SUDAN UNIVERSITY OF SCIENCE AND TECHNOLOGY

GOLLEGE OF GRADUATE STUDIES

THE STUDY OF STRUCTURAL BEHAVIOR OF BRACED CONCRTE TALL BUILDING FRAMES

دراسة السلوك الإنشائي لهياكل مباني خرسانية عالية

A Thesis Submitted to Sudan University of Science and Technology in Partial Fulfillment of the Requirements for the Degree of M.Sc. in Structural Engineering

By

Suhad Abbashar Osman EL-Mubarak

B.Sc. in Civil Engineering (Structures), 2009

Supervised by:

Dr. Abusamra Awad Attaelmanan

October2018

Dedication

To my beloved, sainted mother, who gave me my first lessons in spiritual thing.....

To my father, who taught me that the best kind of knowledge to have is that which is learned for its own sake...

To my husband, who has been a constant source of support and encouragement during my life.....

Acknowledgements

For the ancestors who paved the path before me upon whose shoulder I stand. This is also dedicated to my family and the many friends who supported me on this journey. Thank you.

I would like to express my deepest gratitude to my **supervisor Associate Dr.Abusamra Awad Attaelmanan** for his unwavering support, collegiality, and mentorship throughout this project.

ABSTRACT

In this study and according to European Specifications for Structural Concrete Buildings Standard in which the effect of the wind loads was applied on concrete tall building designed frames depending on the location and topography of the construction place. In order to study the structural behaviour of the frames, different types of tall building forms were assigned with equal dimensions. The frames were defined, simulated, analyzed, and designed by using structural design commercial available software (ETAB 2016).The analysis results obtained were tabulated and figured according to maximum values of beam forces, column forces, lateral story displacements, and story drifts. These results were compared using graphical presentation. The data were resulted and discussed, and eventually. In order to get the most economical frame of all studied cases, that the self-weight of each frame was obtained and compared, whereas the heights of all models and the dimensions were the same. The data were resulted and discussed, and eventually, the affect degree each frame by changing the structural frame.

مستخلص:

في هذا البحث ووفقاً للمواصفات القياسية الأوربية لمباني الخرسانة الإنشائية بأن مقدار تأثير أحمال الرياح على المباني العالية يقاس اعتماداً على موقع وطبوغرافية ومكان الإنشاء للمبنى، لذا ولأجل دراسة السلوك الإنشائي لهيكل مبنى عالي تم إختيار ثلاثة أنواع مختلفة من هياكل المباني العالية بحيث تكون متساوية في الأبعاد والإرتفاع ، بعد ذلك تم تعريف وإدخال ومحاكاة وتحليل وتصميم النماذج جميعها وذلك بإستخدام برنامج الحاسوب التجاري المتاح (ETAB2016)للتحليل والتصميم الإنشائي وتمت جدولة ورسم النتائج المخرجة من البرنامج لكل من القوى على عدد من الأبيام والأعمدة وأيضا تم حساب الإزاحة الجانبية والإنحراف الجانبي لكل طابق، إن جميع المخرجات تمت المقارنة بين بعضها البعض ودراستها في شكل رسوم بيانية، وتمت مناقشة جميع النتائج المتحصل عليها. ومن أجل الحصول على أكثر النماذج المدروسة إقتصادية تم قياس الوزن الذاتي لجميع الهياكل والمقارنة بينها علما بان إرتفاع جميع المباني متساوي، وأيضا تم معرفة وايجاد اكثر النماذج المدروسة ثباتاً، ومن ثم تم مناقشة جميع النتائج المتحصل عليها و وشرح وتحديد مدى درجة تأثير اي تغيير يطرأ على المنشأة.

Table of Contents

Chapter 3

METHODOLOGY AND MODELLING

Chapter 4

ANALYSIS AND DESIGN RESULTS OF TALL BUILDING MODELS

Chapter 5

CONCLUSION

Symbols

V_b basic wind v

$V_{b,0}$ fundamental value of the basic wind velocity

C_{dir} directional factor

C_{season} season factor

$V_m(z)$ mean wind velocity

$C_r(z)$ roughness factor

$C_o(z)$ orography factor

$Z_0$is roughness length

K_r Terrain factor depending on the roughness length

Z_{min} minimum heigh

K_I turbulence factor

C_0 orography factor

Z ρ air density

$q_p(Z_e)$ peak velocity pressure.

Z_e reference height for the external pressure.

C_{pe} pressure coefficient for the external pressure.

$q_p(Z_i)$ peak velocity pressure

Z_i reference height for the internal pressure

C_{pi} pressure coefficient for the internal pressure

C_sC_d structural factor

C_f force coefficient for the structure or structural element

$q_p(Z_e)$ peak velocity pressure at reference height Z_e

A_{ref} reference area of the structure or structural element

C_sC_d structural factor

W_e external pressure on the individual surface at height Z_e

W_i internal pressure on the individual surface at height Z_j,

C_{fr} friction coefficient

A_{fr} area of external surface parallel to the wind

ψ_0 a combination factor

ζ a reduction factor for unfavourable permanent actions G,

γ_G partial factor for permanent actions

γ_P partial factor for pre-stressing actions

γ_Q partial factor for variable actions

P represents actions due to pre-stressing

G_k dead loads.

Q_k Live loads.

W_k Wind load

List of Tables

List of Figures

CHAPTER ONE

INTRODUCTION

1.1. General

Tall towers and buildings have fascinated mankind from the beginning of civilization, their construction being initially for defence and subsequently for ecclesiastical purposes. The growth in modern tall building construction, however, which began in the 1880s, has been largely for commercial and residential purposes.

Tall commercial buildings are primarily a response to the demand by business activities to be as close to each other, and to the city centre, as possible, thereby putting intense pressure on the available land space. Also, because they form distinctive landmarks, tall commercial buildings are frequently developed in city centres as prestige symbols for corporate organizations. Further, the business and tourist community, with its increasing mobility, has fuelled a need for more, frequently high-rise, city centre hotel accommodations.

The rapid growth of the urban population and the consequent pressure on limited space has considerably influenced city residential development. The high cost of land, the desire to avoid a continuous urban sprawl, and the need to preserve important agricultural production have all contributed to drive residential buildings upward. In some cities, local topographical restriction makes tall buildings the only feasible solution for housing needs.

1.2. Problem Statement

The task of structural modelling is arguably the most difficult one facing the structural analyst, requiring critical judgment and a sound knowledge of the structural behaviour of tall building components and assemblies.

The lateral loading due to wind load is the major factor that causes the design of high- rise buildings to differ from low- to medium- rise buildings, the increase in size of the structural members to account of wind loading, incurs a cost premium that increases progressively with height.

The design of tall buildings essentially involves a conceptual design, approximate analysis, preliminary design and optimization, to safely carry gravity and lateral loads. The design criteriaare strength, serviceability, stability and human comfort. The serviceability is satisfied by drift limits.

1.3. Objectives

The objectives of the research were summaries as follows:

1. Aware of the structural forms of tall building.
2. To know the simulation of tall building using structural analysis software.
3. Study the degree of stability of selected types of structural forms of tall buildings using different types of forms.
4. Knowing which building is more economical of all studied case.

1.4. Methodology

1. The European standard EN 1992-1-1 - was used to get the loads that applied to buildings.
2. The structural design software– ETABS 2016 - was used to analyze the tall building forms.
3. The European standard EN 1991-1-4 -Actions on structures - Wind actions – was used for design of tall building.
4. The European standard EN 1992-2-Design of concrete structures – was used for design of tall building.

1.5. Research Outline

This research consists of five chapters. Chapter one was showing brief overview about the tall building, objectives, and the methodology been discussed in the thesis. Chapter two described the types of tall building frames system. Also, the structural concrete types, and the loading types, and how to be determined. Chapter three was explaining the simulated buildings data and depict the analysis results in tables and figures. In chapter four the results were discussed and building behaviour was studied to find the optimum model. Finally, the chapter five was explaining a detailed summery about the obtained results out of the analysed frames.

CHAPTER 2

LITERATURE REVIEW

2.1. Introduction

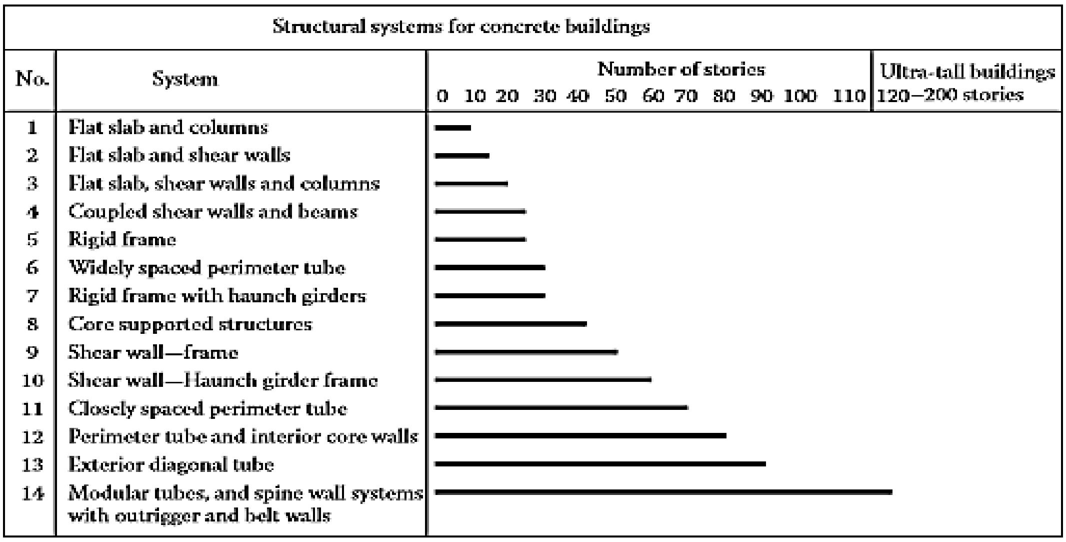
During the last 120 years, three major types of structures have been employed in tall buildings. The first type was used in the cast iron buildings of the 1850 to 1910, in which the gravity load was carried mostly by the exterior walls. The second generation of tall buildings, which began with the 1883 Home Insurance Building, Chicago, and includes the 1913 Woolworth Building and the 1931Empire State Building, are frame structures, in which a skeleton of welded or riveted steel columns and beams runs through, often encased in cinder concrete, and the exterior is a non-bearing curtain wall.

Most high-rises erected since the 1960s use a third type of structure, in which the perimeter structure of these buildings resembles tubes consisting of either closely spaced columns or widely spaced mega columns with braces. Inside the perimeter structure, a core made of steel, concrete, or a wells, mechanical equipment, and toilets (Taranath, 2009).

2.2. Structural Systems for Concrete Buildings

Reinforced concrete offers a wide range of structural systems that may be grouped into distinct categories, each with an applicable height range, as shown in Table (2.1).

Table (2.1): Structural systems for concrete buildings (Taranath,2009).



Structural systems for concrete buildings			
No.	System	Number of stories 0 10 20 30 40 50 60 70 80 90 100 110	Ultra-tall buildings 120–200 stories
1	Flat slab and columns		
2	Flat slab and shear walls		
3	Flat slab, shear walls and columns		
4	Coupled shear walls and beams		
5	Rigid frame		
6	Widely spaced perimeter tube		
7	Rigid frame with haunch girders		
8	Core supported structures		
9	Shear wall—frame		
10	Shear wall—Haunch girder frame		
11	Closely spaced perimeter tube		
12	Perimeter tube and interior core walls		
13	Exterior diagonal tube		
14	Modular tubes, and spine wall systems with outrigger and belt walls		

2.2.1. Shear Wall – Frame Interaction

This system is one of the most popular systems for resisting lateral loads in medium- to high-rise buildings. The system has a broad range of application and has been used for buildings as low as 10 stories to as high as 50 stories or even taller. With the advent of haunch girders, the applicability of the system can be extended to buildings in the 70- to 80-story range.

In this system, resistance to horizontal loading is provided by a combination of shear walls and rigid frames. The shear walls are often placed around elevator and service cores while the frames with relatively deep spandrels occur at the building perimeter shown in Figure (2.1).

The potential advantages of wall-frame structure depend on the amount of horizontal interaction, which is governed by relative stiffness's of the walls and frames, and the height of the structure. The taller the building and, typically proportioned structures, the stiffer the frames, the greater the interaction. It used to be common practice in the design of high rise structures to assume that the shear walls or cores resisted all the lateral loading, and to design the frames for gravity loading only.

The classical mode of interaction between a prismatic shear wall and a moment frame is shown in Figure (2.2). The frame deflects in a so-called shear mode whereas the shear wall predominantly responds in bending as a cantilever. Compatibility of horizontal deflection generates interaction between the two. The linear sway of the moment frame, combined with the parabolic sway of the shear wall, results in enhanced stiffness of the system because the wall is restrained by the frame at the upper levels while at the lower levels, the shear wall restrains the frame (Taranath, 2009). .

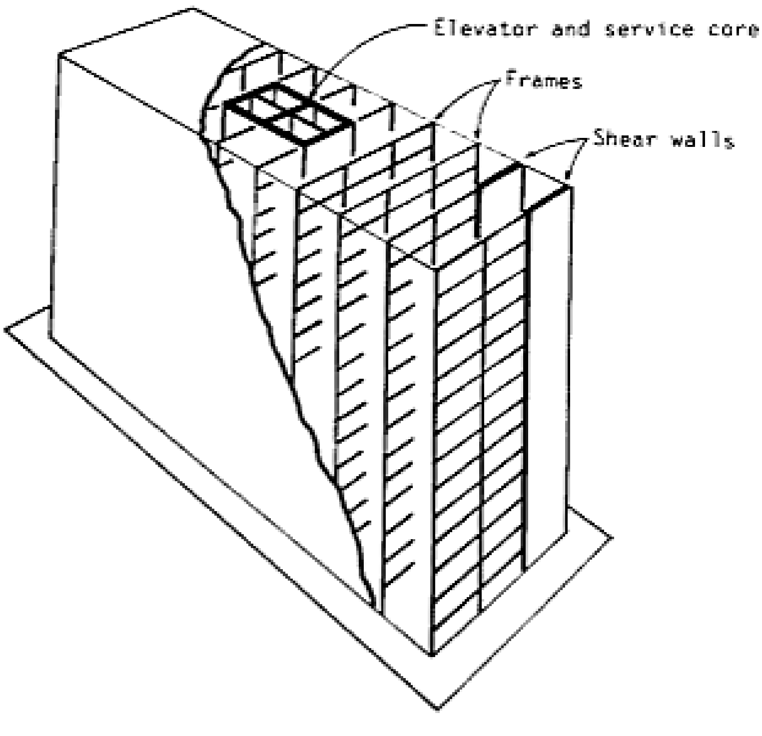


Figure (2.1): Wall-Frame Structure.

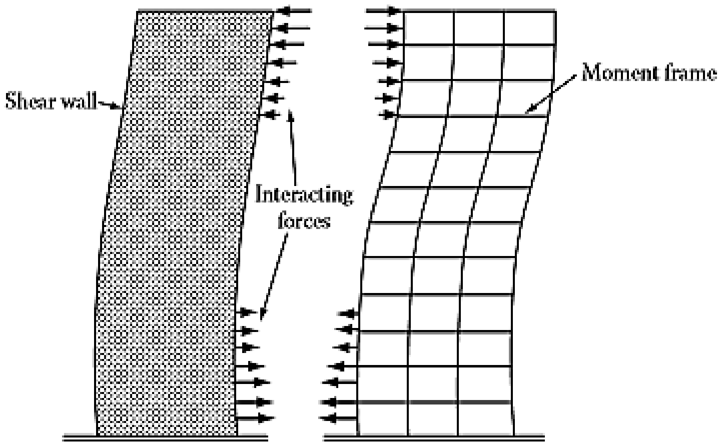


Figure (2.2): Shear Wall – Frame Interaction.

2.2.1.1. Behavior:

If the deflection modes of shear walls and moment frames were similar, the lateral loads would be distributed between the two systems more or less, according to their relative stiffness. However, in general, the two systems deform with their own characteristic shapes. The interaction between the two, particularly at the upper levels of the buildings, results in quite a different lateral load distribution (Taranath, 2009).

