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Sudan University of Science and Technology

College of Graduate Studies

Study and Investigation of Tall building Design Optimization

دراســـــــة وفحص أمثلية تصميــــم المبانى العالـــــــية

A thesis submitted in partial fulfillment of the requirements for the degree of Master of Science in civil engineering [structures]

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Dedication

I would like to dedicate this work to my parents, brothers Sisters, friends, and everybody who taught me a letter for their endless and generous support,,,,

Eltahir

Acknowledgements

All praise be to **ALLAH** Subhanahu wa ta'ala for bestowing me with health, opportunity, patience and knowledge to complete this work.

Acknowledgement is due to Sudan University of Science and Technology for giving me the opportunity to pursue my graduate studies.

I acknowledge, with deep gratitude and appreciation to my supervisor **Dr. Ali Hussein Mohammed Ali** for his encouragement, remarkable assistance and continuous support.

Abstract

In tall buildings where the design is extremely affected by the lateral loads, wind and seismic, which could result excessive drifts and cause damage of nonstructural components. The challenging of designing tall building is how to create and optimize lateral loads resistant systems with economic considerations.

There is a lot of lateral loads resistant systems has been used for different height of buildings, and each one has economical limit, above that limit the controlling of serviceability limits is highly expensive.

The selection of lateral loads resistant system for particular building, is not enough, another effort should be done to optimize the selected system as the location and configuration of that system (i.e. shear walls location, and configurations of a rigid frame).

In this research, computerized models for different lateral loads resistant systems has been developed. Each system with variable configurations such as shear walls system which was configured for different locations with respect to the center of mass, and the results compared with each other's, it was found that the optimum location of shear walls is near to (C.M). For rigid frame the drift affected extremely by major axes of edge columns. For outriggers it's found that the optimum location ranges from 0.4H to 0.6H without significance change in drift.

The drawn recommendations for shear walls to be near the center of mass, with respect to the rigid frame is highly recommended to locate the major axes of edge columns with respect the lateral load. For the outrigger system it is better to use where the wind load governing the lateral loads design.

الملخص

فى المبانى العالية يتأثر التصميم بصورة كبيرة بالأحمال الجانبية من رياح وزلازل، التى يمكن أن تؤدى الى زيادة كبيرة فى الإنحراف الجانبى للمبانى مما قد يؤدى الى تهشيم الأعضاء غير الإنشائية، فالتحدى الكبير فى تصميم المبانى العالية هو كيفية وضع نظام إنشائى لمقاومة الأحمال الجانبية مع الأخذ فى الإعتبار الجانب الإقتصادى.

توجد العديد من أنظمة مقاومة الأحمال الجانبية التى يمكن إستخدامها فى المبانى، وكل نظام لديه حد معين للارتفاع ، فوق ذلك الارتفاع تكون تكلفة التحكم فى حدود الإستخدامية مكلفة جداً.

إختيار النظام المناسب لمقاومة الاحمال الجانبية لمبنى معين ليس كافيا، ولذلك لابد من جهد أخر يجب أن يبذل فى تشكيل وأمثلة النظام المختار، كالموقع والشكل المناسبين (كمثال لذلك موقع حوائط القص أو التشكيل المناسب للإطار الصلب).

فى هذا البحث تمت نمذجة حاسوبية لعدة مبانى استخدمت فيها أنظمة مختلفة لمقاومة الاحمال الجانبية، كل نظام مقاوم للأحمال الجانبية نمذج بعدة طرق مختلفة، كما هو الحال فى نماذج حوائط القص التى نمذجت بعدة مواقع مختلفة منسوبة الى مركز الكتلة، وقورنت النتائج مع بعضها، ووجد ان أمثل موقع لحوائط القص هو بالقرب من مركز الكتلة ،أما بالنسبة للإطارات الصلبة وجد أن الإنحراف يتأثر بشكل كبير بوضع المحاور الرئيسية للأعمدة الموجوده فى المحيط، اما بالنسبة لنظام المداد فوجد أن الموقع الأمثل يتراوح ما بين 40%الى 60% من إرتفاع المبنى دون اي فوارق تذكر فى الإنحراف.

وقد أوصت الدراسة بوضع حوائط القص بالقرب من مركز الكتلة ما لم تكون هنالك اعتبارات معمارية، بالنسبة للإطارت فاوصت بتشكيل محدد للاعمدة، اما لنظام المداد فالأفضل أن يستخدم فى المناطق التى تحكم أحمال الرياح فيها الاحمال الجانبية.

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List of symbols

P: Pressure

C_e: Gust factor coefficient.

C_q: Pressure coefficient.

q_s :Wind stagnation pressure UBC, dynamic pressure BS.

I_w: Importance Factor.

ρ: Density of air, redundancy factor.

V_e: The effective wind speed.

S_b: The terrain and building factor.

V_s: Site wind speed.

V_b: The basic wind speed.

S_a: An altitude factor.

S_d: A direction factor.

S_s: A seasonal factor.

S_p: A probability factor.

p_e :The pressure acting on the external surface.

C_{pi}: The internal pressure coefficient for the building

Ca: The size effect factor for internal pressures BS.

F: Force.

m: Mass.

a: Acceleration.

CR: Center of rigidity.

CM: Center of mass.

K: Stiffness.

T: Period.

R: Over strength factor.

N_a, N_r: Near-source factors.

S_A, S_B, S_C, S_D, S_E, and S_F: Soil profiles.

Z: Zone factor.

C_a,C_v: Seismic coefficients.

W: Seismic dead loads.

V: Base shear.

H: Height.

F_t: Force at top.

F_x: Force at level x.

M_x: Moment at level x.

δ: Lateral displacement.

F_{cu}: Concrete compressive strength.

F_y: Reinforcements tensile strength.

A_c: Concrete area.

A_{sc}: Reinforcement area.

N, F_Z: Axial load.

N_u: Ultimate capacity.

M_X, M_Y: Moments respect x and y.

E: Modulus of elasticity for concrete or steel.

W_x, w_y: wind loads in x and y direction.

S_X, S_Y: seismic loads obtained response spectrum.

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Chapter One

Introduction

Chapter One

Introduction

1.1 General

Tall building and towers have fascinated the mankind from the begging of civilization, their construction being initially for defense and subsequently for ecclesiastical purposes, the growth of modern tall buildings has been largely for commercial and residential purpose, but various other social and economic factors, Such as increase in land values and higher density of population, have also contributed to an increase in the number of tall buildings, and in some cities due to topographical restrictions make the tall building the only feasible solution for housing needs.

It is difficult to distinguish the characteristics of a building which categorize it as tall. After all, the outward appearance of tallness is a relative matter. In a typical single-story neighborhood, a five-story building may appear tall. A 50-story building in a city may be called a high-rise, but the citizens of a small town may point proudly to their skyscraper of six stories.

From the structural design point of view, it is simpler to consider a building as tall when its structural analyses and design are in some way affected by the lateral loads (wind and earthquake), particularly the sway caused by such loads. Sway or drift is the magnitude of the relative lateral displacement between a given floor and the one immediately below it. As the height increases, the forces of nature particularly due to wind, begin to dominate.

The effect of lateral load is increase with height linearly due to the nature of wind loads (velocity profile), it could cause excessive drift which will cause damage for nonstructural elements (cladding, doors, and windows) and structure does not meet the serviceability requirements, human comfort, the second effect due to lateral loads is when flexible structures are subjected to lateral forces, the resulting horizontal displacements lead to additional overturning moments because the gravity load is also displaced, it's called P–Δ Effect and it was considered as one of modes of collapse.

A lot of lateral load resistance systems has been developed to provide conservative design, with relative goals as in seismic design, the codes provision require that structures should be able to resist minor earthquake without any damage, moderate earthquake with negligible structural damage and some nonstructural damage, major earthquake with some structural and nonstructural damage but without collapse, Mainly there are two types of lateral load resistance systems, first the shear wall system, second the moment resistant frame which the resistance provided by the beams and columns and their connections, the others systems called dual systems, because the lateral load is resisted by shear walls and moment frame at the same time.

1.2 Research Problem

The optimization process focusing in some objectives as minimum weight, and optimal sizing of structural sections (minimum stiffness required), and minimum cost and so on. These objectives were subjected to a lot of constrains; the maximum permitted stress (Ultimate Limits State), the permitted deflections (vertical movements), the lateral allowed drift (horizontal movement), and other design codes requirements.

There is a lot of factors affects the optimization process if one is going to consider the construction stage, the cost and other factors (availability of resources), construction ability (common practice, not common practice, limited contractors) which will vary from country to another, and make the optimization process relative matter, The optimization from the structural point of view will only be considered.

In the tall building due to dominate of lateral loads, the main objective is to find optimum lateral load resistance system for structures under consideration which will be subjected to lateral drift limits and inter-story drift limits (the story drift relative to the below one), the other objective is to minimize the gravity load as possible.

This research concern in to find which is optimum the shear wall system or the moment frame, study and investigate the choices in materials and some techniques to minimize the self-weight , for shear wall try to find the optimum location and shape.

Study and investigate the optimum systems (configuration, location) for irregular structures.

1.3 Research Objectives

1. To find the optimum location of shear walls for regular structures.
2. To find applicable configuration of columns layout for rigid frames.
3. To compare between shear walls system and rigid frame.
4. To evaluate the efficiency of outrigger system and to find the optimum location of the outrigger system for vertical irregular structures due to difference in masses (located on top).
5. To find the optimum location of outrigger system for vertical irregular structures due setback.

1.4 Research Methodology

In this research the structural aspect only will be considered , the modeling of the structures under consideration has been made by using sophisticated computer package ETABS (Extended Three Dimensional Analysis of Building System) which is based on a finite element method, the assumptions of materials densities and load intensity will be according to the British standards, the preliminary sizing of beams and slabs would be chosen to satisfy the serviceability limits, and sizing of columns according to tributary area.

For earthquake analysis due to presence of irregularity; the modal response spectrum will conduct according to Uniform Building Code (UBC-97), to check the validation of the program. Another program called SAP 2000 v 16 has been used to check some results randomly.

Iterative method will be adopted to find the optimum configuration and location of lateral loads resistance systems (shear walls, outriggers), for shear walls will start from location very near to center of mass and far away. For rigid frame; several models would be examined some with square columns and another with rectangular columns with different layouts. For outriggers start from optimum theoretical location for regular structures.

For comparison requirements the lateral drift, inter-story drift ratio, and the structural plan density index (which is define as the total area of vertical structural elements divided by gross floor area of footprint of the building) at ground level-will be used .

1.5 Research Organization

This research has been organized as follows:

1. Chapter one presents the general introduction, research problem, the objectives & methodology.
2. Chapter two contains literature review for loads, floor systems, special provisions for wind and seismic loads and lateral loads resistant systems.
3. Chapter three contains the cases study for shear walls, the rigid frame and the outriggers.

4. Chapter four presents the comparison and the discussion between results obtained.
5. Chapter five presents the conclusions and recommendations.

Chapter Two

Literature Review

Chapter Two

Literature Review

2.1 Forward

Safety, functionality, economy, and nowadays, satisfying the design requirements are the principal design objectives. Safety is established by demonstrating that the designed system can withstand the code stipulated loads without collapse and serves as a guarantee of a defined level of performance within the range of loading specified in the applicable code. To establish "safety," it is sufficient to demonstrate that under the code-stipulated loading conditions, the structure can develop an uninterrupted load path–from the point of load application to the foundation–capable of sustaining the applied load and all corresponding actions generated in the structure. Toward this effort the common design procedure for safety aims to ensure two criteria–that an envisaged load path is adequate, and that on demand it would be mobilized. [1]

The adequacy of a load path is implemented by ensuring that, at any point along its path, it can withstand the actions occurring at that point. In design practice, adequacy is determined for only one engineer-selected load path, generally referred to as the "structural system." The engineer selected load path is a "designated path," meaning that the natural load path of the loads may be different from the path selected by the engineer. The designated load path design provides an acceptable design, as long as the engineer can demonstrate that it is adequate.

A structures natural load path is generally more economical than other load paths, because the load always tries to follow a path of least resistance. [1]

2.2 The Floor Systems

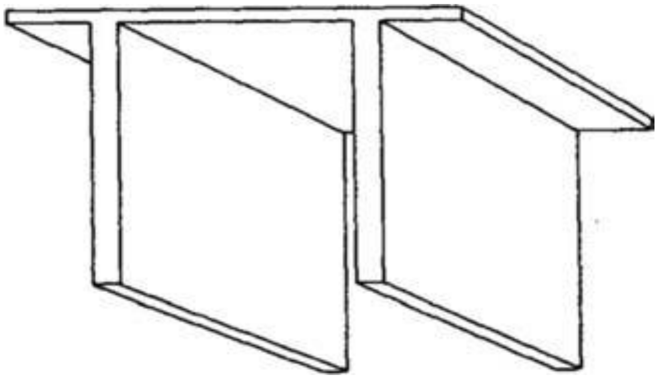
The desire to minimize dead loads is not unique to concrete floor systems only but is of greater significance because the weight of concrete floor system tends to be heavier than steel floors and therefore has a greater impact on the design of vertical elements and foundation systems. Another consideration is the impact of floor depth on the floor-to-floor height. Thus it is important to design a floor system that is relatively of lightweight without being too deep. [1]

However there is a lot of floor systems used, the one way solid slab, the two way solid slabs, the flat slab, the flat slab with column head, the flat slab with drop panels, the ribbed slab, the waffle slab, and the pre- stressed slabs. The selection of the system for a particular building depends on several factors, architectural considerations, the length of spans, and other requirements adopted by design codes like limitation of live loads intensity.

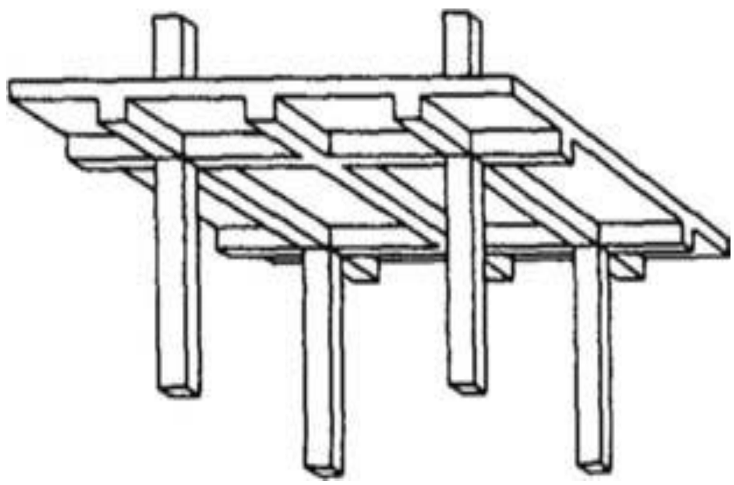
2.2.1 The Solid slab

This system also called the slab with beam –sometimes rested on walls, it has the ability to withstand heavy live loads, the loads transfer from the slab to the beam according to the ratio of the long span to the short, if the ratio exceeds the two the slabs classified as one way slab and the loads transfer directly to the long beams where the others received the loads as reactions from the first ones, the design will consider one way and providing minimum area of steel for the other way. If the span ratio is less than two then the slab is classified as a two way slabs, the design must consider the two way.[2]

From the construction point of view the solid slab is un-favor choice, due to the relative high cost of form work, time of construction, also from the structural point of view due to the increase in the story height which will lead to gathering a lot of wind loads. Figure 2.1 show the solid system.



(a) The One way



(b) The Two way

Figure (2.1): The solid slab

2.2.2 The Flat slab

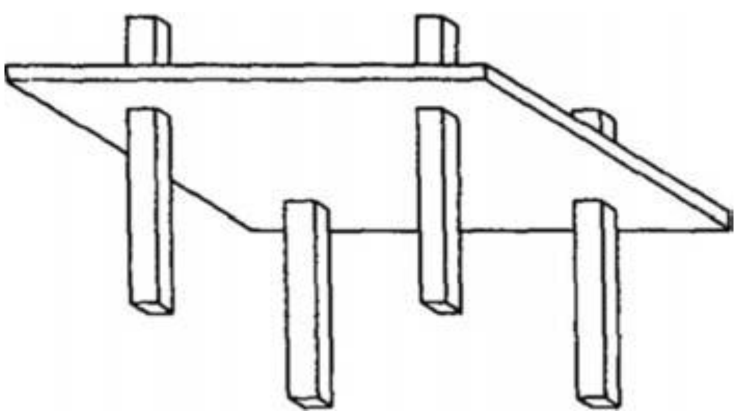
The Concrete slabs are often used to carry vertical loads directly to the walls and columns without the use of beams and girders. Such a system is called a flat slab (Figure2.2) and it is used where spans are not large and loads are not heavy as in apartments and hotel buildings.

The Flat slab is the term used for a slab system without any beams, although column patterns are usually on a rectangular grid. Flat slab can be used with irregularly spaced column layouts. They have been successfully built using columns on triangular grids and other variations.

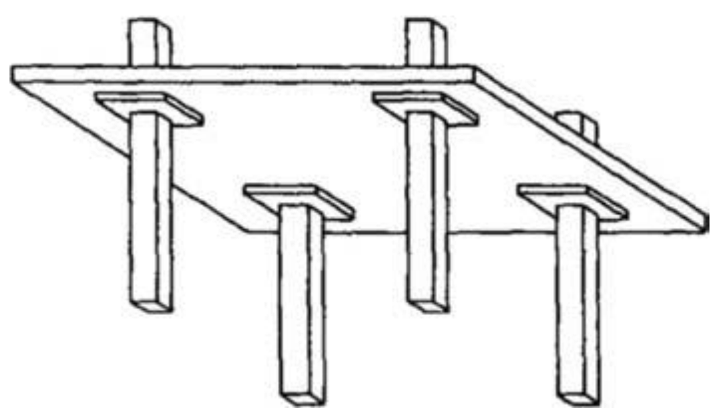
The critical issue in design of flat slab is the punching shear, to overcome this problem drop panels and column heads (capitals) are used to reduce the effective spans which will lead to minimize the bending moment, increase the effective depth and the effective perimeter.

The flat slabs have some structural advantage it was always provide the concept of the strong column - weak beam which is required by most of design codes (ACI stated that the stiffness of all beams in connection must be less the 80% of the column stiffness).

From construction point of view, it’s easy to make the formwork, repeating the same layout from bay to bay for each floor, and from floor to floor to roof, permits a production line work flow and optimum labor productivity. The same equipment can be recycled quickly from one finished area to begin another floor. [1]



(a) The flat slab without drop

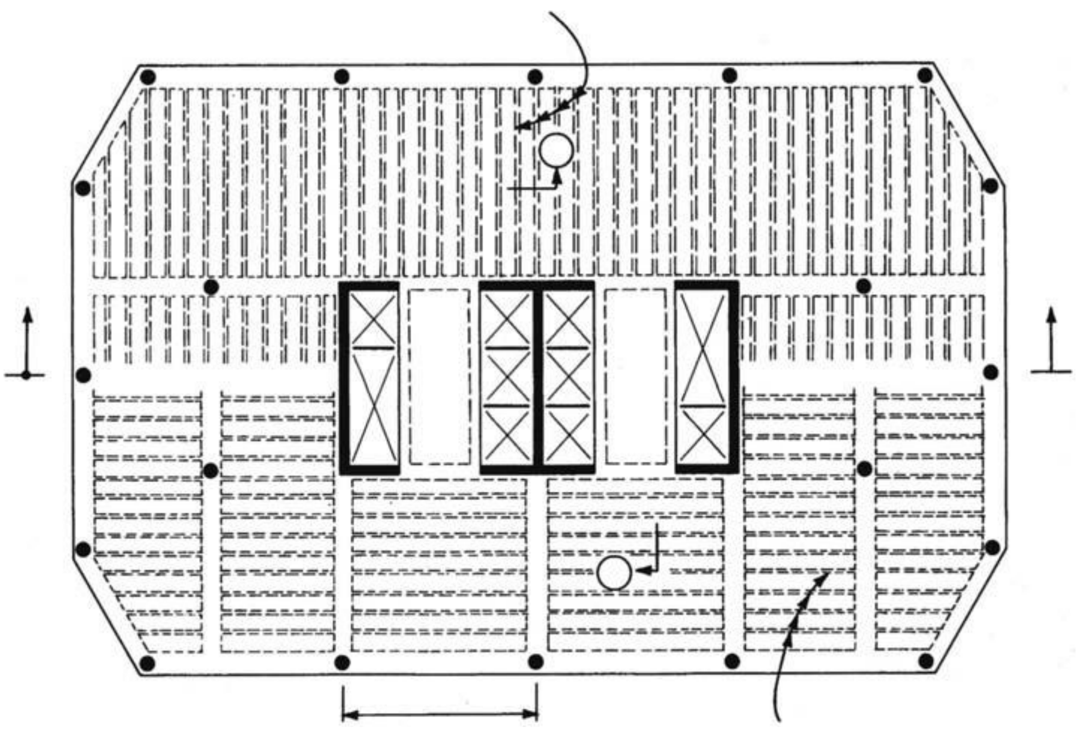


(b)The flat slab with drop

Figure (2.2): The flat slab

2.2.3 The One-way concrete ribbed slabs

This system also referred to as a one-way joist system it is one of the most popular systems for high rise office building construction. The system is based on the well founded premise that concrete in a solid slab below the neutral axis is well in excess of that required for shear and much of it can be eliminated by forming voids. The resulting system shown in Figure (2.3) has voids between the joists made with removable forms of steel, wood, plastic, or other material. The joists are designed as a one-way T-beams for the full-moment tributary to its width. It is a standard practice to use distribution ribs at approximately 3.0 m centers for spans greater than 6 m. For maximum economy of formwork, the depth of beams and girders should be made as the same as for joists.[1]



(a) The Plan of ribs

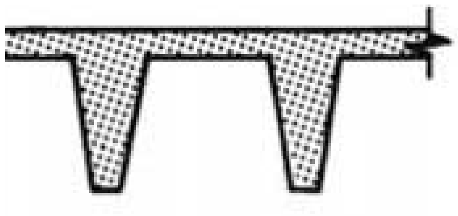
(b) 

Figure (2.3): (a) Plan (b) Section show the ribs (joist)

2.2.4 The Waffle slab

This system also called a two-way joist system (Figure 2.4) is closely related to the flat slab system.To reduce the dead load of a solid slab construction, metal or fiberglass domes are used in the formwork in a rectilinear pattern, as shown in Figure (2.4). Domes are omitted near columns resulting in solid slabs to resist the high bending and shear stresses in these critical areas.

In contrast to a joist which carries loads in a one-way action, a waffle system carries the loads simultaneously in two directions. The system is therefore more suitable for square bays than rectangular bays. The overall behavior of the system is similar to a solid slab. However, the waffle is more efficient for spans in the (10–15 m) range, because it has greater overall depth than a flat slab without the penalty of added dead weight. [3]

Solid head

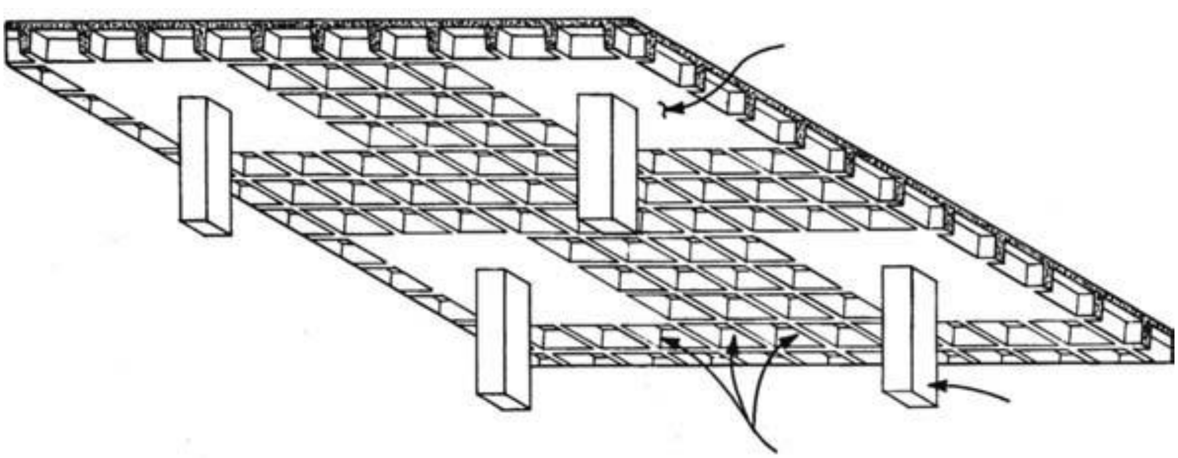
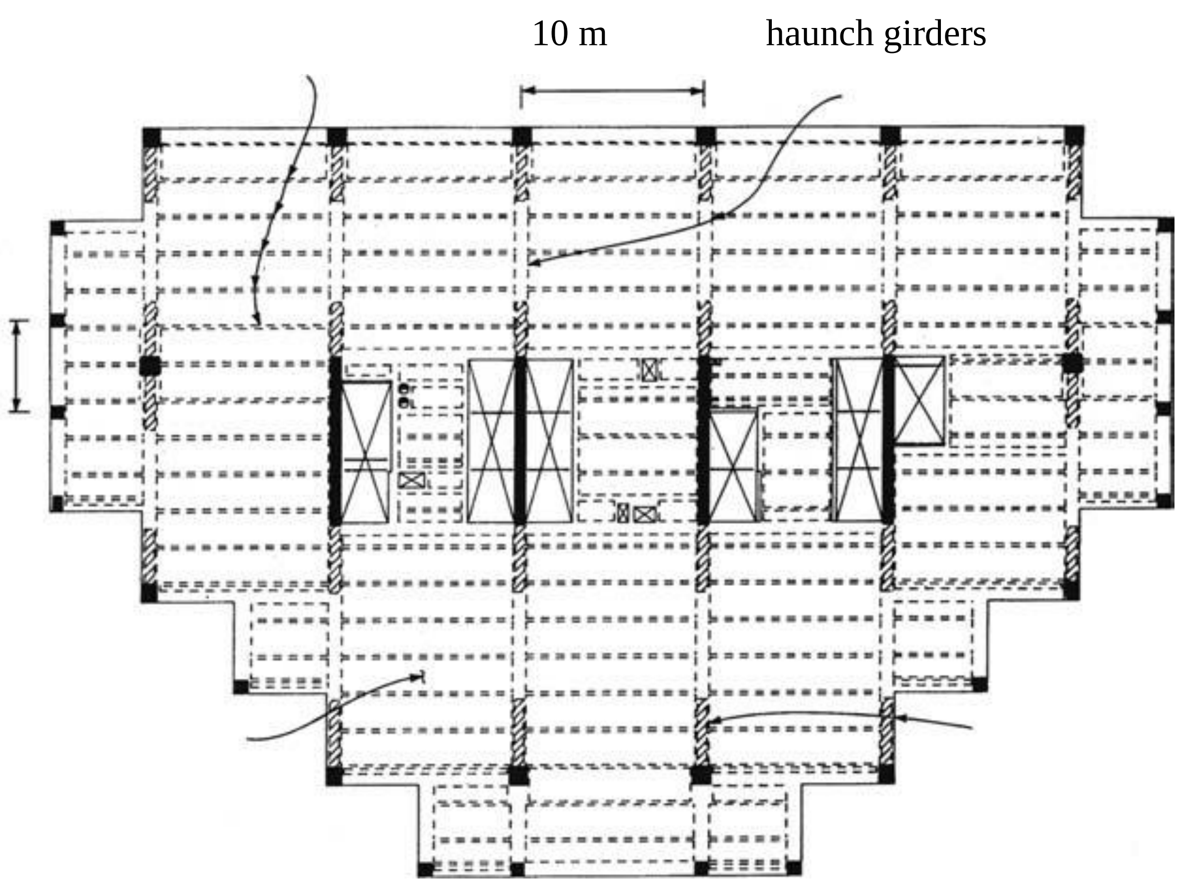


Figure (2.4): The waffle slab The waffle domes

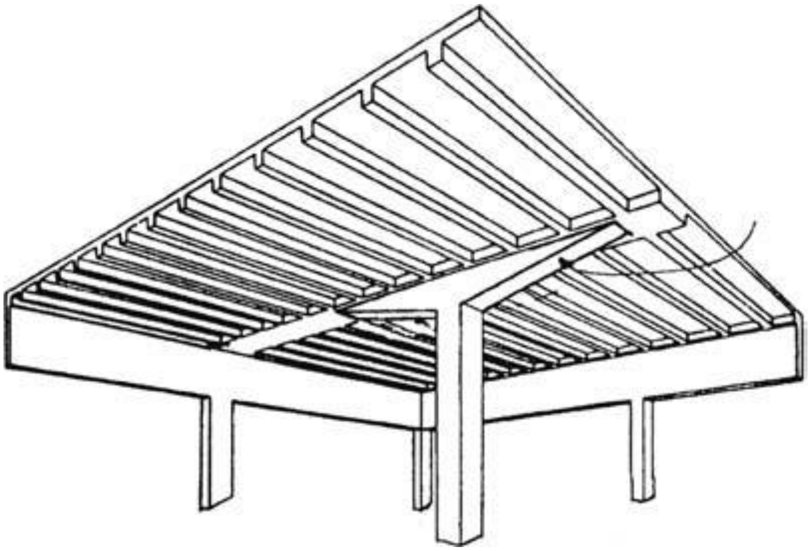
2.2.5 The Haunched girder and joist system

A floor-framing system with girders of a constant depth crisscrossing the interior space between the core and the exterior often presents nonstructural problems because it limits the space available for the passage of air-conditioning ducts, achieves more headroom without making undue compromises in the structure.

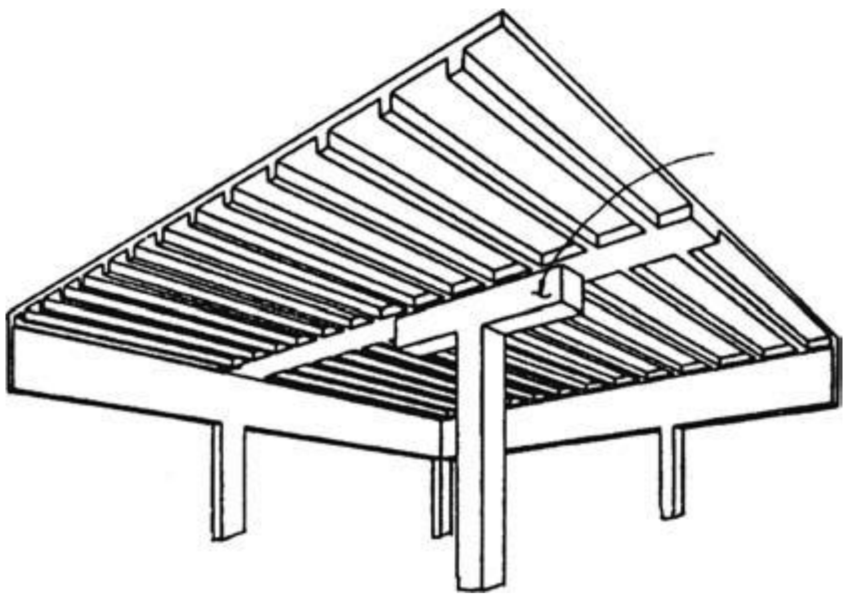
The basic system shown in (Figure 2.5.a) consists of a girder of variable depth. The shallow depth at the center facilitates the passage of mechanical ducts and reduces the need to raise the floor-to-floor height. Two types of haunched girders are in vogue. One uses a tapered haunch (Figure 2.5.b) and the other a square haunch (Figure 2.5.c).[1]



(a) A Plan show haunched girder



(b) the tapered haunch girder



(c) The Hammerhead haunched girder (square haunch)

Figure (2.5): a,b,c showed the haunch girders system

2.2.6 Analysis and design methods

The floor system analyzed and design mainly for gravtiy loads (dead-live), the loads are shared by the structral members propotinal to their relative stiffnesses, there is some methods to analyze the floor system, direct design codes factors which is restrianed by some conditions.

Equivelant frame method, It is currently the most common method of analysis in designing concrete floor systems, including post-tensioned floors. It is flexible and efficient, equally suited for both regular and irregular floor systems. This method involves modeling of the three dimensional slab system as a series of two-dimensional frames which are independently analyzed for loads assumed to act in the plane of each frame. Although similar to the direct design method, this method uses approximations that more accurately capture the actual behavior of the slab.

