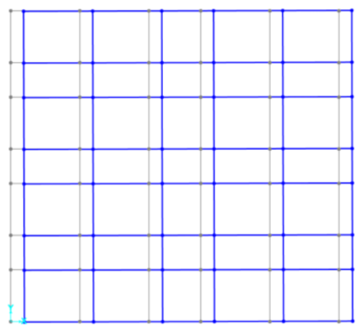


Figure 3.16. Natural periods of building for selected modes

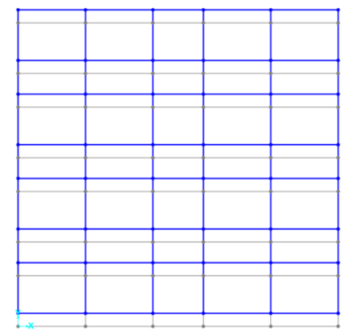
Table 3.8. Modal properties of selected modes

MODE	PERIOD (s)	FREQUENCY (Hz)	MODAL PARTICIPATING MASS RATIO		
			x direction	y direction	z direction
1	1.142956	0.874924319	0.6295	0	0
2	1.134725	0.881270792	0	0.63551	8.49E-10
3	0.859262	1.163789391	0.00021	0	0
4	0.316029	3.16426657	3.108E-16	0.11088	8.304E-10
5	0.304534	3.283705596	0.11745	1.168E-19	4.8E-19
6	0.248733	4.020375262	0.00021	2.169E-15	2.21E-16
7	0.174218	5.739935024	6.863E-15	0.03903	1.408E-07
8	0.16648	6.006727535	8.973E-15	2.411E-08	0.93687
9	0.163974	6.098527815	0.02634	2.118E-15	1.916E-15
10	0.148171	6.748958973	1.536E-13	0.02062	2.865E-08
11	0.147129	6.796756588	0.04497	2.26E-15	3.836E-15
12	0.131388	7.611045149	0.00032	9.525E-15	1.55E-15
13	0.130713	7.650348473	1.991E-14	0.00002196	0.00301
14	0.130049	7.689409376	0.00006096	2.028E-15	2.762E-16
15	0.119904	8.340005338	1.327E-13	3.712E-06	0.00259
16	0.114471	8.735837024	2.6E-16	0.02106	0.000007646
17	0.112958	8.852847961	2.747E-15	0.04283	5.785E-07
18	0.106897	9.35479948	0.00626	2.039E-14	1.17E-13
19	0.103019	9.706947262	0.08095	8.223E-19	1.045E-15
20	0.098089	10.19482307	0.00352	6.296E-14	3.895E-13
21	0.094661	10.56401263	6.921E-15	3.14E-08	0.00045
22	0.092056	10.86295299	1.949E-13	0.00096	0.0000216
23	0.091364	10.94523007	0.00005144	1.092E-17	3.629E-14
24	0.088873	11.2520113	0.000009044	8.1E-15	1.429E-12
25	0.088806	11.26050042	1.428E-13	0.00032	0.00075
26	0.087155	11.47381103	3.108E-14	0.09311	0.000001119
Sum			0.909851444	0.96434573	0.943701115

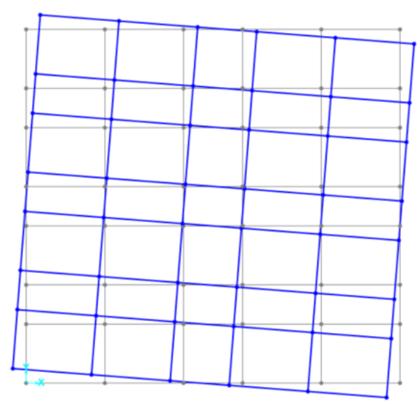
The deformation patterns of the first 3 modes are shown in figure 3.17 which are the fundamental natural modes of vibration of the structure.



(a) Mode 1- translation in x direction



(b) Mode 2- translation in y direction



(c) Mode 3-Torsion

Figure 3.17. First 3 natural modes of vibration of the structure.

The natural period and modes of vibration reflect the flexibility and rigidity of the structure. These intrinsic characteristics of the structure have a role to play in the response of the building to seismic action. In the following section, the seismic response of the building is analysed through response spectrum analysis.

3.3.3. Response spectrum analysis

In this analysis, a seismic force was applied as a response spectrum. The different models were defined and their different seismic responses were analysed in terms of base shear, period, lateral displacement and inter storey drift.

3.3.3.1. Ground conditions and seismic action

The conceived building is used for office area (category B). The soil type is D and considered to be in a medium seismic zone, a_{gR} is 1.6. The importance coefficient is 1, building class II. The spectrum type chosen is T1 and soil factor read from graph is 0.3351. The resulting horizontal elastic response spectrum is shown in figure 3.18. A behaviour factor of 2.457 was used to get the design response spectrum figure 3.19.

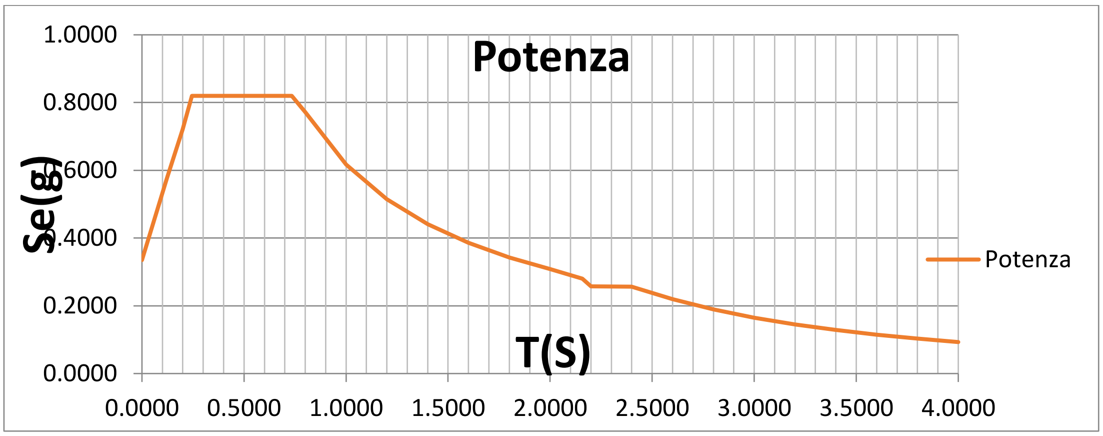


Figure 3.18. Horizontal elastic response spectrum

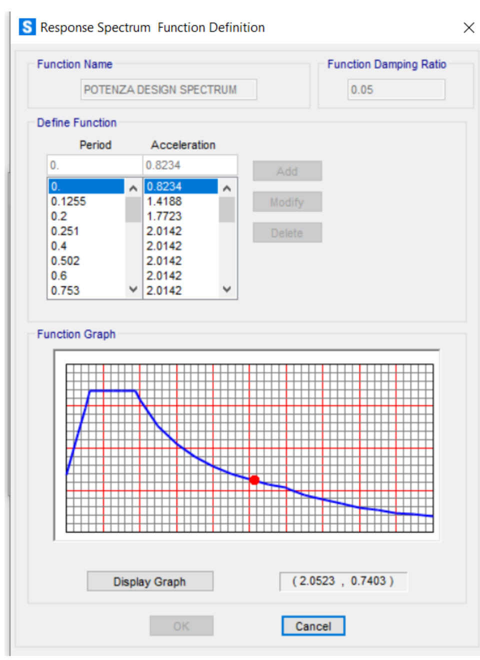


Figure 3.19. Design response spectrum of earthquake motion inputted in SAP2000.

3.3.3.2. Seismic action combination.

For the seismic action, 8 combinations were done as shown in table 3.9 and used to evaluate the structure's response at ULS and SLS.

Table 3.9. Combination of seismic forces in x and y directions

Name	Combination
ALS 1	E_x+0.3 E_y
ALS 2	0.3E_x+E_y
ALS 3	E_x-0.3 E_y
ALS 4	-0.3 E_x+E_y
ALS 5	-E_x+0.3 E_y
ALS 6	0.3E_x-E_y
ALS 7	-E_x+0.3 E_y
ALS 8	-0.3 E_x-E_y

3.3.3.3. Solicitations in frames

Upon addition of the response spectrum of the earthquake, the new solicitation in the frame elements got are shown in table 3.10. The maximum value for each solicitation was taken and used to redesign the elements.

Table 3.10. Solicitations in frame elements

Element	Solicitations		Units	Reinforcement
BEAM	Med+	220	KNm	4 ϕ 16
	Med-	160	KNm	4 ϕ 20
	Ved	145	KN	stirrup 8 with 10 cm spacing at support, and 15 cm at midspan section
Column	M3ed	100	KNm	
	M2ed	140	KNm	
	Ved	150	KN	stirrup 8 with 20 cm spacing
	P	2204	KN	12 ϕ 20

Where:

M is the moment

V is the shear force

P is the axial force.

The resulting detailed cross and transverse section of the elements are presented in appendix 4.

3.3.3.4. Design of shear wall

According to section 2.7.5, the shear wall was designed for its reinforcement and also verified that $M_{RD} > M_{ED}$. The solicitation used for design of shear wall was got after application of seismic forces.

a. Preliminary design

The shear wall thickness was determined using the following formula:

$$\begin{cases} t \geq 15cm \\ t \geq \frac{h}{20} \end{cases}$$ Choose t= 30 cm.

b. Reinforcement

The reinforcement in the vertical, horizontal and transverse directions got is presented in the table 3.11.

Table 3.11. Reinforcement design of shear wall

SHEAR WALL			
Geometry	b	1000	mm
	h	300	mm
	Ac	300000	mm²
Vertical	Asmin	600	mm²
	Asmax	12000	mm²
	reinforcement chosen	18ϕ25	
	Spacing	10	cm
Horizontal	Asmin	150	mm²
	Asmax	3000	mm²
	reinforcement chosen	ϕ12	
	Spacing	13	cm
Transverse	No transverse reinforcement		
MEd	x	52.7	KN. m/m

	y	198.6	KN. m/m
MRd	x	73.6	KN. m/m
	y	275	KN. m/m

3.3.3.5. Response of the structure

The structure's response to the dynamic effect of the seismic forces is reflected in the base shear, displacement of nodes which is further seen as the inter-story drift. These models with openings were kept as practical as possible (door and window openings) and were grouped into 3 sets: model 1-6, model 7 and model 8A-8I, described below with their results.

a. Model 1-6

In the first set of models, 6 models were used comparing centric vs eccentric openings, effects of staggering openings, and the use of many smaller openings over larger ones. Here, the window openings have a ratio of a/b <0.5 (a=height and width of opening, b=height and width of the wall), and door openings are of area less than 65% of RC wall. The models are described as:

- Model 1, no holes: A model with shear wall having no opening on its surface used as reference model.
- Model 2, centric door openings: Door openings of 90 × 220 (cm), placed at the centre of the shear wall in a repetitive manner from bottom to top of building.
- Model 3, eccentric door openings: Door openings of 90 × 220 (cm), placed close to the left boundary of the shear wall in a regular manner from bottom to the top of building.
- Model 4, many smaller window openings: In this model, two windows of 1.2 × 0.5 (m), are placed at each storey in a regular manner all along the height of the building.
- Model 5, regular centric window openings: A regular pattern of windows 1.2 × 1.0(m) is placed at each storey all along the building's height.
- Model 6, staggered window openings: Windows of 1.2 × 1.0(m) are placed in a staggered manner all along the building's height.

The figure 3.20 shows the dimensions of these openings, represented on the RC wall at each storey. For each case, the results of the dynamic effect are presented and discussed.