The lateral deflections of a shear wall may be considered as similar to those of a cantilever column. Near the bottom, the shear wall is relatively stiff, and therefore, the floor-to-floor deflections will be less than half the values near the top. At top floors, the deflections increase rather rapidly, mainly from the cumulative effect of wall rotation. Moment frames, on the other hand, deform predominantly in a shear mode. The relative story deflections depend primarily on the magnitude of shear applied at each story level. Although the deflections are larger near the bottom and smaller near the top as compared to the shear walls, the floor-to-floor deflections can be considered more nearly uniform throughout the height. When the two systems—the shear walls and moment frames—are connected by rigid floor diaphragms, a non-uniform shear force develops between the two. The resulting interaction typically results in a more economical structural system.

The linear sway of the moment frame, when combined with the parabolic deformation of the shear wall, results in an enhanced stiffness of the entire system because the wall is restrained by the frame at the upper levels while at lower levels, the shear wall is restrained by the frame. However, it is not always easy to differentiate between the two modes because a frame consisting of closely spaced columns and deep beams tends to behave more like a shear wall responding predominately in a bending mode (Taranath, 2009).

2.2.2. Frame Tube System

The framed tube system in its simplest form consists of closely spaced exterior columns tied at each floor level by relatively deep spandrels. The system works quite efficiently as a hollow vertical cantilever. However, lateral drift due to the axial displacement of the columns—commonly referred to as chord drift—and web drift, caused by shear and bending deformations of the spandrels and columns, may be quite large depending upon the tube geometry. The economy of the tube system therefore depends on factors such as spacing and size of columns, depth of perimeter spandrels, and the plan aspect ratio of the building. This system should, however, be given consideration for buildings taller than about 40 stories.

A somewhat different type of tube, often referred to as a braced tube, permits greater spacing of columns. As the name implies, the tube has diagonal bracing at the building exterior. Yet another variation of tube called bundled tube uses two or more tubes tied together to form a single, multi cell tube (Taranath, 2009).

2.2.2.1. Behavior:

The behaviour of the tube is in essence similar to that of a hollow perforated tube. The overturning moment under lateral load is resisted by compression and tension in the columns while the shear resisted by bending of columns and beams primarily in the two sides of the building parallel to the direction of the lateral load. The bending moments in the beams and columns of these frames, which are called web frames, can be evaluated using either of the two approximate procedures, namely, the portal or the cantilever analysis. It is perhaps more accurate to use the cantilever method because tube systems are predominately used for very tall buildings in the 40- to 80-story range in which the axial forces in the columns play a dominate role (Taranath, 2009).

The moments in spandrels and columns as well as the racking components of the tube deflection can be evaluated by using the cantilever method, because of the continuity of closely spaced columns and spandrels around the corners of the building, Figure (2.3), the flange frames are coaxed into resisting the overturning moment. Whether or not all the flange columns or only a portion there of contribute to the bending resistance is a function of shear rigidity of the tube (Taranath, 2009).

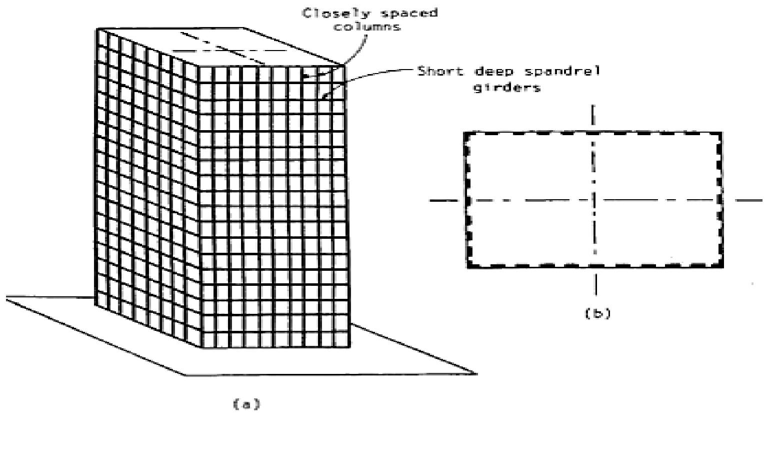


Figure (2.3): Framed- Tube Structure (a) Elevation; (b) Plan.

2.2.3. Core Supported Structures:

Shear walls placed around building services such as elevators and stair cores can be considered as a spatial system capable of transmitting lateral loads in both directions. The advantage is that, being spatial structures, they are able to resist shear forces and bending moments in two directions and also torsion particularly so when link beams are provided between the openings. The shape of the core is typically dictated by the elevator and stair requirements and can vary from a single rectangular core to multiple cores. Floor framing around the core typically consist of systems such as cast-in-place mild steel reinforced or post tensioned concrete (Taranath, 2009).

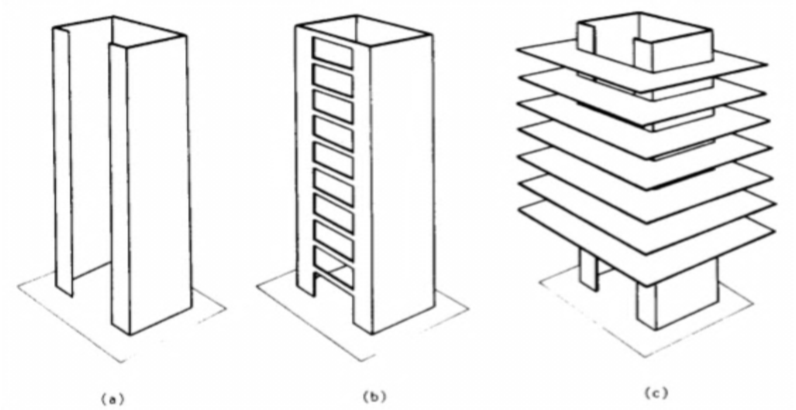


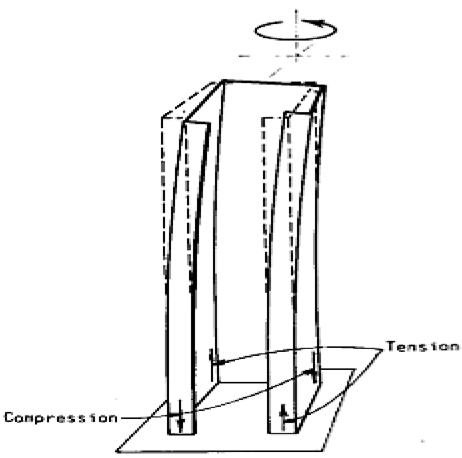
Figure (2.4): Core Structure (a) Open-Section Core (b) Core Partially Closed By Beams (c) Core Closed Partially By Floor Slabs.

Elevator cores are primary components for resisting both horizontal and gravity loading in tall building structures. Reinforced concrete cores usually comprise an assembly of connected shear walls forming a box section with opening that may be partially closed by beams, Figure (2.4-b) or floor slabs Figure (2.4-c).The moments of inertia of a reinforced concrete core are invariably large, so that it is often adequate in itself to carry the whole of the lateral loading. The horizontal load bending deflections and stresses of a core with a fully connected section are calculated conventionally, as for a vertical cantilever, on the basis of the core's moments of inertia about its principal axes (Smith ,B. and Coull, A. ,1991) .

If a building is also subjected to twist, the torsion stiffness of the core can be a significant part of the total torsion resistance of the building

The proportion of the height, length, and thickness of the walls of a typical building core classify it, in terms of its torsion behavior, as a thin-walled beam. Consequently, when the core twists, originally plane section of the core warp, Figure (2.5).Because the base section is prevented from warping by the foundation, the twisting vertical warping strains and stresses throughout the height of the core walls. In structures that are heavily dependent for their torsion stiffness of a core, the vertical warping stresses at the base of the core may be of the same order of magnitude as the bending stresses (Smith ,B. and Coull, A. ,1991).

Partial closure of the core by beams or slabs across the openings restrains the core section from warping and there be increases the core's torsion stiffness, while reducing its rotation and warping stresses, as shown in Figure (2.6).



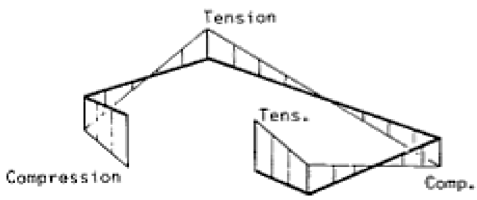


Figure (2.5): Twisted Core **Figure (2.6): Warping Stress In Core**

2.3. Types of Loading on Tall Buildings

The structure must be designed to resist the gravitational and lateral forces, both permanent and transient, that it will be called on to suction during its construction and subsequent service life. These forces will depend on the size and shape of the building, as well as on its geographic location, and maximum probable values must be established before the design can proceed.

2.3.1. Gravity loads

(A). Dead load

Dead loading is calculated from the designed member sizes and estimated material densities. These prone to minor inaccuracies such as differences between the real and the designed sizes, and between the actual and the assumed densities.

(B).Live load

A load produced by the use and occupancy of the building or other structure that does not include construction or environmental loads or dead load.

2.3.2. Wind load

Wind loading competes with seismic loading as the dominant environmental loading for structures.

Wind pressure on a building surface depends primarily on its velocity, the shape and surface structure of the building, the protection from wind offered by surrounding natural terrain or man-made structures, and to a smaller degree, the density of air which decreases with altitude and temperature.

(A). Wind velocity and velocity pressure

1. Basic Values

The basic wind velocity shall be calculated from below Expression.

$$V_b = C_{dir}\, C_{season}\, V_{b,0}$$

Where:

V_b is the basic wind velocity, defined as a function of wind direction and time of year at 10 m above ground of terrain category 11, (table2.2)

$V_{b,0}$ is the fundamental value of the basic wind velocity

C_{dir} is the directional factor

C_{season} is the season factor

Table (2.2): Terrain Category and Terrain Parameters (EN.1991-1-4)

Terrain category		z_0 m	z_{min} m
0	Sea or coastal area exposed to the open sea	0,003	1
I	Lakes or flat and horizontal area with negligible vegetation and without obstacles	0,01	1
II	Area with low vegetation such as grass and isolated obstacles (trees, buildings) with separations of at least 20 obstacle heights	0,05	2
III	Area with regular cover of vegetation or buildings or with isolated obstacles with separations of maximum 20 obstacle heights (such as villages, suburban terrain, permanent forest)	0,3	5
IV	Area in which at least 15 % of the surface is covered with buildings and their average height exceeds 15 m	1,0	10
NOTE: The terrain categories are illustrated in A.1.			

2. Mean Wind

a. Variation with height:

$$V_m(z) = C_r(z)\, C_o(z)\, V_b$$

Where:

$V_m(z)$ is the mean wind velocity

$C_r(z)$ is the roughness factor

$C_o(z)$ is the orography factor

b. Terrain roughness:

The terrain roughness shall be determined As Shown in Appendix A.

$$Cr(z) = KrLn\left(\frac{Z}{Z0}\right) \text{For } Z_{min} \leq Z \leq Z_{max}$$

$$Cr(z) = Cr(Z\text{min}) \text{For } Z \leq Z_{min}$$

Where:

Z_0 is the roughness length

K_r Terrain factor depending on the roughness length Z_0 calculated using

$$\mathbf{Kr = 0.19\ [\frac{Z0}{Z011}]\ ^\wedge\ 0.07}$$

Where:

$Z_{0,\,11}$ = 0.05 m

Z_{min} is the minimum height defined in Table (2.2)

Z_{max} is to be taken as 200 m

3. Wind Turbulence

The turbulence intensity I_v (z) at height z is defined as the standard deviation of the turbulence divided by the mean wind velocity.

$\boldsymbol{\sigma_V = K_{r.}\ V_b\ .K_I}$ **For** $\boldsymbol{Z_{min} \leq Z \leq\ Z\ max}$

$\boldsymbol{I_v(z) = I_v\ (Z_{min})}$ **For** $\boldsymbol{Z \leq Z_{min}}$

$$\mathbf{I_V\ (Z) = \frac{\sigma_V}{Vm\ (Z)} = \frac{K_i}{c_0\,(z)\ \ Ln\,(z/z0)}} \quad \textbf{For} \quad \mathbf{Z_{min}\ \leq\ Z\ \leq\ Z\ max}$$

Where:

K_I is the turbulence factor

C_0 is the orography factor

Z_0 is the roughness length, given in Table (2.2)

4. Peak velocity pressure

$$\boldsymbol{q_p(z) = (1+7.I_V(z)).\frac{1}{2}.\rho.V_m^2(z) = c_e(z).q_b}$$

Where:

ρ is the air density, which depends on the altitude, temperature and barometric pressure to be expected in the region during wind storms.

$$\mathbf{C_e\ (z) = \frac{qp\ (z)}{qb}}$$

$C_e(z)$ is the exposure factor (see Figure 4.2 in Appendix A)

q_b is the basic velocity pressure

$$\mathbf{q_b = 0.5\ \rho v_b^2}$$

(B). Wind actions

Wind actions on structures and structural elements shall be determined taking account of both external and internal wind pressures.

1. Wind pressure on surfaces:

1- The wind pressure acting on the external surfaces, should be obtained from Expression blew;

$$\mathbf{We = q_p(Z_e)\ .\ C_{pe}} \text{................. (2.1)}$$

Where:

$q_p(Z_e)$ is the peak velocity pressure.

Z_e is the reference height for the external pressure.

C_{pe} is the pressure coefficient for the external pressure.

2-The wind pressure acting on the internal surfaces of a structure, W_i, should be obtained from Expression blew;

$$\mathbf{W_i = q_p(Z_i).C_{pi}} \text{..................(2.2)}$$

Where:

$q_p(Z_i)$ is the peak velocity pressure

Z_i is the reference height for the internal pressure

C_{pi} is the pressure coefficient for the internal pressure

3- The net pressure on a wall roof or element is the difference between the pressures on the opposite surfaces taking due account of their signs. Pressure, directed towards the surface is taken as positive, and suction, directed away from the surface as negative. Examples are given in Figure 5.1 in Appendix A.

.

2. Wind Forces

i- The wind forces for the whole structureshould be determined:

*by calculating forces using force coefficients or

*by calculating forces from surface pressures.

ii- The wind force Fw acting on a structure or a structural component may be determined directly by using Expression blew;

$$\boldsymbol{F_w = C_s C_d . C_f . q_p(Z_e) . A_{ref}} \qquad \text{................(2.3)}$$

or by vectorial summation over the individual structural elements by using Expression below

$$\boldsymbol{F_w = C_s C_d . \sum C_f . q_p(Z_e) . A_{ref}} \qquad \text{...........................(2.4)}$$

where :

$C_s C_d$ is the structural factor

C_f is the force coefficient for the structure or structural element

$q_p(Z_e)$ is the peak velocity pressure at reference height Z_e

A_{ref} is the reference area of the structure or structural element

3-The wind force, Fw acting on a structure or a structural element may be determined by vectorial summation of the forces Fw,e, FW,i and calculated from the external and internal pressures using Expressions (2.5) and (2.6) and the frictional forces resulting from the friction of the wind parallel to the external surfaces, calculated using Expression (2.7)

External forces:

$$\mathbf{F_{w,e} = C_s C_d . \sum W_e . A_{ref}} \qquad \text{......................(2.5)}$$

Internal forces:

$$\mathbf{F_{w,i} = \sum_{surfaces} W_i \ . A_{ref}} \qquad \text{.....................(2.6)}$$

Friction forces:

$$\mathbf{F_{fr}} = \mathbf{C_{fr} q_p (Z_e) . A_{fr}} \qquad \text{....................(2.7)}$$

where :

$C_s C_d$ is the structural factor

W_e is the external pressure on the individual surface at height Z_e

W_i is the internal pressure on the individual surface at height Z_j,

is the reference area of the individual surface

C_{fr} is the friction coefficient

A_{fr} is the area of external surface parallel to the wind

4-The effects of wind friction on the surface can be disregarded when the total area of all surfaces parallel with (or at a small angle to) the wind is equal to or less than 4 times the total area of all external surfaces perpendicular to the wind (windward and leeward).

5-In the summation of the wind forces acting on building structures, the lack of correlation of wind pressures between the windward and leeward sides may be taken into account.

