Chapter one

General Introduction

1-1Introduction

Nonlinear behavior may arise because of time-dependent or time-independent material non-linearity or because of large displacements that alter the shape of the structure so that applied loads alter their distribution or magnitude. Non-linearity may also be associated with smaller displacement, as in the problems of contact stress and flat plates whose displacements exceed their thickness. Non-linearity may be mild or severe, the problem may be static or dynamic, and the buckling may not be involved.

For the structural engineer the major difference between low and tall buildings is the influence of the wind forces on the behavior of the structural element Generally, it can be stated that a tall building structure is one in which the horizontal loads are an important factor in the structural in the structural design. In terms of lateral deflections a tall concrete building is one in which the structure, sized for gravity loads only, will exceed the allowable sway due to additionally applied lateral loads this allowable drift is set by the code of practice if the combined horizontal and vertical loads cause excessive bending moments and shear forces the structural system must be augmented by additional bracing elements these could take several forms .Cross-sections of existing beams and columns can be enlarged or efficient lateral-load-resisting bents such as concrete shear walls can be added to the structure.

Though there is increase and magnitude and frequency of seismic and wind load in many parts of the Sudan, building and structures are not designed and constructed in compliance with earthquake and wind loads provisions, or given any consideration for earthquake effects, the main difficulty being that the loads resulting from wind and seismic effects have not been determined yet. Thus it is important to determine such loads and then propose methods for the evaluation of their effects on the design and construction of tower.

Tall building cannot be defined in specific terms related just to height or to the number of floors. The height of a building is matter of a person‘s or community‘s circumstances and their consequent perception, Therefore, a measurable definition of 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. The influence of these actions must therefore be considered from the very beginning of the design process.

1-2 Research Problem Statement:

So as to accurately analyses high-rise building under the effect wind load . Nonlinear analysis methods must be used. The two main nonlinear methods are the P. ∆ and the P. ∆ with large displacement.

In this research a high-rise building is analyzed using linear, nonlinear P. ∆ and P. ∆ with large displacement methods.

The results obtained are then analyzed discussed and compared

1-3 Research Objectives

1-To know the special provisions for the design of framed tall building under wind loading

2-To determine the wind loads and their effects on tall building construction In Khartoum area from previous studies

3-To apply computer program for the linear and nonlinear analysis of specific tall building under wind load

4 - To verify the linear analysis result by comparison with known solution and study the effect on the nonlinear analysis

5- To compare the linear and nonlinear analysis results and drowns conclusions and recommendation.

1-4 Methodology

Comprehensive literature review [analysis and Design of Framed type Tall building under cyclic loads (wind loading) linear and nonlinear]

Case study linear and nonlinear analysis using ETABS Analysis and Discussion of the results obtained [comparing with known solutions for verificatar and studying effect of no linearity].

Drawing conclusions and presenting recommendations and writing up thesis

1-5 Outlines of thesis

Chapter one presents a brief description, the research problem the objectives of the research, the methodology and outlines of thesis.

Chapter two is a literature review that covers previous work in the research area.

Chapter three is concerned**,** Methods of Analysis of Tall Building under Wind Load and dynamic response to wind load using equations.

Chapter four is concerned with analysis and result of dynamic resistant tall building (under wind load) , and application using computer program ETABS v13.

Chapter five is concerned with conclusions and recommendation.

Chapter Two

Literature review

2-1 Introduction

The term dynamic may be defined simply as time varying; thus a dynamic load is any load of which the magnitude, the direction, or position varies with time. Similarly, the structural response to dynamic load, i.e., the resulting deflection and stresses, is also time-varying or dynamic.

Two basically different approaches are available for evaluating structural response to dynamic loads: deterministic and nondeterministic. The choice of method depends upon how the loading is defined, If the time variation of loading is fully known, it will be referred to herein as a prescribed dynamic loading, and the analysis of structural response is defined as a deterministic analysis. On the other hand, if the time variation is not completely known but can be defined in statistical sense, the loading is termed a random dynamic loading. A nondeterministic analysis correspondingly is analysis of response to a random dynamic loading [1]

In general, the structural response to any dynamic loading is expressed basically in terms of the displacements of the structure. Those deterministic analysis leads to displacement – time history corresponding to the prescribed loading history; other aspects of the deterministic structural response, such as stresses, strain, internal forces, etc., are usually response, such obtained as a secondary phase of analysis, from the previously established displacements patterns [1]

All existing buildings and most of the infrastructures are not designed to resist seismic forces. The subsoil conditions within most parts of the Sudan are vulnerable to liquefaction under moderate to high earthquakes. Very limited work has been done in the area of seismic hazard assessment [2], they making use of the historical and instrumental earthquake catalogue of the Sudan, made the first trial in assessing seismic hazard of the Sudan based on probabilistic approach. Their study resulted in several seismic hazard maps of the Sudan based on probable peak ground acceleration for time of exposure of 50, 100, 200 and 500 years. Their study is attempted to provide the design engineers and architects with a criteria to help them design structures subjected to earthquake loads [2].

As stated [2], it is important to understand the expressed objectives and purposes of particular design documents in order to understand it is provisions, for some earthquake codes protection of life is the only objective while in the other codes protection of property is justified from an economical point of view when the return period of damaging earthquakes is short, while it is not justified in locations with long return periods [2].

As stated by Bryan Stafford Smith [3], that in the design process, a thorough knowledge of high-rise structural components and their modes of behavior is a prerequisite to devising an appropriate load resisting system, such a system must be efficient, economic, and should minimize the structural penalty for height while maximizing the satisfaction of the basic serviceability requirement. With the increasing availability of general-purpose structural analysis programs, the formation of a concise and properly representative model has become an important part of tall building analysis. This also requires a fundamental knowledge of structural behavior [3].

Tall buildings, now approaching 500 m in height, project well into the atmospheric boundary layer, and their upper levels may experience the highest winds of large-scale windstorms, such as tropical cyclones. Resonant dynamic response in along-wind, cross-wind and torsional modes are a feature of the overall structural loads experienced by these structures. Extreme local cladding pressures may be experienced on their side walls. John D. Holmes [4]).

2-2 Wind Loading

It is shown in the literature that the lateral loading due to wind or earthquake is the major factor that causes of high-rise buildings to differ from those of low-to medium-rise building. For building of up to about 10 stories and of typical properties, the design is rarely affected by wind loads .Above this height, however, the increase in size of the structural members, and the possible rearrangement of the structure to account for wind loading, incurs accost premium that increase the progressively with height. With innovations in architectural treatment, in the strengths of materials and advance in methods of analysis, tall building structures have become more efficient and lighter and, consequently, more prone to deflect and even to sway under wind loading . This served as spur to research, which has produced significant advances in understanding the nature of wind loading and developing methods for its estimation. These developing have been mainly in experimental and theoretical techniques for determining the increase in wind loading due to gusting and the dynamic interaction of with gust force.[3]

There are some conditions prevailing locally which must be taken into account when designing a high-rise building. The buildings surroundings, for example, have an important effect on the active wind and flow conditions. The location of the building on open ground or surrounded by other high-rise building – has a massive influence on the wind profile. The effect of wind separating off the edges of neighboring building, reduced wind velocities due to obstacles at ground level and effects similar to friction or deflection of the wind loads due to neighboring building cannot be taken into account in the standard loads. Local wind effects have repeatedly been observed in the canyons formed by skyscrapers in large cities. One such effect is known as the “spinning effect”, a tornado-like effect near ground level which affects pedestrians.[2]

Also the shape of the building is another factor influencing the wind forces actually at work. When wind meets an obstacle, it normally generates compressive forces on the wind word side of the building and suction forces on the leeward side. In addition, air streaming around the building produces suction forces on the sides parallel to the wind direction. The shape of the corners and edges of the building is particularly important. Separation effects can cause suction and compressive forces several times greater than the original dynamic pressure. The magnitude of these edge and corner forces depends primarily on the geometry of the building round which air flows. Basically, it may be said that the more sharp-edge and irregular the building is, the more irregularly the wind forces will be distributed. Suction forces cause major problems around the roof in particular. If the roof structure has not been adequately anchored, parts of the roof may be lifted off and catapulted away unhindered. In addition to the roof, such elements as light-metal facades antennas, promotional signs and water tanks are some of the parts most seriously threatened by wind on high-rise building.[2]

As tall-building projects push the envelope to greater heights, designers are faced with the task of not only choosing a structural system to carry the lateral loads, but also insuring a design that meets serviceability and occupant comfort requirements under complex wind environments. This latter issue strongly affects the potential economic viability of tall building projects .An additional limitation tall building design is the inability to provide accurate estimates of structural damping in the design phase, which is critical to insure that the structure can meet both serviceability and habitability requirements. Although the building stiffness may be accurately quantified, inherent damping values are typically assumed in the design stage, resulting in estimates of response characteristics that may have significant inaccuracies. Thus, the accurate prediction of inherent damping for a given design becomes yet another critical consideration.[2]

It is necessary to take account certain precaution during construction. The loss potential during construction is an aspect which cannot be neglected. Although the stability of the building the during the various construction phases is documented by corresponding structural analysis, such equipment items as façade elements or temporary structures are usually not taken into account. Additional precautions must therefore be taken during construction work, particularly if the contractor is given sufficient advance warning of an impending windstorm. A loss of more than 5million was incurred construct ruction of a 90-storey high-rise building. Subcontractors had temporarily stored such electrical installation material as control cabinets and relays on the upper floors of the building shell, but delivery bottlenecks led to delay in assembly of the façade element on these floors.[2]

2.3 Structural Form

Form the structural engineer‛s point of view, the determination of the structural form of a high-rise building would ideally involve only the selection 0f arrangement of the major structural element to resist most efficiently the various combinations of gravity and horizontal loading. In reality, however, the choice of structural form is usually strongly influenced by other than structural considerations. The range of factors than has to be taken into account in deciding the structure form includes the internal planning, the material and method 0f construction, the external architectural treatment, the planned location and routing of the service systems, the nature and magnitude of the horizontal loading, and the height and important the structure factors become, and the more necessary it is to choose an appropriate structural from

One of the structure forms is braced-frame structure, in braced frames the lateral resistance of the structure is provided by diagonal members that, together with the girders, form the “web” of the vertical truss, with the columns acting as the “chords”(Fig.2.1) because the horizontal shear on the building is resisted by the horizontal components of the axial tensile or compressive actions in the web members, bracing systems are highly efficient in resisting lateral loads[3].

Bracing is generally regarded as an exclusively steel system because the diagonal are inevitably are subjected to tension for one or the other directions of lateral loading. Concrete bracing of the double diagonal form is sometimes used, with each diagonal designed as compression member to carry the full external shear.[3]

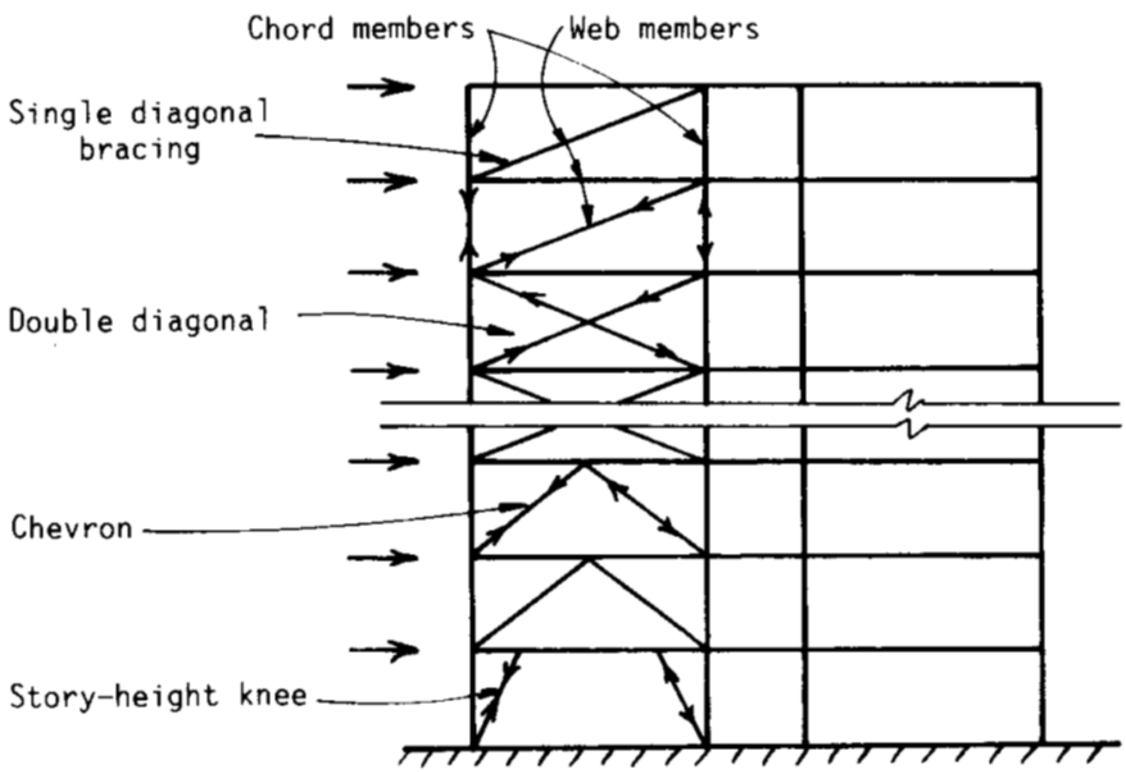

Fig 2.1 braced frame-showing different types of bracing

Consequently, the floor framing design is independent of its in level in the structure and, therefore, can be repetitive up the height of the building with obvious economy in design and fabrication. A major disadvantage of diagonal bracing is that it obstructs the internal planning and the location of windows and doors. For this reason, braced bents are usually incorporated internally along wall and partition lines, and especially around elevator, stair, and service shafts. Another drawback is that the diagonal connections are expensive to fabricate and erect. [3]

The traditional use of bracing has been in story-height, bay-width modules that carefully concealed in the finished building. More recently, however, external larger scale bracing extending over many stories and bays (Fig.2.2), has been used to produce not only produce not only highly efficient structures, but aesthetically attractive building [3]

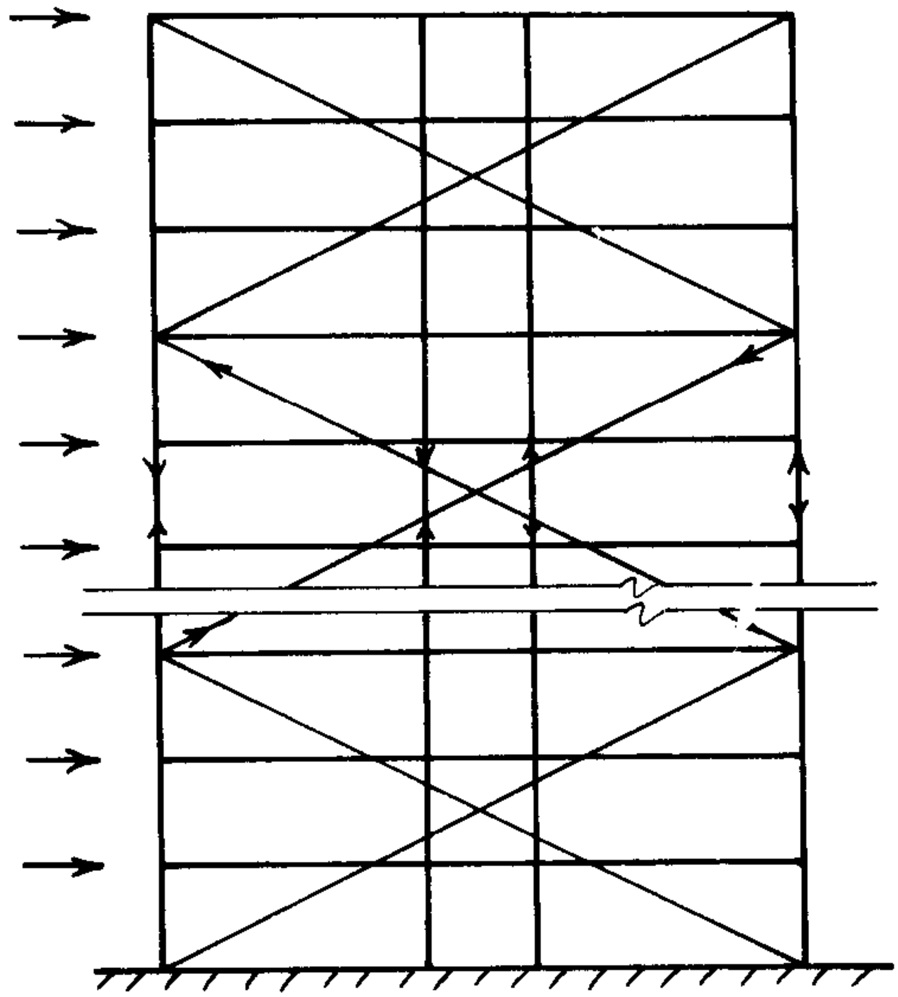

Fig 2. 2 large-scale braced frame

The next type of the structural forms is rigid frame, rigid-frame structure consist of columns and girders joined by moment-resistant connections. The lateral stillness of a rigid-frame ben depends on the bending stiffness of the columns, girders, and connections in the plane of the bent (Fig.2.3). The rigid frame's principal advantage is its open rectangular arrangement. If used as the only source of lateral resistance in a building, in its typical 20 ft(6m)—3o ft(9m) bay size, rigid framing is economic only for building up to about 25 stories. Above 25 stories the relatively high lateral flexibility of the frame calls for uneconomically large members in order to control the drift [3]

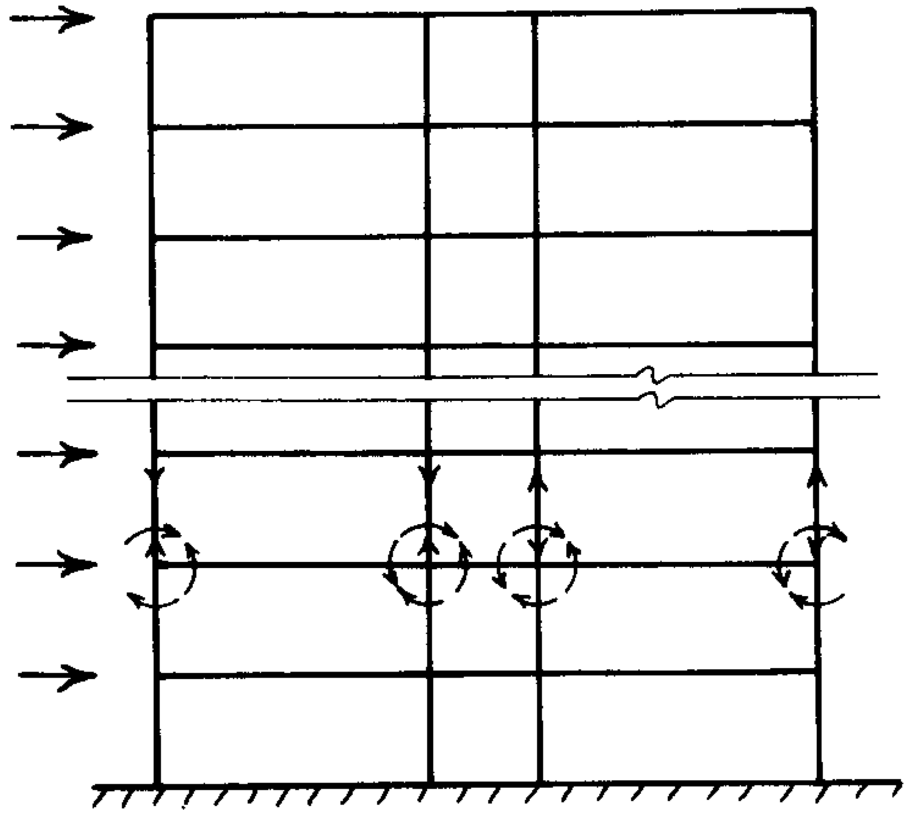

Fig 2. 3 rigid frame

Gravity loading also is resisted by the rigid-frame action. Negative, moments are induced in the girders adjacent to the columns causing the mid-span positive moments to be significantly less than in a simply supported span. While rigid frames of a 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 bents, can be economic up to much greater heights [3].

