

Chapter One

Introduction

1.1 General

Throughout the history of humans, there has been a continual quest for height. Humans have always admired tall structures since ancient times because of their social status. Their builders were held in the highest respect of their societies. And most of modern cities contain skyscrapers for their investments values. Development of tall building structures is going according to develop of societies and humans. [Maha, 2012]

A high commercial building was the result of the concentration of businesses in the city centers and facilities for high business is the only solution to keep these institutions relative to each other as much as possible Available as high-rise buildings that used to represent the progress and prosperity architecture in each city so it has established to meet the requirements of the tourism and hospitality. [Maha, 2012]

The Tall building development involves various complex factors such as economics, aesthetics, technology, municipal regulations, and politics. Among these, economics has been the primary governing factor. This new building type itself would not have been possible, however, without supporting technologies. [Maha, 2012]

In the late nineteenth century, early tall building developments were based on economic factor increasing rentable area by stacking office spaces vertically and maximizing the rents of these offices by introducing as much natural light as possible, new technologies were pursued that have improved upon the conventional load-bearing masonry walls with relatively small punched openings. [Maha, 2012]

1.2 Statement of the Research Problem

Approximations and simplifications adopted in making a preliminary analysis are sometimes huge, concerning loading distribution, plastic hinge formed, or when representing a complex bent as a simple cantilever.

Even with gross approximations made in simplifying the structure and affiliated loadings, it is generally expected that a preliminary analysis should give results for deflections and main member forces that are dependably within about 15% of the values of the accurate analysis. [Smith and coull, 1991]

In this research a verification of this percentage and a definition of the level of accuracy of approximate methods with increase in height of the structure, when subjected to pure wind load has been addressed.

1.3 Objectives of the Research

The aim of this research is:

1. Analysis of tall building (rigid frame) due to lateral load used simplified and finite element methods
2. Development of spread sheets for both cantilever and portal methods.
3. Comparison between simplified methods of analysis mentioned above
4. Comparison between simplified methods of analysis and finite element method.

1.4 Methodology

The methodology of research was summarize as follow:

- Comprehensive literature review (Analysis) of frame type Tall building under wind loads.
- Comprehensive methods of analysis.

- Development of an excel calculation sheet for the portal frame method.
- Development of an excel calculation sheet for the Cantilever frame method.
- Comparison of portal and cantilever method.
- Divided the building in to five different levels, each level was higher than the lower level with five floors
- Applies finite element method of analysis used ETABS.
- Comparison of simplified methods (portal& cantilever) and finite element method analysis.

1.6 Thesis layout

This research consists of five chapters as follows:

- Chapter one, Includes general introduction, Research Problem statement, objectives, methodology, and thesis layout.
- Chapter two, Includes literature review of tall building: definition, system of tall building, loads (dead, live, earthquake, wind),
- Chapter three, include Analysis methods (finite element and simplified methods), Details of program for analysis methods Portal and cantilever and provides numerical examples solved by applying the program and checked manually solution.
- Chapter four, presents the analysis and results and discussion of the results.
- Chapter five, Summaries the conclusions and recommendations.

Chapter Two

Back ground and Literature Review

2.1 Introduction

This chapter includes the tall building: definition, historical background, behavior of tall building also the structural systems and some loads which effect on the tall building.

2.1.1 Definition of Tall Building

The 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 [Smith and Coull ,1991].

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 [Smith and Coull, 1991].

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. High-rise is the demand of new era as it provides accommodation to a well number of people in a small place [Samiul & Ayan, 2010]

2.1.2 Behavior of Tall Buildings

A reasonably and accurate assessment of a proposed tall building behavior is necessary to form a properly representative model for analysis. Tall building structures essentially a vertical cantilever that 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 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 that increase 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. [Kidder, 2013].

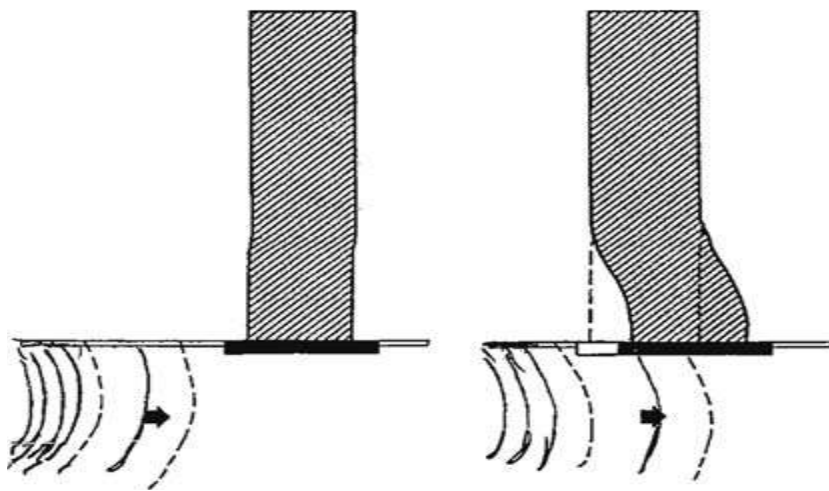


Fig. (2.1) Building behavior during earthquakes.

2.2 Loads

The determination of the loads acting on a structure is a complex problem. The nature of the loads varies essentially with the architectural design, the materials, and the location of the structure. Loading conditions on the same structure may change from time to time, or may change rapidly with time, loads are usually classified into two broad groups' gravity and lateral loads.

2.2.1 Gravity loads

The gravity loads are:

2.2.1.1 Dead load

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 itself. This can be calculated with good accuracy from the design configuration, dimension of the structure and density of the material. For building, floor fill, finish floor and plastered ceiling are usually including as dead loads. [Alaa, 2015].

2.2.1.2 Live load

Live load can be defined as the load whose magnitude and placement change with time. Such loads are due to the weights of people (animals, if the building houses animals), furniture, movable equipment, and stored materials.

Live load depends on the occupancy and the use of the building, and it is different for different occupancies. Based on a large number of surveys, live loads for various commonly encountered occupancies, such as hotels, apartment buildings, libraries, office buildings, and industrial structures, have been determined and are contained in building code tables. [Maha, 2012].

2.2.2 Lateral loads

2.2.2.1 Wind load

While action of lateral loads is orthogonal to the building, which effect negatively on the building's stability as lateral displacement, overturning and twisting, gravity loads appear in the building's own direction and in that way to some extent, have a positive effect on the stability. In all cases, regardless of the direction of the loads the building's main job is to transfer these loads to the ground.

2.2.2.1.1 Outline of procedure for calculating wind load

1. Determines the Dynamic augmentation factor (C_r) depend on:

- Basic geometric (Height) H .
- Structural properties (Type of Building) K_b .

If $C_r < 0.25$ and $H < 300$ then use Method given in part 2 of Bs-6399.

If $C_r > 0.25$ and $H > 300$ the wind load assessed by on of the Methods for Dynamic Buildings.

2. Determines the Basic wind speed (V_b):

3. Determines a site wind speed (V_s) from the basic Speed by applying corrections factors:

(i) S_a : Altitude factor (to adjust the basic wind speed for the altitude of site above sea level)

$$S_a = 1 + 0.001 * \Delta s \dots\dots\dots (2.1)$$

Δs = Height above sea level

(ii) S_p : Probability factor may be used to change the risk of the basic wind speed exceeded from the standard value of $q=0.02$ annually

For all normal design applications standard value of risk $q=0.02$ is use and $=1$

(iii) S_s : Seasonal factor use to basic wind speed for building Which are expect to be exposed to the wind for specific sub annual periods.

For permanent building and building exposed to the wind for a continuous period of more than 6 months a value of 1 should be used for S_s .

(iv) S_d : Direction factor used to adjust the basic wind produce wind speed with same risk of being exceeded in any wind direction

$$V_s = V_b * S_a * S_s * S_d * S_p \dots\dots\dots (2.2)$$

4. Assesses the exposure of the site in terms Terrain roughness and the effective of height.

This stage offers the choice between the standard method and directional method.