Yield line method, this method is an excellent tool to justify moment capacity of existing slab systems that are functioning satisfactorily for serviceability requirements, the method is based on the principle that in a slab failing in flexure under overload conditions, the reinforcement will yield first in a region of highest moment. When this occurs, the slab in this region hinges along a line commonly referred to as a yield line.

Finite element methods are commonly used by computer software pakage, it is accurate and saving time.

2.3 Lateral loads

2.3.1 Wind loads

Windstorms pose a variety of problems in buildings‒particularly in tall buildings‒causing concerns for building owners, insurers, and engineers like. Hurricane winds are the largest single cause of economic and insured losses due to natural disasters in many contureis, designing for wind, a building cannot be considered independent of its surroundings.[1]

The influence of nearby buildings and land configuration on the sway response of the building can be substantial. The sway at the top of a tall building caused by wind may not be seen by a passerby, but may be of concern to those occupying its top floors. However, a modern skyscraper, with lightweight curtain walls, dry partitions, and high-strength materials, is more prone to wind motion problems than the early skyscrapers, which had the weight advantage of masonry partitions, heavy stone facades, and massive structural members.

Structural innovations and lightweight construction technology have reduced the stiffness, mass, and damping characteristics of modern buildings. In buildings experiencing wind motion problems, objects may vibrate, doors and chandeliers may swing, pictures may lean, and books may fall off shelves. If the building has a twisting action, its occupants may get an illusory sense that the world outside is moving, creating symptoms of vertigo and disorientation. In more violent storms, windows may break, creating safety problems for pedestrians below. Sometimes, strange and frightening noises are heard by the occupants as the wind shakes elevators, strains floors and walls, and whistles around the sides.

Following are some of the criteria that are important in designing for wind:

1. Strength and stability.
2. Fatigue in structural members and connections caused by fluctuating wind loads.
3. Excessive lateral deflection that may cause cracking of internal partitions and external cladding, misalignment of mechanical systems, and possible permanent deformations of nonstructural elements
4. Frequency and amplitude of sway that can cause discomfort to occupants of tall, flexible buildings.
5. Possible buffeting that may increase the magnitude of wind velocities on neighboring buildings.
6. Wind-induced discomfort in pedestrian areas caused by intense surface winds.
7. Annoying acoustical disturbances.
8. Resonance of building oscillations with vibrations of elevator hoist ropes.[3]

2.3.1.1Nature of wind

Wind is the term used for air in motion and is usually applied to the natural horizontal motion of the atmosphere. Motion in a vertical or nearly vertical direction is called a current. Movement of air near the surface of the earth is three-dimensional, with horizontal motion much greater than the vertical motion. Vertical air motion is of importance in meteorology but is of less importance near the ground surface. On the other hand, the horizontal motion of air, particularly the gradual retardation of wind speed and the high turbulence that occurs near the ground surface, are of importance in building engineering. In urban areas, this zone of turbulence extends to a height of approximately one-quarter of a mile aboveground, and is called the surface boundary layer. Above this layer, the horizontal airflow is no longer influenced by the ground effect. The wind speed at this height is called the gradient wind speed, and it is precisely in this boundary layer where most human activity is conducted.[3]

Therefore, how wind effects are felt within this zone is of great concern. Although one cannot see the wind, it is a common observation that its flow is quite complex and turbulent in nature.

The sudden variation in wind speed, called gustiness or turbulence, plays an important part in determining building oscillations.

2.3.1.2 Types of wind

Winds that are of interest in the design of buildings can be classified into three major types: prevailing winds, seasonal winds, and local winds.

1. Prevailing winds. Surface air moving toward the low-pressure equatorial belt is called prevailing winds or trade winds. In the northern hemisphere, the northerly wind blowing toward the equator is deflected by the rotation of the earth to become northeasterly and is known as the northeast trade wind. The

corresponding wind in the southern hemisphere is called the southeast trade wind.

2. Seasonal winds. The air over the land is warmer in summer and colder in winter than the air adjacent to oceans during the same seasons. During summer,the continents become seats of low pressure, with wind blowing in from the colder oceans. In winter, the continents experience high pressure with winds directed toward the warmer oceans. These movements of air caused by variations in pressure difference are called seasonal winds. The monsoons of the China Sea and the Indian Ocean are an examples.
3. Local winds. Local winds are those associated with the regional phenomena and include whirlwinds and thunderstorms. These are caused by daily changes in temperature and pressure, generating local effects in winds. The daily variations in temperature and pressure may occur over irregular terrain, causing valley and mountain breezes.

All three types of wind are of equal importance in design, prevailing and seasonal wind speeds fluctuate over a period of several months, whereas the local winds vary almost every minute, The variations in the speed of prevailing and seasonal winds are referred to as fluctuations in mean velocity. The variations in the local winds, are referred to as gusts.[3]

2.3.1.3 Characteristics of wind

Wind flow is complex because numerous flow situations arise from the interaction of wind with structures. However, in wind engineering, simplifi cations are made to arrive at the design wind loads by distinguishing the following characteristics:

1- Variation of wind velocity with height (velocity profile) The roughness of the earths surface which causes drag, converts some of the winds energy into mechanical turbulence. Since turbulence is generated at the surface, surface wind speed is much less than wind speed at high levels

2- Wind turbulence

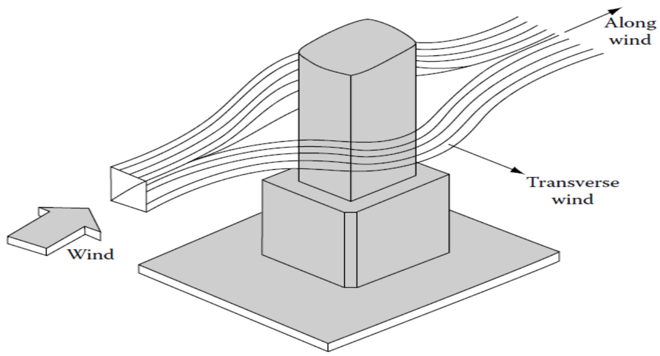
Any movement of air at speeds greater than 2–3 mph (0.9–1.3 m/s) is turbulent, causing particles of air to move randomly in all directions. Every structure has a natural frequency of vibration. Should dynamic loading occur at or near its natural frequency, structural damage, out of all proportion to size of load, may result, it is well known, for example, bridges capable of carrying far greater loads than the weight of a company of soldiers may oscillate dangerously and may even break down under dynamic loading of soldiers marching over them in step.

3- Probabilistic approach

In many engineering sciences, the intensity of certain events is considered to be a function of the duration recurrence interval (return period). For example, the fastest mile wind 33 ft (10 m) aboveground in Dallas, Texas,USA corresponding to a 50 year return period, is 67 mph (30 m/s),compared to the value of 71 mph (31.7 m/s) for a 100 year recurrence interval, However, in structural engineering practice it is believed that the actual probability of overstressing a structure is much less because of the factors of safety and the generally conservative values of wind speeds used in design. Wind velocities (measured with anemometers usually installed at airports across the country) are averages of the fluctuating velocities measured during an infinite interval of time.

4- Vortex shedding

In general, wind buffeting against a bluff body is diverted in three mutually perpendicular directions, giving rise to these sets of forces and moments, in structural engineering the force and moment corresponding to the vertical axis (lift and yawing moment) are of little significance. Therefore, aside from the effects of uplift forces on large roof areas, flow of wind is considered two-dimensional, as shown in Figure (2.6.a), consisting of along wind and transverse wind. the term along wind–or simply wind–is used to refer to drag forces while transverse wind is the term used to describe crosswind. generally, in tall building design, the crosswind motion perpendicular to the direction of wind is often more critical than along-wind motion, Consider a prismatic building subjected to a smooth wind flow. the originally parallel upwind streamlines are displaced on either side of the building, as illustrated in Figure 2.6.b. This results in spiral vortices being shed periodically from the sides into the downstream flow of wind. at relatively low wind speeds of, say, 50–60 mph (22.3–26.8 m/s), the vortices are shed symmetrically in pairs, one from each side. when the vortices are shed, that is, break away from the surface of the building, an impulse is applied in the transverse direction.



(a) Flow of wind

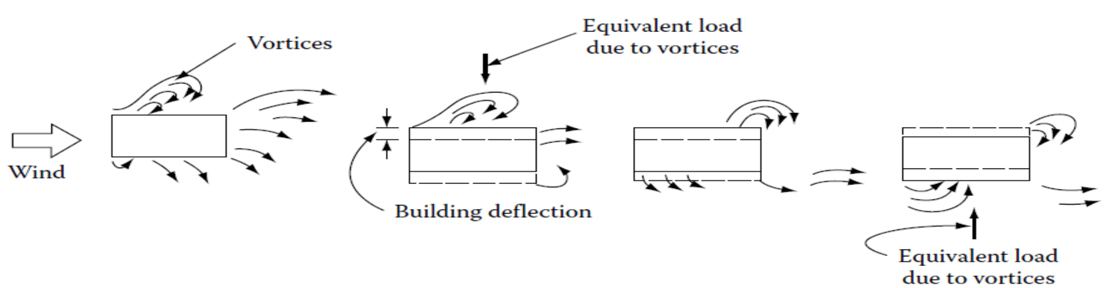


Figure (2.6):(a) (b) wind actions (b)vortix shedding

5- Daynamic nature of wind

Unlike steady flow of wind, which for design purposes is considered static, turbulent wind associated with gustiness cannot be treated in the same manner. This is because gusty wind velocities change rapidly and even abruptly, creating effects much larger than if the same loads were static. Wind loads, therefore, need to be studied as if they were dynamic, some what similar to seismic loads. The intensity of dynamic load depends on how fast the velocity varies and also on the response of the structure itself. Therefore, whether pressures on a building due to wind gust, is dynamic or static entirely depends on the gustiness of wind and the dynamic properties of the building to which it is applied, when we considering building vibrate The time it takes a building to cycle through a complete oscillation is known as the period of the building If the wind gust reaches its maximum value and vanishes in a time much shorter than the period of the building, its effects are dynamic. On the other hand, the gusts can be considered as static loads if the wind load increases and vanishes in a time much longer than the period of the building.

2.3.1.4 Pressures and suctions on exterior surfaces

Detailed measurements of pressures and suctions on exterior surfaces of buildings are made in wind tunnel tests using a rigid models. The model contains numerous (typically 300‑800) ports or “taps” which are connected via tubing to pressure transducers. The transducers convert the pressure at the point where the tap is located to an electrical signal which is then measured simultaneously for a particular wind direction. Measurements are usually made at 10° intervals for the full 360° azimuth range.

These aerodynamic measurements made in the wind tunnel are subsequently combined with the statistics of the full-scale wind climate at the site to provide predictions of pressures and suctions for various return periods. This information is used in the design of cladding.

2.3.1.5 Internal pressures and differential pressures

Estimates of internal pressures are needed in determining net wind loads for the design of the cladding and glazing of buildings. These may be obtained from building code specifications, or from wind-tunnel studies.

Although the importance of determining internal pressures is clear, it is not a quantity which can be determined exactly. In fact, internal pressures are influenced by many factors, which are uncertain in themselves, such as the character of the leakage paths and windows or other exterior openings being left open or being broken during windstorms. The complex distribution of exterior pressures and their influence on the internal pressures must also be taken into account.

In spite of these difficulties, reasonable estimates of the internal pressure can be made by expressing the uncertainties in statistical terms.[3]

2.3.1.6 The Uniform Building Code, 1997: Wind Load Provisions

Wind load provisions of UBC 1997 are based on the ASCE 7-88 standard with certain simplifying assumptions to make calculations easier. The design wind speed is based on the fastest-mile wind speed as compared to the 3-sec gust speeds of the later codes. The prevailing wind direction at the site is not considered in calculating wind forces on the structures: The direction that has the most critical exposure controls the design. Consideration of shielding by adjacent buildings is not permitted because studies have shown that in certain configurations, the nearby buildings can actually increase the wind speed through funneling effects or increased turbulence. Additionally, it is possible that adjacent existing buildings may be removed during the life of the building being designed. [3]

To shorten the calculation procedure, certain simplifying assumptions are made.

These assumptions do not allow determination of wind loads for flexible buildings that may be sensitive to dynamic effects and wind-excited oscillations such as vortex shedding. Such buildings typically are those with a height-to-width ratio greater than 5, and over 400 ft (121.9 m) in height. The general section of the UBC directs the user to an approved standard for the design of these types of structures. The ASCE 7-02, adopted by IBC 2003, is one such standard for determining the dynamic gust response factor required for the design of these types of buildings.

UBC provisions are not applicable to buildings taller than 400 ft. (122 m) for normal force method, Method 1, and 200 ft (61 m) for projected area method, Method 2. Any building, including those not covered by the UBC, may be designed using wind-tunnel.[4]

(1) Design Wind Pressures

The design wind pressure p is given as a product of the combined height, exposure, and gust factor coefficient C_e; the pressure coefficient C_q; the wind stagnation pressure q_s; and building Importance Factor I_w.

$$p = C_e C_q q_s I_w \tag{2.1}$$

The pressure q_s manifesting on the surface of a building due to a mass of air with density ρ , moving at a velocity is given by Bernoulli’ s equation

$$q_s = \frac{1}{2} \rho v^2 \tag{2.2}$$

The density of air ρ is 0.0765 pcf, for conditions of standard atmosphere, temperature (59 F), and barometric pressure (29.92 in. of mercury).

Since velocity given in the wind map is in mph, Eq. (2.2) reduces to

$$q_s = \frac{1}{2}\left[\frac{.0765pcf}{32.2\ ft/s2}\right]\left[\frac{5280ft}{mile} \times \frac{1hr}{3600s}\right] v^2 \tag{2.3}$$

$$q_s = 0.00256\, v^2 \quad \text{(psf)} \qquad (2.4)\ [4]$$

2.3.1.7 The British standard code provition for wind

BS 6399-part 2-1997 gives methods for determining the gust peak wind loads on buildings and components thereof that should be taken into account in design using equivalent static procedures.Two alternative methods are given:

a) a standard method which uses a simplified procedure to obtain a standard effective wind speed which is used with standard pressure coefficients to determine the wind loads for orthogonal design cases,(This procedure is virtually the same as in CP3:Chapter V:Part 2).[5]

b) a directional method in which effective wind speeds and pressure coefficients are determined to derive the wind loads for each wind direction.

Other methods may be used in place of the two methods given in this standard, provided that they can be shown to be equivalent. Such methods include wind tunnel tests.

The methods given in this Part of BS 6399 do not apply to buildings which, by virtue of the structural properties, e.g. mass, stiffness, natural frequency or damping, are particularly susceptible to dynamic excitation. These should be assessed using established dynamic methods or wind tunnel tests. For all structures where the wind loading can be represented by equivalent static loads , the wind loading can be obtained either by the standard method or by the directional method,we will stated the standard method only.

(1) standard method

standard method requires assessment for orthogonal load cases for wind directions normal to the faces of the building, The value of the dynamic pressure q_s as belw

$$q_s = 0.613\, v_e^{\,2} \qquad (2.5)$$

where

q_s is the dynamic pressure (in Pa,N/m^2)

Ve is the effective wind speed (in m/s).

$$V_e = V_s \times S_b \qquad (2.6)$$

Where S_b is the terrain and building factor

V_s is site wind speed which is obtained by the below equation

$$Vs = V_b \times S_a \times S_d \times S_s \times S_p \qquad (2.7)$$

where

V_b is the basic wind speed

S_a is an altitude factor

S_d is a direction factor

S_s is a seasonal factor

S_p is a probability factor

The pressure acting on the external surface of a building p_e is given by

$$p_e = q_s C_{pe} C_a \qquad (2.8)$$

where

q_s is the dynamic pressure

C_{pe} is the external pressure coefficient for the building surface

C_a is the size effect factor for external pressures.

The pressure acting on the internal surface of a building, p_i, is given by

$$p_i = q_s C_{pi} C_a \qquad (2.9)$$

where

q_s is the dynamic pressure

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

C_a is the size effect factor for internal pressures.[5]

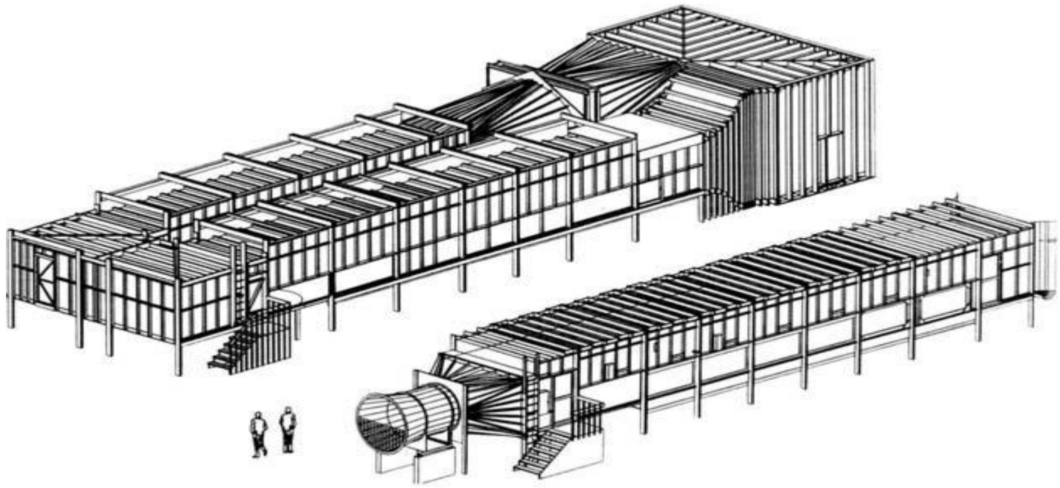
2.3.1.8 The Wind Tunnel Test Method

ASCE standard is American Society of Civil Engineers Minimum Design Loads for Buildings and Other Structures wind-tunnel test procedure is recommended for buildings that possess any of the following characteristics:

1- Have nonuniform shapes.
2- Are flexible with natural frequencies less than 1 Hz.
3- Are subject to significant buffeting by the wake of upwind buildings or other structures.
4- Are subject to accelerated flow of wind by channeling or local topographic features.

Wind tunnels such as those shown in Figures (2.7) are used, among other things, to provide accurate distributions of wind pressure on buildings as well as investigate aeroelastic behavior of slender and light weight structures.[3]

(a)



(b)

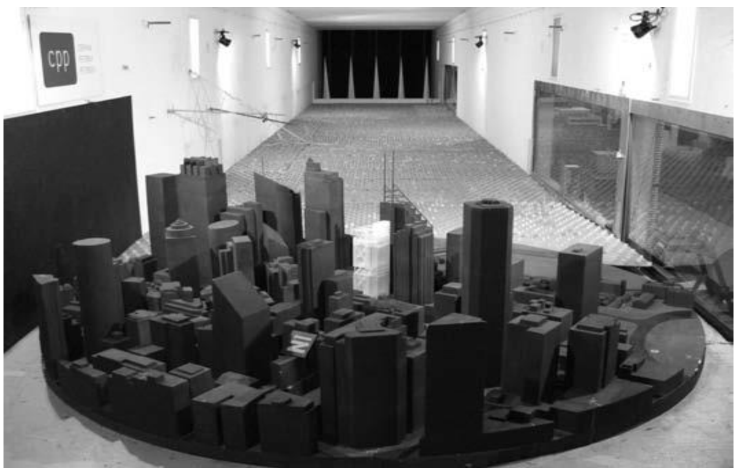


Figure (2.7): (a) wind tunnel (b) Photographs of Rigid model

Services provided by a wind tunnel typically offer the following benefi ts:

1- Provides an accurate distribution of wind loads, especially for structures · in a built-up environment by determining directly the impact of surrounding structures.

2- Provides predictions of wind-induced building motions (accelerations and torsional velocities) likely to be experienced by occupants of the top floors, and compares the test results to available serviceability criteria. Complaints by building occupants of excessive motion can compromise the value of a development. Information provided by tests enables the design team (structural engineers, architect, and developer) to make timely and appropriate modifi cations, if required, to architectural and structural design.

3- Pretest estimate of cladding pressures and overall loads by a wind engineer, based on a review of similar buildings, with appropriate consideration of the local meteorological data can help the engineer, the architect, and the facade engineer to develop a preliminary foundation design and initial cost estimate for the curtain wall.

There are three basic types of wind –tunnel modeling techniques :

1. Rigid pressure model (PM) provide local load pressures for design of clading elements and mean pressures for the determination of overall mean loads
2. Rigid high-frequency base balance model (HFBB/HFFB), measures overall fluctuating loads for the determination of dynamic responses
3. Aeroelastic model (AM) is used for direct measurement of responses such as, deflections, and accelerations, and is deemed necessary, when the lateral motions of a building are considered to have a large infl uence on wind loading, and for measuring effects of higher modes.[1]

2.3.2 Seismic loads

Earthquakes are catastrophic events that occur mostly at the boundaries of portions of the earth crust called tectonic plates. When movement occurs in these regions, along faults, waves are generated at the earth’ s surface that can produce very destructive effects Some of the most destructive effects caused by shaking as a result of the earthquake are those that produce lateral loads in a structure. The input shaking causes the foundation of a building to oscillate back and forth in a more or less horizontal plane. The building mass has inertia and wants to remain where it is and therefore, lateral forces are exerted on the mass in order to bring it along with the foundation.[3]

For analysis purposes, this dynamic action is simplified as a group of horizontal forces that are applied to the structure in proportion to its mass and to the height of the mass above the ground.

In multistory buildings with floors of equal weight, the loading is further simplified as a group of loads, each being applied at a floor line, and each being greater than the one below in a triangular distribution (see Figure 2.8). Seismically resistant structures are designed to resist these lateral forces through inelastic action and must, therefore, be detailed accordingly. These loads are often expressed in terms of a percent of gravity weight of the building and can vary from a few percent to near 50% of gravity weight. There are also vertical loads generated in a structure by earthquake shaking, but these forces rarely overload the vertical load-resisting system. However, earthquake-induced vertical forces have caused damage to structures with high dead load compared to design live load. These vertical forces also increase the chance of collapse due to either increased or decreased compression forces in the columns. Increased compression may exceed the axial compressive capacity of columns while decreased compression may reduce the bending strength of columns.[1]

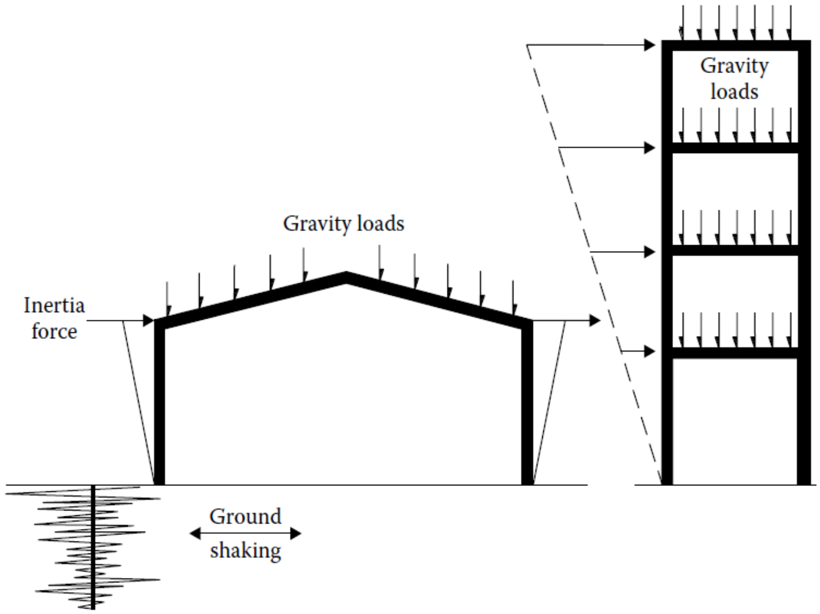


Figure (2.8) :Daynamic actions of earthquake

structural design for seismic loading is primarily concerned with structural safety during major earthquakes, serviceability and the potential for economic loss are also of concern. As such, seismic design requires an understanding of the structural behavior under large inelastic, cyclic deformations. Behavior under this loading is fundamentally different from wind or gravity loading. It requires a more detailed analysis, and the application of a number of stringent detailing requirements to assure acceptable seismic performance beyond the elastic range. Some structural damage can be expected when the building experiences design ground motions because almost all building codes allow inelastic energy dissipation in structural systems

The seismic analysis and design of buildings has traditionally focused on reducing the risk of the loss of life in the largest expected earthquake. Building codes base their provisions on the historic performance of buildings and their deficiencies and have developed provisions around life-safety concerns by focusing their attention to prevent collapse under the most intense earthquake expected at a site during the life of a structure.[1]

There are three goals for new structures subjected to ground motions:

1- minimize the hazard to life from all structures.
2- increase the expected performance of structures having a substantial public hazard due to occupancy or use.
3- improve the capability of essential facilities to function after an earthquake.

In general, most earthquake code provisions implicitly require that structures be able to resist:

1. Minor earthquakes without any damage.
2. Moderate earthquakes with negligible structural damage and some nonstructural damage.
3. Major earthquakes with some structural and nonstructural damage but without collapse.

An idea of the behavior of a building during an earthquake may be grasped by considering the simplifi ed response shape shown in Figure (2.9)

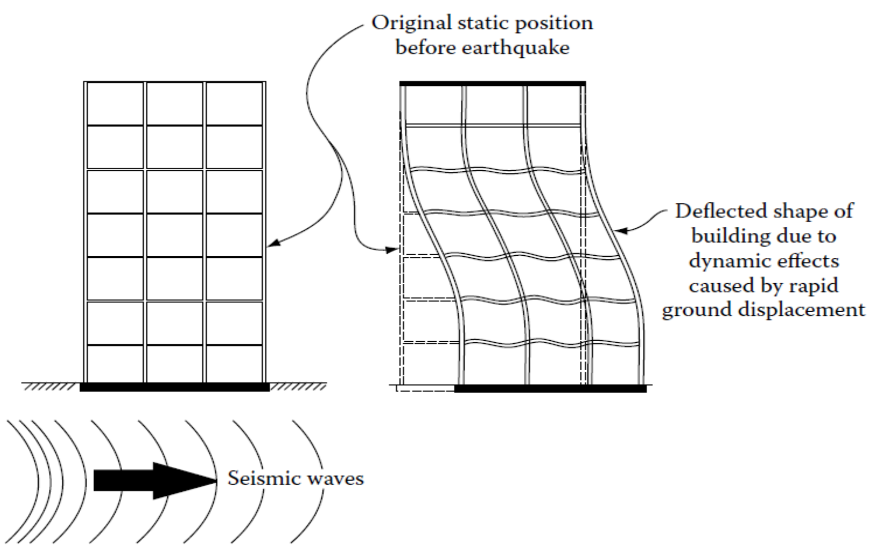


Figure (2.9): Building behavior

As the ground on which the building rests is displaced, the base of the building moves with it. However, the building above the base is reluctant to move with it because the inertia of the building mass resists motion and causes the building to distort. This distortion wave travels along the height of the structure, and with continued shaking of the base, causes the building to undergo a complex series of oscillations.

Both wind and seismic forces are essentially dynamic, there is a fundamental difference in the manner in which they are induced in a structure. Wind loads, applied as external loads, are characteristically proportional to the exposed surface of a structure, while the earthquake forces are principally internal forces resulting from the distortion produced by the inertial resistance of the structure to earthquake motions, the magnitude of earthquake forces is a function of the mass of the structure rather than its exposed surface. Whereas in wind design, one would feel greater assurance about the safety of a structure made up of heavy sections, in seismic design, this does not necessarily produce a safer design.

An increase in mass has two undesirable effects on the earthquake design. First, it results in an increase in the force, and second, it can cause buckling or crushing of columns and walls when the mass pushes down on a member bent or moved out of plumb by the lateral forces. This effect is known as the $P\Delta$ effect and the greater the vertical forces, In general, tall buildings respond to seismic motion differently than low-rise buildings. The magnitude of inertia forces induced in an earthquake depends on the building mass, ground acceleration, the nature of the foundation, and the dynamic characteristics of the structure. If a building and its foundation were infinitely rigid, it would have the same acceleration as the ground, resulting in an inertia force $F = ma$, for a given ground acceleration, a. However, because buildings have certain flexibility, the force tends to be less than the product of buildings mass and acceleration. Tall buildings are invariably more flexible than low-rise buildings, and in general, they experience much lower accelerations than low-rise buildings. But a flexible building subjected to ground motions fora prolonged period may experience much larger forces if its natural period is near that of the ground waves. Thus, the magnitude of lateral force is not a function of the acceleration of the ground alone, but is influenced to a great extent by the type of response of the structure itself and its foundation as well. This interrelationship of building behavior and seismic ground motion also depends on the building period as formulated in the so-called response spectrum.[1]

As a building vibrates due to ground motion, its acceleration will be amplified if the fundamental period of the building coincides with the period of vibrations being transmitted through the soil. This amplified response is called resonance. Natural periods of soil are in the range of 0.5–1.0 s. Therefore, it is entirely possible for the building and ground it rests upon to have the same fundamental period. This was the case for many 5- to 10-story buildings in some earthquake. An obvious design strategy is to ensure that buildings have a natural period different from that of the expected ground vibration to prevent amplification.

2.3.2.1 Damping

Buildings do not resonate forever, because they are damped; the extent of damping depends upon the construction materials, the type of connections, and the influence of nonstructural elements on the stiffness characteristics of the building. Damping is measured as a percentage of critical damping. In a dynamic system, critical damping is defined as the minimum amount of damping necessary to prevent oscillation altogether, the damping of structures is influenced by a number of external and internal sources, external viscous damping caused by air surrounding the building (negligible), internal viscous damping associated with the material viscosity, Friction damping, also called Coulomb damping, occurring at connections and support points of the structure and Hysteretic damping that contributes to a major portion of the energy absorbed in ductile structures.

For analytical purposes, it is a common practice to lump different sources of damping into a single viscous damping, damping ratios used in practice vary anywhere from 1% to 10% of critical(for steel building 2% and 5% for concrete).[1]

2.3.2.2 Load path and diaphragm

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 (rigid or flexible), if there is a discontinuity in the load path, the building is unable to resist seismic forces regardless of the strength of the elements.

For analytical purpose, these are assumed to behave as deep beams. The slab is the web of the beam carrying the shear, and the perimeter spandrel or wall, if any, is the flange of the beam-resisting bending. In the absence of perimeter members, the slab is analyzed as a plate subjected to in-plane bending.

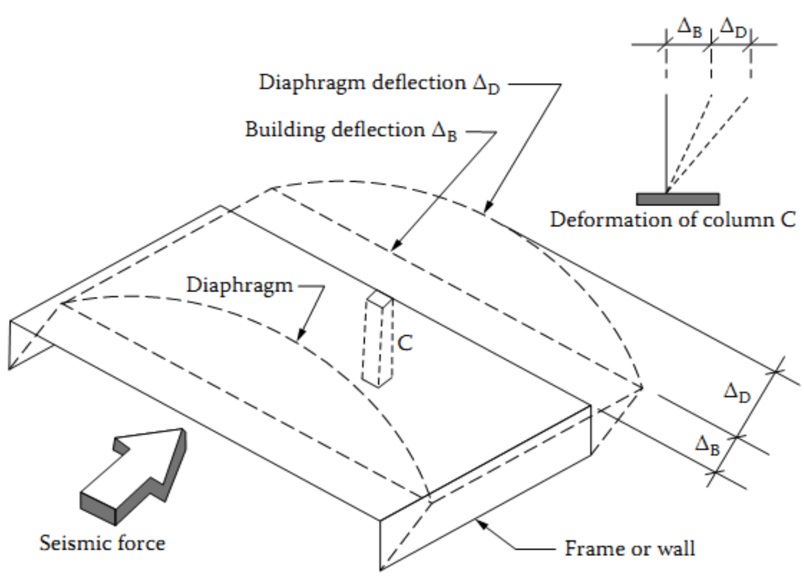


Figure (2.10): show floor acting as diaphragm

2.3.2.3 Ductility & redundancy

All structures are designed for forces much smaller than those the design ground motion would produce in a structure with completely linear-elastic response. This reduction is possible for a number of reasons. As the structure begins to yield and deform in-elastically, the effective period of the response of the structure tends to lengthen, which for many structures, results in a reduction in strength demand.