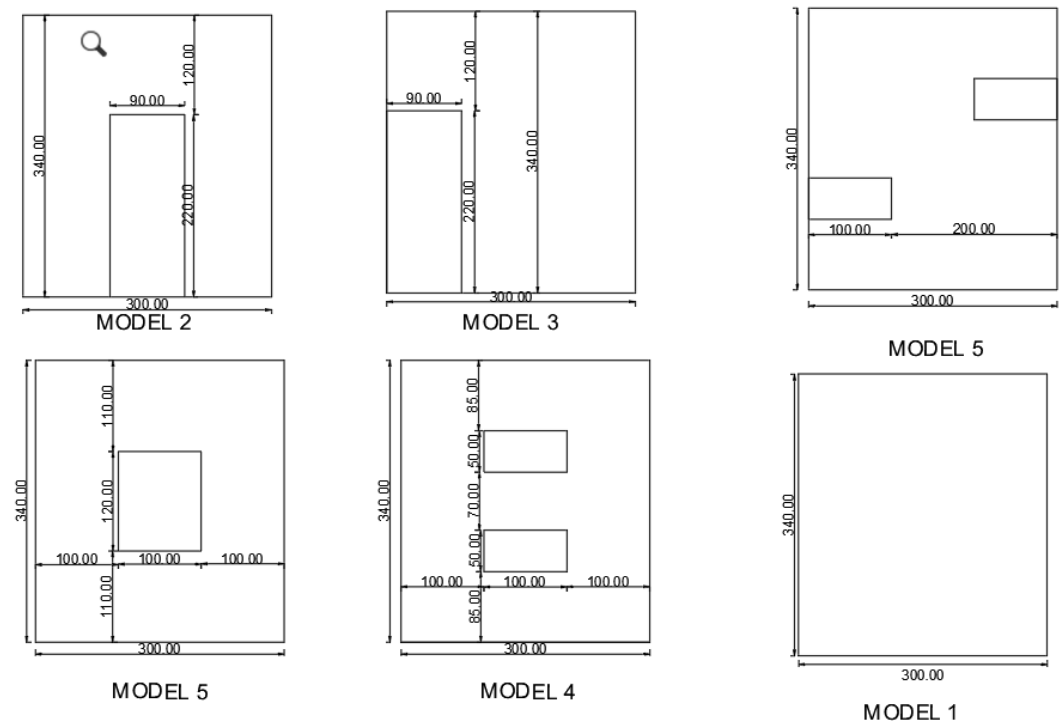


Figure 3.20. Set 1 models

i. Base shear of the structure

With reference to section of methodology 2.8.3.2.a, the base shear of the 6 models is presented in table 3.12. for the maximum seismic load combination.

Table 3.12. Base shear of the various models

Case	Base shear x	Base shear y	d (base shear x)	d (base shear y)
No holes	14,148.702	14,315.615	0	0
Centric Door openings	13,974.576	14,294.529	-174,126	-21,086
Eccentric door openings	14,006.888	14,318.611	-141,814	2,996
Many smaller window openings	14,034.065	14,298.389	-114,637	-17,226
Regular window openings	14,050.929	14,299.493	-97,773	-16,122
Staggered window openings	14,044.16	14,301.044	-104,542	-14,571

The results are illustrated in figure 3.21.

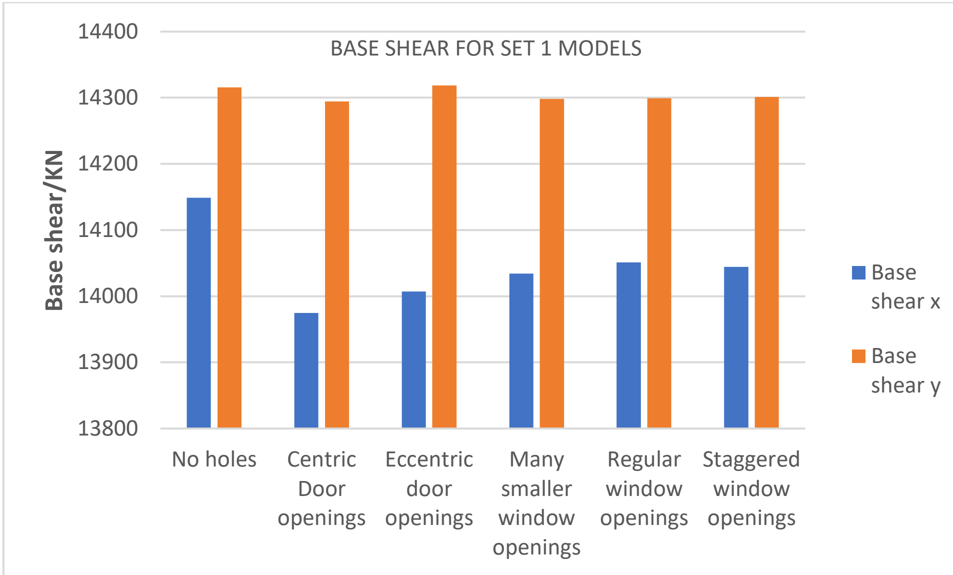


Figure 3.21. Base shear for each set 1 models in x and y direction.

The base shear represents the sum of all lateral forces acting at each storey of the building. According to the results, when buildings of equal mass are considered, centric door openings result in lower base shear than eccentric door openings. Using many smaller openings instead of one large opening result in a lower base shear on the structure and staggered window openings results in lower base shear than regular smaller openings. This implies that using smaller openings rather than a single large opening reduces the lateral force acting on the building by 0.11%, whereas placing door openings with an eccentric distance on the RC wall increases the lateral force the structure receives during a seismic event by 0.3%.

ii. Modal properties of the structure

The period and frequency of each model is presented in table 3.13 and graphically represented in figure 3.22.

Table 3.13. Periods and frequency of the different models

MODEL	MODE	PERIOD	FREQUENCY	Δperiod	Δfrequency
NO HOLES	mode 1	1.146575	0.87216255	0	0
	mode 2	1.134904	0.88113216	0	0
	mode 3	0.861529	1.16072769	0	0
CENTRIC DOOR	mode 1	1.153568	0.86687581	0.006993	-0.0052867
	mode 2	1.134234	0.88165203	-0.00067	0.00051987
	mode 3	0.86555	1.15533542	0.004021	-0.0053923
ECCENTRIC DOOR	mode 1	1.153891	0.86663269	0.007316	-0.0055299
	mode 2	1.134289	0.88160955	-0.00061	0.00047739
	mode 3	0.865674	1.15516949	0.004145	-0.0055582
MANY SMALL OPENINGS	mode 1	1.151334	0.8685577	0.004759	-0.0036049
	mode 2	1.134739	0.88126016	-0.00016	0.000128
	mode 3	0.864303	1.1570011	0.002774	-0.0037266
REGULAR CENTRIC WINDOWS	mode 1	1.150512	0.86917849	0.003937	-0.0029841
	mode 2	1.13466	0.88132113	-0.00024	0.00018897
	mode 3	0.863846	1.1576142	0.002317	-0.0031135
STAGGERED WINDOWS	mode 1	1.151354	0.86854241	0.004779	-0.0036201
	mode 2	1.134979	0.88107378	7.5E-05	-5.839E-05
	mode 3	0.864201	1.15713852	0.002672	-0.0035892

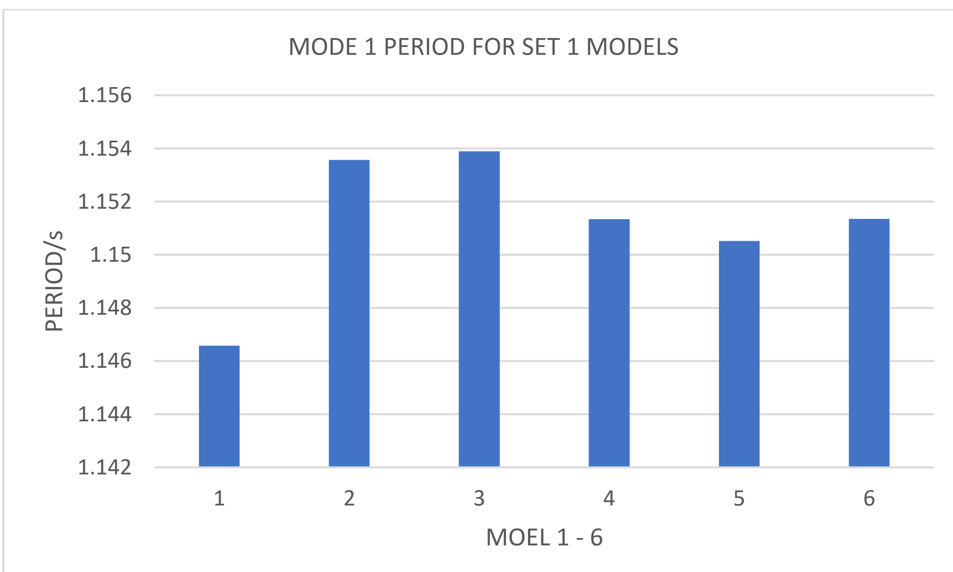


Figure 3.22. Natural period for modes of vibration of the different models.

Moving from mode 1 to mode 3 results in a decrease in building's period for all six models. When eccentric versus centric door opening locations on an RC wall are compared, eccentric openings have a greater period in its first mode – fundamental mode – than centric openings, implying that the building has greater stiffness and thus is more rigid when openings are placed at a centric distance than when openings are placed at the periphery. In addition, when comparing the results of models 4 and 5, a reinforced concrete wall with many small openings has a lower stiffness than one large opening. Staggering window openings result in a greater period than regular centric openings, implying smaller stiffness and rigidity in the event of a seismic event.

iii. Displacement in x-direction

By identifying specific consecutive nodes at each storey, the displacement of the different stories in the x-direction (u1) for each model is got. The results are presented in table 3.14.

Table 3.14. Displacement (mm) in the x-direction of the different models

DISPLACEMENT IN X-DIRECTION/mm						
	MODEL TYPE					
STOREY HEIGHT	No holes	Centric door	Eccentric door opening	Many small openings	Regular centric openings	Staggered windows
0	0.349	0.355	0.353	0.353	0.353	0.353
3.4	2.745	2.817	2.821	2.806	2.798	2.771
6.8	6.02	6.267	6.253	6.197	6.169	6.133
10.2	9.558	9.938	9.924	9.818	9.775	9.758
13.6	13.08	13.54	13.536	13.39	13.338	13.35
17	16.394	16.904	16.904	16.738	16.68	16.715
20.4	19.372	19.912	19.903	19.738	19.677	19.722
23.8	21.92	22.46	22.434	22.291	22.231	22.273
27.2	24.01	24.496	24.455	24.351	24.297	24.33
30.6	25.726	26.11	26.067	25.996	25.954	25.982

A graphical illustration of comparison curves is shown in figure 3.23.

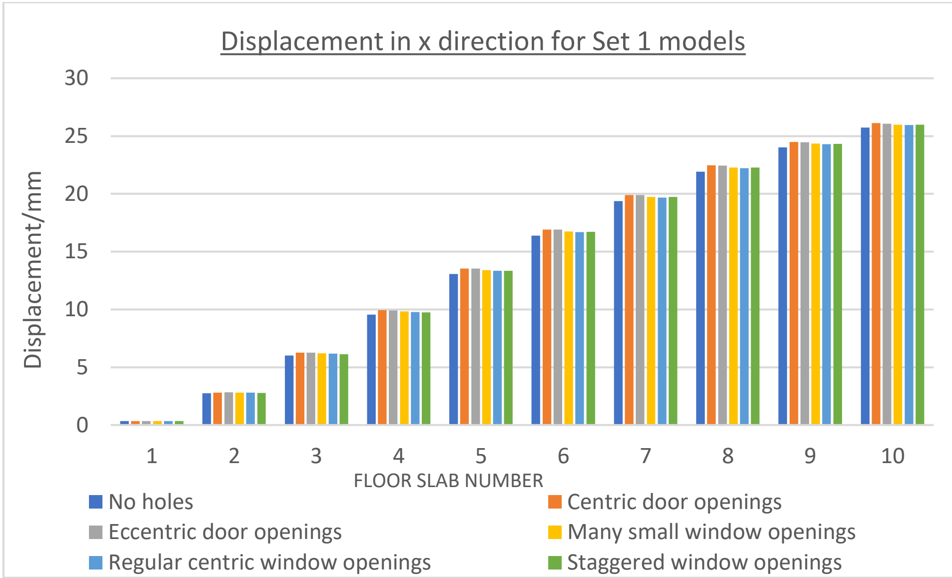


Figure 3.23. Displacement in x-direction for different models

An RC wall with no openings presents the least displacement results. Comparing a wall with centric openings to that with eccentric, displacement in x direction is least with eccentric openings. In the case of window openings, regular centric window openings present least displacement, many smaller window openings result in greater displacement in x direction than regular centric window openings at each storey. Staggered window openings give a lesser displacement in x direction than regular smaller openings.

iv. Displacement in y-direction

By identifying specific consecutive nodes at each storey, the displacement of the different stories in the y-direction (u2) for each model is got. The results are presented in table 3.15.