- Standard method → conservative and simplified for building up to 100 m height.
- The directional method gives more precise for any given wind direction particularly for site in towns.

5. Determines the effective wind speed.

The effective wind speed is a gust wind speed appropriate to the site exposure and height of the building.

Effective wind speed: V_e

$$V_e = V_s * S_b \dots\dots\dots (2.3)$$

S_b : is terrain and building factor.

6. Converts the effective wind speeds in to an equivalent dynamic pressure:

Dynamic pressure (q_s) for area A

$$q_s = 0.613 * V_e^2 \dots \dots \dots (2.4)$$

Selects pressure coefficients corresponding to the form of the building. In standard method these coefficients correspond to number (usually two or three) of orthogonal load case.

7. Determines the wind speed load from dynamic pressure by the size effect factor.

(BS. 6399-2:1997, Loading for buildings -Part 2)

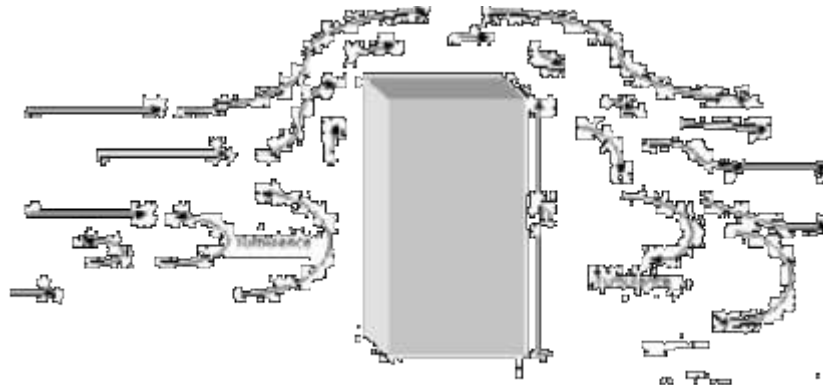


Fig. (2.2) flow of wind around the tall building.

2.2.2.2 Earthquake

An earthquake is sudden, rapid shaking of the earth caused by the beneath the earth surface. Overtime build beneath the earth surface. Occasionally stress is released resulting in the sudden and sometime disastrous shaking which we called an earthquake.

Earthquake destroys construction such as building and high ways. Beside that earthquake causes loss of human and animals lives. [Alaa, 2015]

2.3 Structural System of Tall Buildings

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 efficiently 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 loads, 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 following graph [Smith and Coull, 1991].

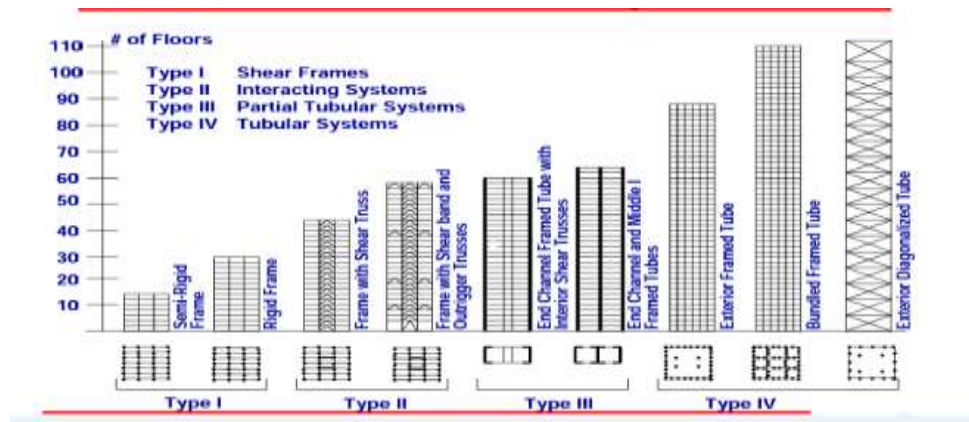


Fig. (2.3) System Structure Classification [Karim and Barua, 2010]

2.3.1 Floor Systems

The floor 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. [Alaa, 2015]

2.3.1.1 Concrete Floor Systems

Slabs of constant thickness are often used with spans 3m to 8m. One or two way systems can be used. The beams are spaced 3m to 8m, and they have usually a depth of $L/15$ to $L/20$. [Alaa, 2015]

2.3.1.2 Steel Floor System

Reinforced concrete on steel beams is used in steel floor system. The thickness of slabs is in the range of $L/30$ to $L/15$ of the span. The spans can be between 1.2 to 9m. Pre-cast concrete slabs (with grouted shear connectors), or concrete slabs on metal decking (with shear connection) are often used. [Alaa, 2015]

2.3.2 Vertical Load Resisting System

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. [Alaa, 2015]

2.3.3 Lateral load Resistance System

In contrast with the vertical load, lateral load effects on buildings are quite variable and 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-story drift in a minimum range. The following lateral systems exist:

2.3.3.1 Moment Resisting Frame

These are column and girder plane frames with fixed or semi rigid connections. They can be constructed from concrete, steel or composite materials. One can observe from (Fig (2.4)) that moment resisting frames can be sufficient for a building up to 30 stories. [Alaa, 2015]

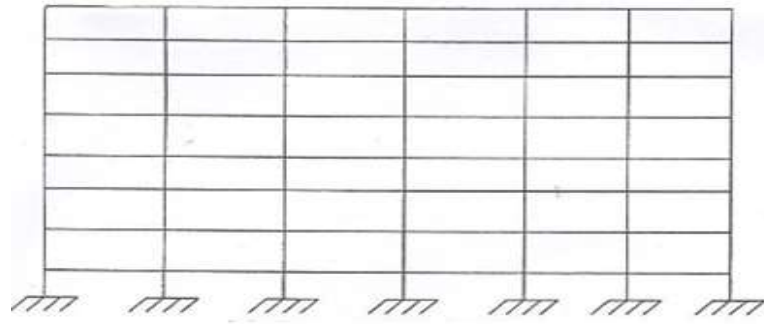


Fig. (2.4) A Typical 8-Storey Moment Resisting Frame [Alaa, 2015]

2.3.3.2 Braced Frames

Braced frames may be grouped into two categories, as either concentric braced frames (CBF) or eccentric braced frames (EBF), depending on their geometric characteristics. CBF (the axes of all members i.e. columns, beams and braces intersect at a common point such that the member forces are axial. They can be figured in various forms, such as the following Fig. (2.5). (a) one story x-bracing; (b) single diagonal bracing; (c),(d) chevron bracing; (e) single diagonal, alternate direction bracing; (f) two story x-bracing.

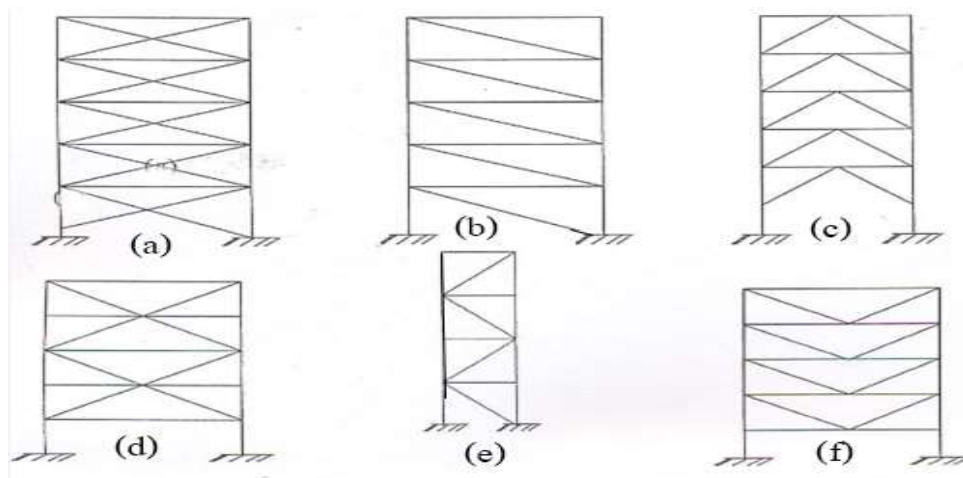


Fig. (2.5) Typical Concentric Braced Frame (CBF) [Taranth, 2005]

EBF (eccentric braced frames) utilize axis offsets to deliberately introduce flexure and shear into frame beam. The primary goal is to increase ductility. Eccentric beam

elements yielding either in shear or in bending act as fuses to dissipate energy during severe earthquakes.