2.4. Load combinations

Method of accounting for load combination and their effects on the design of member vary according to the code used and design philosophy. The load combination expressions, as they appear in to (EN1990), are provided below;

2.4.1. Ultimate limit state

$$\sum_{j\geq 1} \gamma_{G,j} G_{K,j} + \gamma_p P + \gamma_{Q,I} Q_{K,I} + \sum_{i>1} \gamma_{Q,i} \psi_{O,j} Q_{K,i}$$

$$\sum_{j\geq 1} \gamma_{G,j} G_{K,j} + \gamma_p P + \gamma_{Q,I} \psi_{O,1} Q_{K,I} + \sum_{i>1} \gamma_{Q,j} \psi_{O,j} Q_{K,j}$$

$$\sum_{j\geq 1} \zeta \gamma_{G,j} G_{K,j} + \gamma_p P + \gamma_{Q,1} \psi_{O,1} Q_{K,1} + \sum_{i>1} \gamma_{Q,j} \psi_{O,j} Q_{K,j}$$

Where:

$\sum$ implies the combined effect of

ψ_0 is a combination factor

ζ is a reduction factor for unfavourable permanent actions G,

γ_G is partial factor for permanent actions

γ_P is partial factor for pre-stressing actions

γ_Q is a partial factor for variable actions

P represents actions due to pre-stressing

2.4.2. Serviceability Limit State

$$\sum_{j\geq 1} G_{k,j} + P + Q_{K,1} + \sum_{i\geq 1} \psi_{0,i} Q_{k,i}$$

$$\sum_{j\geq 1} G_{k,j} + P + \psi_{1,1} Q_{K,1} + \sum_{i\geq 1} \psi_{2,i} Q_{k,i}$$

$$\sum_{j\geq 1} G_{k,j} + P + \sum_{i\geq 1} \psi_{2,i} Q_{k,i}$$

2.5. Story Drift

Drift is generally defined as the lateral displacement of one floor relative to the floor below. Drift control is necessary to limit damage to interior partitions, elevator and stair enclosures, glass, and cladding systems. Stress or strength limitations in ductile materials do not always provide adequate drift control, especially for tall buildings with relatively flexible moment-resisting frames or narrow shear walls.

Total building drift is the absolute displacement of any point relative to the base. Adjoining buildings or adjoining sections of the same building may not have identical modes of response, and therefore may have a tendency to pound against one another. Building separations or joints must be provided to permit adjoining buildings to respond independently to earthquake ground motion.

CHAPTER THREE

MODELLING AND ANALYSIS

TALL BUILDINGS

3.1. Introduction

3.1.1. Overview:

In this study, a plan of 25- story office building, three models was investigated as shown in Figure (3.1). The bay widths were between 4.8m – 7.2m and the story heights were 3.2m for typical story except the ground which was 4 m. The first step is to choose three types of tall building frame structural systems according to the general classification of concrete tall building frame system mentioned in chapter two.

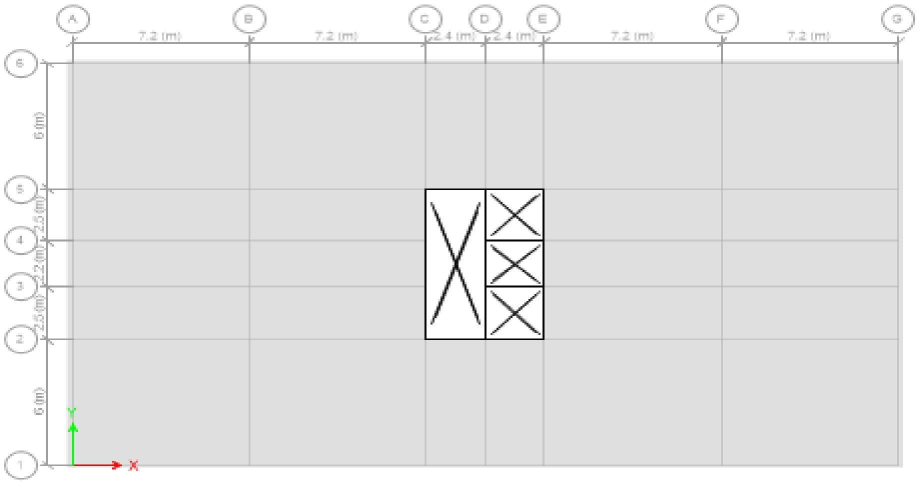


Figure (3.1): Plan of tall building models.

3.1.2 The types of frame systems which studied are as follows:

1- Shear wall- frame system as shown in Figure (3.2).

2- Tube frame system as shown in Figure (3.3).

3- Core system as shown in Figure (3.4).

3.2. Building Data

The modelling and analysis were carried out using structural analysis software (ETABS 2016). The concrete code design was the EN 1992-2, and EN 1992-1-1 - was used to get the loads that applied to buildings.

All display units by metric system (SI), the material properties were presented as shown in the Table (3.1).

Table (3.1): Material Properties.

Name	Type	E (MPa)	ν	Unit Weight (kN/m³)	Design Strengths
Concrete 35	Concrete	24855.6	0.2	24	F_c=35 MPa
Steel 520	Steel	200000	0.3	77	F_y=380 MPa Fu=450 MPa

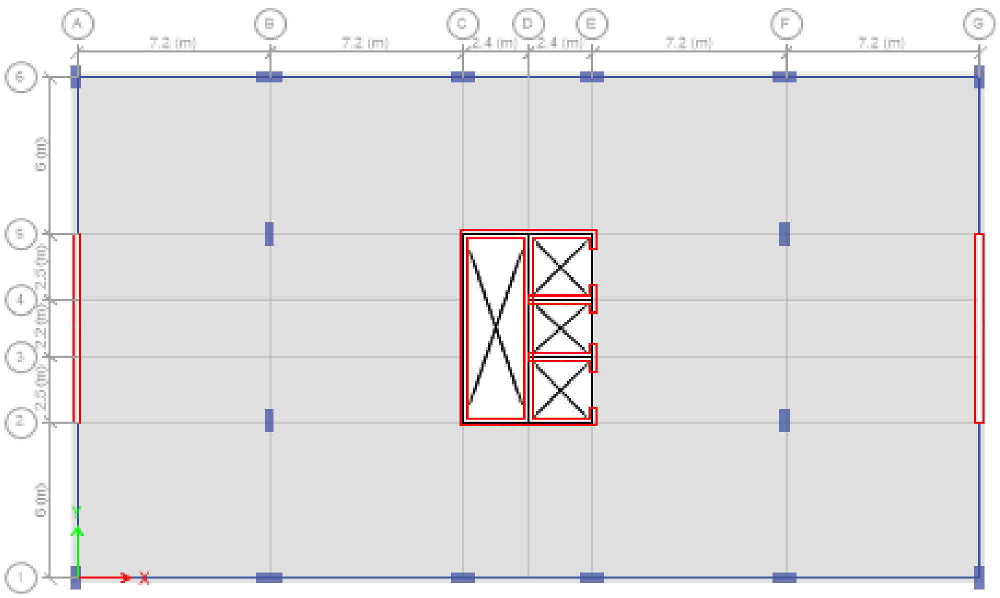


Figure (3.2): Plan of Shear wall- frame system.

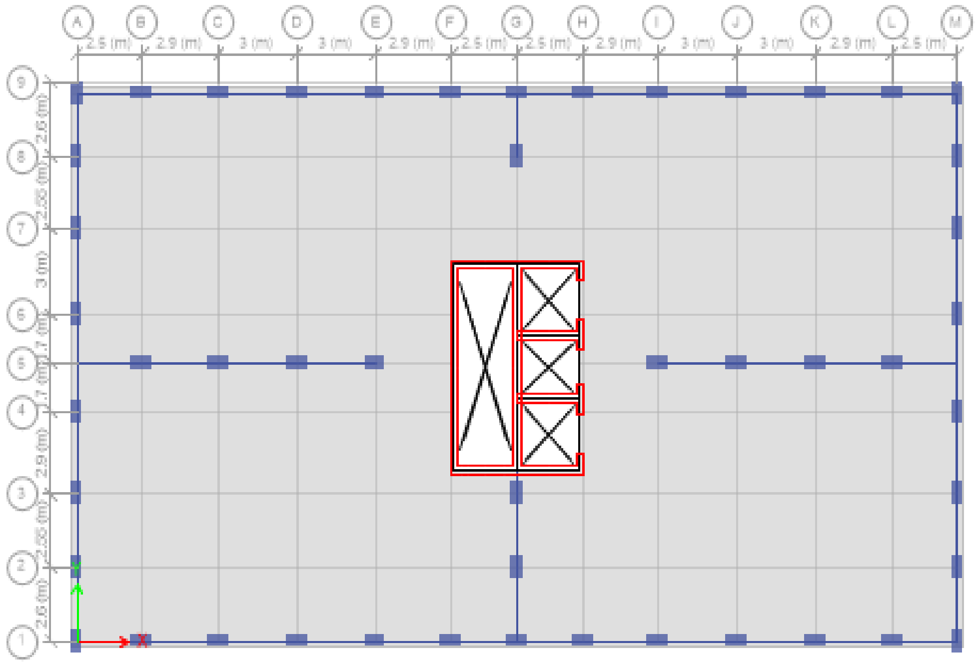


Figure (3.3): Plan of Tube system.

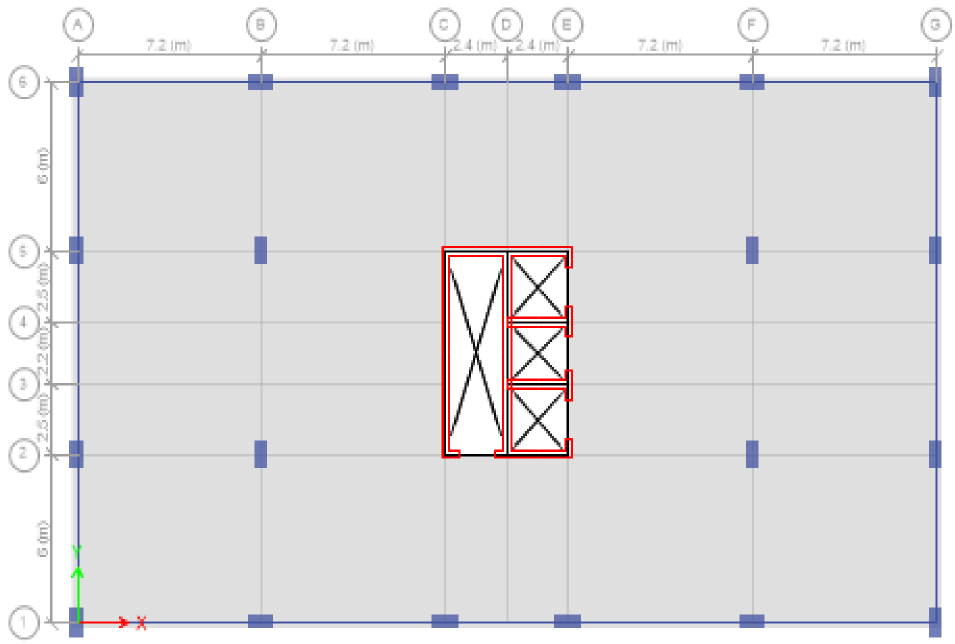


Figure (3.4): Plan of Core system.

3.3. Load Calculations and Combinations

The loads information applied to the models were as shown in Table (3.2).

3.3.1. Dead load

The dead load of structural system of tall building was calculated by (ETABS2016) and the finishing loads and partitions were assumed to be 2KN/m^2.

Table (3.2): Load Patterns.

Name	Type	Self-Weight Multiplier	Note
Dead	Dead	1	EN1991-1-1
Live	Live	0	EN1991-1-1
Wind	Wind	0	EN1991-1-4

3.3.2. Live load

The live load was taken according to (EN1991-1-1) is equal to 3KN/m^2.

3.3.3. Wind load

The wind Load Calculations were calculated according to (EN1991-1-4) as follows:

1. Exposure Parameters

a. Exposure From = Shell Objects

b. Exposure Category = B

c. Windward Coefficient, Cp =0.8

d. Leeward Coefficient, Cp =0.5

e. Top Story = Last Story

f. Bottom Story = Base

2. Factors and Coefficients:

a. Terrain Velocity = 0

b. Orography Factor,$C_0(z)$ =1

c. Turbulence Factor,K_1=1

d. Structural Factor, $C_s\ C_d$ =1

e. Air Density, Rho (Kg/m^3) =1.25

3.3.4 Lateral Loading

a) The Fundamental value of the basic wind velocity $V_{b,0}$ =26 m/s(see European wind map).

b) Basic wind velocityV_b, [EN 1991-1-4 Expression (4.1)].

$$\mathbf{V_b = C_{dir}\ C_{season}\ V_{b,0}}$$

3.4 Load Combinations

All combinations ware selected according to (EN1990).

- Combination (1): $1.35G_k$.
- Combination (2): $1.35G_k + 1.5Q_k + 0.75W_k$.
- Combination (3): $1.35G_k+1.05Q_k+1.5W_k$.
- Combination (4): $1.25G_k+1.5Q_k+0.75W_k$.
- Combination (5): $1.25G_k+1.05Q_k+1.5W_k$.

Where:

G_k= dead loads.

Q_k= Live loads.

W_k= Wind loads.

3.5 Structural Analysis of Shear Wall - Frame System of Tall Building:

When shear-walls are combined with rigid frames the walls and frame, which tend to deflect in a flexural configuration and a shear mode sequently are constrained to adopt a common deflected shape by the horizontal rigidity of the girder and slabs. The interacting wall-frame combination is appropriate for buildings in the 40-60-storeys range, well beyond that of rigid frames or shear-walls alone.

1. Bending Moment at Slabs

The slabs bending moment varied depending on the applied load on floors. Also, the loads are the same in all buildings models, the stresses on the slabs were slightly same for all models. The distribution of bending moments in x-, and y- directions at the 25th floor slab was presented as shown in Figures (3.5)-(3.6).

2. Column Results

In order to study a structural behavior of rigid frame, it was taken edge tension, compression, and internal column (C1, C8, C20), respectively. The column forces summarised as shown in Table (3.3), where P, M2, and M3 were axial forces, moment in x- direction, and moment in y- direction respectively.

3. Beam Forces

In order to study a structural behavior of rigid frame, it was taken the maximum values of bending moment and shear forces of the whole building's beams. The Beam results summarised as shown in Tables (3.4)-(3.5).

4. **Story Displacements**

The story displacement gave an indication to the lateral effect due to wind loads on the building. The Figures (3.7)-(3.8) show the displacement of the storeys.

5. Story Drifts

The story drifts considered as a major effective parameter, which increase gradually from top to base of the frame as resulted in Figures (3.9) - (3.10).

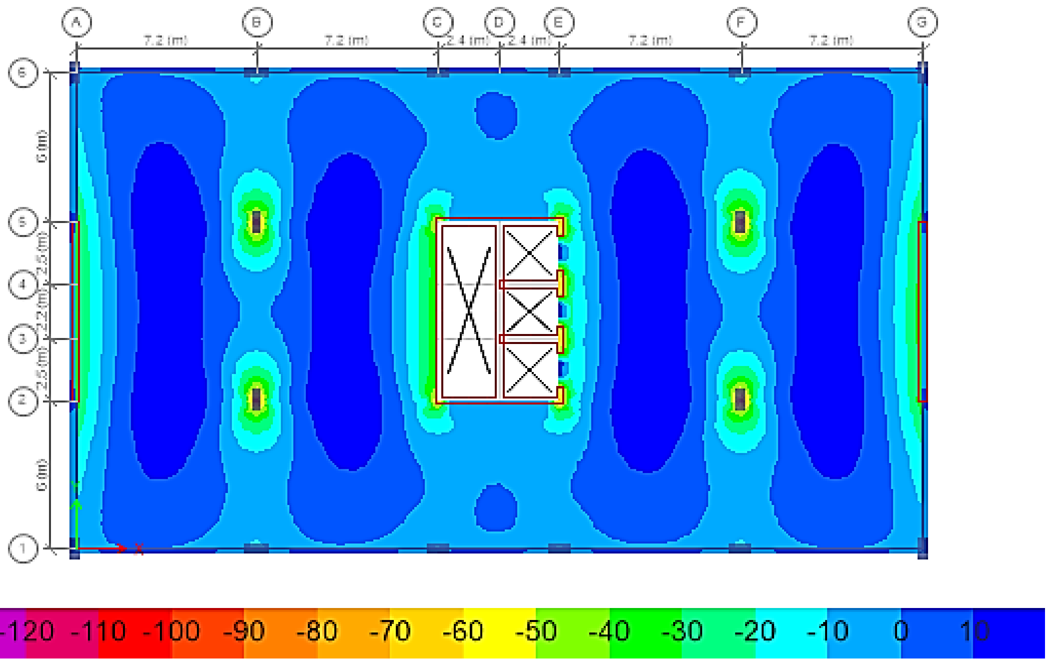


Figure (3.5): The distribution of bending moments in x- direction(combination4).