Ductility is the capacity of building materials, systems, or structures to absorb energy by deforming into the inelastic range. The capability of a structure to absorb energy, with acceptable deformations and without failure, is a very desirable characteristic in any earthquake-resistant design. Concrete, a brittle material, must be properly reinforced with steel to provide the ductility necessary to resist seismic forces. In concrete columns, for example, the combined effects of flexure (due to frame action) and compression (due to the action of the overturning moment of the structure as a whole) produce a common mode of failure: buckling of the vertical steel and spalling of the concrete cover near the floor levels. Columns must, therefore, be detailed with proper spiral reinforcing or hoops to have greater reserve strength and ductility.

Redundancy is a fundamental characteristic for good performance in earthquakes. It tends to mitigate high demands imposed on the performance of members. It is a good practice to provide a building with a redundant system such that the failure of a single connection or component does not adversely affect the lateral stability of the structure. Otherwise, all components must remain operative for the structure to retain its lateral stability.

2.3.2.4 The Irregular buildings

The seismic design of regular buildings is based on two concepts. First, the linearly varying lateral force distribution is a reasonable and conservative representation of the actual response distribution due to earthquake ground motions. Second, the cyclic inelastic deformation demands are reasonably uniform in all of the seismic force‑resisting elements. However, when a structure has irregularities, these concepts may not be valid, requiring corrective factors and procedures to meet the design objectives.[1]

Typical building configuration deficiencies include an irregular geometry, a weakness in a story, a concentration of mass, or a discontinuity in the lateral force‑resisting system. Vertical irregularities are defined in terms of strength, stiffness, geometry, and mass. Although these are evaluated separately, they are related to one another, and may occur simultaneously. For example, a building that has a tall first story can be irregular because of a soft story, a weak story, or both, depending on the stiffness and strength of this story relative to those above.

Because the major effects irregulaty on the structural response the design codes stated that dynamic analysis must be conducted.

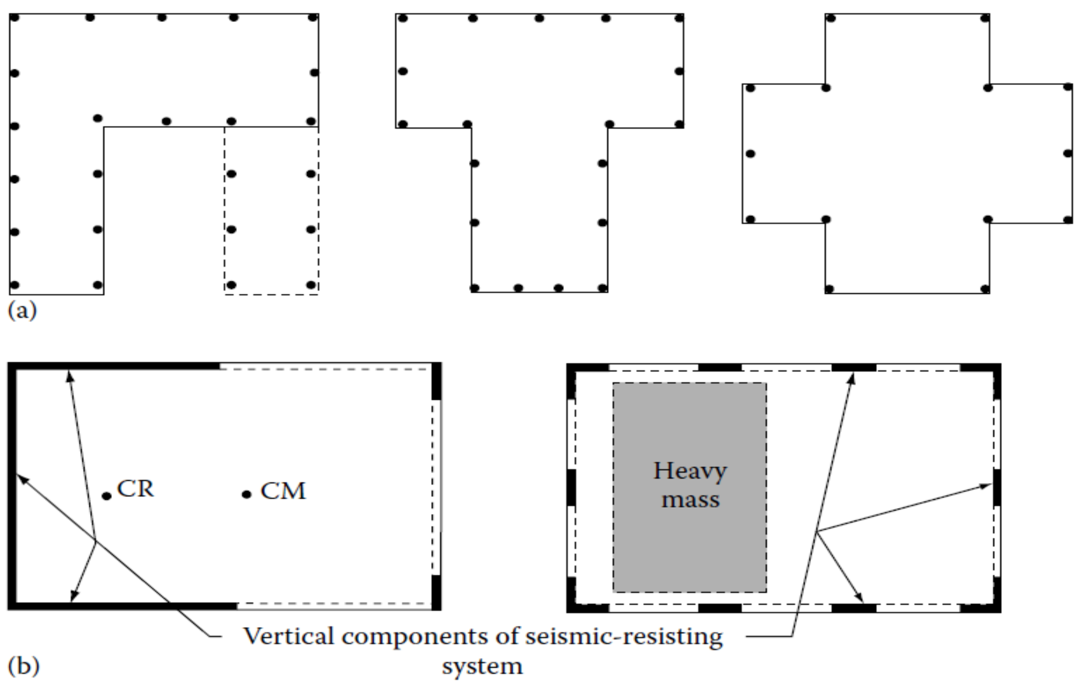


Figure (2.11): Plan irregularities: (a) geometric irregularities, (b) irregularity due to mass-resistance eccentricity

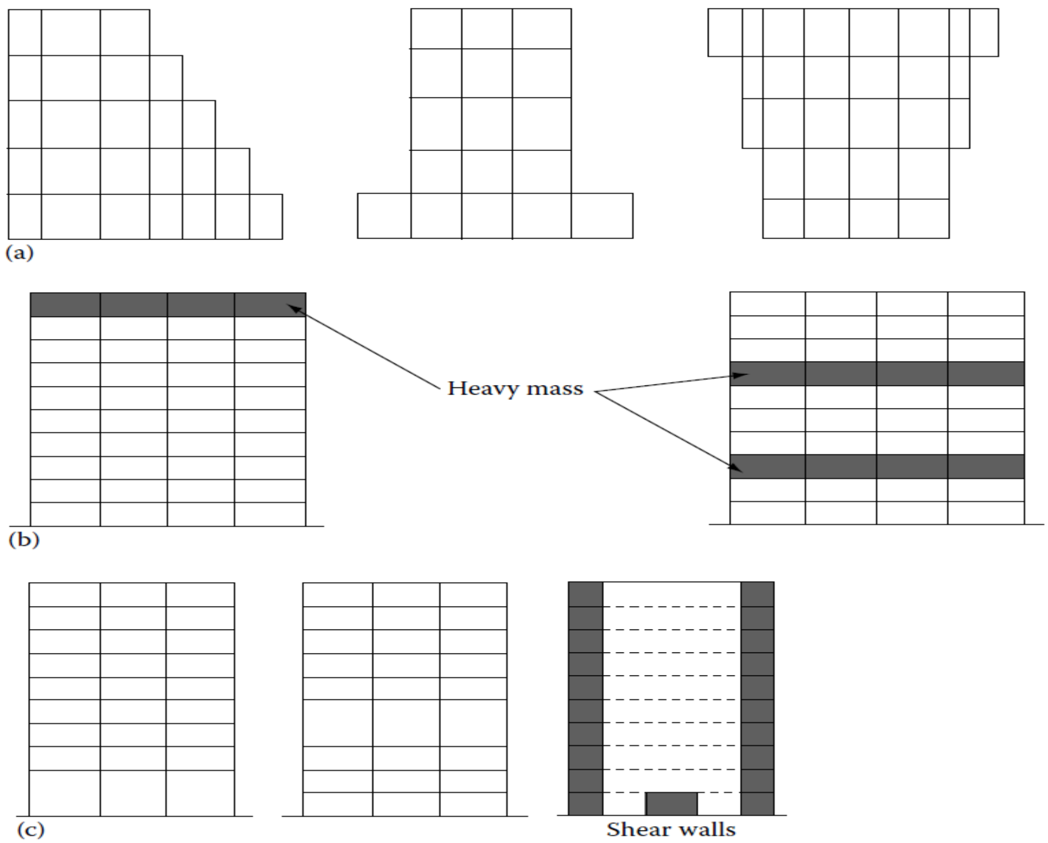


Figure (2.12): Elevation irregularities: (a) abrupt change in geometry, (b) large difference in floor masses, and (c) large difference in story stiffnesses

2.3.2.5 The Overall building torsion

To avoid excessive lateral displacements torsional effects should be minimized by reducing the distance between the center of mass (CM), where horizontal seismic floor forces are applied, and the center of rigidity (CR) of the vertical elements resisting the lateral loads. A conceptual explanation of center of mass, during earthquakes, acceleration-induced inertia forces will be generated at each floor level, where the mass of an entire story may be assumed to be concentrated. Hence the location of seismic force at a particular level will be determined by the center of the accelerated mass at that level. In regular buildings, the positions of the centers of floor masses will differ very little from level to level.

Center of rigidity defined as the center of rigidity or center of stiffness, locates the position of a story shear force V_j, which will cause only relative floor translations and no torsion Hence the seismic requirement that allowance be made in all buildings for so-called accidental torsion, an accidental torsion caused by an assumed displacement of center of mass by a distance equal to 5% of the dimension of the building perpendicular to the direction of force be included. However, the specified accidental torsion need not be applied simultaneously in two directions.

2.3.2.6 Methods of analysis

(1) Equivalent lateral-force procedure (static)

Based on the approximation that the effects of yielding can be adequately accounted for by the linear analysis of the seismic force‐ resisting system for the design spectrum. The distribution of the seismic lateral forces over the height of the building based on simplified formulas that are appropriate for regular structures, more details will discuss later in this research.

(2) Dynamic analysis

Symmetrical buildings with uniform mass and stiffness distribution behave in a fairly predictable manner, whereas buildings that are asymmetrical or with areas of discontinuity or irregularity do not. For such buildings, dynamic analysis is used to determine significant response characteristics such as (1) the effects of the structures' dynamic characteristics on the vertical distribution of lateral forces; (2) the increase in dynamic loads due to torsional motions; and (3) the influence of higher modes, resulting in an increase in story shears and deformations.

Static methods specified in building codes are based on single-mode response with simple corrections for including higher mode effects. While appropriate for simple regular structures, the simplified procedures do not take into account the full range of seismic behavior of complex structures.

Therefore, dynamic analysis is the preferred method for the design of buildings with unusual or irregular geometry.

Two methods of dynamic analysis are permitted:

(1) Elastic response-spectrum analysis.

(2) Elastic or inelastic time-history analysis.

Buildings are analyzed as multi-degree-of-freedom (MDOF) systems by lumping story-masses at intervals along the length of a vertically cantilevered pole. During vibration, each mass will deflect in one direction or another. For higher modes of vibration, some masses may move in opposite directions. Or all masses may simultaneously deflect in the same direction as in the fundamental mode. An idealized MDOF system has a number of modes equal to the number of masses. Each mode has its own natural period of vibration with a unique mode shaped by a line connecting the deflected masses. When ground motion is applied to the base of a multi-mass system, the deflected shape of the system is a combination of all mode shapes, but modes having periods near predominant periods of the base motion will be excited more than the other modes.[1]

Each mode of a multi-mass system can be represented by an equivalent single-mass system having generalized values M and K for mass and stiffness, respectively. The generalized values represent the equivalent combined effects of story masses m1, m2,⋯ and k1, k2,⋯. This concept, shown in Figure (2.13), provides a computational basis for using response spectra based on single-mass systems for analyzing multistoried buildings. Given the period, mode shape, and mass distribution of a multistoried building, we can use the response spectra of a single-degree-of-freedom (SDOF) system for computing the deflected shape, story accelerations, forces, and overturning moments. Each predominant mode is analyzed separately and the results are combined statistically to compute the multimode response.[3]

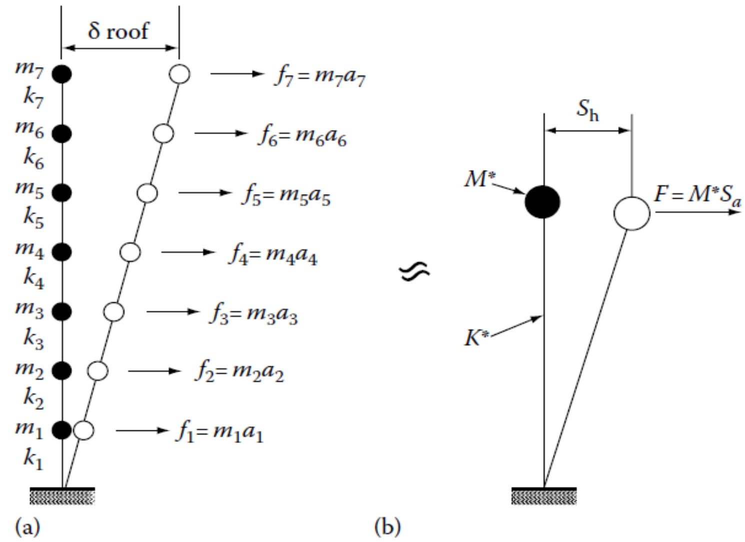


Figure (2.13): (a) mode of multi-mass system (b) equivalent single mass

(a) Response spectrum method.

The word "spectrum" in seismic engineering conveys the idea that the response of buildings having a broad range of periods is summarized in a single graph. For a given earthquake motion and a percentage of critical damping, a typical response spectrum gives a plot of earthquake-related responses such as acceleration, velocity, and deflection for a complete range, or spectrum, of building periods.

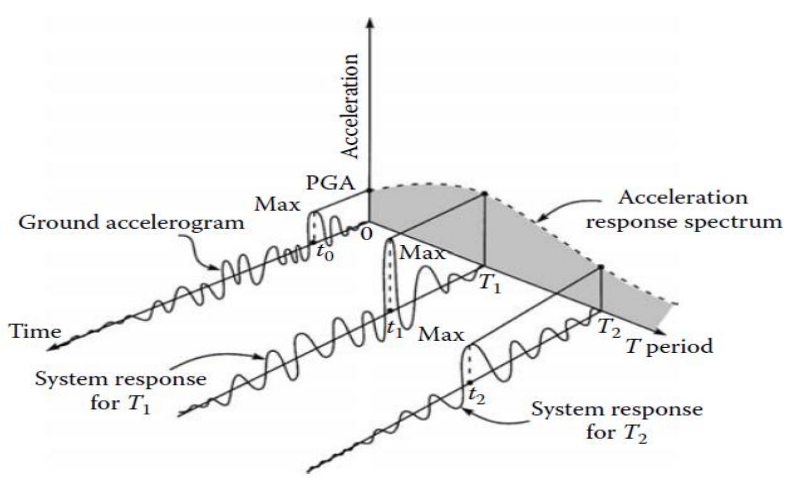


Figure (2.14): Graphical description of response spectrum

Earthquake response spectrum gives engineers a practical means of characterizing ground motions and their effects on structures. Introduced in 1932, it is now a central concept in earthquake engineering that provides a convenient means to summarize the peak response of all possible linear SDOF systems to a particular ground motion. It also provides a practical approach to apply the knowledge of structural dynamics to the design of structures and the development of lateral force requirements in building codes.

A plot of the peak value of response quantity as a function of the natural vibration period T_n of the system (or a related parameter such as circular frequency ω_n or cyclic frequency f_n) is called the response spectrum for that quantity.

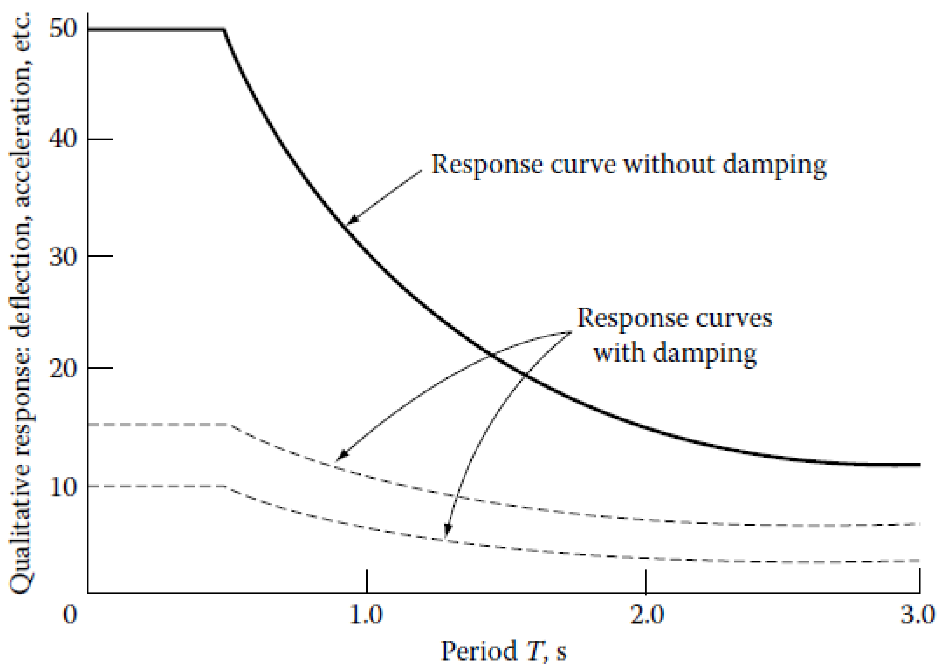


Figure (2.15) Response Spectrum curve

This method is used conjunction with the modal analysis which a lumped mass model of the building with three degrees of freedom at each floor is analyzed to determine the modal shapes and modal frequencies of vibration. The purpose of modal analysis is to obtain the maximum response of the structure in each of its important modes, which are then summed in an appropriate manner. This maximum modal response can be expressed in several ways. In practice, the SRSS (square root of sum of squares) or the (complete quadratic combination) is used for this purpose. Once the story shears and other response variables for each of the important modes are determined and combined to produce design values. The most common combination by the use of the SRSS but un-conservative occurs when two modes have very nearly the same natural period. In this case, the responses are highly correlated and the designer should consider combining the modal quantities more conservatively. The CQC technique provides somewhat better results than the SRSS method for the case of closely spaced modes.

The modal participation factor for each mode may be defined as a constant always less than unity, by which the actual masses of the system are multiplied to give the effective masses for the mode under consideration. Simply stated, model participation factor defines the degree to which that mode participates in the total vibration, most codes make a general statement that all modes having a significant contribution to the total structural response should be included in the analysis. This requirement is deemed satisfied if the sum of the participation factors for the modes considered is at least equal to 90% of unity.

A response spectrum for specific earthquake ground motion does not reflect the total time history of response, but only approximates the maximum value of response for simple structures to that ground motion. The design response spectrum is a smoothed and normalized approximation for many different ground motions, adjusted at the extremes for characteristics of larger structures.

(b) Time history method.

There is to types of time history analysis, linear and nonlinear, linear response history analysis, is a numerically involved technique in which the response of a structural model to a specific earthquake ground motion accelerogram is determined through a process of numerical integration of the equations of motion. The ground shaking accelerogram, or record, is digitized into a series of small time steps, typically on the order of 1/100th of a second or smaller. Starting at the initial time step, a finite difference solution, or other numerical integration algorithm is followed to allow the calculation of the displacement of each node in the model and the forces in each element of model for each time step of the record, The principal advantages of response history analysis, as opposed to response spectrum analysis, is that response history analysis provides a time dependent history of the response of the structure to a specific ground motion, allowing calculation of path-dependent effects such as damping. It also provides information on the stress and deformation state of the structure throughout the period of response. A response spectrum analysis on the other hand, indicates only the maximum response quantities and does not indicate when during the period of response these occur, when response history analyses are used in designs, it is necessary to run the analysis using a suite of ground motion records.

Nonlinear response history procedure, very similar to linear response history analysis, except that the mathematical mode is formulated in such a way that the stiffness and even connectivity of the elements can be directly modified based on the deformation state of the structure.

2.3.2.7 The Uniform building code, 1997: seismic provisions

The following key ideas are contained in the 1997 UBC:

1. Earthquake loads are specified for use with strength or load factor resistance design (LFRD), although allowable stress design (ASD) is also permitted.[4]
2. The structural system coefficient, R, which is a measure of the ductility and over strength of the structural system, has been adjusted to provide a strength

level base shear. It is essentially equal to R_W, the seismic coefficient specified in previous edition, divided by 1.4.

3. Two near-source factors N_a and N_r, new for the 1997 UBC, have been incorporated in seismic zone 4 to amplify ground motions that occur at close distances to the fault.
4. A redundancy-reliability factor, ρ, also new in the 1997 UBC, has been incorporated to promote redundant lateral-force-resisting systems. No redundant systems are penalized through higher lateral load requirements, while super-redundant systems are not rewarded with less stringent seismic design requirements.
5. A set of soil profile categories, S_A through S_F, have been incorporated. These are used in combination with seismic zone factor Z, and near-source factors N_a and N_r, to provide the site-dependent ground motion coefficients C_a and C_r, (hard rock (SA), rock (SB), very dense and soft rock (SC), stiff soil (SD), and soft soil (SE). Soil categories are based on the average shear wave velocity in the upper 30m (100 feet) or below count of a standard penetration test). They are used in combination with the seismic zone factor Z, and the near-source factors N_a and N_V, to determine the site-dependent coefficients C_A and C_V. C_A and C_V define ground motion response within the acceleration and velocity-controlled range of the response spectrum.

Design Base Shear, V The strength level design base shear is given by the formula

$$V = \frac{C_V\, I}{RT}\, W \qquad (2.10)$$

Where

T = fundamental period of the structure in the direction under consideration

I = seismic importance factor

C_v = a numerical coefficient dependent on the soil conditions at the site and the seismicity of the region, (UBC Table 16-R)

W = seismic dead load

R = a factor that accounts for the ductility and over strength of the structural system, (UBC Table 16-N)

Z = seismic zone factor, (UBC Table 16-I). Note that Z does not directly appear in the base shear formula. It does, however, affect the seismic coefficients C_a and C_v.

The base shear as specified by Eq. (2.10) is subject to three limits:

1. The design base shear need not exceed

$$V= \frac{2.5C_a I}{R} W \qquad (2.11) \text{ (UBC Eq. (30.5))}$$

2. It cannot be less than

$$V = 0.11C_a IW \qquad (2.12) \text{ (UBC Eq. (30.6))}$$

Where C_a is a seismic coefficient dependent on soil conditions at the site and on regional seismicity.

3. In the zone of highest seismicity (zone 4), the design base shear must be equal to or greater than

$$V= \frac{0.8ZN_V I}{R} W \qquad (2.13)$$

Five seismic zones−numbered 1, 2A, 2B, 3, and 4−are defined. The zone for a particular site is determined from a seismic zone map. The map accounts for the geographical variations in the expected levels of earthquake ground shaking, and gives an estimated peak horizontal acceleration on rock having a 10% chance of being exceeded in a 50-year period.

Table (2.1): seismic zone factor

Zone	1	2A	2B	3	4
Z	0.075	0.15	0.2	0.3	0.4

(From Table 16-I, UBC 1997.)

The importance factor **I** is used to increase the margin of safety against collapse. For example, I = 1.50 for essential facilities, I = 1.25 for hazardous facilities, and I = 1.15 for special occupancy structures. Essential structures are those that must remain operative immediately following an earthquake such as emergency treatment areas and fire stations. It can be obtained from (Table 16-K, UBC 1997).

The building period **T** may be determined by analysis or by using empirical formulas. It is denoted T_A if determined by empirical formulas, and T_B if determined by analysis. The following single empirical formula may be used for all framing systems:

$$T_A = C_t h_n^{3/4} \qquad (2.14)$$

Where

C_t = 0.035 (0.0853-for m) for steel moment frames

= 0.030 (0.0731) for concrete moment frames

= 0.020 (0.0488) for all other buildings

h_n = the height of the building in feet or meter.

And the period can be obtained from method (B) using the following equation

$$T = 2\pi \sqrt{\left(\sum_{i=1}^{n} w_i \delta_i^{\,2}\right) \div \left(g \sum_{i=1}^{n} f_i \delta_i\right)} \qquad (2.15)$$

The coefficient **R** is a measure of ductility and over strength of a structural system, based primarily on performance of similar systems in past earthquakes. A higher value of R has the effect of reducing the design base shear, (see the appendix).

The dead load **W**, used for calculating the base shear, includes the total dead load of the structure, the actual weight of partitions, and 25% of the floor live load in storage and warehouse occupancies. The total seismic load W represents the total mass of the building and includes the weights of structural slabs, beams, columns, and walls; and nonstructural components such as floor topping, roofing, fireproofing material, fixed electrical and mechanical equipment, partitions, and ceilings.

The seismic coefficients C_v and C_a, given in Tables (2.2) and (2.3), are site-dependent ground motion coefficients that define the seismic response throughout the spectral range. They are measures of expected ground acceleration at a site additionally, in seismic zone 4, they also depend on the seismic source type and near-source factors N_a and N_v.

Table (2.2): Seismic Coefficient C_v.

Soil profile type	Seismic zone factor, Z				
	$Z = 0.075$	$Z = 0.15$	$Z = 0.2$	$Z = 0.3$	$Z = 0.4$
S_A	0.06	0.12	0.16	0.24	$0.32N_v$
S_B	0.08	0.15	0.20	0.30	$0.40N_v$
S_C	0.13	0.25	0.32	0.45	$0.56N_v$
S_D	0.18	0.32	0.40	0.54	$0.64N_v$
S_E	0.26	0.50	0.64	0.84	$0.96N_v$
S_F	Site-specific geotechnical investigation and dynamic site response analysis shall be performed for soil type S_F.				

(From UBC 1997, Table 16-R.)

Table (2.3): Seismic Coefficient C_a.

Soil profile type	Seismic zone factor, Z				
	$Z = 0.075$	$Z = 0.15$	$Z = 0.2$	$Z = 0.3$	$Z = 0.4$
S_A	0.06	0.12	0.16	0.24	$0.32N_a$
S_B	0.08	0.15	0.20	0.30	$0.40N_a$
S_C	0.09	0.18	0.24	0.33	$0.40N_a$
S_D	0.12	0.22	0.28	0.36	$0.44N_a$
S_E	0.19	0.30	0.34	0.36	$0.36N_a$
S_F	Site-specific geotechnical investigation and dynamic site response analysis shall be performed for soil profile S_F.				

(From UBC 1997, Table 16-Q.)

Distribution of Lateral Force F_x is the base shear V distributed over the height of the structure as a force at each level F_i, plus an extra force F_t at the top

$$V = F_T + \sum_{i=1}^{n} Fi \qquad (2.16)$$

The extra force at the top is

$F_t = 0.07TV \leq 0.25V$ if $T > 0.7$ sec

$F_t = 0$ if $T \leq 0.7$ sec

F_t accounts for the greater participation of the higher-mode responses of longer-period structures.

The remaining portion of the total base shear (V – Ft) is distributed over the height, including the top, by the formula

$$F_x = \frac{(V - F_t)(w_x h_x)}{\sum_{i=1}^{n} wi\ hi} \qquad (2.17)$$

where w is the weight at a particular level, and h is the height of that level above the shear base.

The story shear at level x is the sum of all the story forces at and above that level

$$V_x = F_t + \sum_{i=x}^{n} Fi \qquad (2.18)$$

The overturning moment at a particular level M_x is the sum of the moments of the story forces above, about that level. Is given by:

$$Mx = Ft\,(hn - hx) + \sum_{i=x}^{n} Fi(hi - hx) \qquad (2.19)\ [4]$$

2.4 Lateral Loads Resisting Systems

Structural engineering of tall buildings requires the use of different systems for different building heights. Each system, therefore, has an economical height range, beyond which a different system is required. The requirements of these systems and their ranges are somewhat imprecise because the demands imposed on the structure significantly influence these systems. However, knowledge of different structural systems, their approximate ranges of application would be useful for structural engineer to make the preliminary design and the alternatives for new project. Different Structural systems for concrete buildings are listed in table 2.4

Table (2.4): lateral Structural systems for concrete buildings

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		

The principles of efficient tall building structural design, known for some time, are quite simple:

1. Resist overturning forces due to lateral loads by using vertical elements placed as far apart as possible from the geometric center of the building
2. Channel gravity loads to those vertical elements resisting overturning forces
3. Link these vertical elements together with shear-resisting structural elements that experience a minimum of shear lag effects such that the entire perimeter of the building resists the overturning moments

4. Resist lateral forces with members axially loaded in compression rather than those loaded in tension due to overturning.[1]

2.4.1 The Flat slab-frame system

Perhaps one of the simplest framing techniques for a concrete building consists of a two-way floor slab framing directly into columns without beams lateral drift requirements limit their economical height to about 10 stories, The term flat slab-frame signifies that the flat slab behaves as a beam, responding to lateral loads by developing bending moments and shear forces, The slab system has two distinct actions in resisting lateral loads. First, because of its high in-plane stiffness, it distributes the lateral loads to various vertical elements in proportion to their stiffness. Second, because of its significant out-of-plane stiffness, it restrains the vertical displacements and rotations of columns and walls as if they were interconnected by a shallow wide beam.

The partitions and other non -structural element stiffness are neglected in lateral stiffness considerations (it is known that the partitions act as struts and contribute in lateral resistance in the moments frame) due to many reasons as it will be removed in future.[1][6][7]

2.4.2 The Shear walls

A shear walls structure is considered to be one which resistance to horizontal loading is provided entirely by shear walls, the wall may be act as the service core (double function, structurally as shear wall, architect for service as elevator) or may act as partitions, their high in plane stiffness make it well suited for bracing building of up to 40 stories, it can be categorize it to:

- Proportionate system which the ratios of the flexural rigidities of walls remain constant throughout the height

- Non Proportionate system which the ratios of the flexural rigidities of walls are not remain constant throughout the height

A system of interconnected shear walls (see Figure 2.16), exhibits a stiffness that far exceeds the summation of the individual wall stiffnesses. This is because the interconnecting slab or beam restrains the cantilever bending of individual walls by forcing the system to work as a composite unit it’s called coupled shear walls.[1][6]

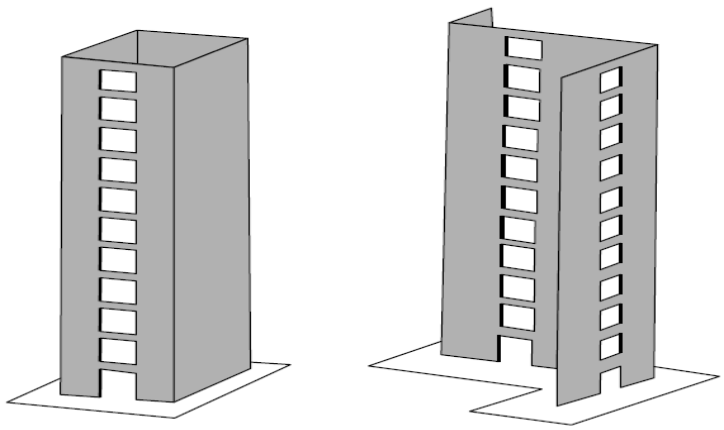


Figure (2.16): coupled shear wall

2.4.3 The Rigid frame

A rigid frame is characterized by flexure of beams and columns and rotation at the joints, the connection is subjected to large shear forces, the advantages of a rigid frame are the simplicity and convenience of its rectangular form. Its unobstructed arrangement, clear of structural walls, allows freedom internally for the layout and externally for the fenestration. Rigid frames are considered economical for buildings of up to about 25 stories.

However, if a rigid frame is combined with shear walls, the resulting structure is very much stiffer so that its height potential may extend up to 50 stories or more. The horizontal stiffness of a rigid frame is governed mainly by the bending resistance of the girders, the columns, and their connections, and in a tall frame, also by the axial rigidity of the column. The rotational deformations of the columns and girders result in shear deflection, often referred to as frame racking, greatly contributing to the horizontal deflection. The deflected shape of a rigid frame due to racking has a shear configuration with concavity upwind that has a maximum inclination near the base and a minimum at the top.

The overall external moment is resisted at each level by a couple resulting from the axial tensile and compressive forces in the columns on opposite sides of the structure. The extension and shortening of the columns cause overall bending and associated horizontal displacements due to curvature of the structure. Because of the cumulative rotation up the height, the story drift due to overall bending increases with height, while that due to racking tends to decrease.

To avoid progressive collapse, the principle of strong column-weak beam is applied (This is specified in the ACI 318 by requiring that the sum of the column flexural strengths exceed the sum of beam flexural strengths at each beam‑column connection by at least 20%).[1]

2.4.4 The Tabular systems

The term tube, in usual building terminology, suggests a system of closely spaced columns (2.43‑4.57 m), tied together with a relatively deep spandrel. However, for buildings with compact plans, it is possible to achieve tube action with relatively widely spaced columns interconnected with deep spandrels. 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 serious consideration for buildings taller than about 40 stories.

It can be consider as hollow tube, the direction which facing the loads is called the flange and the other is web, it is subjected to the effects of shear lag, which has a tendency to modify the axial distribution in the columns. The influence of shear lag, considered presently in the following section, is to increase the axial stresses in the corner columns while simultaneously reducing the same in the inner columns of the flange and the web panels.[1]

However to overcome the shear lag problem and improves the efficiency of the framed tube by increasing its potential for use in taller buildings and allowing greater spacing between the columns. This is achieved by adding diagonal bracing at the faces of the tube to virtually eliminate the shear lag in both the flange and web frames and the result system called braced tube. There is another type called a bundled tube response is to connect two or more individual tubes into a single bundle, The main purpose is to decrease shear lag effects, The cells can be stopped at selected heights without diminishing structural integrity.