The yielding of the link does not because the structure to collapse, the structure continues to retain its vertical load –carry capacity.

Eccentric braced frames can be configured in various forms as long as the brace is connected to at least one link. The underlying principle is to prevent buckling of the brace from large overload that may occur during major earthquakes. This is achieved by designing the link to yield to distress in other structural moments.

The most efficient (but also the most obstructive) types of bracing are those that form a fully triangulated vertical truss. [Taranth, 2005]

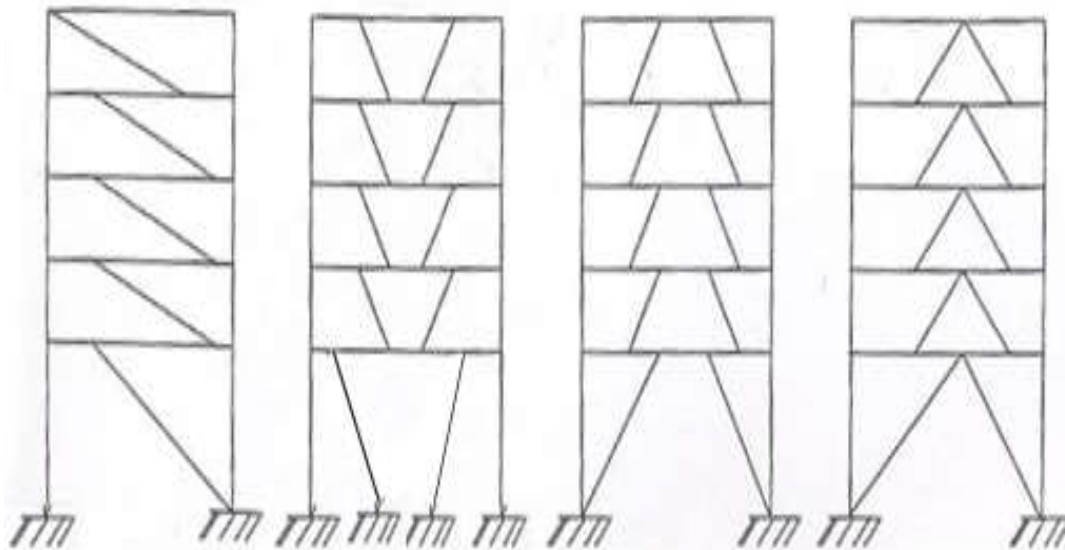


Fig. (2.6) Common Types of Eccentric Braced Frames [Taranth, 2005]

2.3.3.3 Staggered Truss System:

The concept of the staggered truss system was developed by a team of architects and engineers from the department of Architecture and Civil Engineering at the Massachusetts Institute of technology (M.I.T) who combined their respective talents to achieve this imaginative and efficient steel framing system. The system consist of series of story-height trusses spanning the total width between two rows of exterior column and arranged in staggered pattern on adjacent column lines, as shown in the following Fig. (2.7). [Alaa, 2015]

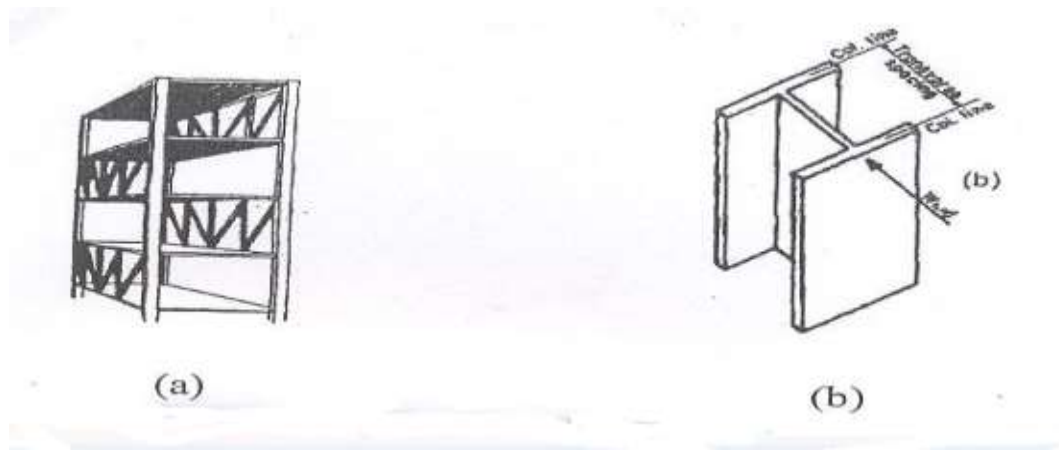


Fig. (2.7) (a) Staggered Truss Arrangement (b) Basic Concept [Alaa, 2015]

The basic concept of the staggered truss system is that the total frame of the building behaves as a cantilever beam when subjected to lateral loads. In this content, all columns are placed on the exterior wall of the building and function as flanges of the beam, while the trusses which span the total transverse width between columns function as the web of the cantilever beam Fig. (2.7(b)). With the columns only on the exterior walls of the building the usual interior columns are omitted, thus providing full width of column free area.

The floor system spans from the top chord of one truss to the bottom chord of the adjacent truss. Therefore, the floor become a major component of the structural system

as a diaphragm transferring the lateral shears from one column line to another, thus enabling the structure to perform as a single braced frame, even though the trusses lie in two parallel lines. The cantilever action of the double planer truss system Fig (2.8) (b) due to lateral loads minimizes the bending moment in the columns. Therefore in general the columns are designed for axial loads only and can be oriented with their webs perpendicular to the trusses, thus eliminating local bending due to the connection of the truss chord.

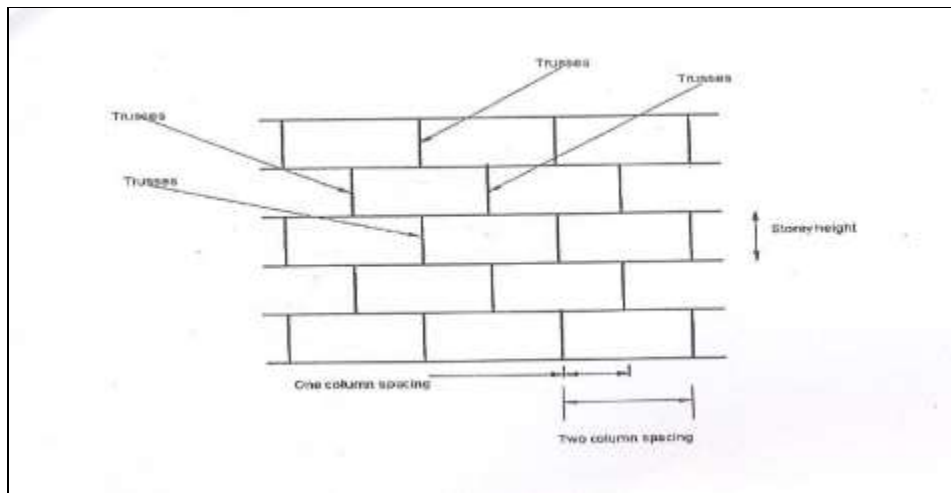


Fig. (2.8) (a). Longitudinal elevation of Fig. (2.7 a) [Alaa, 2015]