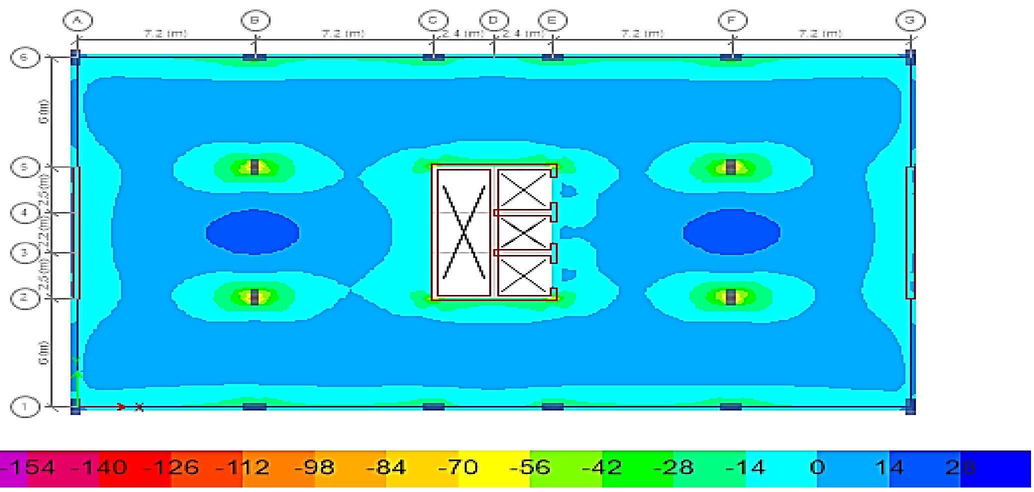


Figure (3.6): The distribution of bending moments in y- direction (combination 4).

Table (3.3): The distribution of axial forces and bending moments on columns for load combination 3(shear wall – frame).

story	Column 1			Column 8			Column 20		
	P	M2	M3	P	M2	M3	P	M2	M3
	KN	KN-m	KN-m	KN	KN-m	KN-m	KN	KN-m	KN-m
story25	-150.70	87.72	172.09	-224.86	-77.36	194.45	-326.02	-1.23	-10.26
story24	-306.32	70.31	137.08	-435.86	-59.17	163.16	-644.50	-1.19	-7.82
story23	-459.74	73.91	146.12	-649.70	-63.18	170.43	-964.86	-1.09	-9.07
story22	-612.38	73.28	145.67	-863.17	-61.84	172.33	-1285.27	-1.17	-8.90
story21	-763.82	73.19	151.38	-1077.00	-61.61	175.90	-1606.37	-1.21	-8.77
story20	-913.39	73.92	145.20	-1291.13	-61.02	179.70	-1928.21	-1.26	-8.34
story19	-1061.72	73.06	156.45	-1505.69	-60.42	183.79	-2250.97	-1.36	-7.62
story18	-1208.79	73.50	153.97	-1720.77	-59.76	187.90	-2574.81	-1.45	-6.71
story17	-1353.64	73.45	156.03	-1936.49	-59.09	191.91	-2899.91	-1.55	-5.60
story16	-1497.32	72.91	163.94	-2152.99	-58.39	195.62	-3226.43	-1.65	-4.29
story15	-1638.51	74.13	150.49	-2370.39	-57.72	198.89	-3554.56	-1.74	-2.85
story14	-1778.78	72.37	168.55	-2588.86	-57.04	201.51	-3884.47	-1.88	-1.12
story13	-1918.22	73.03	160.79	-2808.54	-56.41	203.34	-4216.37	-2.00	0.72
story12	-2055.61	72.63	162.89	-3029.63	-55.84	204.19	-4550.44	-2.12	2.77
story11	-2192.29	72.35	162.20	-3252.30	-55.34	203.89	-4886.88	-2.24	4.97
story10	-2328.10	71.97	161.24	-3476.76	-54.92	202.26	-5225.91	-2.36	7.36
story9	-2463.51	71.52	159.33	-3703.2	-54.62	199.08	-5567.73	-2.47	9.91
story8	-2598.82	70.98	156.48	-3931.86	-54.43	194.17	-5912.55	-2.58	12.71
story7	-2734.49	70.31	152.51	-4162.95	-54.38	187.31	-6260.62	-2.69	15.61
story6	-2871.02	69.56	147.50	-4396.73	-54.54	178.24	-6612.15	-2.79	18.93
story5	-3008.99	68.47	140.88	-4633.44	-54.59	166.91	-6967.40	-2.88	22.26
story4	-3149.14	68.00	134.03	-4873.34	-56.25	152.46	-7326.58	-2.94	26.00
story3	-3292.18	63.42	121.60	-5116.78	-51.76	136.93	-7690.07	-2.98	31.70
story2	-3439.89	77.31	124.80	-5363.62	-75.26	116.53	-8057.59	-3.17	27.58
story1	-3603.56	23.54	82.30	-5632.26	-9.83	106.96	-8453.56	-2.00	74.11

Table (3.4): Beams maximum bending moments for load combination 3 (shear wall – frame).

Story	Beam4		Beam7	
	S.F	Max.(-ve)B.M	S.F	Max.(+ve)B.M
	KN	KN-m	KN	KN-m
Story25	175.77	-159.42	14.67	136.03
Story24	181.13	-158.66	18.44	147.27
Story23	181.82	-158.65	19.37	148.96
Story22	184.03	-158.70	21.31	153.70
Story21	186.37	-158.54	23.48	158.81
Story20	188.97	-158.35	25.87	164.50
Story19	191.68	-158.00	28.40	170.47
Story18	194.39	-157.49	30.93	176.45
Story17	196.93	-156.80	33.34	182.13
Story16	199.23	-155.90	35.56	187.34
Story15	201.18	-154.76	37.50	191.85
Story14	202.71	-153.38	39.08	195.51
Story13	203.66	-151.72	40.19	198.02
Story12	203.91	-149.76	40.69	199.06
Story11	203.34	-147.49	40.50	198.44
Story10	201.84	-144.86	39.51	195.88
Story9	199.27	-141.85	37.60	191.12
Story8	195.47	-138.44	34.65	183.83
Story7	190.30	-134.57	30.49	173.67
Story6	183.55	-130.22	24.98	160.25
Story5	175.01	-125.33	17.92	143.10
Story4	164.45	-119.82	9.09	121.73
Story3	151.56	-113.75	-1.73	95.58
Story2	135.88	-106.59	-15.09	63.24
Story1	119.91	-102.31	-31.91	25.90

Table (3.5): Beams maximum shear forces for load combination 3(shear wall – frame).

Story	Beam4		Beam7	
	Max.(-ve)S.F	B.M	Max.(+ve)S.F	B.M
	KN	KN-m	KN	KN-m
Story25	-57.02	14.76	175.77	-159.42
Story24	-50.64	28.57	181.13	-158.66
Story23	-50.56	29.02	181.82	-158.65
Story22	-49.33	32.03	184.03	-158.70
Story21	-48.04	35.24	186.37	-158.54
Story20	-46.61	38.79	188.97	-158.35
Story19	-45.14	42.49	191.68	-158.00
Story18	-43.67	46.21	194.39	-157.49
Story17	-42.25	49.79	196.93	-156.80
Story16	-40.96	53.11	199.23	-155.90
Story15	-36.07	57.82	201.18	-154.76
Story14	-38.96	58.38	202.71	-153.38
Story13	-38.37	60.05	203.66	-151.72
Story12	-38.13	60.87	203.91	-149.76
Story11	-38.31	60.70	203.34	-147.49
Story10	-38.98	59.37	201.84	-144.86
Story9	-40.19	56.74	199.27	-141.85
Story8	-42.03	52.62	195.47	-138.44
Story7	-44.56	46.83	190.30	-134.57
Story6	-47.87	39.17	183.55	-130.22
Story5	-52.05	29.44	175.01	-125.33
Story4	-57.22	17.36	164.45	-119.82
Story3	-63.44	2.72	151.56	-113.75
Story2	-71.18	-15.44	135.88	-106.59
Story1	-82.63	-38.35	119.91	-102.31

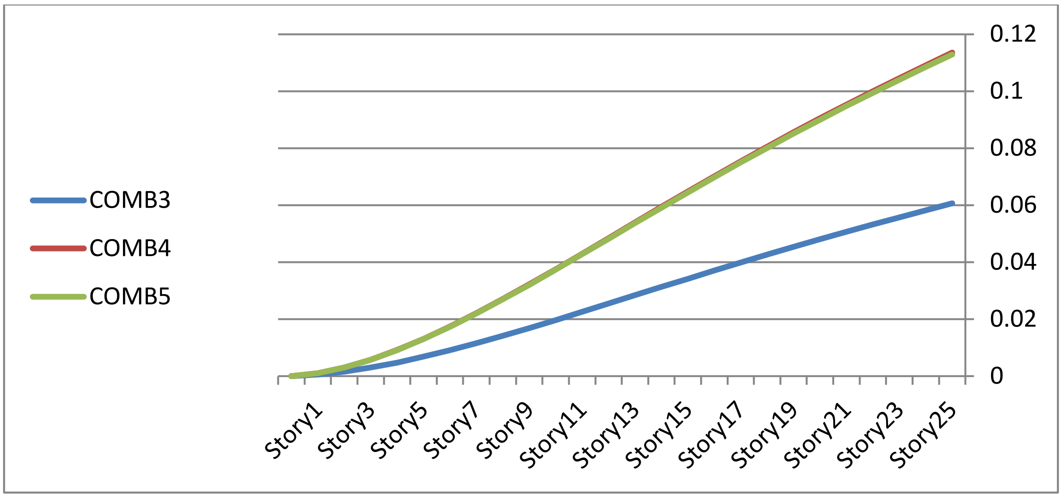


Figure (3.7): The Distribution of Maximum Story Displacement in X-Direction.

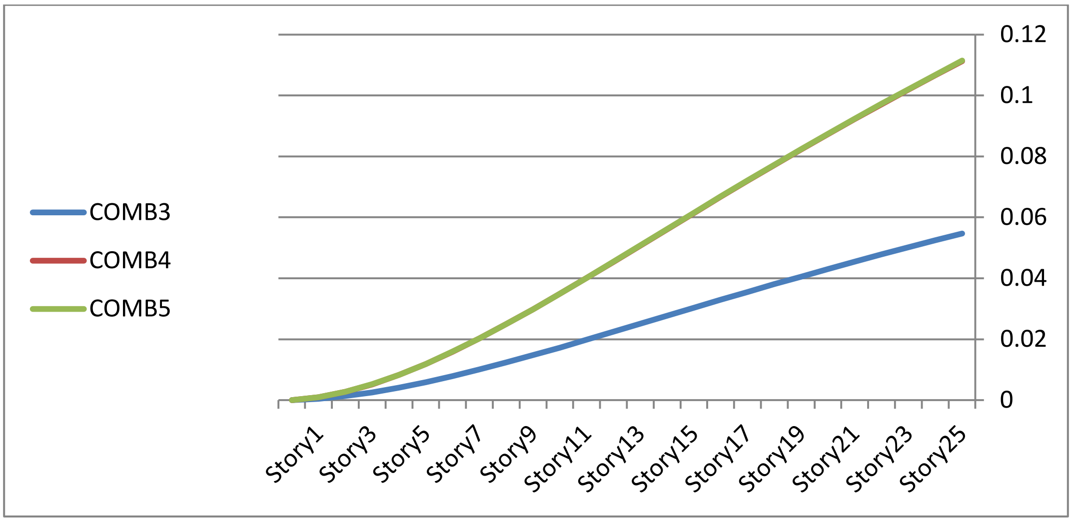


Figure (3.8): The Distribution of Maximum Story Displacement in Y-Direction.

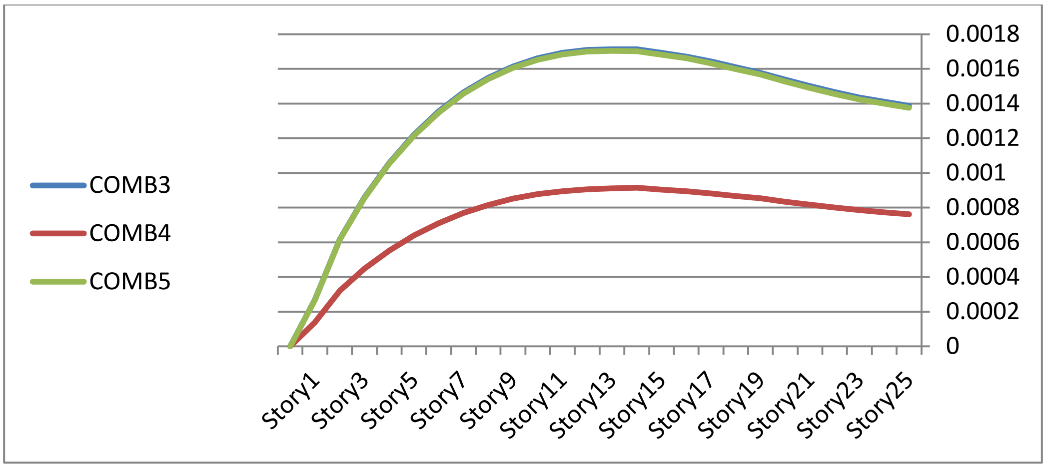


Figure (3.9): The Distribution of Maximum Story Drift in X-Direction.

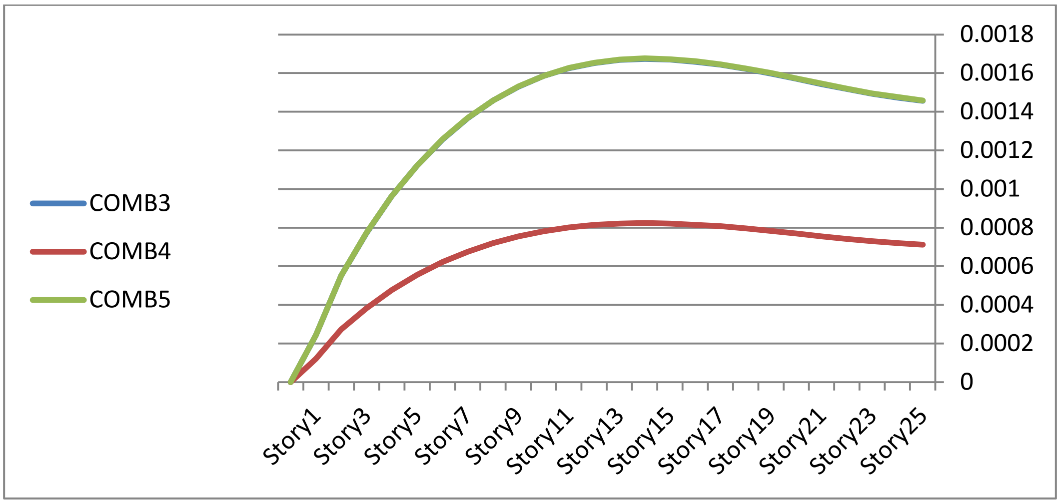


Figure (3.10): The Distribution of Maximum Story Drift in Y-Direction.

3.6 Structural analysis of Tube frame system of tall building:

1. Column results

By the same way used to represent the column forces in Wall-Frame system, it was taken edge tension, compression and internal column (C1, C8, C20), respectively. The column results summarised as shown in Table (3.6).

2. Beams forces

Only the beams had the maximum bending moment and shear forces were tabulated .The beams forces were summarized in Table (3.7).

3. Story displacements

In order to show the lateral effect occurred due to wind loads on the building, only the story displacements at y-direction for all combinations were depicted at Figures. (3.11) – (3.12).Because of the story displacements at y- direction has greater value than x-direction.