Table 3.15. Displacement (m) in y-direction of the different models

DISPLACEMENT IN Y-DIRECTION/mm						
	MODEL TYPE					
STOREY HEIGHT	No holes	Centric door	Eccentric door opening	Many small openings	Regular centric openings	Staggered windows
0	0.352	0.356	0.355	0.355	0.354	0.354
3.4	2.73	2.758	2.754	2.748	2.745	2.747
6.8	6.052	6.116	6.106	6.093	6.086	6.088
10.2	9.572	9.667	9.655	9.629	9.619	9.622
13.6	12.998	13.115	13.107	13.066	13.054	13.057
17	16.165	16.298	16.295	16.238	16.225	16.227
20.4	18.965	19.111	19.114	19.042	19.028	19.026
23.8	21.326	21.479	21.488	21.402	21.389	21.38
27.2	23.197	23.351	23.366	23.267	23.255	23.238
30.6	24.661	24.807	24.829	24.719	24.709	24.683

A graphical illustration is shown in figure 3.24.

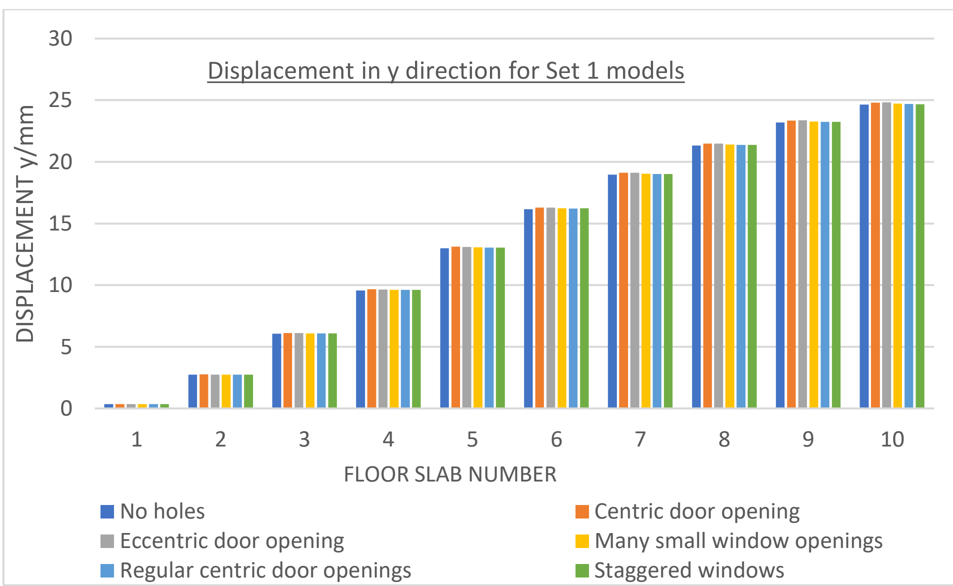


Figure 3.24.Displacement in y-direction for different models

There is very little difference in their displacement in y direction. An RC wall with no openings presents the least displacement results. Comparing a wall with centric openings to that with eccentric, displacement in y direction is least with eccentric openings. In the case of window openings, staggered window openings present least displacement, many smaller window openings result in greater displacement in y direction than regular centric windows at each storey.

v. Inter-storey drift in the x-direction

The inter-storey drift is defined as the difference between the lateral displacements of two adjacent stories divided by the height of that storey. Table 3.16 below presents the maximum inter-story drift for the different models in the x-direction. It is verified for each case with the limiting value, a reduction factor of v=0.5, importance class II.

Table 3.16. Inter storey drifts in x-direction of the different models.

INTERSTORY DRIFT IN X-DIRECTION						
	MODEL TYPE					
STOREY HEIGHT	No holes	Centric door	Eccentric door opening	Many small openings	Regular centric openings	Staggered windows
0	0	0	0	0	0	0
3.4	0.000705	0.00072412	0.000725882	0.000721471	0.000719118	0.000711
6.8	0.000963	0.00101471	0.001009412	0.000997353	0.000991471	0.000989
10.2	0.001041	0.00107971	0.001079706	0.001065	0.001060588	0.001066
13.6	0.001036	0.00105941	0.001062353	0.001050588	0.001047941	0.001056
17	0.000975	0.00098941	0.000990588	0.000984706	0.000982941	0.00099
20.4	0.000876	0.00088471	0.000882059	0.000882353	0.000881471	0.000884
23.8	0.000749	0.00074941	0.000744412	0.000750882	0.000751176	0.00075
27.2	0.000615	0.00059882	0.000594412	0.000605882	0.000607647	0.000605
30.6	0.000505	0.00047471	0.000474118	0.000483824	0.000487353	0.000486

A graphical presentation of comparison curve is shown in figure 3.25 below.

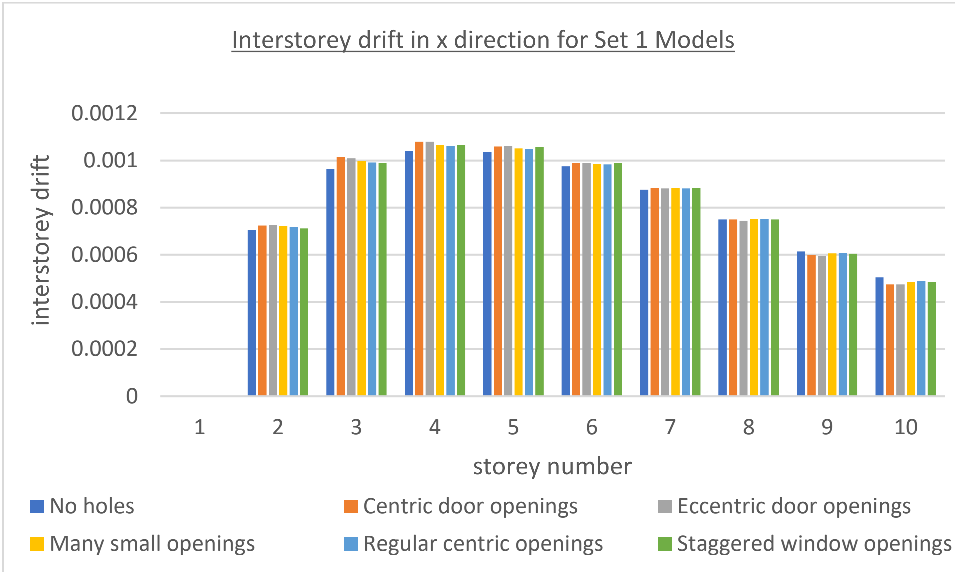


Figure 3.25. Comparison of curves for storey drifts and storey height in x-direction for different models

An RC wall with no openings produces the least inter-storey drift at each level. When a wall with centric openings is compared to one with eccentric openings, the inter-storey drift in the x-direction is the lesser with eccentric openings. When it comes to window openings, staggered window openings have the greater inter-storey drift compared to many small openings placed regularly (Model 4). Regular centric window openings at each storey presents with the least inter storey drift.

vi. Inter-storey drift in the y-direction

Table 3.17 presents the maximum inter-story drift for the different models in the y-direction. It is verified for each case with the limiting value, with a reduction factor of v=0.5, importance class II.

Table 3.17. Storey drifts in y-direction of different models

INTERSTORY DRIFT IN Y-DIRECTION						
	MODEL TYPE					
STOREY HEIGHT	No holes	Centric door	Eccentric door opening	Many small openings	Regular centric openings	Staggered windows
0	0	0	0	0	0	0
3.4	0.000699	0.000706471	0.000705588	0.0007038	0.000703235	0.00070382
6.8	0.000977	0.000987647	0.000985882	0.0009838	0.000982647	0.00098265
10.2	0.001035	0.001044412	0.001043824	0.00104	0.001039118	0.00103941
13.6	0.001008	0.001014118	0.001015294	0.0010109	0.001010294	0.00101029
17	0.000931	0.000936176	0.000937647	0.0009329	0.000932647	0.00093235
20.4	0.000824	0.000827353	0.000829118	0.0008247	0.000824412	0.00082324
23.8	0.000694	0.000696471	0.000698235	0.0006941	0.000694412	0.00069235
27.2	0.00055	0.000550588	0.000552353	0.0005485	0.000548824	0.00054647
30.6	0.000431	0.000428235	0.000430294	0.0004271	0.000427647	0.000425

A graphical presentation of comparison curve is shown in figure 3.26.

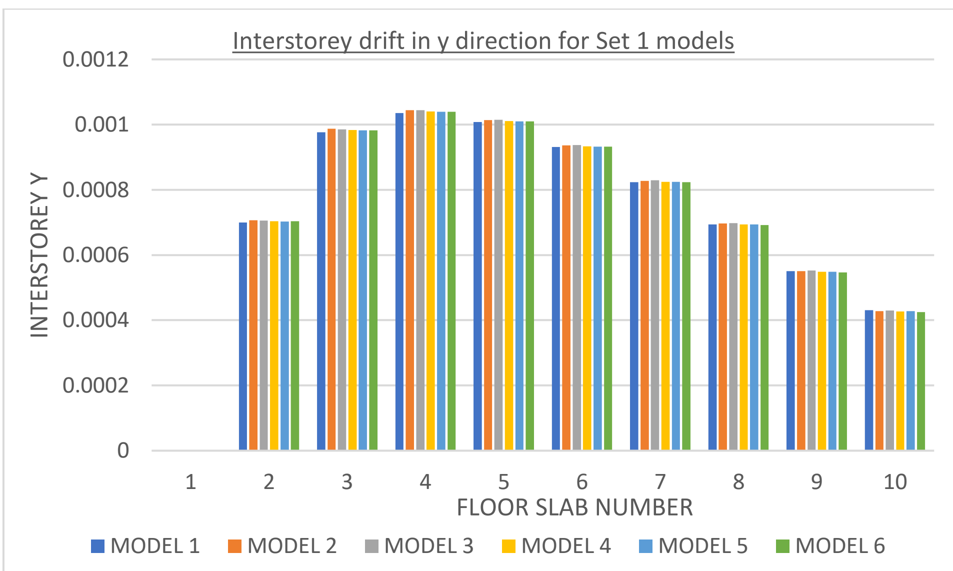


Figure 3.26. Comparison of curves for storey drifts and storey height in y-direction for the different models.

There is very little difference in their displacement in y direction. An RC wall with no openings produces the least inter-storey drift at each level. When a wall with centric openings is compared to one with eccentric openings, the inter-storey drift in the y direction is the least with centric openings. When it comes to window openings, staggered window openings have the least inter-storey drift especially for higher floors, whereas many smaller window openings have more inter-storey drift in the y direction than regular centric window openings at each storey.

b. Model 7

In this set, models used had door openings of area greater than 65% of RC wall area per storey, for orientation check. It consists of two models; 7A for a 2.4 × 3.06(m) centric opening and 7B for a 2.7 × 2.72(m) centric opening as shown in figure 3.27. These were used to compare the orientation with greater displacement effect. For each case, the results of the dynamic response are presented and discussed.

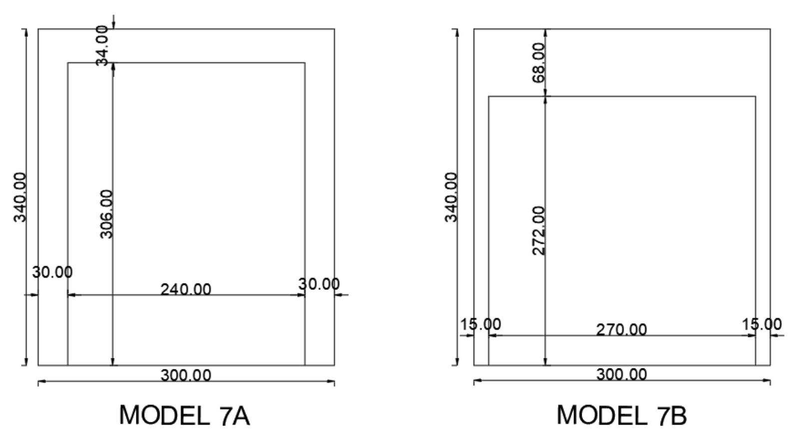


Figure 3.27. Dimension of openings in model 7

i. Base shear of the structure

With reference to section of methodology 2.8.3.2.a, the base shear of the two models is presented in table 3.18 for the maximum seismic load combination.

Table 3.18. Base shear of model 7

Case	Base shear x	Base shear y	d (base shear x)	d (base shear y)
MODEL 7A	12,902.351	14,235.173	-146,845	5,625
MODEL 7B	13,049.196	14,229.548		

The results present that, openings oriented larger in width than height, model 7B, result in a larger value of base shear.

ii. Displacement in x-direction

By identifying specific consecutive nodes at each storey, the displacement of the different stories in the x-direction (u1) for each model was got. The results are presented in table 3.19.