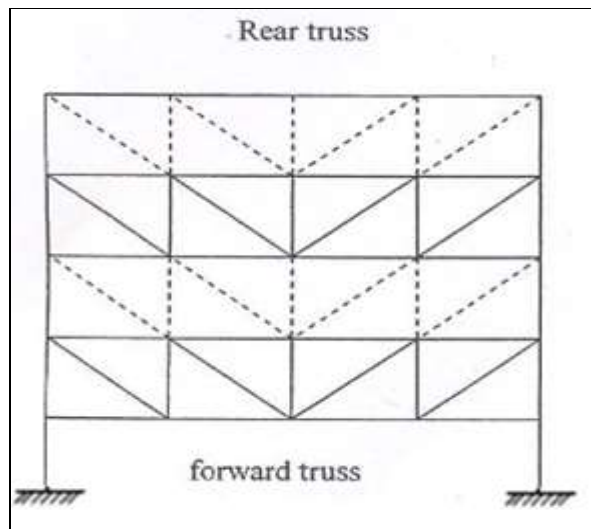


Fig. (2.8) (b) Double planer Braced Frame, side view for Fig. (2.7 a)

2.3.3.4 Outrigger and Belt Truss System

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 (2.9). [Taranth, 2005]

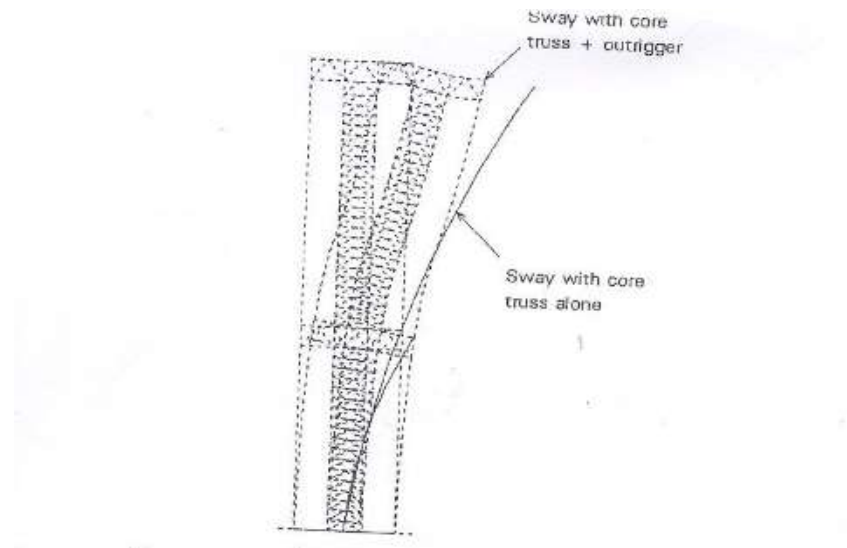


Fig. (2.9) Double outrigger effect on a tall building [Taranth, 2005]

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. Fig (2.10)

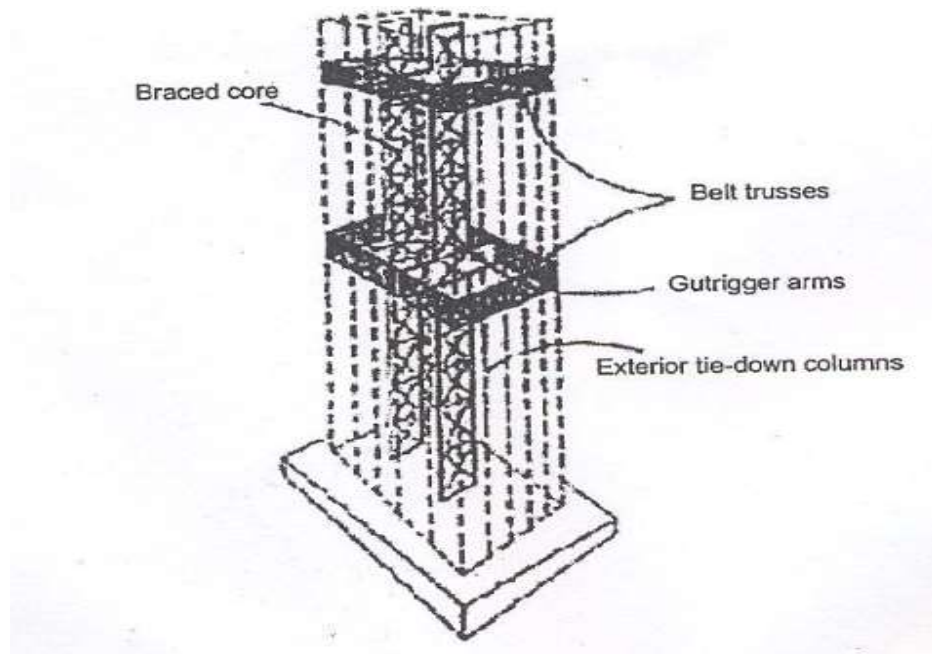


Fig. (2.10) Single Outrigger and Belt Truss Schematic [Taranth, 2005]

2.3.3.5 Framed Tube System

A framed tube can be defined as a three dimensional system 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. 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. [Taranth, 2010]

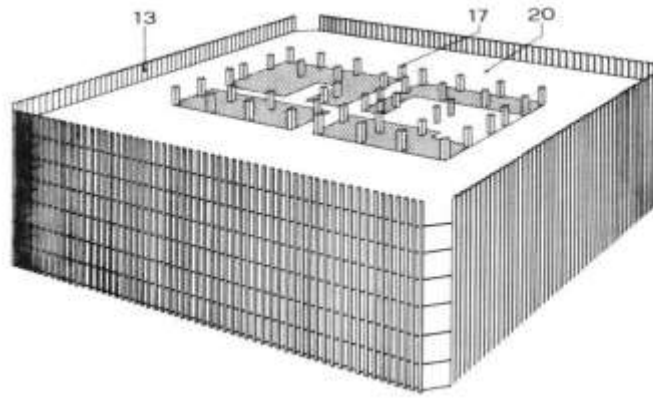


Fig. (2.11) Frame tube [Taranth, 2005]

2.3.3.6 Trussed Tube

A trussed tube improves on the efficiency of the framed tube by increasing its potential for use in taller buildings and allowing greater spacing between the columns. This is achieved by adding diagonal bracing at the faces of the tube to virtually eliminate the shear lag in both the flange and web frames. [Taranth, 2005]

2.3.3.7 Bundle Tube

A bundle tube consists typically of a number of tubes interconnected to form a multi cell tube, in which the frames in the lateral load direction resist the shears, while the flange frames carry most of the overturning moments. The cells can be curtailed at different heights without diminishing structural integrity. The shear lag experienced by conventional framed web tube is greatly reduced by the addition of interior framed web panels across the entire width of the building. When the building is subjected to bending under the action of lateral forces, the height in plane rigidity of the floor slabs constrains the interior web flanges to deflect equally with the exterior web frames. Because a bundled tube is configured from a layout of individual tubes, it is possible to achieve a variety of floor configurations by simply terminating a given tube at any desired level. [Taranth, 2005]

