

Chapter One

Introduction

1.1 General

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

Generally, a tall building refers to a building in which its height creates different conditions in the design, construction and usage compared to the conventional structure. In other words, tall buildings refer to vertical construction for which the wind and earthquake effects are much more significant and greatly emphasized compared to its structural weight or the imposed load on the structure.

The more typical light and flexible modern tall building is much more responsive to dynamic exciting forces than its earlier counterpart. The resulting dynamic displacements and shear forces may be much greater than static values, while induced motions may disturb the comfort and equanimity of the buildings occupants.

Tall building motion may be classified as static or dynamic. The first refers to the motion produced by slowly applied forces such as gravitational or thermal effect, or long period component of wind. Dynamic motion refers to those caused by time-dependent dynamic, forces, notably seismic acceleration, short period wind loads, blasts and machinery vibration, the first two usually being of the greater concern. Carrying out nonlinear dynamic analysis of tall buildings under wind loads have been recommended by many authors.

1.2 Statement of the Research problem

Although for many tall buildings it is possible to treat the dynamic forces due to wind or seismic actions by equivalent static lateral loads, it may be necessary in certain cases to use a dynamic method of analysis to obtain an accurate representation of the peak dynamic forces and stresses, deflection, and accelerations. It is also important to ensure that the building motions do not affect the comfort and equanimity of the occupants.

It is thus important for the engineer to be able to determine in the early design stages if the structure is static or dynamic, particularly in view of the comfort criteria for the occupants. To rectify an unacceptable dynamic response after the structure has been built will, if at all possible, generally be difficult and very expensive.

In this research the nonlinear dynamic analysis of tall building under wind loads will be carried out. The analysis will be based on nonlinear dynamic finite element analysis. The problem of how to verify the accuracy and reliability of the nonlinear dynamic analysis and will be addressed.

1.3 Objectives

The aim of this research is:

1. To study the effect of wind loads on building towers.
2. To study the nonlinear Dynamic behavior of tall buildings under wind loads.
3. To learn how to use computer programs in the nonlinear dynamic analysis of tall buildings under wind loads.
4. To carry out dynamic nonlinear analysis of a specific tall building under wind loads.
5. To study the effect of the non linear dynamic analysis in comparison with nonlinear static analysis.

1.4 Methodology

- Comprehensive literature review (Analysis) of frame type Tall building under wind loads linear, nonlinear static, nonlinear dynamic analysis.
- Carrying out static linear analysis of the selected 20 stories tall building to check the accuracy of the finite element discretization model.
- Carrying out static nonlinear analysis of same finite element model using (ETABS) .and comparing with published results.
- Carrying out nonlinear dynamic analysis of the same finite element model using (ETABS) and evaluation of the results obtained.

1.5 Thesis Components

- This research consists of five chapters as follows:
- Chapter 1: Includes general introduction. Research Problem statement, objectives, methodology, and thesis components.
- Chapter 2: Includes literature review of tall building: definition, systems of tall building, loads, p-delta effect, and dynamic analysis.
- Chapter 3: includes Equations and relations to be used in analysis based on code requirements and analysis program ETABS.
- Chapter 4: Includes the case study static and dynamic analysis results and discussion.
- Chapter 5: Includes conclusions and recommendations.

Chapter Two

Background and Literature Review

2.1 Introduction:

This chapter contains the tall building: definition, historical background and literature review of behavior of tall building, the structural systems and loads which act on tall buildings such as gravity loads and lateral loads and the nonlinear static and dynamic methods.

Smith and Coull, 1991 stated that tall building cannot be defined in specific terms related just to height or to the number of the floors ,The tallness of a building is matter of a persons or community's circumstance and their consequent perception; therefore a measurable definition of a tall building cannot be universally applied. From the structural engineer's point of view, however, a tall building may be defined as one that, because of its height, is affected by lateral forces due to wind or earthquake actions to an extent that they play an important role in the structural design. The influence of these actions must therefore be considered from the very beginning of the design process.

As stated by Karim and Barua,2010 in the U.S, the National Fire Protection Association defines a high-rise as being higher than 75 feet (23 meters), or about 7 stories while most building engineers, inspectors, architects and similar professions define a high-rise as a building that is at least 75 feet (23 m) tall. They also stated that high-rise is the demand of new era as it provides accommodation to a well number of people in a small place.

As stated by Ahmed, 2015 a reasonable and accurate assessment of a proposed tall building behavior is necessary to form a properly representative model for analysis. A tall building structure is essentially a vertical cantilever subjected to an axial loading by gravity and to transverse loading by wind or earthquake .Gravity live loading acts on the slabs, which transfer it horizontally to the vertical walls, beams and columns through which it passes to the foundation. The magnitude of an axial loading in the vertical component estimated from the slab tributary areas, and its calculations are not usually considered a difficult problem. Horizontal loading exerts at each level of a building shear, moment and sometimes torque, which have maximum values at the base of the structure. The maximum value increases rapidly with the building's height. The response of structure to horizontal loading, in having to carry the external shear, moment, torque, is more complex than its first order response to gravity loading. The recognition of the structure behavior under horizontal loading and the formation of the corresponding model are usually the dominant problem of analysis. The principle criterion of a satisfactory model is that under horizontal loading it should deflect similar to the prototype structure.

2.2Loads

As stated by ASCE/SEI7-10 the Loads that act on structures consist mainly of gravity loads such as (Dead, live and construction loads) and environmental loads (wind, seismic, thermal, snow, etc...).

As Dead loads are constant in magnitude and fixed in location throughout the life time of the structure. Usually the major part of the dead load is the weight of the structure it self. This can be calculated with good accuracy from the design conFig.uration, dimension of the structure and density of the material. For building, floor fill, finish floor and plastered ceiling are usually including as dead loads.

Live load as the load whose magnitude and placement change with time. Such loads are due to the weights of people, animals, furniture, movable equipments and stored materials. An earthquake is sudden, rapid shaking of the earth caused by beneath of the earth surface. Earthquake destroys construction such as building and high ways. Beside that earthquake causes loss of human and animals lives.

Wind loads are the forces exerted by winds on buildings and they are considered to be dynamic as they vary greatly in intensity from time to time. Wind loads typically act laterally on walls and may act upward and downward on roofs.

The site wind speed V refers to a standard open country exposure at a height of 10 m above ground. To obtain the effective wind speed the effects of varying ground roughness, the height and distance of obstructions upwind of the site and the effects of topography should be taken into account.

The wind speed acts as pressure when it meets with a structure. The intensity of that pressure is the wind load. Calculating wind load is necessary for the design and construction of safer, more wind-resistant buildings. The code used to estimate the wind loading is American Society of Civil Engineers Minimum Design Loads for Buildings and Other Structures.

Mendis, et.al.,2007 stated that Wind is a phenomenon of great complexity because of the many flow situations arising from the interaction of wind with structures. Wind is composed of a multitude of eddies of varying sizes and rotational characteristics carried along in a general stream of air moving relative to the earth's surface. These eddies give wind its gusty or turbulent character. The gustiness of strong winds in the lower levels of the atmosphere largely arises from interaction with surface features. The average wind speed over a time period of the order of ten minutes or more tends to increase with height, while the gustiness tends to decrease with height.

They, also, stated that the wind vector at a point may be regarded as the sum of the mean wind vector (static component) and a dynamic, or turbulence, component. A consequence of turbulence is that dynamic loading on a structure depends on the size of the eddies. Large eddies, whose dimensions are comparable with the structure, give rise to well correlated pressures as they envelop the structure. On the other hand, small eddies result in pressures on various parts of a structure that become practically uncorrelated with distance of separation. Eddies generated around a typical structure are shown in Fig. (2.1). Some structures, particularly those that are tall or slender, respond dynamically to the effects of wind.

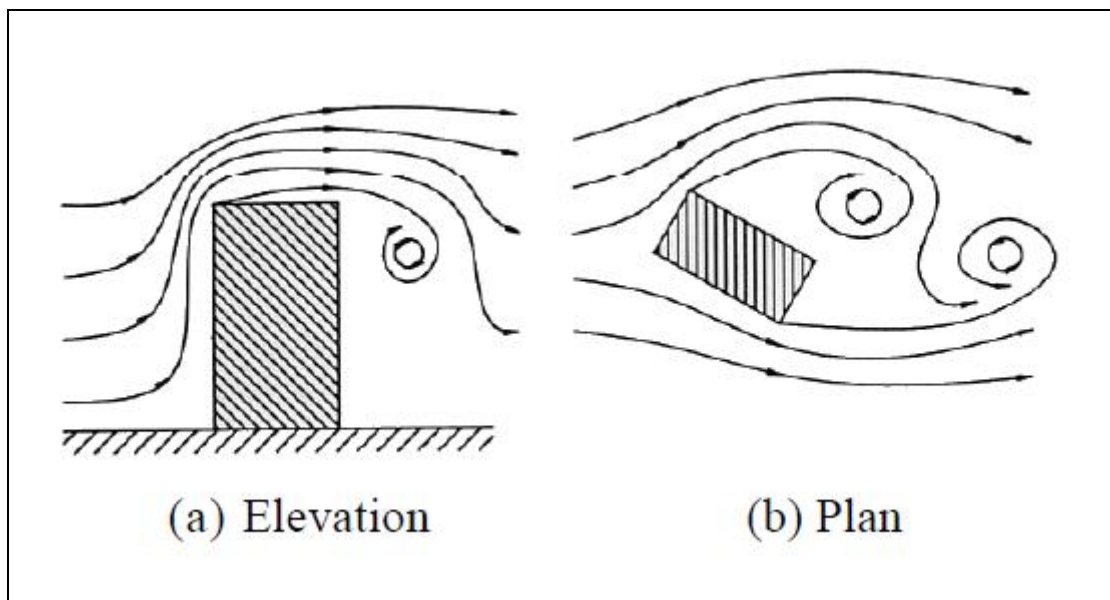


Fig. (2.1): Generation of eddies [Mendis, et.al, 2007]

2.3 Structural Systems of Tall Buildings

As stated by Smith and Coull, 1991 from the structural engineering point of view, the determination of the structural form of a tall building would ideally involve only the selection and arrangement of major structural elements to resist most efficient the various combinations of gravity and horizontal loading. In reality however the choice of structural form is usually strongly influenced by factors other than structural consideration. The range of factors that have to be taken into account in deciding the

structural form includes the internal planning, the material and method of construction, the external architectural treatment, the planned location and routing of the service system, the nature and magnitude of the horizontal loading and the height and proportions of the building. The taller and more slender a building, the more important the structural factors become, and the more necessary to choose an appropriate structural form. In addition to satisfying the previously mentioned nonstructural requirement, the principal objectives in choosing a building's structural form are to arrange to support the gravity, dead and live loading, and to resist at all levels the external horizontal load shear, moment, and torque with adequate strength and stiffness. These requirements should be achieved of course, as economically as possible. While rigid frames of typical scale that serve alone to resist lateral loading have an economic height limit of about 25 stories, smaller scale rigid frames in the form of a perimeter tube, or typically scaled rigid frames in combination with shear walls or braced bent, can be economic up to much greater height. Different structural systems have gradually evolved for residential and office buildings, reflecting their differing functional requirements. In modern office buildings, the need to satisfy the differing requirements of individual clients for floor space arrangements led to the provision of large column free open areas to allow flexibility in planning. The components of tall buildings could be categorized as floor system, vertical load resisting system, lateral loads resisting system, connection, energy dissipation system and damping .The structural classified systems which are most commonly used in the graph in Fig. (2.2).

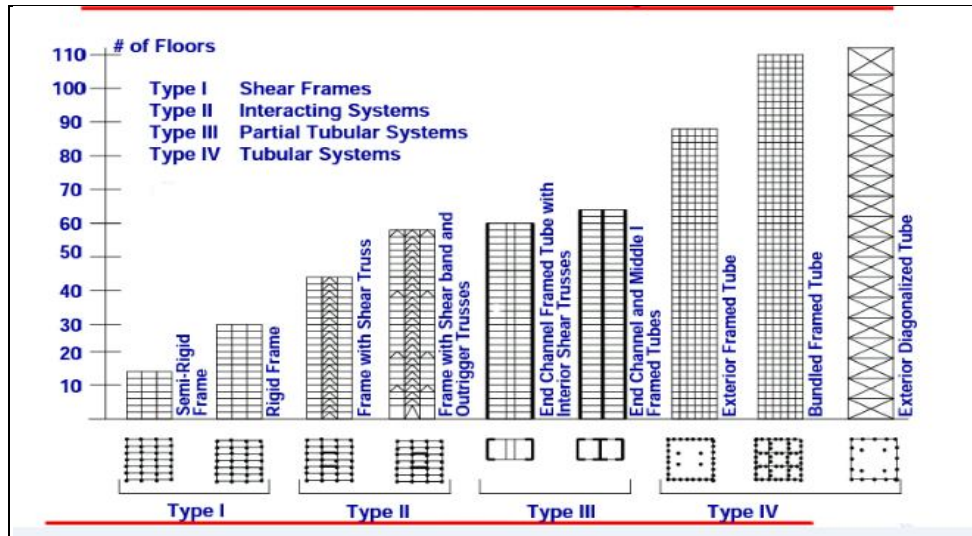


Fig. (2.2): System Structure Classification [Karim and Barua, 2010]

2.3.1 Floor System

Taranath, 2010 stated that, the floor system carries the gravity load during and after construction. It also resists lateral loads through diaphragm action by providing a continuous path for transferring lateral loads from the bottom chord of one truss to the top chord of adjacent truss down the structure. Finally, it accommodates the mechanical system (heating, ventilating, and air condition). It should also have fire resistance properties. It can be classified as: two way systems, one way system and beam and slab systems. Two way systems including flat plates supported by columns, flat slabs supported by columns with capitals or drop panels, slab of constant thickness, slabs with waffles and two way joists are also used. One way system includes slabs of constant thickness with span 3m to 8m.

2.3.2 Vertical Load Resisting System

Taranath, 2010 stated that, Vertical elements are columns, shear walls, hangers, transfer girders and suspended systems such as cable suspended floors. Steel concrete or composite materials are used. Shear walls carry the loads in compression, and sometimes, like staggered trusses between floors. Transfer girders are used to bridge large openings at lower levels of a tall building.

2.3.3 Lateral load Resistance System

Taranath, 2010 stated that, in contrast with the vertical load, lateral load effects on buildings are quite variable and their maximum values increase rapidly with increase in height. For example, under wind load the overturning moment at the base of the building varies in proportion with the square of the building height and lateral deflection varies as the fourth power of the building height. The essential role of the lateral system is to carry the wind and earthquake loads, as well as to resist the P-Delta effects due to secondary moments in columns and to keep the inter-storey drift in a minimum range.

2.3.3.1 Outrigger and Belt Truss System

Taranath, 2005 stated that, the structural arrangement for an outrigger system consists of a main core connected to the exterior columns by relatively stiff horizontal members commonly referred to as outriggers. The main core may consist of a steel braced frame or reinforced concrete shear walls and may be centrally located with outrigger extending on both sides. It may also be located on one side of the building with outriggers extending to the building columns on one side.

The structure response is quite simple when subjected to the lateral loads, the column restrained outrigger 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 as shown in Fig. (2.3).

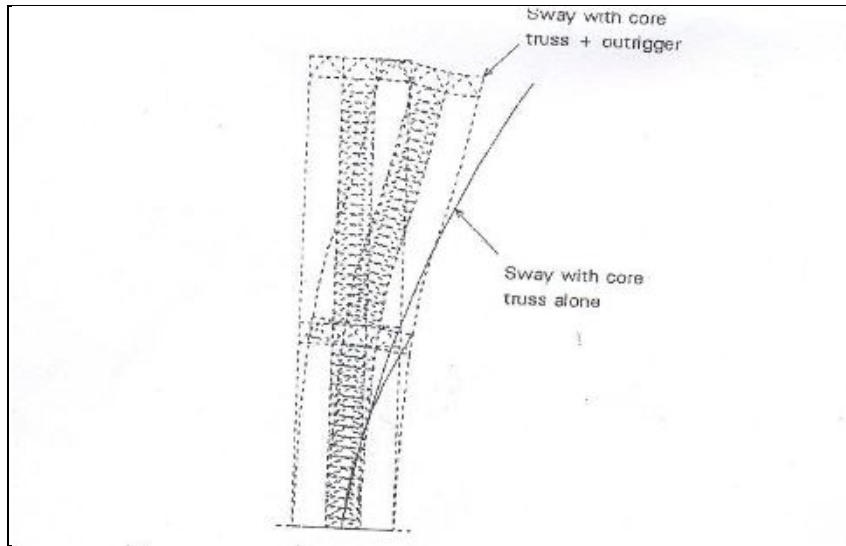


Fig. (2.3): Double outrigger effect on a tall building [Taranath, 2005]

In addition to those columns located at the ends of the outriggers, it is also common to mobilize other peripheral columns to assist the restraining of the outriggers. This is achieved by including a “belt truss”, around the structure at the level of the outriggers. To make the outriggers and belt truss adequately stiff in flexure and shear, they are made at least one or two stories deep as shown in Fig. (2.4).

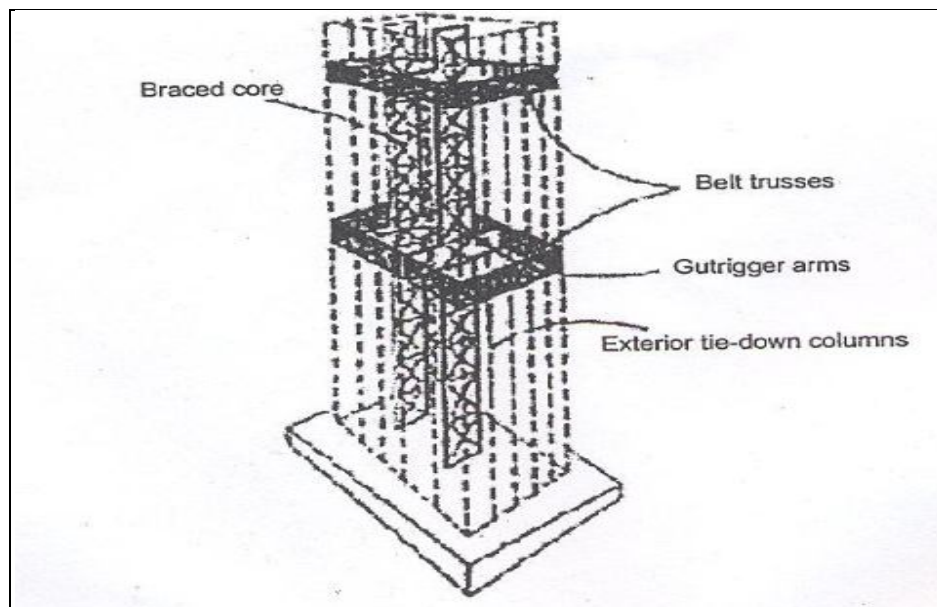


Fig. (2.4): Single Outrigger and Belt Truss Schematic [Taranath, 2005]

2.3.3.2 Framed Tube System

Taranth, 2010 stated that, a framed tube can be defined as a three dimensional system Fig. (2.5) utilizing the entire building perimeter to resist lateral loads. A necessary requirement to achieve a behavior like this is to place columns on the building exterior relatively close to each other, joined by deep spandrel girders. Columns are usually placed 3.05-6.1m apart and with spandrel depths varying from 0.91 to 1.52 m.

Although the structure has a tube like form, its behavior is much more complex than that of a solid tube. Unlike a solid tube, it is subjected to the effects of shear lag, which have a tendency to modify the axial distribution in the columns. The axial stiffness in the corner columns is increased and the stiffness in the inner columns is decreased. The stresses in the inner columns lag behind those in the corner columns (due to the bending of the connecting spandrel), hence the term shear lag.

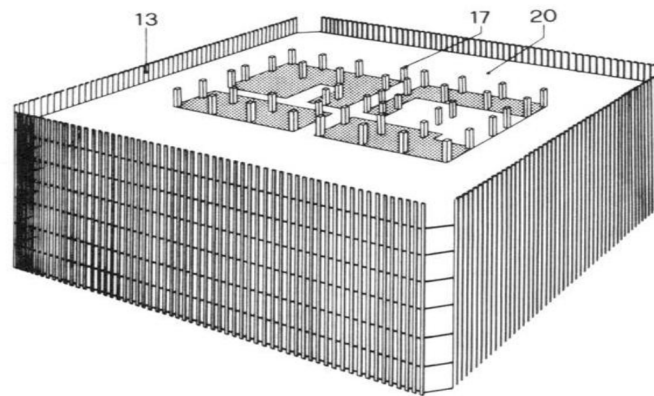


Fig. (2.5): Frame tube [Taranth, 2005]

2.3.3.3 Shear Wall Structures

As stated by Smith and Coull, 1991 Concrete continuous vertical walls may serve both architecturally as partitions and structurally to carry gravity and lateral loading. Their very high value in plane stiffness and strength makes them ideally for bracing tall building. In a shear wall structure, such walls are entirely responsible for lateral load resistance of the building. The shear walls structure is considered to be one whose resistance to horizontal loading is provided by shear walls. It is usual to locate

the walls on plan so that they attract an amount of gravity dead loading sufficient to suppress the maximum tensile bending stresses in the wall caused by lateral loading. In this situation only minimum wall reinforcement is required. The term “shear wall” is in some ways a misnomer because the walls deform predominantly in flexure. Shear wall may be planer, but are often of L.T I or U shaped section to better suit the planning and increase their flexure stiffness. Walls that are connected by floor slab or beams with negligible bending resistance. So that only horizontal interactive forces are transmitted. Walls connected by bending members are termed “coupled walls” as shown Fig. (2.6).

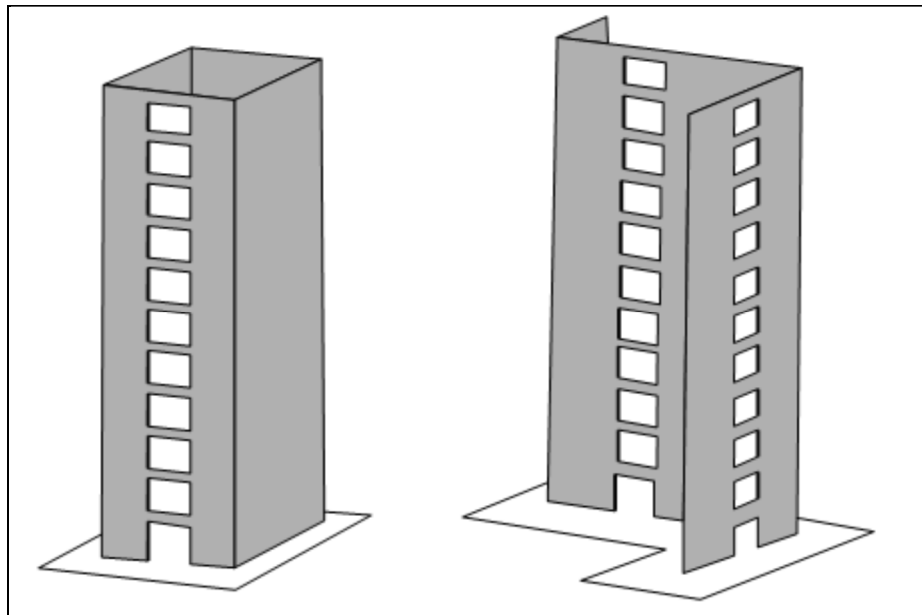


Fig. (2.6): Coupled Shear Walls [Taranth, 2005]

2.3.3.4 Wall-Frame Structures

As stated by Smith and Coull, 1991 a structure, whose resistance to horizontal loading provided by a combination of shear wall and rigid frames or in the case of a steel structure by braced bent and rigid frames, may be categorized as a wall-frame. The shear or braced bents are often parts of the elevator and service cores while the frames are arranged in plan, in conjunction with the walls to support the floor system. The

horizontal interaction can be effective in contributing to lateral stiffness to the extent that wall-frames of up to 50 stories or more are economical.

2.3.3.5 Rigid Frame Structures

As stated by Taranath, 2005 rigid frame structures consist of columns and girder joined by moment resistant connection. The lateral stiffness of a rigid-frame bent depends on bending stiffness of the columns, girders and connection in the plane of the bent as shown in Fig. (2.7). The advantages of a rigid frame are the simplicity and convenience of its rectangular shape.

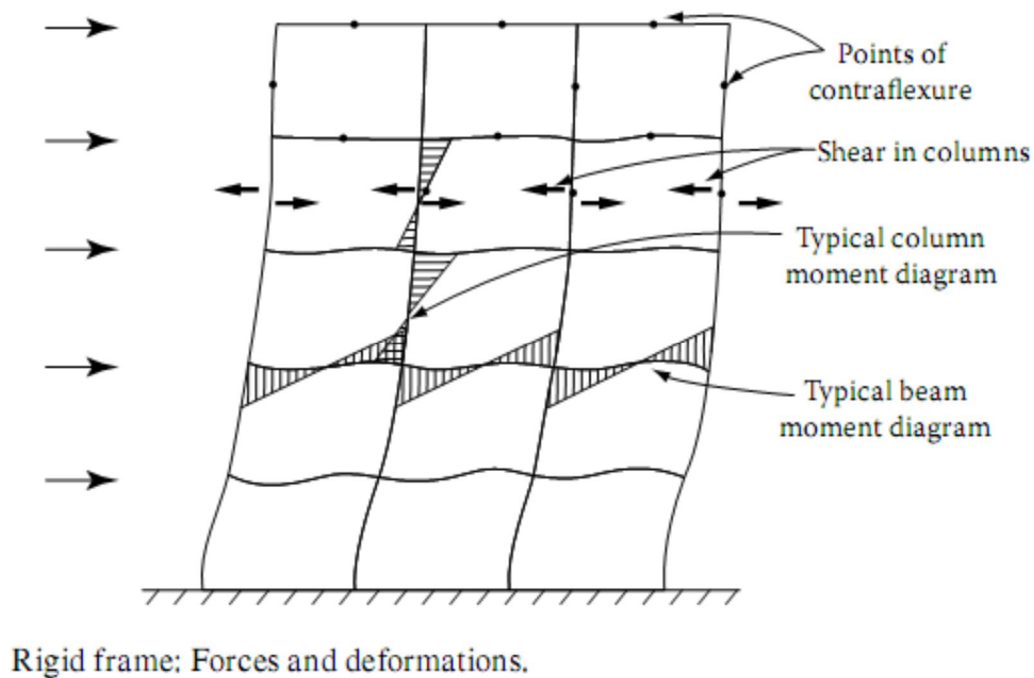


Fig. (2.7): Rigid Frame Structures [Taranath, 2010]

2.4 Static Nonlinear Analysis (P-delta)

Karim and Barua, 2010 stated that, in building analysis the lateral movement of a storey mass to a deformed position generates second-order overturning moments. This second-order behavior has been termed the P-Delta effect since the additional overturning moments on the building are equal to the sum of storey weights “P” times the lateral displacements “Delta”. Many techniques have been proposed for evaluating

this second-order behavior, which takes place in two steps while linear static analysis considers one 1st order loading stage as can be seen from Fig. (2.8).