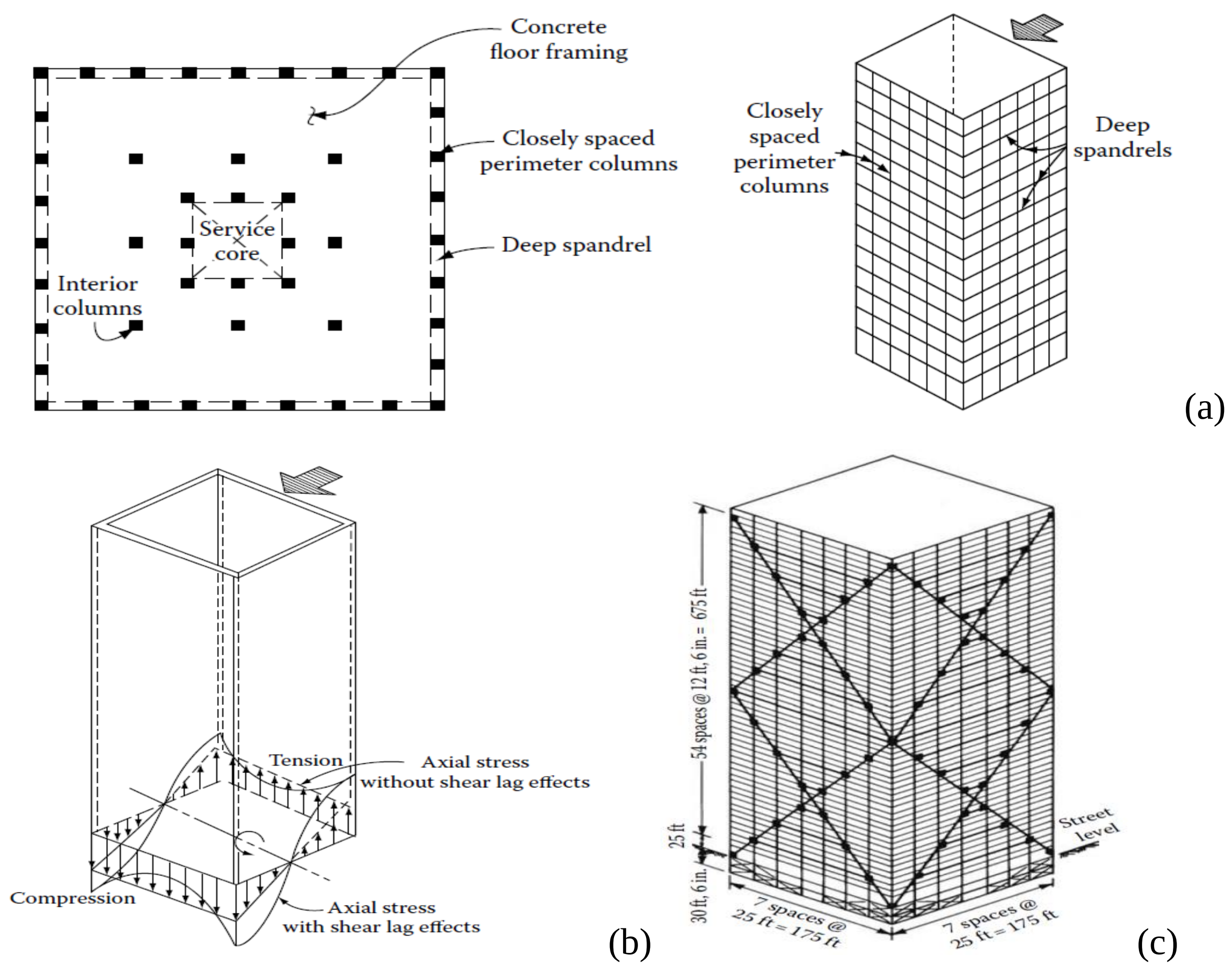


Figure (2.17): (a) tube system (b) shear lag (c) braced tube

2.4.5 The 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.[6]

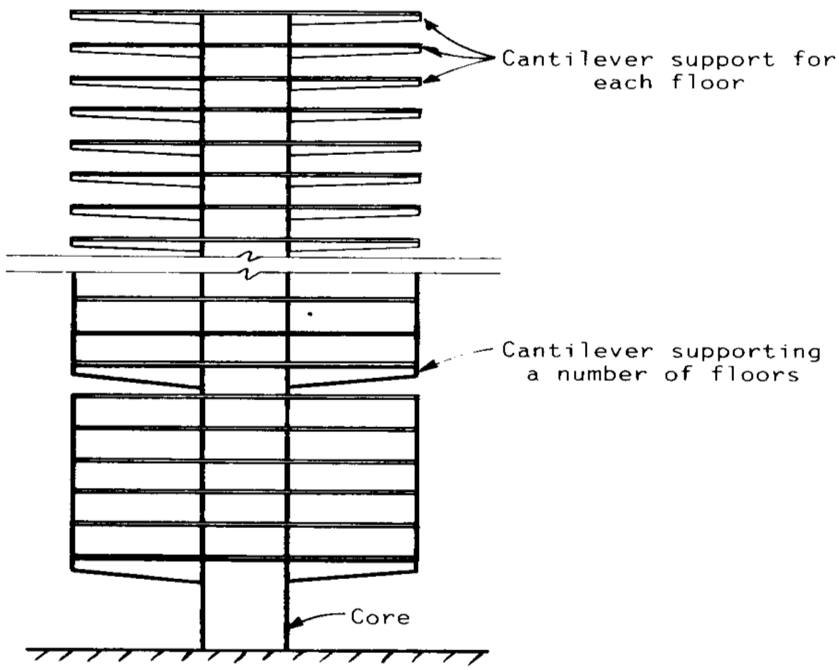
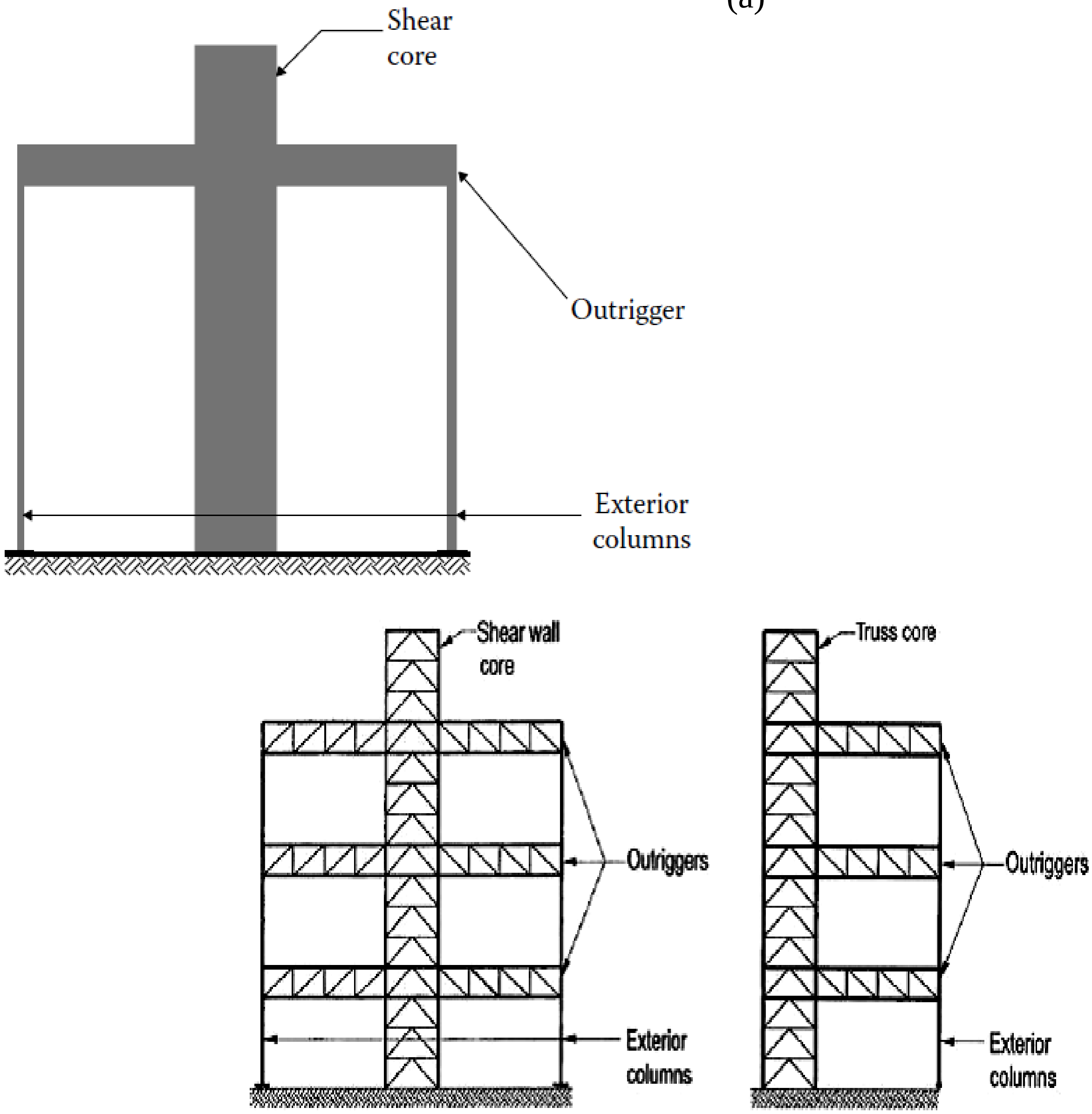


Figure (2.18): core structure

2.4.6 The Outrigger and belt wall system

The structural arrangement for this system consists of a main concrete core connected to exterior columns by relatively stiff horizontal members such as a one or two-story deep walls commonly referred to as outriggers. The core may be centrally located with outriggers extending on both sides (Figure 2.19), or it may be located on one side of the building with outriggers extending to the building columns on one side. Structural response of the system is quite simple, when subjected to lateral loads, the column-restrained outriggers resist the rotation of the core, causing the lateral deflections and moments in the core to be smaller than if the freestanding core alone resisted the loading. The external moment is now resisted not by bending of the core alone, but also by the axial tension and compression of the exterior columns connected to the outriggers.[1][6]

(a)



(b)

Figure (2.19): (a) wall acting as outrigger (b) truss as outrigger (central core on left offset in right)

In addition to those columns located at the ends of the outriggers, it is usual to also mobilize other peripheral columns to assist in restraining the outriggers. This is achieved by including a deep Spandrel Girder, or a Belt Truss, around the structure at the levels of the outriggers. Here, it should be noted that while the outrigger system is very effective in increasing the structure's flexural stiffness, it doesn't increase its resistance to shear, which has to be carried mainly by the core.

2.4.6.1 The Optimum location

Many studies has been done in this field, in 1974, Taranath examined the optimum location of a belt truss which minimized the wind sway and discussed a simple method of analysis (with several assumptions, lateral loads is constant over building and cross sectional area of columns and shear wall are fixed over height), he found the optimum location is H/2 from the top for one outrigger, for tow (H/3,2H/3), for three outrigger is (H/4, H/2, 3H/4) from the top .[1]

McNabb (1975) extended their analysis to two outriggers and investigated governing factors in drift reduction. McNabb (1975) verified the Taranath's optimum outrigger location result and showed that the optimum locations for two outriggers to be 0.312 and 0.685 of the total height from the top of the building. In 1985, Moudarres (1985) investigated the free vibration of high rise structures using dynamic analysis and this treatment took into account the effects of shear deformation and rotatory inertia of the core and included the inertia of the outrigger.

Chan and Kuang (1989a, 1989b) conducted studies on the effect of an intermediate stiffening beam at an arbitrary level along the height of the walls and indicated that the structural behavior of the structure could be significantly affected by the particular positioning of this stiffening beam. For preliminary analysis of outrigger braced structures, simple approximate guidelines for the location of the outriggers were given in Smith (1991). [8]

Moudarres conducted the study of a pair of coupled shear walls stiffened at the top by a flexible outrigger, and investigated the outrigger's influence on the behavior of the walls. The treatment of coupled shear walls stiffened at the top by an outrigger is approached by considering the un-stiffened walls under the influences of external loads and internal forces, respectively. The vertical axial forces and the concentrated moments imposed at the top of the walls are internal forces due to the influence of the stiffening outrigger. [8]

Alex Coull and W. H. Otto Lau conducted a study of a multi outrigger-braced structure based on the continuum approach in which the set of outriggers is smeared over the height to give an equivalent uniform bracing system. After their detail analysis they concluded that, Continuum analysis can give reasonably accurate results for even a very small number of Outriggers. They also presented Design Curves for assessing the lateral drift and the core base moments for any structural configuration defined in terms of two controlling structural parameters. The curves allow a direct assessment of the effectiveness of any number of outriggers.

R. Shankar Nair presented a paper on the detail study of various types of outriggers and their relative behavior and performance subjected to lateral loading along with their advantages and disadvantages. He also conducted an analysis for a typical steel structure employing various types of outriggers.[8]

2.4.6.2 Problems with Outriggers

There are several problems associated with the use of outriggers, problems that limit the applicability of the concept in the real world:

1. The space occupied by the outrigger trusses (especially the diagonals) places constraints on the use of the floors at which the outriggers are located. Even in mechanical equipment floors, the presence of outrigger truss members can be a major problem.

2. Architectural and functional constraints may prevent placement of large outrigger columns where they could most conveniently be engaged by outrigger trusses extending out from the core.
3. The connections of the outrigger trusses to the core can be very complicated, especially when a concrete shear wall core is used.
4. In most instances, the core and the outrigger columns will not shorten equally under gravity load. The outrigger trusses, which need to be very stiff to be effective as outriggers, can be severely stressed as they try to restrain the differential shortening between the core and the outrigger columns. Elaborate and expensive means, such as delaying the completion of certain truss connections until after the building has been topped out, have been employed to alleviate the problems caused by differential shortening.[8]

Chapter Three

The Problem Statements and Analysis

Chapter Three

The Problem Statements and Analysis

3.1 General

Based on the wisdom says "(if you find that you are spending almost all your time on theory, start turning some attention to practical things, it will improve your theories, and If you find that you are spending almost all your time on practice, start turning some attention to theoretical things, it will improve your practice)".

In theory, there has been much research on topic of structural lateral loads resistant system optimization, however, additional investigation should be done on its practical application. In this chapter we will observe the study of several cases to achieve the research objectives, structures cases has been modeled by using sophisticated computer program packages named ETABS and SAP 2000, and the results will be shown in term of figures and tables.

3.2 Description of ETABS program

Extended Three dimensional Analysis of Building System (ETABS), is based on finite element methods, for structural analysis and design. The package is a fully integrated system for modeling, analyzing, designing, and optimizing structures of tall buildings, the program utilize graphical user interface (GUI) as shown in figure (3.1). its provide multiple units systems, preferences for most codes of design such as (ACI,BS, UBC,IBC and Euro code) are included, automated lateral loads- wind, seismic- and provide static and dynamic analysis (response spectrum & time history), linear and nonlinear analysis (time history & push over), with features of importing from other programs (plan, three dimensional frame from AutoCAD) , exporting the objects and results for other program (i.e. floor to SAFE, reactions for designing raft) for more process , and export to the spread sheets (excel).

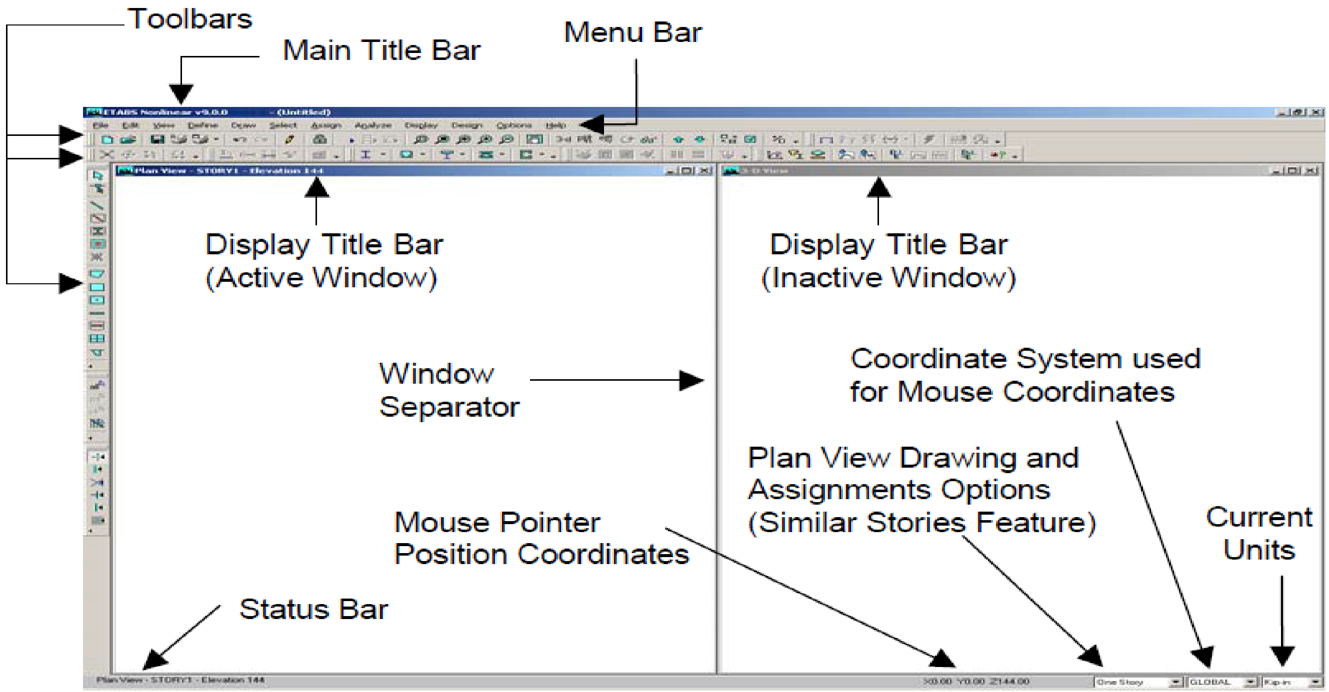


Figure (3.1): ETABS graphic user interface

3.3 Study of Shear walls cases

3.3.1 Problem description

The objective of the present work is to find the optimum location of shear walls interim of the distance from the center of mass, The model considered for this study is a 30 m reinforced concrete building frame , the plan is 25×25m with columns spaced about 5m from center to center in each direction, the typical height of story is 3 m, the floor system is assumed to be typical, to avoid twisting moment the center of rigidity coincide as possible with center of the mass, the assumptions of analysis (materials density and properties ,loading intensity) based on British standard code except the seismic loads which will be done according to UBC 97. The ETABS software program is selected to perform analysis, the analysis would perform for three models, model (A); with shear walls very close to the CM , model (B); shear walls located at mid distance between the CM and the perimeter and model (C); the shear walls located in perimeter of the building,

3.3.2 Model (A): shear walls near the center of building

The model consist of 10 stories with story height equal to 3 m, it's proposed to be used as office building, the shear walls located near mid plan as shown in figure (3.2), the center of rigidity (C.R) coincide with center of mass (C.M), the building assume to be lied in medium seismic zone, zone 2A and the soil type is very dense.

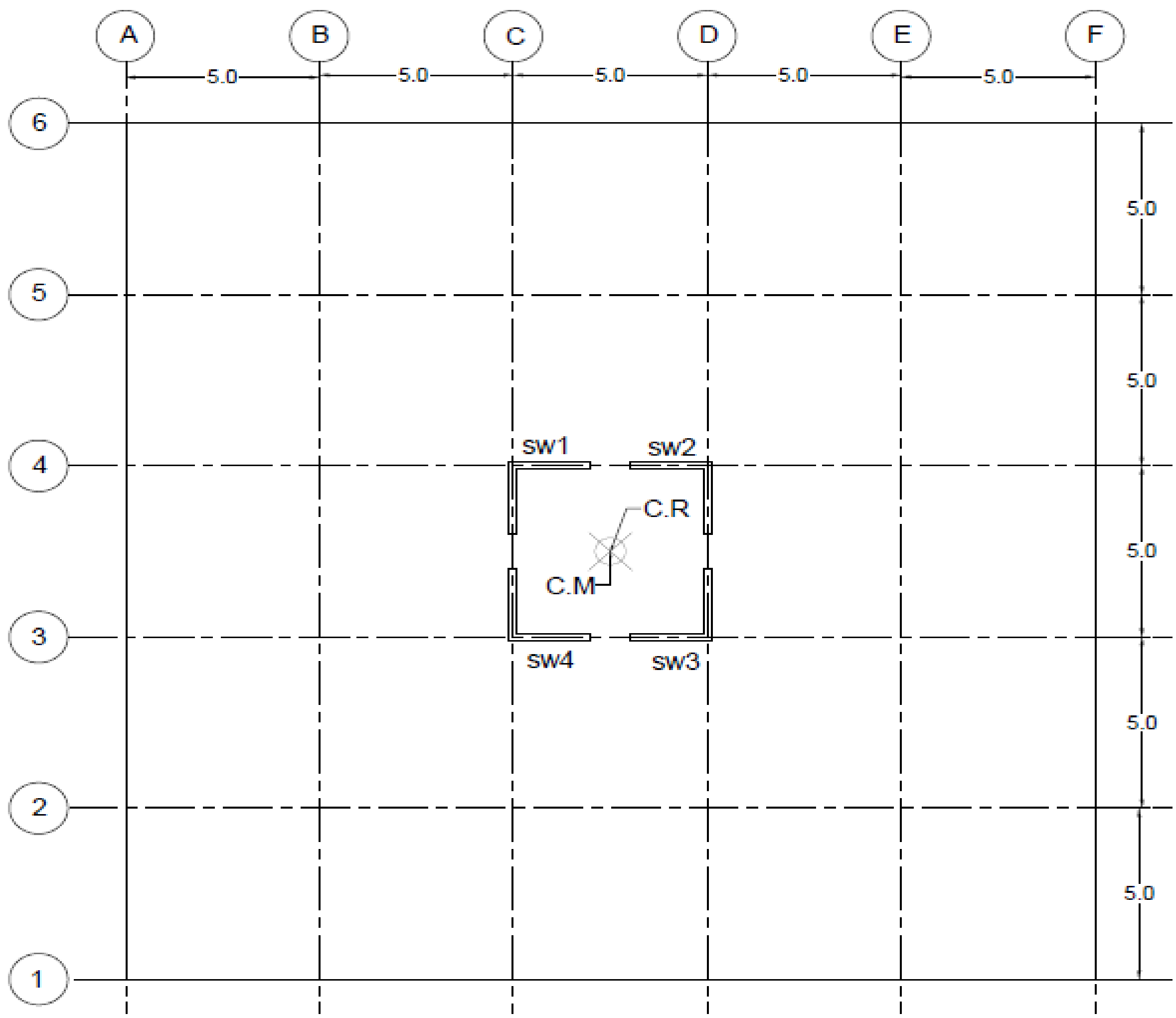


Figure (3.2): Plan of Model (A)

3.3.2.1 Loading

- The dead load calculated from the self-weight of reinforced concrete members based on density 24 KN/m^3

  Floor finish screed = 1.8 KN/m^2

  Block work partition = 2.5 KN/m^2

- Live loads according to BS 6399- part 1 from Table 1 (Minimum imposed floor loads) is 2.5 KN/m^2
- The loads combinations considered as following:
  1- 1.4 D.L (3.1)
  2- 1.4 D.L +1.6 L.L (3.2)
  3- 1.2 D.L +1.2 L.L± 1.2W.L (or E.L) (3.3)
  4- 1D.L±1.4W.L (or E.L) (3.4)
  5- 1.4D.L±1.4W.L (or E.L) (3.5)

Where D.L ≡ dead load, L.L ≡ live load, W.L ≡ wind load & E.L ≡ earth quake loads

3.3.2.2 Materials properties

Table (3.1): Materials Properties

Parameter	description	Strength N/mm^2	E KN/mm^2	Remarks
Reinforcements	F_y 460	460	200	All members
	f_{yv}250	250	200	As shear rein.
concrete	C25	25	25	slabs
	C30	30	26	Walls/columns

3.3.2.3 Preliminary sizing of members

According to the Manual for the design of reinforced concrete building structures, the depth of the flat slab controlled by the deflection (SLS), from table 3; Span / effective depth for final design = 26. From figure (3.2) the maximum span = 5 m, slab thickness = 5/26 = 0.192m so use 20 cm. For columns, the initial size was evaluated for three columns (interior, edge, and corner) using tributary area for evaluating the load carried by each column from the gravity load only , the concept of design repetition is applied, then the sections will be constant every three storey, and the live loads reduction factors applied according to BS 6399-1

Table (3.2): Evaluating of ultimate loads carried by columns

Column type	Location	Dead load KN/m^2	Live load After reduction	tributary area m^2	Load KN
Interior column	Gr – 3rd floor	Slab,0.2*24=4.8 Finish =1.8 Partitions =2.5 Total = 9.1 Design = 127.4	= 2.5*0.5*10 =12.5 Design = 20	25	3685
	4th -6th floor	Design= 76.44	= 2.5*.6*6 = 9 Design =14.4	25	2271
	7th- 9th	Design =32.22	=2.5*.8*3 =6 Design = 9.6	25	1045.5
Edge column	Gr – 3rd floor	Design = 127.4	Design = 20	12.5	1842.5
	4th -6th floor	Design= 76.44	Design =14.4	12.5	1135.5
	7th- 9th	Design =32.22	Design = 9.6	12.5	522.75
Corner column	Gr – 3rd floor	Design = 127.4	Design = 20	6.25	921.25
	4th -6th floor	Design= 76.44	Design =14.4	6.25	567.75
	7th- 9th	Design =32.22	Design = 9.6	6.25	261.4

Referring to the design code (BS 8110-1-97) formula which is used to determine the ultimate capacity of columns;

$$N_u = 0.35f_{cu}A_c + 0.7A_{sc}f_y \qquad (3.6) \quad \text{(BS 8110 eq 39)}$$

By assume the A_{sc} = 1% A_c (optimum percentage)

$$N = 0.35\times30\times A_c + 0.7\times0.01\ A_c \times460$$

$$N = 10.5\ A_c + 3.22\ A_c$$

$$A_{c\ Req} = \frac{N}{13.72} \qquad (3.7)$$

Table (3.3): Columns cross sections

column type	Location	load kN	N KN	Ac reg mm2	B_{req} mm	B_{ch} mm	Ac_{ch} mm^2	A_{sc} mm^2
interior	Gr – 3rd floor	3685	4053.5	268586.01	518.3	550	302500	3025
	4th -6th floor	2271	2498.1	16524.78	406.8	450	202500	2025
	7th- 9th	1046	1150.6	76239.067	276.1	300	90000	900
edge	Gr – 3rd floor	1843	2027.3	134329.45	366.5	400	160000	1600
	4th -6th floor	1136	1249.6	82798.834	287.7	300	90000	900
	7th- 9th	522.8	575.08	38104.956	195.2	200	40000	400
corner	Gr – 3rd floor	921.3	1013.43	67150.146	259.1	300	90000	900
	4th -6th floor	567.8	624.58	41384.84	203.4	250	62500	625
	7th- 9th	261.4	287.54	19052.478	138.0	200	40000	400

Note: the cross-section assumed to be square

Ac_{reg} determined by using Eq (3.7)

Table (3.4): Reinforcement details of columns

column type	Location	A_{sc} mm2	BAR dia./mm	A_S BAR/mm2	BAR NO	act BAR
interior	Gr – 3rd floor	3025	20	314	9.63	10
	4th -6th floor	2025	20	314	6.45	8
	7th- 9th	900	12	113	7.96	8
edge	Gr – 3rd floor	1600	16	201	7.96	8
	4th -6th floor	900	16	201	4.48	6
	7th- 9th	400	12	113	3.54	4
corner	Gr – 3rd floor	900	16	201	4.48	6
	4th -6th floor	625	16	201	3.11	4
	7th- 9th	400	12	113	3.54	4

3.3.2.4 ETABS Inputs

The evaluated loading in clause (3.3.2.1) will be used as gravity loads, the cross sections of columns which determined in Table (3.3) and (3.4) are used as inputs. The slab and walls thickness is 200mm, and the inputs for wind loads and seismic were tabulated below.