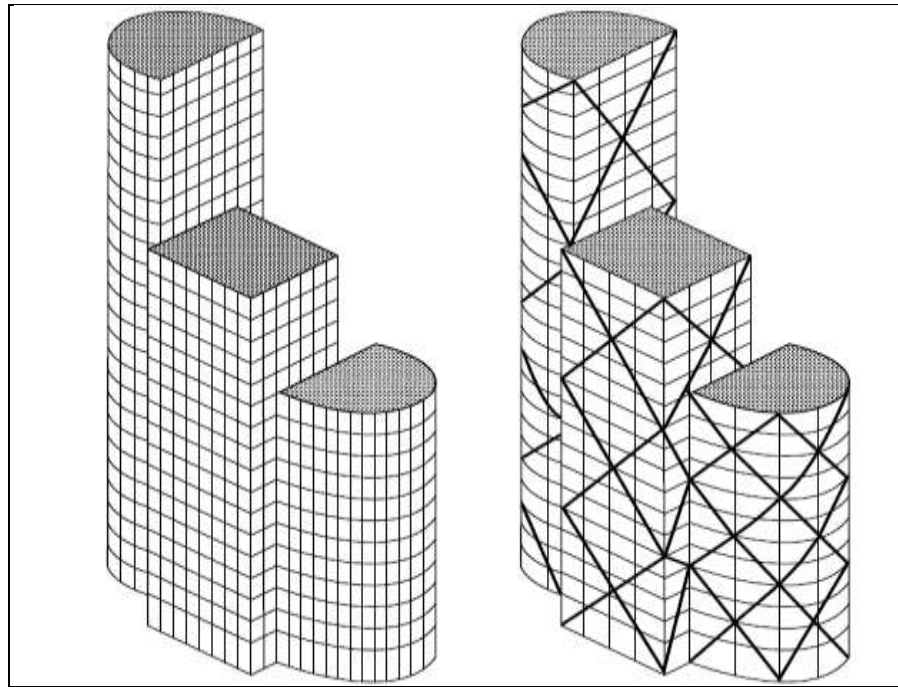


Fig. (2.12) Schematic Bundled Tube [Tarant, 2005]

2.3.3.8 Shear Wall Structures

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”. [Coull, 1991]

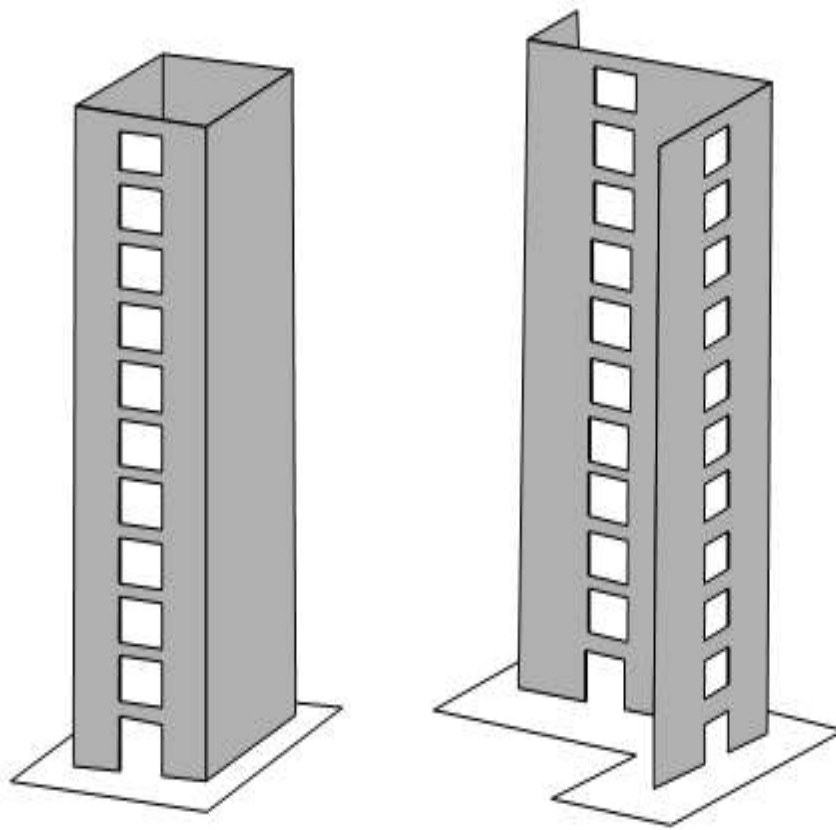


Fig. (2.13) Coupled Shear Walls [Tarant, 2005]

2.3.3.9 Wall-Frame Structures

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. [Coull, 1991]

2.3.3.10 Rigid Frame Structures:

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. The advantages of a rigid frame are the simplicity and convenience of its rectangular shape. [Taranth, 2010]

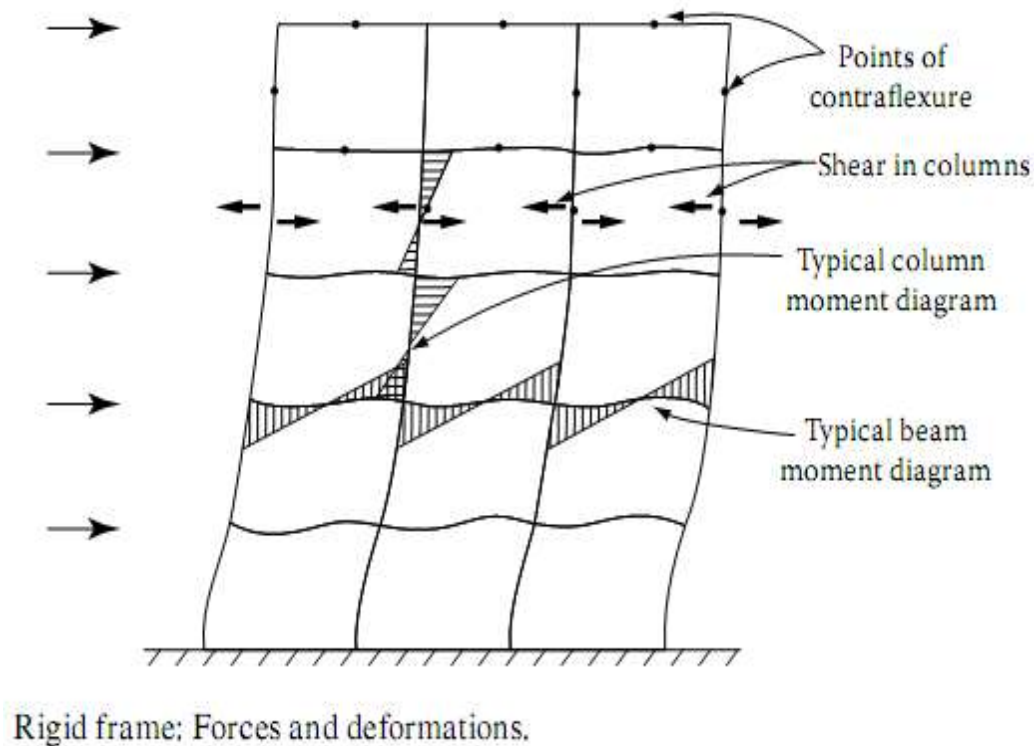


Fig. (2.14) Rigid Frame Structures [Taranth, 2010]

2.3.3.11 Core Structure

Elevator cores are primary components for resisting both horizontal and gravity loading in tall building structures. Reinforced concrete cores usually comprise an assembly of connected shear walls forming a box section with openings that may be partially closed by beams or floor slabs. The moments of inertia of reinforced concrete core are invariably large, so that they are often adequate in itself to carry the whole of the lateral loading. [Coull, 1991]

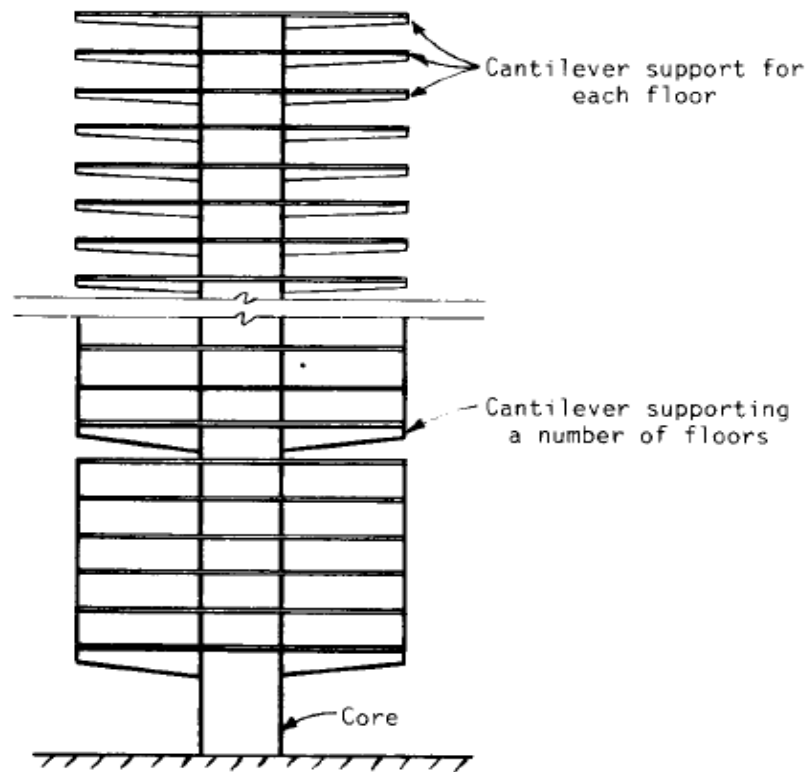


Fig. (2.15) Core Structure [Coull, 1991]