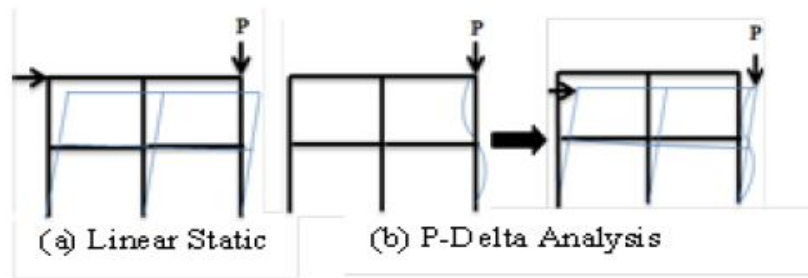


Figure (2.8): (a) Linear Static analysis is performed in one step (b) P- Delta analysis is performed in two steps [Karim and Barua, 2010]

Also they stated that, P-Delta is secondary order loading effect in structure directly related to stiffness as it reduces the stiffness of structural elements. The analysis procedures used to determine P-Delta effects are quite variable. Several methods of accommodating P-Delta effects in analysis have been developed. Some of these methods rely on a constrained problem or set of conditions. The P-Delta effect does not distinguish between directions and types of loading. P-delta does not take into consideration floors, floor levels, or the difference between a column and a beam. Proper care should be taken to work within the limitations of the analysis .A unique procedure has been adopted by Karim and Barua, 2010 to incorporate the P-Delta effect into the analysis. The procedure consists of the following steps:

1. First, the primary deflections are calculated based on the provided external loading.
2. Primary deflections are then combined with the originally applied loading to create the secondary loadings. The load vector is then revised to include the secondary effects.

Lateral loading must be present concurrently with the vertical loading for consideration of the P-Delta effect. The Repeat Load facility has been created with

this requirement in mind. This facility allows the user to combine previously defined primary load cases to create a new primary load case.

3. A new stiffness analysis is carried out based on the revised load vector to generate new deflections.

4. Element/Member forces and support reactions are calculated based on the new deflections.

P-Delta effects are calculated for frame members only not for finite elements or solid elements. So outcomes are compared against frame members only.

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As stated by Mendis, et.al.,2007 the static approach is based on a quasi-steady assumption, and assumes that the Building is a fixed rigid body in the wind. The static method is not appropriate for tall structures of exceptional height, slenderness, or susceptibility to vibration in the wind. In practice, static analysis is normally appropriate for structures up to 50 meters in height.

Mosa, 2015 studied the effect of wind loads on tall buildings by carrying out nonlinear static analysis (P-delta and P-delta plus large displacements) and compared the results obtained with linear static analysis. The results shown that the use of p-delta plus large displacements is important to obtain the correct and accurate displacements and shear forces.

2.5 Dynamic Analysis

Smith and Coull , 1991 carried out dynamic analysis and they stated that, a dynamic analysis is required only when the building is relatively flexible or because of its shape structural arrangement, mass distribution, foundation condition or use, is particularly sensitive to wind or seismic acceleration. Then consideration has to be

given to both the stress levels that occur and the acceleration that may affect the comfort of the occupants.

Mendis, et.al., 2007, stated that, generally, dynamic method is for exceptionally tall, slender, or Vibration-prone buildings. The Codes not only provide some detailed design guidance with respect to dynamic response, but state specifically that a dynamic analysis must be undertaken to determine overall forces on any structure with both a height (and length) to breadth ratio greater than five, and a first mode frequency less than 1 Hertz.

Wind loading codes may give the impression, that wind forces are relatively constant with time. In reality, wind forces vary significantly over short time intervals with large amplitude fluctuations at high frequency intervals. The magnitude and frequency of the fluctuations are dependent on many factors associated with turbulence of the wind and local gusting effects caused by the structure and surrounding environment.

Dynamic wind pressures produce sinusoidal or narrow-band vibration motions and the building will generally undergo translations in both along-wind and cross-wind directions, and possibly rotations about vertical axis. The vast amount of experimental and theoretical research of the past few decades has allowed the inclusion in design codes of simplified methods of dynamic analysis that do not require a formal solution of the equations of motion.

Kareem, 1992 presented results of a recent study addressing issues concerning the dynamic response of high-rise buildings. Utilizing a high-frequency base-balance, the along wind, across wind and torsional components of aerodynamic loads and their statistical correlations for a wide range of generic building shapes of different aspect ratios in two approach flow conditions are quantified. A random-vibration-based procedure for estimating response of high-rise buildings is outlined. A checking procedure for building serviceability is presented for enabling designers to assess the building serviceability performance. The methodology presented here will enable

designers in the preliminary design stages to assess the serviceability of buildings, the need for a detailed wind tunnel test and to evaluate the merit of design modifications that may include a choice of an auxiliary damping system.

2.5.1 Dynamic Response to Wind Loading

Smith and Coull, 1991 carried out a dynamic Response to Wind Loading and they stated that, a complete description of the wind loading process relies on a proper definition of the wind climate from meteorological records, together with an understanding of atmospheric boundary layers, turbulence properties and the variation of wind speed with height, the aerodynamic forces produced by the interaction of the building with the turbulent boundary layer, and the dynamic response of the structure to the wind forces.

2.5.1.1 Sensitivity of Structures to Wind Forces

Smith and Coull, 1991 stated that the principal structural characteristics that affect the decision to make dynamic design analysis are the natural frequencies of the first few normal modes of vibration and the effective size of the building. When a structure is small, the whole building will be loaded by gusts so that the full range of frequencies from both boundary layer turbulence and building-generated turbulence will be encountered. On the other hand, when the building is relatively large or tall, the smaller gusts will not act simultaneously on all parts, and will tend to offset each other's effects, so that only the lower frequencies are significant.

If the structure is stiff, the first few natural frequencies will be relatively high, and there will be little energy in the spectrum of atmospheric turbulence available to excite resonance. The structure will thus tend to follow any fluctuating wind forces without appreciable amplification or attenuation. The dynamic deflections will not be significant, and the main design parameter to be considered is the, maximum loading to which the structure will be subjected during its lifetime. Such a structure is termed "static," and it may be analyzed under the action of static equivalent wind forces.

If a structure is flexible, the first few natural frequencies will be relatively low, and the response will depend on the frequency of the fluctuating wind forces. The dynamic deflections may be appreciably greater than the static values. The lateral deflection of the structure then becomes an important design parameter and the structure is classified as "dynamic." In such structures, the dynamic stresses must also be determined in the design process. Furthermore, the accelerations induced in dynamic structures may be important with regard to the comfort of the occupants of the building and must be considered.

Smith and Coull, 1991 stated that the Australian Code defines a dynamic building as one in which

1. The height exceeds five times the least plan dimension and
2. The natural frequency in the first mode of vibration is less than 1.0 Hz.

In the Canadian Code dynamic buildings are those whose height is greater than four times their minimum effective width, or greater than 120m in height. Such empirical guidelines should be considered applicable only to traditional forms of building structure, and may be inappropriate to apply to more radical innovations.

2.5.1.2 Along-wind Dynamic Response

Chen, 2009 carried out along-wind dynamic response analysis of tall buildings built in urban areas and he stated that in modern cities, buildings get taller and more slender. The effects of wind-induced motions become more pronounced. Accurate prediction of structural response to wind-induced random vibrations displacement and vibration acceleration at the design stage is a basis for structural safety design and serviceability design of modern tall buildings. The wind-induced response consists of along-wind response, across-wind response and torsion response.

Several quasi-static methods in wind codes in various countries and approximate methods based on wind tunnel studies are used to predict the wind-induced response. As a building gets taller, the wind-induced random vibrations becomes more violent,

these methods become less reliable. So the wind-induced response of modern tall buildings should be assessed accurately by using the theory of random vibration. Using the numerical analysis methods of random vibration response of tall buildings, people calculate structural response according to wind load spectra.

2.5.1.3 Across-wind Dynamic Response

Chen, 2013 carrying out across wind dynamic response and he stated that, the study addresses the analysis of crosswind response of tall buildings and flexible structures at wind speed region higher than the vortex lock-in speed, where nonlinear negative aerodynamic damping effect is significant. The modeling of nonlinear aerodynamic damping as a function of time-varying velocity and/or displacement of vibration is established based on motion-induced aerodynamic force information obtained from forced-vibration model testing in wind tunnel, referred to as harmonic balance. Response time history simulations are performed by solving the nonlinear equation of motion to explore the unique hardening non-Gaussian characteristics of crosswind response and its extreme value distribution with reduced peak factor. The response time history simulations also provide a bases for assessing the performance of analytical predictions of root-mean-square (RMS) response using crosswind loading spectrum and equivalent aerodynamic damping models

2.5.2 Response spectrum analysis

Smith and Coull, 1991 defined response spectrum as a graphic representation of the maximum response of a damped single degree of freedom (SDOF) mass-spring system with continuously varying natural periods to a given wind excitation.

Before it is possible to determine the relative displacement-time history it is necessary to know the acceleration-time history or the support, the natural frequency n_0 of the system, and the fraction or critical damping β , which is a measure of the structure's energy dissipative qualities.

2.6 Summary

Nonlinear static analysis of tall buildings under wind loads is based on $P\Delta$ analysis. Nonlinear dynamic analysis of tall building may be necessary to determine the accurate response and to ensure comfort of occupants.

In this research a wall frame structure has been used and the model has been analyzed under wind loads statically and dynamically. The results are then compared to evaluate the importance of the dynamic analysis.

Chapter Three

Nonlinear Analysis procedure of Buildings under wind load

3.1 Introduction

In order to design a structure to resist wind loads, the forces on the structure must be specified. The exact forces that will occur during the life of the structure cannot be anticipated. Most National Building Codes identify some factors according to the boundary conditions of each building considered in the analysis to provide for life safety. A realistic estimate for these factors is important; however the cost of construction and therefore the economic viability of the project are essential. Owing to lack of wind forecasting centers the American code (ASCE/SEI 7-10) give more concentration on calculating these lateral loads and the corresponding additional stresses to be taken into account in the design of the structures.

This chapter considers also the particular circumstances under which the designer may need to undertake a study of the dynamic response, and examines briefly the techniques available for the analysis. Finally, consideration is given to the human response to dynamic motions and its effects on structural design.

3.2 ASCE provision for calculating wind loads

The necessary steps to calculate the wind force are in chapters 26, 27, 28 of ASCE/SEI 7-10.

The design wind loads for buildings and other structures, including the Main Wind Force Resisting System (MWFRS) and component and cladding elements shall be determined using one of the following procedures:

(1) Directional Procedure for buildings of all heights.

(2) Envelope Procedure for low-rise buildings.

(3) Directional Procedure for Building appurtenances (roof top structures and rooftop equipment) and Other Structures such as solid freestanding walls and solid freestanding signs, chimneys, tanks, open signs, lattice frameworks, and trussed towers.

(4) Wind Tunnel Procedure for all buildings and all other structures.

The basic parameters for determining wind loads on MWFRS are:

- Basic wind speed, V.
- Wind directionality factor K_d.
- Exposure category.
- Topographic factor, K_{zt}.
- Gust Effect Factor.
- Enclosure classification.
- Internal pressure coefficient, (G_{cpi}).

3.2.1 Basic wind speed (V)

The basic wind speed, V, used in the determination of design wind loads on buildings and other structures.

3.2.2 Wind directionality factor (K_d)

The wind directionality factor, K_d, shall be determined from Table A-1 (Appendix A).

3.2.3 Exposure categories

For each wind direction considered, the upwind exposure shall be based on ground surface roughness that is determined from natural topography, vegetation and constructed facilities.

3.2.4 Topographic factor (K_{zt})

The wind speed-up effect shall be included in the calculation of design wind loads by using the factor K_{zt} which is given by:

$$K_{zt} = (1 + K_1 K_2 K_3)^2 \quad (3.1)$$

Where K_1 , K_2 , and K_3 are given (ASCE/SEI 7-10 clause 26.6) in Table A-2 (Appendix A).

If site conditions and locations of structures do not meet all the conditions specified in Section 26.8.1 from (ASCE/SEI 7-10) then $K_{zt} = 1.0$.

3.2.5 Gust-Effect Factor

To determine whether a building or structure is rigid or flexible, the fundamental natural frequency n_0 shall be established using the structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis. Low-Rise Buildings are permitted to be considered rigid.

A building is considered flexible, Slender if the fundamental natural frequency less than 1 Hz.

A building is considered rigid, if the fundamental natural frequency is greater than or equal to 1 Hz.

For rigid buildings or other structures, the gust-effect factor shall be taken as 0.85 or calculated from Eq.(26.9-6) from the code:

$$\text{In American units:} \quad G = 0.925 \left(\frac{1 + 1.7 g_Q I_z Q}{1 + 1.7 g_v I_z} \right) \quad (3.2)$$

$$\text{In SI units:} \quad I_z = c \left(\frac{33}{\bar{z}} \right)^{1/6} \quad (3.2-a)$$

$$I_z = c \left(\frac{10}{\bar{z}} \right)^{1/6} \quad (3.2-b)$$

Where I_z is the intensity of turbulence at height $\bar{z}$, where $\bar{z}$ is the equivalent height of the structure defined as $0.6h$, but not less than Z_{min} for all building heights h . Z_{min} and c are listed for each exposure in Table A-3 (Appendix A); g_Q is peak factor

for background response and g_v is peak factor for wind response shall be taken as 3.4(from the code). The background response Q is given by:

$$Q = \sqrt{\frac{1}{1 + 0.63 \left(\frac{B+h}{L_z} \right)^{0.63}}} \quad (3.3)$$

Where B and h are dimensions of the building normal to wind direction, and height respectively and L_z is the integral length scale of turbulence at the equivalent height given by:

$$L_z = \ell \left(\frac{\bar{z}}{33} \right)^{\bar{e}} \quad (3.3-a)$$

$$\text{In SI: } L_z = \ell \left(\frac{\bar{z}}{10} \right)^{\bar{e}} \quad (3.3-b)$$

In which ℓ and $\bar{e}$ are constants listed in Table A-3(Appendix A).

For flexible or dynamically sensitive buildings or other structures as defined previously, the gust-effect factor shall be calculated by:

$$G_f = 0.925 \left(\frac{1 + 1.7 I_z \sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_z} \right) \quad (3.4)$$

And g_R is given by:

$$g_R = \sqrt{2 \ln(3,600 n_1)} + \frac{0.577}{\sqrt{2 \ln(3,600 n_1)}} \quad (3.4-a)$$

And R , the resonant response factor, is given by:

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)} \quad (3.4-b)$$

In which:

$$R_n = \frac{7.47N_1}{(1+10.3N_1)^{5/3}}$$

And: (3.4-b.1)

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_z}$$

And: (3.4-b.1.1)

$$R_\ell = \frac{1}{\eta} - \frac{1}{2\eta^2}(1 - e^{-2\eta}) \quad \text{for } \eta > 0$$

(3.4-b.2)

$$R_\ell = 1 \quad \text{for } \eta = 0$$

Where the subscript ℓ shall be taken as h, B, and L respectively. Where h, B, and L defined in section 26.3 from ASCE/SEI 7-10.

$\bar{V}_z$ = mean hourly wind speed (ft/s) at height z is determined from:

In American units:

$$\bar{V}_z = \bar{b} \left(\frac{\bar{z}}{33} \right)^{\bar{\alpha}} \left(\frac{88}{60} \right) V$$

In SI units: (3.5-a)

$$\bar{V}_z = \bar{b} \left(\frac{\bar{z}}{10} \right)^{\bar{\alpha}} V$$

(3.5-b)

Where $\bar{b}$ and $\bar{\alpha}$ are constants and V is the basic wind speed in mi/h.

3.2.6 Enclosure Classification

For the purpose of determining internal pressure coefficients, all buildings shall be classified as enclosed, partially enclosed, or open. A determination shall be made of the amount of openings in the building envelope for use in determining the enclosure classification.

3.2.7 Internal Pressure Coefficients

Internal pressure coefficients, (GC_{pi}), shall be determined from Table A-4(Appendix A) based on building enclosure classifications.

The steps to Determine MWFRS Wind Loads for Buildings of All Heights are:

Step 1: Determine risk category of the building.

Step 2: Determine the basic wind speed, V, for the applicable risk category.

Step 3: Determine wind load parameters:

- Wind directionality factor, K_d.
- Exposure category.
- Topographic factor, K_{zt}
- Gust Effect Factor G.
- Enclosure classification.
- Internal pressure coefficient, (GC_{pi}).

Step 4: Determine velocity pressure exposure coefficient, K_z or K_h

Step 5: Determine velocity pressure q_z or q_h.

Step 6: Determine external pressure coefficient, C_p or C_N

Step 7: Calculate wind pressure, p, on each building surface.

3.2.8 Velocity Pressure

Based on the exposure category determined, a velocity pressure exposure coefficient K_z or K_h , as applicable, shall be determined from Table A-5(Appendix A).

The numerical coefficient 0.00256 (0.613 in SI units) shall be used except where sufficient climatic data are available to justify the selection of a different value of this coefficient for a design application.

Velocity pressure, q_z evaluated at height z shall be calculated by the following equation:

$$q_z = 0.00256 K_z K_{zt} K_d V^2 \text{ (lb/ft}^2\text{)} \quad (3.6-a)$$

$$[\text{In SI: } q_z = 0.613K_zK_{zt}K_dV^2 \text{ (N/m}^2\text{); } V \text{ in m/s}] \quad (3.6-b)$$

The external pressure coefficient C_p should be determined from Table A.6.1, A.6.2, and A.6.3 (Appendix A).

3.2.9 WIND Pressure

Design wind pressures for the MWFRS of buildings of all heights shall be determined for enclosed and partially enclosed rigid buildings by the following equation:

$$p = qGC_p - q_i(GC_{pi}) \text{ (lb/ft}^2\text{) (N/m}^2\text{)} \quad (3.7)$$

q and q_i shall be evaluated using exposure defined previously. Pressure shall be applied simultaneously on windward and leeward walls and on roof surfaces.

Design wind pressures for the MWFRS of flexible buildings shall be determined for enclosed and partially enclosed flexible buildings from the following equation:

$$p = qG_fC_p - q_i(GC_{pi}) \text{ (lb/ft}^2\text{) (N/m}^2\text{)} \quad (3.8)$$

Where q , q_i , C_p , and (GC_{pi}) , G_f (gust-effect factor) are as determined previously.

3.3 Load Combinations according to ACI318-08

Required strength U shall be at least equal to the effects of factored loads in Eq. (3.9) through (3.15).

$$U = 1.4(D + F) \quad (3.9)$$

$$U = 1.2(D + F + T) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R) \quad (3.10)$$

$$U = 1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (1.0L \text{ or } 0.8W) \quad (3.11)$$

$$U = 1.2D + 1.6W + 1.0L + 0.5(L_r \text{ or } S \text{ or } R) \quad (3.12)$$

$$U = 1.2D + 1.0E + 1.0L + 0.2S \quad (3.13)$$

$$U = 0.9D + 1.6W + 1.6H \quad (3.14)$$

$$U = 0.9D + 1.0E + 1.6H \quad (3.15)$$

Where:

D = dead load

E = earthquake load

F = load due to fluids with well-defined pressures and maximum heights

H = load due to lateral earth pressure, ground water pressure, or pressure of bulk materials

L = live load

L_r = roof live load

R = rain load

S = snow load

T = self-straining force

W = wind load

3.4 Static Analysis

Liu and Quek, 2003, stated that, the static system of equations takes the form

$$KD=F \quad (3.16)$$

There are numerous methods and algorithms to solve the above matrix equation. The methods often used are Gauss elimination for small systems, and iterative methods for large systems. These methods are all routinely available in any math library of any computer system.

3.5 Dynamic Analysis

3.5.1 Types of Dynamic loads

Maguire and Wayatt, 2000 stated that dynamic loading can be classified into a number of types depending on the nature of its variation with time as follows (Fig. 3.1):

- (a) Periodic or harmonic (load amplitude repeats itself regularly many times) e.g. machinery loading.
- (b) Transient (load varies with time but does not repeat itself continuously) e.g. blast loading.

- (c) Stationary random (load not known precisely but statistical properties vary only very slowly) e.g. wind loading.
- (d) Non-stationary random (as for stationary but varying statistical properties) e.g. earthquake loading.

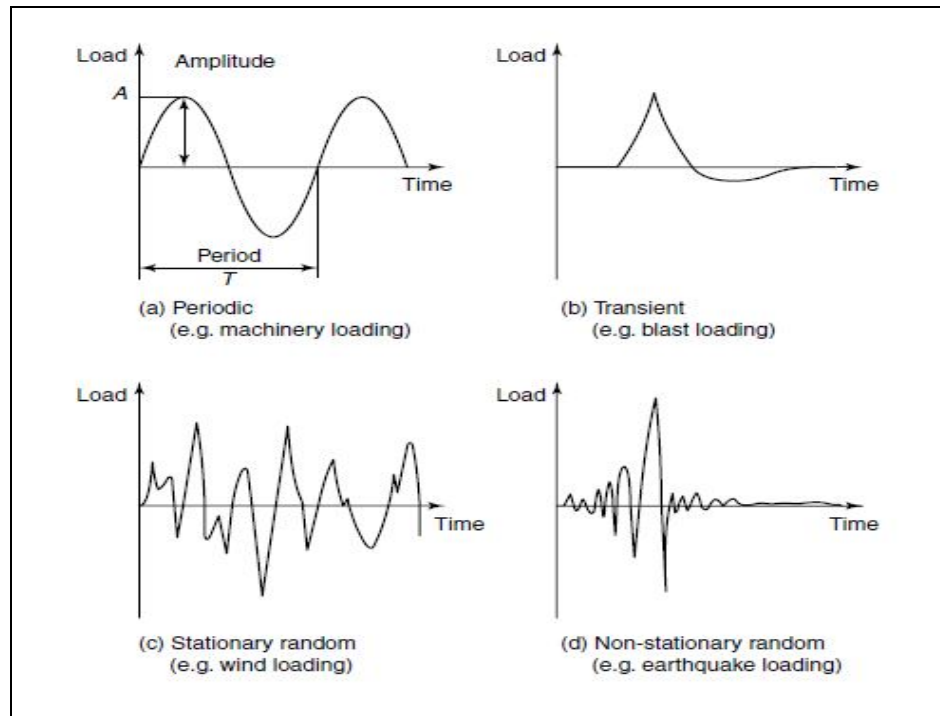


Fig. (3.1): Examples of dynamic loading (time-domain representation) [Maguire and Wayatt, 2000]

3.5.2 Equation of motion

As stated by Maguire and Wayatt 2000, the equation of motion defining the dynamic behavior of the structure is the equation of equilibrium between the inertia force, damping force and stiffness force together with the externally applied force. This is of the form:

Inertia force + damping force + stiffness force = external force

Or, in algebraic form:

$$M\ddot{u} + C\dot{u} + Ku = f(t) \quad (3.17)$$

The mass $\mathbf{M}$ is often simply lumped at the nodes, giving a diagonal mass matrix. A more Complex procedure which significantly reduces the need for additional nodes introduces the so-called consistent mass matrix.

If damping $\mathbf{C}$ is excluded and no external forces $\mathbf{f}(t)$ are applied then the homogeneous undamped equation of motion is:

$$\mathbf{M}\ddot{\mathbf{u}} + \mathbf{C}\dot{\mathbf{u}} = \mathbf{0} \quad (3.18)$$

This has a solution in the form of simple harmonic (periodic) motion:

$$\mathbf{u} = \tilde{\mathbf{u}} \sin \bar{\omega} t \quad \text{and} \quad \ddot{\mathbf{u}} = -\bar{\omega}^2 \sin \bar{\omega} t \quad (3.19)$$

Substituting these into the equation of motion gives:

$$\mathbf{K}\tilde{\mathbf{u}}_0 = \bar{\omega}^2 \mathbf{M}\tilde{\mathbf{u}}_0 \quad (3.20)$$

This is known as the eigenvalue problem, where $\bar{\omega}^2$ the eigenvalue and $\mathbf{u}_0$ is the eigenvector. $\bar{\omega}$ is the natural frequency in radians per second so that the eigenvalue is the square of the natural frequency. $n = \frac{\bar{\omega}}{2\pi}$ Is the corresponding natural frequency in cycles per second (Hz).

3.5.3 Mode of Vibration

Maguire and Wayatt, 2000 stated that, if the system has r degrees of freedom (DOF), equation (3.20) will have r solutions, representing the normal modes of vibration of the system. In each mode, a free vibration consists of harmonic motion of all points in phase, at the frequency n_i (or the circular frequency $\bar{\omega}_i$), in which $\mathbf{y}_i = y_i \Phi_i \sin \bar{\omega}_i t$, in which y_i is the modal generalized amplitude and the eigenvector Φ_i (comprising elements Φ_{ij} for each DOF j) is the mode shape vector Figure (3.2).

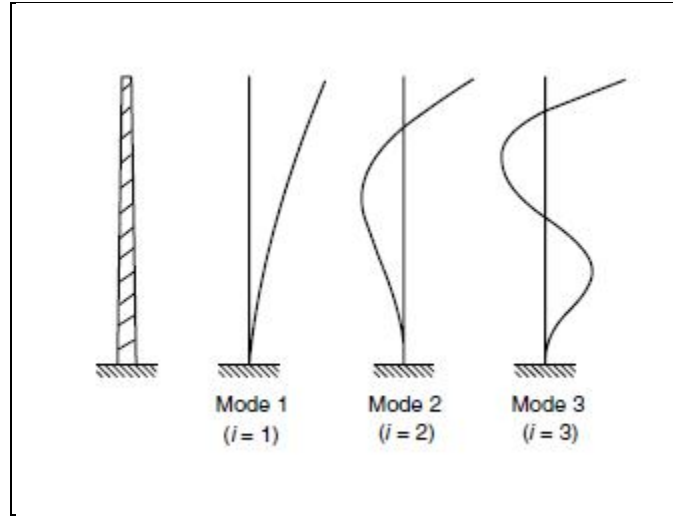


Fig. (3.2): The first three modes of cantilever beam [Maguire and Wayatt, 2000].

3.5.4 Response spectrum analysis

Smith and Coull, stated that for any input acceleration $\ddot{u}$, the solution will yield the maximum absolute value or relative displacement u termed the spectral displacement S_d which will be a function of the natural frequency (or period) and damping factor.

The maximum pseudo relative velocity S_v and maximum absolute pseudo acceleration S_a are then given by

$$\begin{aligned} S_v &= \omega * S_d \\ S_a &= \omega^2 * S_d \end{aligned} \quad (3.21)$$

The pseudo acceleration is identical to the maximum acceleration when there is no damping, which, for normal levels of structural damping, is practically the same as the maximum acceleration.

If the natural frequency of a structure is calculated and the degree of damping present is estimated, other corresponding important design parameters such as the maximum displacement and maximum acceleration can be obtained directly from the response spectrum diagram.

For a building with a known frequency of oscillation ω , and an estimated damping ratio β , a response spectrum diagram can be used with the modal method of analysis, to determine the peak response of the structure.

3.5.5 Modal analysis procedure

In general, the set of governing dynamic equations of motion [Eq. (3.17)] must be solved simultaneously by available computational procedures to determine all displacements u that define the motions of the structure. This approach can be avoided by using the computationally more efficient modal method of analysis. The method which is based on linear elastic structural behavior employs the superposition of a limited number of modal peak responses, as determined from a prescribed response spectrum and with appropriate modal combination rules it will yield results that compare closely with those from a time-history analysis.

This method of analysis is based on the fact that for certain forms of damping that are reasonable approximations for many buildings, the equations of motion can be uncoupled so that the response in each natural mode of vibration can be calculated independently of the others Eq. (3.19). Each mode will respond with its own particular displacement profile, the mode of vibration $\ell \omega_n$, its own frequency the natural frequency of vibration ω_n and with its own modal damping, the damping ratio β_n .

In the structural idealization, the mass is usually lumped at the floor levels. Only one degree of freedom per floor, the horizontal deflection for which the structure is being analyzed, is used, and so the matrices involved are of the same order as the number of stories N in the building.