Table (3.5): Wind loads inputs (BS6399)

Load case	V_e (m/s)	C_a	C_r	Direction angle
W_X	30	1	0.25	0
W_Y	30	1	0.25	90

Table (3.6): Equivalent lateral loads parameters for seismic (UBC97)

Parameter	Value	Remarks
Soil profile	SC	UBC table 16-J
Seismic zone factor	0.15	From Table 2.1
Time period	0.6255 sec	Eq 2.14
Importance factor I	1	UBC table 16-K
Over strength factor R	5.5	UBC table 16-N

The program will calculate the others factors required to calculate the base shear and story shears automatically, and will distribute the shear over structure. To consider the accident torsion eccentricity of ±5% apply to each direction

Table (3.7): Equivalent lateral loads cases

Load Case Name	Direction and Eccentricity	% Eccentricity
EQXA	X Dir + Eccen. Y	5
EQXB	X Dir - Eccen. Y	5
EQYA	Y Dir + Eccen. X	5
EQYB	Y Dir - Eccen. X	5

Table (3.8): Loads combinations

Name	description	Name	description
ULT1	1.4D+1.6 L	ULT4	1.2D+1.2L±1.2 EQXA
ULT2	$1.2D+1.2L\pm1.2W_X$	ULT5	1.2D+1.2L±1.2 EQXB
ULT3	$1.2D+1.2L\pm1.2W_Y$	ULT6	1.2D+1.2L±1.2 EQYA

Table (3.8): (continue)

Name	description	Name	description
ULT7	1.2D+1.2L±1.2 EQYB	ULTWXD	1.4D±1.4WX
WDX	1D±1.4WX	EXAD	1D±1.4 EQXA
UEXAD	1.4D±1.4 EQXA	EYAD	1D±1.4 EQYA
EXBD	1D±1.4 EQXB	EYBD	1D±1.4 EQYA

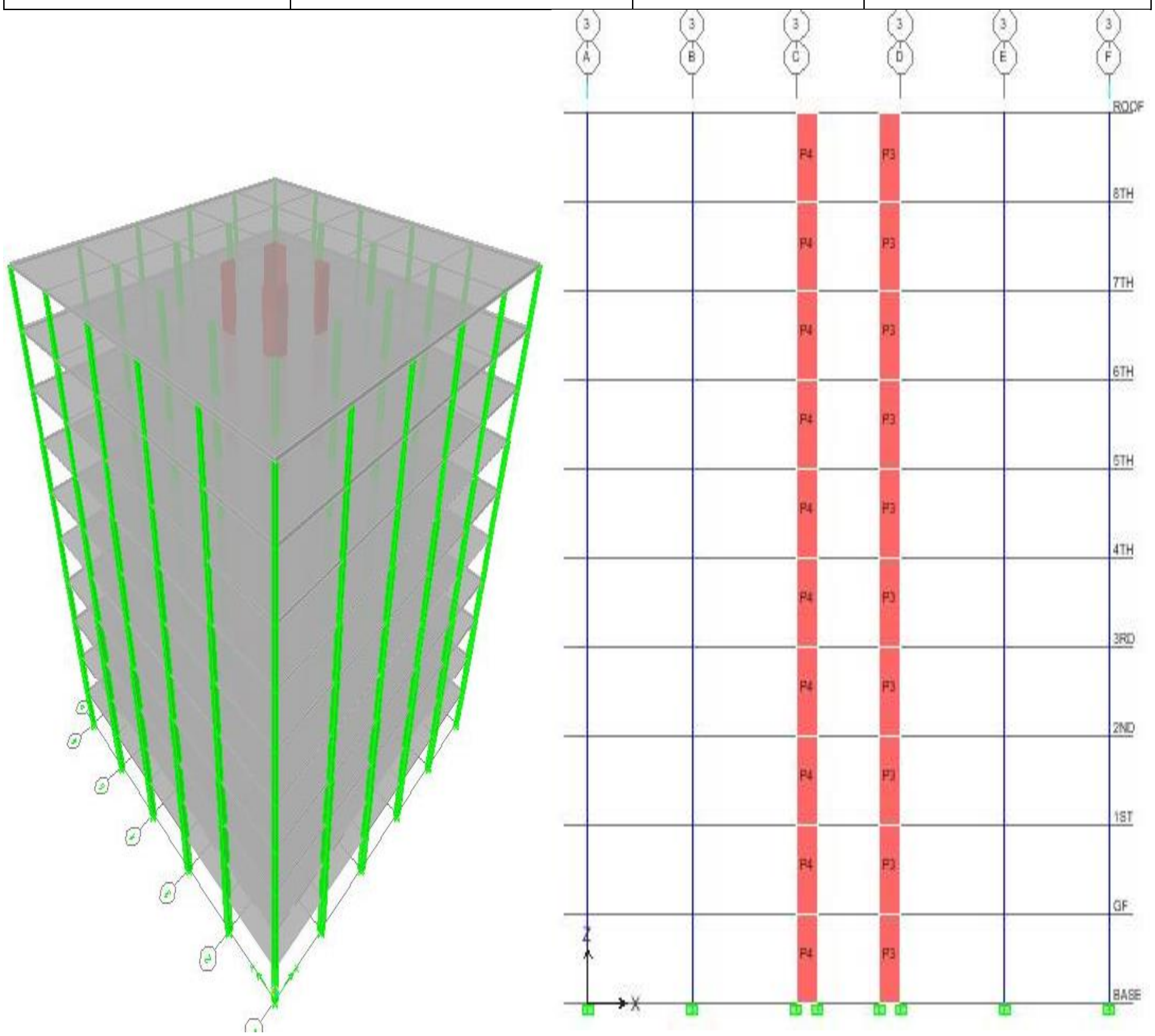


Figure (3.3): 3D view and side elevation

3.3.3 Model (B): shear walls at mid distance from the center

All inputs to the model as same as for model (A) except the following plan

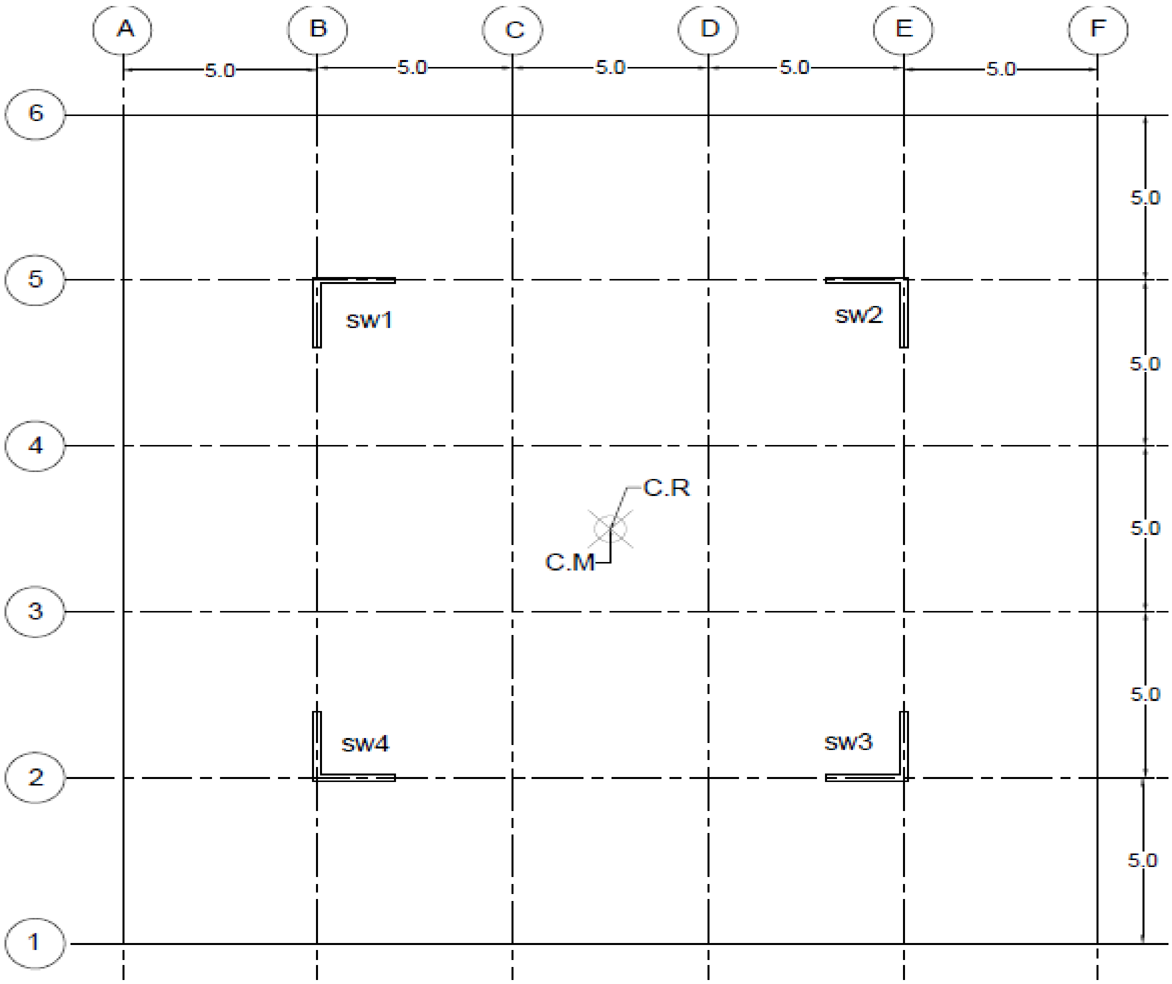


Figure (3.4): Model (B) Plan show the shear walls location

3.3.4 Model (C): shear walls at the perimeter

All inputs to the model as same as for model (A) except the following plan, the analysis results (reactions, drifts, and members forces) for the three models would be mentioned in chapter four.

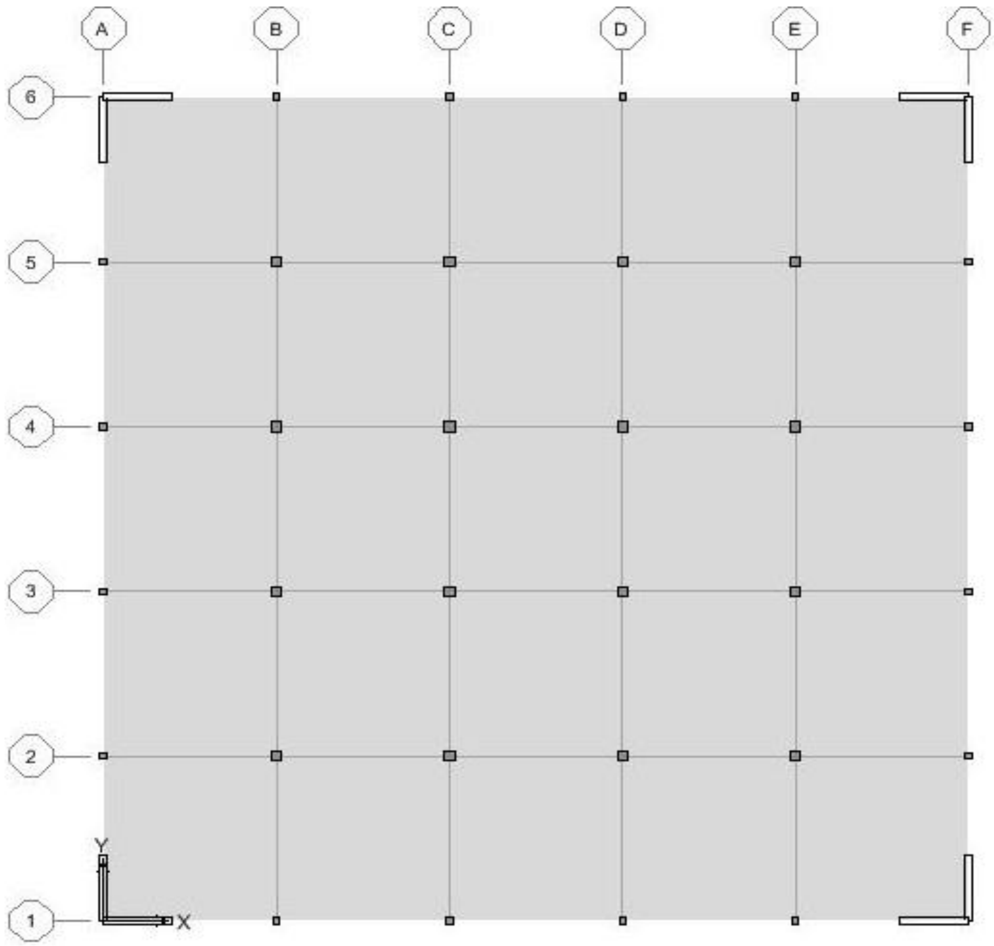


Figure (3.5): Model (C) Plan show shear walls Location

3.4 The study of rigid frame cases

For comparison reasons the same plan and inputs which used in the shear walls models, except small modification the Shear walls cross sections are distributed on edge and corner columns , and beams added with sizing shown below.

- Effective depth = (Max span / 15) = (5000/15) = 333 mm use 400 mm
  Width = 0.6×h = 0.6×400 = 240 use 250 mm as width (f_{cu} grade 25)
- Slab thickness is 150 mm. and Table (3.27) show the different inputs which will be used in the rigid frame model.

Table (3.9): Inputs for Rigid Frame Model

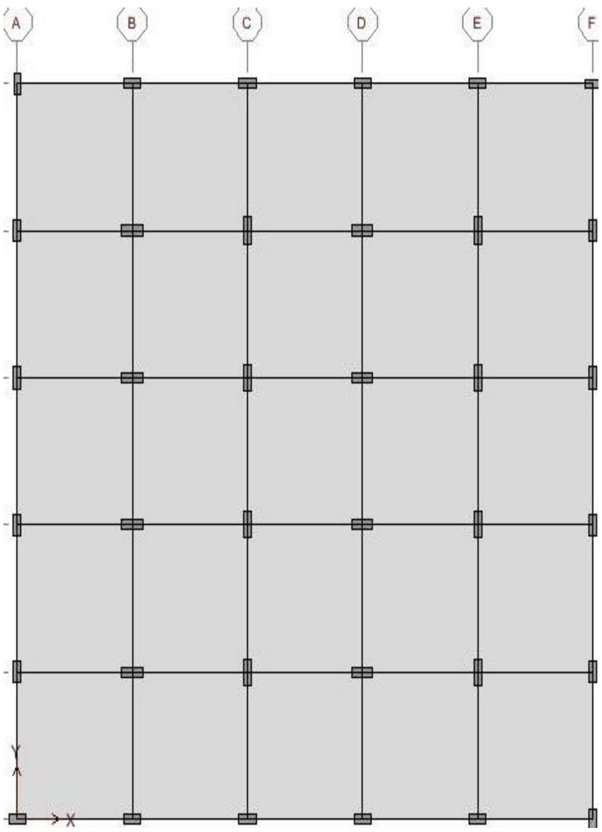
Parameter	Value	Remarks
Building time	0.937	Eq 2.14
Over strength factor R	3.5	UBC table 16-N

Three models would be observed, model (D): the rigid frame with square columns, model (E): the rigid frame with rectangular columns and the major axis of edge columns located with respect the gravity loads and model (F): rectangular columns with edge columns configured in proposed loop. And the best one will be selected and remodeled with beam depth 0.5 m.

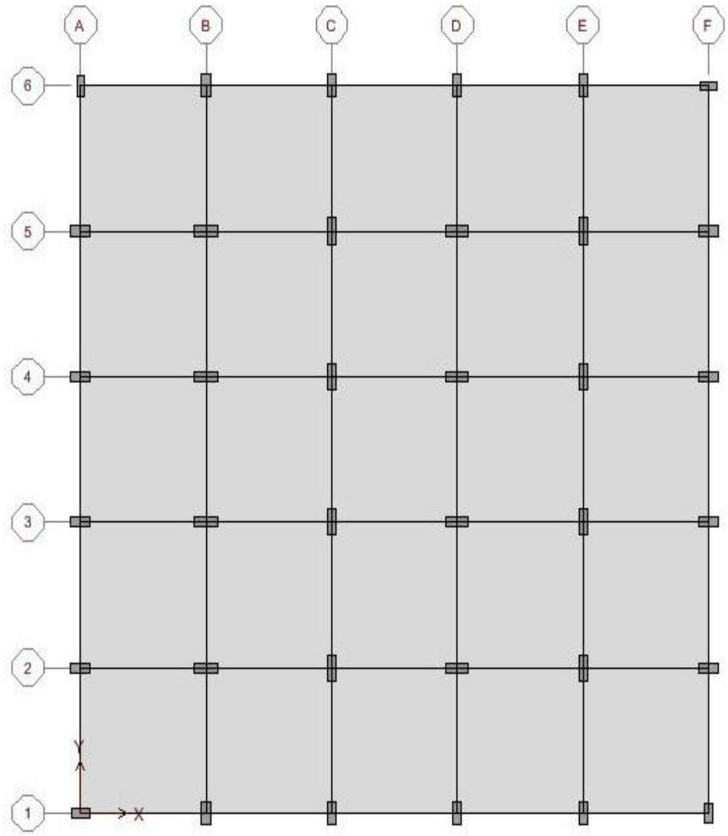
3.4.1 Sections properties

Table (3.10): Columns properties

location	Type	Model D		Model E & F		F_{cu} N/mm2	f_y N/mm2
		B mm	H mm	B mm	H mm		
G^F-3^{RD}	Interior	550	550	350	900	30	460
	edge	500	500	350	750	30	460
	corner	450	450	300	700	30	460
4^{th}- 6^{th}	Interior	450	450	350	600	30	460
	edge	450	450	300	700	30	460
	corner	400	400	250	650	30	460
7^{TH}-9^{TH}	Interior	400	400	250	550	30	460
	edge	350	350	250	550	30	460
	corner	350	350	250	550	30	460



(a) Model (F) Plan



(b) Model (E) plan

Figure (3.6): Models layout

3.5 Study of the Outrigger Cases

In this the efficiency of the outrigger system would be evaluated for regular system so as to investigate the optimum position in presence of vertical irregular due to mass located in the top of building (i.e. swim pool, heavy mechanical equipment, etc.) and irregular due to setback. The dynamic analysis would be adopted by using response spectrum method to evaluate the seismic loads for irregular building.

The following models would be investigated:

(a) For regular models to evaluate the efficiency of system we will investigate:

1- Model without outrigger

2- Models with One outrigger at (0.45, 0.5, 0.55 and 0.6 H).

3- Model with two outriggers (1/3H & 2/3H).

(b) For irregular models due to heavy mass at top (with additional load=15 KN /m^2):

1- Model with one outrigger at 0.4 H.

2- Model with one outrigger at 0.5 H.

3- Model with one outrigger at 0.6 H.

(c) For irregular models due to setback at 0.7 H:

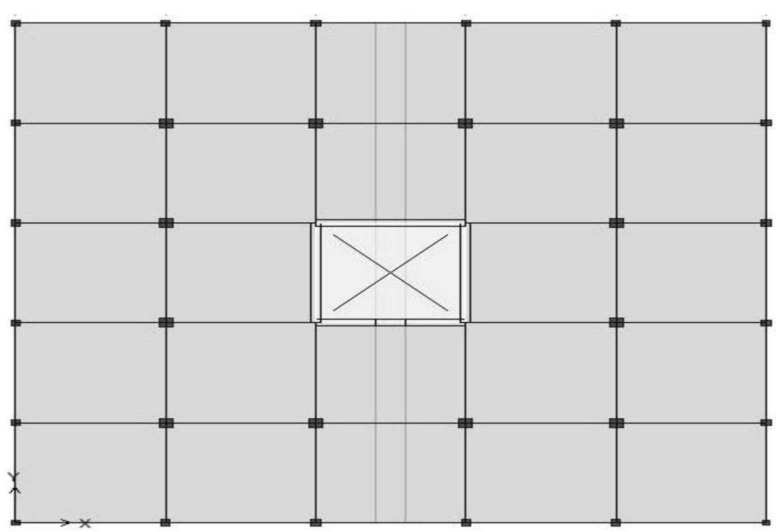
1- Model with one outrigger at 0.5 H.

2- Model with one outrigger at 0.6 H.

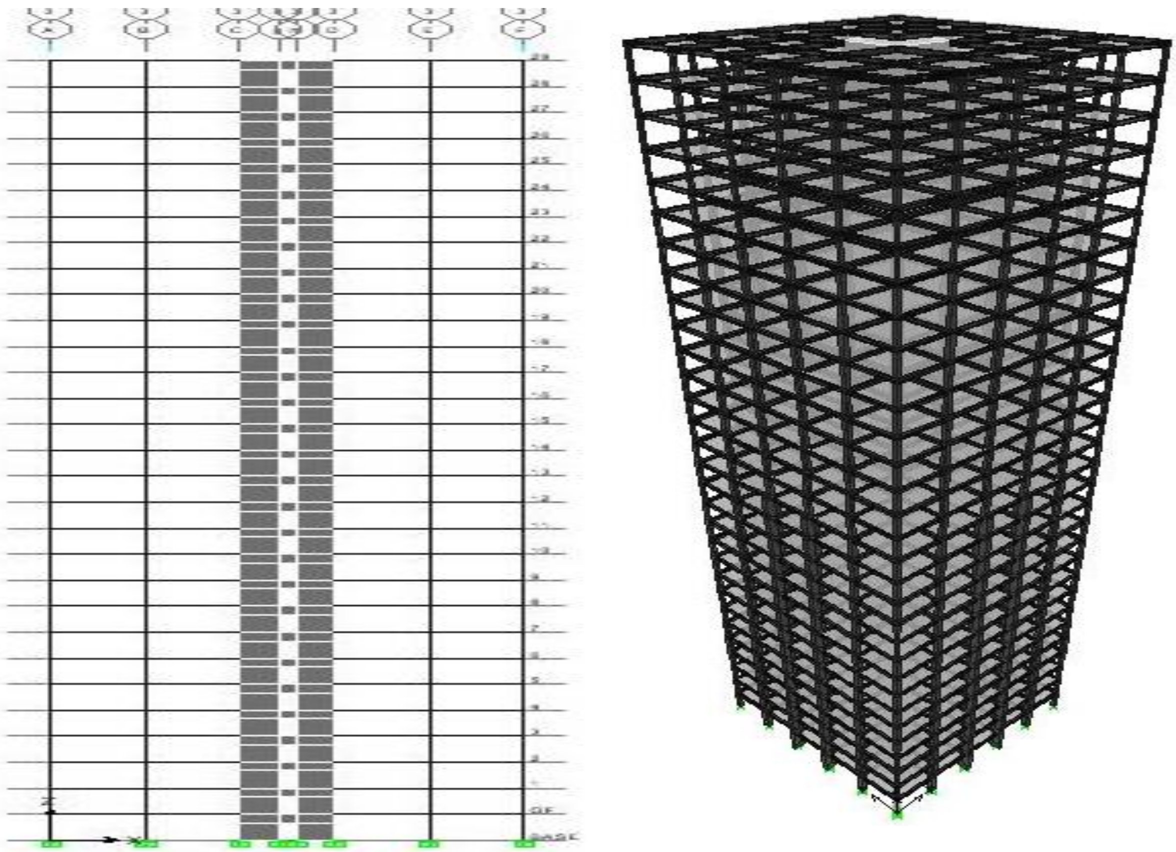
3- Model with one outrigger at 0.7 H.

3.5.1 Data Inputs

All models has the same plan as shown in figure (3.15.a), the overall height is 90m with 30 stories, each storey has a 3 m height, the building is intended to be used as offices, and for the seismic analysis the building was categorized in zone 3 according to UBC 97.



(a) Model plan



(b) Elevation of core (c) 3D view

Figure (3.7): Model system overview

(1) Loading

The dead load is calculated from the self-weight of different members with a density of concrete equal to 24 KN/m^3 and additional loads 1.8 KN/m^2 for block partitions, the live loads has been taken as 2.5 KN/m^2 for offices.

Table (3.11): Automated BS- 6399 Wind load parameters

Load case	V_e(m/s)	C_a	C_r	Direction angle
W_X	60	0.79	0.25	0
W_Y	60	0.79	0.25	90

Table (3.12): Automated UBC 97 Seismic loads parameters

Parameter	Value	Remarks
Soil profile	SC	UBC table 16-J
Seismic zone factor	0.3	From Table 2.1
Time period	1.4246 sec	Eq 2.14
Importance factor I	1	UBC table 16-K
Over strength factor R	5.5	UBC table 16-N

* Mass source from dead loads

Table (3.13): UBC 97 Response spectrum parameters

Parameter	Value	Remarks
C_a	0.33	UBC 97 Table 16-Q
C_V	0.45	UBC 97 Table 16-R
Scale factor	Variable	Equivalent static/ response spectrum

Table (3.14): loads cases and combinations

Load case	description	Load combination	description
D.L	dead	WXD	1 D.L+1.4 WX
L.L	live	EXD	1 D.L+1.4EQX
W_X- W_Y	Wind in X & Y	EYD	1 D.L+1.4EQY
EQ_X - EQ_Y	Static seismic x &y	SXD	1 D.L+1.4 SX
S_X - S_Y	Response spectrum x-y	SYD	1 D.L+1.4 SY

(2) Members properties

The slab is a solid with 0.15 m thickness with f_{cu} 30N/mm^2, the beams are 0.25 m width 0.4 m depth with f_{cu} 35N/mm^2, the shear walls is 0.35 m thickness constant throughout the height with concrete strength f_{cu} 40N/mm^2, the evaluation of modulus of elasticity for different concrete grades based on- BS 8110-97 part2:

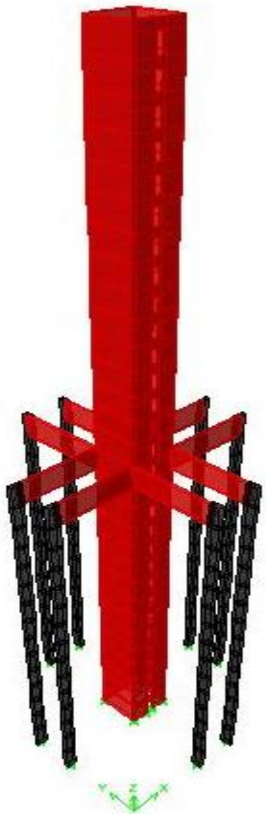
$$E_{c28} = 20 + 0.2\ F_{cu}\ (KN/mm^2) \qquad (3.1)$$

Table (3.15): Columns properties for models

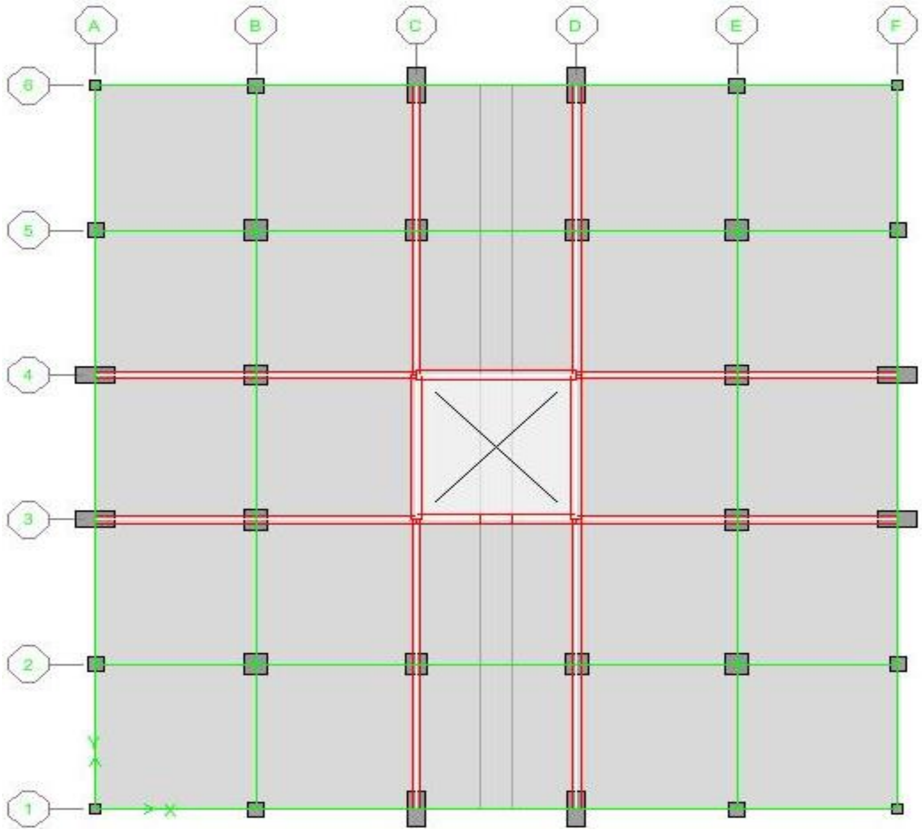
storey	column type	f_{cu}	f_y	B	H	storey	column type	f_{cu}	f_y	B	H
grd-5th	interior	60	460	800	800	16th-20th	interior	50	460	650	650
	edge	60	460	600	600		edge	50	460	450	450
	corner	60	460	400	400		corner	50	460	350	350
6th-10th	interior	60	460	750	750	21th-25th	interior	35	460	600	600
	edge	60	460	550	550		edge	35	460	450	450
	corner	60	460	400	400		corner	35	460	300	300
11th-15th	interior	50	460	700	700	26th-30th	interior	35	460	450	450
	edge	50	460	500	500		edge	35	460	300	300
	corner	50	460	350	350		corner	35	460	250	250

For the outrigger system the shear walls has 0.25 m thickness and f_{cu} equal to 35N/mm^2 and are used to connect the core (main shear walls) with mega columns - 1.2m depth 0.55 m width and f_{cu} equal to 60 N/mm^2- located at the perimeter of building and stopped at storey 14 as shown in figure (3.16) later.

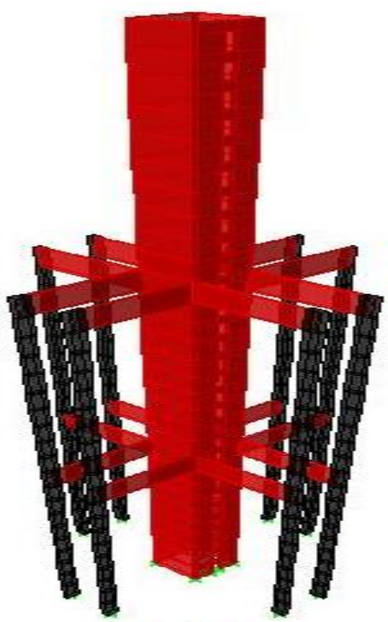
The analysis of all models would include dynamic modal analysis to comply with code requirements (UBC 97) which was stated that any structures has a height exceeds 73.15m the seismic loads should be calculated by dynamic method with effective mass participation not less than 90 percent.



(a) Outrigger 3D view



(b) Plan at the outrigger story



(c) Two outrigger

Figure (3.8): the outriggers system

Chapter Four

Results & Discussions of Results

Chapter Four

Results comparison and Discussions

4.1 General

In this chapter the results of the analysis would be introduced, for the shear walls results would be stated for every model and the vertical elements would be designed. Also for the rigid frame and for the outriggers models the results would be stated.

4.2 Shear Walls Results

4.2.1 Model (A) Analysis Results & design

(1) Reactions and moments at foundation

The reactions and moments at the base will be used to design the vertical elements at footprint to make structural plan density index, thus any column has three combinations to introduce (maximum loads with related biaxial moments, max M_{x},….etc.), and the results obtained to plan shown in figure (3.4).

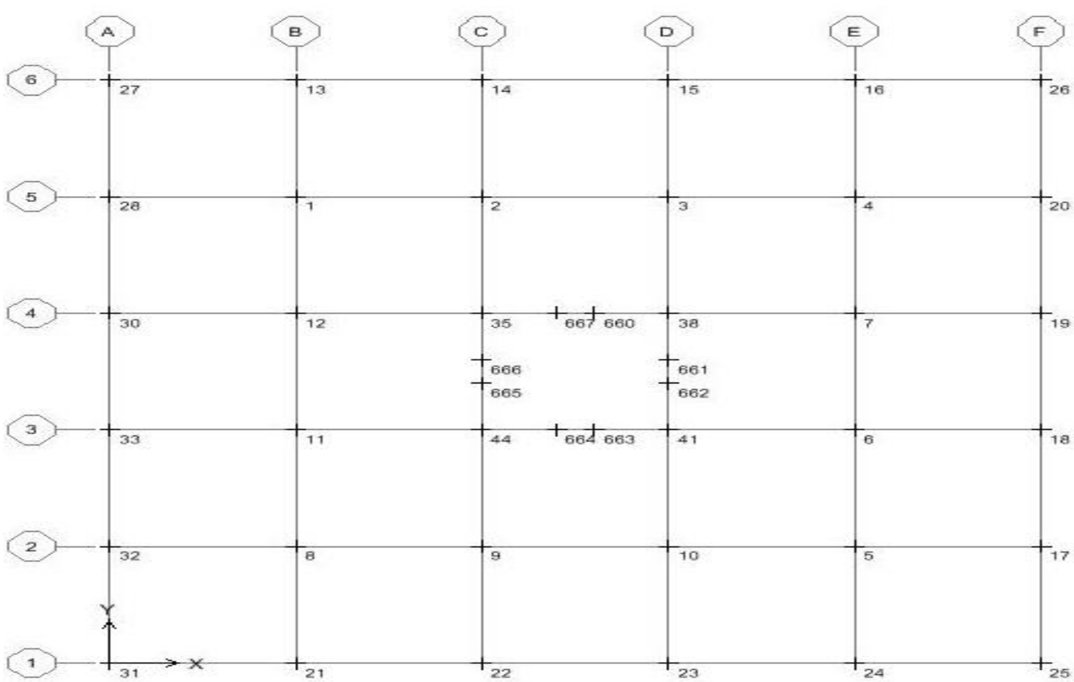


Figure (4.1): ETABS Labeled Columns

Table (4.1): Reactions at foundation.

Point Label	Load comb	FZ KN	MX KN.M	MY KN.M	Point Label	Load comb	FZ KN	MX KN	MY KN
1	ULT1	4933.5	-2.7	-2.7	13	ULT1	2055.0	16.4	-1.3
1	EYBD	2769.1	144.3	-28.6	13	UEYBD	1710.3	55.5	-14.5
1	UEXAD	3753.4	25.1	-147.8	13	UEXAD	1609.1	20.6	-49.3
2	ULT1	4424.6	-3.9	-0.7	14	ULT1	1969.8	15.5	0.2
2	EYBD	2729.4	126.8	-27.3	14	UEYBD	1621.9	49.4	-13.3
2	UEXAD	3318.6	5.5	-146.0	14	EXAD	1100.8	11.1	-48.3
3	ULT1	4424.6	-3.9	0.7	15	ULT1	1969.8	15.5	-0.2
3	EYAD	2729.4	126.8	27.3	15	UEYAD	1621.9	49.4	13.3
3	EXAD	2542.8	-10.6	-145.0	15	UEXAD	1511.4	9.1	-48.5
4	ULT1	4933.5	-2.7	2.7	16	ULT1	2055.0	16.4	1.3
4	EYAD	2769.1	144.3	28.6	16	UEYAD	1710.3	55.5	14.5
4	EXAD	2769.1	-28.6	-144.3	16	EXAD	1114.8	0.8	-47.6
5	ULT1	4933.5	2.7	2.7	17	ULT1	2055.0	1.3	-16.4
5	UEYAD	3753.4	147.8	-25.1	17	UEYAD	1609.1	49.3	-20.6
5	EXBD	2769.1	28.6	-144.3	17	UEXBD	1710.3	14.5	-55.5
6	ULT1	4424.6	0.7	3.9	18	ULT1	1969.8	-0.2	-15.5
6	UEYAD	3318.6	146.0	-5.5	18	EYAD	1100.8	48.3	-11.1
6	EXBD	2729.4	27.3	-126.8	18	UEXBD	1621.9	13.3	-49.4
7	ULT1	4424.6	-0.7	3.9	19	ULT1	1969.8	0.2	-15.5
7	EYAD	2542.8	145.0	10.6	19	UEYAD	1511.4	48.5	-9.1
7	EXAD	2729.4	-27.3	-126.8	19	UEXAD	1621.9	-13.3	-49.4
8	ULT1	4933.5	2.7	-2.7	20	ULT1	2055.0	-1.3	-16.4
8	UEYBD	3753.4	147.8	25.1	20	EYAD	1114.8	47.6	-0.8
8	UEXBD	3753.4	-25.1	-147.8	20	UEXAD	1710.3	-14.5	-55.5
9	ULT1	4424.6	3.9	-0.7	21	ULT1	2055.0	-16.4	-1.3
9	UEYBD	3132.0	131.9	26.4	21	EYBD	1013.7	34.2	12.8
9	UEXBD	3318.6	-5.5	-146.0	21	UEXBD	1609.1	-20.6	-49.3
10	ULT1	4424.6	3.9	0.7	22	ULT1	1969.8	-15.5	0.2
10	UEYAD	3132.0	131.9	-26.4	22	EYBD	990.3	29.2	13.5
10	EXBD	2542.8	10.6	-145.0	22	EXBD	1100.8	-11.1	-48.3
11	ULT1	4424.6	0.7	-3.9	23	ULT1	1969.8	-15.5	-0.2
11	UEYBD	3318.6	146.0	5.5	23	EYAD	990.3	29.2	-13.5
11	UEXBD	3132.0	-26.4	-131.9	23	UEXBD	1511.4	-9.1	-48.5
12	ULT1	4424.6	-0.7	-3.9	24	ULT1	2055.0	-16.4	1.3
12	EYBD	2542.8	145.0	-10.6	24	EYAD	1013.7	34.2	-12.8
12	UEXAD	3132.0	26.4	-131.9	24	EXBD	1114.8	-0.8	-47.6

Table (4.1): Continued

Point Label	Load comb.	FZ KN	MX KN.M	MY KN.M	Point Label	Load comb.	FZ KN	MX KN.M	MY KN.M
25	ULT1	927.9	-6.1	-6.1	30	ULT1	1969.8	0.2	15.5
25	EYAD	467.8	12.9	-7.8	30	UEYBD	1511.4	48.5	9.1
25	UEXBD	769.8	-0.2	-20.9	30	EXAD	990.3	13.5	-29.2
26	ULT1	927.9	6.1	-6.1	31	ULT1	927.9	-6.1	6.1
26	UEYAD	769.8	20.9	-0.2	31	EYBD	467.8	12.9	7.8
26	UEXAD	769.8	0.2	-20.9	31	EXBD	467.8	-7.8	-12.9
27	ULT1	927.9	6.1	6.1	32	ULT1	2055.0	1.3	16.4
27	UEYBD	769.8	20.9	0.2	32	UEYBD	1609.1	49.3	20.6
27	EXAD	467.8	7.8	-12.9	32	EXBD	1013.7	-12.8	-34.2
28	ULT1	2055.0	-1.3	16.4	33	ULT1	1969.8	-0.2	15.5
28	EYBD	1114.8	47.6	0.8	33	EYBD	1100.8	48.3	11.1
28	EXAD	1013.7	12.8	-34.2	33	EXBD	990.3	-13.5	-29.2

These results are checked by another structural program called SAP 2000, and Table (4.2) shows the comparison between results obtained by ETABS and SAP 2000 for selected columns reactions.