2.4 Structural Analysis

To an engineer or architect involved with the design of buildings, it is required to make many technical decisions about structural systems. These decisions include, firstly selecting an efficient, economical, and attractive, secondly evaluating its safety, that is, its strength and stiffness; and thirdly planning its erection under temporary construction loads.

To design structure, structural analysis will be carry out to establish internal forces and deflections at all points produced by the design loads.

The methods of analysis divided to linear and nonlinear analysis are the two basic methods. Linear elastic analysis is generally used for multi-story structures due to its simplicity; Mention some of the approximate ways.

2.4.1 Analysis of Rigid Frames

The horizontal stiffness of rigid frame is governed by the bending resistance of the girders, columns, and joints, and in a tall frame by the axial rigidity of columns.

The accumulated horizontal shear above any story is resisted by shear in the columns of that story. The shear causes the story-height columns to bend in double curvature with points of contra flexure at mid-story height. The moments applied to a joint from the columns above and below are resisted by the girder. The girders also bend in double curvature with points of contra-flexure at mid-span. These deformations in the columns and girders allow racking of the frame and horizontal deflection. The overall deflected shape of the frame due to racking has a shear configuration. The overall moment of the external horizontal load is resisted at each story by the couple resulting from the axial forces in the columns. Because of the cumulative rotation up the height, the story drifts due to bending increases with height, while that of racking tends to decrease. The contribution to story drift from bending may, in the uppermost stories, exceed that from racking. The contribution of bending to the total drift will usually not exceed 10% that of racking, except in very tall, slender frames. The overall deflection shape a high-rise rigid frame usually has a shear configuration. (Hibbler, 2012)

2.5 Previous Studies

- In the research, the structural systems normally used in tall buildings are presented. The linear and nonlinear finite element analysis of tall building under seismic loads is carried out .the main purpose was to study the importance of nonlinear analysis for displacement and shear force.

A selected tall building of reinforced concrete skeleton having height of 20 stories was checked by increasing the number of nodes (one node and two nodes) in the corner columns and compared the curve of the displacement, then have been analyzed and

designed . Dead load, live load, and seismic load were applied to the selected buildings in accordance to BS 8110-1997. The selected building were analyzed using the finite element structural analysis program ETABS in linear and nonlinear mode for any load alone and for all loads together.

The result obtained (the maximum displacement in linear analysis 53.5 mm, in p_{Δ} case 83.4 and p_{Δ} plus large displacement 85.6 also the maximum shear force were 395.8 kN, 641.8 kN and 900.9kN respectively) were analyzed and discussed these clearly show the necessity of nonlinear analysis of real displacement are required and the importance of the p_{Δ} plus large displacement analysis if the correct shear forces are to be obtained . (Alaa, 2015)

- The research is concerned with the study of the effects of wind and earthquakes on tall buildings. Tall buildings are buildings in which the slenderness ratios (heights to the smaller plan dimensions) are large. This makes such buildings more than exposed to the horizontal forces resulting from wind, earthquakes, water pressure and other similar actions. This research is concerned with just wind and earthquakes because they have more probability to occur in Sudan.

Many structural forms are used to resist horizontal forces like from systems, shear wall and combined systems.

A study of a building consisting of twenty stories has been done by analyzing it using the computer program (ETABS9). The analysis was carried out for the building under three different systems of load, namely (D.L+L.L), wind and earthquakes loads .A comparison has been made between wind and earthquakes loads as additional loads applied on the building besides its own loads (D.L+L.L) so as to study the effect of wind and earthquakes loads on tall building and what to expect when those loads are neglected. These results are displayed on figures to explain their effect the on buildings. Based on these results some recommendations have been drawn for the structural

engineers to consider in the design stage of high-rise buildings subject to wind and earthquakes (Abbas Mahgoub-2009).

- This research is concerned with the study of the effect of wind load on tall buildings. In the study the wind load is dealt with since it is more likely to take place in Sudan and affect tall buildings. The importance of carrying out nonlinear $p-\Delta$ and $p-\Delta$ with large as compared to linear static analysis is studied.

A twenty story building is analyzed linearly and nonlinearly for wind loads via the commercial software ETABS v13. The linear results are then presented graphically against results of a previous study of a similar building for the comparison so as to check the accuracy of the model. Also nonlinear analysis using $p-\Delta$ and $p-\Delta$ with large displacement methods and linear results are compared with each other. The results wear compared for displacement and shear.

On bases of these recommendations on the analysis type necessary for tall building subjected to wind loads are drawn. The results shown that the use of the $p-\Delta$ with large displacement analysis is important it the correct displacement and shear force are to be obtained (Khadija- 2015).

Chapter three

Methods of Analysis

3.1 Introduction

This chapter deals with simple program to analyze rigid frame used portal and cantilever methods analysis. Program used is Excel, it's the best application ever written for windows. Excel is one of the oldest windows products and has undergone many reincarnations and face-lifts over the years. Speed of the works, which include large calculations, perform calculations easily with the possibility of the amendment is automatically simply by changing the values used in the calculation, Representation of data in the values used in the calculation, Representation of data in the image graph in different formats and other advantage of excel.

3.2 Finite Element Method

The finite element analysis is a numerical technique. In this method all the complexities of the problems, like varying shape, boundary conditions and loads are maintained as they are but the solutions obtained are approximate. Because of its diversity and flexibility as an analysis tool, it is receiving much attention in engineering. The fast improvements in computer hardware technology and slashing of cost of computers have boosted this method, since the computer is the basic need for the application of this method. A number of popular brand of finite element analysis packages are now available commercially. Some of the popular packages are STAAD-PRO, GT-STRUDEL, NASTRAN, NISA and ANSYS. Using these packages one can analyze several complex structures Civil engineers use this method extensively for the analysis of beams, space frames, plates, shells, folded plates, foundations, rock mechanics problems and seepage analysis of fluid through porous media. Both static and dynamic

problems can be handled by finite element analysis .In a continuum, these unknowns are infinite. The finite element procedure reduces such unknowns to a finite number by dividing the solution region into small parts called elements and by expressing the unknown field variables in terms of assumed approximating functions (Interpolating functions/Shape functions) within each element. The approximating functions are defined in terms of field variables of specified points called nodes or nodal points. Thus in the finite element analysis the unknowns are the field variables of the nodal Points. Once these are found the field variables at any point can be found by using interpolation functions. After selecting elements and nodal unknowns next step in finite element analysis is to assemble element Properties for each element.

Basic equation

$$[k]_e \{\delta\}_e = \{F\}_e \quad \dots \dots \dots (3.1)$$

Where $[k]_e$ is element stiffness matrix, $\{\delta\}_e$ is nodal displacement vector of the element and $\{F\}_e$ is nodal Force vector. (Bhavikatti, 2010)

3.2.1 ETABS Software

The commercially available computer program ETABS (Computers & Structures, Inc., 1995) was chosen for the numerical analysis in this thesis for two reasons. Firstly, 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. Secondly, 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

The package is a fully integrated system for modeling, analyzing, designing, and optimizing structures of tall buildings, the program utilize graphical user interface (GUI). It is provide multiple units systems, preferences for most codes of design such as (ACI, BS, UBC,IBC and Euro code) are included, automated lateral loads- wind, seismic- and provide static and dynamic analysis, linear and nonlinear analysis, with features of importing from other programs (plan, three dimensional frame from AutoCAD) , exporting the objects and results for other program (i.e. floor to SAFE, reactions for designing raft) for more process , and export to the spread sheets (excel).