4. Story drifts

The story drifts considered as a major effective parameter, which increase gradually from top to base of the frame as resulted in Figures.(3.13) - (3.14).

Table (3.6): The distribution of axial forces and bending moments on columns for load combination 3(tube frame).

Story	Column 1			Column 8			Column 20		
	P KN	M2 KN-m	M3 KN-m	P KN	M2 KN-m	M3 KN-m	P KN	M2 KN-m	M3 KN-m
Story25	-91.92	38.20	35.05	-160.51	-81.11	4.22	-172.77	1.12	-80.58
Story24	-252.40	35.39	28.57	-308.29	-63.00	2.00	-364.45	0.73	-70.89
Story23	-400.11	35.13	28.53	-456.33	-67.02	2.14	-552.70	0.82	-71.03
Story22	-546.56	34.67	27.69	-604.53	-66.11	2.34	-740.66	0.78	-70.10
Story21	-690.81	34.31	27.27	-752.93	-66.29	2.82	-928.49	0.79	-69.33
Story20	-833.74	33.98	26.92	-901.61	-66.22	3.40	-1116.58	0.78	-68.64
Story19	-975.61	33.68	26.65	-1050.59	-66.20	4.13	-1305.27	0.77	-67.97
Story18	-1116.6	33.37	26.41	-1199.87	-66.17	4.97	-1494.86	0.77	-67.32
Story17	-1256.77	33.05	26.18	-1349.47	-66.12	5.91	-1685.62	0.77	-66.65
Story16	-1396.15	32.70	25.96	-1499.4	-66.08	6.92	-1877.79	0.77	-65.95
Story15	-1534.70	32.31	25.74	-1649.71	-66.02	8.00	-2071.6	0.77	-65.20
Story14	-1672.36	31.86	25.51	-1800.43	-65.96	9.11	-2267.23	0.77	-64.39
Story13	-1809.01	31.35	25.28	-1951.64	-65.90	10.23	-2464.88	0.78	-63.47
Story12	-1944.53	30.74	25.03	-2103.43	-65.82	11.32	-2664.69	0.78	-62.45
Story11	-2078.71	30.03	24.76	-2255.9	-65.74	12.34	-2866.79	0.79	-61.27
Story10	-2211.33	29.18	24.46	-2409.2	-65.65	13.25	-3071.25	0.80	-59.90
Story9	-2342.09	28.18	24.12	-2563.51	-65.55	14.00	-3278.11	0.81	-58.29
Story8	-2470.58	26.98	23.71	-2719.07	-65.45	14.51	-3487.31	0.82	-56.36
Story7	-2596.29	25.55	23.21	-2876.15	-65.33	14.74	-3698.72	0.84	-54.02
Story6	-2718.54	23.82	22.54	-3035.13	-65.23	14.58	-3912.06	0.85	-51.15
Story5	-2836.38	21.73	21.61	-3196.44	-65.00	13.94	-4126.86	0.88	-47.58
Story4	-2948.53	19.20	20.38	-3360.63	-65.24	12.72	-4342.4	0.88	-43.12
Story3	-3052.99	16.01	18.06	-3528.37	-63.45	10.79	-4557.61	0.89	-37.16
Story2	-3148.10	12.76	17.35	-3700.47	-70.35	8.66	-4771.21	1.05	-31.95
Story1	-3232.46	2.03	1.78	-3888.06	-52.72	1.18	-4989.11	1.56	-9.46

Table (3.7): Beams maximum bending moments for load combination 3 (tube frame).

Story	Beam49		Beam48	
	S.F	Max.(-ve)B.M	S.F	Max.(+ve)B.M
	KN	KN-m	KN	KN-m
Story25	191.53	-140.00	111.17	197.15
Story24	279.88	-181.18	168.59	233.48
Story23	286.06	-178.78	174.92	246.99
Story22	299.84	-177.48	182.12	255.22
Story21	315.42	-175.46	191.17	267.13
Story20	332.66	-173.04	200.75	279.50
Story19	350.92	-170.16	210.86	292.64
Story18	369.71	-166.86	221.14	306.00
Story17	388.63	-163.12	231.31	319.29
Story16	407.30	-158.93	241.07	332.13
Story15	425.38	-154.26	250.13	344.18
Story14	442.55	-149.10	258.17	355.08
Story13	458.46	-143.43	264.86	364.45
Story12	472.75	-137.22	269.84	371.90
Story11	484.96	-130.44	272.75	376.96
Story10	494.58	-123.04	273.16	379.16
Story9	500.92	-114.99	270.60	377.91
Story8	503.18	-106.24	264.64	372.67
Story7	500.19	-96.73	254.67	362.62
Story6	490.64	-86.41	240.35	347.23
Story5	472.34	-75.20	220.88	325.31
Story4	443.15	-63.05	196.49	296.73
Story3	398.40	-49.61	166.03	259.66
Story2	336.56	-34.75	131.92	214.60
Story1	215.73	-10.06	77.21	150.92

Table (3.8): Beams maximum shear forces for load combination 3 (tube frame).

Story	Beam45		Beam52	
	Max.(-ve)S.F	B.M	Max.(+ve)S.F	B.M
	KN	KN-m	KN	KN-m
Story25	-182.72	-63.20	65.45	-100.60
Story24	-255.06	-152.46	163.72	-99.77
Story23	-249.36	-144.42	173.61	-99.82
Story22	-243.55	-138.54	195.19	-98.26
Story21	-236.92	-131.85	220.81	-96.35
Story20	-229.80	-124.70	247.61	-94.18
Story19	-222.27	-117.21	275.06	-91.87
Story18	-214.36	-109.40	302.54	-89.38
Story17	-206.07	-101.27	329.90	-86.72
Story16	-197.41	-92.84	356.91	-83.84
Story15	-188.36	-84.10	383.47	-80.71
Story14	-178.91	-75.04	409.48	-77.29
Story13	-169.05	-65.67	434.81	-73.52
Story12	-158.78	-55.98	459.37	-69.36
Story11	-148.08	-45.9	482.95	-64.72
Story10	-136.96	-35.67	505.44	-59.55
Story9	-125.43	-25.11	526.33	-53.73
Story8	-113.54	-14.37	545.57	-47.18
Story7	-101.35	-3.55	561.77	-39.78
Story6	-88.98	7.18	575.30	-31.44
Story5	-76.65	17.54	582.15	-22.17
Story4	-64.67	27.11	584.78	-12.07
Story3	-53.69	35.09	570.61	-1.75
Story2	-43.93	41.19	554.06	7.59
Story1	-51.35	28.08	492.39	19.60

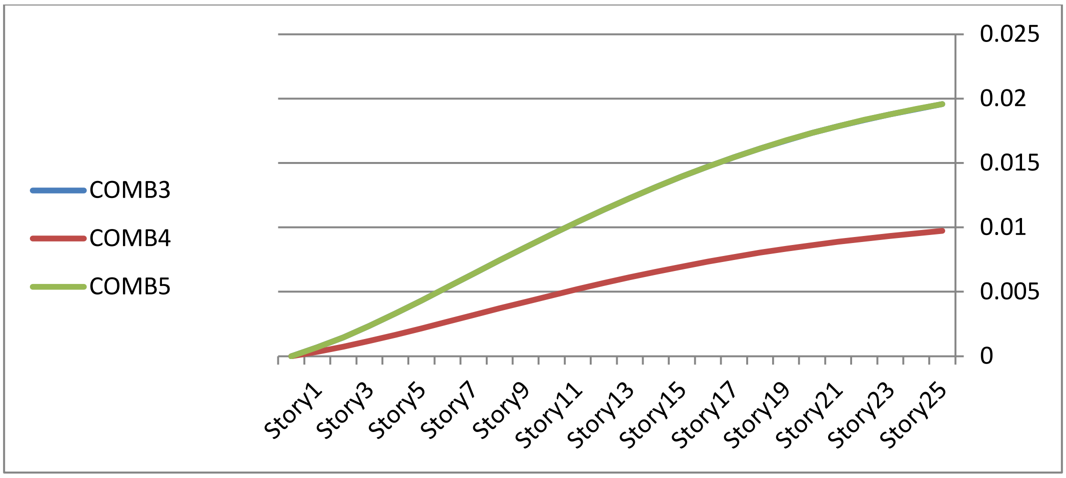


Figure (3.11): The Distribution of Maximum Story Displacement in X-Direction.

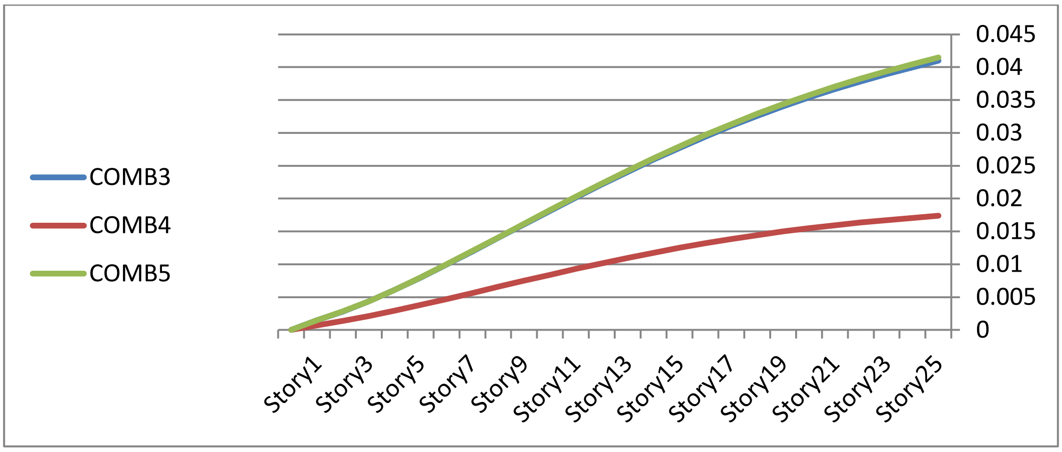


Figure (3.12): The Distribution of Maximum Story Displacement in Y-Direction.

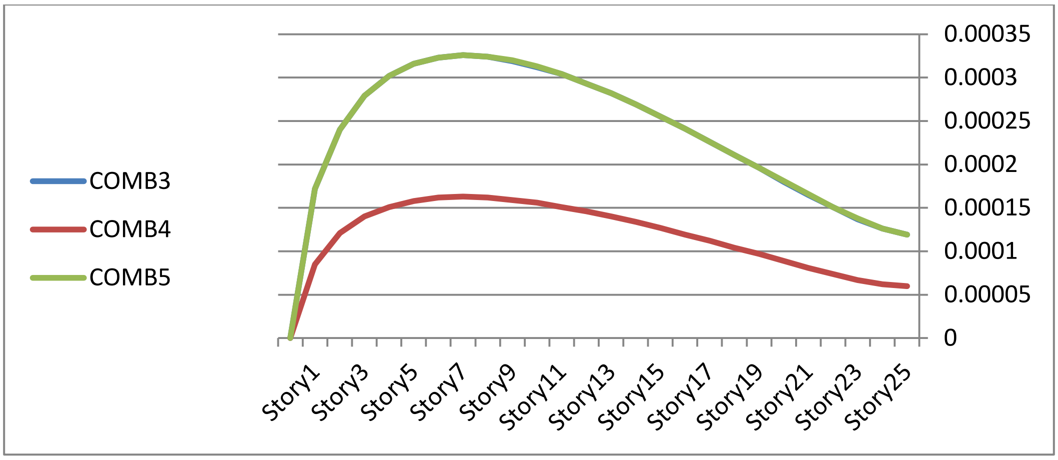


Figure (3.13): The Distribution of Maximum Story Drift in X-Direction.

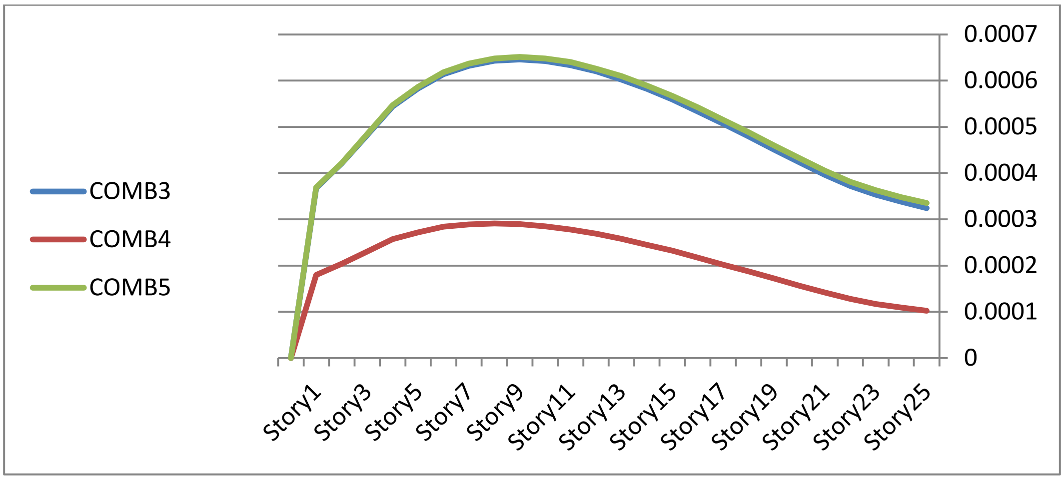


Figure (3.14): The Distribution of Maximum Story Drift in Y-Direction.

3.7 Structural Analysis of Core System of Tall Building

1. Column results:

By the same way used to represent the column forces in above systems, it was taken edge tension, compression, and internal column (C1, C8, C20), respectively. The column results summarised as shown in Table (3.6).

2. Beams forces:

Only the beams had the maximum bending moment and shear forces were tabulated .The beams forces were summarized in Table (3.7).

3. Story displacement:

In order to show the lateral effect occurred due to wind loads on the building, only the story displacements at y-direction for all combinations were depicted at Figures (3.11) – (3.12).Because of the story displacements at Y- direction has greater value than X-direction.

4. Story drifts:

The story drifts considered as a major effective parameter, which increase gradually from top to base of the frame as resulted in Figures (3.13)-(3.14).

Table (3.9): The distribution of axial forces and bending moments on columns for load combination3 (core).

Story	Column 1			Column 8			Column 20		
	P	M2	M3	P	M2	M3	P	M2	M3
	KN	KN-m	KN-m	KN	KN-m	KN-m	KN	KN-m	KN-m
Story25	-195.52	82.14	240.52	-264.96	-136.08	155.59	-376.02	6.11	-15.98
Story24	-395.37	64.33	194.66	-527.97	-102.19	131.71	-748.64	4.67	-10.73
Story23	-592.88	69.14	202.80	-792.34	-110.54	141.15	-1122.28	5.08	-13.55
Story22	-789.30	68.90	200.99	-1057.62	-108.43	146.51	-1496.17	4.91	-13.04
Story21	-983.98	69.80	201.55	-1324.08	-108.90	153.81	-1870.7	4.84	-12.67
Story20	-1176.82	70.46	202.13	-1591.83	-108.69	161.51	-2245.91	4.73	-11.68
Story19	-1367.6	71.16	203.07	-1860.99	-108.62	169.65	-2621.93	4.61	-10.29
Story18	-1556.21	71.85	204.22	-2131.66	-108.46	177.98	-2998.87	4.46	-8.48
Story17	-1742.55	72.55	205.49	-2403.9	-108.27	186.33	-3376.84	4.30	-6.29
Story16	-1926.57	73.26	206.76	-2677.8	-108.04	194.55	-3755.95	4.12	-3.71
Story15	-2108.26	73.97	207.95	-2953.42	-107.76	202.44	-4136.31	3.91	-0.77
Story14	-2287.64	74.71	208.96	-3230.79	-107.44	209.87	-4518.03	3.69	2.52
Story13	-2464.77	75.45	209.73	-3509.96	-107.07	216.60	-4901.21	3.44	6.12
Story12	-2639.75	76.25	209.91	-3790.96	-106.66	222.76	-5285.96	3.15	10.21
Story11	-2812.73	77.06	209.67	-4073.79	-106.18	227.78	-5672.39	2.85	14.52
Story10	-2983.88	77.89	208.85	-4358.45	-105.66	231.81	-6060.6	2.51	19.30
Story9	-3153.46	78.74	207.32	-4644.92	-105.07	234.17	-6450.69	2.15	24.08
Story8	-3321.73	79.64	205.01	-4933.18	-104.42	235.65	-6842.78	1.77	29.76
Story7	-3489.04	80.56	201.81	-5223.18	-103.8	234.15	-7236.95	1.23	34.34
Story6	-3655.8	81.37	197.68	-5514.86	-102.52	233.10	-7633.32	1.34	42.22
Story5	-3822.48	82.90	193.87	-5808.16	-103.94	223.42	-8031.98	-1.66	40.23
Story4	-3989.7	82.70	184.45	-6103.04	-99.21	227.87	-8433.11	2.91	61.91
Story3	-4158.32	80.58	179.26	-6399.36	-99.14	204.45	-8836.72	-4.96	54.58
Story2	-4329.85	114.37	87.70	-6696.93	-119.40	231.74	-9242.82	0.15	51.53
Story1	-4516.5	66.63	218.90	-7009.05	-102.30	589.28	-9665.87	-0.85	467.89

Table (3.10): Beams maximum bending moments for load combination 3(core).