Another type of structural forms as shown in [3] is the infilled-frame structure. In many countries infilled frames are the most usual form of construction for tall buildings of up to 30 stories in height. Column and girder framing of reinforced concrete, or sometimes steel, is infilled by panels of brickwork, block work, or cast-in-place concrete [3].

When an infilled frame is subjected to lateral loading, the infill behaves effectively as a strut along its compression diagonal to brace the frame (Fig. 2.7). Because the infills serve also as external walls or internal partitions, the system is an economical way of stiffening and strengthening the structure. The complex interactive behavior of the infill in the frame, and the rather random quality of masonry, has made it difficult to predict with accuracy the stiffness and strength of an infilled frame, and there is no method of analyzing infilled frames for their design has gained general acceptance. For these reasons, and because of the fear of the unwitting removal of bracing infills at some time in the life of the building, the use of the infills for bracing tall buildings has mainly been supplementary to the rigid-frame action of concrete frames [3].

Also on of the structural forms [3], is the shear wall structure. Concrete or masonry continuous vertical walls may serve both architecturally as partitions and structurally to carry gravity and lateral loading. Their very high in plane stiffness and strength makes them ideally suited for bracing tall buildings.

In a shear wall structure, such walls are entirely responsible for the lateral load resistance of the building. They act as vertical cantilevers in the form of separate planar walls, and as nonplanar assemblies of connected walls

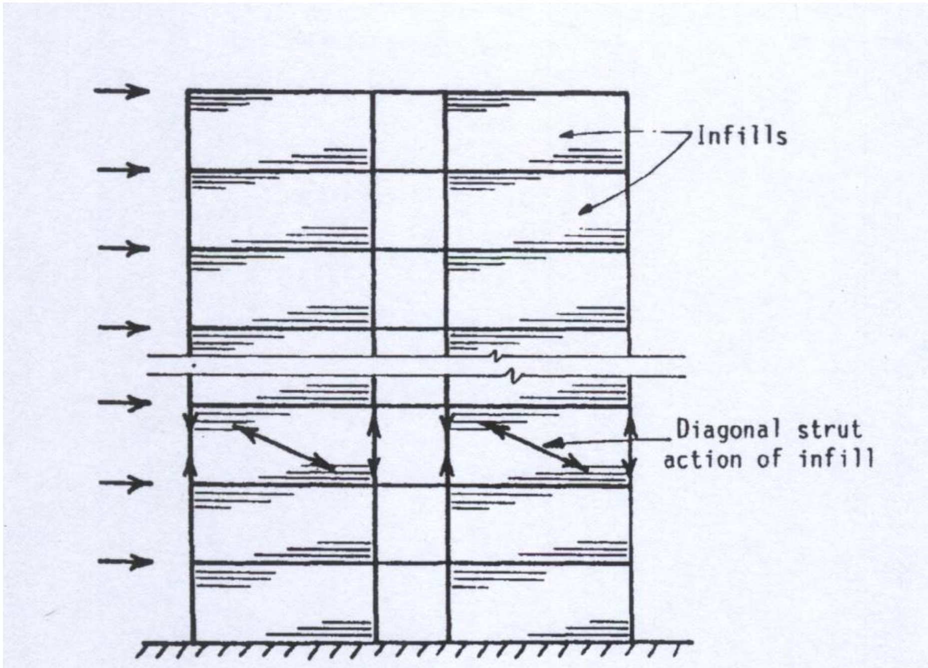

Fig (2.4) Infilled frame [3]

around elevator, stair, and service shafts (Fig. 2.5). Because they are much stiffer horizontally than rigid frames, shear wall structures can be economical up to abut 35 stories [3].

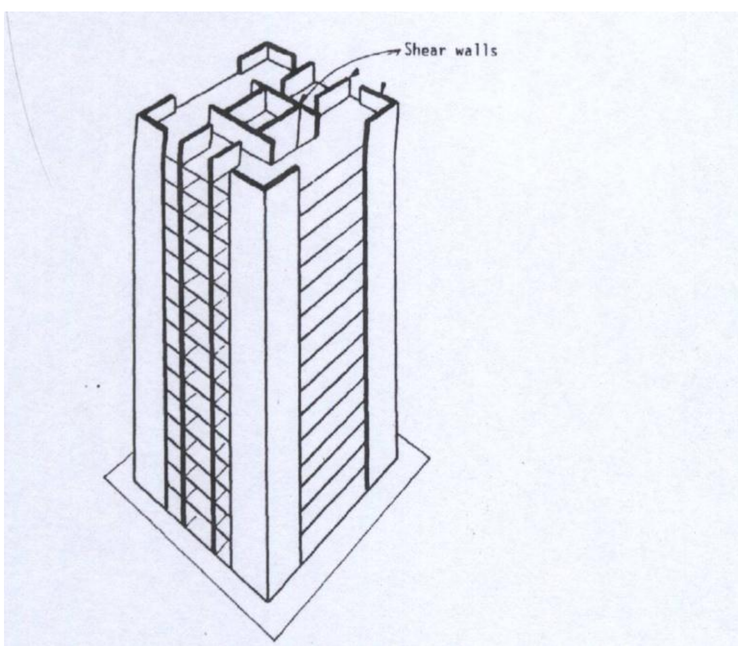

Fig (2.5) Shear wall structure [3]

A coupled wall structure is a particular, but very common, form of shear wall structure with its own special problems of analysis and design. It consists of two or more shear walls in the same plane, or almost. The same plane, connected at the floor levels by beams or stiff slabs (Fig. 2.9). The effect of the shear-resistant connecting members is to cause the set of walls to behave in their plane partly as a composite cantilever, bending about the common centroidal axis of the walls. This results in a horizontal stiffness very much greater than if the walls acted as a set of separate uncoupled cantilevers [3]. Another way of improving the efficiency of the framed tube which you can [3], thereby increasing its potential for use to even greater heights as well as allowing greater spacing between the columns, is to add diagonal bracing to the faces of the tube. This arrangement was first used in a steel structure in 1969 in Chicago's John Hancock Building (Fig. 2.10) and in a reinforced concrete structure in 1985, in New York's 780 Third Avenue Building (Fig.2.8). In the steel tube the bracing traverses the faces of the rigid Frames, whereas in the concrete structure the bracing is formed by a

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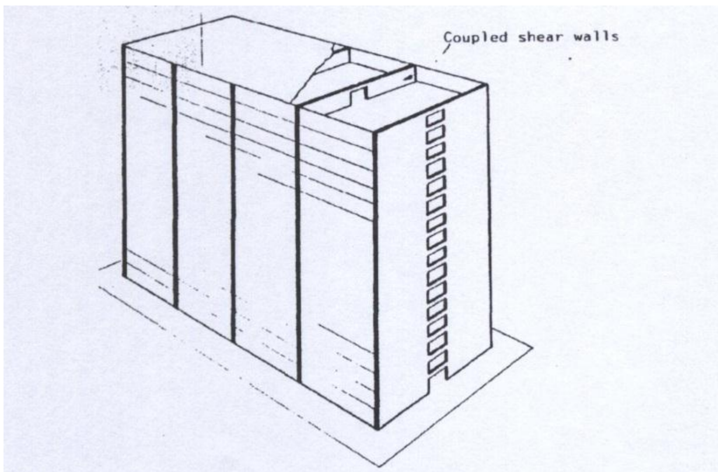
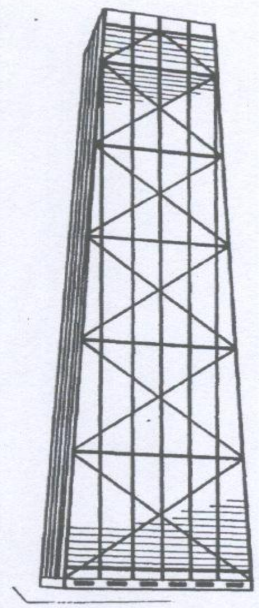
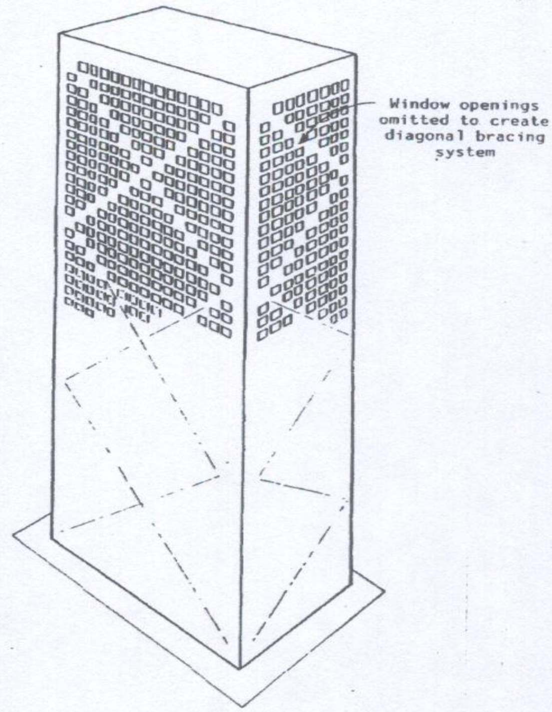


Fig(2.6) Coupled shear wall structure [3]

diagonal pattern of concrete window-size panels, poured integrally with the frame. Because the diagonals of a braced tube are connected to the columns at each intersection, they virtually eliminate the effects of shear lag in both the flange and web frames. As a result, the structure behaves under lateral loading more like a frame, with greatly diminished bending in the members of the frames. Consequently, the spacing of the columns can be larger and the depth of the spandrels less, thereby allowing larger size windows than in the conventional tube structure [3].

In the braced-tube structure the bracing contributes also to the improved performance the tube in carrying gravity loading: differences between gravity loads in the columns are evened out by the braces transferring axial loading from the more highly to the less highly stressed columns [3].





Fig(2.7) steel- braced tube **Fig(2.8) concrete-braced tub**

Outrigger-Braced structure is one of the structural forms as ref [3] stated, this efficient structural form consists of a central core, comprising either braced or shear walls, with horizontal cantilever “outrigger” trusses or girders connecting the core to the outer columns. When the structure is loaded horizontally, vertical plane rotations of the core are restrained by the outriggers through tension in the windward columns and compression in the leeward columns (fig 2.9). The effective structural depth of the building is greatly increased, thus augmenting the lateral stiffness of the building and reducing the lateral deflection and moments in the core. In effect, the outriggers join the columns to the core to make the structure behave as a partly composite cantilever [3].

Perimeter columns, other than those connected directly to the ends of the out riggers, can also be made to participate in the outrigger action by joining all the perimeter columns with a horizontal truss or girder around the face of the building at the outrigger level. The large, often two-story, depths of the outrigger and perimeter trusses make it desirable to locate them within the plant levels in the building. Outrigger-braced structures have been used for buildings from 40 to 70 stories high, but the system should be effective and efficient for much greater heights [3].

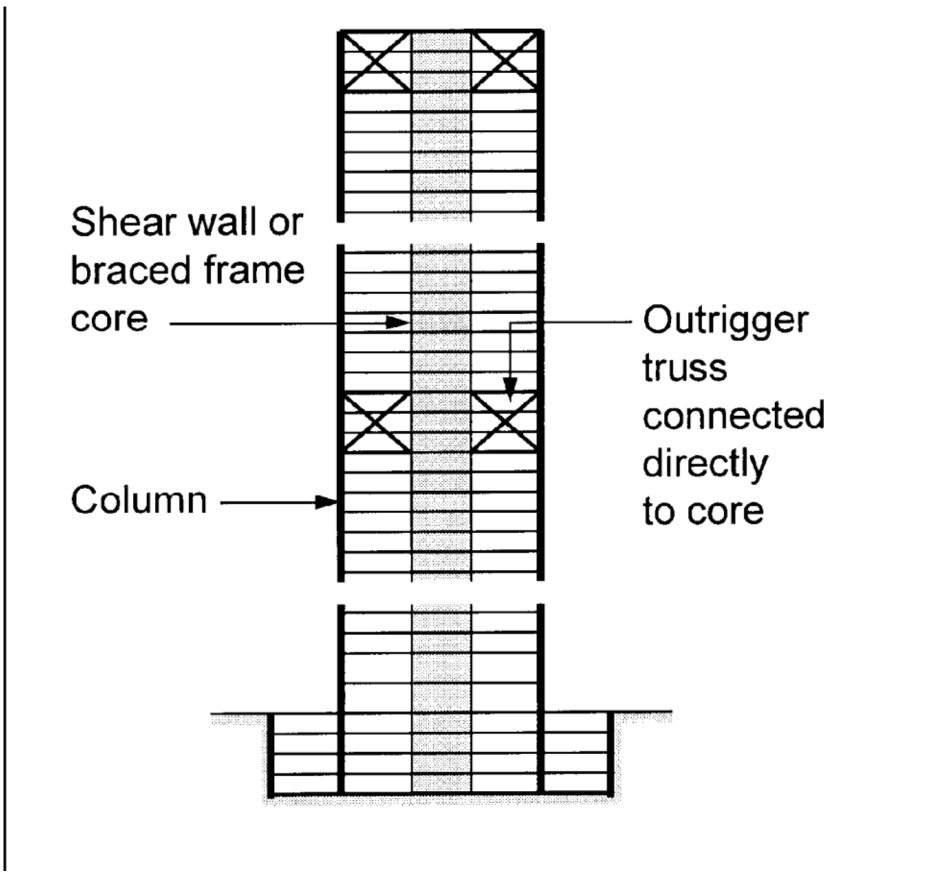


Fig (2.9) R. S As stated by ref [3],

the suspended structure is one of the structural forms, suspended structure is consists of a central core, or cores, with horizontal cantilevers at roof level, to which vertical hangers of steel cable, rod, or plate are attached. The floor slabs are suspended from the hangers (Fig. 2. 10a) [3].

The advantages of this structural form are primarily architectural in that, except for the presence of the central core, the ground story can be entirely free of major vertical members, thereby allowing an open concourse; also, the hangers, because they are in tension and consequently can be of high strength steel, have a minimum sized section and are therefore less obtrusive. The potential of this latter benefit tends to be offset, however, by the need to proof the hangers against fire and rust, thereby significantly increasing their bulk. The suspended structure has some construction advantages in allowing the core, cantilevers, and hangers to be constructed while the slabs are being poured on top of each other at ground level; the slabs are then lifted in sets and fixed in position (Fig. 2.10b).

The structural disadvantages of the suspended structure are that it is inefficient in first transmitting the gravity loads upward to the roof-level cantilevers before returning them through the core to the ground, and that the structural width of the building at the base is limited to the relatively narrow depth of the core, which restricts the system to buildings of lesser height. A further problem is caused by the vertical extension of the slender hangers that, over the range from zero to full live loading, can result in significant changes in the levels of the edges of the slabs. This effect increases at each level down the length of the hanger and, consequently, is worst at the lowest hung floor. The problem can be limited by restricting the maximum number of floors supported by a single length of hanger to about 10, and by having multilevel cantilever systems (Fig. 2.11)

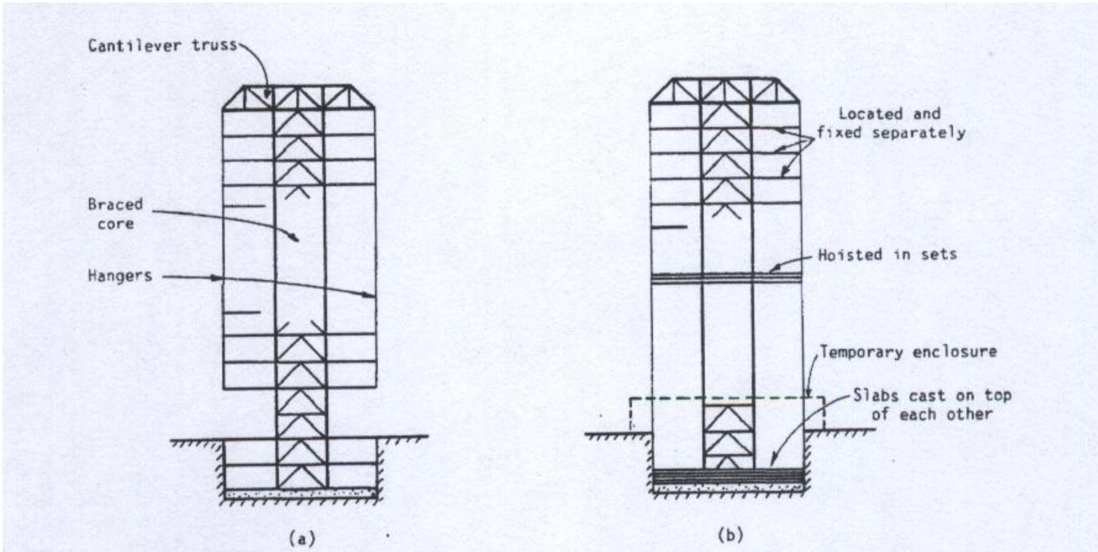


Fig (2.10) (a) suspended structure (b) sequence of construction-sus – structure

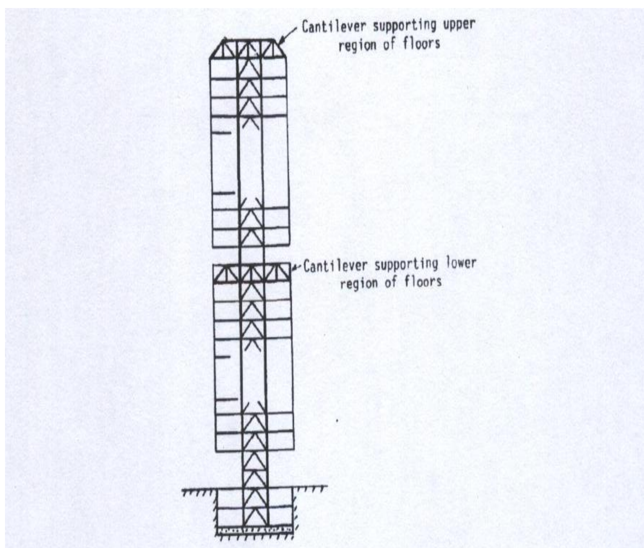


Fig (2.11) two tiered suspended structure [3]

Core structure is one of the structural forms [3] stated, in these structures a single core serves to carry the entire gravity and horizontal loading (Fig. 2.15). In some, the slabs are supported at each level by cantilevers from the core. In others, the slabs are supported between the core and perimeter columns, which terminate either on major cantilevers at intervals down the height, or on a single massive cantilever a few stories above the ground [3].