3.3 Simplified Methods

There are many simplified methods used to analyze structural systems like portal and cantilever methods used to analyze due to lateral loads.

3.3.1 The Portal Method

The portal method was initially developed by A. Smith in 1915 and is generally considered to be appropriate for the approximate analysis of relatively low building frames.

The following are the simplifying assumptions made in the portal method:

1. A hinge is placed at the center of each girder, since this is assumed to be a point of zero moment.
2. A hinge is placed at the center of each column, since this is assumed to be a point of zero moment.

3. The axial forces in internal columns equal zero. The above assumptions convert the indeterminate multi-story frame to a determinate structure. The steps involved in the analysis of the frame are detailed below:

Step 1: The Axial forces on the external columns of the floor according to assumption (2).

Step 2: The beam shears are determined joint by joint from the column axial forces.

Step 3: The horizontal shears on each level are distributed between the columns of that floor according to assumption (1).

Step 4: The moment in each column is equal to the column shear multiplied by half the column height according to assumption (2).

Step 5: The moment in each girder is equal to the axial force in column multiplied by half the girder length according to assumption (1).

Assumptions of the Portal method of analysis are diagrammatically shown in Fig. (3.1)

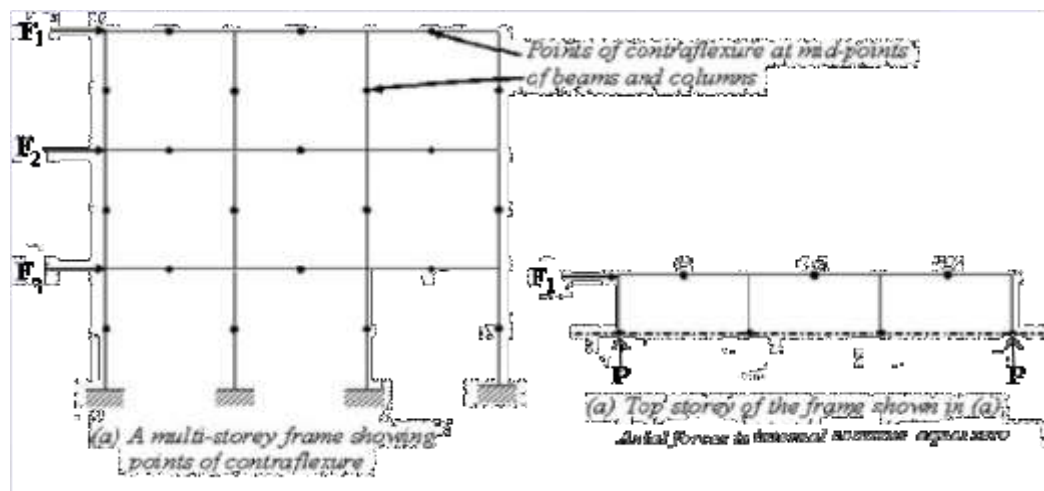


Fig. (3.1) multi-story frame subjected to wind loading

3.3.2 The cantilever method:

The cantilever method was initially developed by A. C. Wilson in 1908 and is generally considered to be appropriate for the approximate analysis of relatively tall building frames.

The simplifying assumptions made in the cantilever method are:

1. A hinge is placed at the center of each girder, since this is assumed to be a point of zero moment.
2. A hinge is placed at the center of each column, since this is assumed to be a point of zero moment.
3. The axial stress in a column is proportional to its distance from the centroid of the cross-sectional areas of the columns at a given floor level. Since stress equals force per area, then in the special case of the columns having equal cross-sectional areas, the force in a column is also proportional to its distance from the centroid of the column areas.

The steps involved in the application of this method are:

Step 1: The center of gravity of columns is located by taking moment of Areas of all the columns and dividing by sum of the areas of columns.

Step 2: A lateral force P acting at the top story of building frame is shown in Fig. (3.2). The axial forces in the columns are represented by F_1 , F_2 , F_3 and F_4 and the columns are at a distance of x_1 , x_2 , x_3 and x_4 from the centroidal axis respectively as shown in Fig. (3.3).

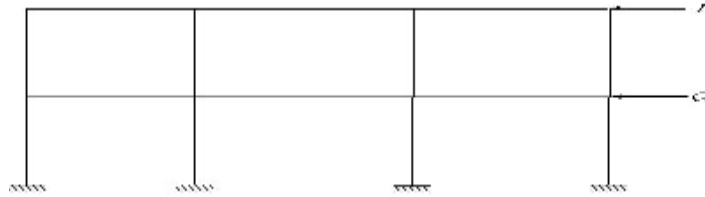


Fig (3.2) typical frame

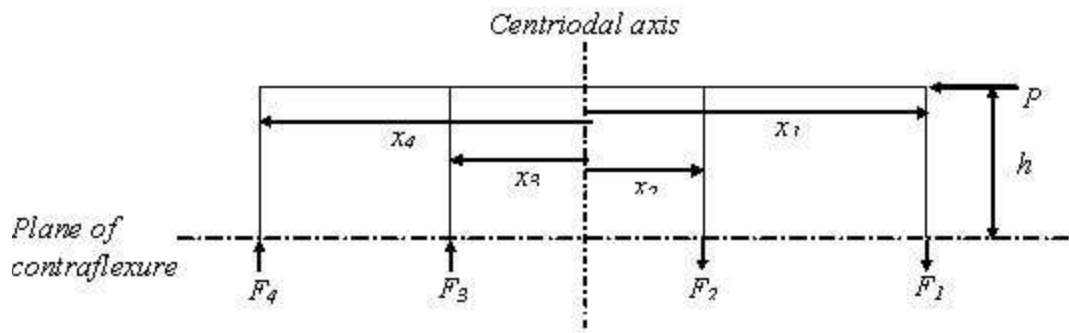


Fig (3.3) Top story of the frame above plane of contra flexure

By taking the moments about the center of gravity of columns of the story,

$$Ph - F_1X_1 - F_2X_2 - F_3X_3 - F_4X_4 = 0 \dots \dots \dots (3.2)$$

The axial force in one column may be assumed as F and the axial forces of remaining columns can be expressed in terms of F using assumption (3).

Step 3: The beam shears are determined joint by joint from the column axial forces.

Step 4: The beam moments are determined by multiplying the shear in the beam by half the span of beam according to assumption (1), heights using assumption (2)

Step 5: The column moments are found joint by joint from the beam moments.

The column shears are obtained by dividing the column moments by the half-column heights using assumption (2).

3.4 Developed Program for Portal Method:

Developed portal program by used the assumption shown in section (3.3.1) and drive some equation to analyzed shown in section (3.4.1).