The contribution of the n^{th} mode to the modal displacement $u_j(t)$ at the j^{th} floor is then equal to the product of the amplitude generalized coordinate and the mode shape,

$$u_{jn}(t) = u_n(t) l_{jn} \quad (3.22)$$

To determine the dynamic storey shears and moments, it is convenient to introduce the concept of equivalent lateral forces, defined as the static external forces p that would produce the same structural displacements u . Hence, at any time t , the equivalent forces corresponding to the modal displacements $u_n(t)$ will be, from Eq. (3.20) as:

$$p_n(t) = K u_n(t) = K \ell_n u_n(t) = \omega_n^2 M \ell_n u_n(t) \quad (3.23)$$

The equivalent lateral force at the j^{th} floor level is then.

$$P_{jn}(t) = \omega_n^2 m_j l_{jn} u_n(t) \quad (3.24)$$

The contributions from each mode may then be summed to give the total equivalent force P at each floor level. For example, at level j .

$$P_j = \sum_{n=1}^N P_{jn}(t) \quad (3.25)$$

The internal dynamic shears and moments at any level can then be obtained by summing all the storey forces and the moments of these forces above the level concerned.

In a similar manner, the displacement at any level may be obtained by combining the responses from each mode at that position. The drift U_N at the top of the building is then.

$$U_N = \sum_{n=1}^N U_{Nn} \quad (3.26)$$

And the inter storey drift is given by the difference between the total displacements of the floors above and below the level concerned.

3.5.6 Dynamic Structural Response due to wind forces

Smith and Coull, 1991 stated that the prediction of the structural response involves two stages: (1) the prediction of the occurrence of various mean wind speeds and their associated directions, and (2) given the occurrence of that wind, the prediction of the maximum dynamic response of the structure. The former requires an assessment of the wind climate of the region, adjusted to take account of the local topography of the site, and of the local wind characteristics (mean velocity profile and turbulence structure). The steady pressures and forces due to the mean wind, and the fluctuating pressures on the exterior, may then be determined. The properties of the mean wind can be conveniently expressed only in statistical terms.

The peak response value can be expressed statistically in terms of the number of standard deviations by which the peak exceeds the mean value. For design purposes, the conventional practice is to define the peak value of the variable, $x(\max)$ say, by the relationship.

$$x(\max) = \bar{x} + g_p \delta \quad (3.27)$$

Where $x(\max)$, $\bar{x}$, and δ are the peak, mean, and standard deviations, respectively, of the variable x concerned, referred to a record period of 1 hour, and g_p is the peak factor. When considering the response of a tall building to wind actions, both along-wind and cross-wind motions must be considered.

3.5.6.1 Along-Wind Response

Smith and Coull, 1991 stated that the along-wind response of most structures is due almost entirely to the action of the incident turbulence of the longitudinal component

of the wind velocity, superimposed on a mean displacement due to the mean drag. The resulting analytical methods, using spectral correlation considerations to predict the structural response, have been developed to such a level that they are now employed in modern design wind Codes. The work has led to the development of the gust factor method for the prediction of the building response. Fig. (3.3) shows the along and across wind direction.

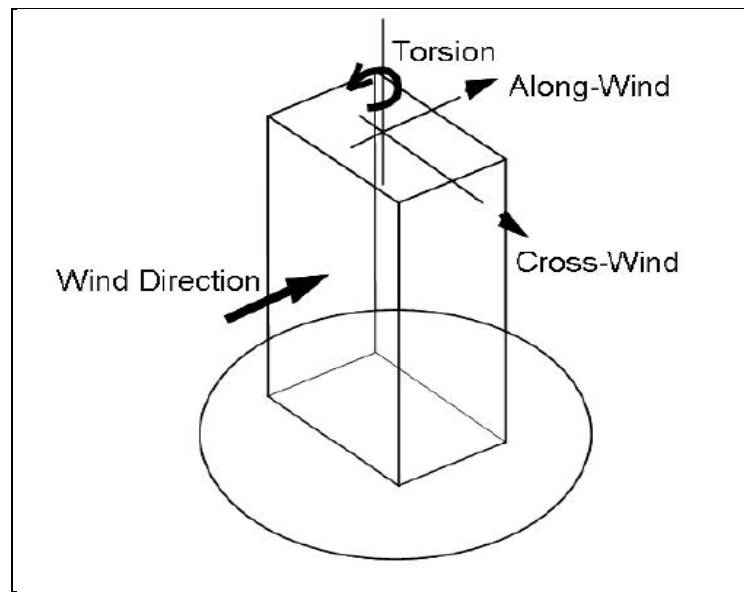


Fig. (3.3): Along and cross wind direction (Mosa, 2015)

The gust factor method is based on the assumption that the fundamental mode of vibration of a structure has an approximately linear mode shape. In essence, the aim of the method is to determine a gust factor G that relates the peak to mean response in terms of an equivalent static design load or load effect $\bar{Q}$, such that,

$$\text{Design value, } Q(\text{max}) = G \bar{Q} \quad (3.28)$$

Where $\bar{Q}$ defines the mean value of the quantity concerned.

For example, if the mean pressures acting on the face of a tall building are summed to give the mean base overturning moment $\bar{M}$, the design dynamic base overturning moment $M(\text{max})$ will be obtained by multiplying $\bar{M}$ by the gust factor G .

$$M(\max) = G\bar{M} \quad (3.29)$$

Smith and Coull, 1991 carried out Schematic representation of Davenport's design procedure as shown in Fig. (3.4). The procedure is a combination of two parts, the first involves the modeling of the wind forces, and the second involves the use of structural dynamic analysis to determine the resulting response. In the diagram, the force spectrum is found, from the wind velocity spectrum, represented by an algebraic expression based on observations of the real wind, through the aerodynamic admittance, which relates the size of the gust disturbance to the size of the structure. The aerodynamic admittance may be determined theoretically, or measured experimentally in a wind tunnel. To find the response of the structure in this mode to the force spectrum, it is necessary to know the mechanical admittance, which is a function of the natural frequency, the damping, and the stiffness of the structure.

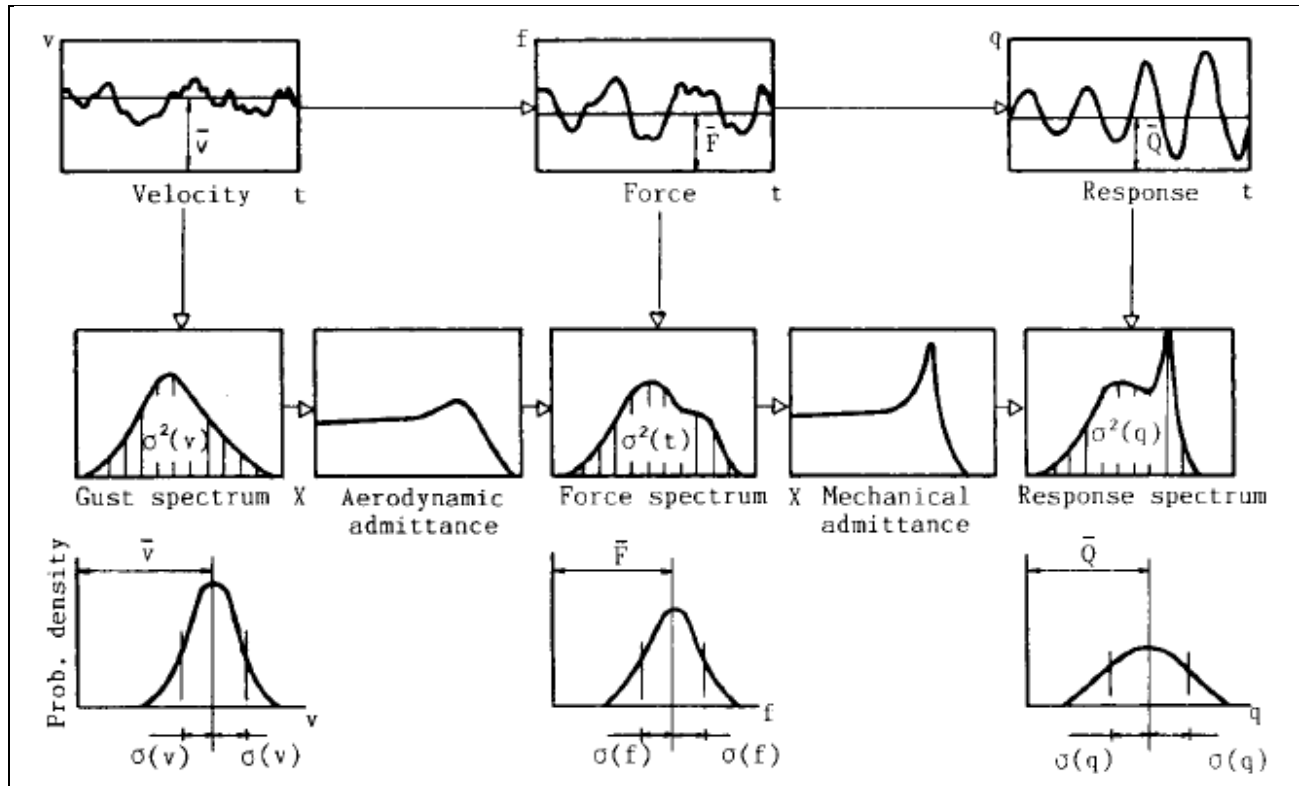


Fig. (3.4): Schematic representation of Davenport's design procedure [Smith and Coull, 1991].

The mechanical admittance has a sharp resonance peak at the natural frequency, similar in form to the dynamic magnification curve found in the response of dynamic systems. As a result of the peak in the mechanical admittance function, the response has a peak at the natural frequency, the amplitude of which is determined by the damping present. Smith and Coull, 1991 stated that for the orders of damping found in most buildings, this peak usually contains most of the area in the response spectrum. For this reason most of the fluctuations take place at or near the natural frequency. The area under the loading effect spectrum is taken as the sum of two components. The area under the broad hump of the diagram, which must be integrated numerically for each structure, and the area under the resonance peak, for which a single analytic expression is available. These backgrounds and resonant excitation components are represented in Eq. (3.30) by B and R, respectively, combined to give the peak response.

In Davenport's analysis, the response of a tall slender building to a randomly fluctuating wind force is determined by treating it as a rigid spring-mounted cantilever whose dynamic properties are specified by the fundamental natural frequency n_0 and an appropriate damping ratio. Consequently, only a single linear mode of vibration needs to be considered.

The gust factor can be regarded as a relationship between the wind gusts and the magnification due to the structural dynamic properties. As such, it will depend on the properties of the structure (height H, and breadth/height ratio (W / H), fundamental natural frequency n_0 and critical damping ratio, the mean design wind' speed V, and the particular location of the building.

It may be shown that the gust factor G may be expressed as:

$$G = 1 + g_p r (B + R)^{\frac{1}{2}} \quad (3.30)$$

In Eq. (3.30), g_p is a peak factor that accounts for the time history of the excitation, and is determined from the duration time T over which the mean velocity is averaged

and the fundamental frequency of vibration n_0 : in practice, T is taken as 3600 sec (1 hour). r is a roughness factor, which depends on the location and height of the building (Fig.. 3.5); B is the excitation due to the background turbulence or gust energy, which depends on the height and aspect ratio of the building (Fig.. 3.6); and R is the excitation by the turbulence resonant with the structure, which depends on the size effect S , the gust energy ratio F at the natural frequency of the structure, and the

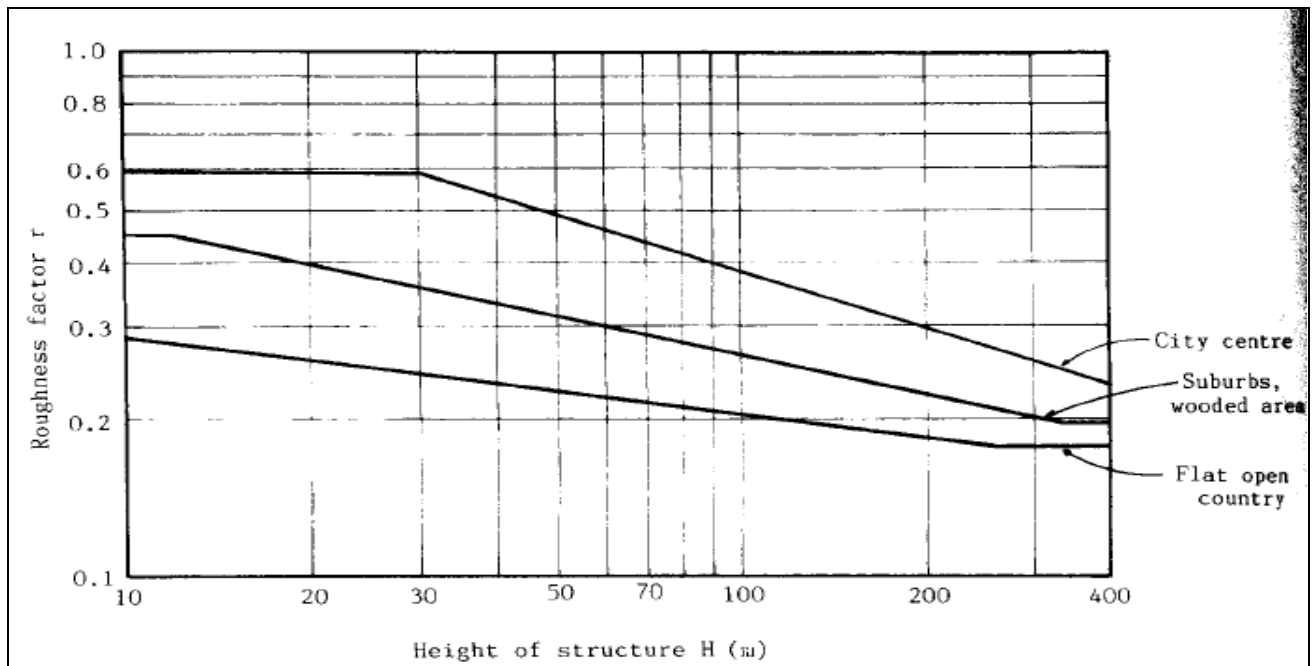


Fig. (3.5): Variation of roughness factor with building height [Smith and Coull,1991]

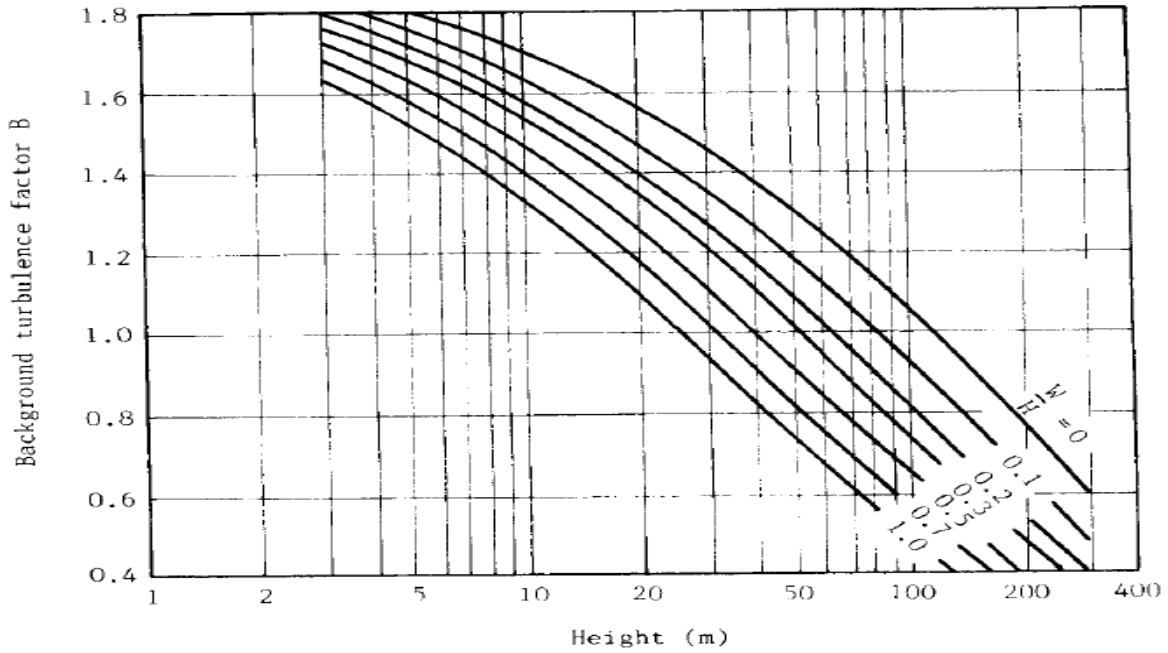


Fig. (3.6): Variation of background turbulence factor with height and aspect. [Smith and Coull, 1991]

Critical damping ratio β , that is.

$$R = \frac{SF}{\beta} \quad (3.31)$$

The size reduction factor S depends on the aspect ratio W / H , the natural frequency n_0 , and the mean wind velocity at the top of the structure, V_H , as shown in Fig.. (3.7). The gust energy ratio F is a function of the inverse wavelength, n_0/V_H as shown in Fig.. (3.8). If resonant effects are small, then R will be small compared to the background Turbulence B , and vice versa.

The peak factor g_p in Eq. (3.30) gives the number of standard deviations by which the peak load effect is expected to exceed the mean load effect, and is shown in Fig.. (3.9) as a function of the average fluctuation rate v given by:

$$v = \frac{n_0}{\sqrt{(1 + \frac{B}{R})}} \quad (3.32)$$

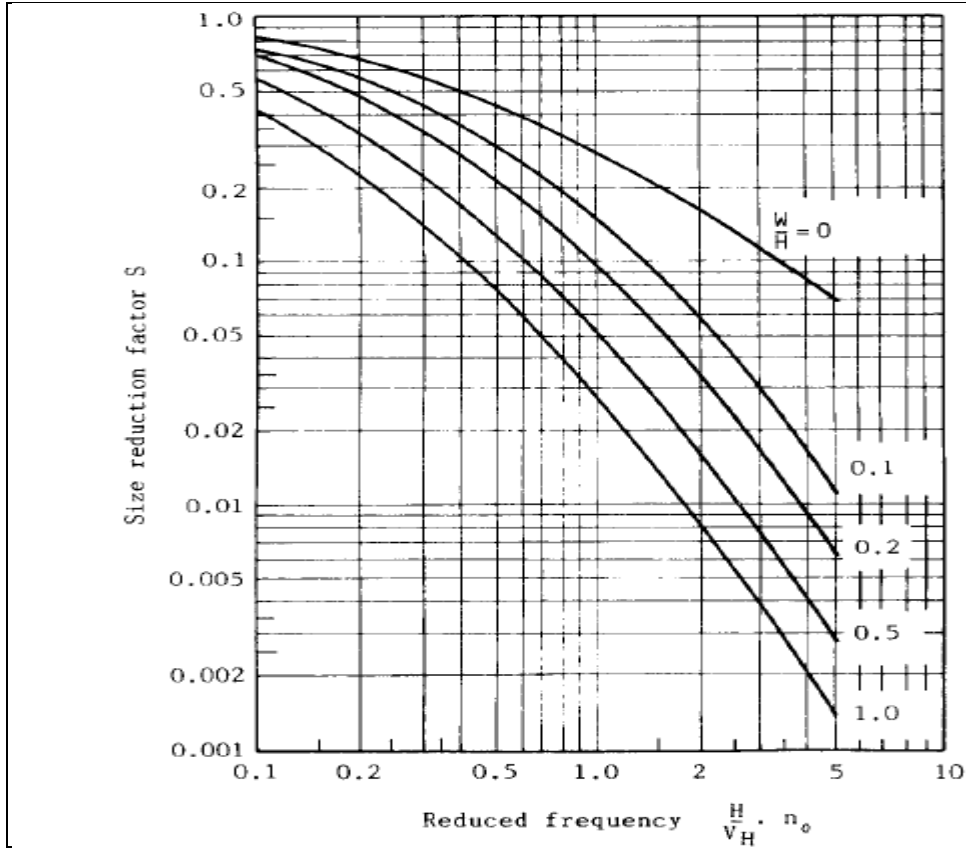


Fig. (3.7): Variation of size reduction factor with reduced frequency and aspect ratio of building. [Smith and Coull, 1991]

In the above formulas, the variables V_H, n_0 and β must relate to the along-wind direction. Substitution of the known values of g_p , r , B , and R into Eq. (3.30) then produces the desired value of the gust factor. Once the gust factor G has been determined, the peak dynamic forces and displacements may be determined by multiplying the values due to the mean wind loading by G .

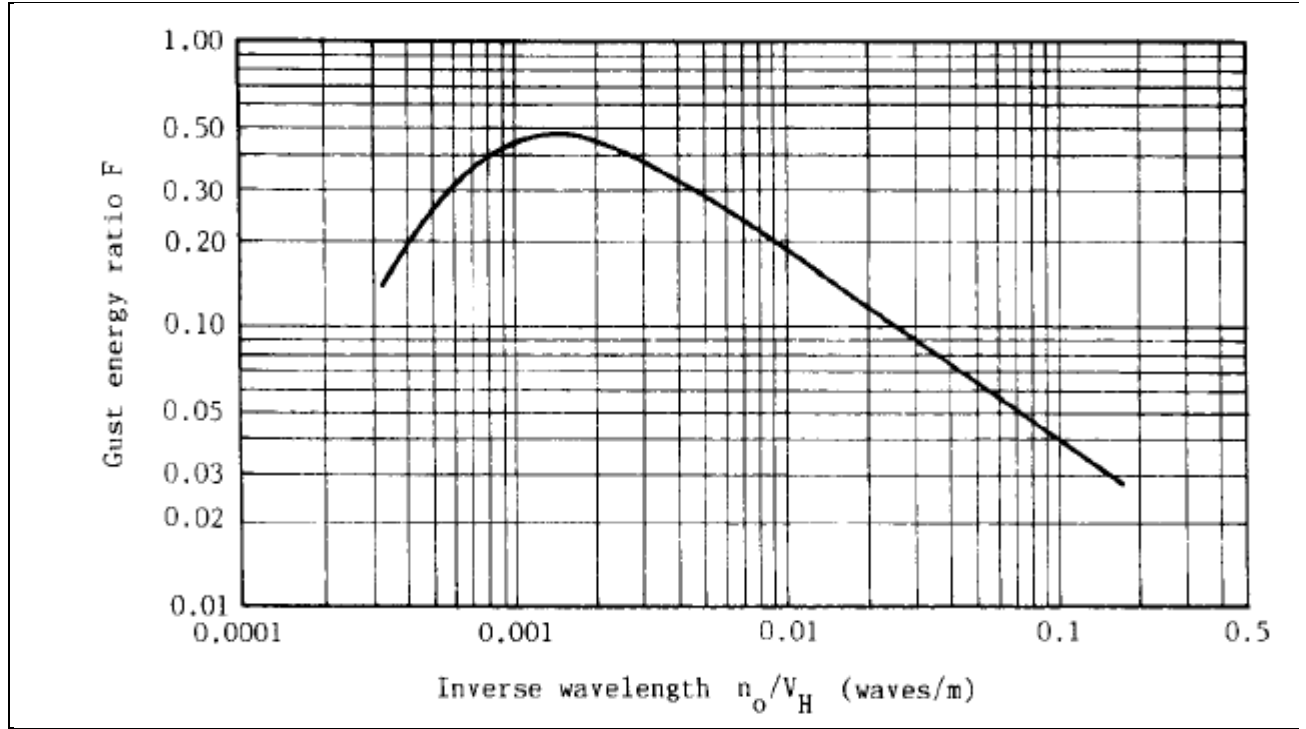


Fig. (3.8): variation of gust energy ratio with inverse wavelength [Smith and Coull, 1991].

Peak along - wind Accelerations. The most important criterion for the comfort of the building's occupants is the peak acceleration they experience. It is thus important to be able to estimate at an early stage in design the likely maximum accelerations in both the along-wind and across-wind directions.

The maximum acceleration a_D in the along-wind direction may be estimated as:

$$a_D = \pi^2 n_0^2 g_p r \sqrt{R} \left(\frac{\Delta}{G} \right) \quad (3.33)$$

Where Δ = the maximum wind-induced deflection at the top of the building in the along- wind direction (m).

The natural frequency n_0 and β damping ratio must be again in the along-wind Direction.

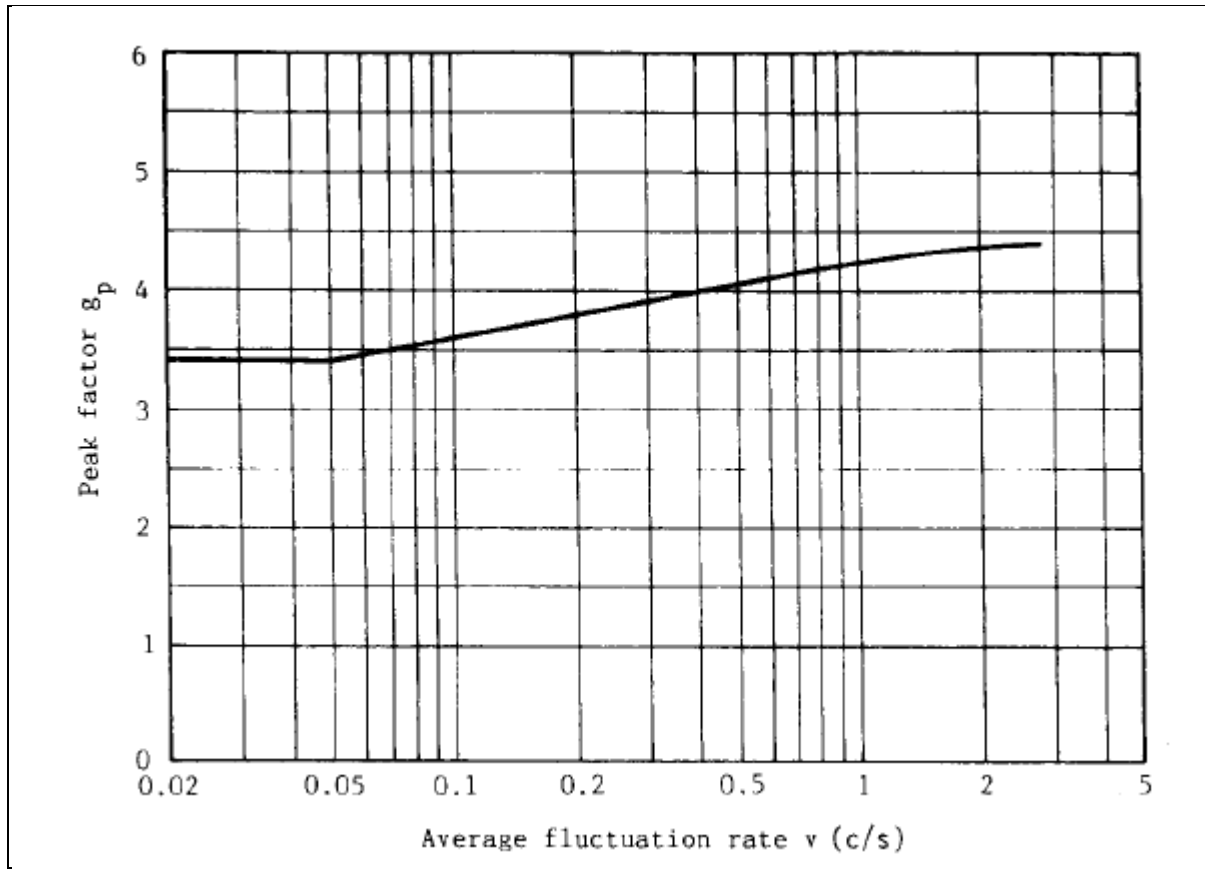


Fig. (3.9): Variation of peak factor with average fluctuating rate [Smith and Coull, 1991]

3.5.6.2 Across-Wind Response

Smith and Coull, 1991 stated that, the cross-wind excitation of tall buildings is due predominantly to vortex shedding. However; generalized empirical methods of predicting the response have been difficult to derive, even assuming that the motions are due entirely to wake excitation, because of the effects of building geometry and density, structural damping, turbulence, operating reduced frequency range, and interference from upstream buildings. The last effect can alter significantly the across-wind motions. Consequently, the most accurate method of determining the across-wind structural response has been from tests on an aero elastic model in a wind tunnel. The maximum lateral wind loading and deflection are in the along-wind direction, the maximum acceleration of the building may often occur in the across-wind direction. Across-wind accelerations are likely to exceed along-wind accelerations if the

building is slender about both axes, such that the geometric ratio $\sqrt{\frac{WD}{H}}$ is less than one-third, where D is the along-wind plan dimension. Based on a wide range of turbulent boundary layer wind tunnel studies, a tentative formula for the peak acceleration a_w at the top of the building, namely,

$$a_w = n_0^2 g_p [WD]^{\frac{1}{2}} \left(\frac{a_r}{\rho g \sqrt{\beta}} \right) \quad (3.34)$$

ρ = average density of the building (kg/m^3).

$$a_r = 78.5 \times 10^{-3} \left[\frac{V_H}{n_0 \sqrt{WD}} \right]^{3.3} \text{ (Pa)}.$$

g = acceleration due to gravity (m/sec^2).