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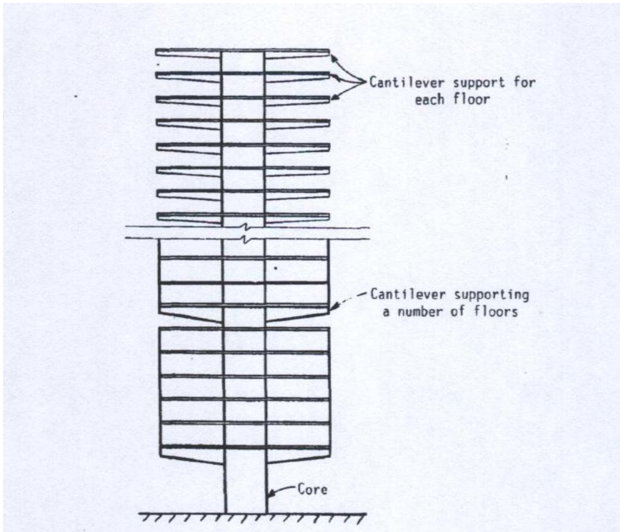

Fig (2.12) core structure[3].

The last type of the structural forms [3] stated is the space structure, the primary load-resisting system of a space structure consists essentially of a three-dimensional triangulated frame—as distinct from an assembly of planar bents—whose members serve dually in resisting both gravity and horizontal loading. The result is a highly efficient, relatively lightweight structure with a potential for achieving the greatest heights [3].

Although simple in their overall concept, space structures are usually geometrically complex, which calls for considerable structural ingenuity in transferring both the gravity loading and the lateral loading from the floors to the main structure? One solution is to have an inner braced core, which serves to collect the lateral loading, and the inner region gravity loading, from the slabs over a number of multi-story regions at the bottom of each region. The lateral and gravity loads are transferred out to the main joints of the space frame (Fig. 2.10) [3].

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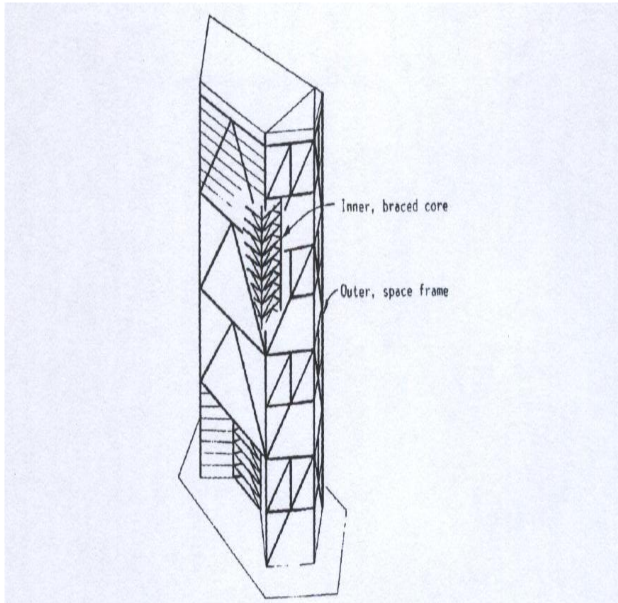

Fig (2.13) space structure[3].

As ref [3] stated the exciting forces on a structure due to wind actions tend to be random in amplitude and spread over a wide range of frequencies. The structure’s response is dominated by the action of its resonant response to wind energy available in the narrow bands close to the natural frequencies of the structure. The major part of the exciting energy will generally be at frequencies much lower than the, fundamental natural structural frequency and the amount of energy decreases with increasing frequency. Consequently, for design purposes, it is usually necessary to, consider the structure's response only in the fundamental modes the contribution from higher modes is rarely significant [3].

The fluctuations in response of a structure can be considered as those associated with the mean wind speed, and those associated with the turbulence of the wind, which are predominantly dynamic in character. Consequently, it is convenient to describe wind speeds, forces, deflections, etc. in term of an hourly mean value together with the average maximum fluctuation likely to occur in an hour. When these are added, the sum can be used as an average hourly maximum, or peak response, to define equivalent static design da

2.4Nonlinear analysis (P Delta)

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 in the simple cantilever modal in figures below the total base moment is [4].

$$Mub = VuH + Pu\Delta \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.1)$$

Therefore in addition to the over turning 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[4]..

It is necessary to recognize when assessing seismic design forces that the importance of $P\,\Delta$ effects will generally be more significant for structures in regions of low to moderate seismicity than for structures in regions of high seismicity where design lateral forces will be correspondingly higher . Therefore in most situations, particularly in regions where large seismic design forces need to be considered, $P\,\Delta$ phenomena will not control the design of frames [4].

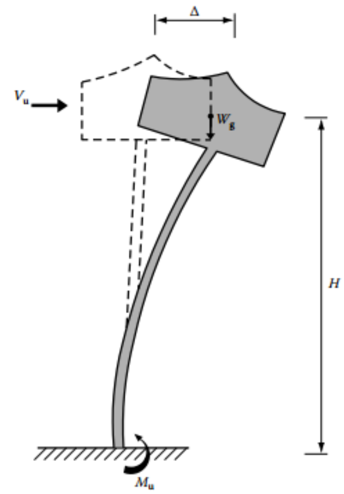

Fig (2.14) p-Δ simple cantilever modal[4].

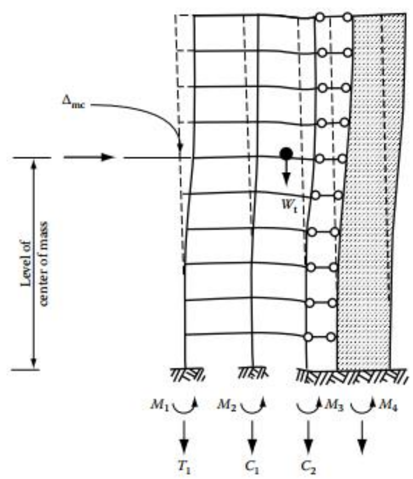

Fig (2.15) p-Δ shear wall system[4].

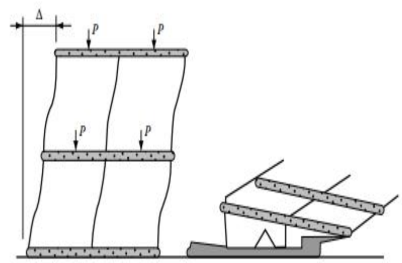

Fig (2.16) collapse due to p-Δ effect [4].

Expected P_ Δ phenomena will increase drift, but analyses of typical building frames have indicated that effects are small when maximum understory drift isles than 1%. However, for greater understory drift P_ Δ effects may lead to rapidly increasing augmentation of these drifts [4].

It should be recognized that in so far as the elastic behavior is concerned that increasing strength of a frame is more effective in controlling drift than increasing stiffness. This is because the more vigorously a frame responds in the elastic range , the less is the significance of stiffness[4].

Chapter Three
Analysis of Tall Buildings under Wind Load

3.1 Introduction

Advances in structural design concepts and analytical techniques, combined with the availability of newer and more efficient materials and construction methods for, buildings, have led to significant reductions in their structural weight and stiffness. In addition, heavy masonry cladding and partition walls, which were very effective in increasing the stiffness of structural frames, are less frequently used. Consequently, the more typical light and flexible modern tall building is much more responsive to dynamic exciting forces than its earlier counterpart. The resulting dynamic stresses may be much greater than static values, while induced motions may disturb the comfort and equanimity of the building's occupants.

Tall building motions may be classified as static or dynamic. The first refers to the motions produced by slowly applied forces such as gravitational or thermal effects, or the long period component of wind. Dynamic motions refer to those caused by time-dependent dynamic forces, notably seismic accelerations, short period wind loads, blasts, and machinery vibrations, the first two usually being of the greatest concern.

Although the deformations arising from static forces may be of possible detriment to the integrity of the structure, unless they lead to the collapse of the building they are unlikely to provoke any reaction from the occupants.

Dynamic wind pressures produce sinusoidal or narrow-band random vibration motions of the building, which will generally oscillate in both along-wind and cross-wind directions, and possibly rotate about a vertical axis. The magnitudes of the three displacement components will depend on the velocity distribution and direction of the wind, and on the shape, mass, and stiffness properties of the structure. In certain cases, the effects of cross-wind motions of the structure may be greater than those due to the along-wind motions.

The ground shaking which occurs in an earthquake may be described as a series of virtually multi-directional random acceleration pulses. The response of a building to such acceleration-time histories may be determined from an elastic analysis, unless the inertial forces are sufficiently large to cause inelastic deformations or localized failures.

The intensity of the ground shaking to which the building may be subjected during its lifetime can be estimated from the recorded earthquake history in the area in which it is situated. Continuing records are used to produce maps showing regions of relative possible seismic hazards, and these can be extended and refined as knowledge of such events is accumulated.

When designing a tall building to resist seismic forces, the design loads may be determined from a dynamic analysis of the building's response to time-history base accelerations, based on an actual recorded local event, or an artificially generated time-history. Such a time con-summing rigorous approach may be simplified by the use of earthquake response spectra, which, although requiring less computational effort, yield acceptably similar results for peak responses.

The seismic response of the building will depend on the dynamic properties of the structure, the ground motion at the foundation, and the mode of soil-structure, interaction. The motion of a very stiff building will be almost identical to the ground motion, but that of a flexible structure will be quite different. The response will depend on the proximity of the natural frequencies of the structure to that of the predominant ground-motion frequency, the damping inherent in the structure, the foundation behavior, the ductility of the structure, and the duration of the earthquake.

The equivalent static loads for wind effects will be based on a statistical knowledge of the likely occurrence and magnitude of wind veloci1ties and pressures, and for earthquakes on the time-history of accelerations. For the majority of tall buildings, the quasistatic loadings are adequate for design purposes, and have proved satisfactory in most situations.

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 accelerations. Then consideration has to be given to both the stress levels that occur and the accelerations that may affect the comfort of the occupants.

This chapter considers 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. The field of structural dynamics is very extensive, and is well covered by existing textbooks. Consequently, this chapter will highlight briefly only the major techniques that are important for tall structures. Finally, consideration is given to the human response to dynamic motions and its effect on structural design

3.2 Dynamic Response to Wind Loading:

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.

3.2.1 Sensitivity of Structures to Wind Forces:

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 he 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. At frequencies below the first natural frequency, the structure will tend to follow closely the fluctuating force actions. The dynamic response will be attenuated at frequencies above the natural frequency, but will be amplified at frequencies at or near the natural frequency, consequently 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.

When a structure is very flexible, its oscillations may interact with the aerodynamic forces to produce various kinds of instability, such as vortex-capture resonance, galloping oscillations, divergence,' and flutter. In this exceptional case, the potential for disaster is so great that the design must he changed or the aerodynamic effects modified to ensure that this form of unstable behavior cannot occur.

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.

Unfortunately, it is not yet possible, particularly in the early design stages, to assess accurately whether a dynamic analysis will be required Although several empirical guidelines are available in Design Codes. For example, 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.

A more sophisticated, but necessarily more complicated, approach for structural classification has been devised. It allows a judgment on whether a structure is potentially dynamic, that is, not stiff' enough to be analyzed by static methods alone, but sufficiently stiff to prevent aerodynamic instability.

The procedure requires knowledge of the damping ratio and the lowest natural frequency of vibration of the structure. In the early stages of design, it is not possible to calculate accurately the fundamental natural frequency, but empirical formulas are available to allow an assessment to be made.

3.2.2 Dynamic Structural Response due to Wind Forces:

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 oil 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 term 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.

$$x(max) = x' + gpa \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \qquad (3.1)$$

Where x (max), x`, and a are the peak, mean, and standard deviations, respectively, of the variable x concerned, referred to a record period of 1 hour, and gp is the "peak" factor. When considering the response of a tall building to wind actions, both along- wind and cross-wind motions must considered.

3.2.3 Along-Wind Response

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 (3.1) shows the long and cross wind direction. . Fig (3.1) shows the long and cross wind direction.

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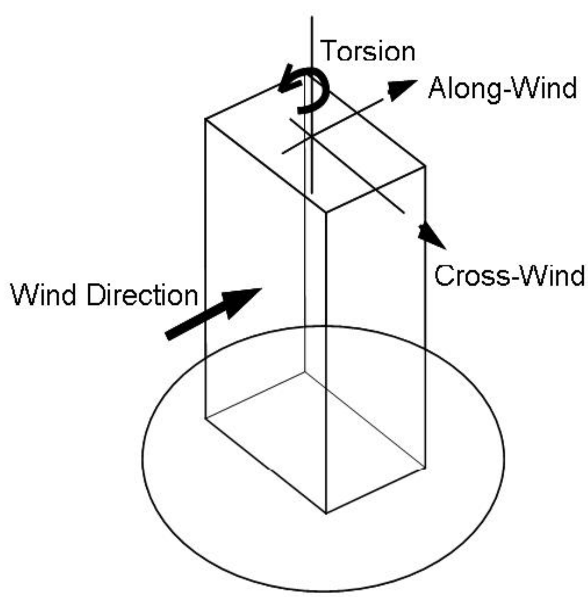


Fig (3.1). Along and cross wind direction

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 Q`, such that.

Design value,

$Q(max) = GQ' \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots ..$ (3.2)

where 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 M, the design dynamic base overturning moment M (max) will be obtained by multiplying M by the gust factor

GM $M(Max) = GM \ldots\ldots\ldots\ldots\ldots\ldots \ldots\ldots\ldots$ (3.3)

Schematic representation of Davenport's design procedure is shown in fig (3.2). 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 (3.3), 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 [3].

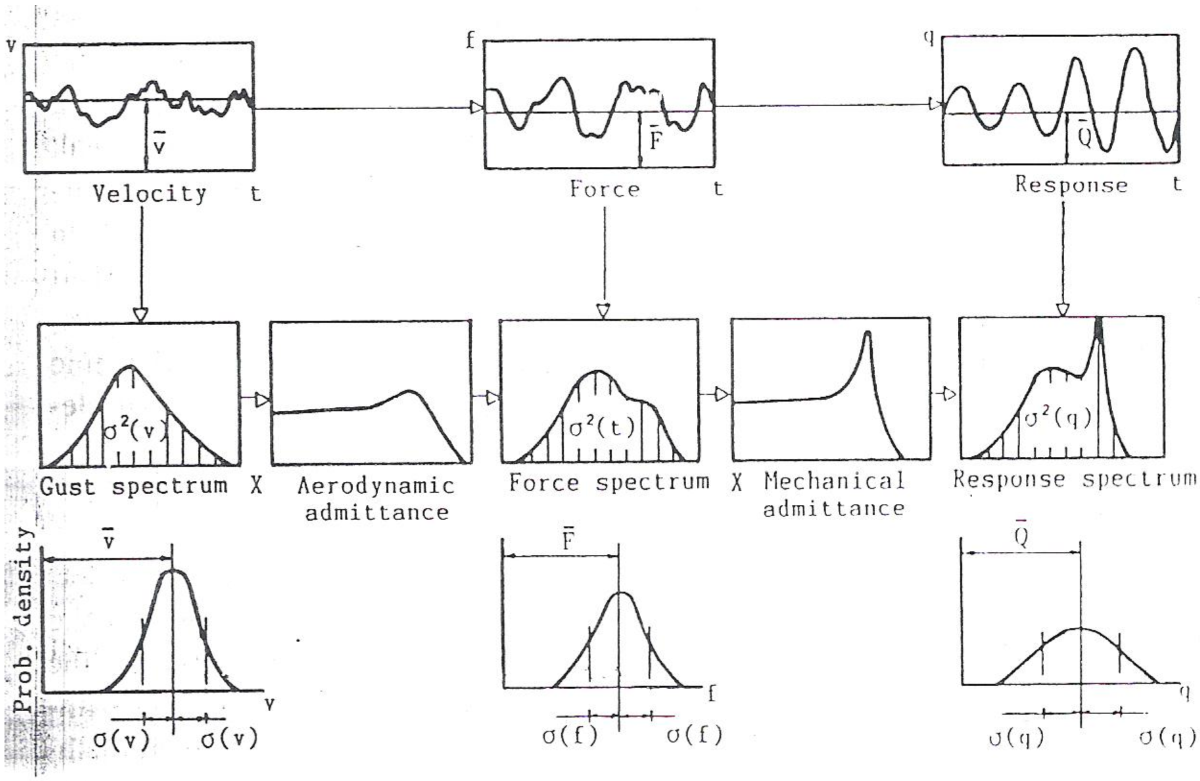


Fig (3.2) Schematic representation of Davenport's design procedure [3]

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. 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 board 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. (3.4) by B and R (B is the excitation due to the background turbulence or gust energy, which depends on the height and aspect ratio of the building, and R is the excitation by the turbulence resonant with the structure,, respectively, combined vector ally to give the peak response) [3].

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 (o) and an appropriate damping ratio . Consequently, only a single linear mode of vibration need he 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 n0, and critical damping ratio , the mean design wind' speed V, and the particular location of the building (i.e., whether it is sited in the center of a large city, in suburbs or wooded areas, or in flat open country).

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

$$G = 1 + gp.r(B + R)^{1/2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.4)$$

In Eq. (3.4), gp 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 no: 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. 3.3); B is the excitation due to the background turbulence or gust energy, which depends on the height and aspect ratio of the building.(fig. 3.4); and R is the excitation by the turbulence resonant with the structure, which depends on the size effect S, the gust energy ratio at the natural Frequency of the structure, F, and the critical damping ratio   that is.

$$R = \frac{SF}{B} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.5)$$

The size reduction factor S depends on the aspect ratio W / H, the natural frequency n0, and the mean wind velocity at the top of the structure, VH, as shown in Fig. (3.5). the gust energy ratio F is a function of the inverse wavelength, n0/VH as shown in Fig. (3.6).