Table 3.19. Displacement (mm) in x-direction of different models

DISPLACEMENT X-DIRECTION/mm		
storey	MODEL 7A	MODEL 7B
0	0.372	0.369
3.4	3.321	3.263
6.8	7.554	7.393
10.2	11.959	11.688
13.6	16.147	15.779
17	19.932	19.489
20.4	23.194	22.699
23.8	25.809	25.288
27.2	27.703	27.18
30.6	29.031	28.523

An illustration of the displacement is presented in the figure 3.28.

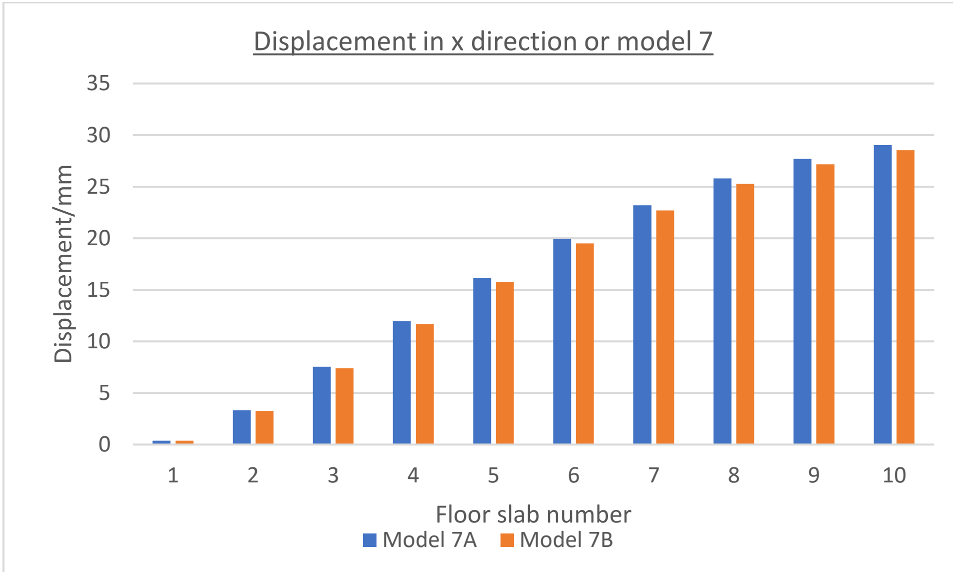


Figure 3.28. Displacement in X-direction for different models

Comparing both models, displacement in x direction is least with centric openings 2.7 × 2.72(m) at each storey.

iii. Displacement in y-direction

By identifying specific consecutive nodes at each storey, the displacement of the different stories in the y-direction (u2) for each model is got. The results are presented in table 3.20.

Table 3.20. Displacement (mm) in y-direction of different models

DISPLACEMENT Y-DIRECTION/mm		
storey	MODEL 7A	MODEL 7B
0	0.376	0.373
3.4	2.915	2.894
6.8	6.47	6.418
10.2	10.219	10.132
13.6	13.844	13.723
17	17.174	17.021
20.4	20.102	19.921
23.8	22.545	22.342
27.2	24.451	24.231
30.6	25.913	25.68

Graphically presented in the figure 3.29.

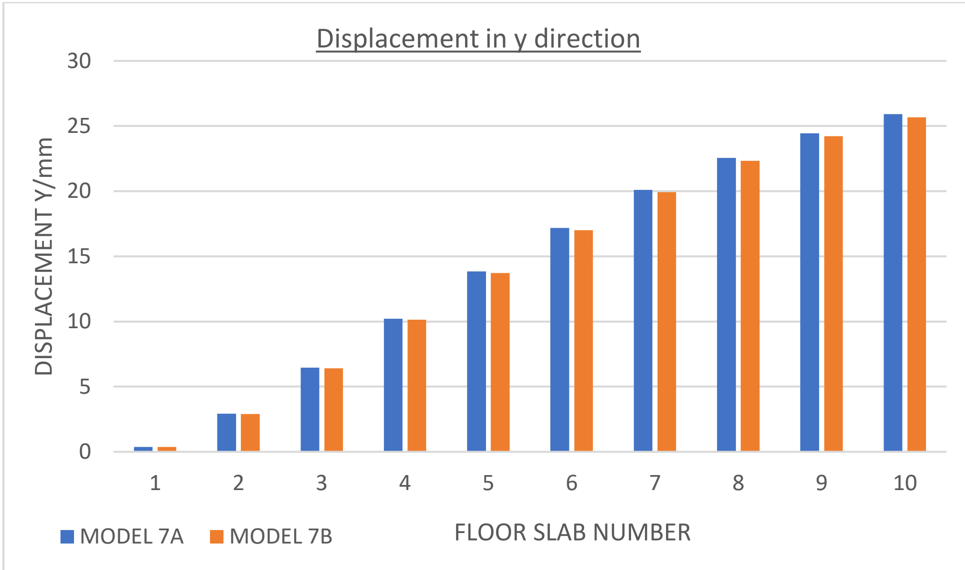


Figure 3.29. Displacement in y-direction for different models

Comparing both models, displacement in y direction is least with centric openings 2.7 × 2.72(m) at each storey.

iv. Inter-storey drift in the x-direction

The inter-storey drift is defined as the difference between the lateral deflections of two adjacent stories divided by the height of that storey. Table 3.21 presents the maximum inter-story drift for the different models in the x-direction. It is verified for each case with the limiting value, a reduction factor of v=0.5, importance class II.

Table 3.21. Inter storey drifts in x-direction for model 7.

INTESTOREY X-DIRECTION		
storey	MODEL 7A	MODEL 7B
0	0	0
3.4	0.000867353	0.000851
6.8	0.001245	0.001215
10.2	0.001295588	0.001263
13.6	0.001231765	0.001203
17	0.001113235	0.001091
20.4	0.000959412	0.000944
23.8	0.000769118	0.000761
27.2	0.000557059	0.000556
30.6	0.000390588	0.000395

Graphically presented in the figure 3.30.

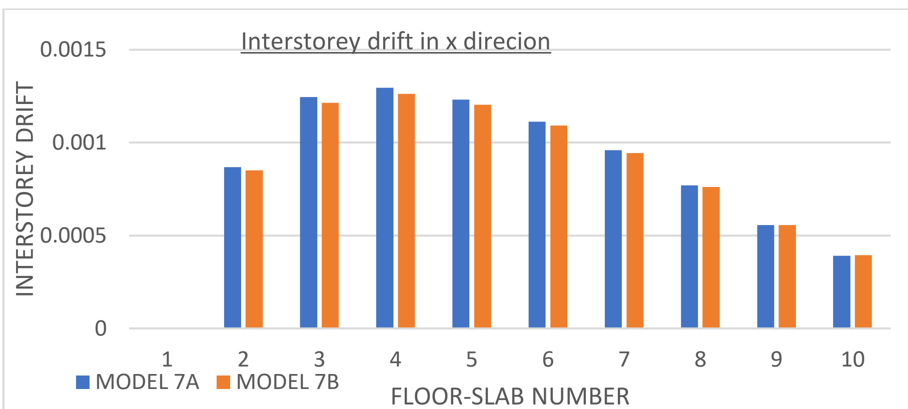


Figure 3.30. Comparison curves for inter storey drifts for the different models.

Comparing both models, inter story drift in x-direction is least with centric openings 2.7 × 2.72(m) at each storey, averagely 3.11% less than that of openings 2.4 x 3.06(m).

v. Inter-storey drift in the y-direction

Table 3.22 presents the maximum inter-story drift for the different models in the y-direction. It is verified for each case with the limiting value, a reduction factor of v=0.5, importance class II. Graphically presented in figure 3.31.

Table 3.22. Inter storey drifts in y-direction for model 7.

INTER-STOREY Y-DIRECTION		
storey	MODEL 7A	MODEL 7B
0	0	0
3.4	0.000746765	0.000741
6.8	0.001045588	0.001036
10.2	0.001102647	0.001092
13.6	0.001066176	0.001056
17	0.000979412	0.00097
20.4	0.000861176	0.000853
23.8	0.000718529	0.000712
27.2	0.000560588	0.000556
30.6	0.00043	0.000426

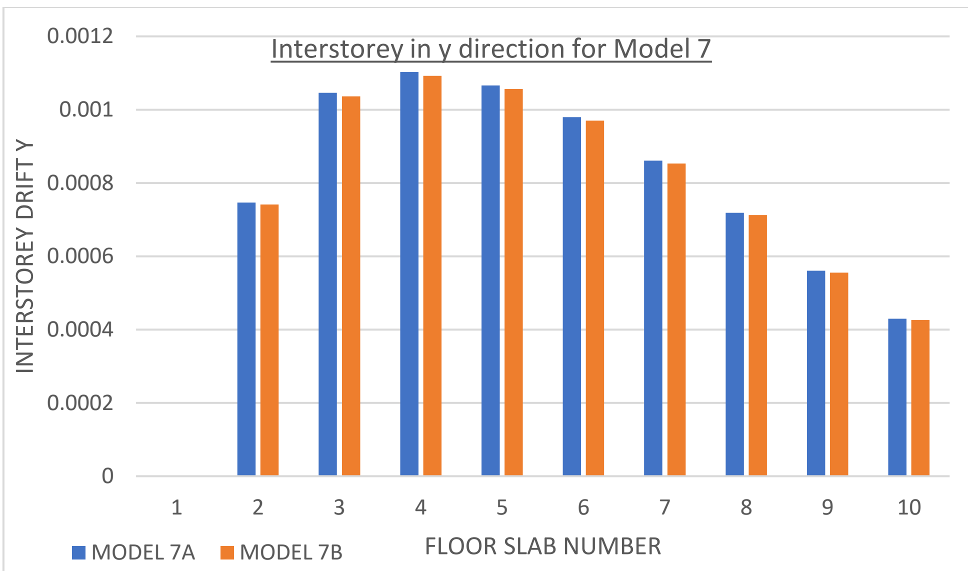


Figure 3.31. Comparison curves for inter storey drifts for different models in y direction.

Comparing both models, inter story drift in y direction is least with centric openings 2.7 × 2.72 at each storey.

c. Model 8

In this set, models had a door opening of area greater than 65% of RC wall area which varied per storey. This model is divided into nine sub-models, numbered A to I. For each model, a centric door opening 3.06 × 2.4(m), the worst case of model 7, was placed on each floor and varied to compare response and determine which sub-model has the greatest response. Centric door openings of 90 × 220(cm) were placed at the rest of the stories. For each case, the results of the dynamic response are presented and discussed.

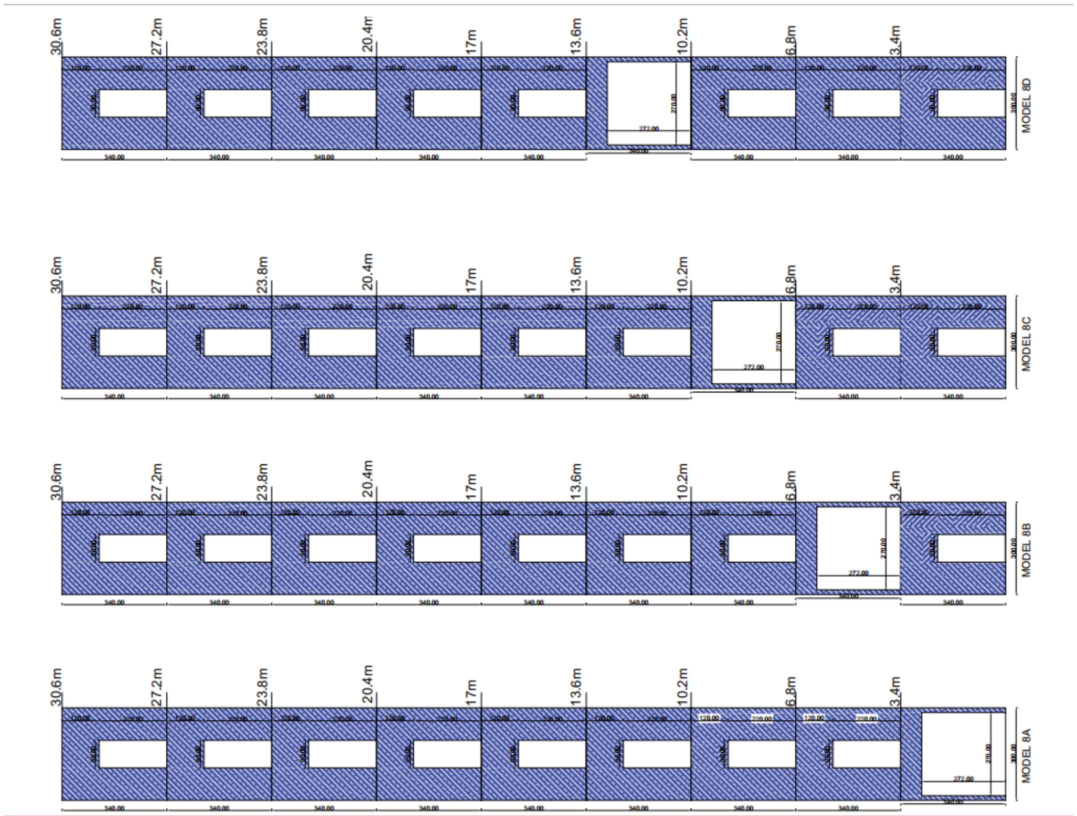


Figure 3.32. Set 2 models used for analysis and verification of buildings behaviour

i. Base shear of the structure

With reference to section of methodology 2.8.3.2.a, the base shear values in KN of the 9 models are presented in table 3.23. for the maximum seismic load combination and illustrated in Figure 3.33.