Table (4.2): Comparison between SAP 2000 and ETABS.

Column type	program	Label	Loads comb	N	M_X	M_Y
Interior	ETABS	1(B,5)	ULT1	4934	-2.7	-2.7
	SAP 2000	1(B,5)	ULT1	4935	-2.7	-2.7
	ETABS	1(B,5)	EYBD	2769	144	-28.6
	SAP 2000	1(B,5)	EYBD	2769	148	-29.4
	ETABS	1(B,5)	UEXAD	3753	25.1	-148
	SAP 2000	1(B,5)	UEXA	3755	25.9	-151
Edge	ETABS	14(C,6)	ULT1	1970	15.5	0.172
	SAP 2000	14(C,6)	ULT1	1970	15.5	0.172
	ETABS	14(C,6)	UEYBD	1622	49.4	-13.3
	SAP 2000	14(C,6)	UEYBD	1625	50.2	-13.7

Table (4.2): Continued

Column type	program	Label	Loads comb	N	M_X	M_Y
Edge	ETABS	14(C,6)	EXAD	1101	11.1	-48.3
	SAP 2000	14(C,6)	EXAD	1101	11.2	-49.4
corner	ETABS	26(F,6)	ULT1	927.9	6.12	-6.12
	SAP 2000	26(F,6)	ULT1	927.9	6.12	-6.12
	ETABS	26(F,6)	UEYBD	769.8	20.9	-0.23
	SAP 2000	26(F,6)	UEYBD	770.8	21.3	-0.1
	ETABS	26(F,6)	UEXAD	769.8	0.23	-20.9
	SAP 2000	26(F,6)	UEXAD	770.8	0.1	-21.3

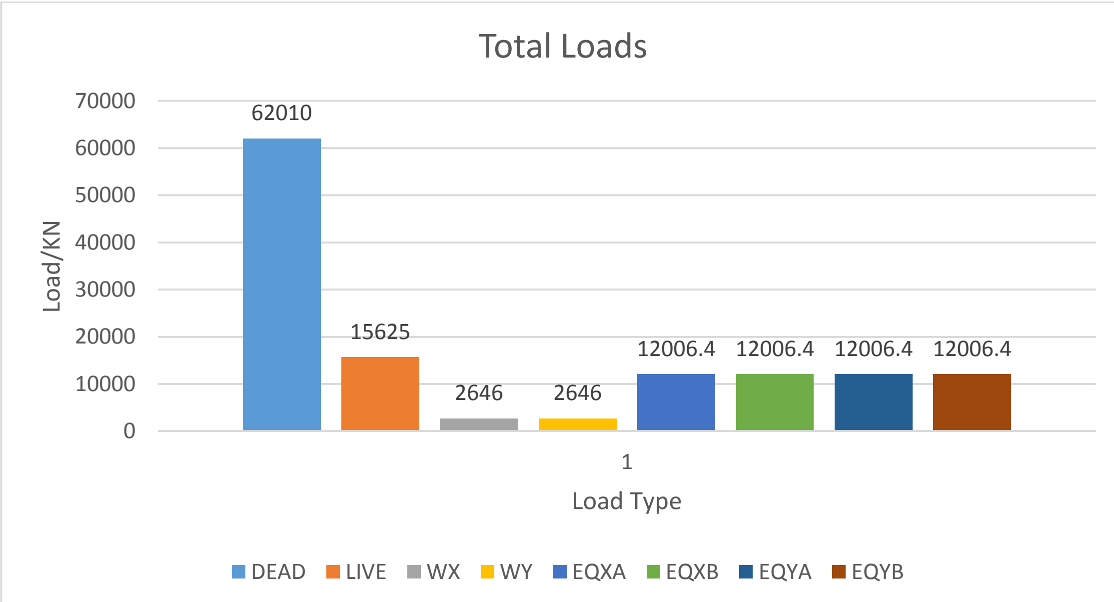
(2) Shear walls forces

Table (4.3) shows the analysis results for the shear walls which is labeled in the program as piers & spandrel (SW1…SW4), the combination shown maximum axial load, biaxial moments and torsion.

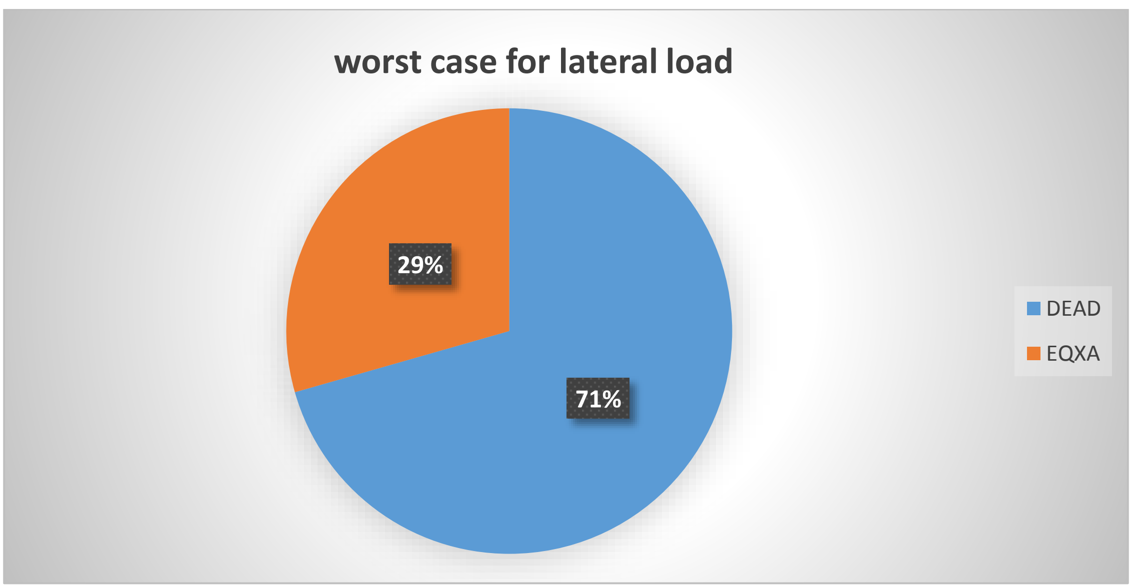
Table (4.3): Shear Walls Forces

Wall No	Load Comb.	P kN	M_x KN.M	M_y KN.M
SW1	UEYBD	-7979.5	1862.0	108.0
SW1	EYBD	-6769.2	1865.7	104.1
SW1	UEXAD	-492.6	83.1	1886.3
SW2	UEYBD	-7979.5	1720.0	35.2
SW2	EYAD	-6769.2	1865.7	-105.2
SW2	EXAD	-6769.2	105.2	1865.7
SW3	UEXBD	-7979.5	1769.5	34.0
SW3	UEYAD	-492.6	1886.3	83.7
SW3	EXBD	-6769.2	105.2	1865.7
SW4	ULT1	-7979.5	15.8	15.8
SW4	UEYBD	-492.6	1886.3	-83.7
SW4	UEXBD	-492.6	83.7	1886.3

The next figures the shows comparison between the total loads for every load case, and shows clearly that the lateral loads for the worst case can be about 30 % of the total loads acting on the building.



(a) Total loads for each case



(b) Worst case for lateral loads

Figure (4.2): Loads Acting on the Model

(3) Drifts

The maximum drift in each directions is shown in Table (4.4) with the limits value, and the maximum inter-storey drift for each storey shown in table (4.5)

Table (4.4): The Maximum Lateral Building Drift

Prog.	Load comb	Max drift x(m)	drift limits	status	load comb	max drift y(m)	status
ETABS	EQXAD	0.061692	0.6	Ok	EYAD	0.061692	Ok
SAP	EQXAD	0.06319	0.6	Ok	EYAD	0.06319	Ok
ETABS	WXD	0.00652	0.06	Ok	WYD	0.00652	Ok

Note: the drift limits for seismic obtained by 0.02H (for wind H/400).

Table (4.5): Maximum Inter-storey Drift Ratio

story	Load comb	Max drift x	limits	Status	Load comb	Max drift y	Status
9TH	EXAD	0.001766	0.02	ok	EYAD	0.001766	Ok
8TH	EXAD	0.00201	0.02	ok	EYAD	0.00201	Ok
7TH	EXAD	0.00224	0.02	ok	EYAD	0.00224	Ok
6TH	EXAD	0.002317	0.02	ok	EYAD	0.002317	Ok
5TH	EXAD	0.002449	0.02	ok	EYAD	0.002449	Ok
4TH	EXAD	0.002506	0.02	ok	EYAD	0.002506	Ok
3RD	EXAD	0.002393	0.02	ok	EYAD	0.002393	Ok
2ND	EXAD	0.002209	0.02	ok	EYAD	0.002209	Ok
1ST	EXAD	0.001787	0.02	ok	EYAD	0.001787	Ok
GF	EXAD	0.000889	0.02	ok	EYAD	0.000889	Ok

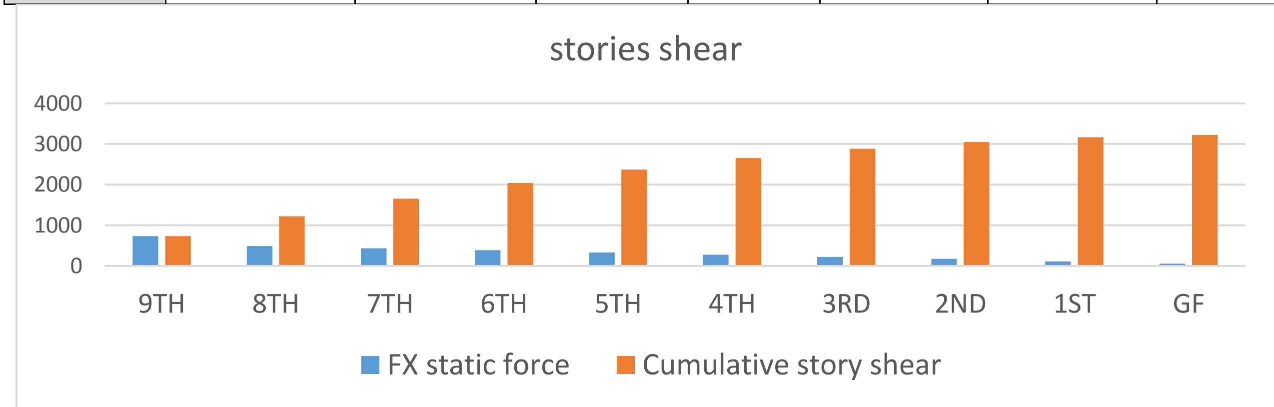


Figure (4.3): Stories static force & cumulative shear from seismic loads

(4) Final design for columns and shear walls

The final and optimized design will be done in the footprint level (reactions), the next details statements for design of the interior column labeled by (1) in ETABS which lies in the intersection of main grids B-5, to find the final cross sections of this column based on the case of maximum vertical load with reinforcement area ratio equal to one percent of concrete cross section, and the cases of maximum moment in each direction will be checked by using another program called CSI COLUMN specialized in columns and shear walls design. The below data will be used:

Load combinations mentioned in Table (3.9)

Column height L = 2.8m,

Effective height = β×L (when the slab depth will be less than column width or height β will be 1 at top and bottom) it's clearly this column is short

Final cross-section = $\frac{4935.41 \times 1000}{13.72} = 359723.76$ mm^2

The new width = $\sqrt{359723.76} = 599.76$ mm use 600 mm

$A_{sc} = 0.01 * 600^2 = 3600$ mm^2, use T20 (As =314 mm^2), No of Bars = $\frac{3600}{314} = 11.46$

use 12 T20 with $A_{sp} = 3768$ mm^2

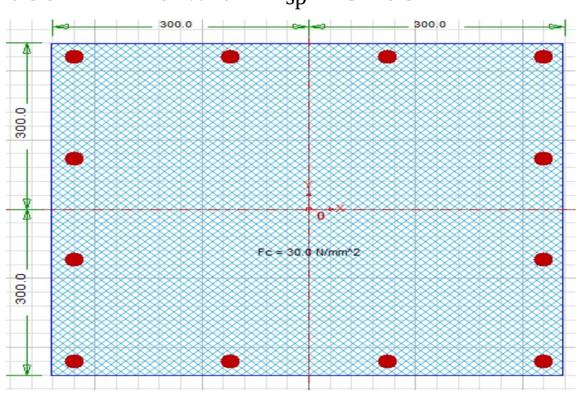


Figure (4.4): Column cross-section details

- Check for other combinations:

1- Case of Max M_x: (N = 2769.1, M_x = 144, M_Y= -28.6)

$h' = b' = 600 – 8 – 10 - 25 = 557$ mm

$M_x/h' = 144000/557 = 258.5$ $M_y/b' = 28600/557 = 48.8$

$M_x' = M_x + \beta \frac{h'}{b'} M_Y$, from BS 8110-97 table (3.22) with $\frac{N}{bhf_{cu}} = \frac{2769.1*10^3}{30*600^2-(3768)}$

= 0.26 and with interpolation β = 0.722

M_x'= 144 + 0.722×1×27.2 = 164.6 KN.M

$\frac{N}{bh} = \frac{2769.1*10^3}{600^2}$ = 7.69, $\frac{d}{h} = \frac{557}{600}$ = 0.93

Use BS 8110: part 3, design chart 30, where d/h = 0.95

100 A_{sc}/ bh = 1.05 = 3768 mm^2, $\frac{\text{Mux'}}{bh^2}$ = 4

$M_{ux} = 4×600×600^2 /10^6$ = 864 KN.M, then section satisfactory

2- Case of Max M_Y (N = 3753.4, M_x = 25.1, M_Y= -147.8)

$M_x/h' = 25100/557 = 45.1$ $M_y/b' = 147800/557 = 265.4$

$M_y' = M_y + \beta \frac{h'}{b'} M_x$, from BS 8110-97 table (3.22) with $\frac{N}{bhf_{cu}} = \frac{3753.4*10^3}{30*600^2-(3768)}$

= 0.35 and with interpolation β = 0.59

M_y'= 147.8+ 0.59×1×25.1 = 162.61KN.M

$\frac{N}{bh} = \frac{3753.4*10^3}{600^2}$ = 10.43 , $\frac{d}{h} = \frac{557}{600}$ = 0.93

Use BS 8110: part 3, design chart 30, where d/h = 0.95

100 A_{sc}/ bh = 1.05 = 3768 mm^2, $\frac{\text{My'}}{bh^2}$ = 2.85

$M_{uy} = 2.85×600×600^2 /10^6$ = 615 KN.M, then section satisfactory.

These results has been checked by CSI COLUMN, the capacity ratio for every combination and interactions diagrams introduced in Table (4.6) below.

Table (4.6): Capacity Results for column (1) obtained from CSI column

Load Comb	Load-N (kN)	Mx (kN-m)	My (kN-m)	Mxy (kN-m)	Mx-My Angle (Deg)	Load Vector	Capacity Vector	Capacity Ratio	N/A Angle (deg)	N/A Depth (mm)	Capacity Method	Remarks
ULT1	4933.5	-2.7	-2.7	3.8	225.0	N/A	N/A	0.77	136.2	693.2	4	OK
EYBD	2769.1	144.3	-28.6	147.1	348.8	N/A	N/A	0.53	15.6	431.5	4	OK
UEXAD	3753.4	25.1	-147.8	149.9	279.6	N/A	N/A	0.68	79.0	498.4	4	OK

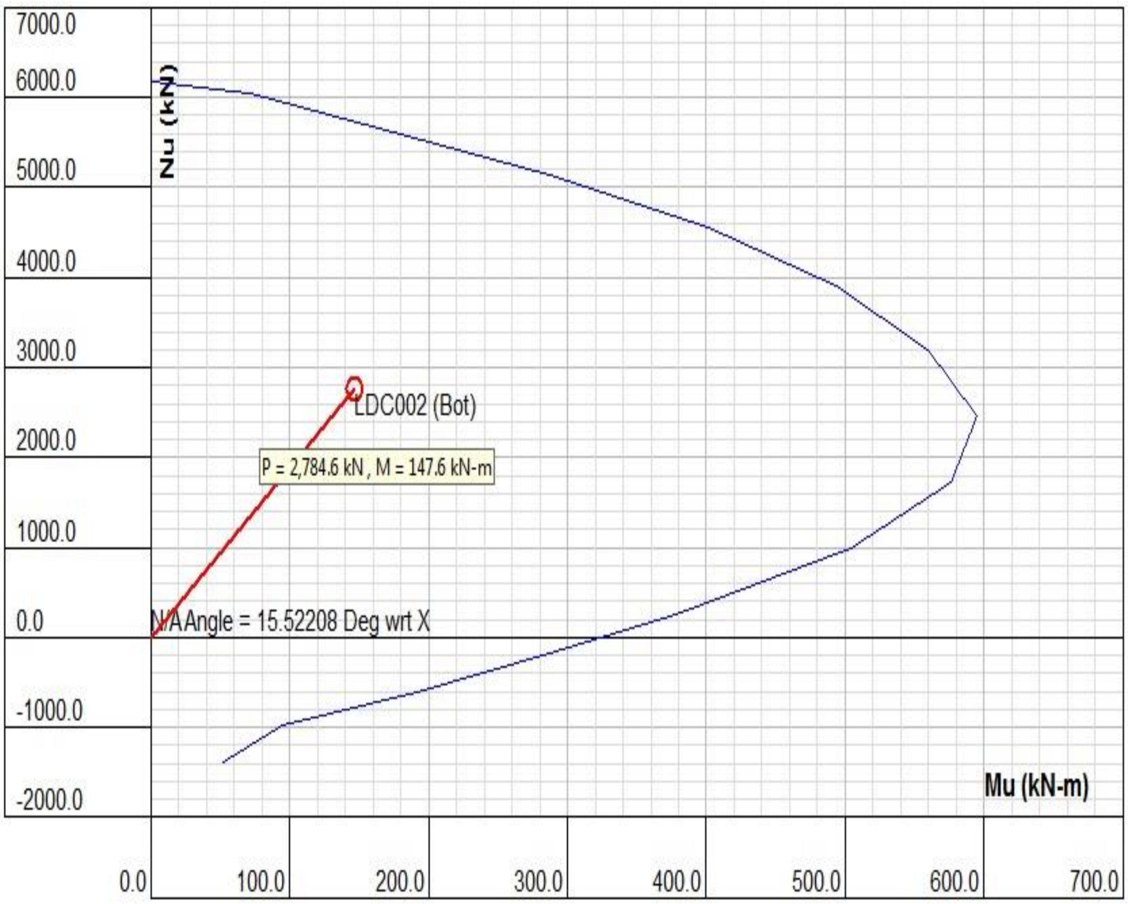


Figure (4.5): Interaction diagram for column (1) N and M_{xy} for EYBD comb.

By the same manner the columns design are tabulated in table (4.7).

Table (4.7): Details of Columns Design

Point	Load comb	FZ KN	Ac mm2	B_{req} mm	B_{ch} mm	Ac_{ch} mm2	$A_{s}c$ mm2	dia mm	BAR NO
1	ULT1	4933.5	359581.633	599.651	600	360000	3600	20	12
2	ULT1	4424.6	322490.525	567.882	600	360000	3600	20	12
3	ULT1	4424.6	322490.525	567.882	600	360000	3600	20	12
4	ULT1	4933.5	359581.633	599.651	600	360000	3600	20	12
5	ULT1	4933.5	359581.633	599.651	600	360000	3600	20	12
6	ULT1	4424.6	322490.525	567.882	600	360000	3600	20	12
7	ULT1	4424.6	322490.525	567.882	600	360000	3600	20	12
8	ULT1	4933.5	359581.633	599.651	600	360000	3600	20	12
9	ULT1	4424.6	322490.525	567.882	600	360000	3600	20	12
10	ULT1	4424.6	322490.525	567.882	600	360000	3600	20	12
11	ULT1	4424.6	322490.525	567.882	600	360000	3600	20	12
12	ULT1	4424.6	322490.525	567.882	600	360000	3600	20	12
13	ULT1	2055	149781.341	387.016	400	160000	1600	16	8
14	ULT1	1969.8	143567.784	378.903	400	160000	1600	16	8
15	ULT1	1969.8	143567.784	378.903	400	160000	1600	16	8
16	ULT1	2055	149781.341	387.016	400	160000	1600	16	8
17	ULT1	2055	149781.341	387.016	400	160000	1600	16	8
18	ULT1	1969.8	143567.784	378.903	400	160000	1600	16	8
19	ULT1	1969.8	143567.784	378.903	400	160000	1600	16	8
20	ULT1	2055	149781.341	387.016	400	160000	1600	16	8
21	ULT1	2055	149781.341	387.016	400	160000	1600	16	8
22	ULT1	1969.8	143567.784	378.903	400	160000	1600	16	8
23	ULT1	1969.8	143567.784	378.903	400	160000	1600	16	8
24	ULT1	2055	149781.341	387.016	400	160000	1600	16	8
25	ULT1	927.85	67627.551	260.053	300	90000	900	16	6
26	ULT1	927.85	67627.551	260.053	300	90000	900	16	6
27	ULT1	927.85	67627.551	260.053	300	90000	900	16	6
28	ULT1	2055	149781.341	387.016	400	160000	1600	16	8
30	ULT1	1969.8	143567.784	378.903	400	160000	1600	16	8
31	ULT1	927.85	67627.551	260.053	300	90000	900	16	6
32	ULT1	2055	149781.341	387.016	400	160000	1600	16	8
33	ULT1	1969.8	143567.784	378.903	400	160000	1600	16	8

The design of shear walls (walls) by assume the area of reinforcement equal 1% of area of concrete and check for loads combinations conducted by CSI COLUMN

SW1 Shear walls height is 2.8 m the walls classified as braced walls

The effective height = $\beta \times L = 1 \times 2.8 = 2.8/0.2 = 14 < 15$ wall is stocky

$A_C = 800{,}000\ mm^2$ $A_{SC} = 0.01 \times 800{,}000 = 8{,}000\ mm^2$

$N_u = 0.35 \times 30 \times (800000 - 8000) + 0.7 \times 460 \times 8000 = 10892000/1000 = 10{,}892$ KN

$N_u > 7979$ OK

The section has been checked for the other combinations and the results is below

Load Comb	Load-N (kN)	Mx (kN-m)	My (kN-m)	Mxy (kN-m)	Mx-My Angle (Deg)	Load Vector	Capacity Vector	Capacity Ratio	N/A Angle (deg)	N/A Depth (mm)	Capacity Method	Remarks
UEYBD	7979.5	1862.0	108.0	1865.1	3.3	N/A	N/A	0.70	30.6	1086.4	4	OK
EYBD	6769.2	1865.7	104.1	1868.6	3.2	N/A	N/A	0.61	27.1	921.0	4	OK
UEXAD	492.6	83.1	1886.3	1888.1	87.5	N/A	N/A	0.70	236.4	686.2	4	OK

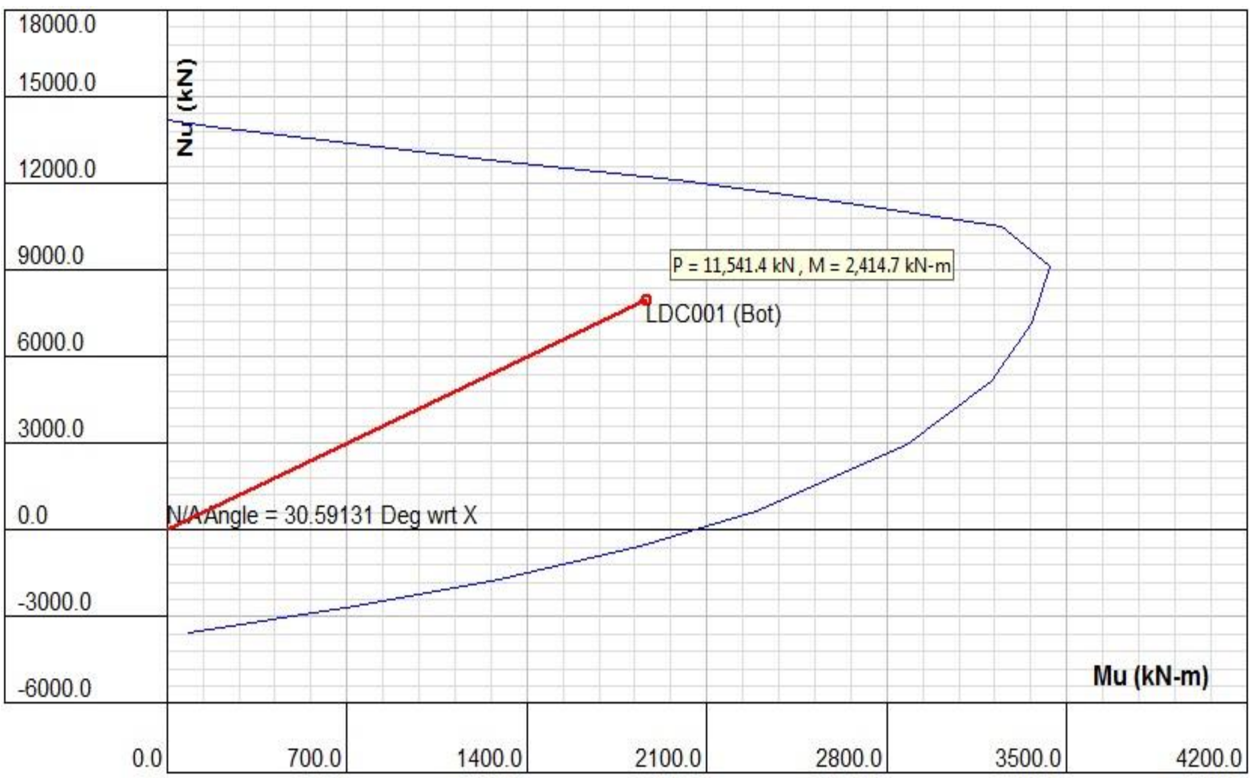


Figure (4.6): Show the interaction diagram for worst case (UEYBD) SW1

The Check of capacity ratio for SW2, SW3 & SW4 has been performed by the same criteria, the capacity ratios was found satisfactory.

Table (4.8): (a) Capacity Ratio Results for SW2

Load Comb	Load-N (kN)	Mx (kN-m)	My (kN-m)	Mxy (kN-m)	Mx-My Angle (Deg)	Load Vector	Capacity Vector	Capacity Ratio	N/A Angle (deg)	N/A Depth (mm)	Capacity Method	Remarks
UEYBD	7979.5	1720.0	35.2	1720.4	1.2	N/A	N/A	0.70	328.2	1093.3	4	OK
EYAD	6769.2	1865.7	-105.2	1868.7	356.8	N/A	N/A	0.61	333.5	911.8	4	OK
EXAD	6769.2	105.2	1865.7	1868.7	86.8	N/A	N/A	0.62	300.5	975.4	4	OK

Table (4.8): (b) Capacity Ratio Results for SW3

Load Comb	Load-N (kN)	Mx (kN-m)	My (kN-m)	Mxy (kN-m)	Mx-My Angle (Deg)	Load Vector	Capacity Vector	Capacity Ratio	N/A Angle (deg)	N/A Depth (mm)	Capacity Method	Remarks
UEYBD	7979.5	1720.0	35.2	1720.4	1.2	N/A	N/A	0.84	17.8	1780.6	4	OK
EYAD	6769.2	1865.7	-105.2	1868.7	356.8	N/A	N/A	0.79	24.7	1499.4	4	OK
EXAD	6769.2	105.2	1865.7	1868.7	86.8	N/A	N/A	0.61	243.5	911.8	4	OK

Table (4.8): (c) Capacity Ratio Results for SW4

Load Comb	Load-N (kN)	Mx (kN-m)	My (kN-m)	Mxy (kN-m)	Mx-My Angle (Deg)	Load Vector	Capacity Vector	Capacity Ratio	N/A Angle (deg)	N/A Depth (mm)	Capacity Method	Remarks
ULT1	5316.5	15.8	15.8	22.3	45.0	N/A	N/A	0.38	315.0	826.1	4	OK
UEYBD	492.6	1886.3	-83.7	1888.2	357.5	N/A	N/A	0.57	326.4	686.2	4	OK
UEXBD	492.6	83.7	1886.3	1888.2	87.5	N/A	N/A	0.69	305.1	651.2	4	OK

It can be observed that the cross section of shear wall is more safe and the maximum capacity ratio for the worst case is 84%.

4.2.2 Model (B) Analysis Results & design

(1) Analysis results; the maximum drift in each directions shown in table (4.9) with the limits value, and the max inter-story drift for every story shown in table (4.10).