If resonant effects are small, then R will be small compared to the background Turbulence B, and vice versa

The peak factor gp in Eq. (3.4) 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 (3.7) as a function of the average fluctuation rate v given by:

$$V = \frac{no}{\sqrt{1+B/R}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.6)$$

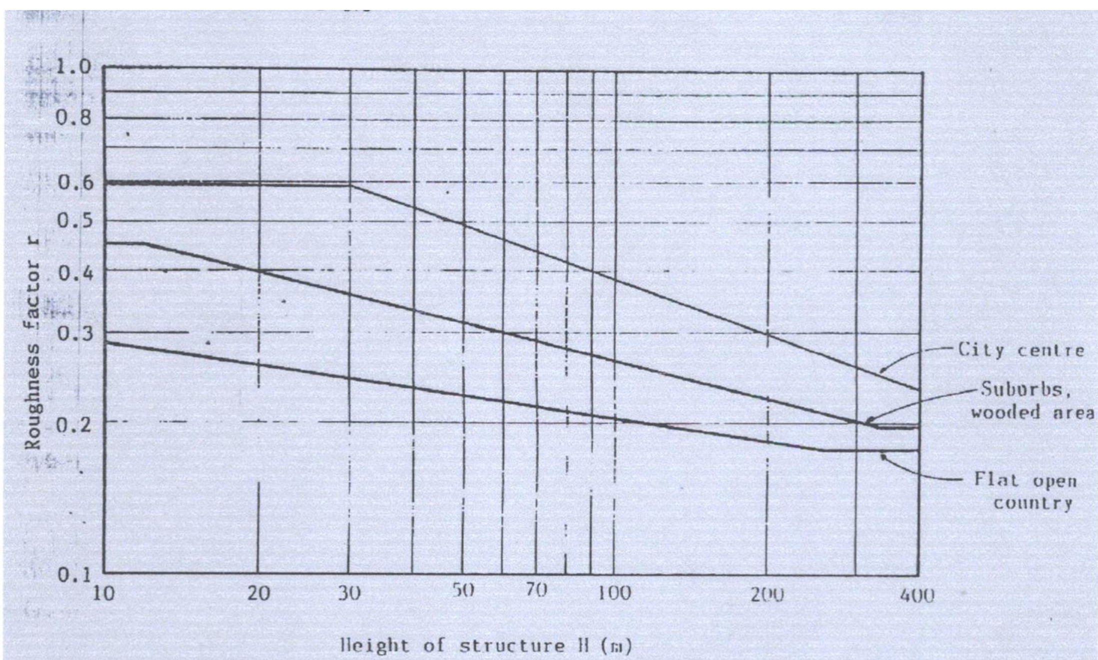
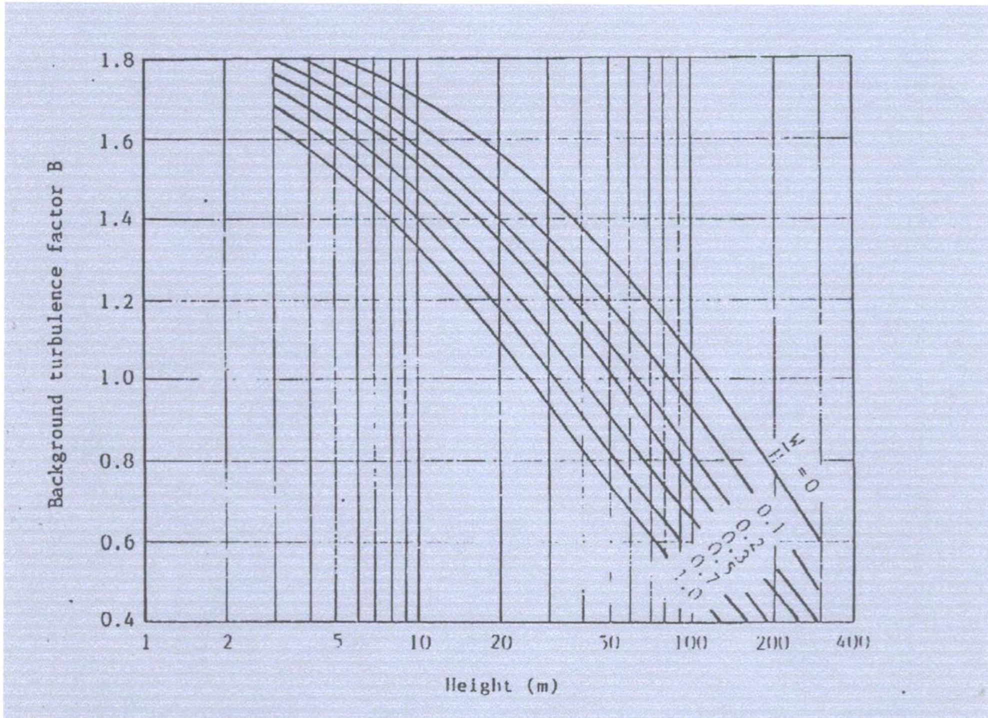


Fig (3.3) variation of roughness factor with building height



Fig(3.4) variation of background turbulence factor with height and aspect [3].

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 aD in the along-wind direction may be estimated from:

$aD = 4\ 2n^2_{o.gp.r}$ ………………………………………… (3.7)

Where = the maximum wind-induced deflection at the top of the building in the along- wind direction (m). In the above formulas, the variables VH, no' and must relate to the along-wind direction. Substitution of the known values of gp, r, B, and R into Eq. (3.4) 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.

The natural frequency no and damping ratio must be again in the along-wind Direction [3].

3.2.4 Cross-Wind Response

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 cross-wind motions. Consequently,

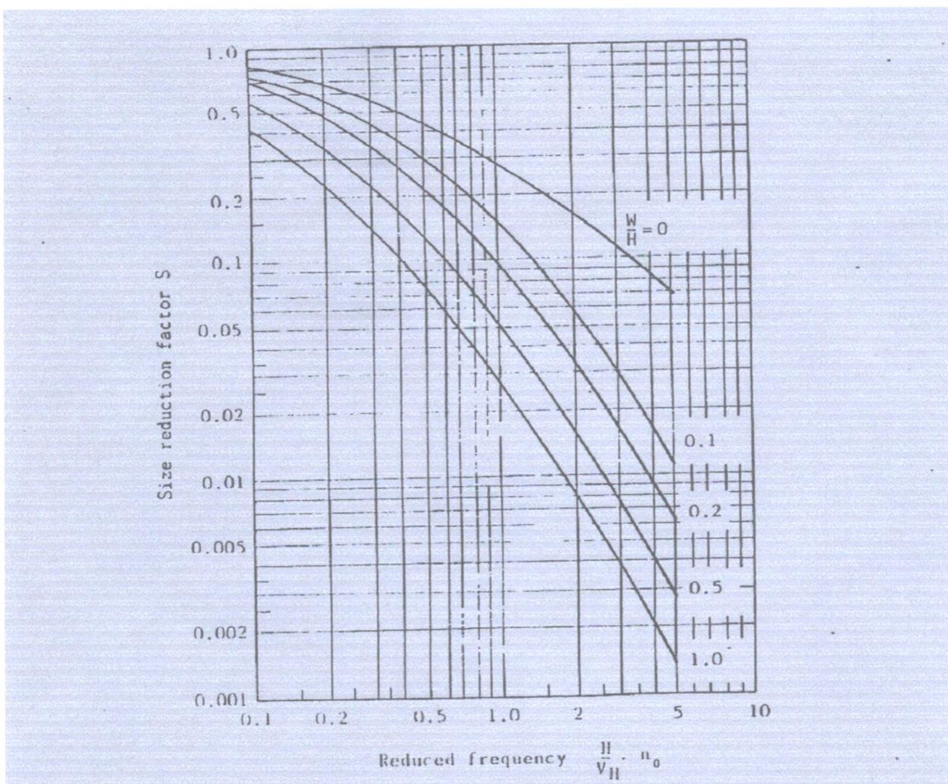


Fig (3.5) Variation of size reduction factor with frequency and aspect ratio of building [3]

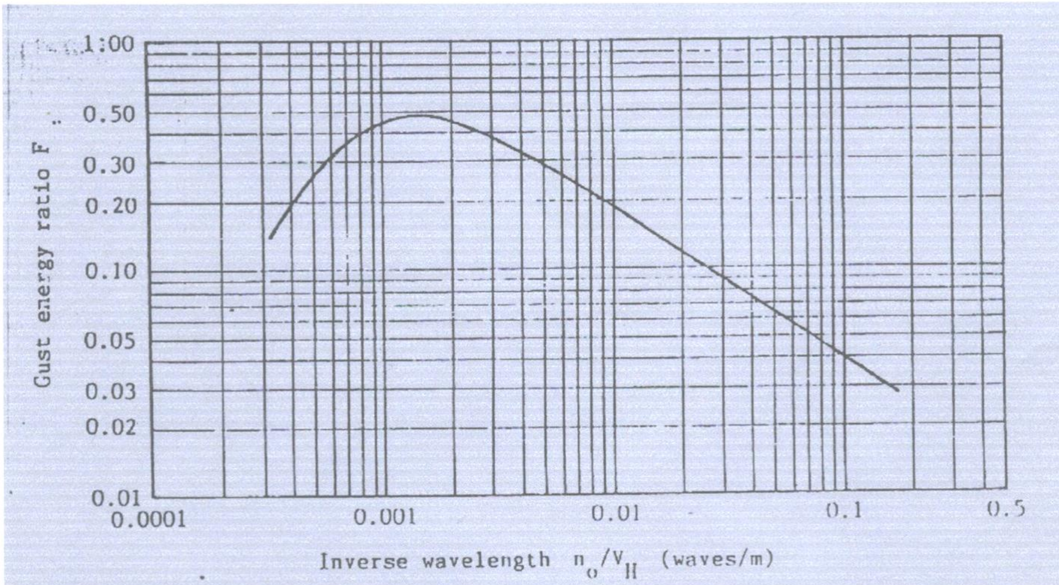


Fig (3.6) variation of gust energy ratio with inverse wavelength [3]

that the maximum lateral wind loading and defection are in the along-wind direction, the maximum acceleration of the building may often occur in the cross-¬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 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 is given in the NBCC for the peak acceleration Aw at the top of the building, namely,

$AW = n2o\ gp[WD]\ {}^{1}/_{2}$ ………………………………………………….. (3.8)

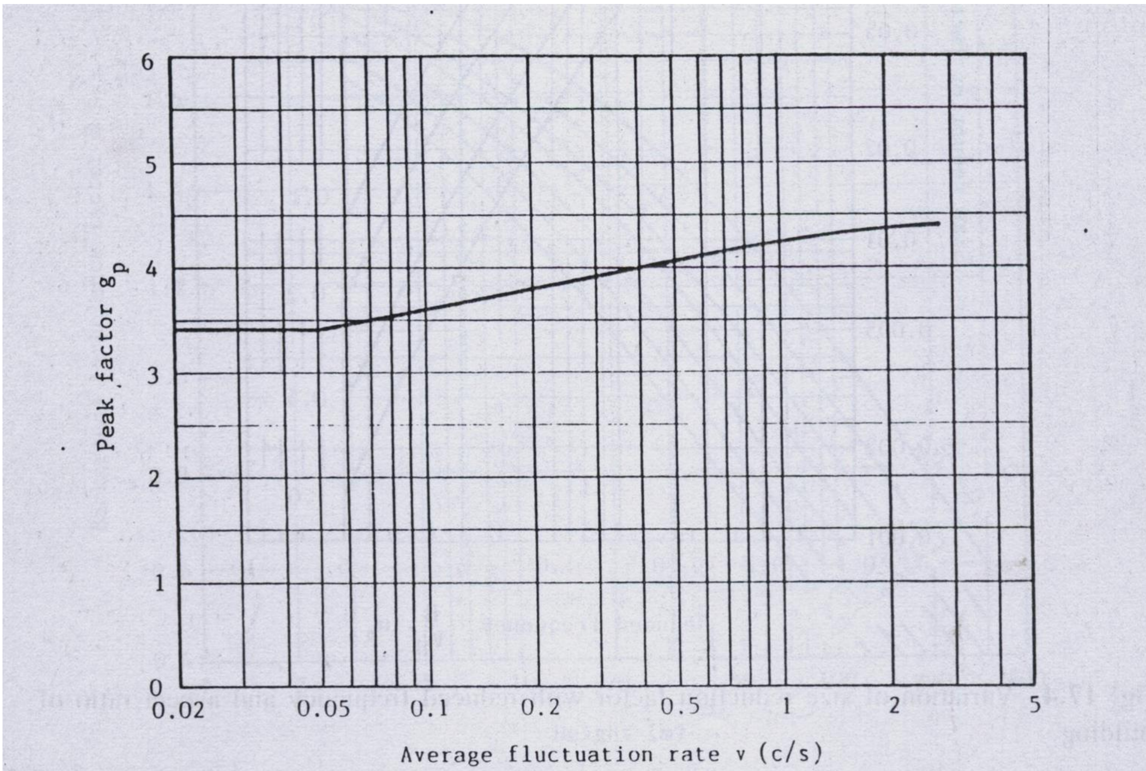
where

P = average, density of the building (kg/m1)

ar = 78.5 x I 0**3 , ((VH / (no))** 3.3 (pa)

g = acceleration due to gravity (m / sec2)

Because of the relative sensitivities of the expressions in Esq. (3.7) and (3.8) to the natural frequencies, it is recommended that the latter be determined using fairly rigorous analytical methods, and that approximate formulas be used with caution.].



Fig(3.7) variation of peak factor with average fluctuation rate [3].

3.4 Analysis using computer program:

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

3.4.1 General

The commercially available computer program ETABS (Computers & Structures, Inc, 1995) was chosen for the numerical analysis in this thesis for two reasons. First, ETABS is a special purpose computer program for the analysis of building system .Building systems represent a unique class of structures that are defined floor – by –floor , column-by column ,bay – by –bay and wall –by –wall and not as a sequence of non- descriptive nodes and elements as in general purpose computer programs. Second ETABS is among the most commonly used structural analysis software packages in analysis and design of a variety of commercial and residential buildings by engineering consultants and thus serves as a useful benchmark. 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.

3.4.2 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 multistory building structures, such as office buildings apartments and hospitals. 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 story 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. The floor elements may span between adjacent levels for the creation of sloped floors. Modeling of partial diaphragms, such as in mezzanines, a setback, atriums and floor opening is possible. It is also possible to consider situations with multiple diaphragms at each level thereby allowing the modeling of buildings consisting of several towers rising from a combined structure below or vice versa.

3.4.3Basic Process

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

1. Set the units.

2. Open a file.

3. Set up grid lines.

4. Define story 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

Chapter four

Linear and Nonlinear Analysis Results

4.1 Introduction

The building has been analyzed by using ETABSV13 computer program. The plan of 20 story high residential building, with flat slab floor system and spandrels beam which will be investigated shown in Fig. (4.1) . The total plan area of the building is about 150 m^2 (with overall dimensions 12 m * 12.5 m) and height of floor is 3m .Shown fig in appendix (b)

The floor slab thickness = 220 mm .

The beam section = 600x300 mm

The wall thickness = 300 mm.

The columns section = 750 x 350 mm.

Number of floor = 20

The floors height=3.2

Basement =4

Height of building=64.8

Concrete Design cod ACI 318-11

4.2 The case Study Building Properties:

. The 3D multi story building model Fig (4.2) was analysis use ETABS. To check model the result obtained for linear static analysis wear compared with result how reference (2)

4.2.1 The Materials:

The materials strengths are as follows:

F_{yv} = 250 N/mm^2

Reinforced Concrete = 24 kN/ m^3

$Fu = 620.53$Mpa

$F_y = 413.69$ Mpa

$F_c = 27.58$ Mpa

4.2.2 The Building Loads:

The area loads are taken as follows:

Max. Floor slab thick=220 mm

D.L = Slab self weight + finishes (1.5 kN/ m^2) + Brick works partitions (4.5 KN/ m^2) = 6 kN/ m^2

L.L = 2.5 kN/ m^2 .

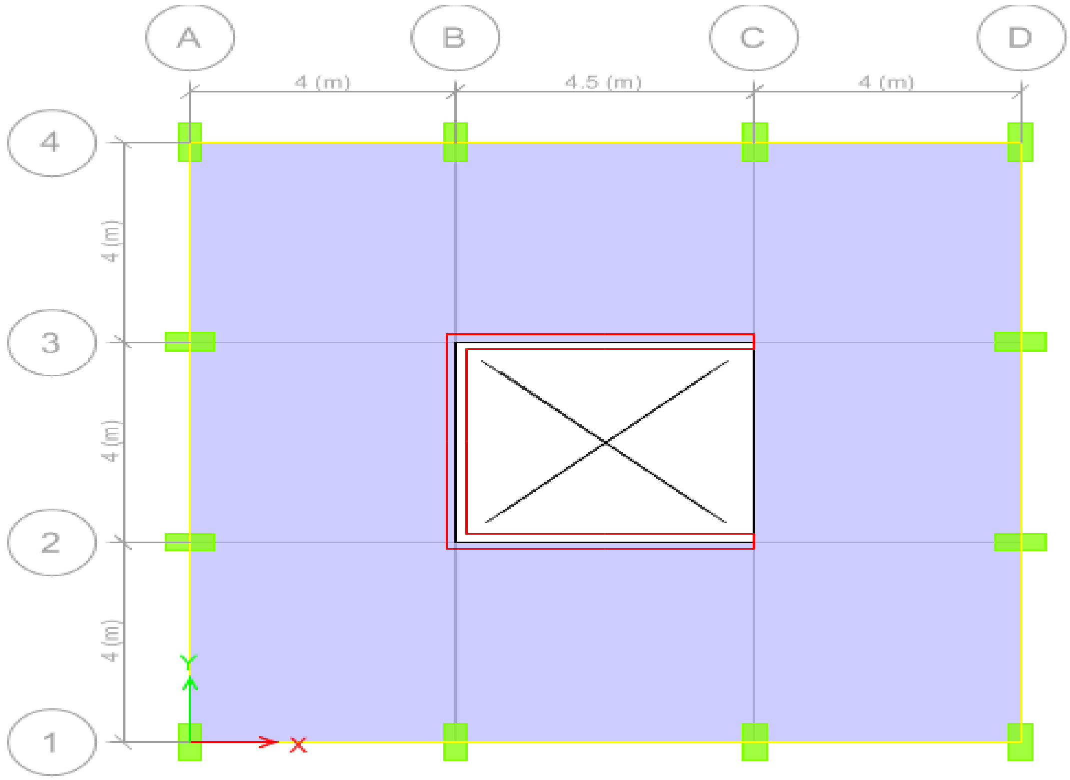


Fig (4.1) Plan of multi –story building

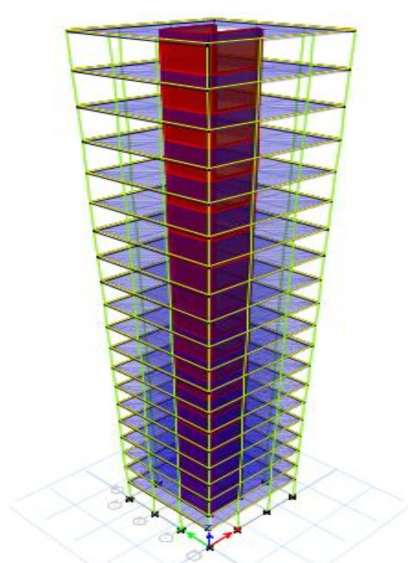


Fig (4.2) 3D of multi –story building model

4.3 Wind load:

The building is located in Khartoum the capital of the republic of Sudan which is considered as one of the low-rise building city when calculating wind loads, although many scattered tall reinforced concrete buildings are encountered.