Table 3.23. Base shear for model 8

Case	Base shear x/KN	Base shear y/KN	d (base shear x)	d (base shear y)
MODEL 8A	13,936.835	14,282.012	0	0
MODEL 8B	13,886.072	14,279.647	-50,763	-2,365
MODEL 8C	13,862.668	14,280.005	-74,167	-2,007
MODEL 8D	13,916.524	14,282.629	-20,311	0.617
MODEL 8E	13,911.515	14,285.174	-25.32	3,162
MODEL 8F	13,935.56	14,288.142	-1,275	6.13
MODEL 8G	13,919.362	14,290.435	-17,473	8,423
MODEL 8H	13,961.177	14,293.17	24,342	11,158
MODEL 8I	13,943.936	14,295.445	7,101	13,433

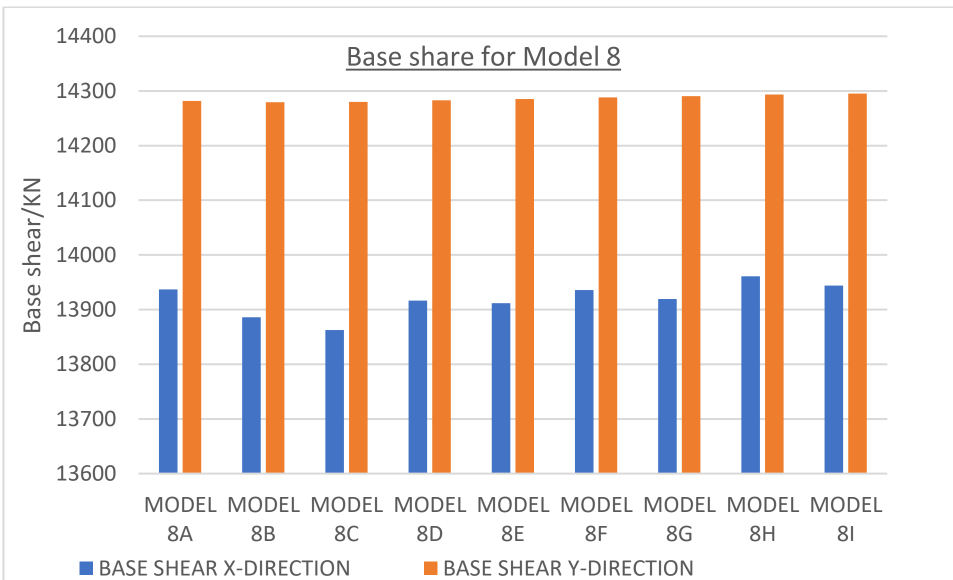


Figure 3.33 . Graphical illustration of base shear for model 8

According to the results, model 8C has the lowest base shear in the x direction and model 8H has the highest. The difference in base shear in the y direction is very minimal for the different models.

ii. Displacement in x-direction

By identifying specific consecutive nodes at each storey, the displacement of the different stories in the x-direction (u1) for each model is got. The results are presented in table 3.24 below and illustrated in figure 3.34.

Table 3.24. Displacement (mm) in x-direction of different models

DISPLACEMENT IN X-DIRECTION/mm									
	MODEL TYPE								
STOREY HEIGHT	8A	8B	8C	8D	8E	8F	8G	8H	8I
0	0.357	0.362	0.356	0.356	0.355	0.355	0.354	0.354	0.354
3.4	3.109	2.849	2.792	2.801	2.808	2.811	2.812	2.813	2.814
6.8	6.593	6.723	6.254	6.225	6.244	6.253	6.258	6.26	6.262
10.2	10.255	10.434	10.265	9.902	9.894	9.912	9.923	9.928	9.933
13.6	13.852	14.017	13.89	13.767	13.499	13.501	13.52	13.53	13.539
17	17.214	17.365	17.225	17.141	17.061	16.866	16.878	16.896	16.914
20.4	20.224	20.363	20.208	20.117	20.071	20.012	19.886	19.909	19.942
23.8	22.776	22.904	22.739	22.638	22.587	22.555	22.494	22.455	22.522
27.2	24.816	24.936	24.762	24.654	24.596	24.56	24.517	24.448	24.571
30.6	26.434	26.547	26.367	26.253	26.188	26.146	26.105	26.049	26.012

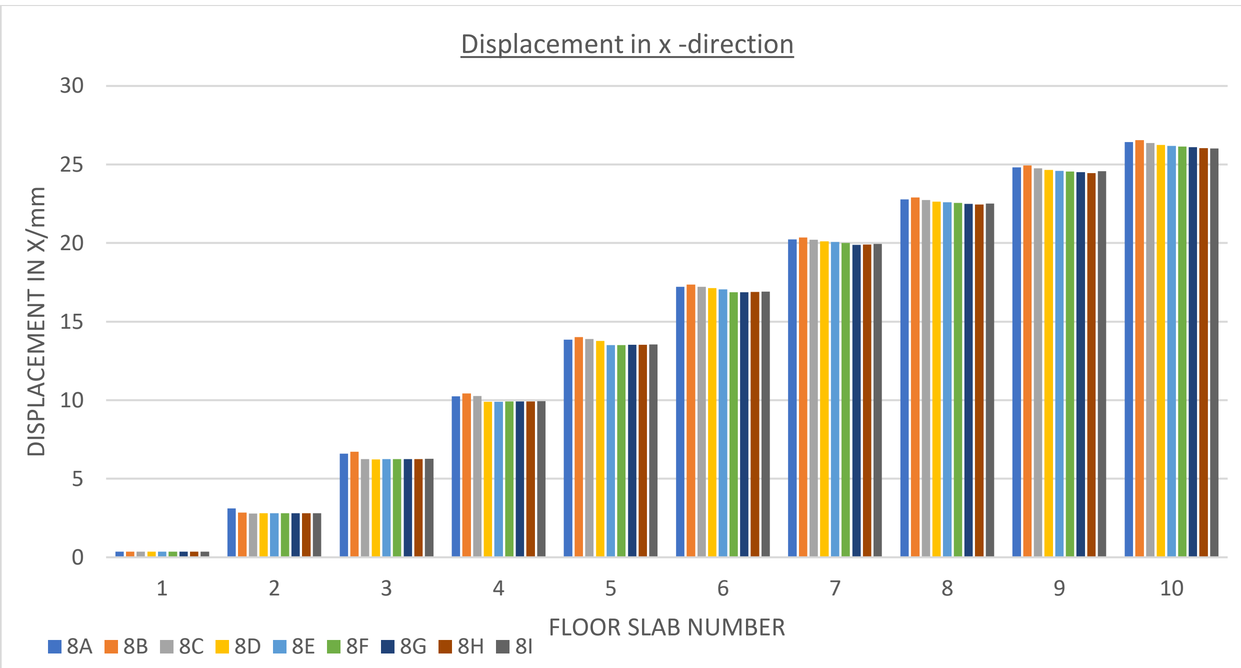


Figure 3.34. Comparism chart of displacement in x direction for different models.

It is observed that model 8B has the greatest displacement at each storey in the x direction followed by model 8A and then 8C, while model 8I has the least displacement in x.

iii. Displacement in y-direction

By identifying specific consecutive nodes at each storey, the displacement of the different stories in the y-direction (u2) for each model is got. The results are presented in table 3.25 and graphically presented in figure 3.35.

Table 3.25. Displacement (mm) in y-direction of different models

	DISPLACEMENT IN Y-DIRECTION/m								
	MODEL TYPE								
STOREY HEI	8A	8B	8C	8D	8E	8F	8G	8H	8I
0	0.358	0.358	0.358	0.357	0.357	0.357	0.356	0.356	0.356
3.4	2.803	2.772	2.763	2.761	2.761	2.76	2.759	2.758	2.759
6.8	6.169	6.19	6.13	6.122	6.121	6.121	6.119	6.116	6.119
10.2	9.719	9.752	9.727	9.679	9.674	9.673	9.67	9.666	9.671
13.6	13.166	13.203	13.182	13.161	13.127	13.122	13.117	13.114	13.122
17	16.348	16.388	16.364	16.347	16.333	16.306	16.299	16.297	16.308
20.4	19.16	19.204	19.176	19.158	19.147	19.134	19.112	19.11	19.125
23.8	21.527	21.573	21.543	21.522	21.511	21.501	21.487	21.478	21.497
27.2	23.396	23.444	23.412	23.39	23.377	23.369	23.357	23.345	23.37
30.6	24.849	24.899	24.865	24.842	24.829	24.82	24.81	24.8	24.808

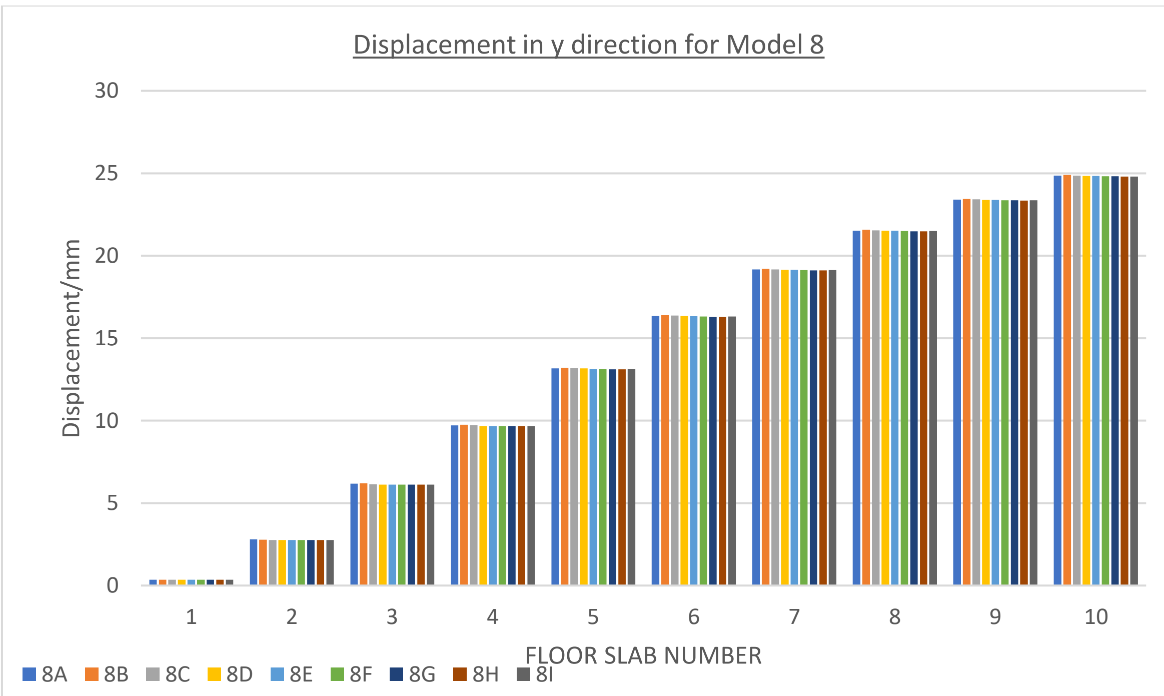


Figure 3.35. Comparism chart of displacement in y direction for different models.

It is observed that model 8B has the greatest displacement in the y direction at each storey, while model 8I has the least. It is also observed that there is not much difference in displacement n y direction between the nine models.

iv. Inter-storey drift in the x-direction

The inter-storey drift is defined as the difference between the lateral deflections of two adjacent stories divided by the height of that storey. Table 3.26 presents the maximum inter-story drift for the different models in the x-direction. Graphically presented in figure 3.36. It is verified for each case with the limiting value, a reduction factor of v=0.5, importance class II.