Table (4.9): Maximum Lateral Building Drift

Prog-gram	Load comb	Max drift x(m)	drift limits	status	load comb	max drift y(m)	Status
ETABS	EXAD	0.085243	0.6	Ok	EYAD	0.085243	Ok
SAP	EXAD	0.08649	0.6	Ok	EYAD	0.08649	Ok

Table (4.10): Maximum inter-storey Drift Ratio

story	Load comb	Max drift x	limits	Status	Load comb	Max drift y	Status
9TH	EXAD	0.00274	0.02	Ok	EYAD	0.00274	Ok
8TH	EXAD	0.002975	0.02	Ok	EYAD	0.002975	Ok
7TH	EXAD	0.003209	0.02	Ok	EYAD	0.003209	Ok
6TH	EXAD	0.003318	0.02	Ok	EYAD	0.003318	Ok
5TH	EXAD	0.003446	0.02	Ok	EYAD	0.003446	Ok
4TH	EXAD	0.003456	0.02	Ok	EYAD	0.003456	Ok
3RD	EXAD	0.003247	0.02	Ok	EYAD	0.003247	Ok
2ND	EXAD	0.002875	0.02	Ok	EYAD	0.002875	Ok
1ST	EXAD	0.002173	0.02	Ok	EYAD	0.002173	Ok
GF	EXAD	0.000976	0.02	Ok	EYAD	0.000976	Ok

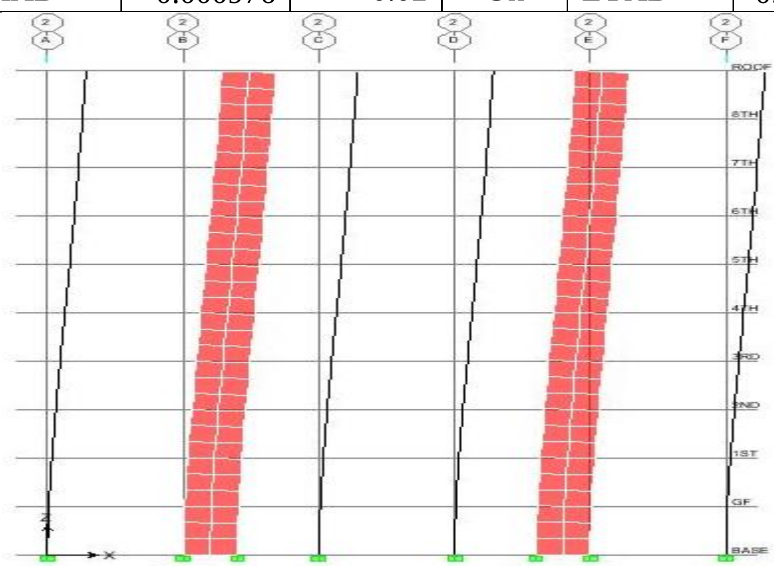


Figure (4.7) Show Deflected Shape (elevation) due to Seismic Load

Table (4.11): Shear Walls Forces for Model (B)

Wall No	Load comb	P KN	M_x KN.M	M_y KN.M
SW1	ULT1	-7753	12.059	12.059
SW1	EYBD	-4874.87	2424.3	126.071
SW1	UEXAD	-5554	115.07	2435.3
SW2	ULT1	-7753	12.059	12.059
SW2	EYAD	-4875	2424.3	126.07
SW2	EXAD	-4875	126.07	2424.3
SW3	ULT1	-7753	12.059	12.059
SW3	UEYAD	-5554	2435.3	115.07
SW3	EXBD	-4875	126.07	2424.3
SW4	ULT1	-7753	12.059	12.059
SW4	UEYBD	-5554	2435.3	115.07
SW4	UEXBD	-5554	-115.1	2435.3

Note: tabulated columns forces stated in appendix

(2) Final design for columns & shear walls

By the same manner followed in model (A) the designed columns are shown in table (4.12).

Table (4.12): Final Design of Columns

Point Label	Load comb	FZ KN	Ac mm2	B_{req} mm	B_{ch} mm	Ac_{ch} mm2	A_{sc} mm2	dia mm	Bar NO
2	ULT1	3775.39	275174	524.57	550	302500	3025	20	10
3	ULT1	3775.39	275174	524.57	550	302500	3025	20	10
6	ULT1	3775.39	275174	524.57	550	302500	3025	20	10
7	ULT1	3775.39	275174	524.57	550	302500	3025	20	10
9	ULT1	3775.39	275174	524.57	550	302500	3025	20	10
10	ULT1	3775.39	275174	524.57	550	302500	3025	20	10
11	ULT1	3775.39	275174	524.57	550	302500	3025	20	10
12	ULT1	3775.39	275174	524.57	550	302500	3025	20	10
13	ULT1	1903.06	138707	372.434	400	160000	1600	16	8
14	ULT1	1888.35	137635	370.992	400	160000	1600	16	8
15	ULT1	1888.35	137635	370.992	400	160000	1600	16	8

Table (4.12): Continued

Point Label	Load comb	FZ KN	Ac mm2	B_{req} mm	B_{ch} mm	Ac_{ch} mm2	A_{sc} mm2	dia mm	BAR NO
16	ULT1	1903.06	138707	372.434	400	160000	1600	16	8
17	ULT1	1903.06	138707	372.434	400	160000	1600	16	8
18	ULT1	1888.35	137635	370.992	400	160000	1600	16	8
19	ULT1	1888.35	137635	370.992	400	160000	1600	16	8
20	ULT1	1903.06	138707	372.434	400	160000	1600	16	8
21	ULT1	1903.06	138707	372.434	400	160000	1600	16	8
22	ULT1	1888.35	137635	370.992	400	160000	1600	16	8
23	ULT1	1888.35	137635	370.992	400	160000	1600	16	8
24	ULT1	1903.06	138707	372.434	400	160000	1600	16	8
25	ULT1	964.17	70274.8	265.094	300	90000	900	16	6
26	ULT1	964.17	70274.8	265.094	300	90000	900	16	6
27	ULT1	964.17	70274.8	265.094	300	90000	900	16	6
28	ULT1	1903.06	138707	372.434	400	160000	1600	16	8
30	ULT1	1888.35	137635	370.992	400	160000	1600	16	8
31	ULT1	964.17	70274.8	265.094	400	160000	1600	16	6
32	ULT1	1903.06	138707	372.434	400	160000	1600	16	8
33	ULT1	1888.35	137635	370.992	400	160000	1600	16	8
35	ULT1	4225.95	308014	554.99	600	360000	3600	20	12
38	ULT1	4225.95	308014	554.99	600	360000	3600	20	12
41	ULT1	4225.95	308014	554.99	600	360000	3600	20	12
44	ULT1	4225.95	308014	554.99	600	360000	3600	20	12

For shear walls it's clearly that the existing cross section with 1% reinforcement ratio is satisfactory to withstand the all combinations, and the check for wall 1 show the results below

Table (4.13): Capacity Ratio for SW1.

Load Comb	Load-N (kN)	Mx (kN-m)	My (kN-m)	Mxy (kN-m)	Mx-My Angle (Deg)	Load Vector	Capacity Vector	Capacity Ratio	N/A Angle (deg)	N/A Depth (mm)	Capacity Method	Remarks
ULT1	7753.0	12.1	12.1	17.1	45.0	N/A	N/A	0.55	278.7	2349.7	4	OK
EYBD	4874.9	2424.3	126.1	2427.6	3.0	N/A	N/A	0.68	22.2	689.7	4	OK
UEXAD	5554.0	115.1	2435.3	2438.0	87.3	N/A	N/A	0.83	245.5	1400.6	4	OK

4.2.3 Model (C) Analysis Results & design

The maximum drift in each directions shown in Table (4.14) with the limits value, and the max inter-story drift for every story shown in Table (4.15)

Table (4.14): Maximum Lateral Building Drift

Prog-gram	Load comb	Max Drift x(m)	drift limits	status	load comb	max drift y(m)	status
ETABS	EXAD	0.110595	0.6	Ok	EYAD	0.0110595	Ok
ETABS	WXD	0.013613	0.075	Ok	WYD	0.013613	Ok

Table (4.15): Inter-storey Drift Ratio

story	Load comb	Max drift x	Drift limits	Status	Load comb	Max drift y	Status
9TH	EXAD	0.004067	0.02	Ok	EYAD	0.004067	Ok
8TH	EXAD	0.004258	0.02	Ok	EYAD	0.004258	Ok
7TH	EXAD	0.004428	0.02	Ok	EYAD	0.004428	Ok
6TH	EXAD	0.004415	0.02	Ok	EYAD	0.004415	Ok
5TH	EXAD	0.004446	0.02	Ok	EYAD	0.004446	Ok
4TH	EXAD	0.004332	0.02	Ok	EYAD	0.004332	Ok
3RD	EXAD	0.003955	0.02	Ok	EYAD	0.003955	Ok
2ND	EXAD	0.003405	0.02	Ok	EYAD	0.003405	Ok
1ST	EXAD	0.002492	0.02	Ok	EYAD	0.002492	Ok
GF	EXAD	0.001067	0.02	Ok	EYAD	0.001067	Ok

Table (4.16): Shear Walls Forces for Model (C)

Wall No	Load comb	P KN	M_x KN.M	M_y KN.M
SW1	ULT1	-2998	2728.2	115.3
SW1	UEYBD	-2998	2728.2	115.3
SW1	EXAD	-1035	122.05	2704.2
SW2	ULT1	-2998	2728.2	115.3
SW2	UEYAD	-2998	2728.2	115.3

Table (4.16): continued

Wall No	Load comb	P KN	M_x KN.M	M_y KN.M
SW2	UEXAD	-2998	115.3	2728.2
SW3	ULT1	-2998	115.3	2728.2
SW3	EYAD	-1035	2704.2	122.05
SW3	UEXBD	-2998	115.3	2728.2
SW4	ULT1	-2843	18.437	18.388
SW4	EYBD	-1035	2704.2	122.05
SW4	EXBD	-1035	-122.1	2704.2

Moreover, the detailed designed columns for model (c) shown in Table (4.17)

Table (4.17): Final Design of Columns for Model (c)

Point Label	Load comb	FZ KN	Ac mm2	B_{req} mm	B_{ch} mm	Ac_{ch} mm2	A_{sc} mm2	dia mm	Bar NO
1	ULT1	4638.5	338085	581.5	600	360000	3600	20	12
2	ULT1	4690.6	341881	584.7	600	360000	3600	20	12
3	ULT1	4690.6	341881	584.7	600	360000	3600	20	12
4	ULT1	4638.5	338085	581.5	600	360000	3600	20	12
5	ULT1	4638.5	338085	581.5	600	360000	3600	20	12
6	ULT1	4690.6	341881	584.7	600	360000	3600	20	12
7	ULT1	4690.6	341881	584.7	600	360000	3600	20	12
8	ULT1	4638.5	338085	581.5	600	360000	3600	20	12
9	ULT1	4690.6	341881	584.7	600	360000	3600	20	12
10	ULT1	4690.6	341881	584.7	600	360000	3600	20	12
11	ULT1	4690.6	341881	584.7	600	360000	3600	20	12
12	ULT1	4690.6	341881	584.7	600	360000	3600	20	12
13	ULT1	1516.7	110545	332.5	350	122500	1225	16	8
14	ULT1	1969.1	143523	378.8	400	160000	1600	16	8
15	ULT1	1969.1	143523	378.8	400	160000	1600	16	8
16	ULT1	1516.7	110545	332.5	350	122500	1225	16	8
17	ULT1	1516.7	110545	332.5	350	122500	1225	16	8
18	ULT1	1969.1	143523	378.8	400	160000	1600	16	8

Table (3.17) continued.

Point Label	Load comb	FZ KN	Ac mm2	B_{req} mm	B_{ch} mm	Ac_{ch} mm2	A_{sc} mm2	dia mm	Bar NO
19	ULT1	1969.1	143523	378.8	400	160000	1600	16	8
20	ULT1	1516.7	110545	332.5	350	122500	1225	16	8
21	ULT1	1516.7	110545	332.5	350	122500	1225	16	8
22	ULT1	1969.1	143523	378.8	400	160000	1600	16	8
23	ULT1	1969.1	143523	378.8	400	160000	1600	16	8
24	ULT1	1516.7	110545	332.5	350	122500	1225	16	8
28	ULT1	1516.7	110545	332.5	350	122500	1225	16	8
30	ULT1	1969.1	143523	378.8	400	160000	1600	16	8
32	ULT1	1516.7	110545	332.5	350	122500	1225	16	8
33	ULT1	1969.1	143523	378.8	400	160000	1600	16	8
35	ULT1	4385.7	319658	565.4	600	360000	3600	20	12
38	ULT1	4385.7	319658	565.4	600	360000	3600	20	12
41	ULT1	4385.7	319658	565.4	600	360000	3600	20	12
44	ULT1	4385.7	319658	565.4	600	360000	3600	20	12

A simple check for capacity of SW1 has been done, the result are shown below

Table (4.18): Capacity Ratio for SW1(c)

Load Comb	Load-N (kN)	Mx (kN-m)	My (kN-m)	Mxy (kN-m)	Mx-My Angle (Deg)	Load Vector	Capacity Vector	Capacity Ratio	N/A Angle (deg)	N/A Depth (mm)	Capacity Method	Remarks
ULT1	2842.6	18.4	18.4	26.0	44.9	N/A	N/A	0.20	88.3	271.0	4	OK
UEYBD	2997.6	2728.2	115.3	2730.6	2.4	N/A	N/A	0.86	15.4	470.6	4	OK
EXAD	1034.7	122.1	2704.2	2707.0	87.4	N/A	N/A	0.89	236.6	758.1	4	OK

4.3 The rigid frame results

(1) Drifts: The maximum building drift in each directions shown in table (4.19)

Table (4.19): Maximum Building Drift

Model	Drift (m)			
	EXAD	EYAD	WXD	WYD
D	0.081891	0.081891	0.009731	0.009731
E	0.084687	0.08247	0.010604	0.010047
F	0.079307	0.07783	0.010045	0.00967

Moreover, the static equivalent force in each story with cumulative story shear shown below

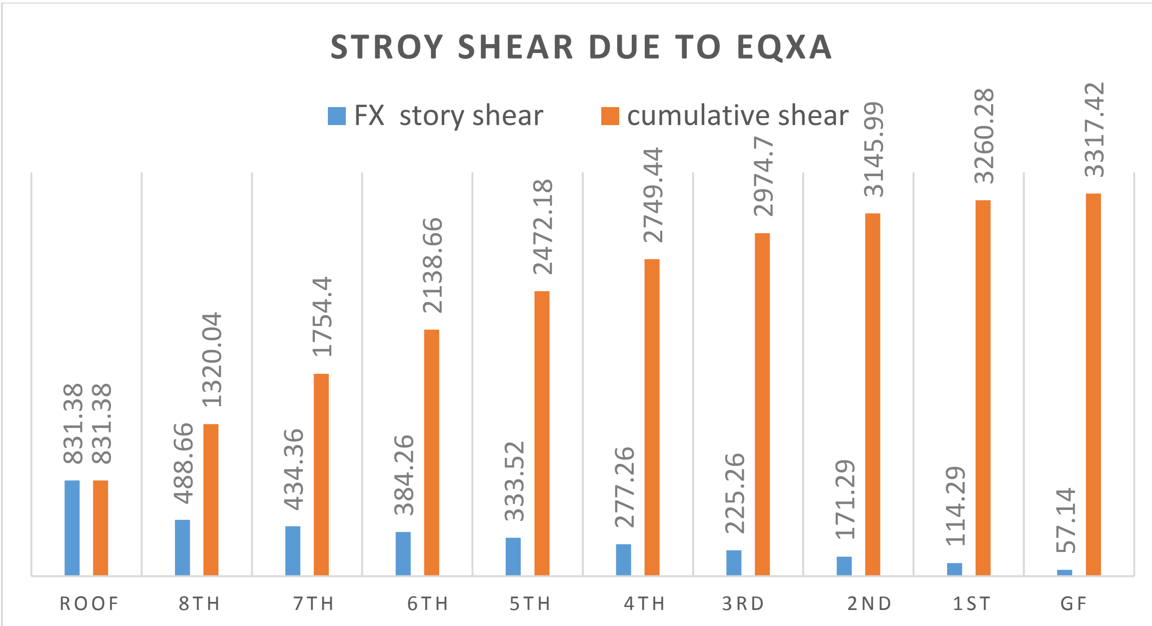


Figure (4.8): Static Equivalent force & cumulative shear

Due to previous knowledge that the beam has a major contribution in joints rotation (frame racking) and the total drift in the building, Model (F) has been selected to remodeled with beam depth increased to 0.5 m and table (4.20) show the drifts.

Table (4.20) Drifts of Model (F) with 0.5 Beam Depth

Prog-gram	Load comb	Max Drift x(m)	drift limits	status	load comb	max drift y(m)	status
ETABS	UEXBD	0.058802	0.6	Ok	EYAD	0.05833	Ok
ETABS	WXD	0.007373	0.075	Ok	WYD	0.006976	Ok

Table (4.21): Maximum inter-storey Drifts

story	Load comb	Max drift x	limits	Status	Load comb	Max drift y	Status
9TH	UEXBD	0.0011	0.02	Ok	UEYAD	0.001	Ok
8TH	UEXBD	0.0017	0.02	Ok	EYBD	0.0016	Ok

Table (4.21): continued

story	Load comb	Max drift x	limits	Status	Load comb	Max drift y	Status
7TH	UEXBD	0.0022	0.02	Ok	EYBD	0.0021	Ok
6TH	UEXBD	0.002	0.02	Ok	EYBD	0.002	Ok
5TH	UEXBD	0.0022	0.02	Ok	EYBD	0.0022	Ok
4TH	UEXBD	0.0024	0.02	Ok	EYBD	0.0024	Ok
3RD	UEXBD	0.0023	0.02	Ok	EYBD	0.0023	Ok
2ND	UEXBD	0.0023	0.02	Ok	EYBD	0.0024	Ok
1ST	UEXBD	0.0021	0.02	Ok	EYBD	0.0022	Ok
GF	UEXBD	0.0012	0.02	Ok	EYBD	0.0012	Ok

4.4 The outriggers analysis results

(a) For regular models; the acting of lateral force on the building for model with outrigger at 0.5H was shown in table (4.22).

Table (4.22): lateral force in the building (KN)

Story NO	F_X Wind	F_X (static)	S_X (dynamic)	Story NO	F_X Wind	F_X (static)	S_X (dynamic)
29	76.41	1563.92	984.3	14	152.82	246.84	220
28	152.82	418.47	825.7	13	152.82	223.81	179
27	152.82	404.04	634	12	152.82	207.82	138
26	152.82	389.61	465	11	152.82	191.83	109
25	152.82	375.18	334	10	152.82	175.85	96
24	152.82	368.21	254	9	152.82	161.03	109
23	152.82	361.15	222	8	152.82	146.01	144
22	152.82	346.1	216	7	152.82	129.79	195

Table (4.22): continued

Story NO	F_X Wind	F_X (static)	S_X (dynamic)	Story NO	F_X Wind	F_X (static)	S_X (dynamic)
21	152.82	331.06	226	6	152.82	113.57	246
20	152.82	316.01	231	5	152.82	97.34	278
19	152.82	302.42	230	4	152.82	80.61	283
18	152.82	288.7	223	3	152.82	64.96	254
17	152.82	273.51	263	2	152.82	49.36	192
16	152.82	258.31	282	1	152.82	32.91	114
15	152.82	285.67	256	GF	152.82	16.45	42

Table (4.23): Building drift for regular models (m)

Model	W_XD	W_YD	E_XD	E_YD	S_XD	S_YD
without outrigger	0.143953	0.145311	0.292622	0.297011	0.196492	0.203358
outrigger at 13	0.086933	0.08662	0.183024	0.183823	0.145535	0.14718
outrigger at 14	0.087303	0.087015	0.183198	0.182503	0.148	0.143537
outrigger at 15	0.087449	0.087293	0.180162	0.180982	0.148454	0.14307
outrigger at 17	0.089072	0.089215	0.17964	0.18077	0.14514	0.141488
two outrigger	0.062887	0.062136	0.132365	0.13181	0.112302	0.103132

Notice: the static combination (E_XD& E_YD) shown for comparison only

(b) The building drift for irregular models: due to huge mass located on the top was shown below:

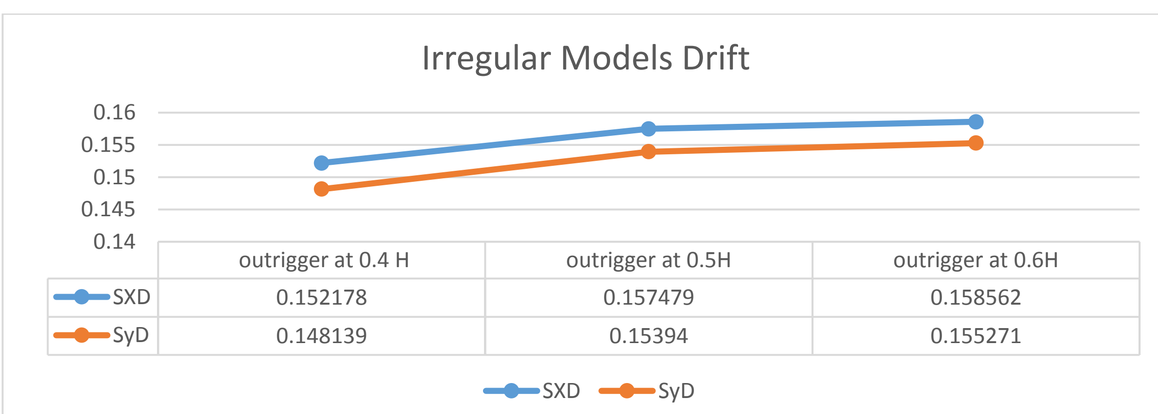


Figure (4.9): Irregularity due to mass difference models drift

(a) The deflected shape of a setback model was shown in figure (4.10)

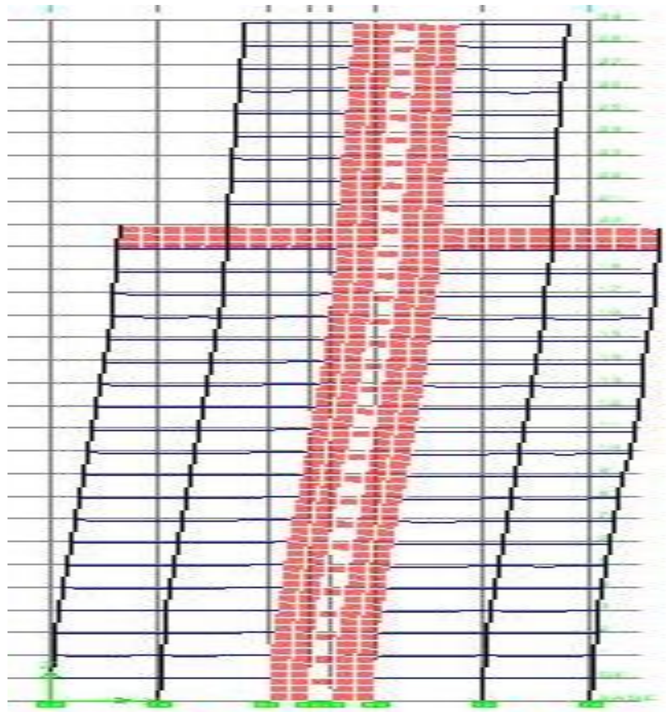


Figure (4.10): Deflected shape due to wind

Table (4.24): The Setback Models Drift (m)

Setback model	S_XD	S_YD	W_XD	W_YD
outrigger at 0.5 H	0.110595	0.109284	0.072485	0.071222
outrigger at 0.6H	0.107173	0.105716	0.073564	0.07271
outrigger at 0.7H	0.108661	0.107838	0.077129	0.076742

4.5 Results comparison.

The comparison between the models would be introduced interim of figures and tables, the variables which were increase or decrease such as drifts, forces and plan density index. The shear walls results would be compared with each other's to find the optimum position, and the model which would gave the optimum position would be selected to compare with the selected rigid frame. The results of outrigger systems would discussed in three level, first; to evaluate the efficiency of the outrigger, second; optimum position for irregular due to mass, third; the optimum position due to setbacks.

4.5.1 Shear walls comparison

The Tables and figures introduce the comparison between the shear walls model for observed, Table (4.25) shows the forces in SW1 obtained from three models.

Table (4.25): Forces in SW1 for all models.

Model	wall No	Load Comb.	P KN	M_x KN.M	M_y KN.M
A	SW1	UEYBD	-7980	1862	108
	SW1	EYBD	-6769	1865.7	104.1
	SW1	UEXAD	-492.6	83.1	1886.3
B	SW1	ULT1	-7753	12.059	12.059
	SW1	EYBD	-4875	2424.3	126.071
	SW1	UEXAD	-5554	115.07	2435.3
C	SW1	UEYBD	-2998	2728.2	115.299
	SW1	UEYBD	-2998	2728.2	115.299
	SW1	EXAD	-1035	122.05	2704.21

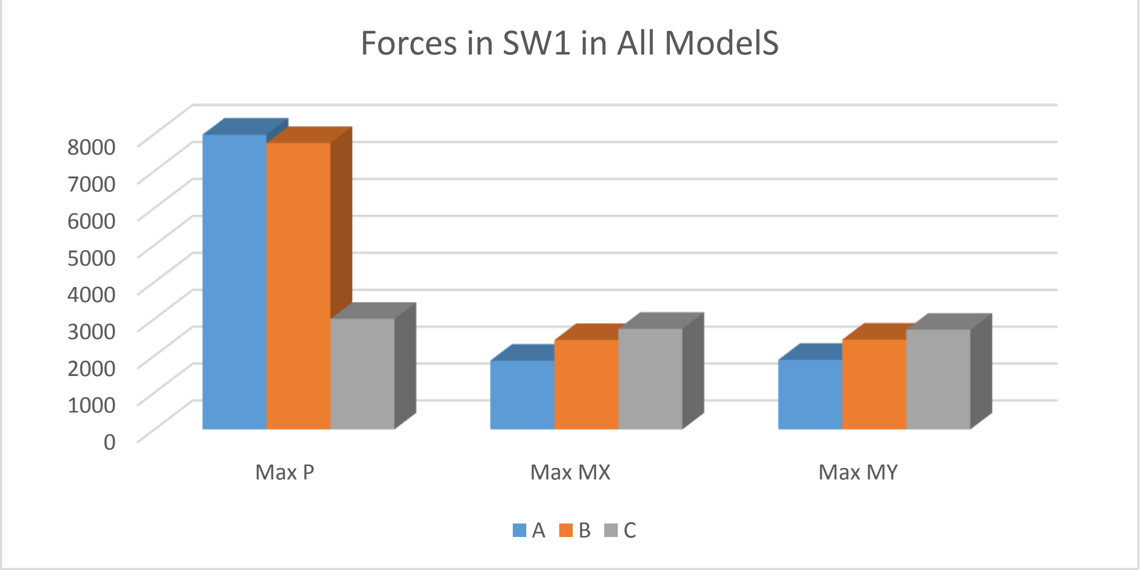


Figure (4.11): Results Comparison between the Forces in SW1.

Table (4.26): The Forces as percentage of model (A)

Model	wall	Load	P	M_x	M_y
B	SW1	ULT1	97.2%		
	SW1	EYBD	72.0%	129.9%	126.07
	SW1	UEXAD	1127.5%		129.1%
C	SW1	UEYBD	37.6%		
	SW1	UEYBD	44.3%	146.2%	
	SW1	EXAD	210.1%		143.4%

Moreover, Table (4.27) shows the forces for the edge column which is labeled as (13) and interior column is labeled as (12).

Table (4.27): The Forces in columns.

Column	Force	Model		
		A	B	C
Interior (12)	P	4443	3775	4690
	M_X	145	176	179.3
	M_Y	-131.9	-158.8	-177.8
Edge (13)	P	2055	1917	1743
	M_X	55.5	62.6	63.22
	M_Y	-49.3	-51.5	-55.31

For drift and drift ratio comparisons. Figure (4.12) showed the building drift due to seismic loads for all models

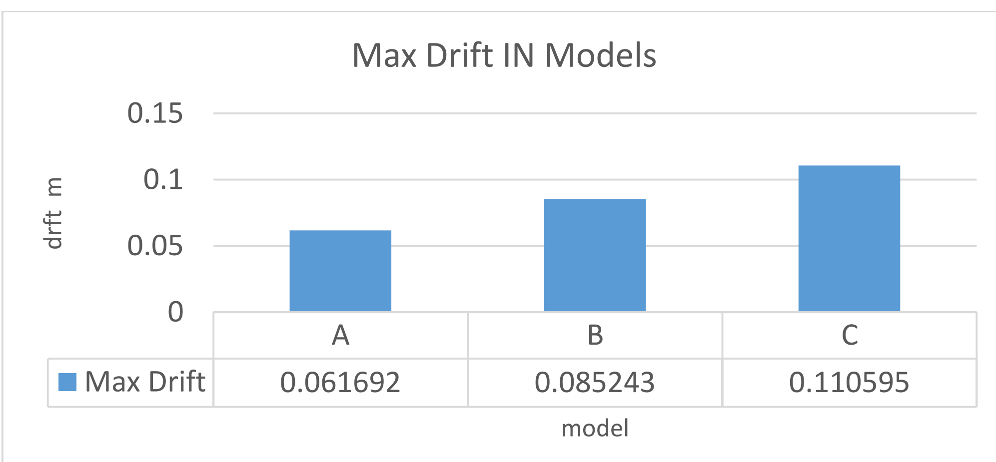


Figure (4.12): Models Drift: A, B & C.

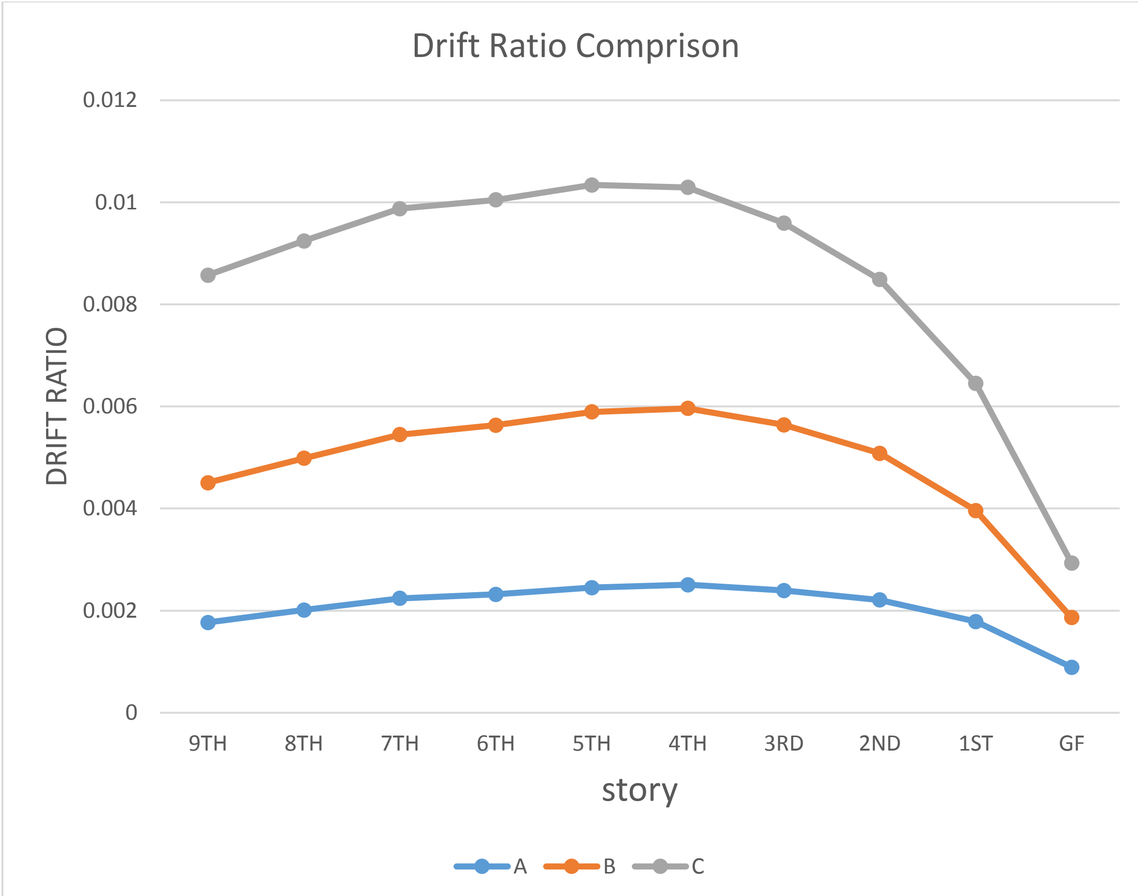


Figure (4.13): Comparison of drift Ratios.

Also table (4.28) showed the plan density index (PDI) - prepared by using the cross sections as obtained from columns design for each model- in the footprint for all models.

Table (4.28): The Plan Density index.

Model	Elements area (m)	Plan Area(m)	PDI
A	10.44	625	1.67%
B	10.05	625	1.61%
C	11.22	625	1.80%

4.5.1.1 The final observations

From the figures and tables mentioned above it could be observed the following:

1- The position of shear walls in lateral loads combinations (dead+ seismic) in model (A) make it gathering a minimum overturning moments (M_x =1865.7& M_y=1886.3), where in model (B) the M_x is increased about 30% and M_y increased by 29.1%, and in model (c) the M_x is increased by 46% and M_y increased by 43.5%, and generally the axial loads was maximum in (A) and tend to decrease in the other models.