3.6 P-delta effect

Taranath, 2005 stated that when flexible structures are subjected to lateral forces the resulting horizontal displacement lead to additional overturning moments because the gravity load is also displaced .Thus the total base moment is:

$$M_{ub} = V_u H + P_u \Delta \quad (3.35)$$

Therefore in addition to the overturning moment produced by lateral force V_u the secondary moment $P_u \Delta$ must also be resisted. This moment increment in turn will produce additional lateral displacement and hence Δ will increase further .In very flexible structures, instability, resulting in collapse, may occur as shown in Fig. (3.10).

P-Delta effect typically involves large external forces upon relatively small displacements. If deformations become sufficiently large as to break from linear compatibility relationships, then Large-displacement and Large-deformation analysis become necessary. The two sources of P-Delta effect are described as follows:

- **P-Delta** ($P-\Delta$), is associated with local deformation relative to the element chord between end nodes. Typically, $P-\Delta$ only becomes significant at unreasonably large

displacement values, or in especially slender columns, it is not advisable to model $P-\Delta$, since it may significantly increase computational time without providing the benefit of useful information. An easier way to capture this behavior is to subdivide critical elements into multiple segments, transferring behavior into $P-\Delta$ effect.

- **P-Delta plus large displacement ($P-\Delta-w$)**, is associated with displacements relative to member ends. Unlike $P-\Delta$, this type of P -Delta effect is critical to nonlinear modeling and analysis. The gravity loading will influence structural response under significant lateral displacement. $P-\Delta$ may contribute to loss of lateral resistance, residual deformations, and dynamic instability. To consider $P-\Delta$ effect directly, gravity load should be present during nonlinear analysis. Application will cause minimal increase to computational time, and will remain accurate for drift levels up to 10%.

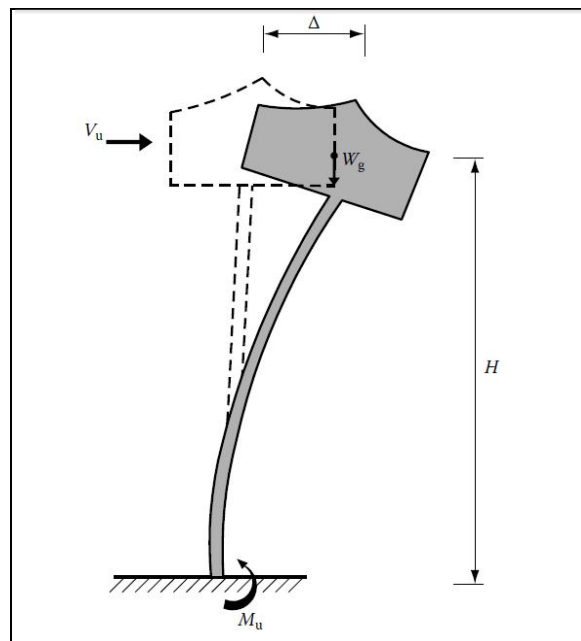


Fig. (3.10): P-delta effect (simple cantilever model) [Taranath, 2005]

As stated by ACI 318-08, Columns and stories in structures shall be designated as non-sway or sway columns or stories. It shall be permitted to assume a column in a

structure is non-sway if the increase in column end moments due to second-order effects does not exceed 5 percent of the first-order end moments.

In ACI 318-08, section 6.6.4.4.1 permitted to assume a storey with in a structure is non-sway if:

$$Q = \frac{\Sigma P_u \Delta_0}{V_{us} l_c} \leq 0.05 \quad (3.36)$$

The P-delta analysis should be used if $Q \leq 0.05$, where:

ΣP_u : Total storey factored vertical load.

V_{us} : Horizontal storey shear.

Δ_0 : First order relative lateral deflection between top and bottom that storey due to V_{us} .

l_c : Height of the storey.

3.7 Analysis using computer program

There are many programs that are used to analyze and design buildings, in this research analysis and design has been carried out using ETABS.

3.7.1 General

The special features of the ETABS program greatly reduce the amount of input required. This includes the definition of beams and columns as a simple grid system rather than a complex matrix of nodes and elements. The inherent assumption of rigid floor system in ETABS makes it ideal for defining floor systems in high rise buildings.

3.7.2 ETABS Features

ETABS is special purpose computer program for the linear and non-linear, static and dynamic analysis of buildings. ETABS offers a comprehensive 3-D analysis and design for multi storey building structures. A complete suite of windows graphical

tools and utilities are included with the base package, a modeler and a post –processor for viewing all results, including mode shapes, force diagram and deflected shapes. The ETABS buildings may be un-symmetrical and non – rectangular in plan. The program considers a building system as an assemblage of vertical frames interconnected at each storey level by horizontal floor diaphragms. The vertical frames are idealized as an assemblage of column, beam, brace and wall elements interconnected by horizontal floor diaphragm slabs which may be rigid or flexible in their own plane.

3.7.3 Basic Process

The following provides a broad overview of the basic modeling, analysis, and design processes: (Appendix B)

1. Set the units.
2. Open a file.
3. Set up grid lines.
4. Define storey levels.
5. Define member properties.
6. Draw structural objects.
7. Assign properties.
8. Define load cases.
9. Assign loads.
10. Edit the model geometry.
11. View the model.

12. Analyze the model.
13. Display results for checking.
14. Design the model.
15. Generate output.
16. Save the model.

3.7.4 Non Linear Static Procedures

From define menu the basic process are as follows:

- a- Add load combination → Convert Combos to nonlinear case.
- b- Load cases... → Modify / show case → 1-load case type → nonlinear static → 2- Geometric nonlinearity options → P-Delta, P-Delta plus large displacements.

3.7.5 Non Linear Dynamic Procedures

- a- Functions → Response spectrum → Add new function → Convert to user defined.
- b- Mass source... → Specified load pattern (Dead).
- c- Load cases... → Modify / show case → load case type → Response spectrum → Add load applied.
- d- Modal cases... → Modify / show case → 1- use preset P-Delta settings → Non iterative based on mass.
2- Set minimum and maximum number of modes.

Chapter Four

Analysis Results and Discussion

4.1 Introduction

A concrete frame of 20 stories as shown in Fig. (4.1) under wind loading has been analyzed using finite element program ETABS in linear, static non linear (P-Delta and P-Delta plus large displacement) , and dynamic cases. And the building code requirements for structural concrete is ACI318-08. The building was idealized using a finite element model .On entering the data in section (4.2) the displacements in y-y direction were obtained for the linear and static nonlinear cases. Also, the shear forces were obtained. These results were then analyzed, discussed and compared with published results as shown in the following sections.

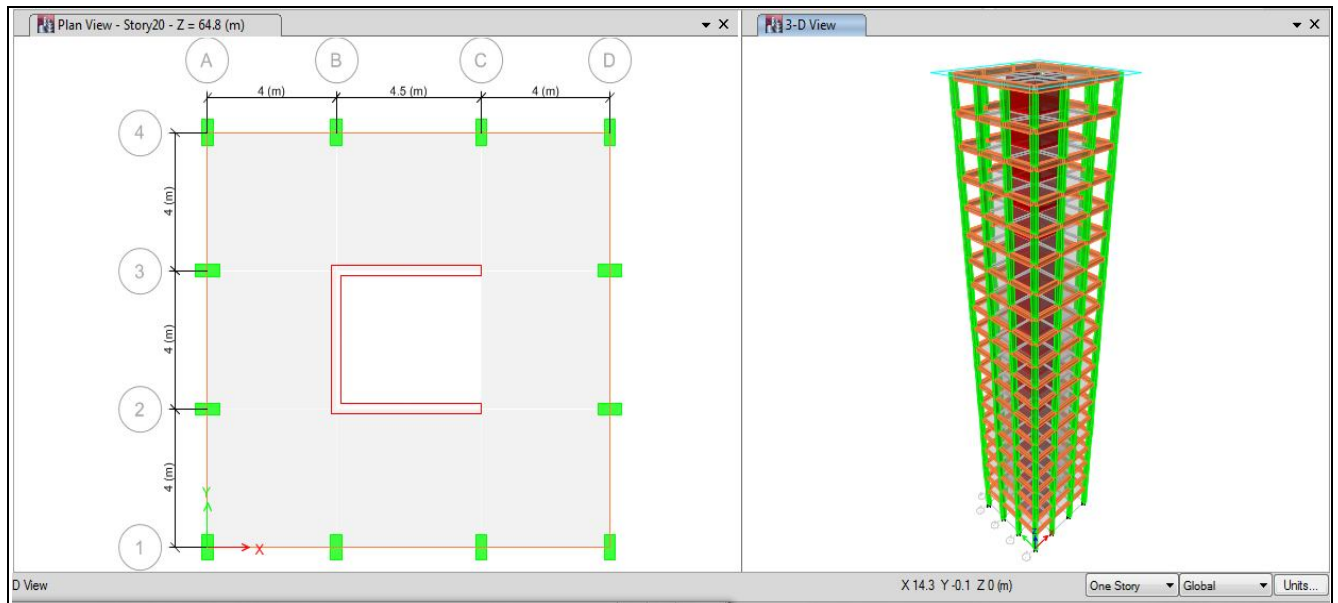


Fig. (4.1a): Plan View of model

Fig. (4.1b): 3D View of model

4.2 The finite element Model properties

The plan of the model building is shown in Fig. (4.1) 12m *12.5m and is of a height of 64.8m, it was idealized by 336 nodes and 560 elements.

4.2.1 The Materials Properties

The materials properties are as follows:

Characteristic concrete strength ($f_c' = 25 \text{ N/mm}^2$)

Characteristic reinforcement strength (labeled Y, high yields), $F_y = 460 \text{ N/mm}^2$

Characteristic reinforcement strength (labeled R, mild steel), $F_{yv} = 250 \text{ N/mm}^2$

Unit weight of reinforced Concrete = 24 kN/ m^3 .

4.2.2 Geometric Properties

The floor slab thickness = 220 mm.

The shear wall thickness=300mm.

The columns section = 750mm x 350 mm.

The beam section = 600mmx300 mm.

The floor height = 3.20 m.

Basement height= 4.00 m.

4.2.3 The Building Loads

The loads per unit floor area are taken as follows:

Dead load D.L = Self weight of slab (not included) + finishes (1.5 kN/ m^2) + Brick works partitions (4.5 kN/ m^2) = 6 kN/ m^2

Live load L.L = 2.5 kN/ m².

4.2.4 Wind loads parameters

- Exposure From extents of Diaphragms (Appendix A)
- Exposure Category = B
- Importance factor = 1.
- Topographical factor $K_{zt} = 1$.
- Gust factor = 0.85.
- Directionality factor $K_d = 0.85$
- Wind Direction = 0° for wind x, 90° for wind y.
- Basic Wind Speed $V = 100\text{mph}$.
- Wind ward coefficient C_q , wind = 0.8.
- Lee ward coefficient C_q , lee = 0.5.
- Top Storey = Storey20.
- Bottom Storey = Base.
- Parapet Height = 1.5

4.2.5 Load Combinations According To ASCE 7-05

In this research the following two load combinations have been used for the linear and nonlinear static and dynamic analysis.

$$U1 = 1.2 \text{ D.L} + 1.0 \text{ L.L} + 1.6 \text{ W.L}$$

$$U2 = 0.9 \text{ D.L} + 1.6 \text{ W.L}$$

Where W.L is wind load

4.3 Linear and nonlinear static analysis

4.3.1 Lateral loads from the program used

The lateral loads applied to stories in y-direction were given in Table (4.1) and Figure (4.2). The maximum lateral load in storey 19 is equal to (55.95 kN). Also the lateral loads applied to storey in x-direction are given in Table (4.2) and Figure (4.3). The maximum lateral load in storey 19 is equal to (53.37kN).

Table (4.1): lateral load to storey in y-direction:

Storey	Storey height (m)	Load in Y-direction kN
Storey 20	64.8	28.14
Storey 19	61.6	55.95
Storey 18	58.4	55.43
Storey 17	55.2	54.90
Storey 16	52	54.33
Storey 15	48.8	53.75
Storey 14	45.6	53.13
Storey 13	42.4	52.48
Storey 12	39.2	51.80
Storey 11	36	51.08
Storey 10	32.8	50.31
Storey 9	29.6	49.46
Storey 8	26.4	48.58
Storey 7	23.2	47.61
Storey 6	20	46.53
Storey 5	16.8	45.32
Storey 4	13.6	43.93
Storey 3	10.4	42.28
Storey 2	7.2	40.19
Storey 1	4	43.02
Base	0	0

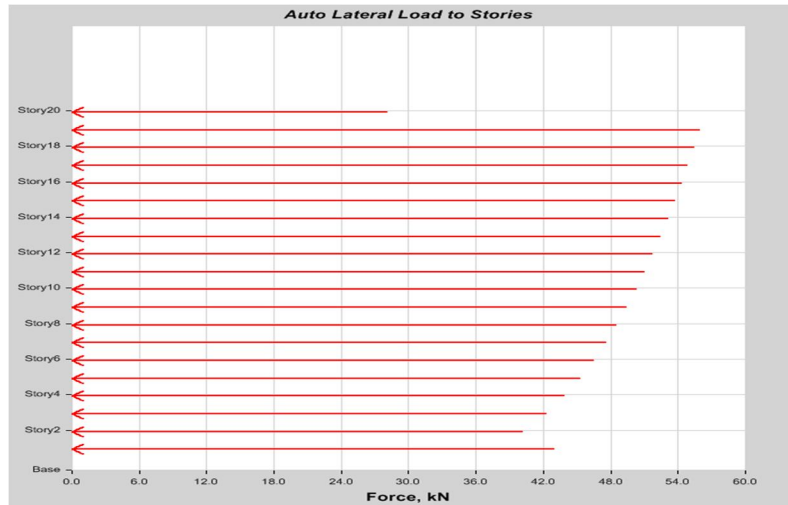


Fig. (4.2): Lateral Load to storey in y-direction.

Table (4.2): lateral load to storey in x-direction:

Storey	Storey height (m)	Load in X-Dir (kN)
Storey 20	64.8	26.84
Storey 19	61.6	53.37
Storey 18	58.4	52.87
Storey 17	55.2	52.35
Storey 16	52	51.81
Storey 15	48.8	51.25
Storey 14	45.6	50.66
Storey 13	42.4	50.04
Storey 12	39.2	49.38
Storey 11	36	48.69
Storey 10	32.8	47.95
Storey 9	29.6	47.15
Storey 8	26.4	46.29
Storey 7	23.2	45.36
Storey 6	20	44.32
Storey 5	16.8	43.16
Storey 4	13.6	41.83
Storey 3	10.4	40.23
Storey 2	7.2	38.24
Storey 1	4	40.91
Base	0	0

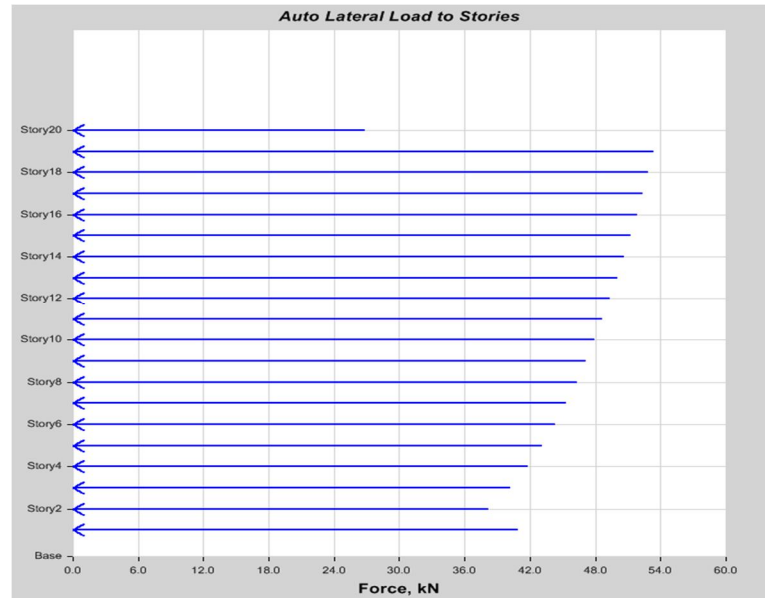


Fig. (4.3) Lateral Load to storey in x-direction.

4.3.2 Model check

To check the adequacy of the finite element model the results obtained were compared with results from Mosa, 2015. The results of the drift variation with respect to height (m) were used in the comparison as shown in Table (4.3).

As results of this comparison were unsatisfactory, the model was checked by increasing the number of the elements by adding one node and two nodes for the corner columns and comparing the displacements for the three cases as follows:

Case one: without adding nodes in the corner columns, the total number of nodes was 336 as shown in Fig. (4.4). In this case the max storey displacements curves were as shown in Fig. (4.5).

Table (4.3): The values of drift in y-direction for case study and Mosa, 2015 model:

Storey	Storey height (m)	Drift of case study	Drift of Mosa, 2015
Storey 20	64.8	0.000769	0.00000314
Storey 19	61.6	0.000776	0.00000356
Storey 18	58.4	0.000836	0.00000409
Storey 17	55.2	0.000924	0.00000466
Storey 16	52	0.001014	0.000001
Storey 15	48.8	0.001104	0.000001
Storey 14	45.6	0.001193	0.000001
Storey 13	42.4	0.001278	0.000001
Storey 12	39.2	0.001358	0.000001
Storey 11	36	0.001434	0.000001
Storey 10	32.8	0.001503	0.000001
Storey 9	29.6	0.001565	0.000001
Storey 8	26.4	0.001619	0.000001
Storey 7	23.2	0.00166	0.000001
Storey 6	20	0.001686	0.000001
Storey 5	16.8	0.001689	0.000001
Storey 4	13.6	0.001667	0.000001
Storey 3	10.4	0.001581	0.000001
Storey 2	7.2	0.001368	0.000001
Storey 1	4	0.000772	0.00000372
Base	0	0	0

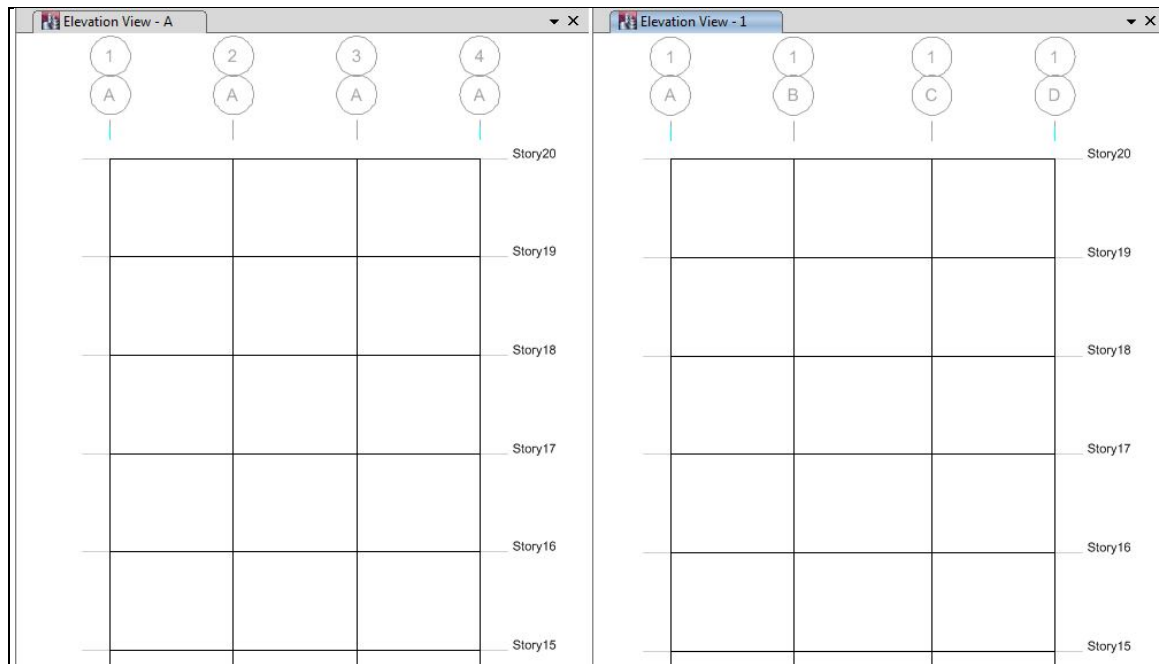


Fig. (4.4): Case One Model

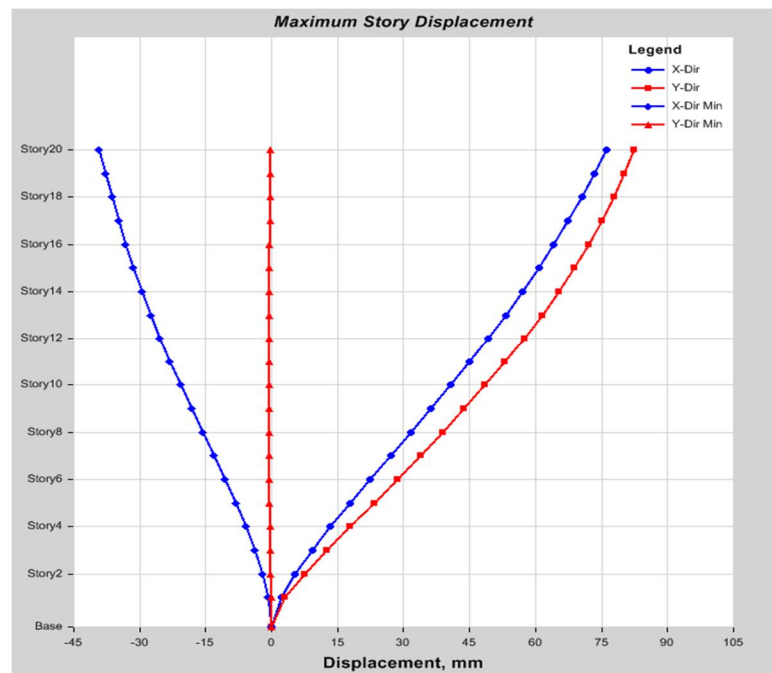


Fig. (4.5): Case one displacement curves.

Case two: by adding one node in each corner column so the total number of nodes was 496 as shown in Fig. (4.6). In this case the maximum storey displacements curves were as shown in Fig. (4.7).

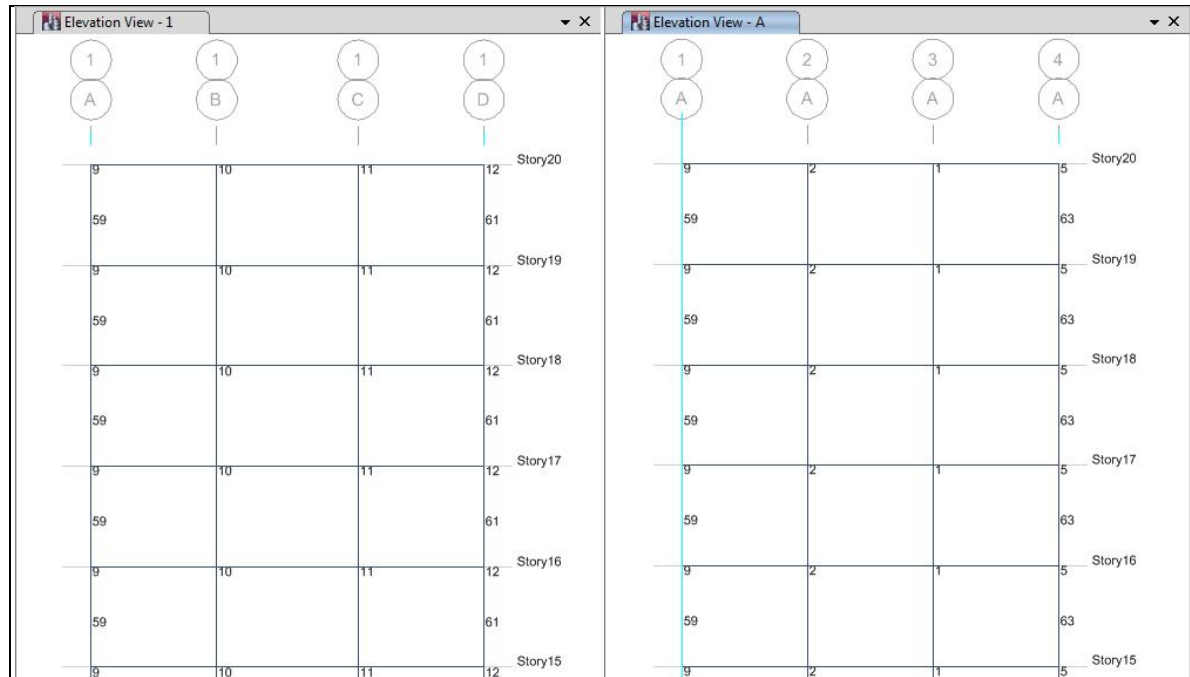


Fig. (4.6): Case Two Model.

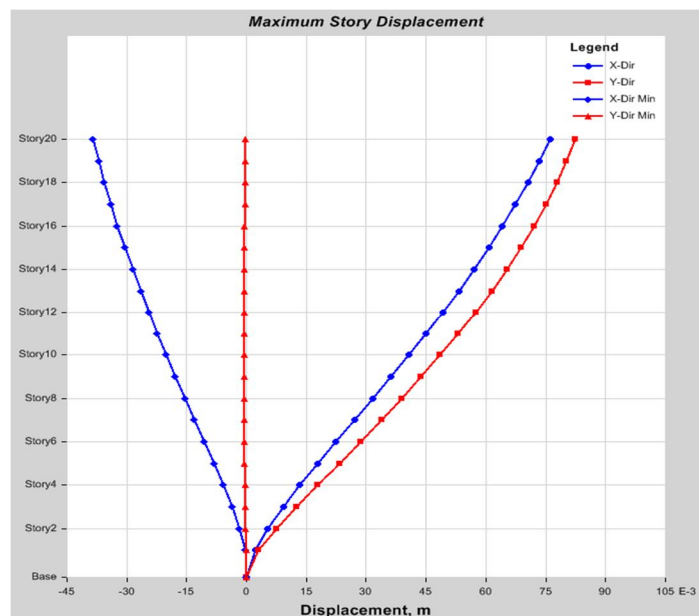


Fig. (4.7): Case Two displacements curves.