Table 3.26. Inter storey in x-direction of the different models

	INTERSTOREY DRIFT IN X								
STOREY	8A	8B	8C	8D	8E	8F	8G	8H	8I
0	0	0	0	0	0	0	0	0	0
3.4	0.000809412	0.000731471	0.00071647	0.00071912	0.000721471	0.00072235	0.00072294	0.00072324	0.00072353
6.8	0.001024706	0.001139412	0.00101824	0.00100706	0.001010588	0.00101235	0.00101353	0.00101382	0.00101412
10.2	0.001077059	0.001091471	0.00117971	0.00108147	0.001073529	0.00107618	0.00107794	0.00107882	0.00107971
13.6	0.001057941	0.001053824	0.00106618	0.00113676	0.001060294	0.00105559	0.00105794	0.00105941	0.00106059
17	0.000988824	0.000984706	0.00098088	0.00099235	0.001047647	0.00098971	0.00098765	0.00099	0.00099265
20.4	0.000885294	0.000881765	0.00087735	0.00087529	0.000885294	0.00092529	0.00088471	0.00088618	0.00089059
23.8	0.000750588	0.000747353	0.00074441	0.00074147	0.00074	0.00074794	0.00076706	0.00074882	0.00075882
27.2	0.0006	0.000597647	0.000595	0.00059294	0.000590882	0.00058971	0.000595	0.00058618	0.00060265
30.6	0.000475882	0.000473824	0.00047206	0.00047029	0.000468235	0.00046647	0.00046706	0.00047088	0.00042382

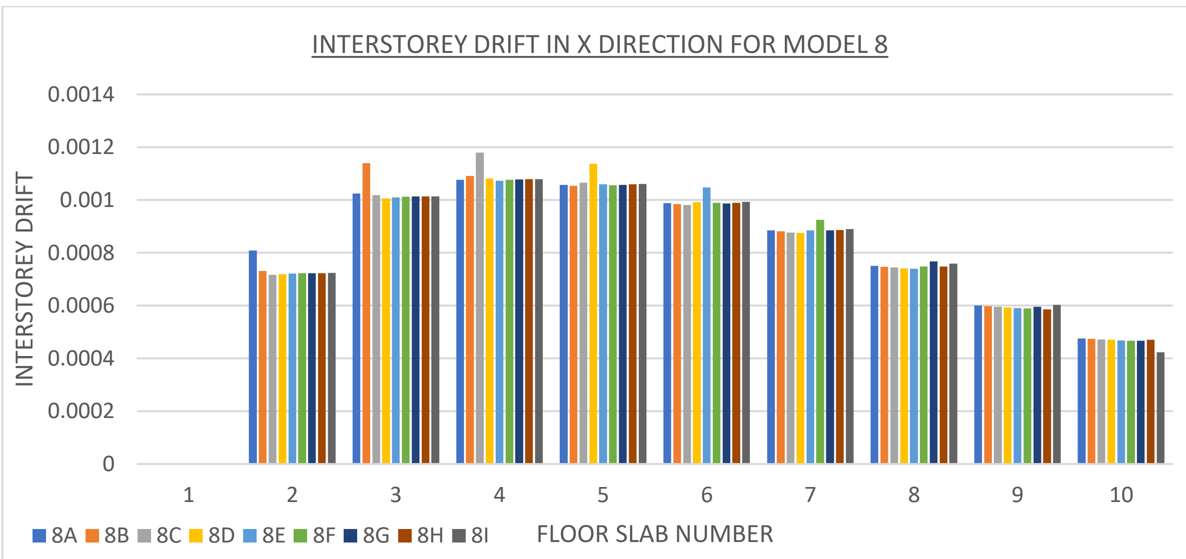


Figure 3.36. Comparism chart of inter-storey in x direction for the different models.

As can be seen, the inter storey drift in the x direction at each storey is greatest when the opening 3.06 × 2.4(m) is located at that storey. In general, the maximum inter storey drift is obtained when the opening is placed on storey 3, model 8C.

Conclusion

The main objective of this chapter was to present the case study, model, results of the analysis and design of structural elements, and to compare the seismic behaviour of the building, considering openings of different sizes and location. For the defined case study, vertical columns of 60 × 60 (cm) were obtained with beam elements of 30 × 60 (cm). The RC wall was designed with a thickness of 30 cm for a height of 340 cm per storey and a raft foundation of 100 cm thick adopted. The models described in section 3.3.3.5 were used to vary the size and location of openings on the RC wall, and the seismic response, mainly observed in its modal properties, base shear and inter storey drift, were presented. From a general point of view, openings on RC walls reduce their efficiency to resist lateral loads. Eccentric door openings gave higher values of base shear and natural period compared to centric door openings while giving a lesser displacement and inter storey drift in x direction than centric door openings. For window openings, regular centric window openings model experienced the greatest base shear and smaller openings the least. The natural period and inter storey drift are greatest for staggered windows and least for regular centric windows. For the model 7, model 7A show a greater inter storey drift than model 7B. In the models 8A to 8I, it was found that the base shear is greatest for 8H and least for 8C, and the inter storey drift greatest for the model 8C.

GENERAL CONCLUSION

The purpose of this work was to analyse and verify the effect of the size and position of openings on reinforced concrete walls on the behaviour of tall buildings. This study was done firstly through a literature review on tall buildings and reinforced concrete walls with openings. Secondly, the methodology of the analysis and design of structural elements was presented alongside the dynamic analysis procedure. Following this, a nine-storey building was modelled, modal analysis performed and a response spectrum analysis was carried out. This response spectrum analysis was carried out on the different defined models and a comparative study on their response to the seismic force was done. The analysis was performed using the software SAP 2000 (version 22). The results obtained from the analysis revealed that:

(1) Centric door openings result in lower base shear and vibration period than eccentric door openings. However, they presented greater lateral displacement and inter-storey drift than eccentric door opening model.

(2) For window openings, smaller window openings though having the least base shear, gave greater displacement, period and inter storey drift over one large window opening. Also, staggered windows presented a greater inter storey drift, base shear, period over regular smaller window openings but lower displacement values

(3) For the model 7, model 7B gave better results than model 7A. Thus, this work recommends that for openings greater than 65% of wall area, they produce a better behaviour when oriented larger in width than in height. This brings forward the aspect of load path when openings with greater height produce a great interruption to the load path-load transfer to ground mechanism.

(4) For the analysis on critical storey for large openings, greater displacements were got when openings were placed at the first 2 storey, storey 1 and 2. However, the greatest Inter storey drift occurs between the 3^{rd} and 4^{th} floors and when the large opening is placed a storey 3.

From the above results, upon recommending opening position and sizes, it was seen that lateral displacement alone is not enough to conclude on an opening's size and position especially when the displacement value is below limiting value. To add, a proportional relationship was observed between periods and inter storey drift, they followed the same pattern for all the models. However, the displacement did not follow the same pattern as the period as was expected to reflect the model's rigidity.

From these, it can be concluded that, openings on RC walls influence the seismic response of the building but the choice of the position and sizes of openings depends on the parameter the designer wants to control most: seismic force on building, period, displacement or inter storey drift. To the construction industry to whom this work most applies to, it is recommended that, complete analysis, design and verification are carried out as such since other building properties like aspect ratio, slenderness, building shape just to name a few could intrinsically affect the building's behaviour to lateral loading alongside the opening effects. This thus makes each building definite analysis and verification independent. The big question to guide this, as stated above, will be which building behaviour parameter do you want to control? Is it lateral displacement for fear of touching neighbouring structures or inter storey drift for fear of collapsing down?

Nonetheless, this research has shortcomings related to the non-consideration of elements material and geometry non-linearity and that it worked on a building regular in plan and elevation with squat shear walls only. To this effect, further research could be carried on the nonlinear analysis on tall building's behaviour with openings, a detailed study on the influence of openings on walls strength, load path interruption and a performance-based design approach for walls with openings to understand the collapse mechanism of the various model.

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APPENDIX

APPENDIX 1: Categories of use of the building (EC 1 Part 1) and the recommended values of factors for building (Table 1.1.1 of EN 1990)

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

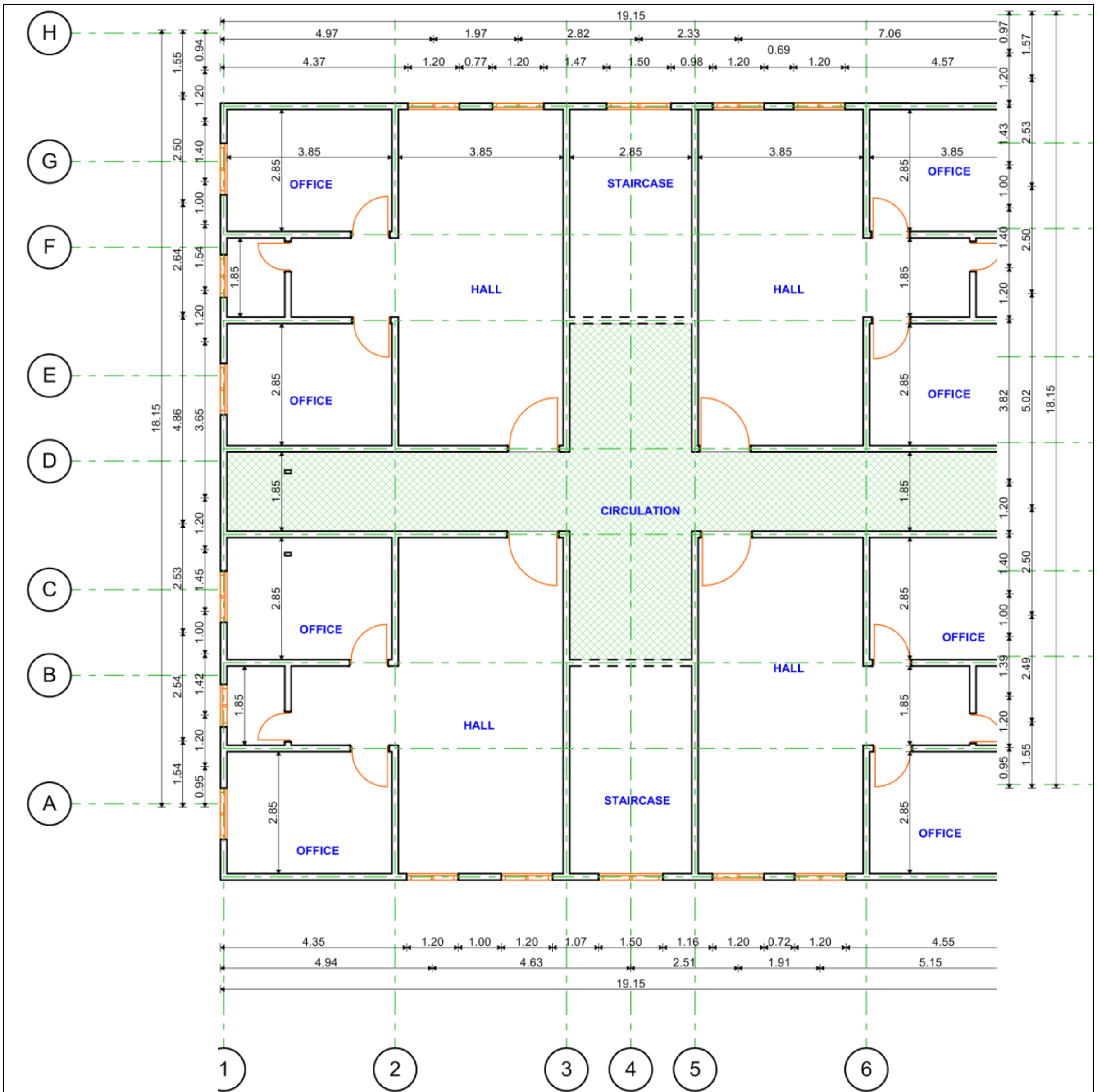
Action	ψ_0	ψ_1	ψ_2
Imposed loads in buildings, category (see EN 1991-1-1)			
Category A : domestic, residential areas	0,7	0,5	0,3
Category B : office areas	0,7	0,5	0,3
Category C : congregation areas	0,7	0,7	0,6
Category D : shopping areas	0,7	0,7	0,6
Category E : storage areas	1,0	0,9	0,8
Category F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