2- It should be noted that the columns has small contributions in resistance of lateral forces according to their lateral stiffness (is more realistic than the other assumptions), then could observe from table (4.27) for interior column (12) the minimum moments carried by column are M_x=145 & M_y 131.9 KN.M in model (A), where in model (B) are increased by 21% & 20% respectively, and in model (C) are increased by 23.6% and 35% respectively. For the edge column (13) the moments also yielded in model (A), where are increased in model (B) by 13% & 10.5% M_x and M_y respectively and increased by 14% and 12% in model (C).

3- Figure (4.12) showed clearly that the minimum building drift are obtained in model (A) and increase linearly with shifting the shear walls far away from the center of mass, and the drift increased by 39.6% and 79.2% in model(B) and (C) respectively.

4- From the observed structural cases It could be drawn that the optimum position of shear walls is near to center of mass, because it would yield minimum overturning moments on vertical elements (shear walls, columns), and less lateral drift, while the minimum plan density index was obtained from model (B), but the difference in very small (0.06%) and it could be sacrificed.

5- Model (A) will be selected for the comparison with the rigid frame results.

4.5.2 The Rigid Frame Results comparisons

For comparison purposes figure (4.14) showed the models drifts for seismic loads

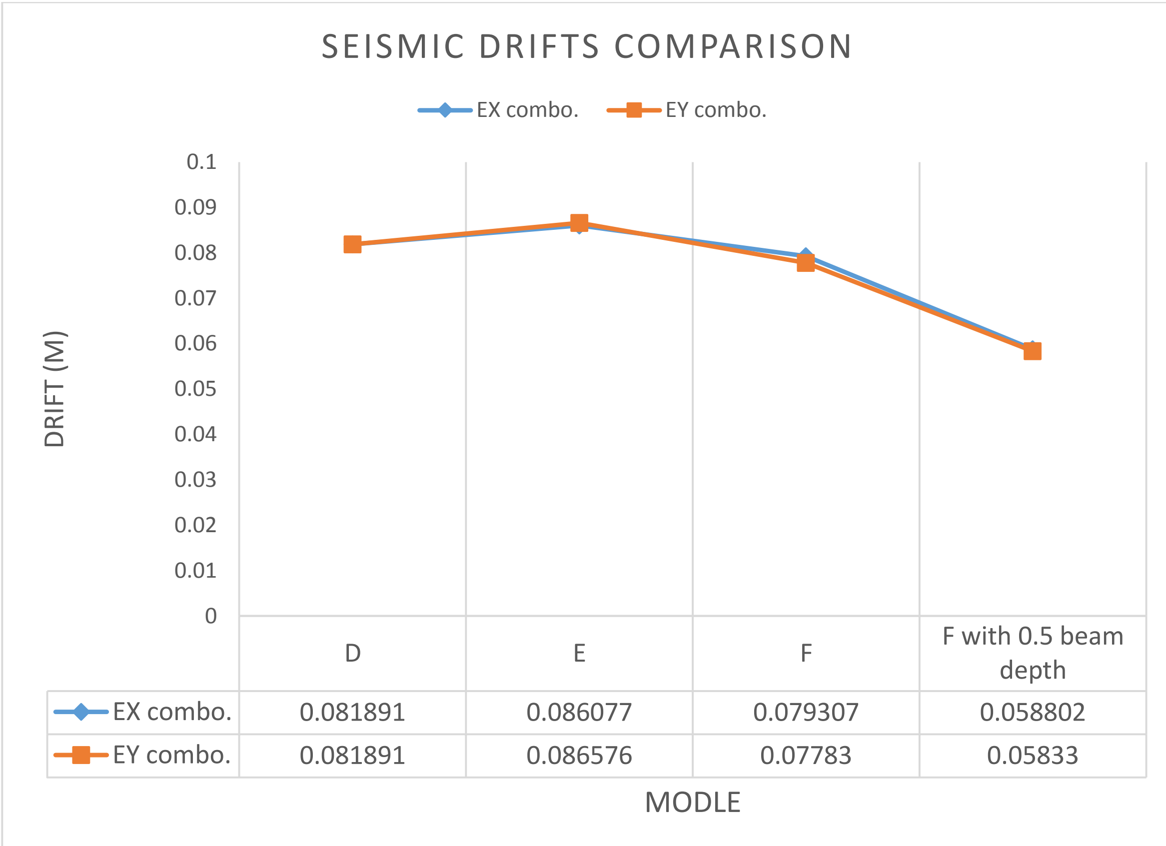


Figure (4.14): Comparison between Drifts.

From the previous figure the following points could be observed:

1. The rectangular columns which has a large stiffness - when compared with square- could produce higher drift than the square columns if the applicable layout did not followed as in model (E) when compared with (D).
2. When the beams depth increased by 0.1 m the drift was reduced by 26%.
3. Model (F) with 0.5 beams depth has been selected for comparison with the shear walls model.

4.5.3 Shear Walls vs Rigid Frame

In the following figures the comparison between would be introduced the shear walls and rigid frame with some parameters such as time period, drift and the plan density index.

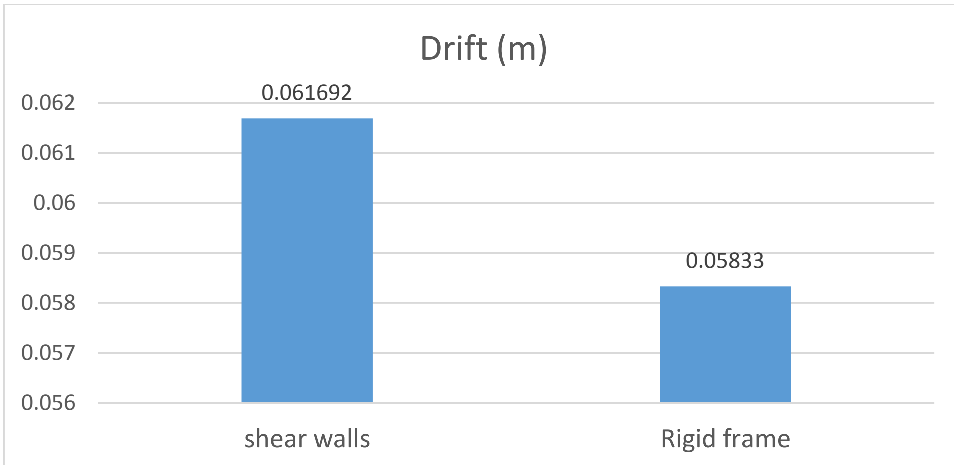


Figure (4.15): Comparison between shear wall and rigid frame drift.

Table (4.29) The Plan density index and buildings periods

Model	T_a	T_{used}	PDI
Shear walls	0.625	0.875	1.67%
Rigid frame	0.9375	1.3124	1.66%

From the previous figures and tables the following points could be observed:

1- The code (UBC 97) provision consider the shear walls system having more rigid than the rigid frame, because the time period used (T_{used}) in shear walls is less than 1 second, while in the rigid frame it is greater than 1 sec, and then in the case of rigid frame the model will be classified as a flexible structure.

2- The rigid frame with applicable layout produce a minimum lateral drift while in the case of the shear walls the drift was increased by 6 %, where the difference in (PDI) is very small and could be neglected.

3- The rigid frame was categorized as ordinary moment frame with over strength factor R equal to 3.5, with special detailing, could be categorized as a special moment resistant frame (SMRF) with R equal to 8.5, which mean the system will examine less forces (about two third less than the first situation).

4.5.4 The Outrigger Results comparisons

(1) Evaluation of the outrigger efficiency:

Table (4.30) showed the drift of each model as percentage of drift without the outrigger.

Table (4.30): Drift as a percentage of the without outrigger case

Outrigger location	story	WXD	WYD	SXD	SYD
0.467H	13	59.0%	58%	73%	72%
0.5H	14	59.2%	58.5%	74.7%	70.0%
0.53H	15	59.3%	58.6%	75.0%	69.8%
0.6H	17	60.4%	59.9%	73.3%	69.0%
two outriggers	9/19	42.7%	41.7%	56.7%	50.3%

From the previous table (4.30) the following points could be observed:

1- The loads combinations which produce the maximum drift has been selected for the comparison purposes.

2- One outrigger: for the wind load the maximum reduction in drift is 42% occurred at 0.467 H while for seismic load the maximum reduction in drift is 31% occurred at 0.6 H. These differences in reductions and their outrigger location is due to different nature of forces caused by wind which is the increment lightly and could be represent uniform distributed along the height of building, where the forces caused by seismic has much variation and it is represented by triangle start from the top as illustrated in figure (4.8).

3- Two outriggers the maximum reduction in wind is 58.3%, where in seismic is 49.7%.
4- The reduction in the considered direction is depended on the stiffness of shear walls (core) in that direction.
5- For these models it is clear that the seismic load controlling the design.

(2) The Irregular models due to mass:

In these study we will observe the drift due to dynamic seismic loads, figure (4.16) show the drifts comparison.

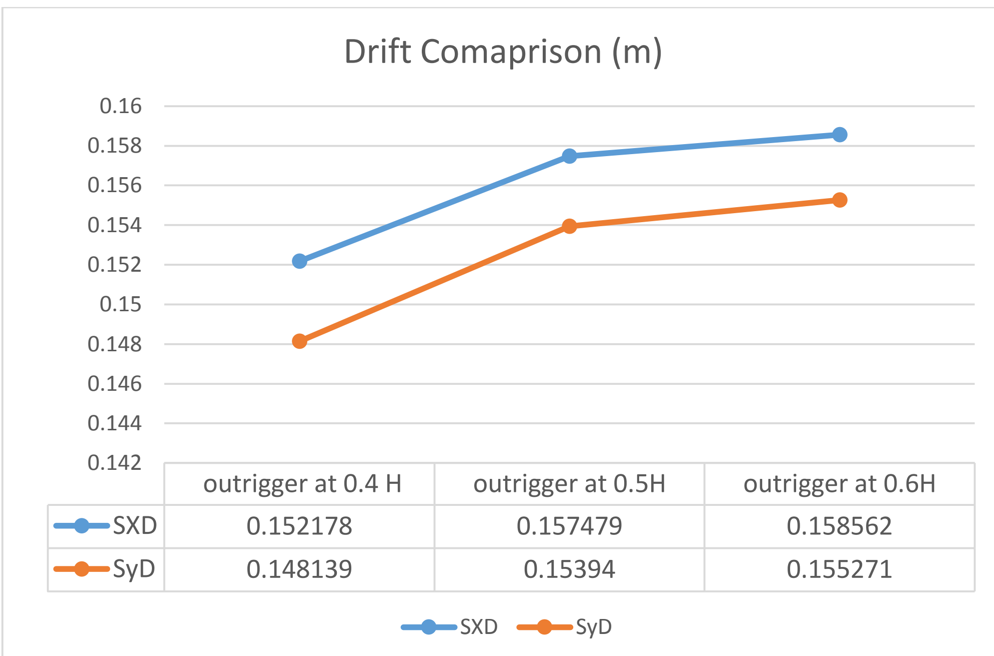
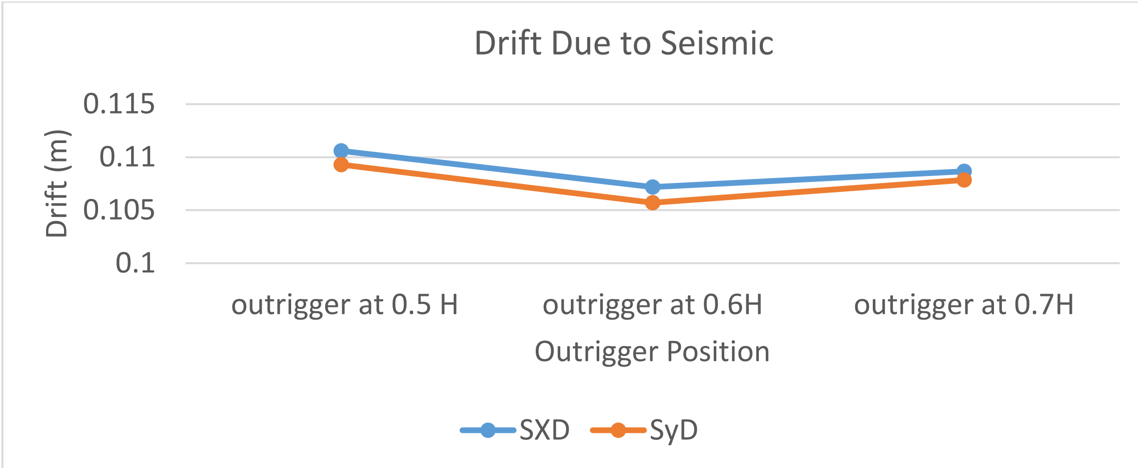


Figure (4.16): The Irregular models drifts.

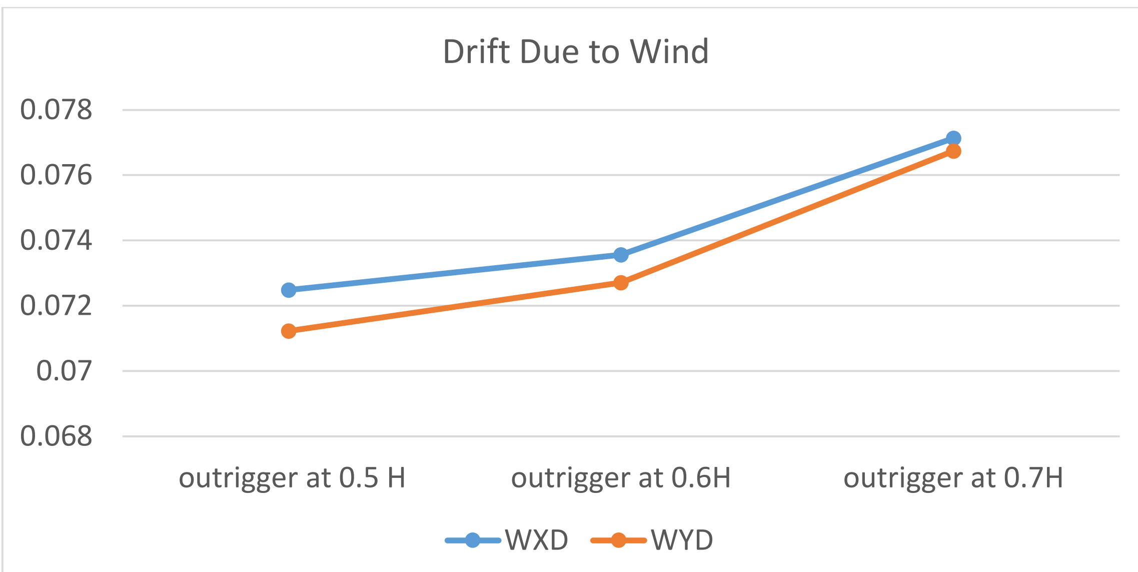
It could be observe the following:

1- The optimum position of the outrigger for the irregular due to mass located on top is 0.4 H, where for regular is 0.6 H in case of seismic load.
2- The difference between the drifts is very small – 5, 6, and 7mm – so could be neglected.

(3) Setbacks models: figure (4.17) A and B showing the drift for the models



(A) Drifts due to seismic.



(B) Wind drift.

Figures (4.17): (A) & (B) Setbacks Drifts

The followings could be observed:

1- The position of the outrigger which produce a minimum lateral drift in case of wind is 0.5H, where in the case of seismic is 0.6H.

2- The difference between drifts is very small and could be neglected, which will give the architectural aspects more flexibility to satisfy the client requirements.

Chapter Five

Conclusions and Recommendations

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Conclusions and Recommendations

5.1 Conclusions

The conclusions that can be drawn from the results of this study are as follows:

1- For the models based on shear the walls, the rigid frame and the outrigger system are classified as a rigid structure - the aspect ratio (H/W) less than 4- which means that it is not sensitive for lateral loads and the static approach for winds load can be considered conservative.

2- The gravity loads, wind load and design of vertical elements were carried out according to the British code, while the static and dynamic analysis for seismic loads carried out according to UBC 97.

3- For shear walls in the models studied it could be drawn that the optimum position is near to the center of mass, which will minimize the drifts and moments on the vertical elements, and the plan density will be in the acceptable range.

4- The UBC 97 provision consider the shear walls system more rigid than the rigid frame, because the time period was used (T_{used}) in shear walls is less than 1 second, while in rigid frame is greater than 1 sec.

5- The rigid frame system has an advantage –if the applicable layout followed- because it is produce the minimum lateral drift while in case of shear walls the drift was increased by 6%, but in the rigid frame must keep in mind the concept of strong column- weak beam as critical matter and clear height of storey.

6- The efficiency –in term of the reduction of drift- of the outrigger system for regular models for one outrigger was found to be 42% for wind and 31% for seismic, where reduction for the two outriggers was 58.3% for wind and 49.7% for seismic, while the theoretical reduction consider as 75%.

7- For the irregular models due to mass the optimum position was found at 0.4H, where for setbacks for wind was 0.5H and for seismic was 0.6H

8- It could be drawn that the outrigger system is more applicable for wind load rather than the seismic.

9- The reduction of drift in particular direction affected by the shear walls (core) stiffness in that direction.

5.2 Recommendations

These following recommendation may be made:

1- The position of shear walls must be near to the center of mass from the structural point of view, unless for architectural considerations.

2- It is better to use the outrigger system when the wind load governing the lateral loads design.

3- The expected reduction should be considered about 50% for one outrigger and about 60% for two outriggers.

4- For the rigid frame it is highly recommended to put the major axes of edge columns in direction of lateral loads as in plan (F).

5.3 Suggestion for Future Researches.

1. The extension of analysis to include the P-delta effects, which could maximize the drifts and members forces.
2. The extension of observation to taller buildings with different dynamic properties.
3. The extensions of observations to dynamic effects or blast loads.

References

1. Reinforced Concrete Design of Tall Building, Bungale S. Taranath, first Edition, 2010.
2. Reinforced Concrete Design Theory and Examples, P. Bhatt, Thomas Joseph MacGinley, B. S., (2006).
3. Wind And Earthquake Resistant Buildings structural Analysis and Design, Bungale S.Taranath.
4. 1997 Uniform Building code, Volume 2, Structural Design Requirements.
5. British Standard BS 6399-part 2-1997, Loading for Buildings Part 2: Code of Practice for Wind loads.
6. Tall Building Structures Analysis and Design, Bryan Stafford smith, Alex Coull, 1991.
7. Reinforced Concrete Design, W. H. Mosley, J. H. Bungey, Scholium International, Inc. (1991).
8. www.ajer.org, American Journal of Engineering Research, Optimum Position of Outrigger System for High Rise Reinforced Concrete Building Under Wind and Earthquake Loading, Raj Kiran Nanduri, B.Suresh, md. Ihtesham hussian, 2013.
9. Reinforced and Prestressed Concrete, F K Kong & R H Evans, third edition.
10. Reinforced concrete design to BS 8110, A.H.ALLEN, 1988.
11. Design of Reinforced Concrete, Jack C. McCormac, RussellH.Brown,Wiley, 7th edition, (2013).
12. Chan HC, Kuang JS. 1989a. Stiffened coupled shear walls. Journal of Engineering Mechanics, ASCE 115(4).
13. Chan HC, Kuang JS. 1989b. Elastic design charts for stiffened coupled shear walls. Journal of Structural Engineering, ASCE 115(2).

14. Coull A, Lao WHO. 1988. Outrigger braced structures subjected to equivalent static seismic loading. In Proceedings of 4th International Conference on Tall Buildings, Hong Kong.
15. Moudares FR. 1984. Outrigger-braced coupled shear walls. Journal of Structural Engineering, ASCE 110(12).

Appendix [A]

The 1997 Uniform Building Code Tables

TABLE 16-I—SEISMIC ZONE FACTOR *Z*

ZONE	1	2A	2B	3	4
Z	0.075	0.15	0.20	0.30	0.40

NOTE: The zone shall be determined from the seismic zone map in Figure 16-2.

TABLE 16-J—SOIL PROFILE TYPES

SOIL PROFILE TYPE	SOIL PROFILE NAME/GENERIC DESCRIPTION	AVERAGE SOIL PROPERTIES FOR TOP 100 FEET (30 480 mm) OF SOIL PROFILE		
		Shear Wave Velocity, $\bar{v}_s$ feet/second (m/s)	Standard Penetration Test, $\bar{N}$ [or $\bar{N}_{CH}$ for cohesionless soil layers] (blows/foot)	Undrained Shear Strength, $\bar{s}_u$ psf (kPa)
S_A	Hard Rock	> 5,000 (1,500)	—	—
S_B	Rock	2,500 to 5,000 (760 to 1,500)		
S_C	Very Dense Soil and Soft Rock	1,200 to 2,500 (360 to 760)	> 50	> 2,000 (100)
S_D	Stiff Soil Profile	600 to 1,200 (180 to 360)	15 to 50	1,000 to 2,000 (50 to 100)
S_E[1]	Soft Soil Profile	< 600 (180)	< 15	< 1,000 (50)
S_F	Soil Requiring Site-specific Evaluation. See Section 1629.3.1.			

[1]Soil Profile Type S_E also includes any soil profile with more than 10 feet (3048 mm) of soft clay defined as a soil with a plasticity index, $PI > 20$, $w_{mc} \geq 40$ percent and $s_u < 500$ psf (24 kPa). The Plasticity Index, *PI*, and the moisture content, w_{mc}, shall be determined in accordance with approved national standards.

TABLE 16-K—OCCUPANCY CATEGORY

OCCUPANCY CATEGORY	OCCUPANCY OR FUNCTIONS OF STRUCTURE	SEISMIC IMPORTANCE FACTOR, *I*	SEISMIC IMPORTANCE[1] FACTOR, I_p	WIND IMPORTANCE FACTOR, I_w
1. Essential facilities[2]	Group I, Division 1 Occupancies having surgery and emergency treatment areas Fire and police stations Garages and shelters for emergency vehicles and emergency aircraft Structures and shelters in emergency-preparedness centers Aviation control towers Structures and equipment in government communication centers and other facilities required for emergency response Standby power-generating equipment for Category 1 facilities Tanks or other structures containing housing or supporting water or other fire-suppression material or equipment required for the protection of Category 1, 2 or 3 structures	1.25	1.50	1.15
2. Hazardous facilities	Group H, Divisions 1, 2, 6 and 7 Occupancies and structures therein housing or supporting toxic or explosive chemicals or substances Nonbuilding structures housing, supporting or containing quantities of toxic or explosive substances that, if contained within a building, would cause that building to be classified as a Group H, Division 1, 2 or 7 Occupancy	1.25	1.50	1.15
3. Special occupancy structures[3]	Group A, Divisions 1, 2 and 2.1 Occupancies Buildings housing Group E, Divisions 1 and 3 Occupancies with a capacity greater than 300 students Buildings housing Group B Occupancies used for college or adult education with a capacity greater than 500 students Group I, Divisions 1 and 2 Occupancies with 50 or more resident incapacitated patients, but not included in Category 1 Group I, Division 3 Occupancies All structures with an occupancy greater than 5,000 persons Structures and equipment in power-generating stations, and other public utility facilities not included in Category 1 or Category 2 above, and required for continued operation	1.00	1.00	1.00
4. Standard occupancy structures[3]	All structures housing occupancies or having functions not listed in Category 1, 2 or 3 and Group U Occupancy towers	1.00	1.00	1.00
5. Miscellaneous structures	Group U Occupancies except for towers	1.00	1.00	1.00

[1]The limitation of I_p for panel connections in Section 1633.2.4 shall be 1.0 for the entire connector.
[2]Structural observation requirements are given in Section 1702.
[3]For anchorage of machinery and equipment required for life-safety systems, the value of I_p shall be taken as 1.5.

TABLE 16-L—VERTICAL STRUCTURAL IRREGULARITIES

IRREGULARITY TYPE AND DEFINITION	REFERENCE SECTION
1. **Stiffness irregularity—soft story** A soft story is one in which the lateral stiffness is less than 70 percent of that in the story above or less than 80 percent of the average stiffness of the three stories above.	1629.8.4, Item 2
2. **Weight (mass) irregularity** Mass irregularity shall be considered to exist where the effective mass of any story is more than 150 percent of the effective mass of an adjacent story. A roof that is lighter than the floor below need not be considered.	1629.8.4, Item 2
3. **Vertical geometric irregularity** Vertical geometric irregularity shall be considered to exist where the horizontal dimension of the lateral-force-resisting system in any story is more than 130 percent of that in an adjacent story. One-story penthouses need not be considered.	1629.8.4, Item 2
4. **In-plane discontinuity in vertical lateral-force-resisting element** An in-plane offset of the lateral-load-resisting elements greater than the length of those elements.	1630.8.2
5. **Discontinuity in capacity—weak story** A weak story is one in which the story strength is less than 80 percent of that in the story above. The story strength is the total strength of all seismic-resisting elements sharing the story shear for the direction under consideration.	1629.9.1

TABLE 16-M—PLAN STRUCTURAL IRREGULARITIES

IRREGULARITY TYPE AND DEFINITION	REFERENCE SECTION
1. **Torsional irregularity—to be considered when diaphragms are not flexible** Torsional irregularity shall be considered to exist when the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts of the two ends of the structure.	1633.1, 1633.2.9, Item 6
2. **Re-entrant corners** Plan configurations of a structure and its lateral-force-resisting system contain re-entrant corners, where both projections of the structure beyond a re-entrant corner are greater than 15 percent of the plan dimension of the structure in the given direction.	1633.2.9, Items 6 and 7
3. **Diaphragm discontinuity** Diaphragms with abrupt discontinuities or variations in stiffness, including those having cutout or open areas greater than 50 percent of the gross enclosed area of the diaphragm, or changes in effective diaphragm stiffness of more than 50 percent from one story to the next.	1633.2.9, Item 6
4. **Out-of-plane offsets** Discontinuities in a lateral force path, such as out-of-plane offsets of the vertical elements.	1630.8.2; 1633.2.9, Item 6; 2213.9.1
5. **Nonparallel systems** The vertical lateral-load-resisting elements are not parallel to or symmetric about the major orthogonal axes of the lateral-force-resisting system.	1633.1

TABLE 16-P—R AND Ω_o FACTORS FOR NONBUILDING STRUCTURES

STRUCTURE TYPE	R	Ω_o
1. Vessels, including tanks and pressurized spheres, on braced or unbraced legs.	2.2	2.0
2. Cast-in-place concrete silos and chimneys having walls continuous to the foundations.	3.6	2.0
3. Distributed mass cantilever structures such as stacks, chimneys, silos and skirt-supported vertical vessels.	2.9	2.0
4. Trussed towers (freestanding or guyed), guyed stacks and chimneys.	2.9	2.0
5. Cantilevered column-type structures.	2.2	2.0
6. Cooling towers.	3.6	2.0
7. Bins and hoppers on braced or unbraced legs.	2.9	2.0
8. Storage racks.	3.6	2.0
9. Signs and billboards.	3.6	2.0
10. Amusement structures and monuments.	2.2	2.0
11. All other self-supporting structures not otherwise covered.	2.9	2.0

TABLE 16-Q—SEISMIC COEFFICIENT C_a

SOIL PROFILE TYPE	SEISMIC ZONE FACTOR, Z				
	Z = 0.075	Z = 0.15	Z = 0.2	Z = 0.3	Z = 0.4
S_A	0.06	0.12	0.16	0.24	$0.32N_a$
S_B	0.08	0.15	0.20	0.30	$0.40N_a$
S_C	0.09	0.18	0.24	0.33	$0.40N_a$
S_D	0.12	0.22	0.28	0.36	$0.44N_a$
S_E	0.19	0.30	0.34	0.36	$0.36N_a$
S_F	See Footnote 1				

[1]Site-specific geotechnical investigation and dynamic site response analysis shall be performed to determine seismic coefficients for Soil Profile Type S_F.

TABLE 16-R—SEISMIC COEFFICIENT C_v

SOIL PROFILE TYPE	SEISMIC ZONE FACTOR, Z				
	$Z = 0.075$	$Z = 0.15$	$Z = 0.2$	$Z = 0.3$	$Z = 0.4$
S_A	0.06	0.12	0.16	0.24	$0.32N_v$
S_B	0.08	0.15	0.20	0.30	$0.40N_v$
S_C	0.13	0.25	0.32	0.45	$0.56N_v$
S_D	0.18	0.32	0.40	0.54	$0.64N_v$
S_E	0.26	0.50	0.64	0.84	$0.96N_v$
S_F	See Footnote 1				

[1]Site-specific geotechnical investigation and dynamic site response analysis shall be performed to determine seismic coefficients for Soil Profile Type S_F.

TABLE 16-N—STRUCTURAL SYSTEMS[1]

BASIC STRUCTURAL SYSTEM[2]	LATERAL-FORCE-RESISTING SYSTEM DESCRIPTION	R	Ω_o	HEIGHT LIMIT FOR SEISMIC ZONES 3 AND 4 (feet) × 304.8 for mm
1. Bearing wall system	1. Light-framed walls with shear panels			
	a. Wood structural panel walls for structures three stories or less	5.5	2.8	65
	b. All other light-framed walls	4.5	2.8	65
	2. Shear walls			
	a. Concrete	4.5	2.8	160
	b. Masonry	4.5	2.8	160
	3. Light steel-framed bearing walls with tension-only bracing	2.8	2.2	65
	4. Braced frames where bracing carries gravity load			
	a. Steel	4.4	2.2	160
	b. Concrete[3]	2.8	2.2	—
	c. Heavy timber	2.8	2.2	65
2. Building frame system	1. Steel eccentrically braced frame (EBF)	7.0	2.8	240
	2. Light-framed walls with shear panels			
	a. Wood structural panel walls for structures three stories or less	6.5	2.8	65
	b. All other light-framed walls	5.0	2.8	65
	3. Shear walls			
	a. Concrete	5.5	2.8	240
	b. Masonry	5.5	2.8	160
	4. Ordinary braced frames			
	a. Steel	5.6	2.2	160
	b. Concrete[3]	5.6	2.2	—
	c. Heavy timber	5.6	2.2	65
	5. Special concentrically braced frames			
	a. Steel	6.4	2.2	240
3. Moment-resisting frame system	1. Special moment-resisting frame (SMRF)			
	a. Steel	8.5	2.8	N.L.
	b. Concrete[4]	8.5	2.8	N.L.
	2. Masonry moment-resisting wall frame (MMRWF)	6.5	2.8	160
	3. Concrete intermediate moment-resisting frame (IMRF)[5]	5.5	2.8	—
	4. Ordinary moment-resisting frame (OMRF)			
	a. Steel[6]	4.5	2.8	160
	b. Concrete[7]	3.5	2.8	—
	5. Special truss moment frames of steel (STMF)	6.5	2.8	240
4. Dual systems	1. Shear walls			
	a. Concrete with SMRF	8.5	2.8	N.L.
	b. Concrete with steel OMRF	4.2	2.8	160
	c. Concrete with concrete IMRF[5]	6.5	2.8	160
	d. Masonry with SMRF	5.5	2.8	160
	e. Masonry with steel OMRF	4.2	2.8	160
	f. Masonry with concrete IMRF[3]	4.2	2.8	—
	g. Masonry with masonry MMRWF	6.0	2.8	160
	2. Steel EBF			
	a. With steel SMRF	8.5	2.8	N.L.
	b. With steel OMRF	4.2	2.8	160
	3. Ordinary braced frames			
	a. Steel with steel SMRF	6.5	2.8	N.L.
	b. Steel with steel OMRF	4.2	2.8	160
	c. Concrete with concrete SMRF[3]	6.5	2.8	—
	d. Concrete with concrete IMRF[3]	4.2	2.8	—
	4. Special concentrically braced frames			
	a. Steel with steel SMRF	7.5	2.8	N.L.
	b. Steel with steel OMRF	4.2	2.8	160
5. Cantilevered column building systems	1. Cantilevered column elements	2.2	2.0	35[7]
6. Shear wall-frame interaction systems	1. Concrete[8]	5.5	2.8	160
7. Undefined systems	See Sections 1629.6.7 and 1629.9.2	—	—	—

N.L.—no limit

[1]See Section 1630.4 for combination of structural systems.

[2]Basic structural systems are defined in Section 1629.6.

[3]Prohibited in Seismic Zones 3 and 4.

[4]Includes precast concrete conforming to Section 1921.2.7.

[5]Prohibited in Seismic Zones 3 and 4, except as permitted in Section 1634.2.

[6]Ordinary moment-resisting frames in Seismic Zone 1 meeting the requirements of Section 2211.6 may use a R value of 8.

[7] Total height of the building including cantilevered columns.

[8]Prohibited in Seismic Zones 2A, 2B, 3 and 4. See Section 1633.2.7.