Wind loads are calculated to ASCE 7-05

4.3.1. Procedures wind load

Automatic wind loads for the ASCE 7-05 are based on Section 6.5 of A7-05.

The wind loads applied are a modified version of those described in ASCE 7-05 Section 6.5 (Method 2 – Analytical Procedure). Windward and leeward Horizontal wind loads are applied on the vertical projected area of the building as determined from the story heights and the input diaphragm exposure widths. The programs do not apply vertical wind loads automatically over the projected horizontal area of roof surfaces. To include those vertical wind loads in the same load pattern, the user must include them manually. The following equation is used to determine the velocity pressure, *qz*, at any height *z* on the surface of the vertical projected area, in pounds per square foot (psf).

$qz = .00256\ KzKztKdV2I$ … … … … … … … … … …………… …. (1.4)

where,

Kz = The velocity pressure exposure coefficient.

Kz = The velocity pressure exposure coefficient..

C6-4a and C6-4b).

Kzt = Topographic factor as input by the user.

Kd = Directionality factor as input by the user.

V = Basic wind speed in miles per hour (mph) as input by the user

I = Importance factor as input by the user.

The velocity pressure exposure coefficient, *Kz*, is obtained using (Eqns. C4a and C6-4b in ASCE 7-05 Commentary Section 6.5.6.6*z* 2.01

$$\ldots Kz = 2.01\left(\frac{z}{Zq}\right)^{z/\propto} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(4.2)$$

For 15feet<z<zg

$$Kz = 2.01\left(\frac{z}{Zq}\right)^{z/} \ \ldots\ldots Kz =$$

$$2.01\left(\frac{z}{Zq}\right)^{z/\propto} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(4.3)$$

For z< 15feet

Where,

z = Distance (height) from input bottom story/minimum height to point considered.

zg = Gradient height. See (ASCE 7-05 Eqn. C6-6).

□ = Empirical exponent. See (ASCE 7-05 Eqn. C6-5).

The gradient height, *zg*, and the empirical exponent □ are obtained ASCE 05 Table 6-2.

ASCE 7-05 Eqn. 6-17 is used to determine the wind pressure, *p*, at any point on the surface of the vertical projected area.

p = *q G Cp*-windward + *qh G Cp*-leeward (ASCE 7-05 Eqn. 6-17)

where,

q = Velocity pressure, *qz*, at any height *z* on the surface of the vertical projected area calculated using (ASCE 7-05 Eqn. 6-15).

G = Gust effect factor as input by the user.

Cp-windward = Windward external pressure coefficient as input by the user.

qh = Velocity pressure at the top story height on the surface of the vertical projected area calculated using (ASCE 7-05 Eqn. 6-15).

Cp-leeward = Leeward external pressure coefficient as input by the user.

4.3.2 Wind speed and load:

The maximum wind speed to be used for Khartoum area was shown speed in table (c) appendix (a) the meter speed is100 mph and result wind load is calculated as follows

Form this speed the wind load is calculated as follows.

4.3.2.1 ASCE 7-05 Auto Wind Load Calculation

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

4.3.2.2 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] $C_{q,wind} = 0.8$

Leeward Coefficient, $C_{p,lee}$ [ASCE 6.5.11.2.1] $C_{q,lee} = 0.5$

Wind Case = All Cases

Top Story = Story20

Bottom Story = Base

Include Parapet = Yes, Parapet Height = 1.5

4.3.2.3Factors 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 63] $K_z = 2.01(^{\frac{z}{z_g}}$ for15ft $\leq z \leq z_g$

$K_z = 2.01(^{\frac{15}{z_g}}$ for $z \leq$ 15ft

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] $G = 0.85$

4.3.2.4 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] $p = qGC_{p,wind} + q_h(GC_{p,lee})$

Table (4.1) lateral load to story:

Story	Elevation M	Location	Load in X-Dir kN	Load inY-Dir kN
Story20	64.8	Top	86.6178	0
Story19	61.6	Top	53.3666	0
Story18	58.4	Top	52.8692	0
Story17	55.2	Top	52.352	0
Story16	52	Top	51.8128	0
Story15	48.8	Top	51.2494	0
Story14	45.6	Top	50.6589	0
Story13	42.4	Top	50.038	0
Story12	39.2	Top	49.3825	0
Story11	36	Top	48.6876	0
Story10	32.8	Top	47.9469	0
Story9	29.6	Top	47.1525	0
Story8	26.4	Top	46.2939	0
Story7	23.2	Top	45.3571	0
Story6	20	Top	44.3222	0
Story5	16.8	Top	43.1602	0
Story4	13.6	Top	41.8254	0
Story3	10.4	Top	40.2381	0
Story2	7.2	Top	38.2396	0
Story1	4	Top	40.9123	0
Base	0	Top	0	0

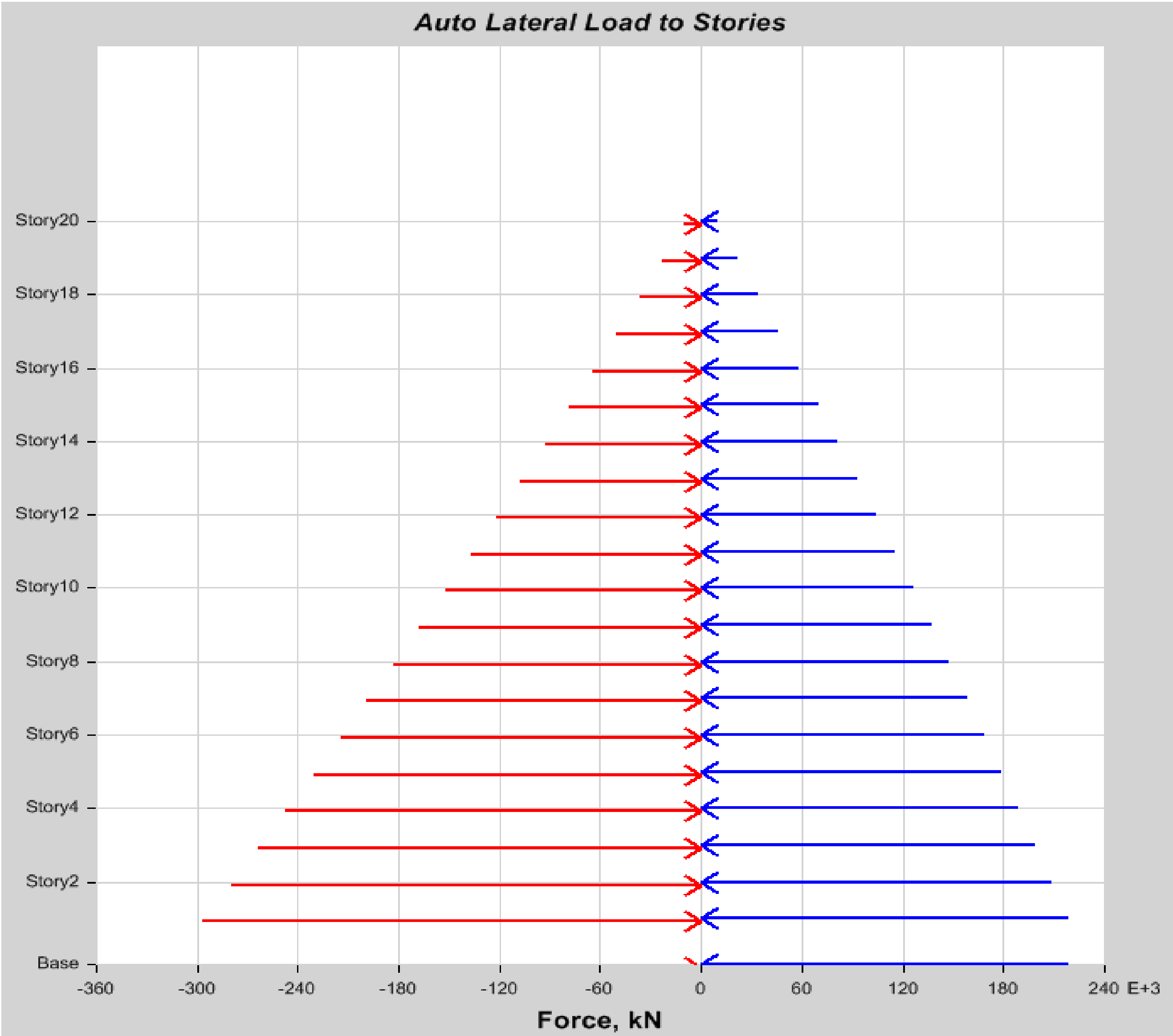


Fig (4.3) lateral load to story

4.4 Finite element method:

The finite –element type of idealization is applicable to structures of all types: framed structures, which comprise assemblages of one-dimensional members (beam, column, etc); plane-stress or shell-type structures, which are made up of two-dimensional components, and general three-dimensional solids. The advantages of this special procedure are as follows:

1- Any desired number of generalized coordinates can be introduced merely by dividing the structure into an appropriate number of segments.

2- Since the displacement functions chosen for each segment may be identical, computations are simplified.

3- The equations, which are developed by this approach, are largely uncoupled because each nodal displacement affects only the neighboring elements, thus the solution process is greatly simplified.

In general the finite-element approach provides the most efficient procedure for expressing the displacements of arbitrary structural configuration by means of a discrete set of coordinates.

In this study the tall building was modeled with 2253shell finite elements and 2620 frame finite elements with a total of 3656 nods.

4.5 Result of Analysis Model check:

The maximum story drifts result obtained for linear static analysis for case study and reference (2) to check the accuracy of the case study model. The comparison is shown in table (4.3) and fig (4.5) .The difference reference (2) is great than the case study and this is out to the building in reference (2) being higher (130) than the case study (64.8).

Table (4.2) Maximum story drifts:

Story	**Hight** m	**Location**	**Drifts X-Dir kn**	**Drifts Y-Dir kn**
Story20	64.8	Top	5.453E-08	3.143E-07
Story19	61.6	Top	7.723E-08	3.561E-07
Story18	58.4	Top	1.058E-07	4.085E-07
Story17	55.2	Top	1.371E-07	0.000000466
Story16	52	Top	1.692E-07	0.000001
Story15	48.8	Top	2.009E-07	0.000001
Story14	45.6	Top	2.314E-07	0.000001
Story13	42.4	Top	2.605E-07	0.000001
Story12	39.2	Top	2.878E-07	0.000001
Story11	36	Top	0.000000313	0.000001
Story10	32.8	Top	0.000000336	0.000001
Story9	29.6	Top	3.565E-07	0.000001
Story8	26.4	Top	3.743E-07	0.000001
Story7	23.2	Top	3.887E-07	0.000001
Story6	20	Top	3.988E-07	0.000001
Story5	16.8	Top	4.029E-07	0.000001
Story4	13.6	Top	3.977E-07	0.000001
Story3	10.4	Top	3.766E-07	0.000001
Story2	7.2	Top	3.271E-07	0.000001
Story1	4	Top	1.989E-07	3.722E-07

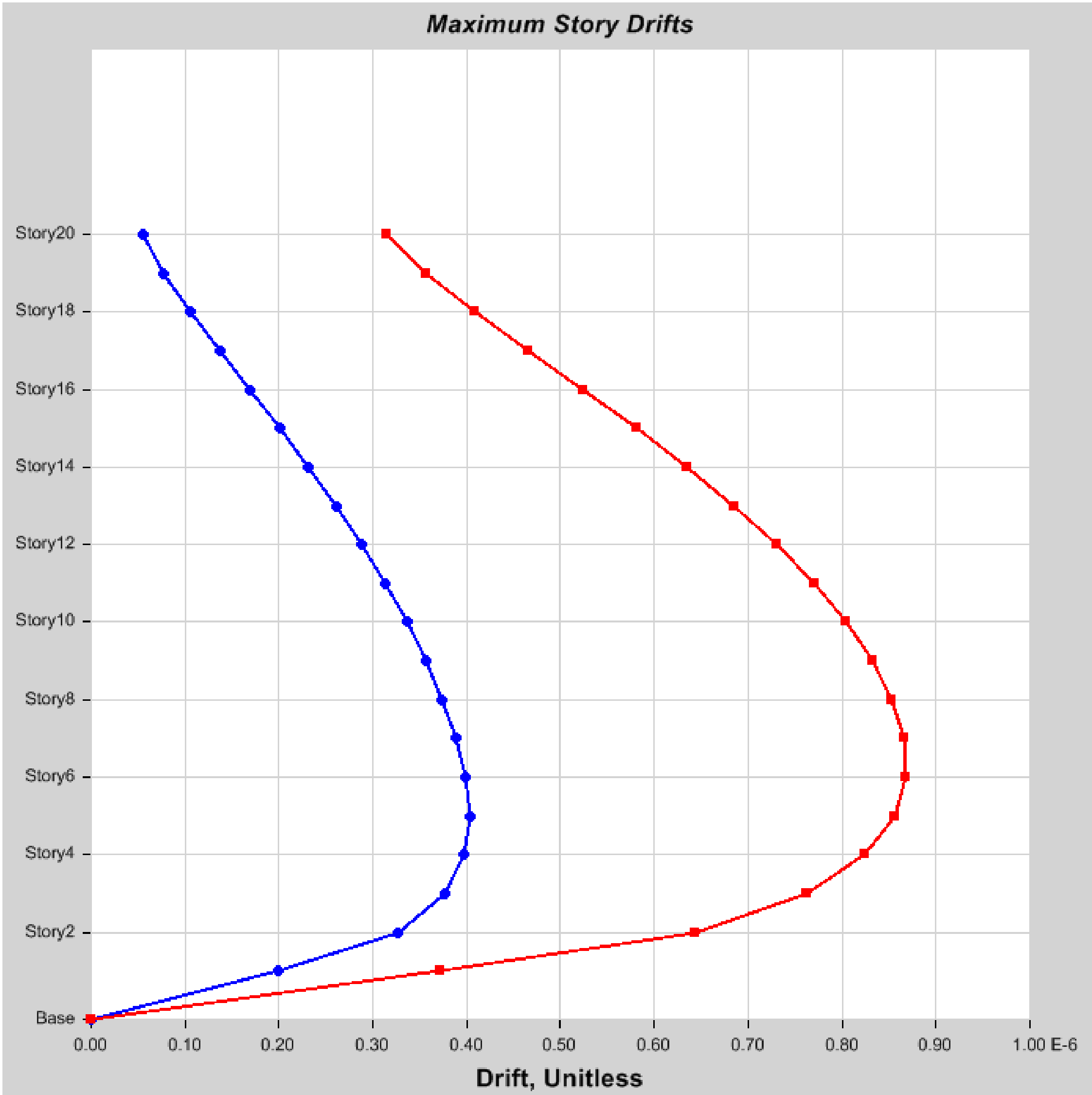


Fig (4.4) Maximum story drifts

Table (4.3) comparison of drifts case study and reference (2):

h1	(case s)	h2	reference
0	0	0	0
4	0.00000372	3	0.000139
7.2	0.000001	6	0.000157
10.4	0.000001	9	0.000189
13.6	0.000001	12	0.000239
16.8	0.000001	15	0.000295
20	0.000001	18	0.000349
23.2	0.000001	21	0.000397
26.4	0.000001	24	0.00044
29.6	0.000001	27	0.000479
32.8	0.000001	30	0.000514
36	0.000001	33	0.000545
39.2	0.000001	36	0.000572
42.4	0.000001	39	0.000596
45.6	0.000001	42	0.000617
48.8	0.000001	45	0.000636
52	0.000001	48	0.000632
55.2	0.00000466	51	0.000665
58.4	0.00000409	54	0.000677
61.6	0.00000356	57	0.000687
64.8	0.00000314	60	0.000687
		63	0.0007