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

APPENDIX 2: Importance coefficient

building class	importance coefficient γ_1
I	0.8
II	1
III	1.2
IV	1.4

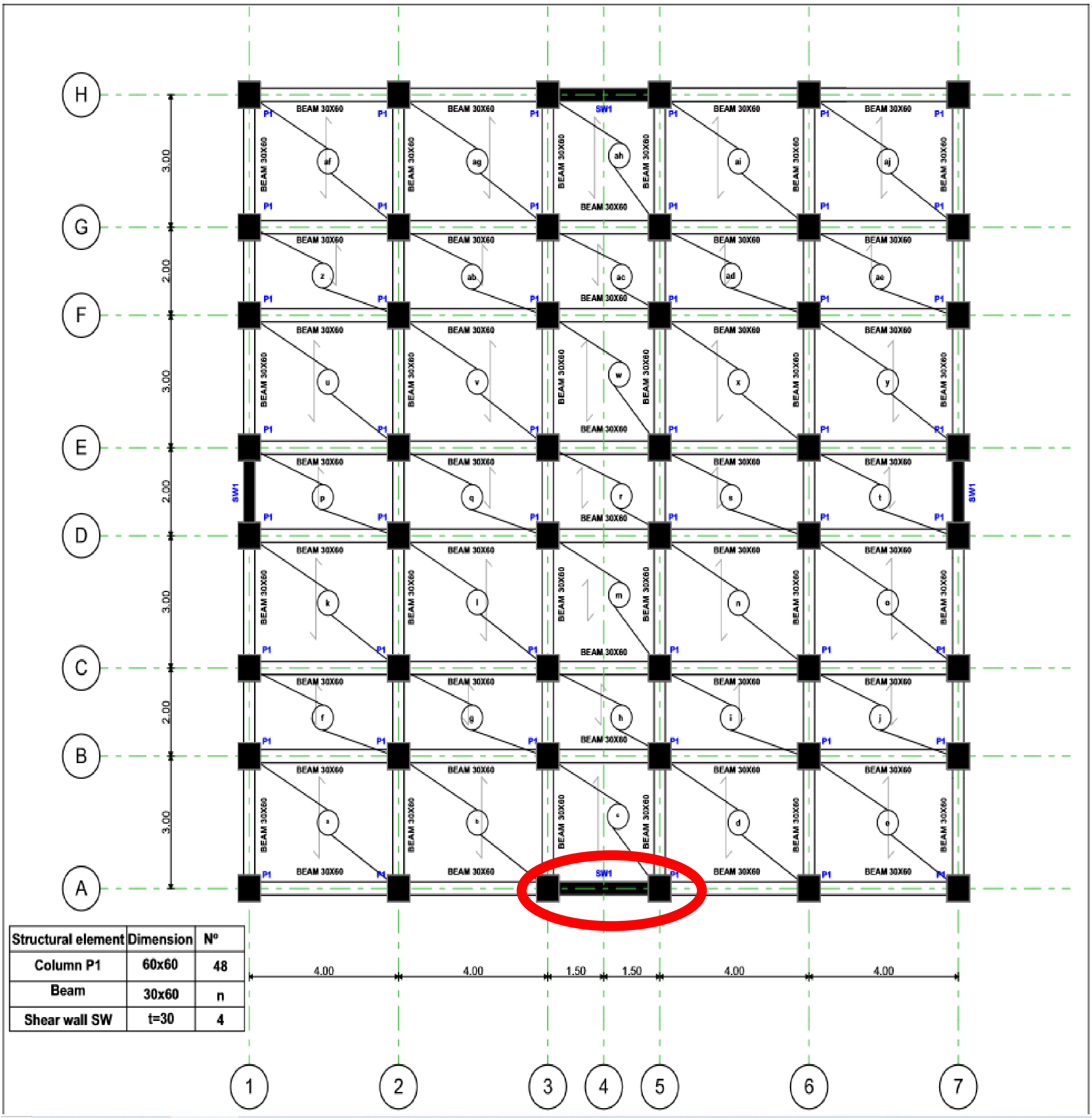
APPENDIX 3: DISTRIBUTION PLAN



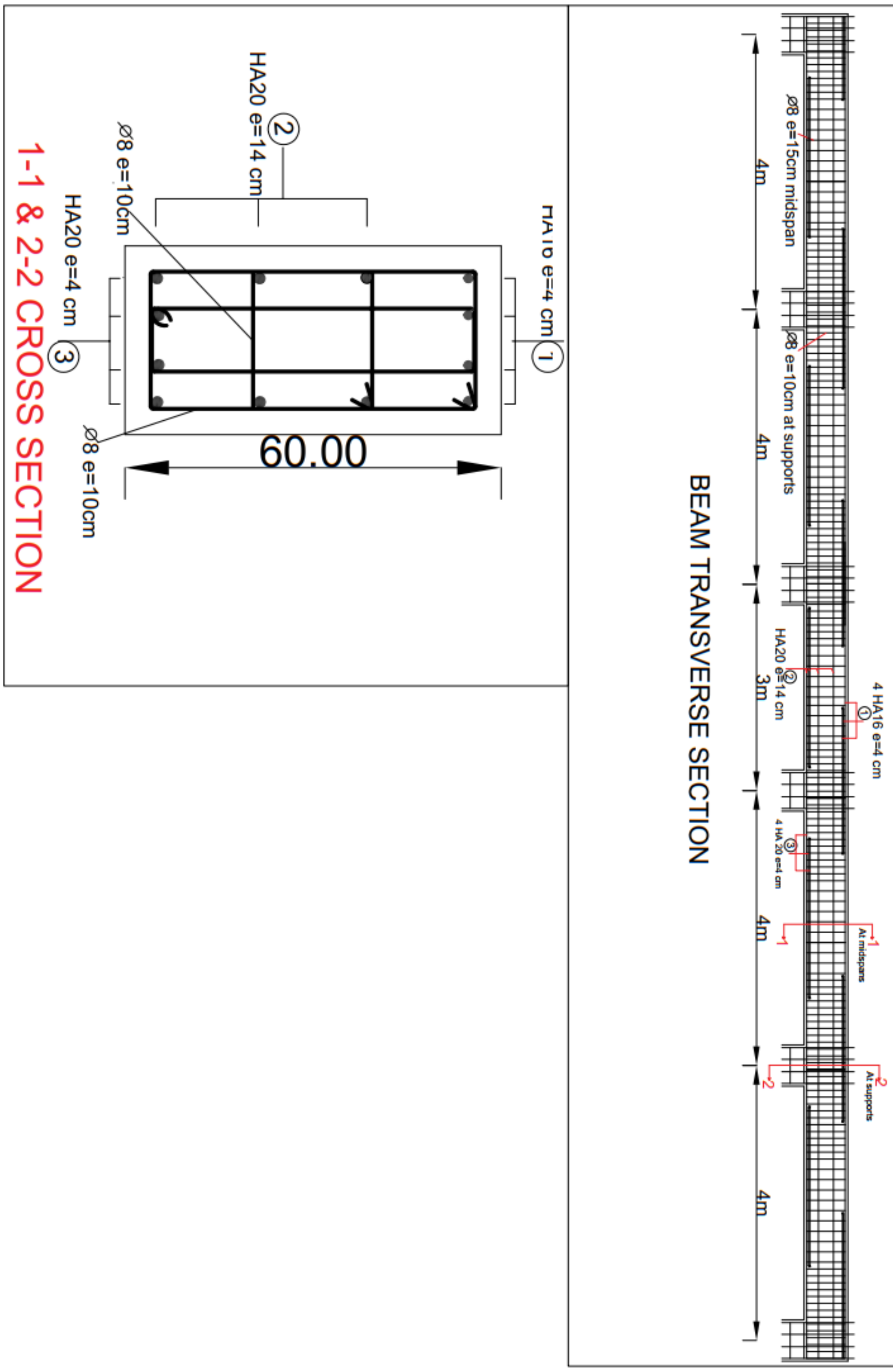
DISTRIBUTION PLAN OF GROUND FLOOR- STOREY 8