Story	Beam7		Beam11	
	S.F	Max.(-ve)B.M	S.F	Max.(+ve)B.M
	KN	KN-m	KN	KN-m
Story25	108.97	-135.34	7.81	126.06
Story24	121.86	-167.16	20.87	158.24
Story23	120.11	-160.85	20.49	157.02
Story22	121.49	-162.87	22.99	162.87
Story21	122.38	-162.91	25.15	167.84
Story20	123.45	-163.15	27.44	173.15
Story19	124.50	-163.06	29.70	178.35
Story18	125.53	-162.74	31.86	183.33
Story17	126.47	-162.15	33.86	187.93
Story16	127.29	-161.27	35.64	192.02
Story15	127.94	-160.09	37.14	195.46
Story14	128.38	-158.61	38.31	198.11
Story13	128.56	-156.80	39.07	199.81
Story12	128.43	-154.67	39.38	200.46
Story11	127.93	-152.17	39.15	199.86
Story10	127.03	-149.30	38.32	197.86
Story9	125.66	-146.03	36.81	194.29
Story8	123.76	-142.36	34.59	189.03
Story7	121.27	-138.28	31.55	181.88
Story6	118.10	-133.76	27.68	172.81
Story5	114.27	-129.05	22.86	161.52
Story4	109.07	-122.82	17.35	148.60
Story3	103.41	-117.50	10.46	132.48
Story2	96.80	-112.06	2.88	114.75
Story1	86.81	-95.27	-8.52	87.95

Table (3.11): Beams maximum shear forces for load combination 3(core).

Story	Beam15		Beam7	
	Max.(-ve)S.F	B.M	Max.(+ve)S.F	B.M
	KN	KN-m	KN	KN-m
Story25	-39.06	88.45	169.36	-102.78
Story24	-31.394	109.65	178.82	-104.94
Story23	-32.76	105.75	178.61	-105.02
Story22	-32.46	106.45	180.42	-105.66
Story21	-32.38	106.48	181.98	-106.22
Story20	-32.23	106.68	183.61	-106.80
Story19	-32.11	106.75	185.18	-107.39
Story18	-32.05	106.57	186.64	-107.97
Story17	-32.11	106.00	187.94	-108.55
Story16	-32.34	104.88	189.04	-109.13
Story15	-32.80	103.07	189.87	-109.71
Story14	-33.53	100.42	190.40	-110.29
Story13	-34.59	96.76	190.56	-110.87
Story12	-36.03	91.95	190.32	-111.45
Story11	-37.90	85.83	189.62	-112.02
Story10	-40.25	78.29	188.39	-112.60
Story9	-43.14	69.14	186.60	-113.18
Story8	-46.61	58.23	184.19	-113.76
Story7	-50.74	45.38	181.11	-114.33
Story6	-55.58	30.41	177.35	-114.89
Story5	-61.22	13.11	172.81	-115.41
Story4	-67.80	-6.99	167.70	-115.74
Story3	-75.26	-29.67	161.50	-115.49
Story2	-83.84	-55.58	154.72	-114.45
Story1	-88.12	-69.25	145.12	-111.78

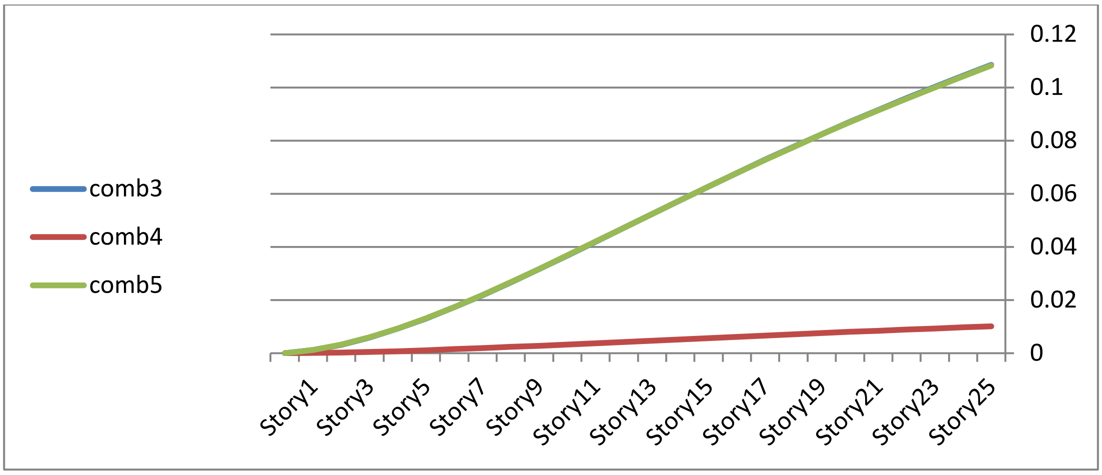


Figure (3.15): The Distribution of Maximum Story Displacement in X- Direction.

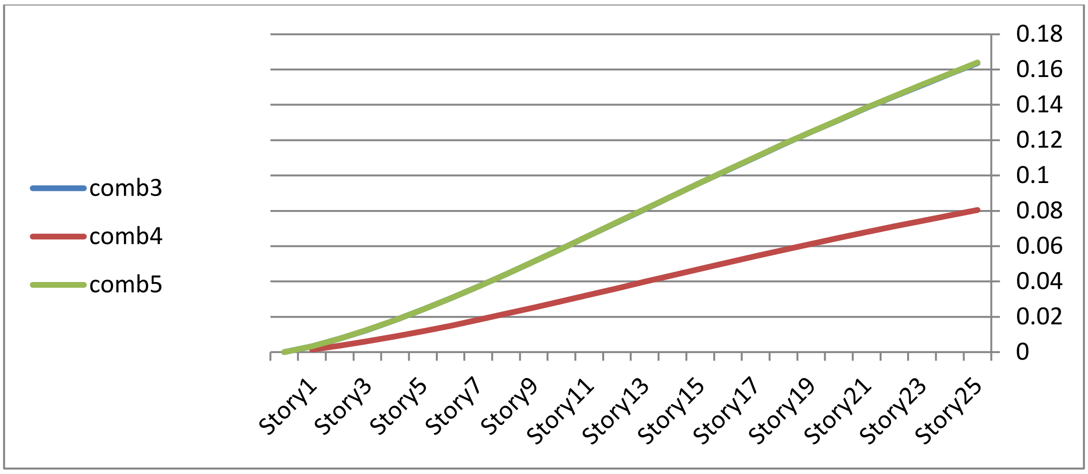


Figure (3.16): The Distribution of Maximum Story Displacement in Y-Direction.

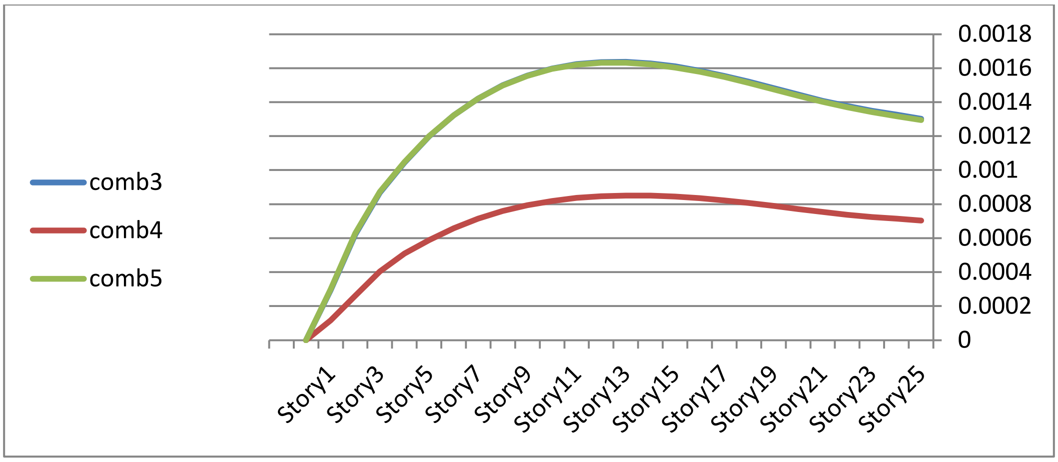


Figure (3.17): The Distribution of Maximum Story Drift in X-Direction.

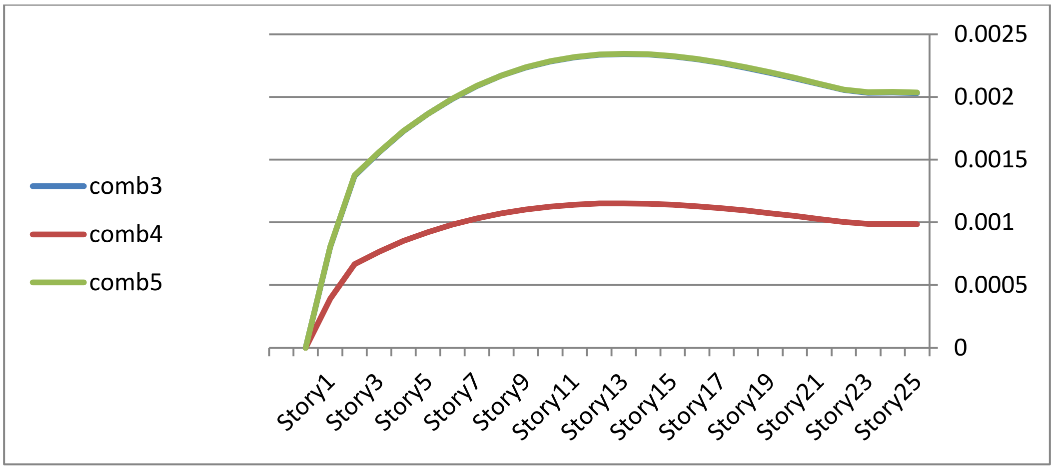


Figure (3.18): The Distribution of Maximum Story Drift in Y-Direction.

CHAPTER FOUR

ANALYSIS RESULTS AND DESIGN OF TALL BUILDING MODELS

4.1. Introduction

In chapter three, the analysis results data were tabulated and viewed in a graphical presentation. In order to make an adequate study, comparison between Shear wall- frame, tube and core systems was made for each system. These results will be presented in charts to get the stress distribution for each frame. The distribution of story displacements charts was created to decide which system has the minimum story displacement values.

4.2. Results Analysis

In order to study the structural behavior of three selected types of tall building frames, it was considered stresses in beams and columns, story displacements and story drifts. As known, each frame was analyzed. In addition, the shear wall - frame results were taken as an example as shown in chapter three. Therefore, beam and column sample of shear wall - frame was picked to show the graphical presentation for the distribution of forces, for the other two systems of tall building.

4.2.1. Beams Forces

For a comparison purpose, (B7) was selected which has the maximum (+ve) bending moment for plan area of rigid frame. The results of bending moments and shear forces were depicted in graphical presentations with respect of the story heights as shown in Figures (4.1) – (4.2).

4.2.2. Column Forces

The distribution of axial forces and moment of the interior column (C20) was represented for all systems as shown in Figures (4.3) – (4.4) Because of shear lag in the frame tube, the axial force in the interior columns will be less than the outer ones. Note that, the M3 is the bending moment about the major axis, and M2 was neglected because it was too small comparing with M3.

4.2.3. Story Displacement

In order to compare between the three systems, the combination that gave the maximum story displacement of the Shear wall - frame system was taken as reference. Then, the comparison of displacements with respect of storey heights was made about x- and y- axis. At top floors in the shear-wall frame, the displacement increase rather rapidly, mainly from the cumulative effect of wall rotation, as depicted in Figures (4.5) – (4.6).

4.2.4. Story Drift

As known, the story drift depended primarily on the story displacements. In addition, the story drift was an indicator to show the acceptance degree of the displacement over the building height. Because of the cumulative rotation up the height in the shear-wall frame, the story drift due to overall bending increases with height. So, the results were slightly the same with the displacements as shown in Figures (4.7) – (4.8).

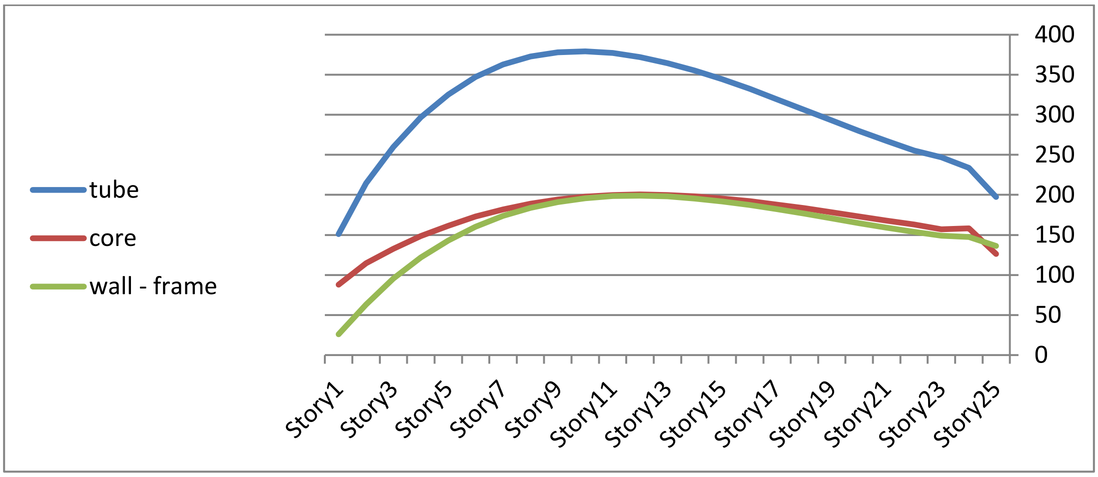


Figure (4.1): Distribution of Bending Moments of B7 for All Models.

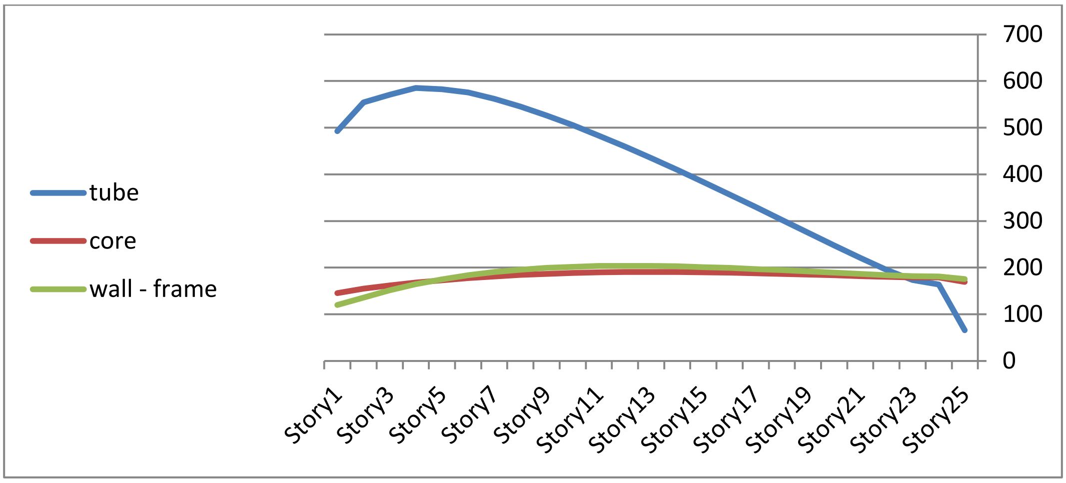


Figure (4.2): Distribution of Shear Forces of B7 for All Models.