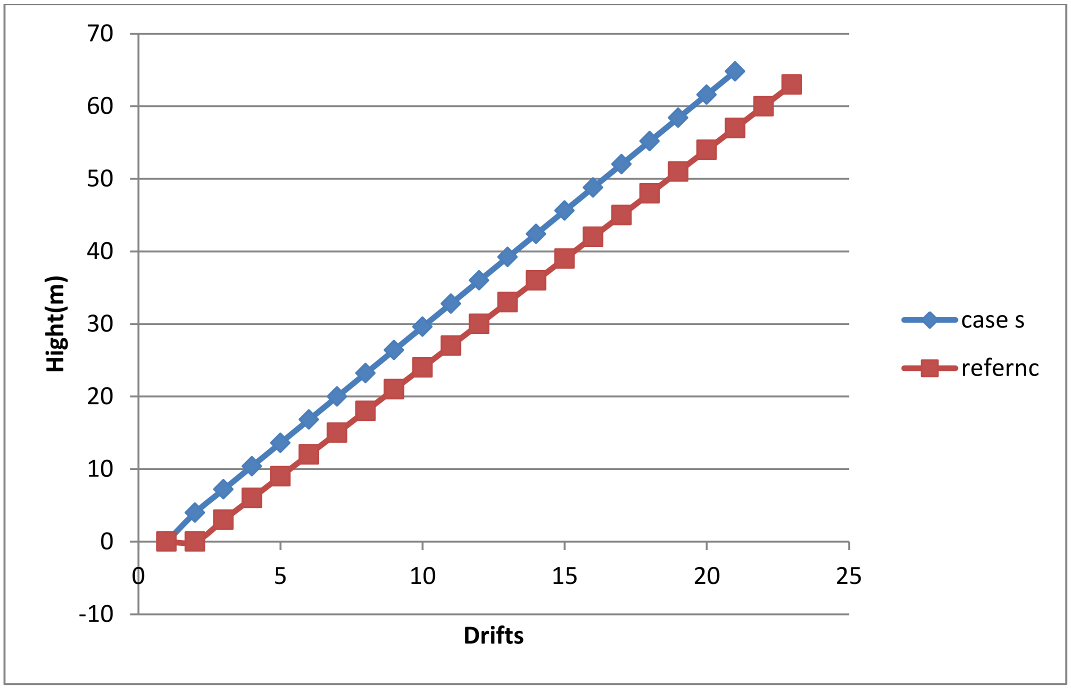
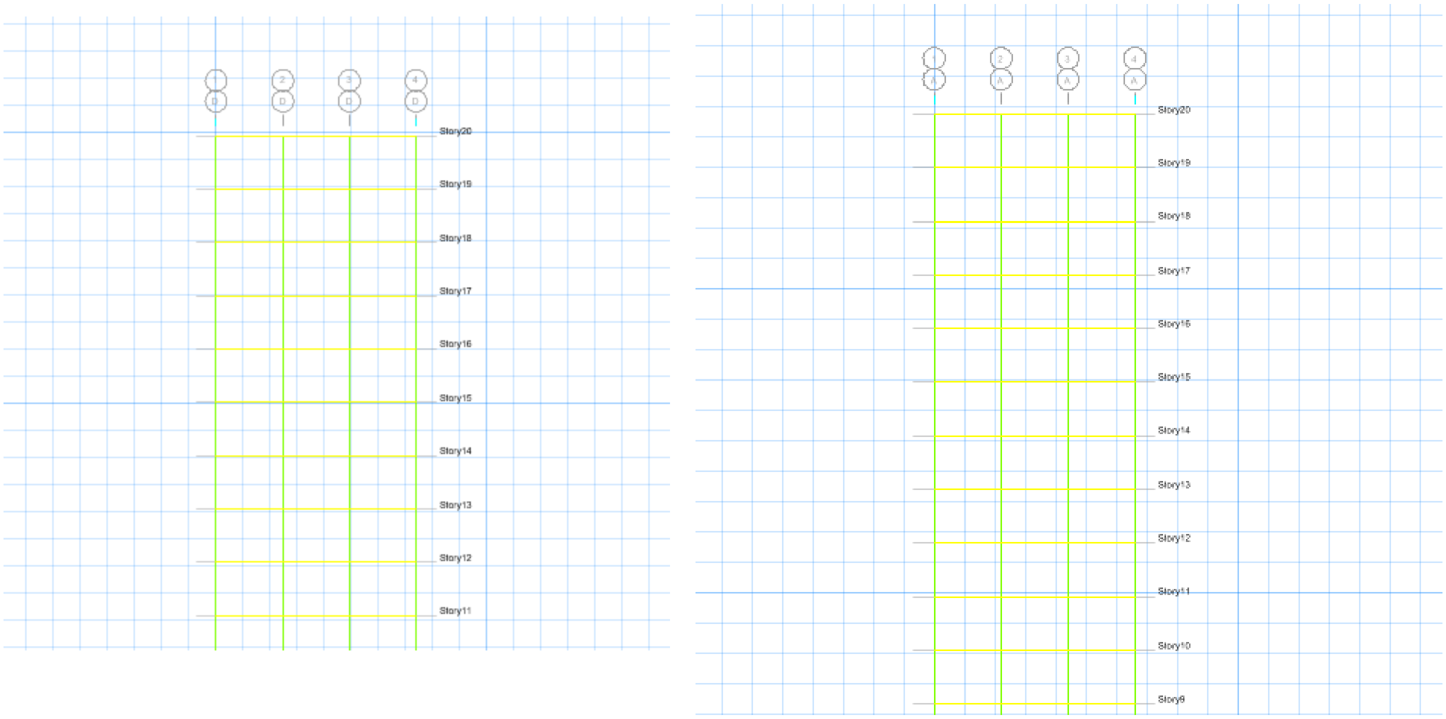


Fig (4.5) comparison of drifts case study and reference (2)

Also the model was checked by increasing the number of the element by adding one node and two nodes for the corner column and compared the displacement diagram in each case:

Case one : without nodes in the column, the number of node was 336 as shown in fig(4.6)



Fig(4.6) the corner column without interior nodes

In this case the displace ment carve was as show in fig(4.)

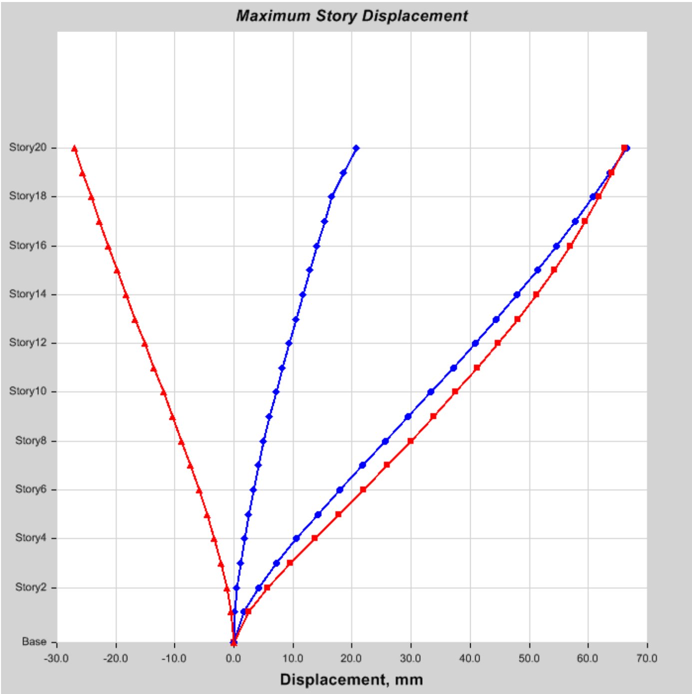
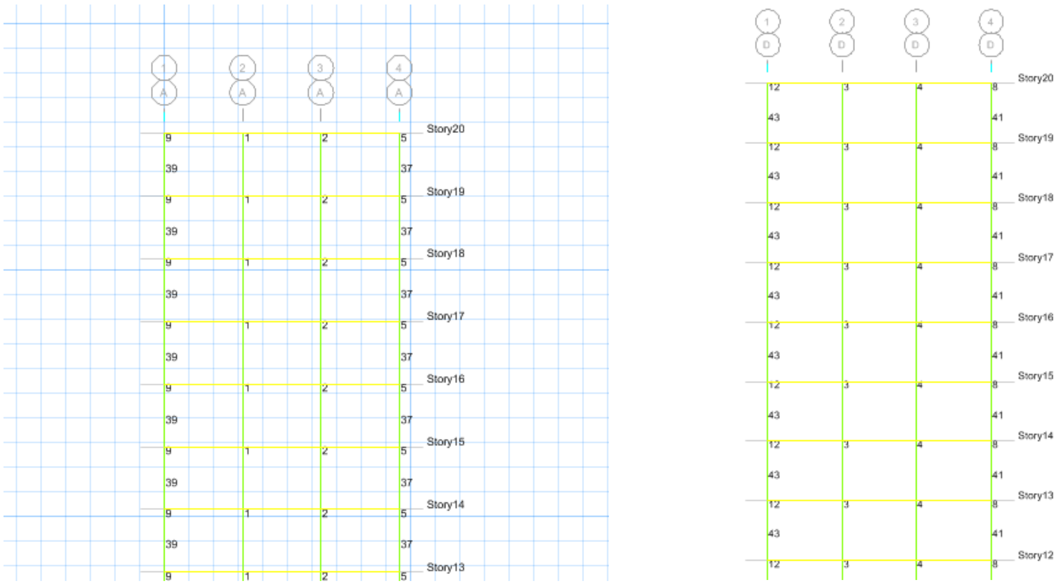


Fig (4.7) the displacement without interior nodes

Case two : by adding one node at each corner column so the number of nodes was 496 as shown in fig(4.8)



Fig(4.8) the corner column with one interior nodes

In this case the displace ment carve was as show in fig(4.9)

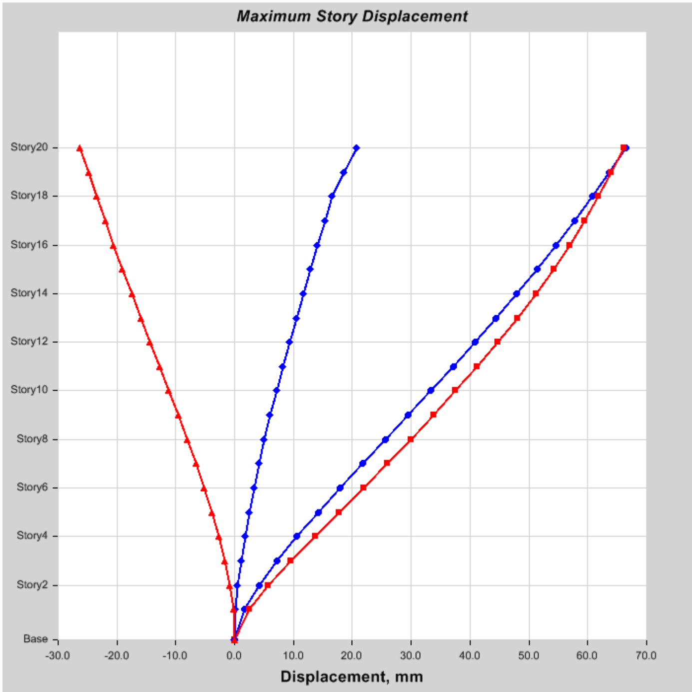
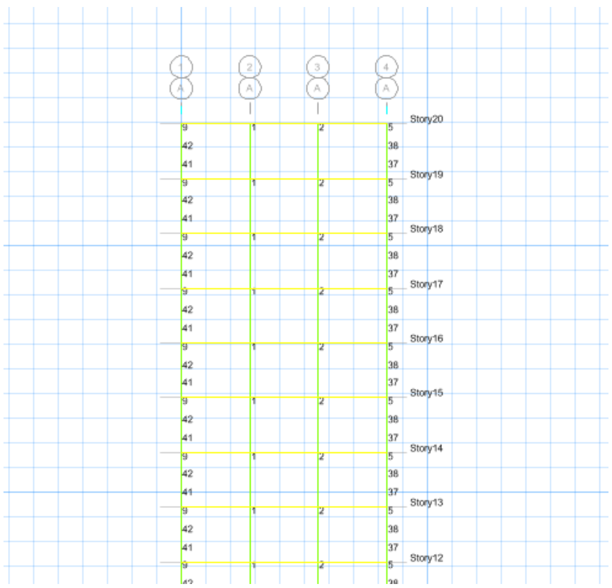
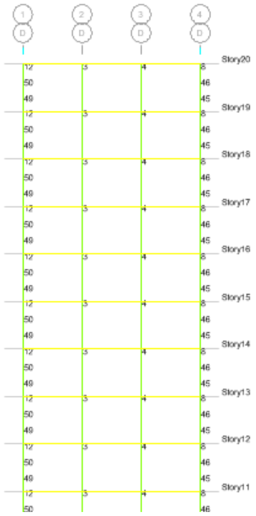


Fig (4.9) the displacement with one interior node

Case three: by adding two node at each corner column so the number of nodes was 656 as shown in fig (4.10)





Fig(4.10)the corner column with two interior nodes

In this case the displace ment carve was as show in fig(4.11)

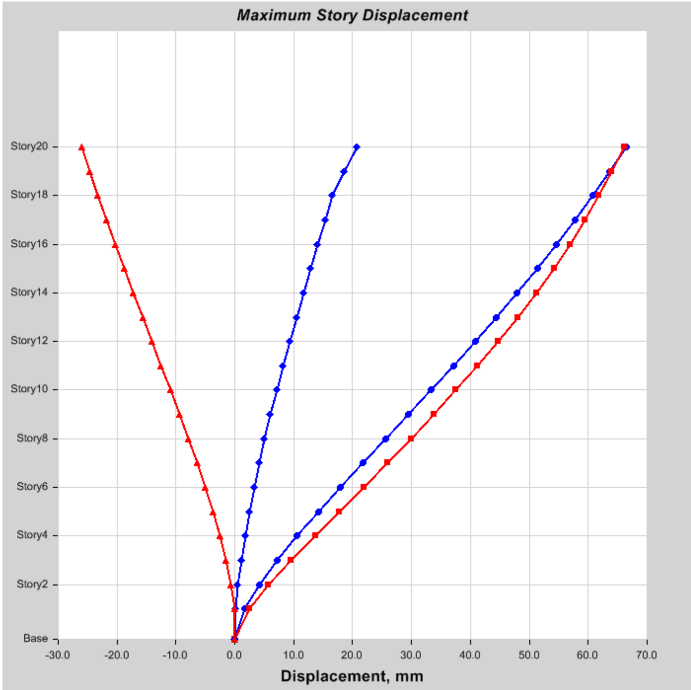


Fig (4.11) the displacement with two interior nodes

Then by comparing fig(4.7) ,fig(4.9) and fig(4.11) the carves had same curvature so the model is OK

4.6 Analysis and Discussion of Results:

4.6.1 Maximum Displacements:

Figure (4.12) and Table (4.4) show the Max Displacement in y-y direction and the percentage difference three analysis types.

Max difference (P Delta- Linear) =81.1-66.1=15 mm

Max difference (Large displacement- Linear) =78.5-66.5=12 mm

Max difference (P Delta- Large displacement) =82.1-78.5=3.6 mm

-Referring to table (4.4) and figure (4.13) the maximum difference displacement y-y direction between linear analysis and p Δ analysis is 22.9%. The maximum displacement y-y direction between linear analysis and p ∆ with large displacement analysis is 18.7% The maximum difference displacement y-y direction between p ∆ with large displacement analysis and p ∆ analysis is 3.3%.These show the large difference between the linear and nonlinear displacement results.

Table (4.4) Max Displacement in y-y direction

Story	**Elevation**	**Linear**	**P ∆**	**P ∆ W**	different percentage	different percentage	different percentage
	M	Mm	mm	mm	(P ∆-Linear) /Linear	(P ∆ W-Linear)/Linear	(P ∆ -P ∆ W)/P ∆ W
Story20	64.8	66.1	81.1	78.5	22.69289	18.75946	3.312102
Story19	61.6	64	78.6	76	22.8125	18.75	3.421053
Story18	58.4	61.9	75.9	73.4	22.61712	18.57835	3.405995
Story17	55.2	59.5	73	70.6	22.68908	18.65546	3.399433
Story16	52	56.9	69.9	67.5	22.8471	18.62917	3.555556
Story15	48.8	54.2	66.5	64.2	22.69373	18.45018	3.582555
Story14	45.6	51.2	62.9	60.7	22.85156	18.55469	3.624382
Story13	42.4	48.1	59	57	22.66112	18.50312	3.508772
Story12	39.2	44.7	54.9	53	22.81879	18.56823	3.584906
Story11	36	41.2	50.6	48.8	22.81553	18.4466	3.688525
Story10	32.8	37.5	46.1	44.5	22.93333	18.66667	3.595506
Story9	29.6	33.8	41.5	39.9	22.78107	18.04734	4.010025
Story8	26.4	30	36.8	35.3	22.66667	17.66667	4.249292
Story7	23.2	26	31.9	30.4	22.69231	16.92308	4.934211
Story6	20	21.9	26.9	25.5	22.83105	16.43836	5.490196
Story5	16.8	17.8	21.8	20.6	22.47191	15.73034	5.825243
Story4	13.6	13.6	16.7	15.6	22.79412	14.70588	7.051282
Story3	10.4	9.6	11.6	10.9	20.83333	13.54167	6.422018
Story2	7.2	5.7	6.9	6.5	21.05263	14.03509	6.153846
Story1	4	2.4	2.9	2.7	20.83333	12.5	7.407407