Case three: by adding two nodes in each corner column so that the total number of nodes was 656 as shown in Fig. (4.8). In this case the max storey displacements curves were as shown in Fig. (4.9).



Fig. (4.8): Case Three Model.

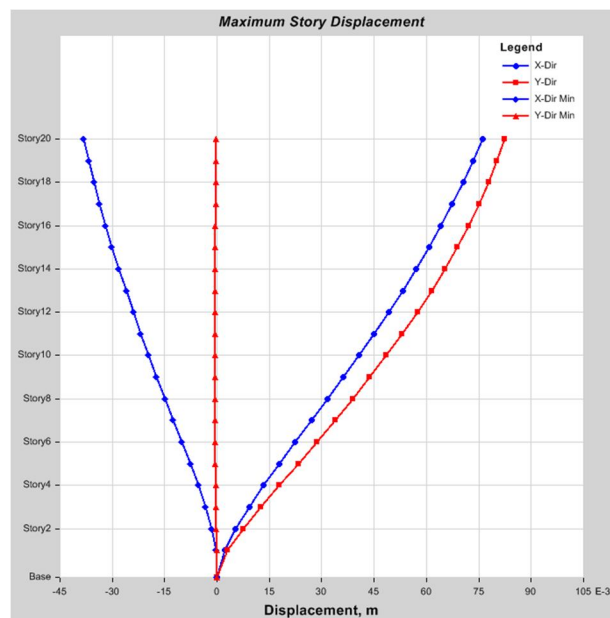


Fig. (4.9): Case three Displacements Curves.

Then by comparing results for the three cases, the displacements results show monotonic convergence as can be seen from Table (4.4) and Fig. (4.10) for x-direction. This verifies that the finite element model is ok.

Table (4.4): The variation of displacements of three cases in x-direction (mm):

Storey	Storey height (m)	Case1 displacement (m)	Case2 displacement (m)	Case3 displacement (m)
Storey 20	64.8	-0.03910542	-0.03840424	-0.03818356
Storey 19	61.6	-0.03775045	-0.0370403	-0.03674954
Storey 18	58.4	-0.03633083	-0.03558024	-0.03526952
Storey 17	55.2	-0.03481684	-0.03400973	-0.0336745
Storey 16	52	-0.03318679	-0.03231516	-0.03195355
Storey 15	48.8	-0.03142653	-0.03048614	-0.03009662
Storey 14	45.6	-0.02952869	-0.02851855	-0.0281008
Storey 13	42.4	-0.02749216	-0.02641428	-0.02597583
Storey 12	39.2	-0.02532177	-0.02444142	-0.02402
Storey 11	36	-0.02302824	-0.02236854	-0.02191846
Storey 10	32.8	-0.02062823	-0.02016271	-0.01968746
Storey 9	29.6	-0.01814466	-0.01784444	-0.01734865
Storey 8	26.4	-0.01560725	-0.0154399	-0.01492977
Storey 7	23.2	-0.01305321	-0.01298258	-0.01246568
Storey 6	20	-0.0105283	-0.01051409	-0.00999993
Storey 5	16.8	-0.00808812	-0.00808663	-0.00758734
Storey 4	13.6	-0.00579978	-0.005767	-0.00529762
Storey 3	10.4	-0.00374401	-0.00364121	-0.00322678
Storey 2	7.2	-0.00201775	-0.00182625	-0.00147982
Storey 1	4	-0.00073772	-0.00034267	-0.00033488
Base	0	0	0	0

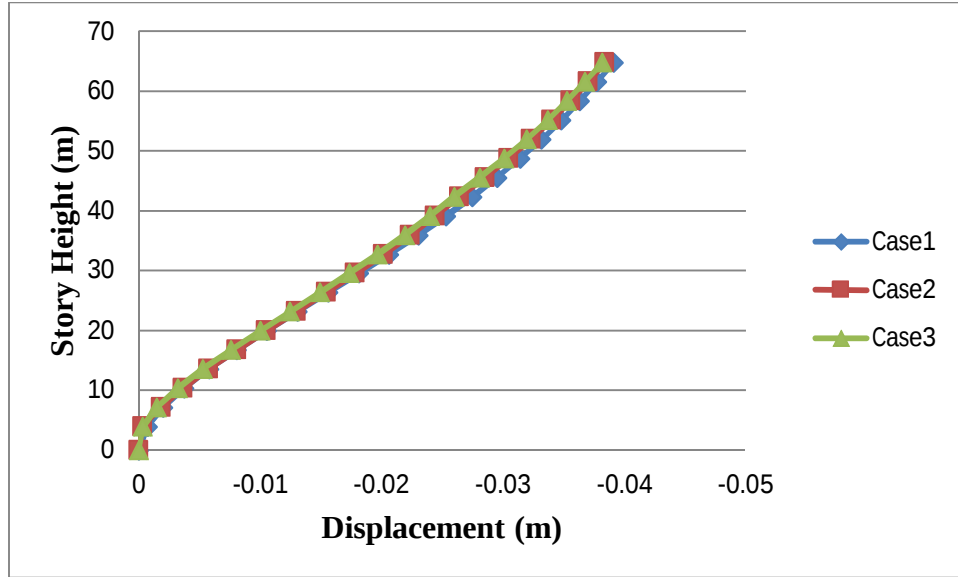


Fig. (4.10): The variation of displacements of three cases in x-direction (min).

4.3.3 Analysis and Discussion of the linear and nonlinear static results

4.3.3.1 Max storey displacements results

The results for maximum displacements in the direction of wind load (y-direction) for the linear, nonlinear P Delta for the two load combinations are presented in Table (4.5). As can be seen from Table (4.5), the results of combination U1 were the same as those of U2 in linear analysis. But results for U1 were more than those for U2 for p-delta results. So the first combination has been used in the rest of the analysis. The Comparison between the displacements in the three cases (linear, p-delta, p-delta-w) for case study and Mosa, 2015 were shown in Tables (4.6 and 4.7) and Figures (4.11a, b, and c). In all cases the case study results was greater than Mosa, 2015 results with maximum percentage difference of about 20%. As can be seen from Table (4.8) and Figures (4.12 and 4.13) the p-delta's results were greater than the linear results with a maximum percentage difference of about 23%, which means there is need to carryout nonlinear analysis, especially, for flexible buildings.

To check the code requirements according to ASCE 7-05, the max horizontal deflection must not exceed $H/500$.

Max allowable displacement $< 64800/500 = 129.6\text{mm}$

From Table (4.4) max displacement $= 101.4\text{mm} < 129.6\text{mm} \quad \therefore \text{OK.}$

Referring to Table (4.8) and Fig.(4.14), the maximum percentage difference in displacement in y-direction between $P\Delta$ analysis and linear analysis was about 23.1%. Also, the maximum percentage difference between $P\Delta$ with large displacement analysis and $P\Delta$ analysis is 8.56%, and the maximum percentage difference between $P\Delta$ with large displacement analysis and linear analysis was about 13.34%.

Table (4.5): Max storey displacement in y-direction for combination U1 and U2:

Storey	Linear Displacement (mm)		P Delta Displacement (mm)	
	U1	U2	U1	U2
20	82.4	82.4	101.4	98.5
19	80.2	80.2	98.4	95.6
18	77.8	77.8	95.2	92.4
17	75.1	75.1	91.7	89
16	72.1	72.1	87.9	85.3
15	68.9	68.9	83.8	81.2
14	65.4	65.4	79.4	76.9
13	61.5	61.5	74.7	72.3
12	57.5	57.5	69.6	67.3
11	53.1	53.1	64.3	62.1
10	48.5	48.5	58.7	56.7
9	43.7	43.7	52.8	51
8	38.8	38.8	46.7	45
7	33.8	33.8	40.4	38.9
6	28.6	28.6	33.9	32.7
5	23.3	23.3	27.4	26.4
4	17.9	17.9	20.9	20.1
3	12.5	12.5	14.5	14
2	7.5	7.5	8.5	8.2
1	3.1	3.1	3.4	3.3
Base	0	0	0	0

Table (4.6): Linear static displacements y-direction for case study and Mosa, 2015:

Storey	Linear Disp.(mm) Mosa, 2015	Linear Disp.(mm) Case study	Difference%
20	66.1	82.4	19.78
19	64	80.2	20.20
18	61.9	77.8	20.44
17	59.5	75.1	20.77
16	56.9	72.1	21.10
15	54.2	68.9	21.34
14	51.2	65.4	21.71
13	48.1	61.5	21.79
12	44.7	57.5	22.26
11	41.2	53.1	22.41
10	37.5	48.5	22.68
9	33.8	43.7	22.65
8	30	38.8	22.68
7	26	33.8	23.10
6	21.9	28.6	23.43
5	17.8	23.3	23.61
4	13.6	17.9	24.02
3	9.6	12.5	23.20
2	5.7	7.5	24
1	2.4	3.1	22.58
Base	0	0	0

Table (4.7): Non linear displacements in y-direction for case study and Mosa, 2015:

Storey	P delta			P delta w		
	Case study Disp.(mm)	Mosa, 2015 Disp. (mm)	Difference %	Case study Disp.(mm)	Mosa, 2015 Disp. (mm)	Difference %
20	101.4	81.1	20.02	93.4	78.5	15.95
19	98.4	78.6	20.12	90.5	76	16.02
18	95.2	75.9	20.27	87.5	73.4	16.11
17	91.7	73	20.39	84.2	70.6	16.15
16	87.9	69.9	20.48	80.6	67.5	16.25
15	83.8	66.5	20.64	76.8	64.2	16.41
14	79.4	62.9	20.78	72.7	60.7	16.51
13	74.7	59	21.02	68.3	57	16.54
12	69.6	54.9	21.12	63.6	53	16.67
11	64.3	50.6	21.31	58.7	48.8	16.87
10	58.7	46.1	21.47	53.6	44.5	16.98
9	52.8	41.5	21.40	48.2	39.9	17.22
8	46.7	36.8	21.20	42.6	35.3	17.14
7	40.4	31.9	21.04	36.9	30.4	17.62
6	33.9	26.9	20.65	31	25.5	17.74
5	27.4	21.8	20.44	25	20.6	17.60
4	20.9	16.7	20.10	19	15.6	17.89
3	14.5	11.6	20	13.2	10.9	17.42
2	8.5	6.9	18.82	7.7	6.5	15.58
1	3.4	2.9	14.71	3.1	2.7	12.90
Base	0	0	0	0	0	0

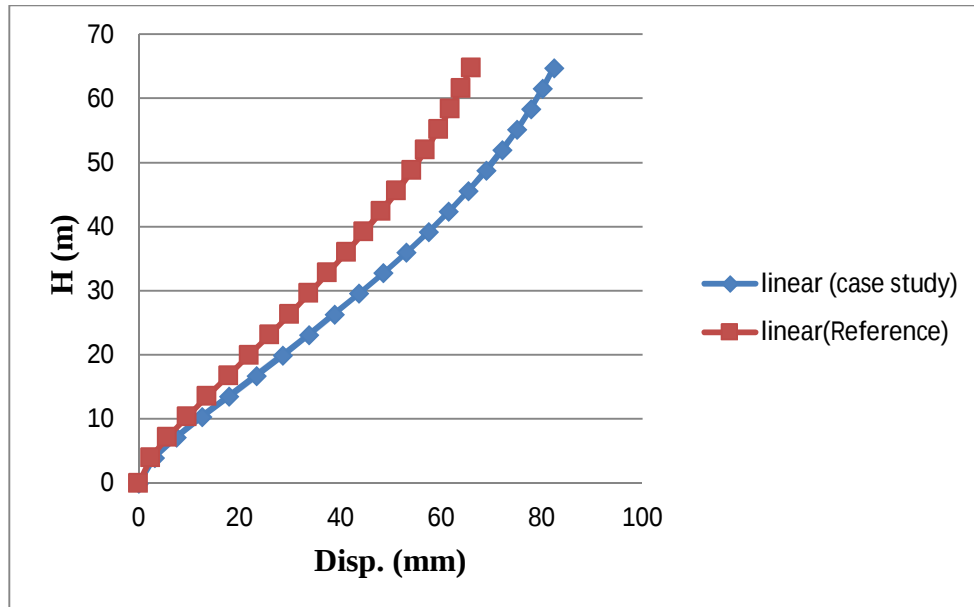


Fig. (4.11a): Linear displacements case study and Mosa, 2015 model.

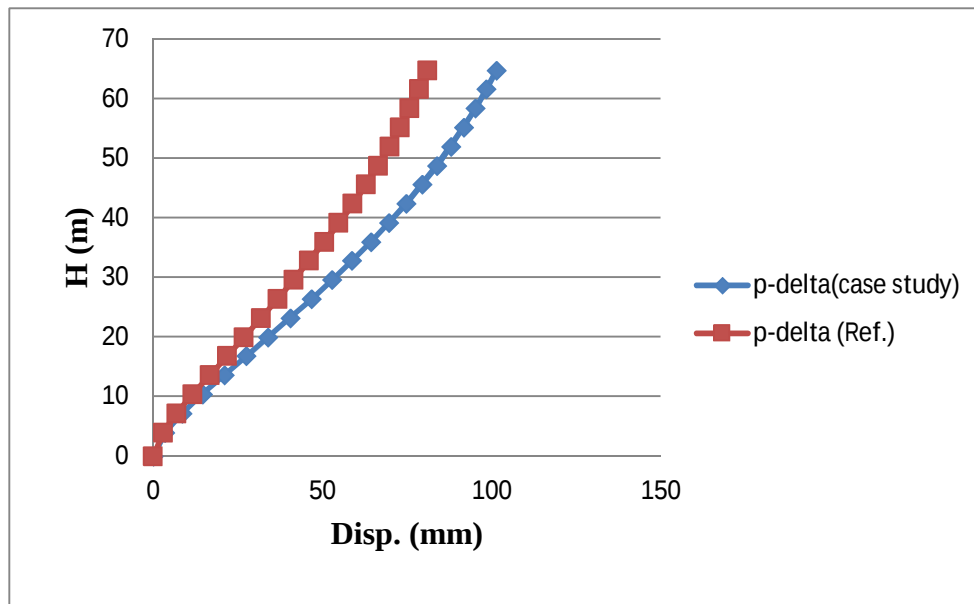


Fig. (4.11b): P-delta displacements for case study and Mosa, 2015 model.

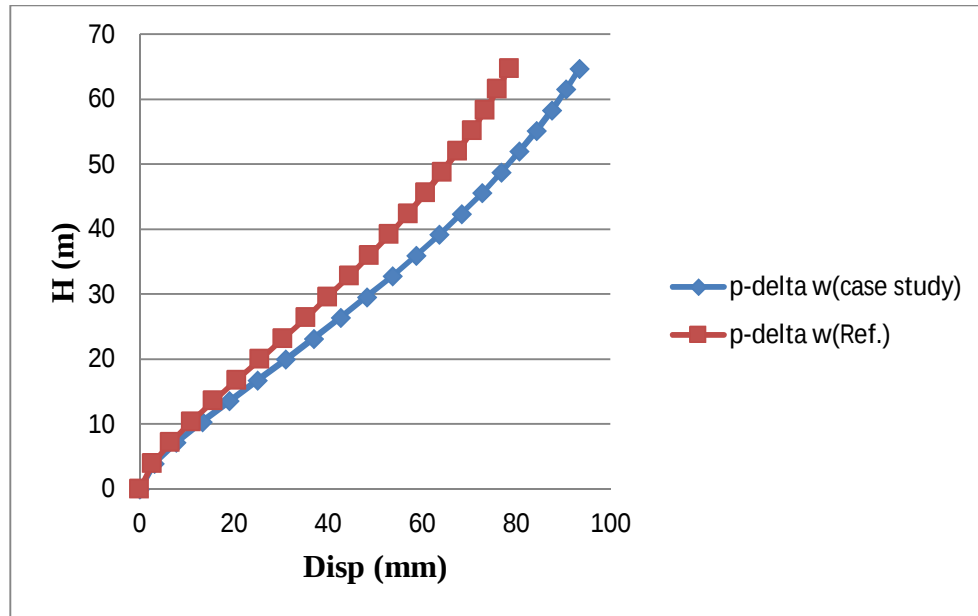
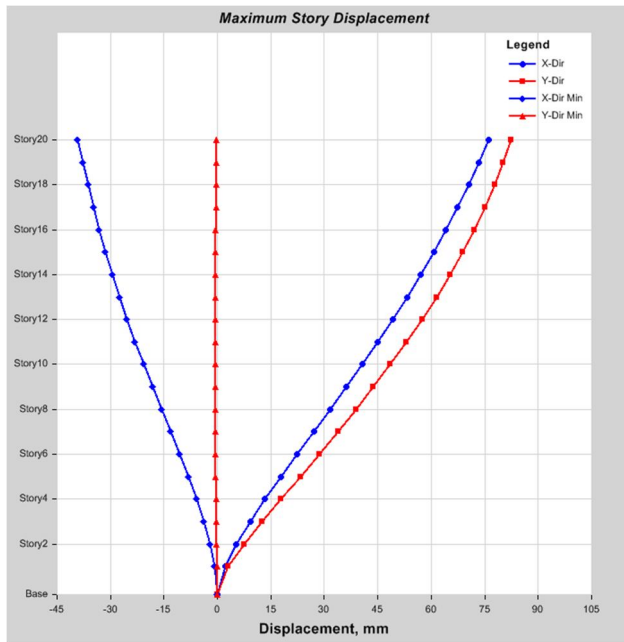


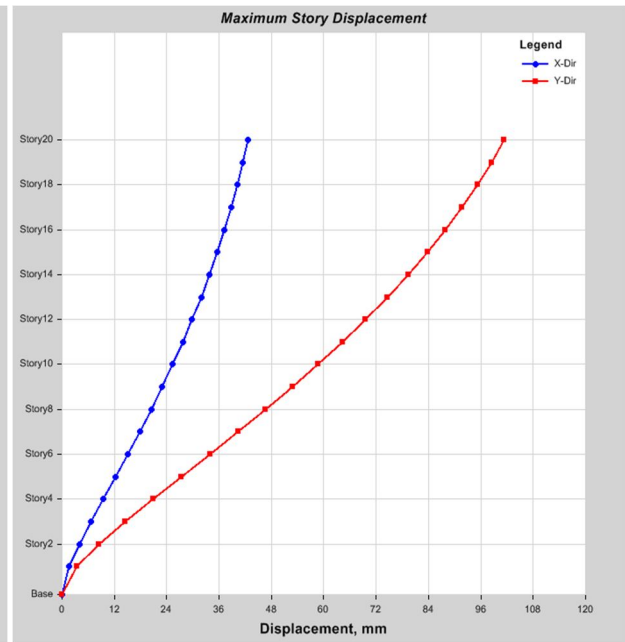
Fig. (4.11c): P-delta w displacements for case study and Mosa, 2015 model.

Table (4.8): Maximum storey displacement in y-direction for case study:

Storey	Linear Disp. (mm) (L)	P Delta Disp. (mm) (PΔ)	Percentage difference (PΔ/L)%	P Delta + large Disp.(mm) (PΔW)	Percentage difference (PΔ_w/ L)%
20	82.4	101.4	23.06	93.4	13.35
19	80.2	98.4	22.69	90.5	12.84
18	77.8	95.2	22.37	87.5	12.47
17	75.1	91.7	22.10	84.2	12.11
16	72.1	87.9	21.91	80.6	11.79
15	68.9	83.8	21.63	76.8	11.47
14	65.4	79.4	21.41	72.7	11.16
13	61.5	74.7	21.46	68.3	11.06
12	57.5	69.6	21.04	63.6	10.61
11	53.1	64.3	21.09	58.7	10.55
10	48.5	58.7	21.03	53.6	10.52
9	43.7	52.8	20.82	48.2	10.29
8	38.8	46.7	20.36	42.6	9.79
7	33.8	40.4	19.53	36.9	9.17
6	28.6	33.9	18.53	31	8.39
5	23.3	27.4	17.59	25	7.29
4	17.9	20.9	16.76	19	6.15
3	12.5	14.5	16	13.2	5.6
2	7.5	8.5	13.33	7.7	2.67
1	3.1	3.4	9.68	3.1	0
Base	0	0	0	0	0



a. linear max displacement



b. P Delta Mode



c. P Delta+ large displacement mode

Fig. (4.12): Max Storey displacements.

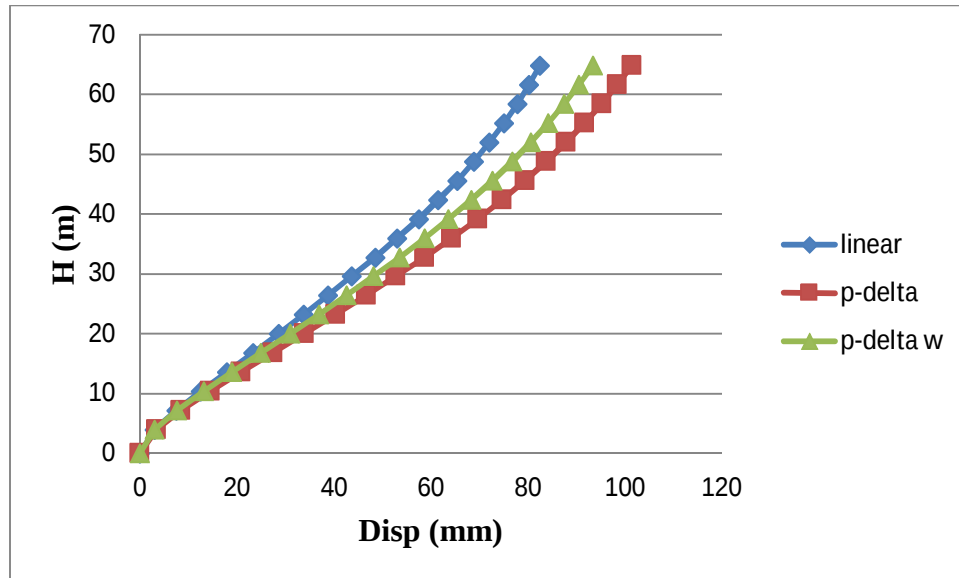


Fig. (4.13): Maximum storey displacement in y-direction (Linear, P- Δ , P- Δ -w).

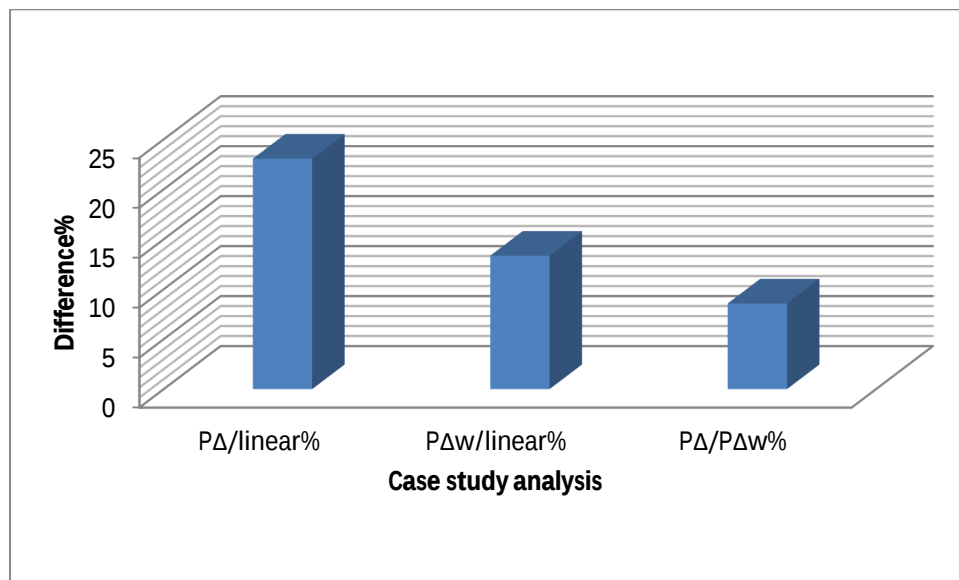


Fig. (4.14): The maximum storey displacement in y-direction differences (%).

4.3.3.2 Max storey Shear results

The P- Δ -w results were used for maximum shear comparison between static and dynamic analysis since they give more accurate results. The maximum storey shear in y-direction for top and bottom of the stories were shown in

Table (4.9) and Fig. (4.15), Also the comparison between the case study and Mosa, 2015 shear was given in Table (4.10).

Table (4.9): Maximum storey Shear in y-direction P-delta w (case study):

Storey	Elevation	Location	Shear in y-direction
	m		kN
Storey20	64.8	Top	-143.5017
		Bottom	-143.1694
Storey19	61.6	Top	-231.5471
		Bottom	-231.2035
Storey18	58.4	Top	-318.7619
		Bottom	-318.4055
Storey17	55.2	Top	-405.1165
		Bottom	-404.7457
Storey16	52	Top	-490.5782
		Bottom	-490.1926
Storey15	48.8	Top	-575.1112
		Bottom	-574.7111
Storey14	45.6	Top	-658.6763
		Bottom	-658.2624
Storey13	42.4	Top	-741.229
		Bottom	-740.8027
Storey12	39.2	Top	-822.7191
		Bottom	-822.2819
Storey11	36	Top	-903.0883
		Bottom	-902.6425
Storey10	32.8	Top	-982.2687
		Bottom	-981.8169
Storey9	29.6	Top	-1060.1797
		Bottom	-1059.7249
Storey8	26.4	Top	-1136.7236
		Bottom	-1136.2697
Storey7	23.2	Top	-1211.7799

Storey	Elevation	Location	Shear in y-direction
	m		kN
		Bottom	-1211.3316
Storey6	20	Top	-1285.1959
		Bottom	-1284.759
Storey5	16.8	Top	-1356.771
		Bottom	-1356.3529
Storey4	13.6	Top	-1426.2297
		Bottom	-1425.8407
Storey3	10.4	Top	-1493.1656
		Bottom	-1492.8209
Storey2	7.2	Top	-1556.9116
		Bottom	-1556.6354
Storey1	4	Top	-1625.3513
		Bottom	-1625.1766
Base	0	Top	0
		Bottom	0

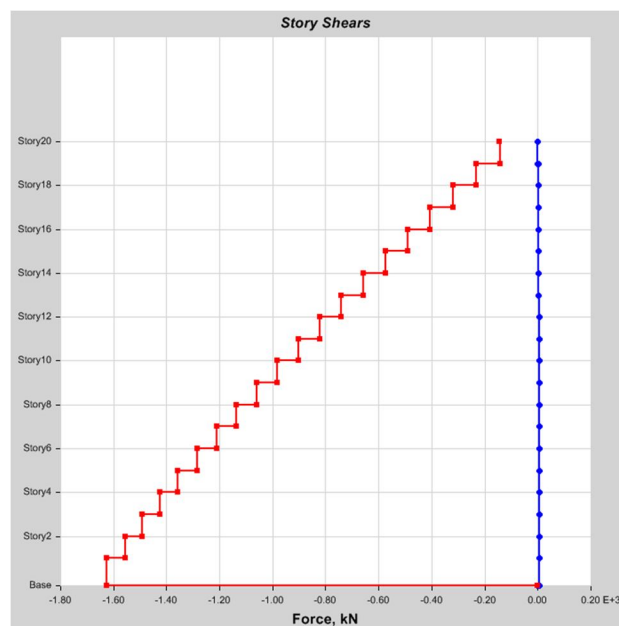


Fig. (4.15): Maximum storey shear y-direction.