.

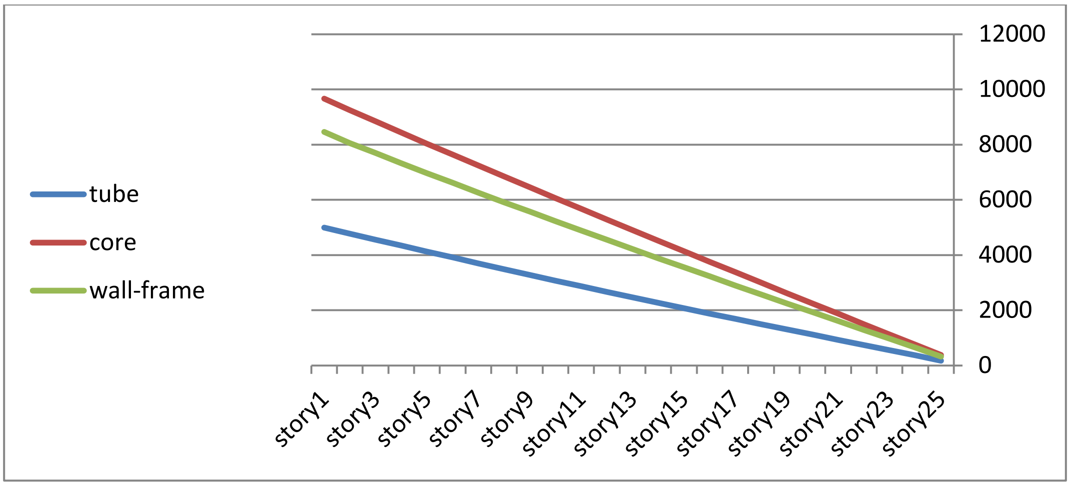
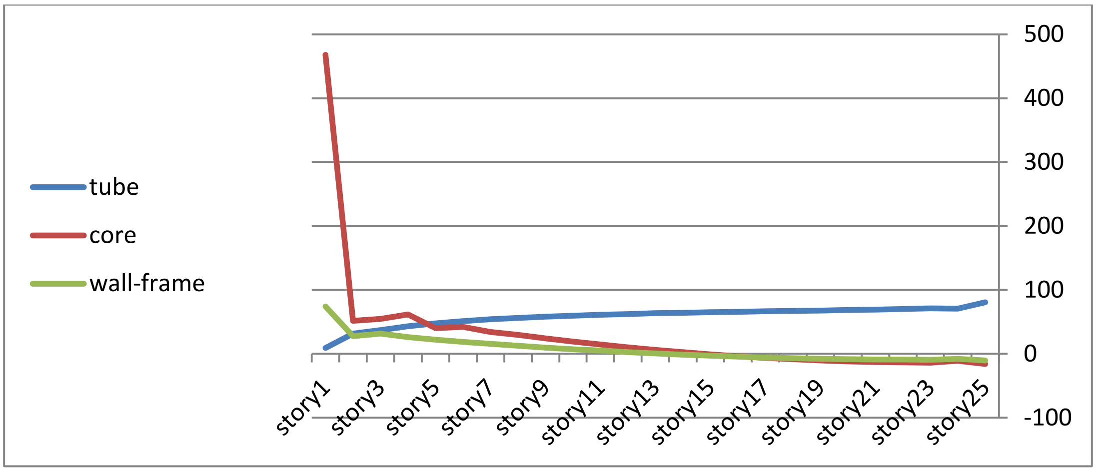


Figure (4.3): Distribution of Axial Forces of C20 for All Models.



Figure(4.4): Distribution of Bending Moments of C20for All Models.

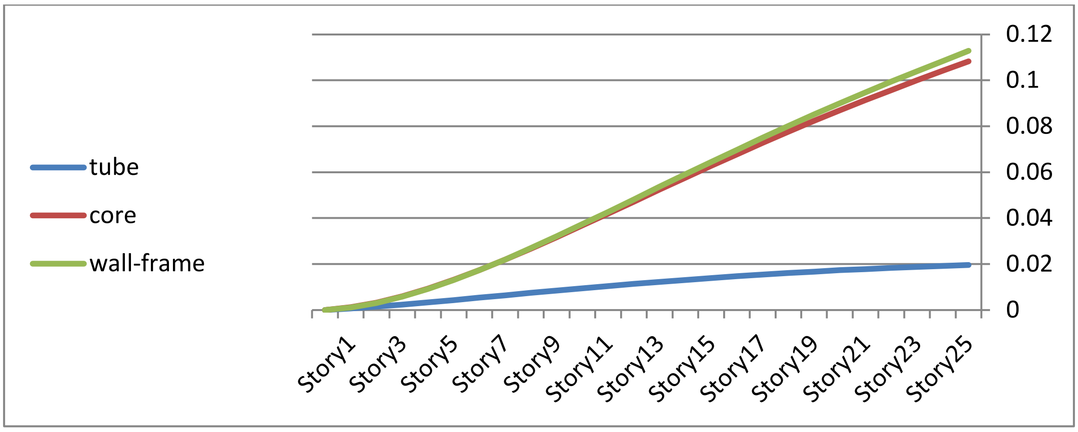


Figure (4.5): The Distribution of Maximum Story Displacements in X-Direction for All Models.

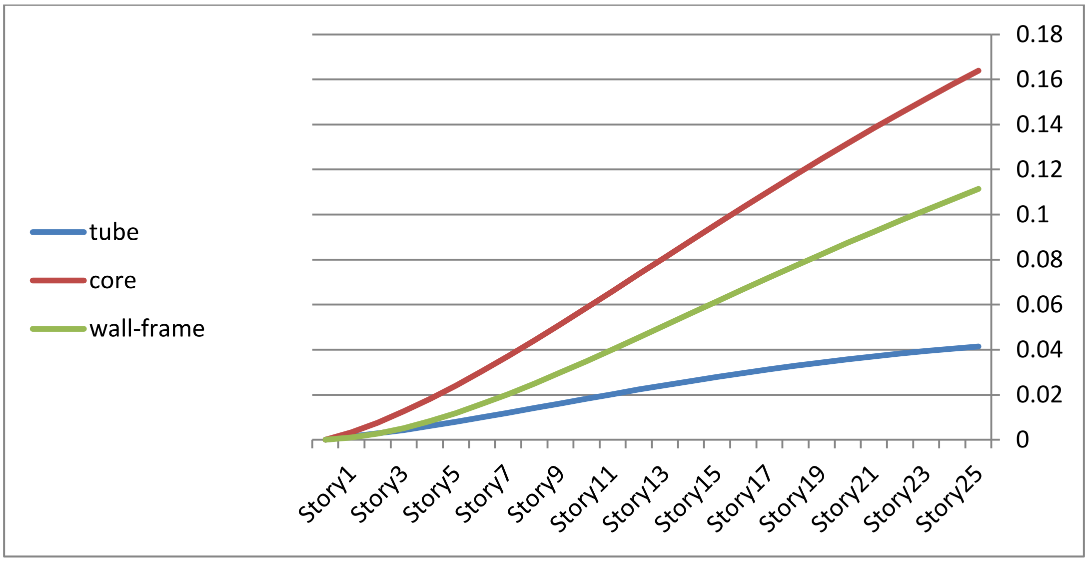


Figure (4.6): The Distribution of Maximum Story Displacements in Y-Direction for All Models.

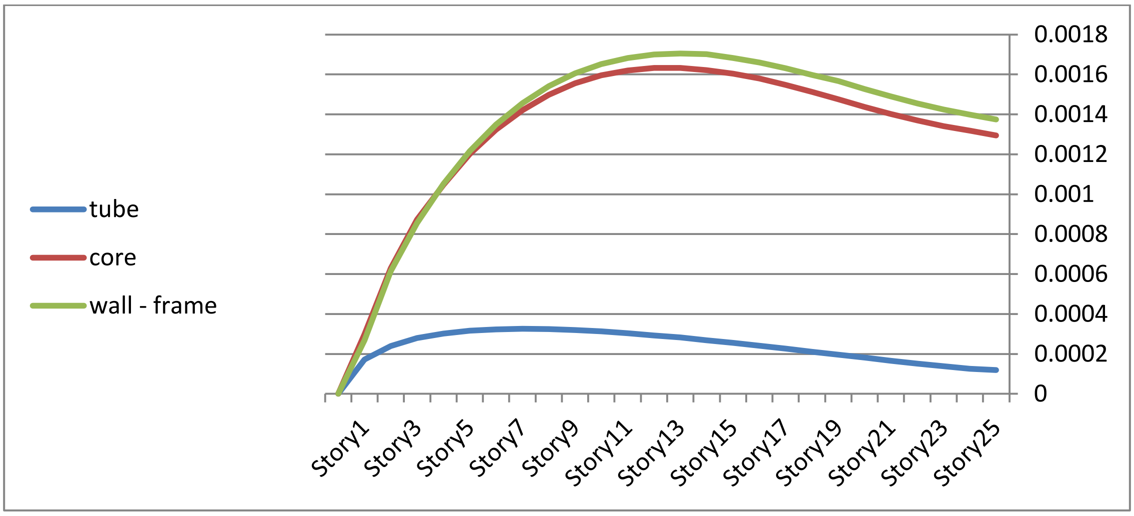


Figure (4.7): The Distribution of Maximum Story Drift in X-Direction for All Models

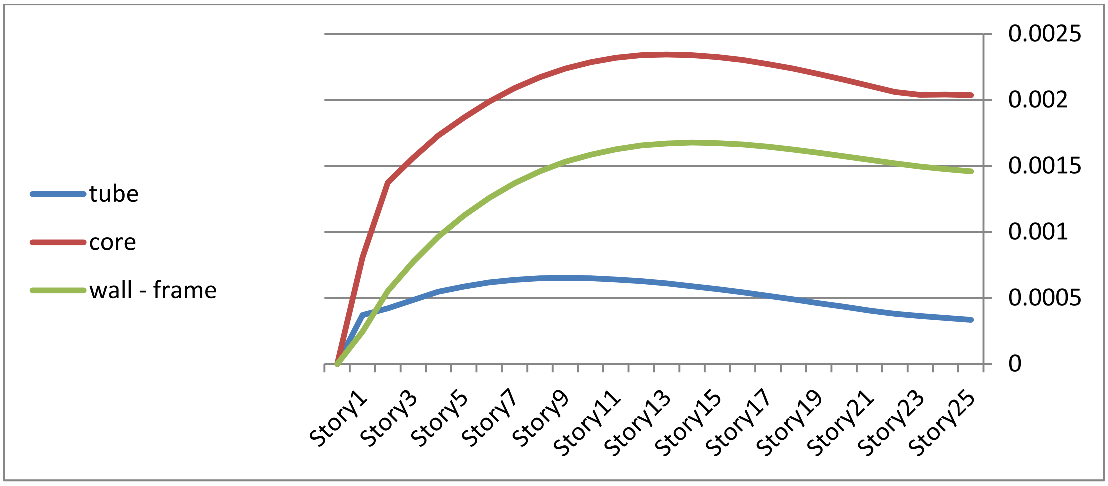


Figure (4.8): The Distribution of Maximum Story Drift in Y-Direction for All Models.

4.3. Design Results:

The design results of column (C1, C8 and C20) in all types frame were presented in Tables (4.1) – (4.3), respectively.

In order to get the most economical frame of the three systems of tall building, it was consider that dimensions building height and applied loads were all the same. The weights of the whole structural elements of all types system frame were presented in Figure (4.9).

Table (4.1): Column 1 Designed Sections.

Story	Shear - Wall Frame	Tube Frame	Core Frame
Story25	0.4×0.9/ 12Ø20	0.45×0.8/8Ø25	0.6×0.9/ 18Ø25
Story24	0.4×0.9/ 12Ø20	0.45×0.8/8Ø25	0.6×0.9/ 18Ø25
Story23	0.4×0.9/ 12Ø20	0.45×0.8/8Ø25	0.6×0.9/ 18Ø25
Story22	0.4×0.9/ 12Ø20	0.45×0.8/8Ø25	0.6×0.9/ 18Ø25
Story21	0.4×0.9/ 12Ø20	0.45×0.8/8Ø25	0.6×0.9/ 18Ø25

Story20	0.4×0.9/ 12Ø20	0.45×0.8/8Ø25	0.6×0.9/ 18Ø25
Story19	0.4×0.9/ 12Ø20	0.45×0.8/8Ø25	0.6×0.9/ 18Ø25
Story18	0.4×0.9/ 12Ø20	0.45×0.8/8Ø25	0.6×0.9/ 18Ø25
Story17	0.4×0.9/ 12Ø20	0.45×0.8/8Ø25	0.6×0.9/ 18Ø25
Story16	0.4×0.9/ 12Ø20	0.45×0.8/8Ø25	0.6×0.9/ 18Ø25
Story15	0.4×0.9/ 12Ø20	0.45×0.8/8Ø25	0.6×0.9/ 18Ø25
Story14	0.4×0.9/ 12Ø20	0.45×0.8/8Ø25	0.75×0.9/ 22Ø20
Story13	0.4×0.9/ 12Ø20	0.45×0.8/8Ø25	0.75×0.9/ 22Ø20
Story12	0.4×0.9/ 12Ø20	0.45×0.8/8Ø25	0.75×0.9/ 22Ø20
Story11	0.75×0.9/22Ø20	0.45×0.8/8Ø25	0.75×0.9/ 22Ø20
Story10	0.75×0.9/22Ø20	0.45×0.8/8Ø25	0.75×0.9/ 22Ø20
Story9	0.75×0.9/22Ø20	0.45×0.8/8Ø25	0.75×0.9/ 22Ø20
Story8	0.75×0.9/22Ø20	0.45×0.8/8Ø25	0.75×0.9/ 22Ø20
Story7	0.75×0.9/22Ø20	0.45×0.8/8Ø25	0.75×0.9/ 22Ø20
Story6	0.75×0.9/22Ø20	0.45×0.8/8Ø25	0.75×0.9/ 22Ø20
Story5	0.75×0.9/22Ø20	0.45×0.8/8Ø25	0.75×0.9/ 22Ø20
Story4	0.75×0.9/22Ø20	0.45×0.8/8Ø25	0.75×0.9/ 22Ø20
Story3	0.75×0.9/22Ø20	0.45×0.8/8Ø25	0.75×0.9/ 22Ø20
Story2	0.75×0.9/22Ø20	0.45×0.8/8Ø25	0.75×0.9/ 22Ø20
Story1	0.75×0.9/22Ø20	0.45×0.8/8Ø25	0.75×0.9/ 22Ø20

Table (4.2): Column 8 Designed Sections.