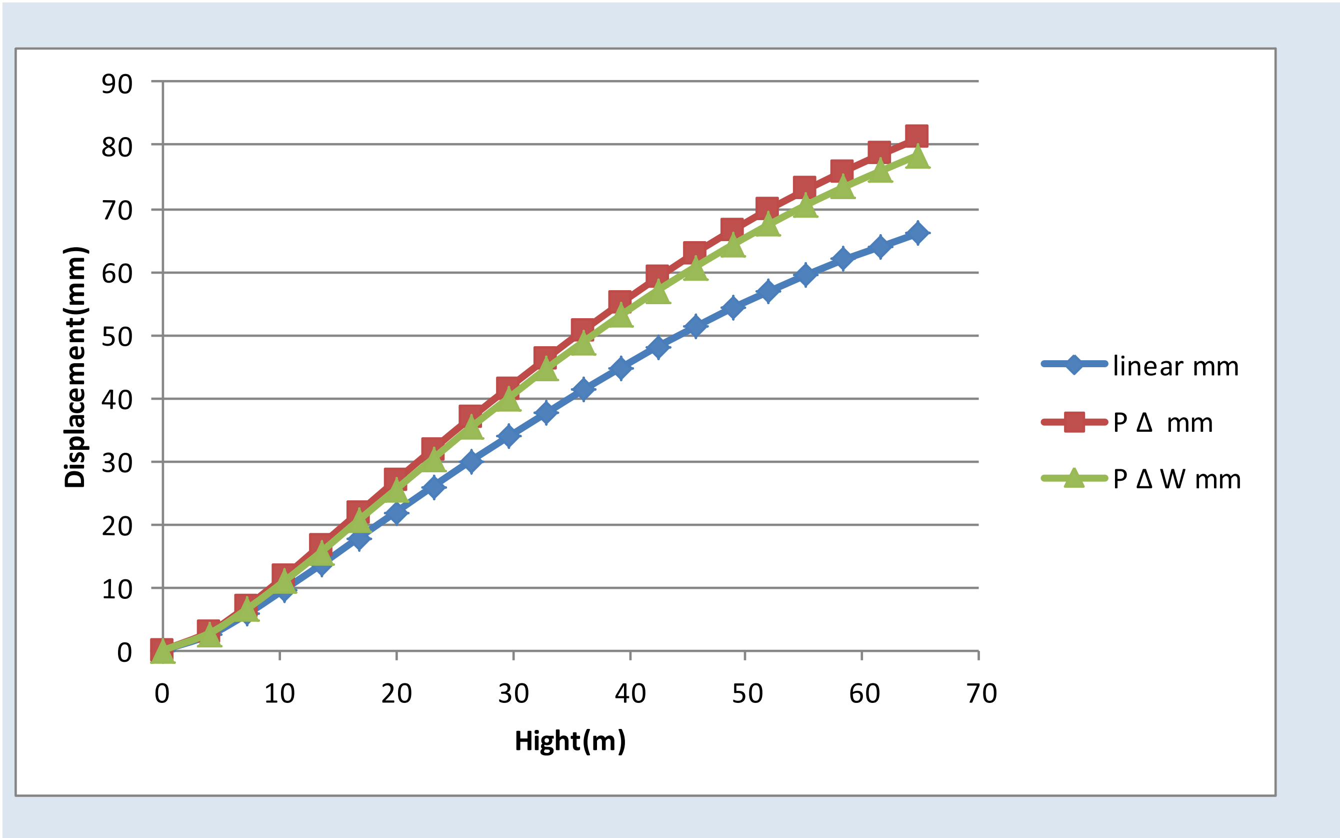


Fig (4.12) Max different displacement in y-y direction

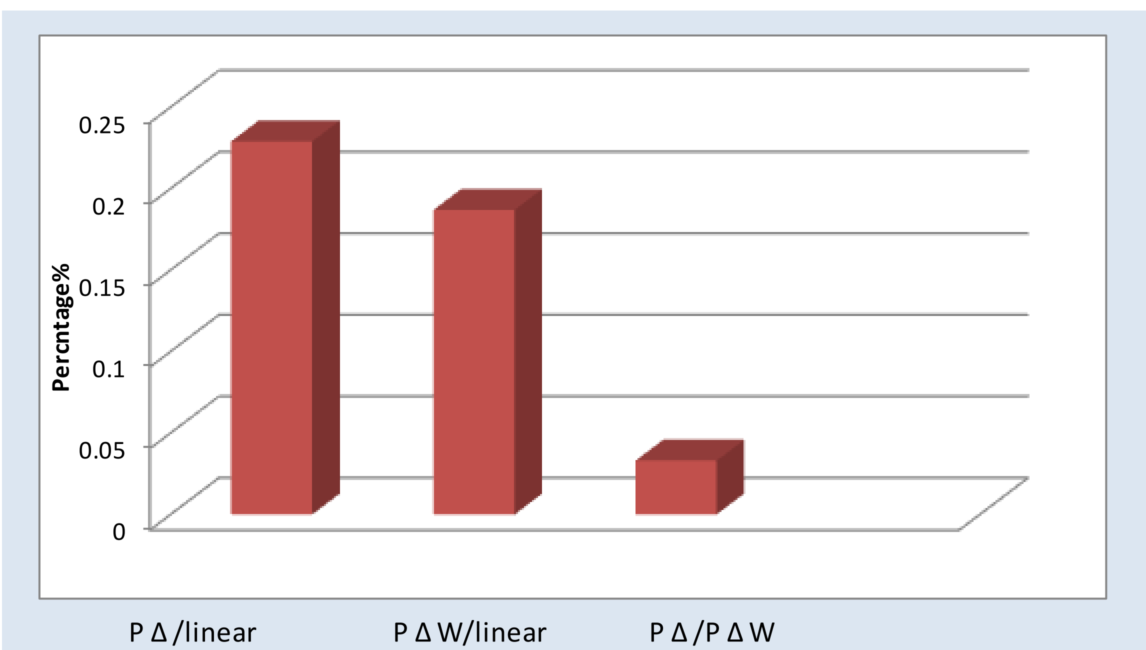


Fig (4.13) displacement percentage

Red: Global Y Displacement

Blue: Global X Displacement

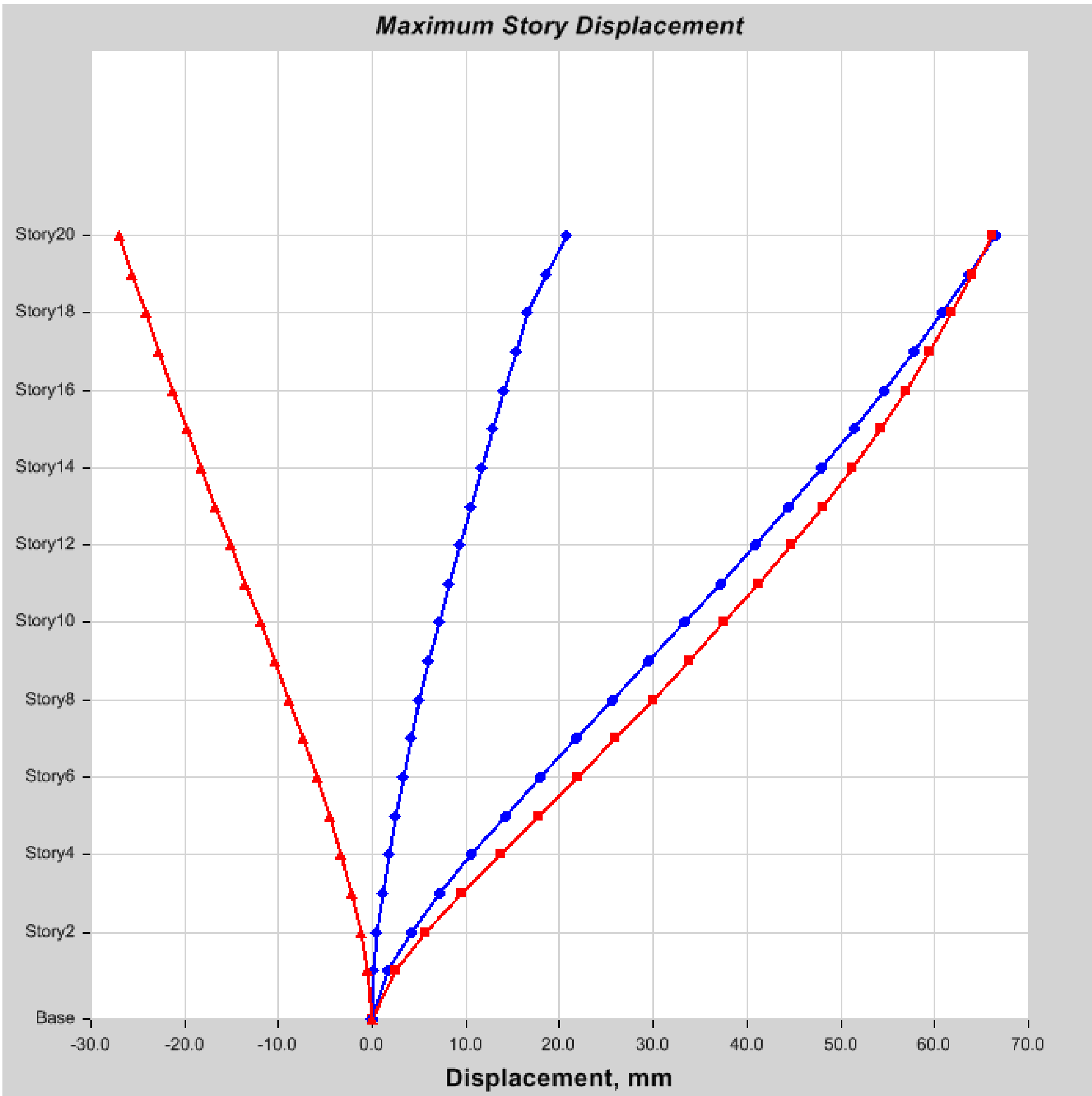


Fig (4.14) Max displacement linear in y-y direction

Red: Global Y Displacement

Blue: Global X Displacement

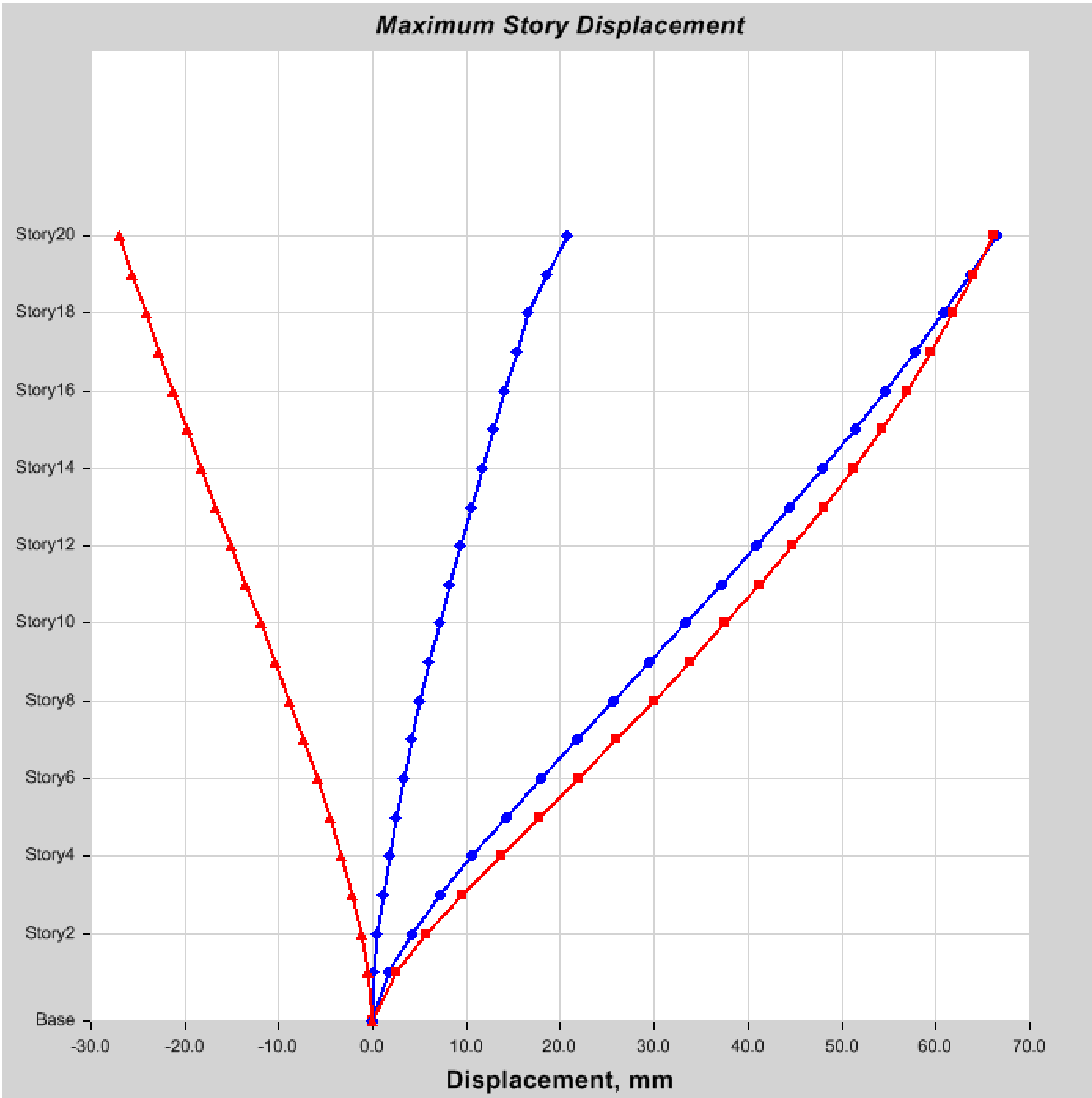


Fig (4.15) Max displacement p ∆ in y-y direction

Red: Global Y Displacement

Blue: Global X Displacement

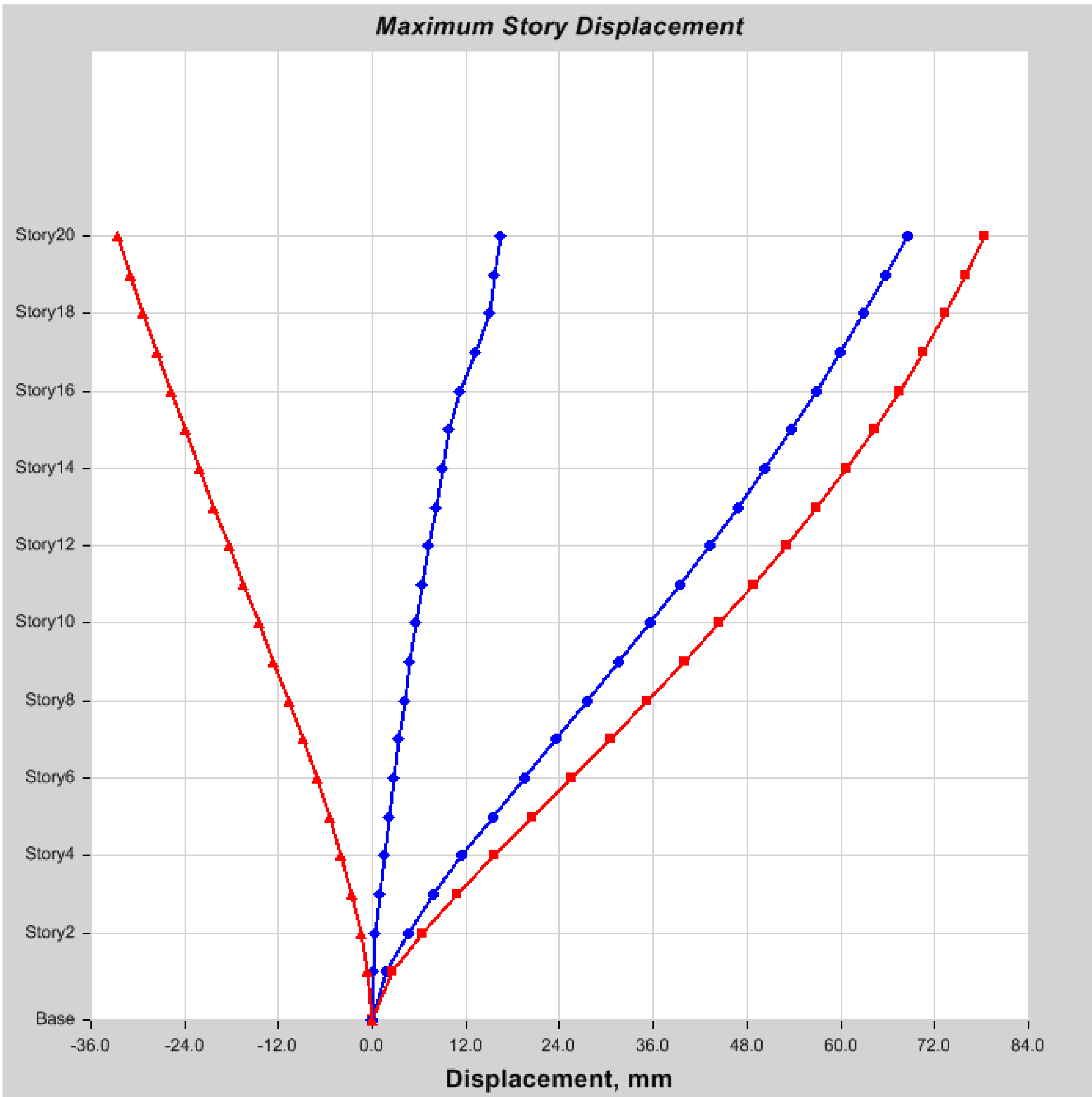


Fig (4.16) Max displacement p ∆ W in y-y direction

4.6.2 Maximum shears:

Figure (4. 5) and Table (4.17) Show the max shears in y-y direction and the percentage difference three analysis types.

Max difference (P Delta- Linear) =938.7-767.6=171.1 kn

Max difference (Large displacement- Linear) =938.5-767.6=170.9 kn

Max difference (P Delta- Large displacement) =938.7-938.5=0.2 kn

- Referring to table (4.5) and figure (4. 18) the maximum shear y-y direction between linear analysis and p ∆ analysis is 22 %. The maximum difference shear y-y direction between linear analysis and p ∆ with large displacement analysis is 22% . The maximum difference in shear y-y direction between p ∆ with large displacement analysis and p ∆ analysis is.02%.These show that there is a large difference between the linear and nonlinear. Aloes it can be seen that the nonlinear results in close agreement.

Table (4.5) Max shears in y-y direction

Story	Elevation	Linear	P Δ	P Δ W	different percentage	different percentage	different percentage
	m	Kn	kn	Kn	(P Δ-Linear) /Linear	(P Δ W-Linear)/Linear	(P Δ -P Δ W)/P Δ W
Story20	64.8	67.6702	82	81.99	21.175939	21.161161	0.012197
Story19	61.6	109.3628	133.03	133.01	21.640997	21.622709	0.015036
Story18	58.4	150.6669	183.58	183.57	21.844944	21.8383069	0.005448
Story17	55.2	191.5669	233.67	233.64	21.978275	21.9626146	0.01284
Story16	52	232.0457	283.27	283.24	22.075091	22.0621628	0.010592
Story15	48.8	272.0843	232.31	332.26	14.618374	22.1165646	30.08186
Story14	45.6	311.6616	380.82	380.76	22.190222	22.1709701	0.015758
Story13	42.4	350.7537	428.74	428.67	22.233921	22.2139638	0.01633
Story12	39.2	389.3338	476.04	475.96	22.270401	22.2498535	0.016808
Story11	36	427.371	522.67	522.58	22.298893	22.2778335	0.017222
Story10	32.8	464.8295	568.6	568.5	22.324422	22.3029089	0.01759
Story9	29.6	501.6674	613.76	613.66	22.344007	22.3240737	0.016296
Story8	26.4	537.8346	658.1	657.98	22.361038	22.3387264	0.018238
Story7	23.2	573.2698	701.52	701.39	22.3717	22.3490231	0.018535
Story6	20	607.8965	743.93	743.8	22.37774	22.3563551	0.017478
Story5	16.8	641.6155	785.2	785.06	22.37859	22.3567697	0.017833
Story4	13.6	674.2916	825.16	825	22.374356	22.3506269	0.019394
Story3	10.4	705.7276	863.54	863.38	22.361659	22.3389875	0.018532
Story2	7.2	735.6023	899.94	899.78	22.340564	22.3188128	0.017782
Story1	4	767.565	938.71	938.53	22.297134	22.2736837	0.019179