Table (4.10): Comparison of maximum shear (P- Δ -w) between case study and Mosa, 2015:

Storey	Max shear (case study)P-Δ-w (kN)	Max shear (Mosa, 2015) P-Δ-w (kN)	Difference (%)
20	-143.5017	81.99	42.86
19	-231.5471	133.01	42.56
18	-318.7619	183.57	42.41
17	-405.1165	233.64	42.33
16	-490.5782	283.24	42.26
15	-575.1112	332.26	42.23
14	-658.6763	380.76	42.19
13	-741.229	428.67	42.17
12	-822.7191	475.96	42.15
11	-903.0883	522.58	42.13
10	-982.2687	568.5	42.12
9	-1060.1797	613.66	42.12
8	-1136.7236	657.98	42.12
7	-1211.7799	701.39	42.12
6	-1285.1959	743.8	42.13
5	-1356.771	785.06	42.14
4	-1426.2297	825	42.16
3	-1493.1656	863.38	42.18
2	-1556.9116	899.78	42.21
1	-1625.3513	938.53	42.26
Base	0	0	0

The maximum storey shear for case study show a pronounced percentage difference of 42% compared with Mosa, 2015.

4.4 Dynamic Analysis

In Dynamic analysis for wind load case, the building was carried out using modal and response spectrum analysis.

4.4.1 Wind load spectra

-As presented in chapter 3 section (3.5.6) and according to Smith and Coull, 1991 the along-wind and across-wind accelerations were obtained as follows:

- Height of the building (H) = 64.8m

-Breadth of the building (B) = 12m

- Depth of the building (D) = 12.5.

-The fundamental natural frequency (n_0) (from the program) = 0.4Hz.

-Max deflection at top of the building (Δ) = (64.8/500) = 0.13m.

- Average building density $\rho = 472 \text{ kg/m}^3$.

- Critical damping ratio = 0.015.

- Mean wind speed at top of building = $(100\text{mph} * \frac{8}{5} * \frac{1000}{60*60}) = 45 \text{ m/sec}$

-The maximum along-wind acceleration (0.58552) and the maximum across-wind acceleration (1.61625) were first obtained. Then these were used to obtain the first two response spectra assuming 4 mode shapes as shown in Table (4.11) and Fig. (4.16a and 4.16b) and 6 mode shapes as shown in Table (4.12).

Table (4.11): Calculations of along-wind (a_D) and across-wind (a_w) accelerations for 4 periods (10, 5, 3.3, and 2.5 sec):

Bread	Depth	n0	Δ	ρ	β	VH	r	W/H	B	n0*H/V	S	n0/VH	F	R	V	gp	G	aD	ar	aw
12	12.5	0.1	0.13	472	0.015	45	0.46	0.185185185	1.07	0.144	0.69	0.00222	0.48	22.08	0.09766165	3.6	8.96775	0.04449	11479	8.93051
12	12.5	0.2	0.13	472	0.015	45	0.46	0.185185185	1.07	0.288	0.4	0.00444	0.44	11.7333	0.19146049	3.8	7.25465	0.16926	1165.5	3.82839
12	12.5	0.3	0.13	472	0.015	45	0.46	0.185185185	1.07	0.432	0.32	0.00667	0.3	6.4	0.27768405	3.9	5.90324	0.35475	305.79	2.31944
12	12.5	0.4	0.13	472	0.015	45	0.46	0.185185185	1.07	0.576	0.2	0.00889	0.3	4	0.35529247	3.95	5.09128	0.58552	118.34	1.61623

Table (4.12) Calculations of along-wind (a_D) and across-wind (a_w) accelerations (for 6 periods (10, 6.7, 5, 4, 3.3, and 2.5 sec)):

heigh(H)	Bread	Depth	n0	Δ	ρ	β	VH	r	W/H	B	n0*H/V	S	n0/VH	F	R	V	gp	G	aD	ar	aw
64.8	12	12.5	0.1	0.13	472	0.015	45	0.46	0.185185185	1.07	0.144	0.69	0.00222	0.48	22.08	0.09766165	3.6	8.96775	0.04449	11479	8.93051
64.8	12	12.5	0.15	0.13	472	0.015	45	0.46	0.185185185	1.07	0.216	0.49	0.00333	0.4	13.0667	0.14421159	3.7	7.39931	0.09592	3011.8	5.41822
64.8	12	12.5	0.2	0.13	472	0.015	45	0.46	0.185185185	1.07	0.288	0.4	0.00444	0.44	11.7333	0.19146049	3.8	7.25465	0.16926	1165.5	3.82839
64.8	12	12.5	0.25	0.13	472	0.015	45	0.46	0.185185185	1.07	0.36	0.37	0.00556	0.39	9.62	0.2371585	3.85	6.79038	0.25921	558.11	2.90211
64.8	12	12.5	0.3	0.13	472	0.015	45	0.46	0.185185185	1.07	0.432	0.32	0.00667	0.3	6.4	0.27768405	3.9	5.90324	0.35475	305.79	2.31944
64.8	12	12.5	0.4	0.13	472	0.015	45	0.46	0.185185185	1.07	0.576	0.2	0.00889	0.3	4	0.35529247	3.95	5.09128	0.58552	118.34	1.61623

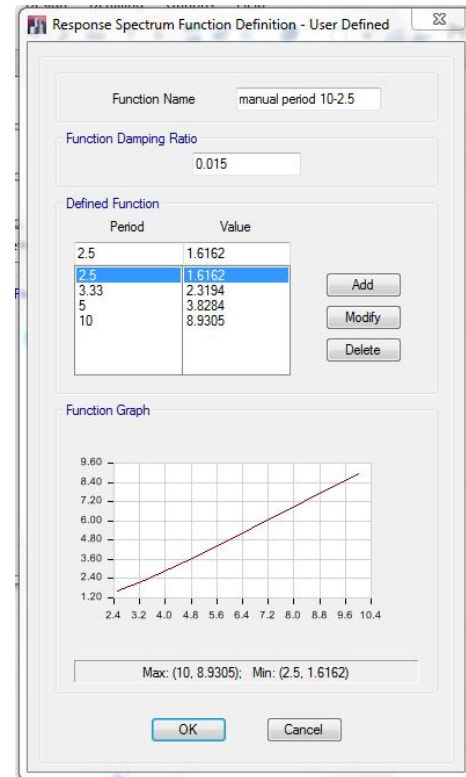
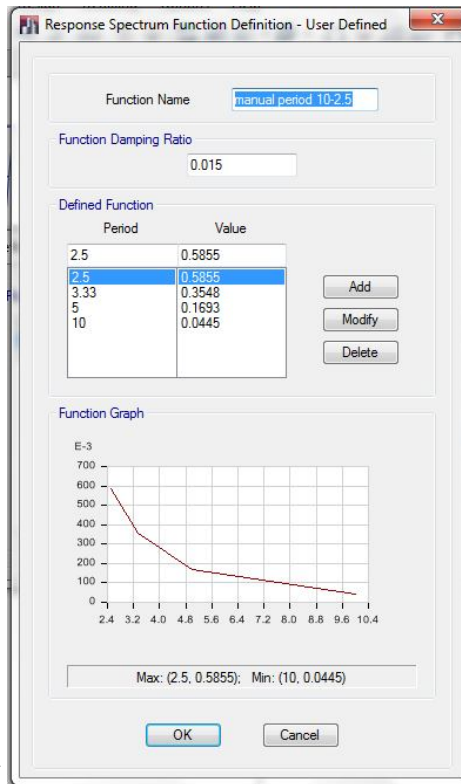


Fig. (4.16a): Along-wind response spectra Fig. (4.16b): Across-wind response spectra (4 periods)

4.4.1.1 The along- wind Response

In the displacements obtained from 4 and 6 mode shapes. The mode shapes of building were given in Fig. (4.17 and 4.18). The maximum storey displacements (u_2) in y-direction were given in Table (4.13) and Fig. (4.19). As can be seen from the Table, it was found there is a very small difference between 4 and 6 modes in terms of displacements results. Hence the 6 mode wind spectra were used for the comparison between static and dynamic analysis, as it gives more accurate results which were slightly than 4 mode wind spectra results. The maximum storey displacement in storey 20 (0.114m) is greater than the nonlinear static analysis results (0.093m) by about 25%.

Also the maximum storey shear results in y-y direction are given in Table (4.14) and Figure (4.20), the maximum storey shear in storey 1 (1170.40kN) is less than the nonlinear static analysis results (1625.18 kN). The maximum storey drifts were given in Table (4.15) and Figure (4.21), the maximum drift in storey 6(0.00232878m).

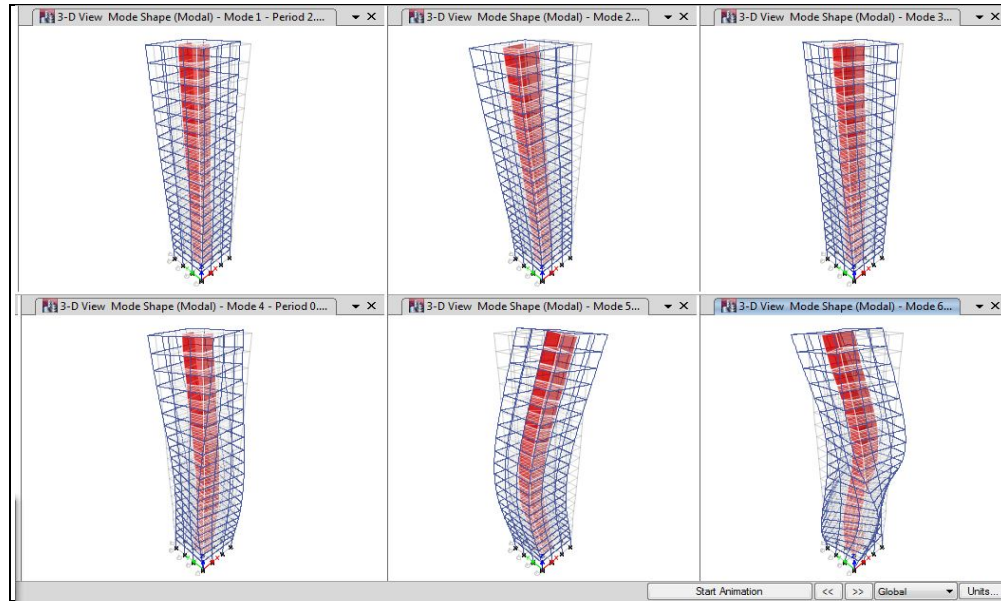


Fig. (4.17): Six mode shapes.

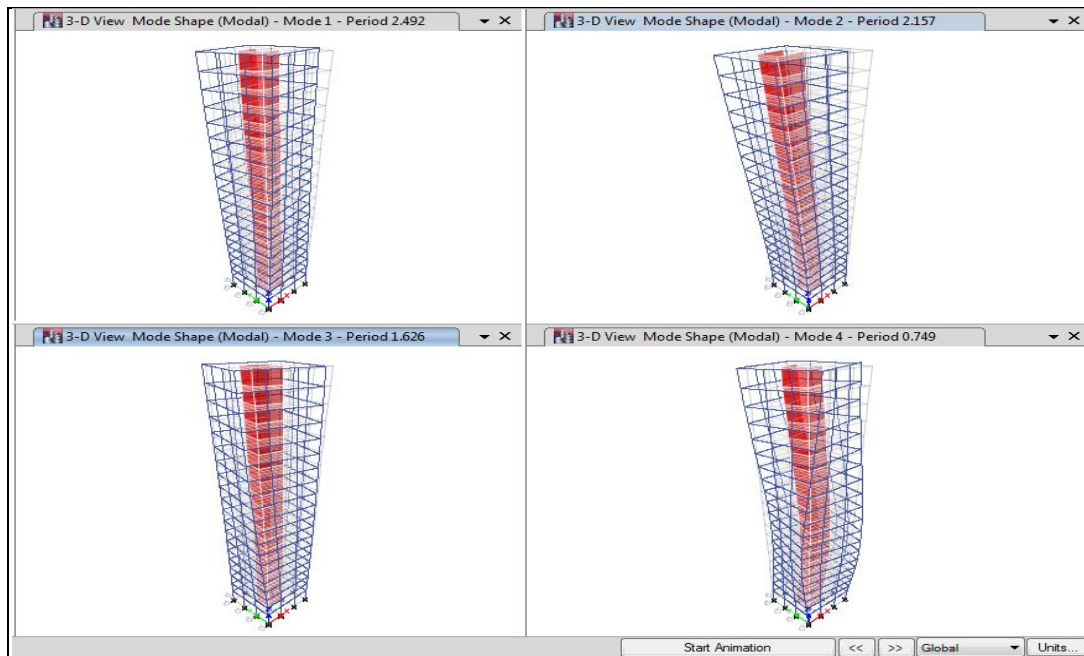


Fig. (4.18): Four mode shapes.

Table (4.13): Maximum storey displacements (along-wind accelerations):

Storey	(4 spectra) displacement (m)	(6 spectra) displacement (m)	Difference %
	4 modes	6 modes	
20	0.11384788	0.11384856	0.0006
19	0.11108062	0.1110809	0.0003
18	0.10796003	0.10796005	0.00002
17	0.10439659	0.10439659	0.0000
16	0.1003527	0.10035298	0.0003
15	0.09582583	0.09582662	0.0008
14	0.09083189	0.09083323	0.0015
13	0.08539718	0.08539889	0.0020
12	0.07955422	0.07955593	0.0021
11	0.07333965	0.07334097	0.0018
10	0.0667933	0.06679398	0.0010
9	0.05995829	0.05995834	0.00008
8	0.05288177	0.05288148	-0.0005
7	0.04561679	0.04561661	-0.0004
6	0.03822581	0.03822618	0.0010
5	0.03078747	0.03078863	0.0038
4	0.02340996	0.0234118	0.0079
3	0.01625754	0.01625967	0.0131
2	0.00960629	0.00960814	0.0193
1	0.00395318	0.00395424	0.0268
Base	0	0	0

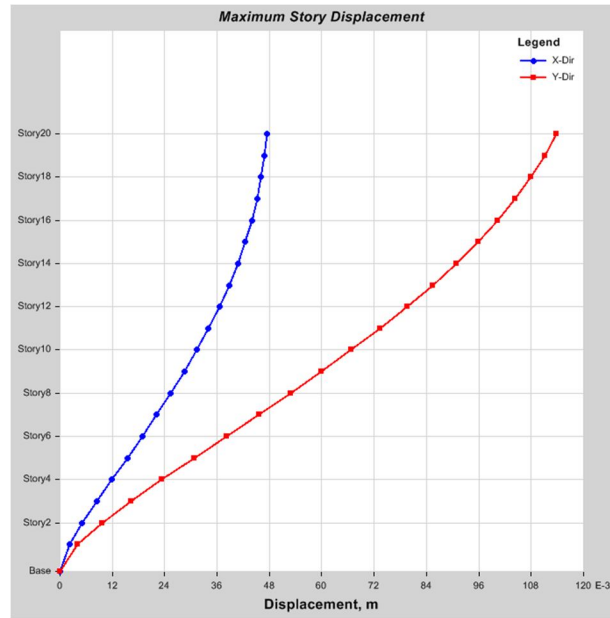


Fig. (4.19): Maximum storey displacement for 6 modes (along-wind).

Table (4.14): Max storey shear in y-direction for 6modes (along-wind):

Storey	Elevation	Location	Shear in Y-Direction
	M		kN
Storey20	64.8	Top & bottom	104.9072
Storey19	61.6	Top & bottom	217.6913
Storey18	58.4	Top & bottom	323.7708
Storey17	55.2	Top & bottom	422.9831
Storey16	52	Top & bottom	515.3156
Storey15	48.8	Top & bottom	600.9127
Storey14	45.6	Top & bottom	680.0217
Storey13	42.4	Top & bottom	752.9194

Storey	Elevation	Location	Shear in Y-Direction
	M		kN
Storey12	39.2	Top & bottom	819.8441
Storey11	36	Top & bottom	880.953
Storey10	32.8	Top & bottom	936.3095
Storey9	29.6	Top & bottom	985.8957
Storey8	26.4	Top & bottom	1029.6389
Storey7	23.2	Top & bottom	1067.4423
Storey6	20	Top & bottom	1099.2147
Storey5	16.8	Top & bottom	1124.9043
Storey4	13.6	Top & bottom	1144.5415
Storey3	10.4	Top & bottom	1158.3043
Storey2	7.2	Top & bottom	1166.6163
Storey1	4	Top & bottom	1170.4024
Base	0	Top & bottom	0

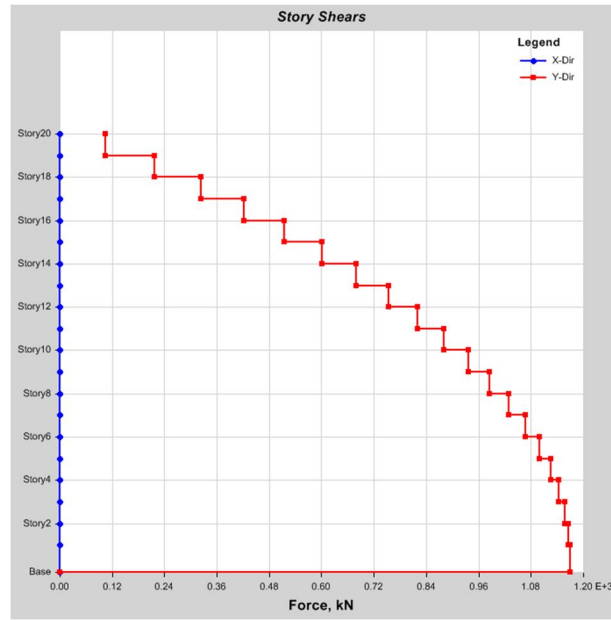


Fig. (4.20): Maximum storey shear in y-direction for 6 modes (along-wind).

Table (4.15): Max storey drift for 6 modes (along-wind):

Storey	Elevation	Location	X-direction	Y-direction
	M			
Storey20	64.8	Top	0.0001891	0.00090498
Storey19	61.6	Top	0.00024568	0.00101597
Storey18	58.4	Top	0.00031976	0.00115459
Storey17	55.2	Top	0.00040179	0.00130494
Storey16	52	Top	0.00048523	0.00145544
Storey15	48.8	Top	0.00056668	0.00159999
Storey14	45.6	Top	0.0006445	0.00173524
Storey13	42.4	Top	0.00071794	0.00185936
Storey12	39.2	Top	0.00078669	0.00197127
Storey11	36	Top	0.0008506	0.00207026
Storey10	32.8	Top	0.00090952	0.0021556
Storey9	29.6	Top	0.00096314	0.00222634
Storey8	26.4	Top	0.00101081	0.00228094
Storey7	23.2	Top	0.00105122	0.00231672
Storey6	20	Top	0.00108187	0.00232878
Storey5	16.8	Top	0.00109791	0.00230793

Storey	Elevation	Location	X-direction	Y-direction
	M			
Storey4	13.6	Top	0.00108997	0.00223652
Storey3	10.4	Top	0.00103917	0.00207941
Storey2	7.2	Top	0.00090854	0.00176728
Storey1	4	Top	0.00053728	0.00098856
Base	0	Top	0	0

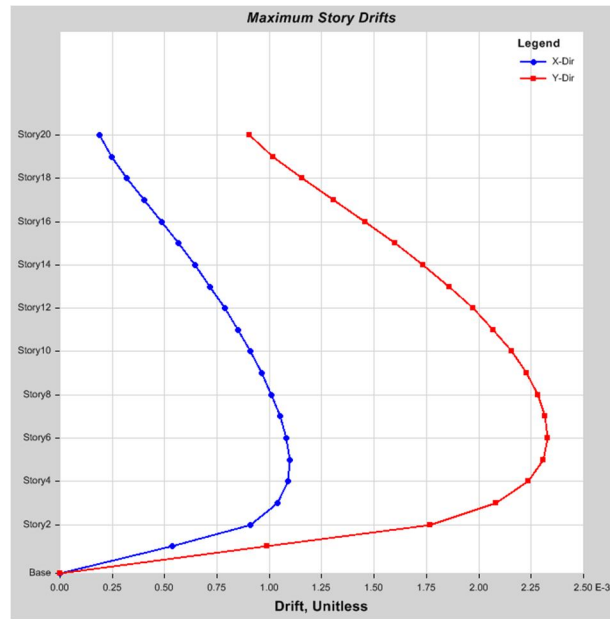


Fig. (4.21): Maximum storey drift for 6 modes (along-wind).

4.4.1.2 Along-wind Response assuming constant Acceleration

Instead of using the calculation in Tables (4.11 and 4.12) for response spectrum, a constant acceleration was assumed for the spectrum as shown in Fig. (4.22), it was found that Fig. (4.23) and Table (4.16) gave the maximum storey displacements. The maximum values of shear for top and bottom of the stories were shown in Fig. (4.24) and Table (4.18). Also the maximum storey drift were given in Fig. (4.25) and Table (4.18).

After comparing these results with section (4.4.1.1) shows that the results are exactly the same. Therefore the dynamic wind analysis can be based on constant acceleration obtained from Eq. (3.33).

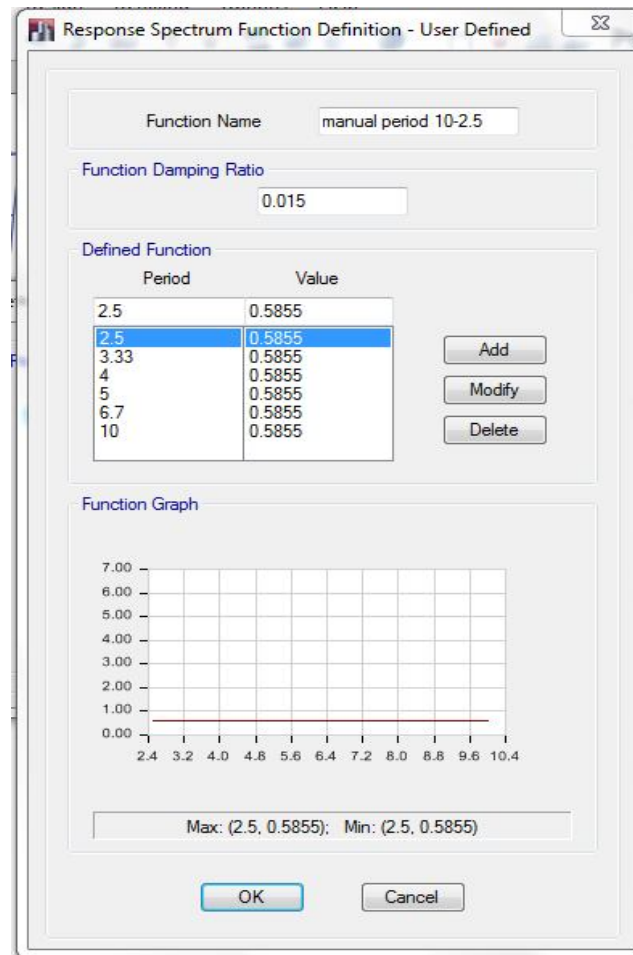


Fig. (4.22): The application of Constant acceleration spectrum.

Table (4.16): Max storey displacements in y-direction for constant acceleration:

Storey	Elevation	Location	Y-direction
	m		m
Storey20	64.8	Top	0.11384856
Storey19	61.6	Top	0.1110809
Storey18	58.4	Top	0.10796005
Storey17	55.2	Top	0.10439659
Storey16	52	Top	0.10035298
Storey15	48.8	Top	0.09582662
Storey14	45.6	Top	0.09083323
Storey13	42.4	Top	0.08539889
Storey12	39.2	Top	0.07955593
Storey11	36	Top	0.07334097
Storey10	32.8	Top	0.06679398
Storey9	29.6	Top	0.05995834
Storey8	26.4	Top	0.05288148
Storey7	23.2	Top	0.04561661
Storey6	20	Top	0.03822618
Storey5	16.8	Top	0.03078863
Storey4	13.6	Top	0.0234118
Storey3	10.4	Top	0.01625967
Storey2	7.2	Top	0.00960814
Storey1	4	Top	0.00395424
Base	0	Top	0

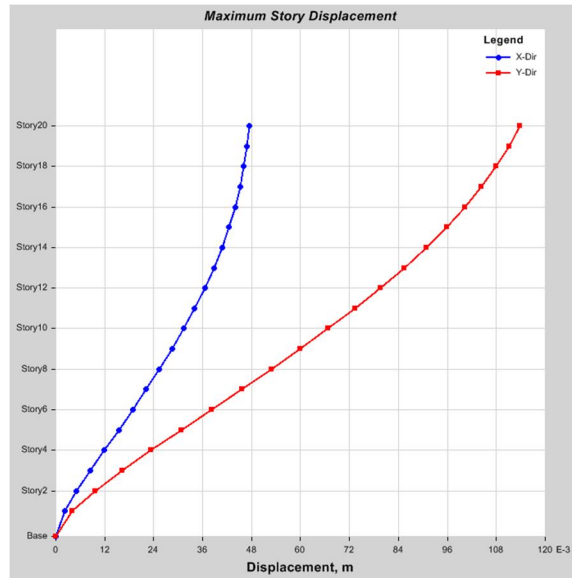


Fig. (4.23): Maximum storey displacement for constant acceleration.

Table (4.17): Max storey shear in y-direction for constant acceleration:

Storey	Elevation	Location	Shear in Y-direction
	m		kN
Storey20	64.8	Top & Bottom	104.9072
Storey19	61.6	Top & Bottom	217.6913
Storey18	58.4	Top & Bottom	323.7708
Storey17	55.2	Top & Bottom	422.9831
Storey16	52	Top & Bottom	515.3156
Storey15	48.8	Top & Bottom	600.9127
Storey14	45.6	Top & Bottom	680.0217
Storey13	42.4	Top & Bottom	752.9194
Storey12	39.2	Top & Bottom	819.8441
Storey11	36	Top & Bottom	880.953

Storey	Elevation	Location	Shear in Y-direction
	m		kN
Storey10	32.8	Top & Bottom	936.3095
Storey9	29.6	Top & Bottom	985.8957
Storey8	26.4	Top & Bottom	1029.6389
Storey7	23.2	Top & Bottom	1067.4423
Storey6	20	Top & Bottom	1099.2147
Storey5	16.8	Top & Bottom	1124.9043
Storey4	13.6	Top & Bottom	1144.5415
Storey3	10.4	Top & Bottom	1158.3043
Storey2	7.2	Top & Bottom	1166.6163
Storey1	4	Top & Bottom	1170.4024
Base	0	Top & Bottom	0

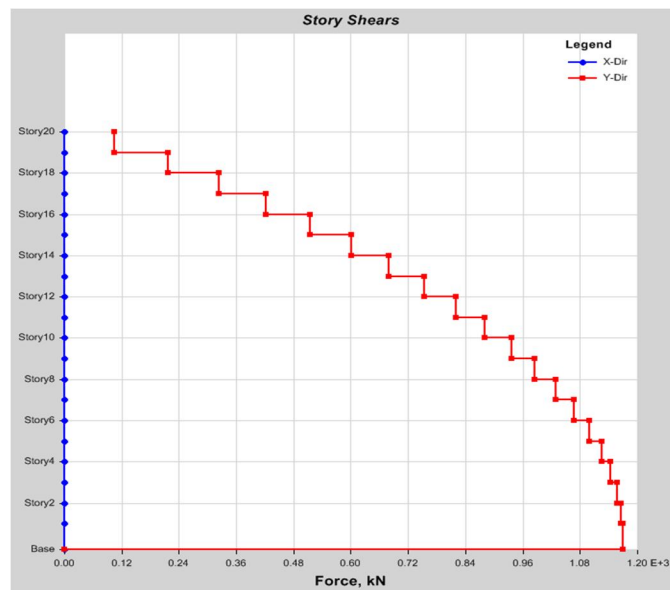


Fig. (4.24): Maximum storey shear y-direction for constant acceleration.