APPENDIX 4: STRUCTURAL PLAN AND REINFORCEMENT

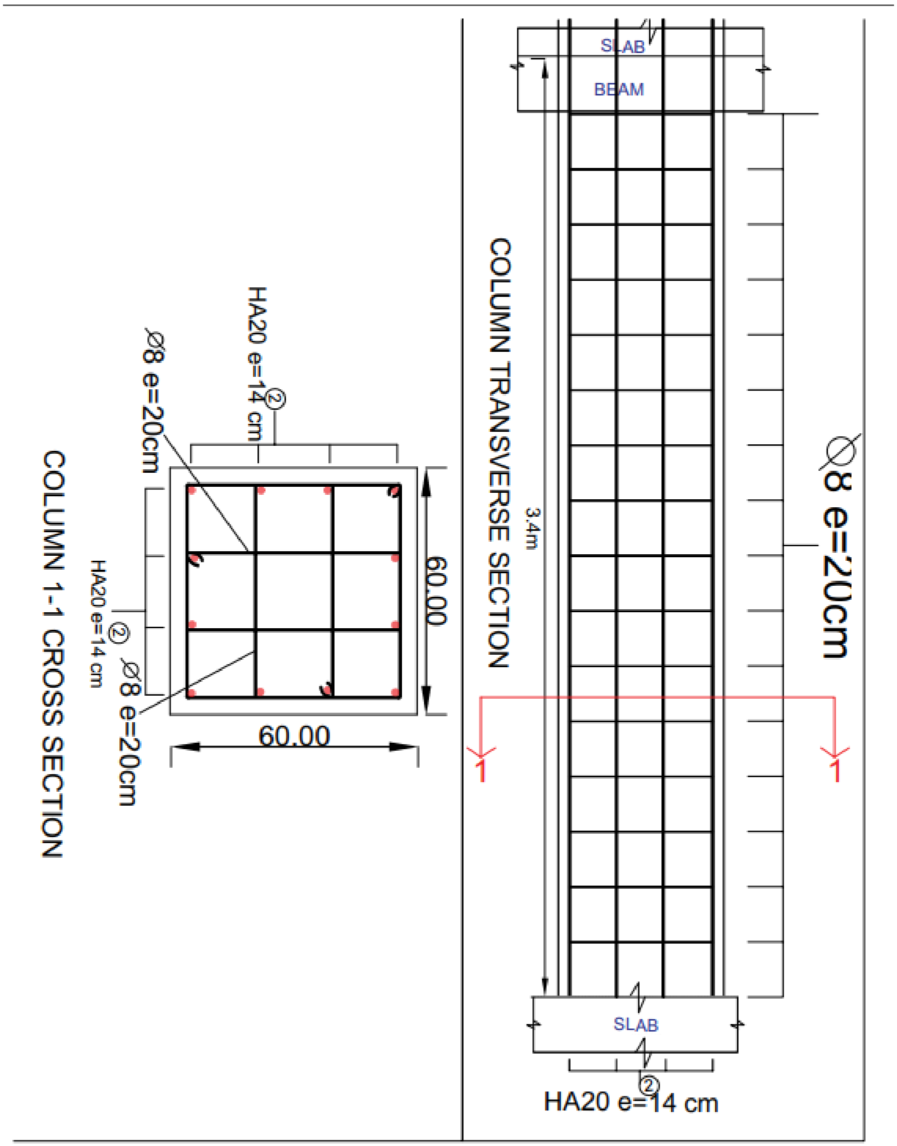
a) FLOOR BEAM PLANS



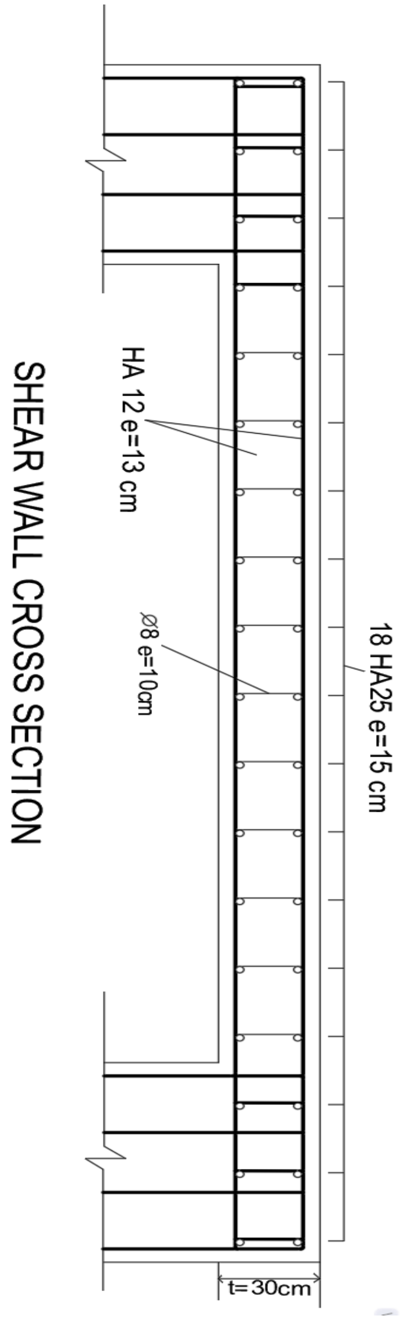
b) BEAM REINFORCEMENT



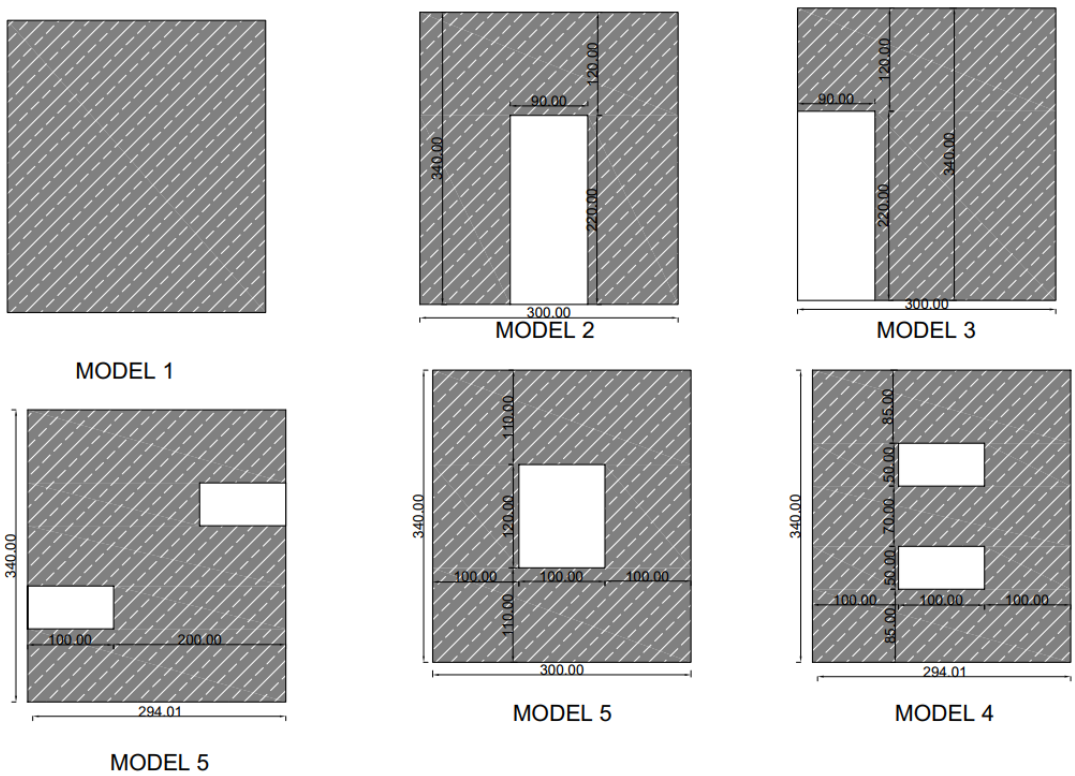
c) COLUMN REINFORCEMENT



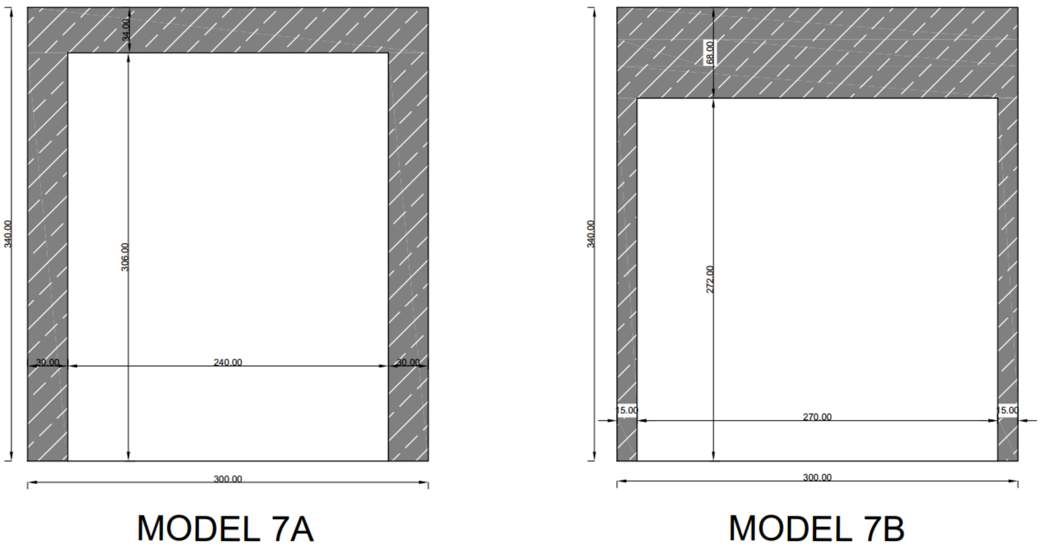
d) SHEAR WALL REINFORCEMENT



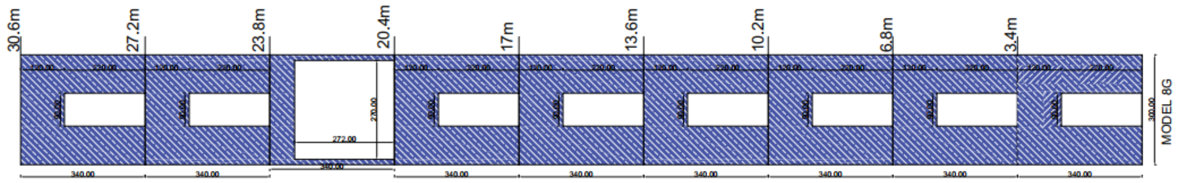
APPENDIX 5: MODELS 1-6

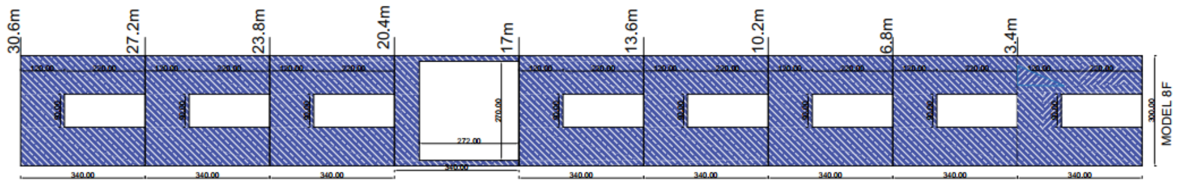


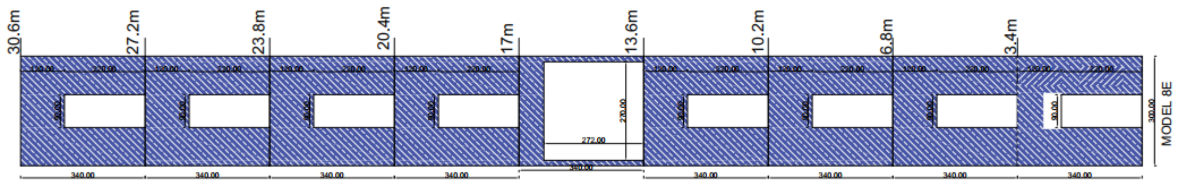
APPENDIX 6: MODEL 7

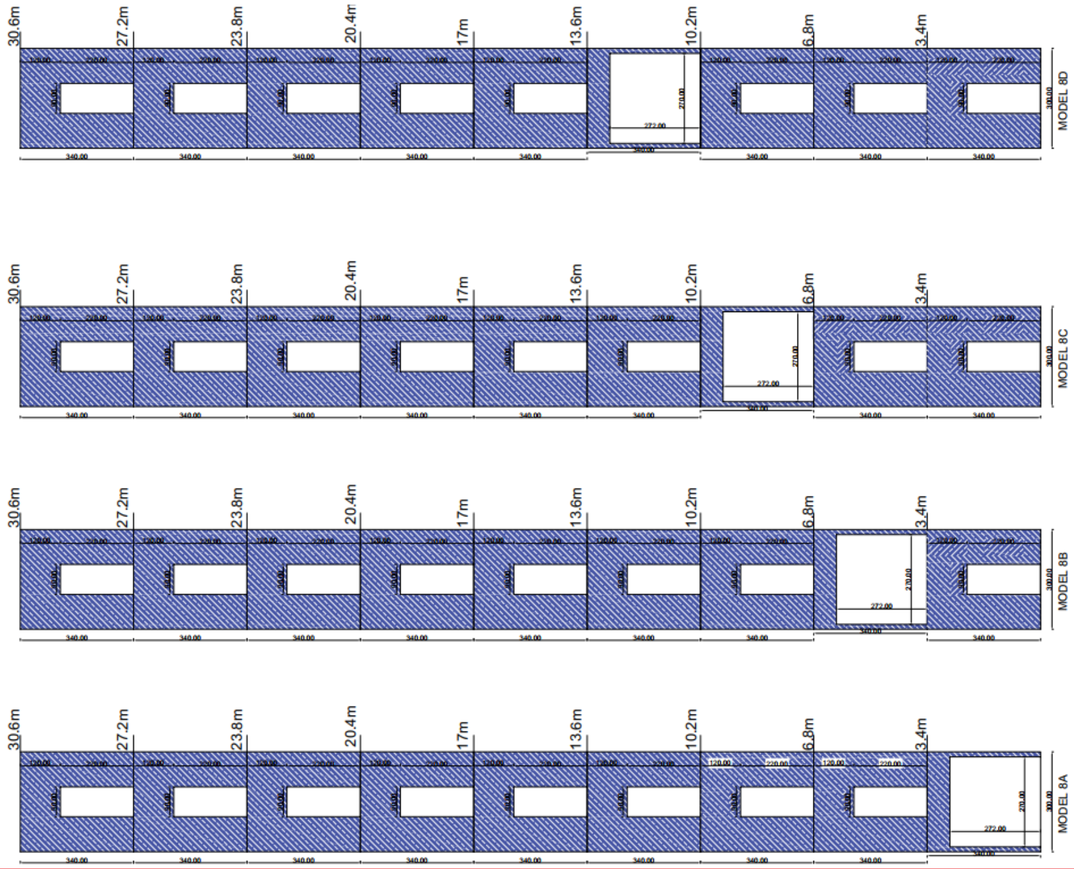


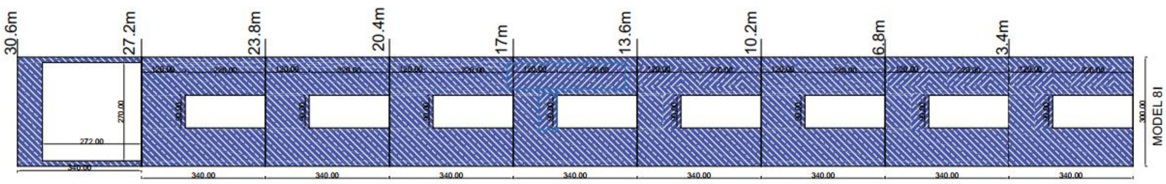
APPENDIX 7: MODEL 8








30.6m
27.2m
23.8m
20.4m
17m
13.6m
10.2m
6.8m
3.4m
MODEL 8D
MODEL 8C
MODEL 8B
MODEL 8A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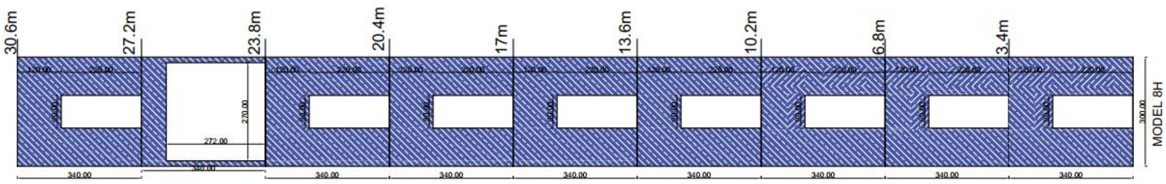
27.2m
23.8m
20.4m
17m
13.6m
10.2m
6.8m
3.4m
MODEL 8I


30.6m
27.2m
23.8m
20.4m
17m
13.6m
10.2m
6.8m
3.4m
MODEL 8H