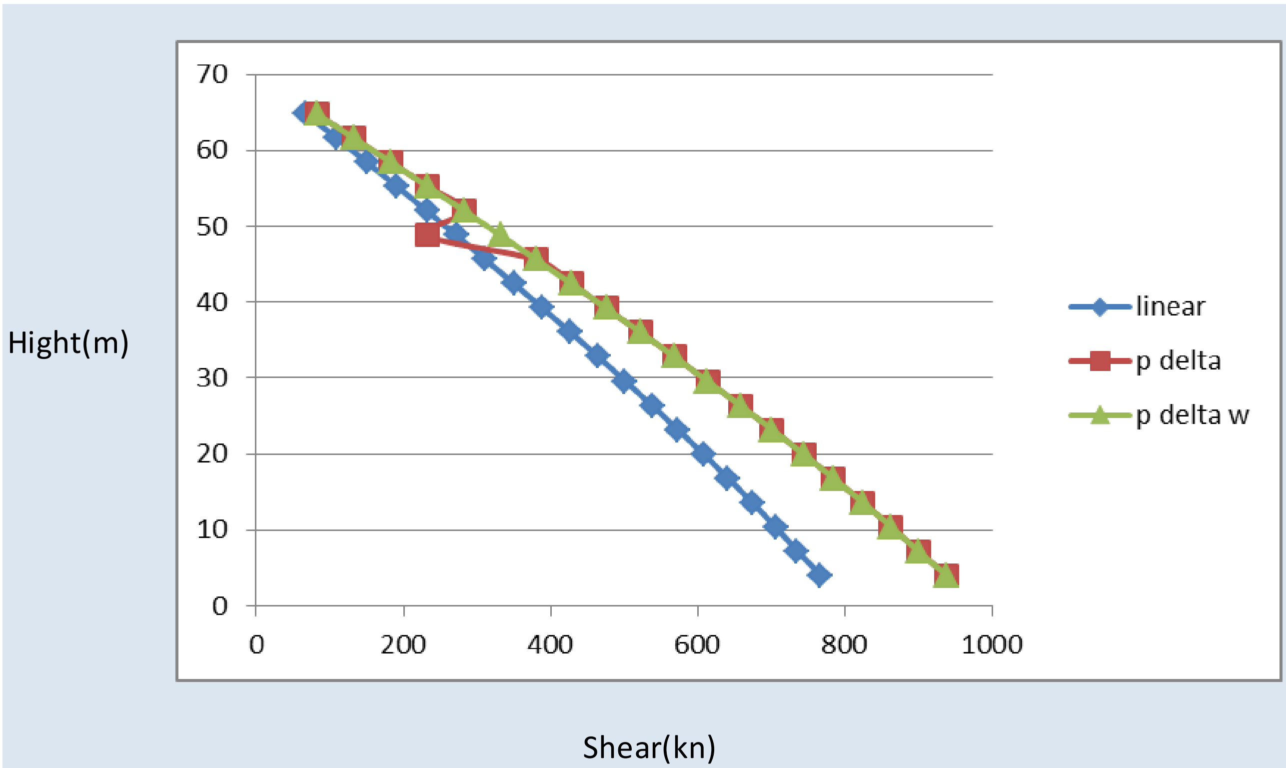


Fig (4.17) Max different shears in y-y direction

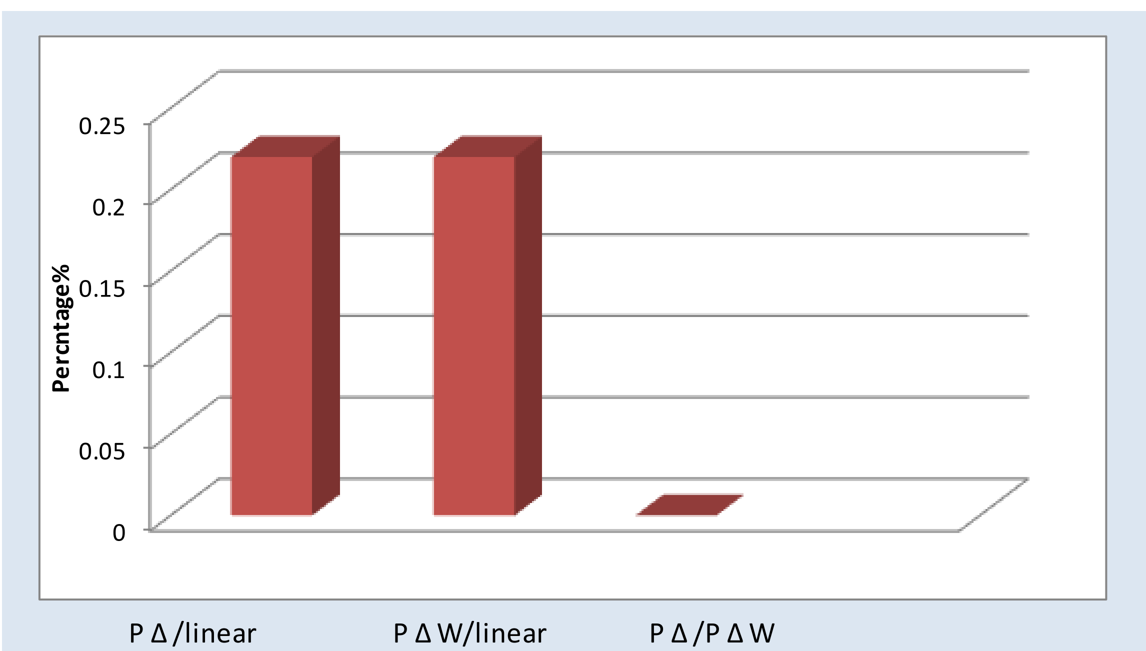


Fig (4.18) shears percentage

Red: Global Y shear

Blue: Global X shear

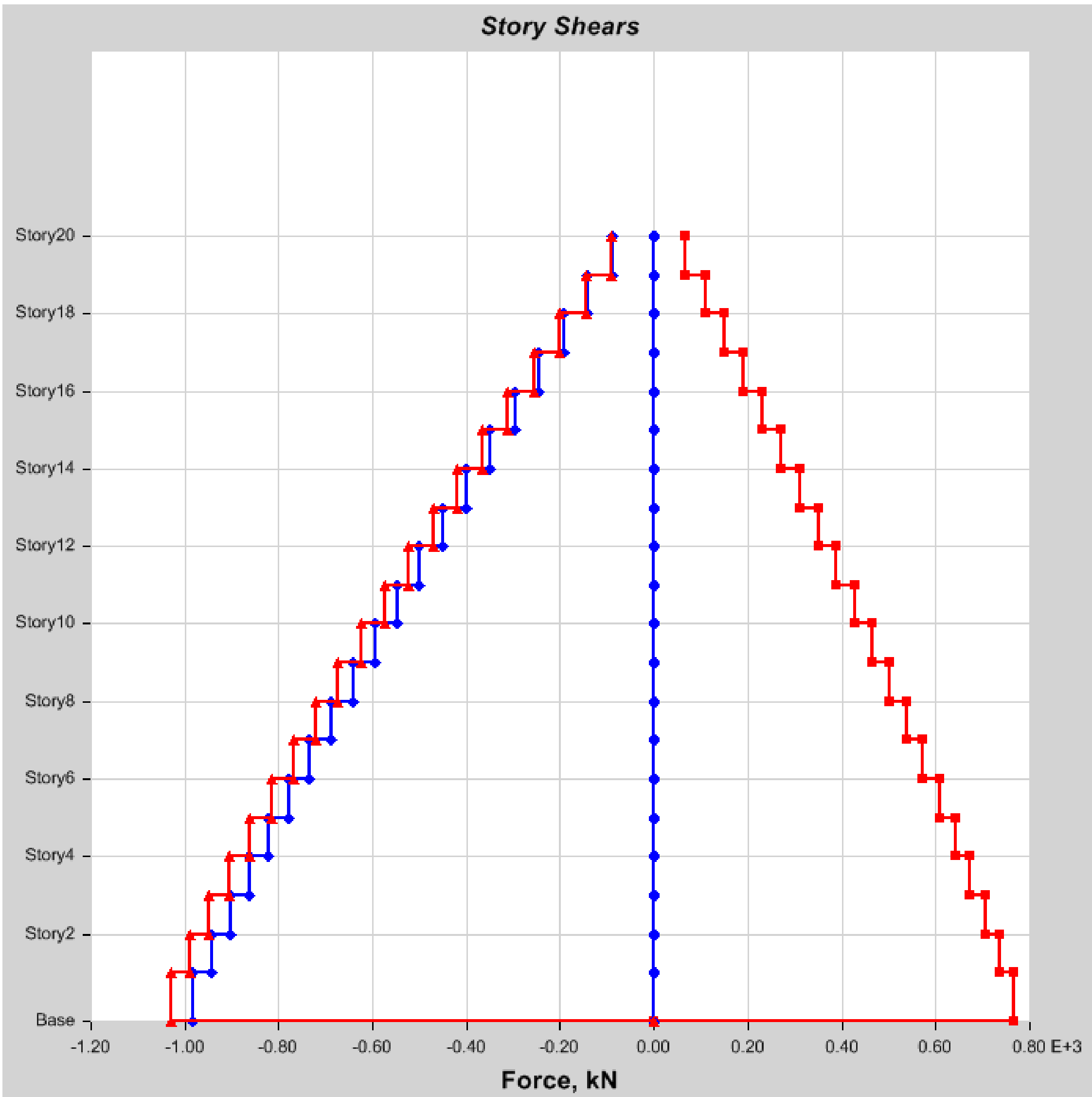


Fig (4.19) Max Shear force linear

Red: Global Y shear

Blue: Global X shear

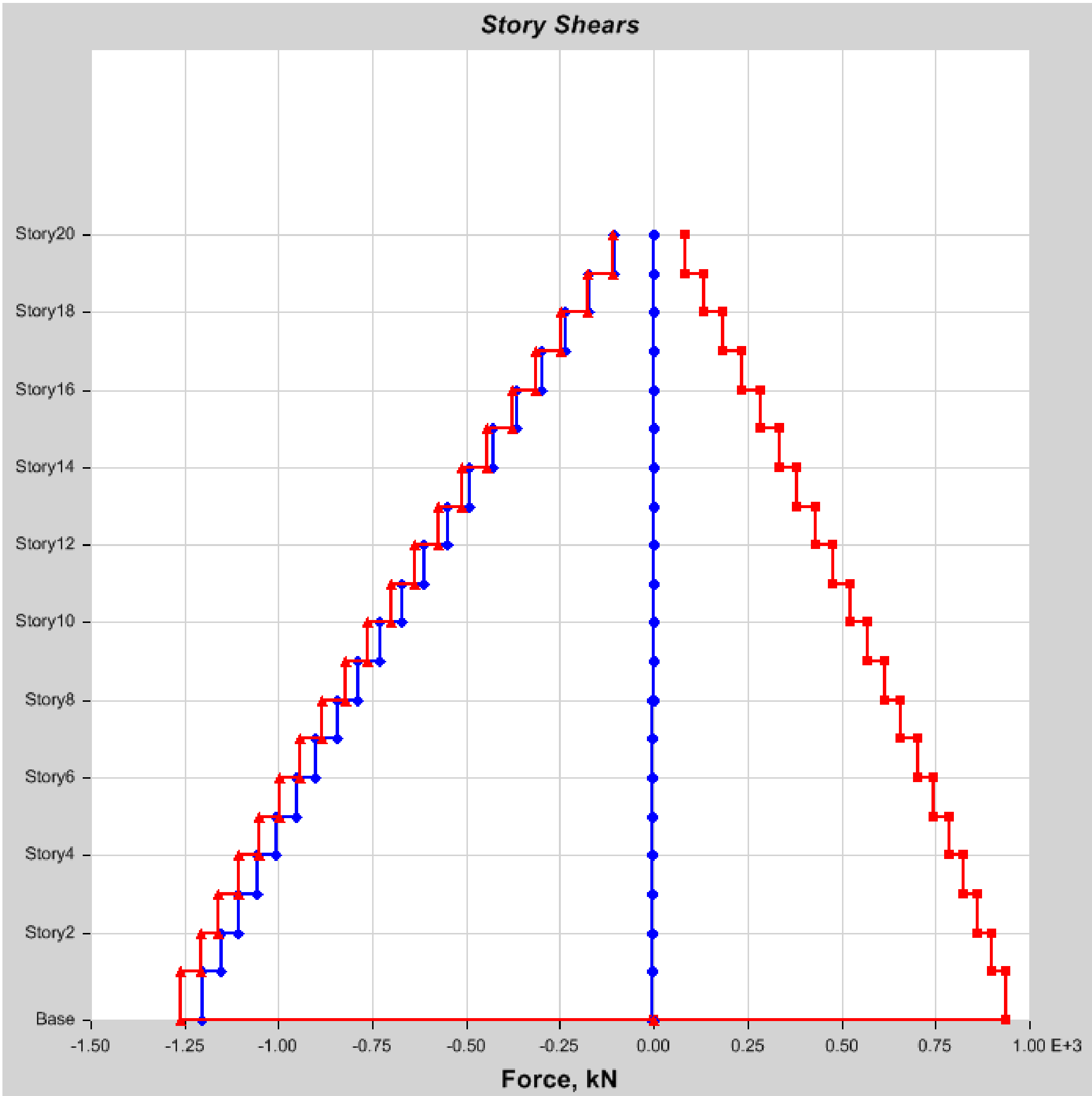


Fig (4.20) Max Shear force (p Δ)

Red: Global Y shear

Blue: Global X shear

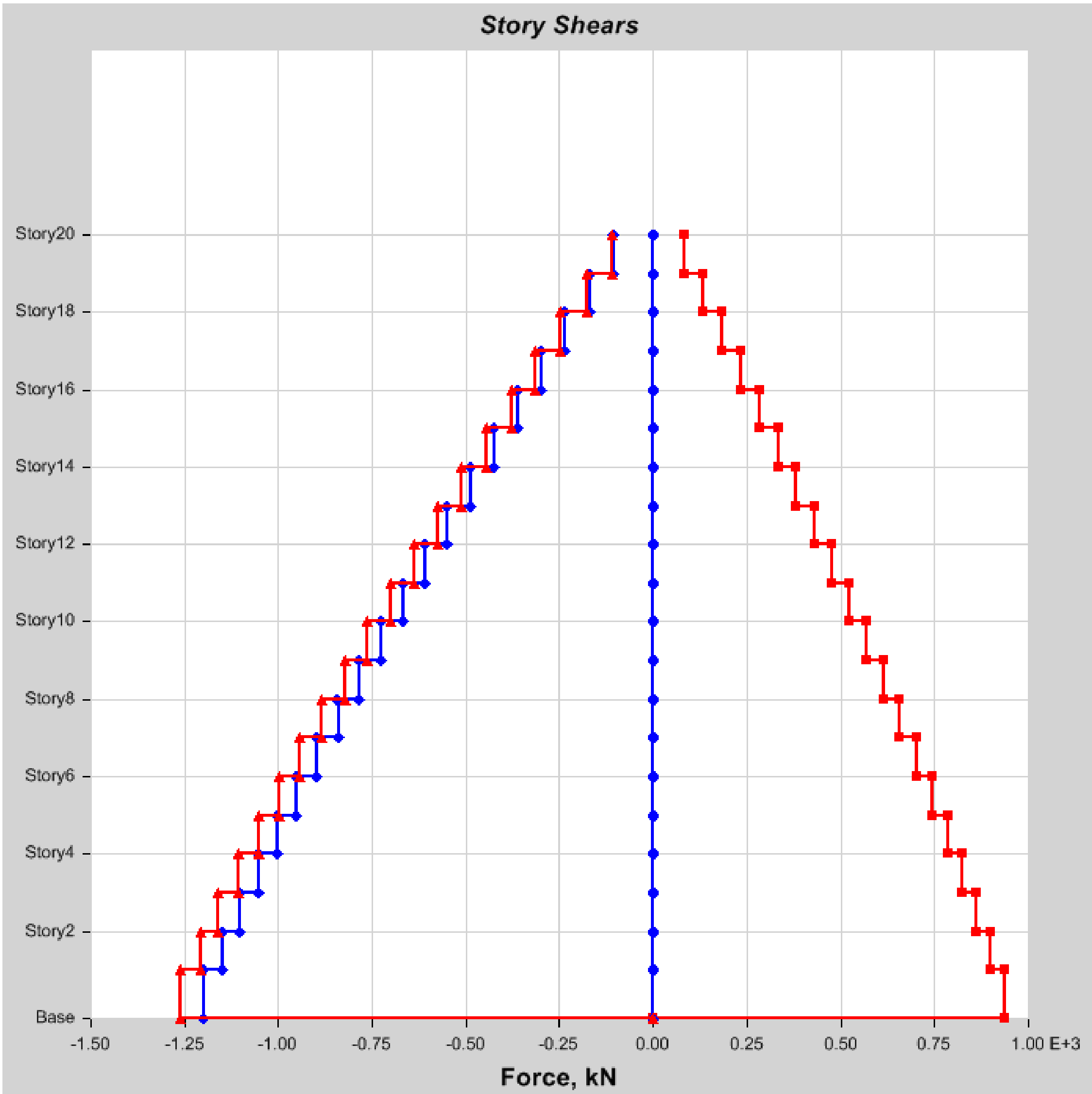


Fig (4.21) Max Shear force

Chapter Five

Conclusions and Recommendations

5.1 Conclusions:

From the research and the result it can be concluded that:

1- Comprehensive literature review was carried out to know the provisions.

2- The wind loading was determine using ASCE-05 for Khartoum area from previous study

3- The linear analysis and nonlinear analysis of the building under the wind loads was carried out using the computer program ETABSv13

4- Comparison of the results of linear analysis of the building and known previous result forwas carried out to check model.

5-From comparison of the results of linear analysis of the building and nonlinear analysis (p.Δ & p.Δ *with large displacement pluse*) it is found that: the maximum differences displacement in y-y direction between linear analysis and pΔ analysis is 22.6%. The maximum differences displacement y-y direction between linear analysis and pΔ with large displacement analysis is 18.7%. The maximum differences displacement y-y direction between pΔ with large displacement analysis and pΔ analysis is 3.3%.This shows the importance of carrying out nonlinear analysis.

6- From comparison of the results of linear analysis of the building and nonlinear analysis (p.Δ & p.Δ *with large displacement pluse*) it is seen that: the maximum differences shears in y-y direction between linear analysis and pΔ analysis is 22 %. The maximum shear y-y direction between linear analysis and pΔ with large displacement analysis is 22%. The maximum differences in shear y-y direction between pΔ with large displacement analysis and pΔ analysis is.02%.These nonlinear analysis must be used to obtain correct shear values.

5.2 Recommendations:

The following recommendations are suggested to be a guide for future study in the subject of this research:

1. To use the nonlinear static analysis it the correct displacement and shear are required
2. To use the p Δ analysis since it gives almost the same result as from p Δ with large displacement analysis while being simple and requiring less computation.
3. To re analysis the case study building system with very large height.
4. To use dynamic analysis and compare with the static analysis result.

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16 – تحليل وتصميم المباني العالية تحت تأثير احمال الرياح والزلازل ’ بحث لنيل درجة البكلاريوس مرتبة الشرفجا, معة السودان للعلوم والتكنلوجيا ا قسم الانشاءات ا, لمعز ساتي محمد احمد , احمد الرشيد الامام الخضر سبتمبر2012

17 – اختيار النظم الإنشائية لمقاومة أحمال الرياح في المباني العالية بحث لنيل درجة البكلاريوس مرتبة الشرفجا, معة السودان للعلوم والتكنلوجيا قسم الانشاءات , حمدالنيل عبدالرحمن علي محمد , محمد السعودي عبدالباقي عبدالباقي, محمد احمد محمد الباقر, اغسطس 2014

Appendices