Table (4.18): Maximum storey drift for constant acceleration:

Storey	Elevation	Location	X-direction	Y-direction
	m			
Storey20	64.8	Top	0.0001891	0.00090498
Storey19	61.6	Top	0.00024568	0.00101597
Storey18	58.4	Top	0.00031976	0.00115459
Storey17	55.2	Top	0.00040179	0.00130494
Storey16	52	Top	0.00048523	0.00145544
Storey15	48.8	Top	0.00056668	0.00159999
Storey14	45.6	Top	0.0006445	0.00173524
Storey13	42.4	Top	0.00071794	0.00185936
Storey12	39.2	Top	0.00078669	0.00197127
Storey11	36	Top	0.0008506	0.00207026
Storey10	32.8	Top	0.00090952	0.0021556
Storey9	29.6	Top	0.00096314	0.00222634
Storey8	26.4	Top	0.00101081	0.00228094
Storey7	23.2	Top	0.00105122	0.00231672
Storey6	20	Top	0.00108187	0.00232878
Storey5	16.8	Top	0.00109791	0.00230793
Storey4	13.6	Top	0.00108997	0.00223652
Storey3	10.4	Top	0.00103917	0.00207941
Storey2	7.2	Top	0.00090854	0.00176728
Storey1	4	Top	0.00053728	0.00098856
Base	0	Top	0	0

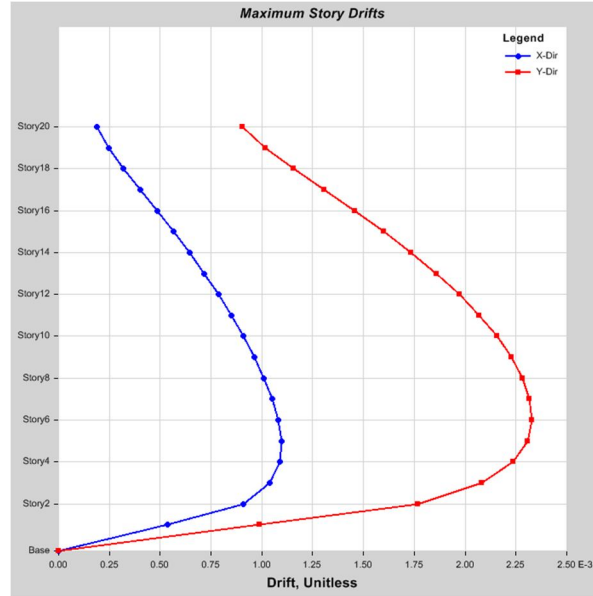


Fig. (4.25): Max storey drift for constant acceleration.

4.4.1.3 Across-wind Response

As the peak across-wind accelerations are considerably greater than the peak along-wind accelerations in this case, subsequently induce large shear, drift and displacements. This requires confirmation based on more studies as stated in literature, the across-wind response is presented in appendix C.

4.4.2 Discussion and comparison between nonlinear static and along-wind dynamic results

4.4.2.1 Maximum storey displacement

The comparison between max storey displacements in y-y direction for nonlinear static and dynamic analysis (p-delta-w) is presented in Table (4.19) and Figure (4.26). These show that there is considered difference between the static and dynamic results.

Table (4.19): Comparison between nonlinear static and dynamic displacements (along-wind):

Storey	Nonlinear Static disp. of P-delta-w(m)	Nonlinear Dynamic disp. (m)	Percentage difference (%)
20	0.0934	0.11384856	21.89
19	0.0905	0.1110809	22.74
18	0.0875	0.10796005	23.38
17	0.0842	0.10439659	23.99
16	0.0806	0.10035298	24.51
15	0.0768	0.09582662	24.77
14	0.0727	0.09083323	24.94
13	0.0683	0.08539889	25.03
12	0.0636	0.07955593	25.088
11	0.0587	0.07334097	24.94
10	0.0536	0.06679398	24.62
9	0.0482	0.05995834	24.39
8	0.0426	0.05288148	24.13
7	0.0369	0.04561661	23.62
6	0.031	0.03822618	23.31
5	0.025	0.03078863	23.15
4	0.019	0.0234118	23.22
3	0.0132	0.01625967	23.18
2	0.0077	0.00960814	24.78
1	0.0031	0.00395424	27.56
Base	0	0	0

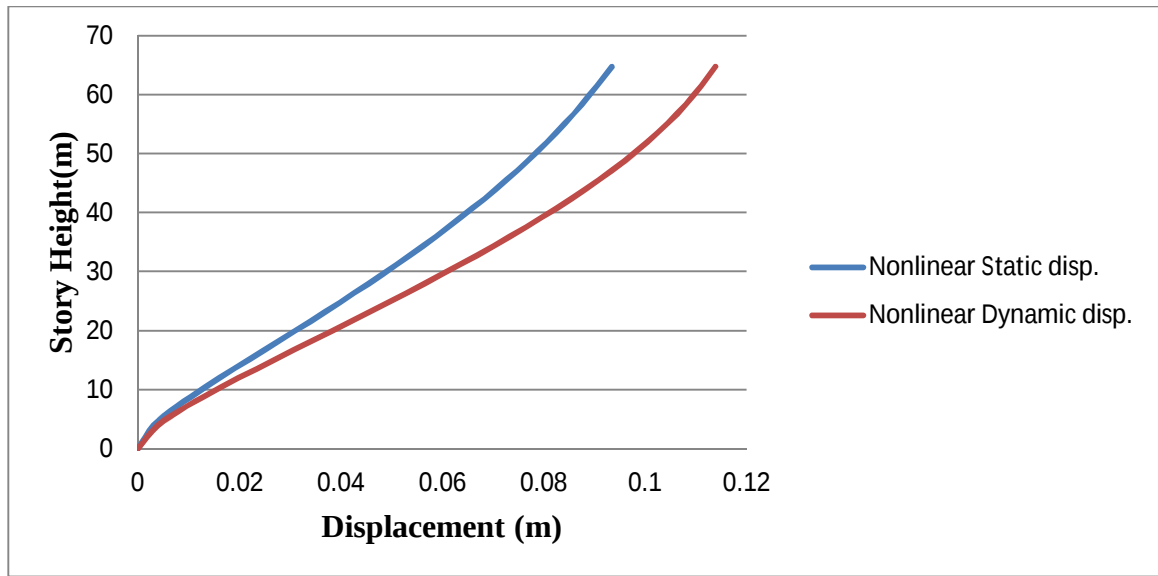


Fig. (4.26): Nonlinear static and dynamic displacements (along-wind).

In first storey, the actual values are very small. For example insignificant difference in 1st storey was about 0.0008 (27.56%). As can be see from the results obtained from Table (4.19) and Fig. (4.26), the percentage difference between nonlinear static and dynamic analysis displacements reached about 23%. It was found that the dynamic results always greater than the static once.

4.4.2.2 Maximum storey shear

The comparison between the maximum storey shear forces in y-direction for nonlinear static and dynamic analysis were given in Table (4.20). The maximum percentage difference is about 28% with the dynamic results less than the static results.

Table (4.20): Comparison between nonlinear static and dynamic storey shear forces at the top for along-wind:

Storey	Nonlinear Static Shear. (kN)	Nonlinear Dynamic Shear. (kN)	Percentage difference (%)
20	143.5017	104.9072	-26.89
19	231.5471	217.6913	-5.98
18	318.7619	323.7708	1.57
17	405.1165	422.9831	4.41
16	490.5782	515.3156	5.04
15	575.1112	600.9127	4.49
14	658.6763	680.0217	3.24
13	741.229	752.9194	1.58
12	822.7191	819.8441	-0.35
11	903.0883	880.953	-2.45
10	982.2687	936.3095	-4.68
9	1060.1797	985.8957	-7.01
8	1136.7236	1029.6389	-9.42
7	1211.7799	1067.4423	-11.91
6	1285.1959	1099.2147	-14.47
5	1356.771	1124.9043	-17.09
4	1426.2297	1144.5415	-19.75
3	1493.1656	1158.3043	-22.43
2	1556.9116	1166.6163	-25.07
1	1625.3513	1170.4024	-27.99
Base	0	0	0

The insignificant difference in 20th storey was about (27%). As can be see from the results obtained from Table (4.20) the percentage difference between nonlinear static and dynamic analysis shear reached about 28%.

4.4.2.3Maximum storey drift

The comparison between maximum storey drift in y-direction for nonlinear static and dynamic analysis are given in Table (4.21) and Fig.ure (4.27). The dynamic results are considerably greater than the static results.

Dynamic analysis despite resulting in less shear results in greater oscillating displacements, which affects comfort of occupants. This agrees with published findings that point out the importance of dynamic analysis as a check for comfort.

Table (4.21): Comparison between nonlinear static and dynamic storey (along-wind):

Storey	Nonlinear Static drift. (m)	Nonlinear Dynamic drift. (m)	Difference (%)
20	0.000769	0.00090498	15.03
19	0.000776	0.00101597	23.62
18	0.000836	0.00115459	27.59
17	0.000924	0.00130494	29.19
16	0.001014	0.00145544	30.33
15	0.001104	0.00159999	30.99
14	0.001193	0.00173524	31.25
13	0.001278	0.00185936	31.27
12	0.001358	0.00197127	31.11
11	0.001434	0.00207026	30.73
10	0.001503	0.0021556	30.27
9	0.001565	0.00222634	29.71
8	0.001619	0.00228094	29.02
7	0.00166	0.00231672	28.35
6	0.001686	0.00232878	27.60
5	0.001689	0.00230793	26.82
4	0.001667	0.00223652	25.46
3	0.001581	0.00207941	23.97
2	0.001368	0.00176728	22.59
1	0.000772	0.00098856	21.91
Base	0	0	0

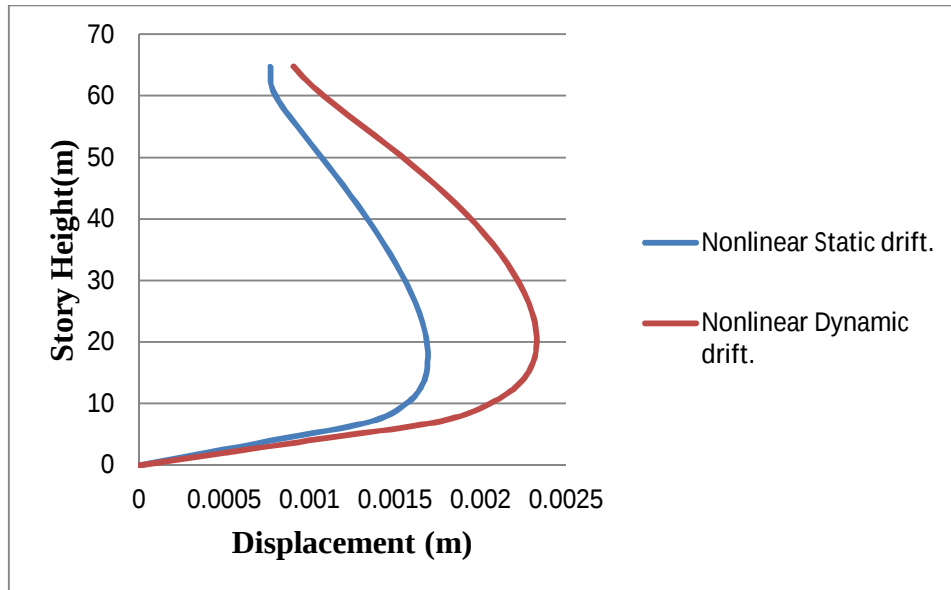


Fig. (4.27): comparison between nonlinear static and dynamic storey drift for along-wind.

Chapter Five

Conclusions and Recommendations

5.1 Conclusions:

Conclusions can be summarized as follows:

1. The wind forces were calculated using ASCE7-05 code of practice. The maximum wind force in y-direction is 55.95 KN at 19th floor.
2. A 20 stories tall building was analyzed for wind load dynamically (nonlinear) and statically using nonlinear P- Δ and non linear P- Δ plus large displacement using ETABS finite element program.
3. The results obtained for the four modes and six modes of dynamic analysis were analyzed and compared for horizontal displacements and shear forces.
4. The Six mode shapes give more accurate results in the dynamic analysis as shown in Table (4.11).
5. The difference between the linear static results and non linear static results for displacement was about 23% and was too large.
6. The values of displacement for the two non linear static modes were in very close agreement as shown in table (4.8) and Fig. (4.13).
7. The percentage difference between the nonlinear static results and nonlinear dynamic (along-wind) results for displacement was very large (max _displacement difference in between (21%-25%)).
8. As can be seen from appendix C the difference between the nonlinear dynamic for along-wind and cross-wind displacements in the upper storey was very large reached to 46.76%.

9. The value of shear in nonlinear static analysis in 1st storey was greater than nonlinear dynamic analysis with percentage difference of 28% for along-wind.
10. The value of shear in cross-wind was very large comparing to along-wind and that means cross-wind must be taken in to consideration. As can be seen from appendix C, this needs more investigate.
11. The values of displacements and shear forces after assuming constant acceleration in along-wind were the same results as for along wind results based on variable acceleration response spectrum.
12. The values of displacement and shear forces were very small in the x direction for all cases of static and dynamic (along-wind) analysis and were neglected.
13. The results from the dynamic analysis point out that there is a need for making dynamic analysis for wind loads, This also may affect the design of tall buildings, especially as a check of human comfort.

5.2 Recommendations

The following recommendations are suggested to be a guide for future studies:

1. Carrying out the study building by using another software programs for dynamic analysis.
2. Carrying out Dynamic analysis for other buildings with different structural systems.
3. Study of the effect of building height and flexibility of the dynamic analysis results.
4. Carrying out study to look into alternative acceptable methods to obtain across-wind dynamic response.

From the results obtained it is recommended to:

- 1- Use of P-Delta plus large displacement analysis for the type of tall building considered in this study to obtain the shear values.
- 2- Use of the dynamic analysis for the type of building considered to obtain displacements and check for comfort.
- 3- Use constant acceleration to obtain the dynamic analysis results.

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Appendix A

Table A.1

Wind Directionality Factor, K_d	
Table 26.6-1	
Structure Type	Directionality Factor K_d *
Buildings	
Main Wind Force Resisting System	0.85
Components and Cladding	0.85
Arched Roofs	0.85
Chimneys, Tanks, and Similar Structures	
Square	0.90
Hexagonal	0.95
Round	0.95
Solid Freestanding Walls and Solid Freestanding and Attached Signs	0.85
Open Signs and Lattice Framework	0.85
Trussed Towers	
Triangular, square, rectangular	0.85
All other cross sections	0.95

*Directionality Factor K_d has been calibrated with combinations of loads specified in Chapter 2. This factor shall only be applied when used in conjunction with load combinations specified in Sections 2.3 and 2.4.

Table A.2

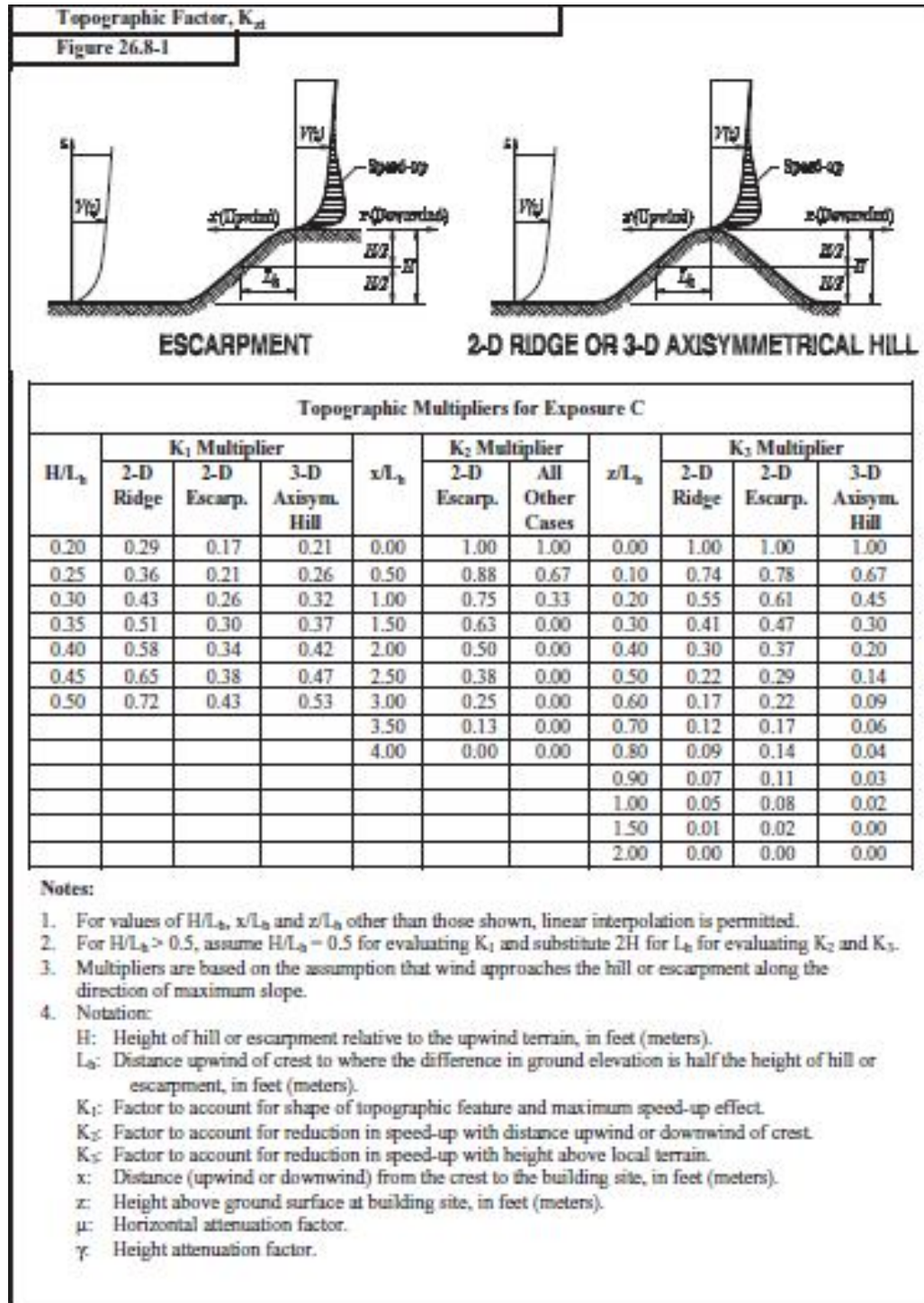


Table A.3

CHAPTER 26 WIND LOADS: GENERAL REQUIREMENTS

Terrain Exposure Constants

Table 26.9-1

Exposure	α	z_g (ft)	$\hat{\alpha}$	$\hat{b}$	$\bar{\alpha}$	$\bar{b}$	c	ℓ (ft)	$\bar{\epsilon}$	z_{min} (ft)*
B	7.0	1200	1/7	0.84	1/4.0	0.45	0.30	320	1/3.0	30
C	9.5	900	1/9.5	1.00	1/6.5	0.65	0.20	500	1/5.0	15
D	11.5	700	1/11.5	1.07	1/9.0	0.80	0.15	650	1/8.0	7

* z_{min} = minimum height used to ensure that the equivalent height $\bar{Z}$ is greater of $0.6h$ or z_{min} .
 For buildings with $h \leq z_{min}$, $\bar{Z}$ shall be taken as z_{min} .

In metric

Exposure	α	z_g (m)	$\hat{\alpha}$	$\hat{b}$	$\bar{\alpha}$	$\bar{b}$	c	ℓ (m)	$\bar{\epsilon}$	z_{min} (m)*
B	7.0	365.76	1/7	0.84	1/4.0	0.45	0.30	97.54	1/3.0	9.14
C	9.5	274.32	1/9.5	1.00	1/6.5	0.65	0.20	152.4	1/5.0	4.57
D	11.5	213.36	1/11.5	1.07	1/9.0	0.80	0.15	198.12	1/8.0	2.13

* z_{min} = minimum height used to ensure that the equivalent height $\bar{Z}$ is greater of $0.6h$ or z_{min} .
 For buildings with $h \leq z_{min}$, $\bar{Z}$ shall be taken as z_{min} .

Table A.4

Main Wind Force Resisting System and Components and Cladding		All Heights
Table 26.11-1	Internal Pressure Coefficient, (GC_{pi})	Walls & Roofs
Enclosed, Partially Enclosed, and Open Buildings		

Table A.5

Main Wind Force Resisting System – Part I		All Heights		
Velocity Pressure Exposure Coefficients, K_h and K_z				
Table 27.3-1				
Height above ground level, z		Exposure		
		B	C	D
ft	(m)			
0-15	(0-4.6)	0.57	0.85	1.03
20	(6.1)	0.62	0.90	1.08
25	(7.6)	0.66	0.94	1.12
30	(9.1)	0.70	0.98	1.16
40	(12.2)	0.76	1.04	1.22
50	(15.2)	0.81	1.09	1.27
60	(18)	0.85	1.13	1.31
70	(21.3)	0.89	1.17	1.34
80	(24.4)	0.93	1.21	1.38
90	(27.4)	0.96	1.24	1.40
100	(30.5)	0.99	1.26	1.43
120	(36.6)	1.04	1.31	1.48
140	(42.7)	1.09	1.36	1.52
160	(48.8)	1.13	1.39	1.55
180	(54.9)	1.17	1.43	1.58
200	(61.0)	1.20	1.46	1.61
250	(76.2)	1.28	1.53	1.68
300	(91.4)	1.35	1.59	1.73
350	(106.7)	1.41	1.64	1.78
400	(121.9)	1.47	1.69	1.82
450	(137.2)	1.52	1.73	1.86
500	(152.4)	1.56	1.77	1.89

Notes:

- The velocity pressure exposure coefficient K_z may be determined from the following formula:
For $15 \text{ ft} \leq z \leq z_g$ For $z < 15 \text{ ft}$.
 $K_z = 2.01 (z/z_g)^{\alpha}$ $K_z = 2.01 (15/z_g)^{\alpha}$
- α and z_g are tabulated in Table 26.9.1.
- Linear interpolation for intermediate values of height z is acceptable.
- Exposure categories are defined in Section 26.7.

Table A.6.1

Main Wind Force Resisting System – Part 1										All Heights		
Figure 27.4-1 (cont.)			External Pressure Coefficients, C_p							Walls & Roofs		
Enclosed, Partially Enclosed Buildings												
Wall Pressure Coefficients, C_p												
Surface		L/B		C_p		Use With						
Windward Wall		All values		0.8		q_z						
Leeward Wall		0-1		-0.5		q_h						
		2		-0.3								
		≥ 4		-0.2								
Side Wall		All values		-0.7		q_h						
Roof Pressure Coefficients, C_p , for use with q_h												
Wind Direction	Windward									Leeward		
	Angle, θ (degrees)									Angle, θ (degrees)		
	h/L	10	15	20	25	30	35	45	$\geq 60^\circ$	10	15	≥ 20
Normal to ridge for $\theta \geq 10^\circ$	≤ 0.25	-0.7 -0.18	-0.5 0.0*	-0.3 0.2	-0.2 0.3	-0.2 0.3	0.0* 0.4	0.4	0.01 θ	-0.3	-0.5	-0.6
	0.5	-0.9 -0.18	-0.7 -0.18	-0.4 0.0*	-0.3 0.2	-0.2 0.2	-0.2 0.3	0.0* 0.4	0.01 θ	-0.5	-0.5	-0.6
	≥ 1.0	-1.3** -0.18	-1.0 -0.18	-0.7 -0.18	-0.5 0.0*	-0.3 0.2	-0.2 0.2	0.0* 0.3	0.01 θ	-0.7	-0.6	-0.6
Normal to ridge for $\theta < 10^\circ$ and Parallel to ridge for all θ	≤ 0.5	Horiz distance from windward edge			C_p		*Value is provided for interpolation purposes. **Value can be reduced linearly with area over which it is applicable as follows					
		0 to $h/2$			-0.9, -0.18							
		$h/2$ to h			-0.9, -0.18							
		h to $2h$			-0.5, -0.18							
	≥ 1.0	$> 2h$			-0.3, -0.18		Area (sq ft) Reduction Factor					
		0 to $h/2$			-1.3**, -0.18							
		$> h/2$			-0.7, -0.18							

Notes:

- Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
- Linear interpolation is permitted for values of L/B , h/L and θ other than shown. Interpolation shall only be carried out between values of the same sign. Where no value of the same sign is given, assume 0.0 for interpolation purposes.
- Where two values of C_p are listed, this indicates that the windward roof slope is subjected to either positive or negative pressures and the roof structure shall be designed for both conditions. Interpolation for intermediate ratios of h/L in this case shall only be carried out between C_p values of like sign.
- For monoslope roofs, entire roof surface is either a windward or leeward surface.
- For flexible buildings use appropriate G_f as determined by Section 26.9.4.
- Refer to Figure 27.4-2 for domes and Figure 27.4-3 for arched roofs.
- Notation:
 B : Horizontal dimension of building, in feet (meter), measured normal to wind direction.
 L : Horizontal dimension of building, in feet (meter), measured parallel to wind direction.
 h : Mean roof height in feet (meters), except that eave height shall be used for $\theta \leq 10^\circ$ degrees.
 z : Height above ground, in feet (meters).
 G : Gust effect factor.
 q_z, q_h : Velocity pressure, in pounds per square foot (N/m^2), evaluated at respective height.
 θ : Angle of plane of roof from horizontal, in degrees.
- For mansard roofs, the top horizontal surface and leeward inclined surface shall be treated as leeward surfaces from the table.
- Except for MWFRS's at the roof consisting of moment resisting frames, the total horizontal shear shall not be less than that determined by neglecting wind forces on roof surfaces.