Story	Wall - Frame	Tube	Core
Story25	0.4×0.9/ 12Ø20	0.45×0.8/ 8Ø25	0.75×0.9/22Ø20
Story24	0.4×0.9/ 12Ø20	0.45×0.8/ 8Ø25	0.75×0.9/22Ø20
Story23	0.4×0.9/ 12Ø20	0.45×0.8/ 8Ø25	0.75×0.9/22Ø20
Story22	0.4×0.9/ 12Ø20	0.45×0.8/ 8Ø25	0.75×0.9/22Ø20

Story21	0.4×0.9/ 12Ø20	0.45×0.8/ 8Ø25	0.75×0.9/22Ø20
Story20	0.4×0.9/ 12Ø20	0.45×0.8/ 8Ø25	0.75×0.9/22Ø20
Story19	0.75×0.9/22Ø20	0.45×0.8/ 8Ø25	0.65×1.0/22Ø20
Story18	0.75×0.9/22Ø20	0.45×0.8/ 8Ø25	0.65×1.0/22Ø20
Story17	0.75×0.9/22Ø20	0.45×0.8/ 8Ø25	0.65×1.0/22Ø20
Story16	0.75×0.9/22Ø20	0.45×0.8/ 8Ø25	0.65×1.0/22Ø20
Story15	0.75×0.9/22Ø20	0.45×0.8/ 8Ø25	0.65×1.0/22Ø20
Story14	0.75×0.9/22Ø20	0.45×0.8/ 8Ø25	0.65×1.0/22Ø20
Story13	0.75×0.9/22Ø20	0.45×0.8/ 8Ø25	0.65×1.0/22Ø20
Story12	0.75×0.9/22Ø20	0.45×0.8/ 8Ø25	0.65×1.0/22Ø20
Story11	0.75×0.9/22Ø20	0.45×0.8/ 8Ø25	0.65×1.0/22Ø20
Story10	0.75×0.9/22Ø20	0.45×0.8/ 8Ø25	0.65×1.0/22Ø20
Story9	0.75×0.9/22Ø20	0.45×0.8/ 8Ø25	0.65×1.0/22Ø20
Story8	0.75×0.9/22Ø20	0.45×0.8/ 8Ø25	0.65×1.0/22Ø20
Story7	0.75×0.9/22Ø20	0.45×0.8/ 8Ø25	0.65×1.0/22Ø20
Story6	0.75×0.9/22Ø20	0.45×0.8/ 8Ø25	0.65×1.0/22Ø20
Story5	0.85×1.0/28Ø20	0.45×0.8/ 8Ø25	0.65×1.0/22Ø20
Story4	0.85×1.0/28Ø20	0.45×0.8/ 8Ø25	0.65×1.0/22Ø20
Story3	0.85×1.0/28Ø20	0.45×0.8/ 8Ø25	0.65×1.0/22Ø20
Story2	0.85×1.0/28Ø20	0.45×0.8/ 8Ø25	0.65×1.0/22Ø20
Story1	0.85×1.0/28Ø20	0.45×0.8/ 8Ø25	0.65×1.0/22Ø20

Table (4.3): Column 20 Designed Sections.

Story	Wall - Frame	Tube	Core
Story25	0.4×0.9/ 12Ø20	0.45×0.8 /8Ø25	0.65×1.0/22Ø20
Story24	0.4×0.9/ 12Ø20	0.45×0.8 /8Ø25	0.65×1.0/22Ø20

Story23	0.4×0.9/ 12Ø20	0.45×0.8 /8Ø25	0.65×1.0/22Ø20
Story22	0.4×0.9/ 12Ø20	0.45×0.8 /8Ø25	0.65×1.0/22Ø20
Story21	0.4×0.9/ 12Ø20	0.45×0.8 /8Ø25	0.65×1.0/22Ø20
Story20	0.4×0.9/ 12Ø20	0.45×0.8 /8Ø25	0.65×1.0/22Ø20
Story19	0.85×1.0/ 18Ø25	0.6×0.9/12Ø25	0.65×1.0/22Ø20
Story18	0.85×1.0/ 18Ø25	0.6×0.9/12Ø25	0.65×1.0/22Ø20
Story17	0.85×1.0/ 18Ø25	0.6×0.9/12Ø25	0.65×1.0/22Ø20
Story16	0.85×1.0/ 18Ø25	0.6×0.9/12Ø25	0.65×1.0/22Ø20
Story15	0.85×1.0/ 18Ø25	0.6×0.9/12Ø25	0.65×1.0/22Ø20
Story14	0.85×1.0/ 18Ø25	0.6×0.9/12Ø25	0.8×1.0/22Ø20
Story13	0.85×1.0/ 18Ø25	0.6×0.9/12Ø25	0.8×1.0/22Ø20
Story12	0.85×1.0/ 18Ø25	0.6×0.9/12Ø25	0.8×1.0/22Ø20
Story11	0.85×1.0/ 18Ø25	0.6×0.9/12Ø25	0.8×1.0/22Ø20
Story10	0.85×1.0/ 18Ø25	0.6×0.9/12Ø25	0.8×1.0/22Ø20
Story9	0.85×1.0/ 18Ø25	0.6×0.9/12Ø25	0.8×1.0/22Ø20
Story8	0.85×1.0/ 18Ø25	0.6×0.9/12Ø25	0.8×1.0/22Ø20
Story7	0.85×1.0/ 18Ø25	0.6×0.9/12Ø25	0.8×1.0/22Ø20
Story6	0.85×1.0/ 18Ø25	0.6×0.9/12Ø25	0.8×1.0/22Ø20
Story5	0.85×1.0/ 18Ø25	0.6×0.9/12Ø25	0.8×1.0/22Ø20
Story4	0.85×1.0/ 18Ø25	0.6×0.9/12Ø25	0.8×1.0/22Ø20
Story3	0.85×1.0/ 18Ø25	0.6×0.9/12Ø25	0.8×1.0/22Ø20
Story2	0.85×1.0/ 18Ø25	0.6×0.9/12Ø25	0.8×1.0/22Ø20
Story1	0.85×1.0/ 18Ø25	0.6×0.9/12Ø25	0.8×1.0/22Ø20

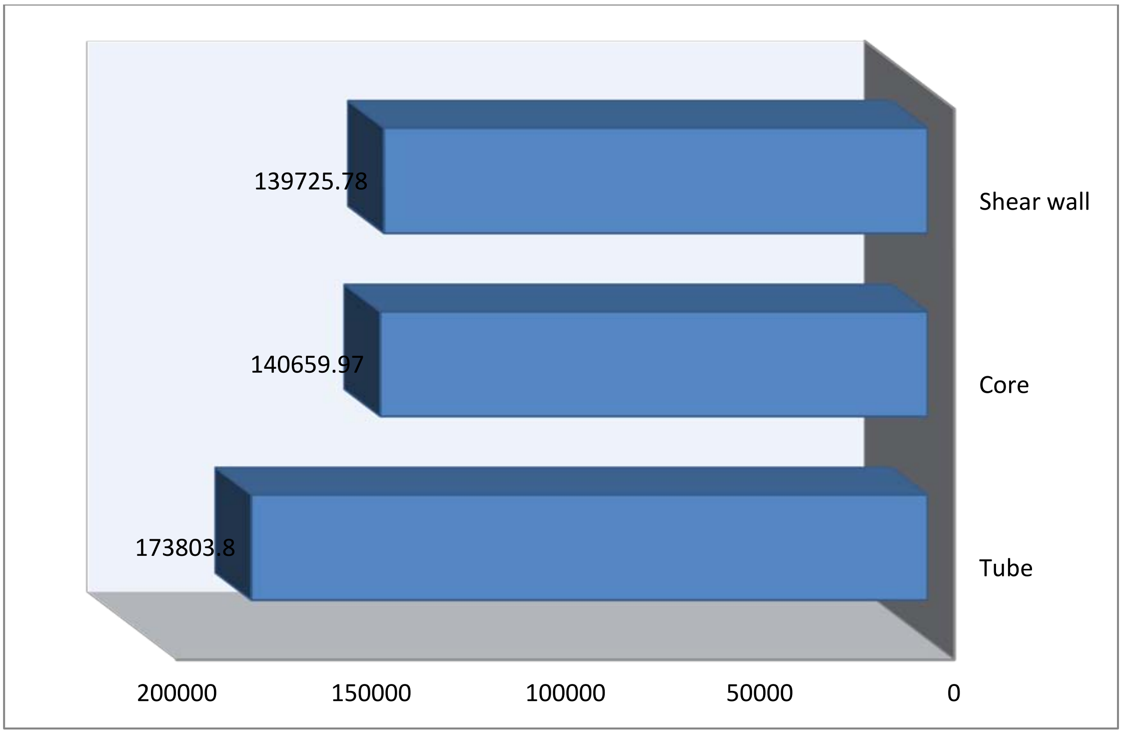


Figure (4.9): The total weights of tall building frames.

CHAPTER FIVE

CONCLUSION AND RECOMMNDATIONS

5.1. Conclusion

The drift and lateral displacement results essentially were depending on the type of tall building frame system. By assigning the same dimensions for all models and subjecting to equal loads distributions over the buildings that made an adequate comparing. Also, changing the structural frames form types led to different analysis.

1- In shear wall- frames, generally, the designed forces were less comparing with the two remains types of structural frames. Therefore, the designed sections were lesser too. But, there was a slightly increasing in drift and lateral displacement values for all other frames.
2- The tube frame were have a high degree of stability resulted as decreasing of drift story and lateral displacement values. Whereas the designed sections were increased because of the massive dead load come from shear-walls.
3- The stability factor values of core frames were between the tube and shear wall- frames.

In order to find the best economical frame of all studied frames, after designing the frames the loads were released, and calculate the total self-weight of each frame as shown in chapter 4. Finally, then the most economical one that which has the minimum weight.

5.2. Recommendations:

Recommendations were summarized as follows:

1. Increasing the height of the building led to changing all loads factor and then redistributes the designing force over the frame members that made a possibility for another structural frame be more suitable.
2. There were many suggestions stated that, if the concrete used in the lower floors and core of the tall building and the structural steel used to build the upper floors, that will significant a huge capacity of resisting the different loads.
3. In actual designing cases of tall building the effect of earthquake loads shouldn’t be ignored.
4. One of the most important concepts to choose any system was to across the architectural requirements and allows the structural efficiency of the building.

References

1. Taranath, B. (2009) Reinforced concrete design of tall buildings. 1st Edition. United States: Taylor & Francis, Inc.
2. Smith, B. S., Stafford-Smith, B. and Coull, A. (1991) Tall building structures: analysis and design. United States: Wiley, John & Sons, Incorporated.
3. John D.Holmes (2007) Wind loading of structure.Second Edition. London and New York:Taylor& Francis.
4. Taranath, B. (2004) Wind and earthquake resistant building: structure analysis and design. Los Angeles, California:John A. Martin & Associates.
5. Taranath, B. (2011) Structural analysis and design of tall buildings: steel and composite construction. United States: CRC/Taylor & Francis.

Appendix A

Terrain roughness

1- The roughness factor, C_f (Z), accounts for the variability of the mean wind velocity at the site of the structure due to:

The height above ground level

The ground of roughness of the terrain upwind of the structure in the wind direction considered

2- The terrain roughness to be used for a given wind direction depends on the ground roughness and the distance with uniform terrain roughness in an angular sector around the wind direction. Small areas (less than 10% of the area under consideration) with deviating roughness may be ignored. See figure 4.1.

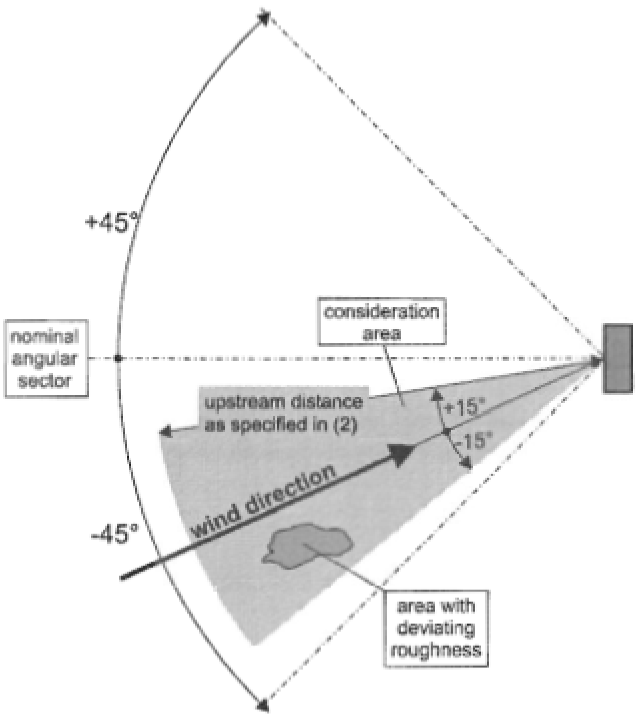


Figure 4.1 — Assessment of terrain roughness

3- When a pressure or force coefficient is defined for a nominal angular sector, the lowest roughness length within any 30° angular wind sector should be used.

4- When there is choice between two or more terrain categories in the definition of a given area, then the area with the lowest roughness length should be used.

Peak velocity pressure

For flat terrain where $C_o(z) = 1$, the exposure factor $C_e(z)$ is illustrated in Figure 4.2 as a function of height above terrain and afuction of terrain category.

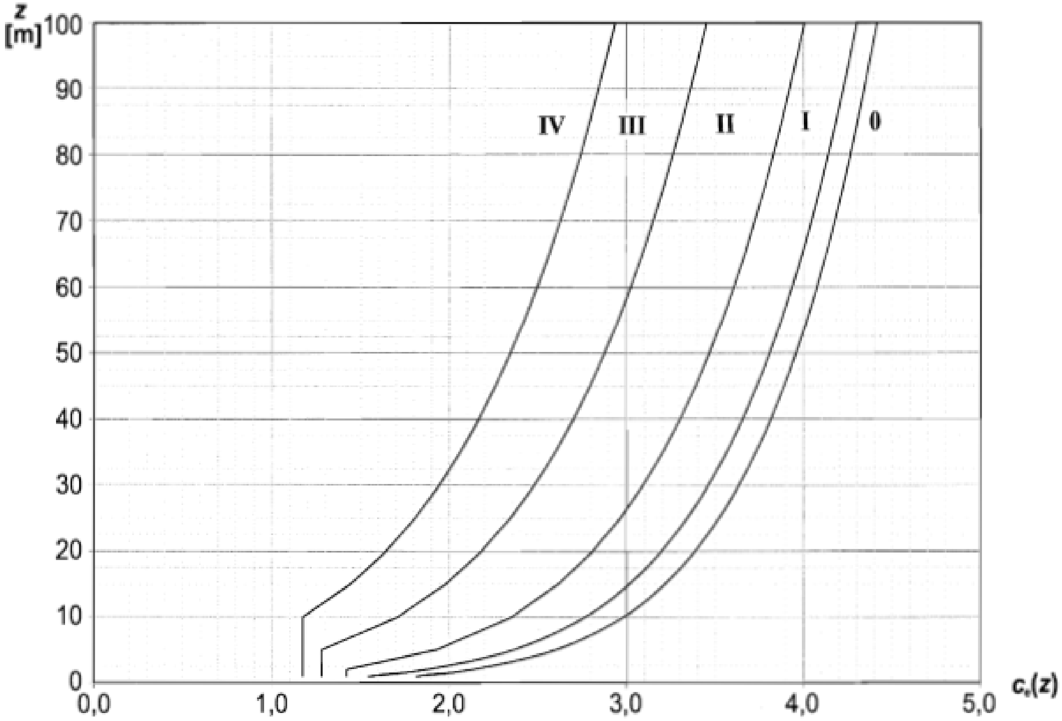


Figure 4.2 — Illustrations of the exposure factor $c_e(z)$ for c_O=1,0, k_I=1,0

Wind actions

Wind pressure on surfaces

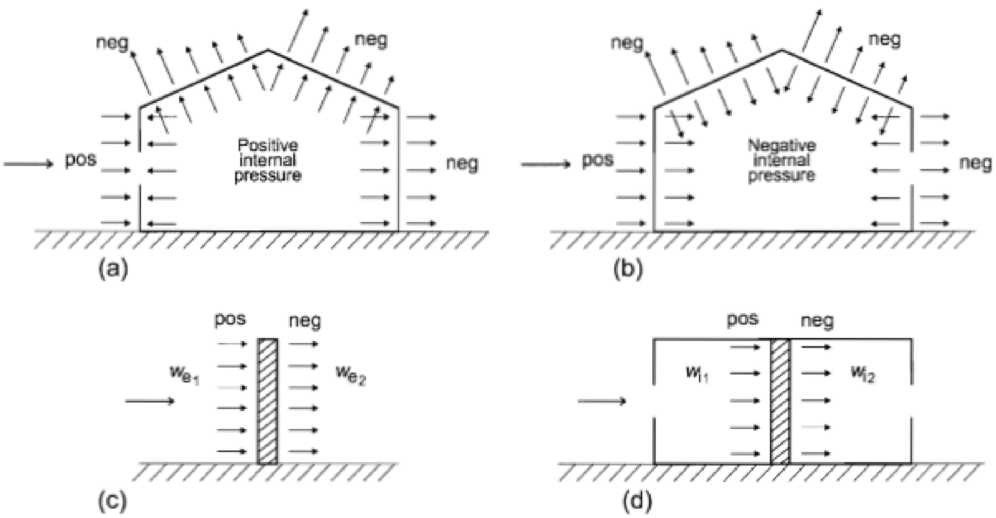


Figure 5.1 — Pressure on surfaces