#For roof slopes greater than 80° , use $C_p = 0.8$

A.6.2

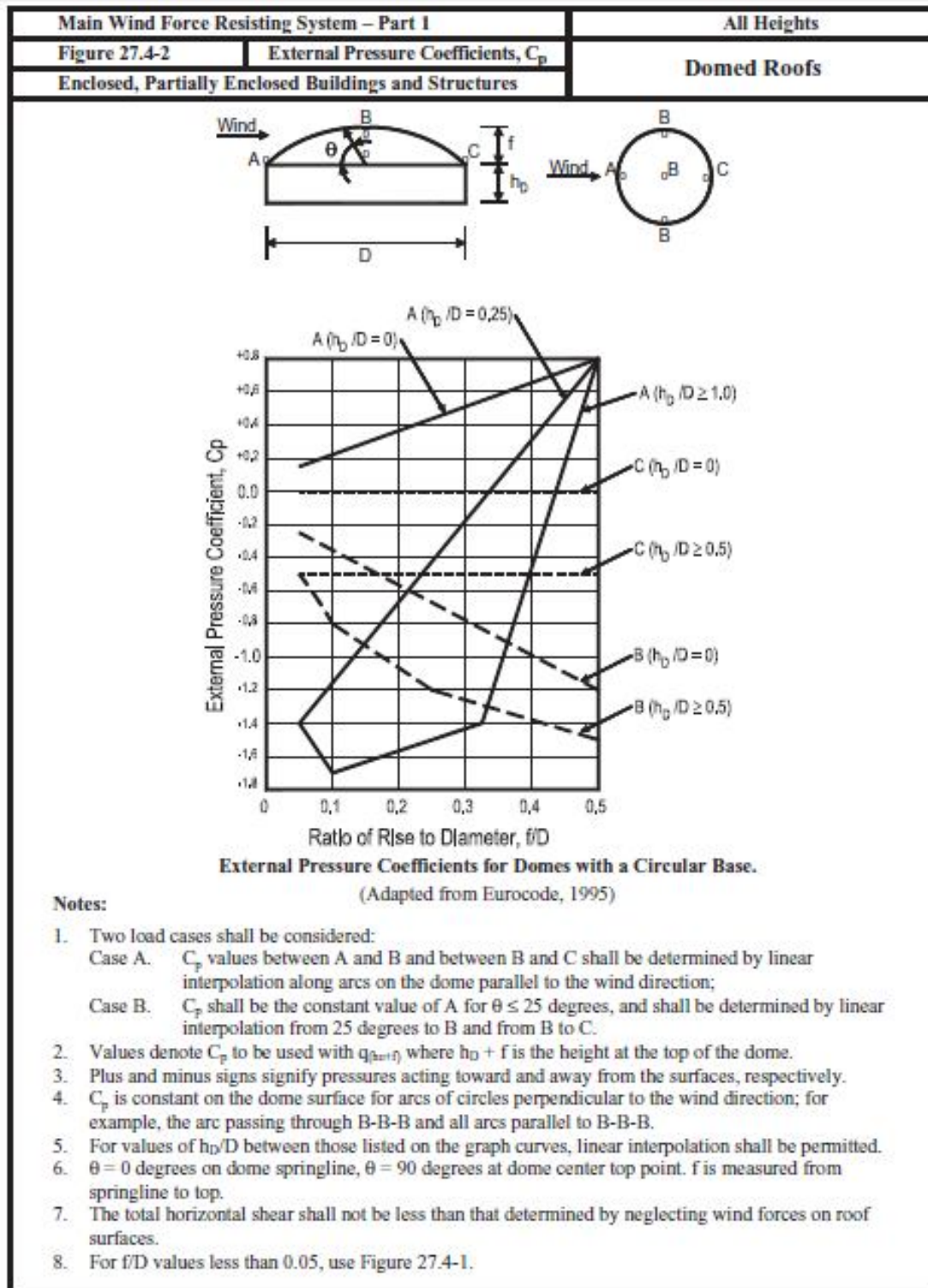


Table A.6.3

Main Wind Force Resisting System and Components and Cladding – Part I		All Heights		
Figure 27.4-3	External Pressure Coefficients, C_p	Arched Roofs		
Enclosed, Partially Enclosed Buildings and Structures				
Conditions	Rise-to-span ratio, r	C_p		
		Windward quarter	Center half	Leeward quarter
Roof on elevated structure	$0 < r < 0.2$	-0.9	$-0.7 - r$	-0.5
	$0.2 \leq r < 0.3^*$	$1.5r - 0.3$	$-0.7 - r$	-0.5
	$0.3 \leq r \leq 0.6$	$2.75r - 0.7$	$-0.7 - r$	-0.5
Roof springing from ground level	$0 < r \leq 0.6$	$1.4r$	$-0.7 - r$	-0.5

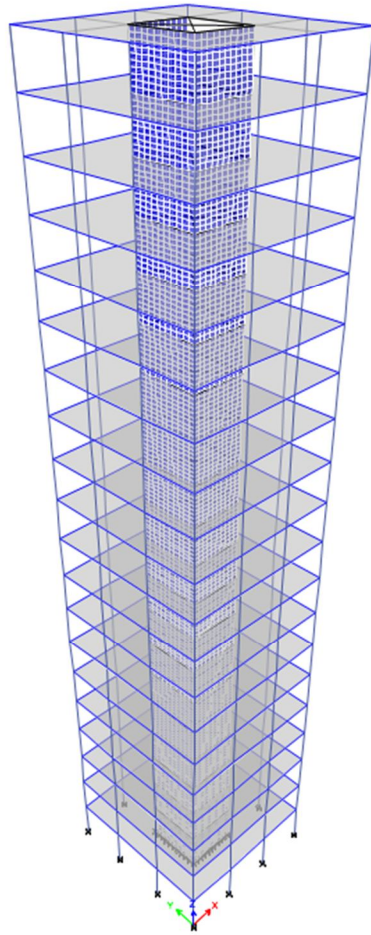
*When the rise-to-span ratio is $0.2 \leq r \leq 0.3$, alternate coefficients given by $6r - 2.1$ shall also be used for the windward quarter.

Notes:

- Values listed are for the determination of average loads on main wind force resisting systems.
- Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
- For wind directed parallel to the axis of the arch, use pressure coefficients from Fig. 27.4-1 with wind directed parallel to ridge.
- For components and cladding: (1) At roof perimeter, use the external pressure coefficients in Fig. 30.4-2A, B and C with θ based on spring-line slope and (2) for remaining roof areas, use external pressure coefficients of this table multiplied by 0.87.

Appendix B

ETABS[®] 2013
Integrated Building Design Software



Project Report

1 Structure Data

This chapter provides model geometry information, including items such as storey levels, point coordinates, and element connectivity.

1.1 Storey Data

Table 1.1 - Storey Data

Name	Height m	Elevation m	Master Storey	Similar To	Splice Storey
Storey20	3.2	64.8	Yes	None	No
Storey19	3.2	61.6	No	Storey20	No
Storey18	3.2	58.4	No	Storey20	No
Storey17	3.2	55.2	No	Storey20	No
Storey16	3.2	52	No	Storey20	No
Storey15	3.2	48.8	No	Storey20	No
Storey14	3.2	45.6	No	Storey20	No
Storey13	3.2	42.4	No	Storey20	No
Storey12	3.2	39.2	No	Storey20	No
Storey11	3.2	36	No	Storey20	No
Storey10	3.2	32.8	No	Storey20	No
Storey9	3.2	29.6	No	Storey20	No
Storey8	3.2	26.4	No	Storey20	No
Storey7	3.2	23.2	No	Storey20	No
Storey6	3.2	20	No	Storey20	No
Storey5	3.2	16.8	No	Storey20	No
Storey4	3.2	13.6	No	Storey20	No
Storey3	3.2	10.4	No	Storey20	No
Storey2	3.2	7.2	No	Storey20	No
Storey1	4	4	No	Storey20	No
Base	0	0	No	None	No

1.2 Grid Data

Table 1.2 - Grid Systems

Name	Type	Storey Range	X Origin m	Y Origin m	Rotation deg	Bubble Size m	Color
G1	Cartesian	Default	0	0	0	1.25	ffa0a0a0

Table 1.3 - Grid Lines

Grid System	Grid Direction	Grid ID	Visible	Bubble Location	Ordinate m
G1	X	A	Yes	End	0
G1	X	B	Yes	End	4
G1	X	C	Yes	End	8.5
G1	X	D	Yes	End	12.5
G1	Y	1	Yes	Start	0
G1	Y	2	Yes	Start	4
G1	Y	3	Yes	Start	8
G1	Y	4	Yes	Start	12

1.3 Mass

Table 1.3 - Mass Source

Include Elements	Include Added Mass	Include Loads	Lateral Only	Lump at Stories	Load Pattern	Multiplier
No	No	Yes	Yes	Yes	Dead	1

2 Properties

This chapter provides property information for materials, frame sections, shell sections, and links.

2.1 Materials

Table 2.1 - Material Properties - Summary

Name	Type	E kN/m ²	ν	Unit Weight kN/m ³	Design Strengths
4000Psi	Concrete	24855578.06	0.2	24	Fc=27579.03 kN/m ²
A615Gr60	Rebar	199947979	0.3	76.9729	Fy=413685.47 kN/m ² , Fu=620528.21 kN/m ²

2.2 Frame Sections

Table 2.2 - Frame Sections - Summary

Name	Material	Shape
beam 600*300	4000Psi	Concrete Rectangular
col 750*350	4000Psi	Concrete Rectangular

2.3 Shell Sections

Table 2.3 - Shell Sections - Summary

Name	Design Type	Element Type	Material	Total Thickness m
Slab1	Slab	Shell-Thin	4000Psi	0.22
Wall1	Wall	Shell-Thin	4000Psi	0.3

3 Loads

This chapter provides loading information as applied to the model.

3.1 Load Patterns

Table 4.1 - Load Patterns

Name	Type	Self Weight Multiplier	Auto Load
Dead	Dead	1	

Live	Live	0	
windx	Wind	0	ASCE 7-05
windy	Wind	0	ASCE 7-05

ASCE 7-05 Auto Wind Load Calculation

This calculation presents the automatically generated lateral wind loads for load pattern windx according to ASCE 7-05, as calculated by ETABS.

Exposure Parameters

Exposure From = Diaphragms

Exposure Category = B

Wind Direction = 0 degrees

Basic Wind Speed, V [ASCE 6.5.4]

V = 100 mph

Windward Coefficient, $C_{p,wind}$ [ASCE 6.5.11.2.1]

$C_{p,wind} = 0.8$

Leeward Coefficient, $C_{p,lee}$ [ASCE 6.5.11.2.1]

$C_{p,lee} = 0.491667$

Wind Case = All Cases

Top Storey = Storey20

Bottom Storey = Base

Include Parapet = Yes, Parapet Height = 1.5

Factors and Coefficients

Gradient Height, z_g [ASCE Table 6-2]

$z_g = 1200$

Emperical Exponent, α [ASCE Table 6-2]

$\alpha = 7$

Velocity Pressure Exposure Coefficient, K_z , [ASCE Table 6-3]

$$K_z = 2.01 \left(\frac{z}{z_g} \right)$$

for $15\text{ft} \leq z \leq z_g$

$$K_z = 2.01 \left(\frac{15}{z_g} \right)$$

for $z \leq 15\text{ft}$

Topographical Factor, K_{zt} [ASCE 6.5.7.2]

$K_{zt} = 1$

Directionality Factor, K_d [ASCE 6.5.4.4]

$K_d = 0.85$

Importance Factor, I [ASCE 6.5.5]

I = 1

Gust Effect Factor, G [ASCE 6.5.8]

G = 0.85

Lateral Loading

Velocity Pressure, q_z [ASCE 6.5.10 Eq. 6-15]

$$q_z = 0.00256 K_z K_{zt} K_d V^2 I$$

Design Wind Pressure, p [ASCE 6.5.12.2.1 Eq. 6-17]

$$p = q G C_{p,wind} + q_h (G C_{p,lee})$$

4 Modal Results

Table 4.16 - Modal Periods and Frequencies

Case	Mode	Period sec	Frequency cyc/sec	Circular Frequency rad/sec	Eigenvalue rad ² /sec ²
Modal	1	2.492	0.401	2.5217	6.3588
Modal	2	2.157	0.464	2.9125	8.4824
Modal	3	1.626	0.615	3.8637	14.928
Modal	4	0.749	1.334	8.3833	70.2804
Modal	5	0.53	1.888	11.8609	140.6818
Modal	6	0.428	2.335	14.6702	215.2157
Modal	7	0.401	2.492	15.6583	245.1818
Modal	8	0.275	3.639	22.8652	522.8169
Modal	9	0.229	4.359	27.3896	750.1919
Modal	10	0.198	5.056	31.7674	1009.167

Appendix C

C.1 Across-wind Response

In this case also 6 modes used and the max storey displacements in x-direction (u_1) were given in Table (C.1) and Figure (C.1). The maximum storey displacement in 20th storey (0.214m) was very large compared to the nonlinear static analysis results (0.093m).

Also the max storey shear in x-direction(u_1) were given in Table (C.2) and Fig.(C.2), the maximum storey shear in 1st storey (4093.8184KN) was very large comparing with the nonlinear static analysis results (1625.18 kN).Also the maximum storey drifts were given in Table (C.3) and Fig.(C.3).

Table (C.1): Maximum storey displacements in x-direction for 6 modes (cross-wind):

Storey	Elevation	Location	Displacement X-direction
	m		m
Storey20	64.8	Top	0.2139076
Storey19	61.6	Top	0.20323494
Storey18	58.4	Top	0.19232221
Storey17	55.2	Top	0.18110483
Storey16	52	Top	0.16953406
Storey15	48.8	Top	0.157599
Storey14	45.6	Top	0.14531741
Storey13	42.4	Top	0.13273243
Storey12	39.2	Top	0.11990964
Storey11	36	Top	0.10693505
Storey10	32.8	Top	0.09391408
Storey9	29.6	Top	0.08097129
Storey8	26.4	Top	0.06825093
Storey7	23.2	Top	0.05591822

Storey	Elevation	Location	Displacement X-direction
	m		m
Storey6	20	Top	0.04416145
Storey5	16.8	Top	0.03319491
Storey4	13.6	Top	0.02326285
Storey3	10.4	Top	0.01464456
Storey2	7.2	Top	0.00766092
Storey1	4	Top	0.00268497
Base	0	Top	0

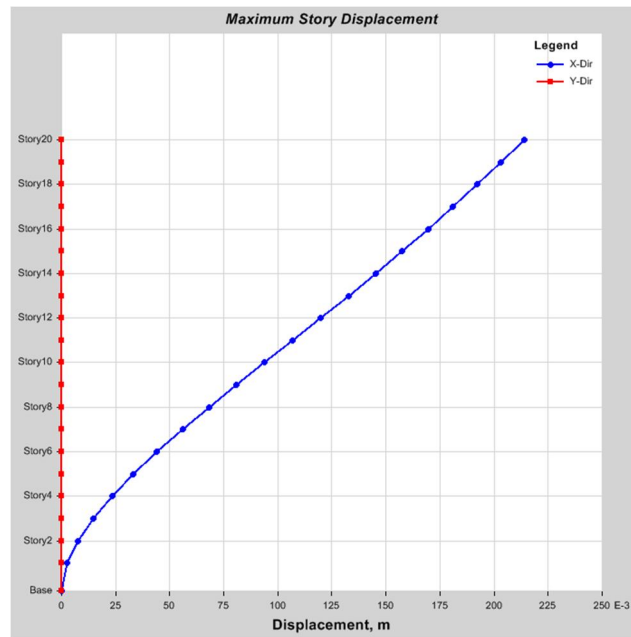


Fig. (C.1): Maximum storey displacement x-direction for 6 modes in cross-wind.

Table (C.2): Max storey shear in x-direction for 6 modes in cross-wind:

Storey	Elevation	Location	Shear in X-direction
	M		kN
Storey20	64.8	Top & bottom	405.7749

Storey	Elevation	Location	Shear in X-direction
	M		kN
Storey19	61.6	Top & bottom	832.0504
Storey18	58.4	Top & bottom	1223.2238
Storey17	55.2	Top & bottom	1580.4805
Storey16	52	Top & bottom	1905.5706
Storey15	48.8	Top & bottom	2200.8254
Storey14	45.6	Top & bottom	2468.9887
Storey13	42.4	Top & bottom	2712.9233
Storey12	39.2	Top & bottom	2935.2523
Storey11	36	Top & bottom	3138.0156
Storey10	32.8	Top & bottom	3322.4215
Storey9	29.6	Top & bottom	3488.7541
Storey8	26.4	Top & bottom	3636.4658
Storey7	23.2	Top & bottom	3764.447
Storey6	20	Top & bottom	3871.4276
Storey5	16.8	Top & bottom	3956.4507
Storey4	13.6	Top & bottom	4019.349
Storey3	10.4	Top & bottom	4061.1649
Storey2	7.2	Top & bottom	4084.4663
Storey1	4	Top & bottom	4093.8184
Base	0	Top & bottom	0

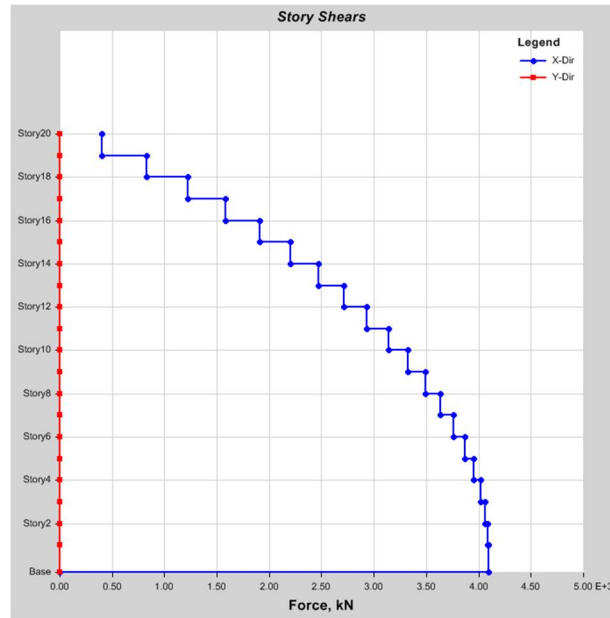


Fig. (C.2): Maximum storey shear x-direction for 6 modes in cross-wind.

Table (C.3): Maximum storey drift in x-direction for 6 modes in across-wind:

Storey	Elevation	Location	Drift in X-direction
	M		
Storey20	64.8	Top	0.00335076
Storey19	61.6	Top	0.0034281
Storey18	58.4	Top	0.00352535
Storey17	55.2	Top	0.00363721
Storey16	52	Top	0.00375158
Storey15	48.8	Top	0.00385943
Storey14	45.6	Top	0.00395285
Storey13	42.4	Top	0.00402503
Storey12	39.2	Top	0.00406982
Storey11	36	Top	0.00408145
Storey10	32.8	Top	0.0040542
Storey9	29.6	Top	0.00398212
Storey8	26.4	Top	0.0038588
Storey7	23.2	Top	0.00367711
Storey6	20	Top	0.0034289

Storey	Elevation	Location	Drift in X-direction
	M		
Storey5	16.8	Top	0.00310479
Storey4	13.6	Top	0.00269372
Storey3	10.4	Top	0.00218261
Storey2	7.2	Top	0.00155507
Storey1	4	Top	0.00067124
Base	0	Top	0

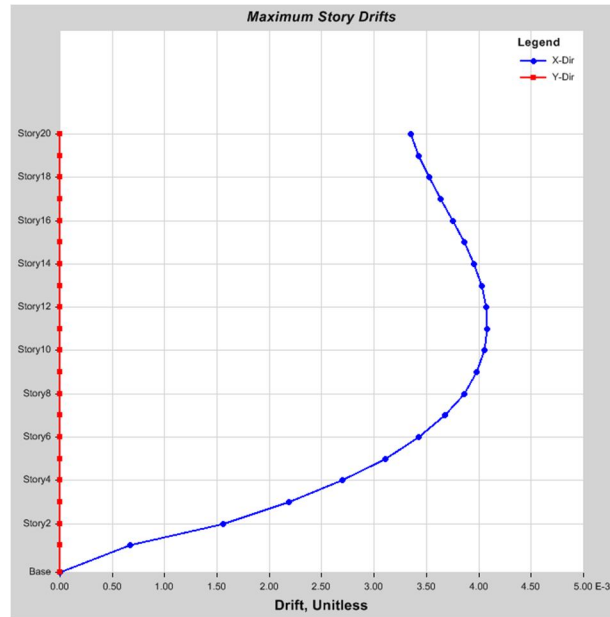


Fig. (C.4): Maximum storey drifts x-direction for 6 modes in cross-wind.

The peak across-wind accelerations were therefore considerably greater than the peak along-wind accelerations in this case, subsequently induce large shear, drift and displacements.

C.2 Cross-wind and along-wind response comparison

C.2.1 Maximum storey displacement

The comparison between the along-wind (y-direction) and cross-wind (x-direction) displacements for nonlinear dynamic analysis was given in Table (C.4) and Figure (C.4). These show that there was considered difference between the along-wind and cross-wind results.

Table (C.4): Difference between the along-wind and cross-wind displacements:

Storey	Along-wind disp.(m) (y-direction)	Across-wind disp.(m) (x-direction)	Difference (%)
20	0.11384856	0.2139076	46.78
19	0.1110809	0.20323494	45.34
18	0.10796005	0.19232221	43.87
17	0.10439659	0.18110483	42.36
16	0.10035298	0.16953406	40.81
15	0.09582662	0.157599	39.20
14	0.09083323	0.14531741	37.49
13	0.08539889	0.13273243	35.66
12	0.07955593	0.11990964	33.65
11	0.07334097	0.10693505	31.42
10	0.06679398	0.09391408	28.88
9	0.05995834	0.08097129	25.95
8	0.05288148	0.06825093	22.52
7	0.04561661	0.05591822	18.42
6	0.03822618	0.04416145	13.44
5	0.03078863	0.03319491	7.25
4	0.0234118	0.02326285	-0.64

3	0.01625967	0.01464456	-11.03
2	0.00960814	0.00766092	-25.42
1	0.00395424	0.00268497	-47.27
Base	0	0	0

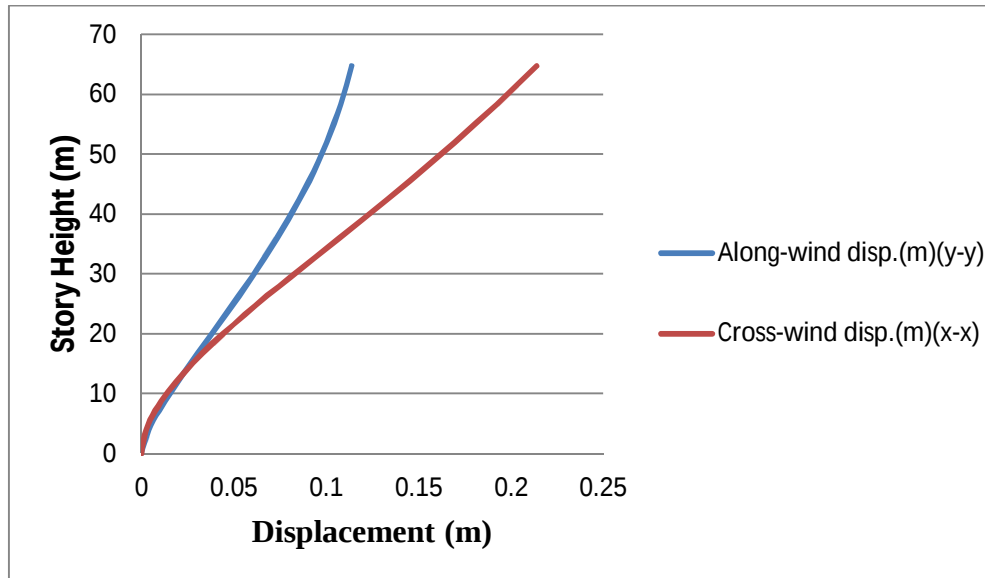


Fig. (C.4):The along-wind and cross-wind displacements.

C.2.2Max storey shear

The comparison between along-wind (y-direction) and cross-wind(x-direction) shear for nonlinear dynamic analysis were given in Table (C.5). These show that there was considered difference between the along-wind and cross-wind shear results reached to 72%.

Table (C.5): The along-wind and across-wind storey shear forces:

Storey	Along-wind Shear.(y-y) (kN)	Across-wind Shear.(x-x) (kN)	Difference (Along/ cross) (%)
20	104.9072	405.7749	74.15

19	217.6913	832.0504	73.84
18	323.7708	1223.2238	73.53
17	422.9831	1580.4805	73.24
16	515.3156	1905.5706	72.96
15	600.9127	2200.8254	72.70
14	680.0217	2468.9887	72.46
13	752.9194	2712.9233	72.25
12	819.8441	2935.2523	72.07
11	880.953	3138.0156	71.93
10	936.3095	3322.4215	71.82
9	985.8957	3488.7541	71.74
8	1029.6389	3636.4658	71.69
7	1067.4423	3764.447	71.64
6	1099.2147	3871.4276	71.61
5	1124.9043	3956.4507	71.57
4	1144.5415	4019.349	71.52
3	1158.3043	4061.1649	71.48
2	1166.6163	4084.4663	71.44
1	1170.4024	4093.8184	71.41
Base	0	0	0

C.2.3 Maximum storey drift

The comparison between along-wind (y-direction) and cross-wind (x-direction) storey drift for nonlinear dynamic analysis were given in Table (C.6) and Figure (C.5).

Table (C.6): The along-wind and across-wind storey drifts:

Storey	Along-wind drift. (y-direction)	Across-wind drift. (x-direction)	Difference (Along/ cross) (%)
20	0.00090498	0.00335076	72.99
19	0.00101597	0.0034281	70.36
18	0.00115459	0.00352535	67.25
17	0.00130494	0.00363721	64.12
16	0.00145544	0.00375158	61.20
15	0.00159999	0.00385943	58.54
14	0.00173524	0.00395285	56.10
13	0.00185936	0.00402503	53.81
12	0.00197127	0.00406982	51.56
11	0.00207026	0.00408145	49.28
10	0.0021556	0.0040542	46.83
9	0.00222634	0.00398212	44.09
8	0.00228094	0.0038588	40.89
7	0.00231672	0.00367711	36.99
6	0.00232878	0.0034289	32.08
5	0.00230793	0.00310479	25.67
4	0.00223652	0.00269372	16.97
3	0.00207941	0.00218261	4.73
2	0.00176728	0.00155507	-13.65
1	0.00098856	0.00067124	-47.27
Base	0	0	0

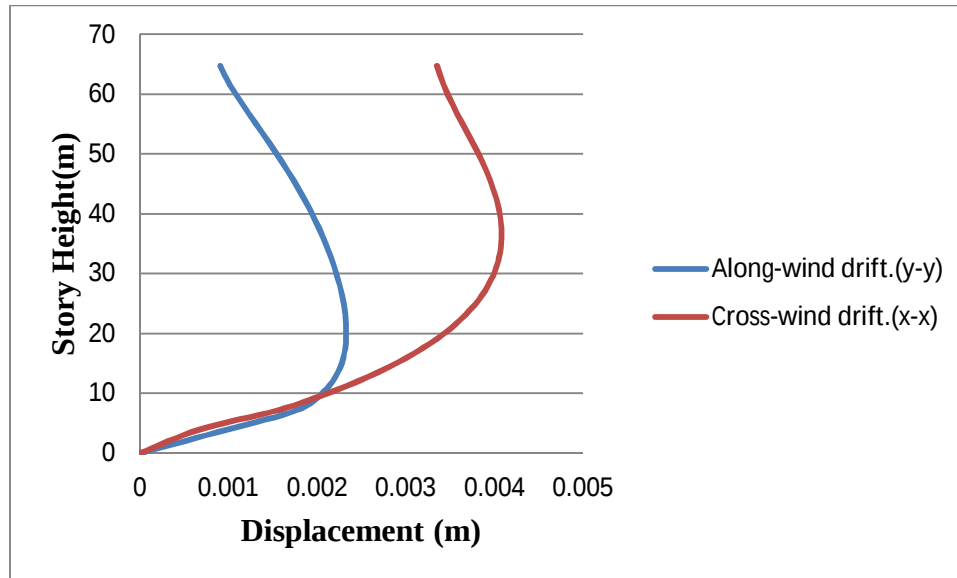


Fig. (C.5) Along-wind and cross-wind storey drift.

It's clear that there are pronounced differences between the along-wind and cross-wind in terms of displacements shear and drift .and that because the accelerations of a cross-wind are greater than along-wind, subsequently induces large response.