3.4.1 Equations for developed program:

The derivation of general equations shown below from Figs. (3.4) (3.5)

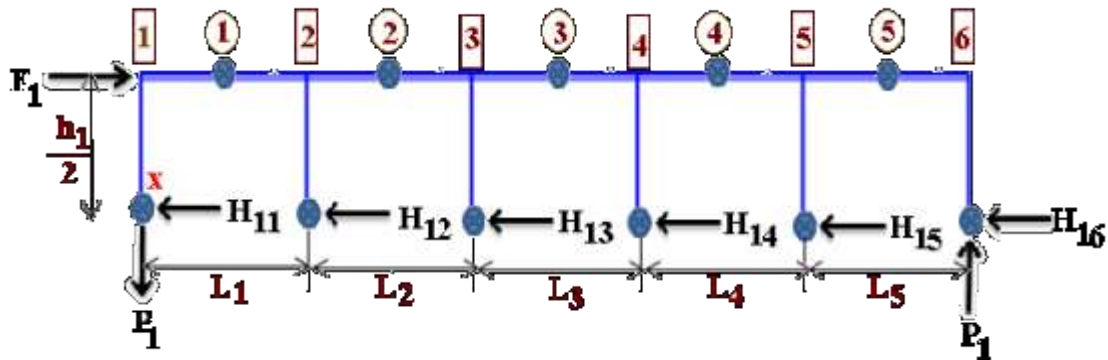


Fig (3.4) Top story

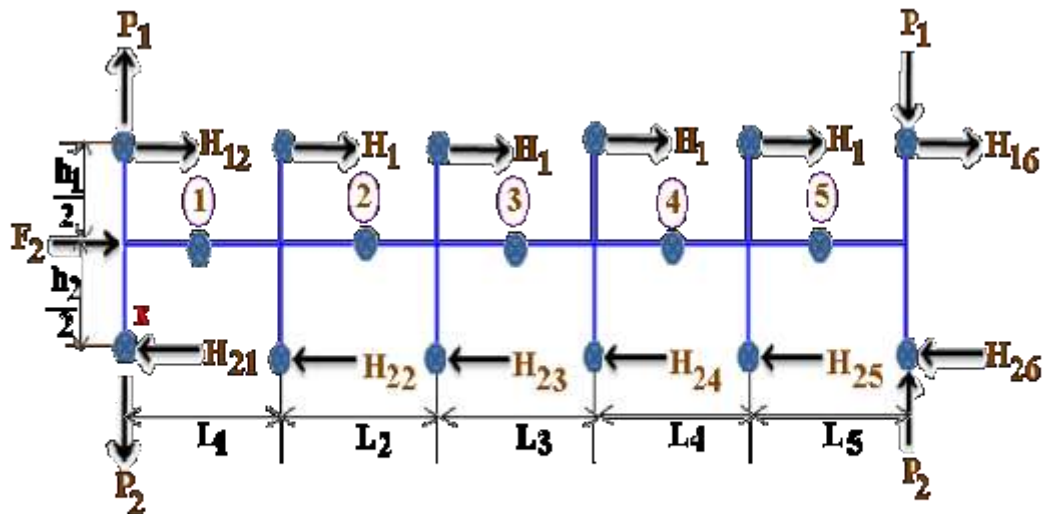


Fig (3.5) Under Top Story

3.4.1.1 Axial forces in columns and shear forces in beams:

*Top story

$$\sum M_x = 0$$

$$F_1 * \frac{h_1}{2} = P_1 [L_1 + L_2 + L_3 + L_4 + L_5] \dots \dots \dots (3.3)$$

Where:

$$[L_1 + L_2 + L_3 + L_4 + L_5] = L \dots \dots \dots (3.4)$$

sub (3.4) in (3.3)

$$\Rightarrow P_1 = \frac{F_1 h_1}{2L} \quad \Rightarrow V_1 = P_1$$

*Under top story

$$\sum M_x = 0$$

$$F_1 \left[h_1 + \frac{h_2}{2} \right] + F_2 \left[\frac{h_2}{2} \right] = P_2 \Rightarrow P_2 = \frac{F_1 [2h_1 + h_2] + F_2 h_2}{2L} \Rightarrow V_2 = P_2 - P_1$$

*In general

$$P_n = K_{n1}F_1 + K_{n2}F_2 + K_{n3}F_3 + \dots + K_{nn}F_n \dots \dots \dots (3.5)$$

$$P_1 = K_{11}F_1 \quad P_2 = K_{21}F_1 + K_{22}F_2 \quad P_3 = K_{31}F_1 + K_{32}F_2 + K_{33}F_3$$

$$P_4 = K_{41}F_1 + K_{42}F_2 + K_{43}F_3 + K_{44}F_4$$

Where:

$$K_{21} = \frac{[2h_1 + h_2]}{2L} \quad K_{22} = \frac{h_2}{2L} \quad K_{31} = \frac{[2h_1 + 2h_2 + h_3]}{2L} \quad K_{32} = \frac{[2h_2 + h_3]}{2L} \quad K_{33} = \frac{h_3}{2L}$$

Shear forces:

$$V_n = P_n - P_{n-1} \dots \dots \dots (3.6)$$

3.4.1.2 Shear forces in columns:

*Top story

$$\Sigma M_1 = 0$$

$$H_{11} * \frac{h_1}{2} = \frac{P_1 L_1}{2} \Rightarrow H_{11} = \frac{P_1 L_1}{h_1} \dots \dots \dots (3.7)$$

$$\Sigma M_2 = 0$$

$$(H_{11} + H_{12}) \frac{h_1}{2} = P_1 \left[L_1 + \frac{L_2}{2} \right] \frac{H_{12} h_1}{2} = \frac{P_1 L_1}{2} + \frac{P_1 L_2}{2}$$

$$\Rightarrow H_{12} = \frac{P_1 [L_1 + L_2]}{h_1} \dots \dots \dots (3.8)$$

Similarly:

$$H_{13} = \frac{V_1 [L_2 + L_3]}{h_1} \quad H_{14} = \frac{V_1 [L_3 + L_4]}{h_1} \quad H_{15} = \frac{V_1 [L_4 + L_5]}{h_1} \quad H_{16} = \frac{V_1 L_5}{h_1}$$

*Under top story

$$\Sigma M_1 = 0$$

$$H_{21} * \frac{h_2}{2} + H_{11} * \frac{h_1}{2} = (P_2 - P_1) \frac{L_1}{2}$$

$$\therefore H_{21} = \frac{[V_2 - V_1] L_1}{h_2} \dots \dots \dots (3.9)$$

$$\Sigma M_2 = 0$$

$$(P_2 - P_1) * \left[L_1 + \frac{L_2}{2} \right] - (H_{11} + H_{21}) * \frac{h_1}{2} = (H_{21} + H_{22}) * \frac{h_2}{2}$$

$$\Rightarrow H_{22} = \frac{[V_2 - V_1][L_1 + L_2]}{h_2} \dots\dots\dots (3.10)$$

Similarly:

$$H_{23} = \frac{[V_2 - V_1][L_2 + L_3]}{h_2} \qquad H_{24} = \frac{[V_2 - V_1][L_3 + L_4]}{h_2}$$

$$H_{25} = \frac{[V_2 - V_1][L_4 + L_5]}{h_2} \qquad H_{26} = \frac{[V_2 - V_1]L_5}{h_2}$$

*In general:

$$H_{ni} = \frac{[V_n - V_{n-1} + V_{n-2} - V_{n-3} + \dots - V_1][L_i + L_{i-1}]}{h_n} \dots\dots\dots (3.11)$$

3.4.1.3 Bending Moments in columns:

*Top story

$$\begin{array}{lll} M_{11} = \frac{H_{11}h_1}{2} & M_{12} = \frac{H_{12}h_1}{2} & M_{13} = \frac{H_{13}h_1}{2} \\ M_{14} = \frac{H_{14}h_1}{2} & M_{15} = \frac{H_{15}h_1}{2} & M_{16} = \frac{H_{16}h_1}{2} \end{array}$$

*Under Top story

$$\begin{array}{lll} M_{21} = \frac{H_{21}h_2}{2} & M_{22} = \frac{H_{22}h_2}{2} & M_{23} = \frac{H_{23}h_2}{2} \\ M_{24} = \frac{H_{24}h_2}{2} & M_{25} = \frac{H_{25}h_2}{2} & M_{26} = \frac{H_{26}h_2}{2} \end{array}$$

*In general:

$$M_{ni} = \frac{H_{ni}h_n}{2} \dots\dots\dots (3.12)$$

3.4.1.4 Bending Moments in beams:

Top story:

$$M_{11} = \frac{P_1 L_1}{2} \quad M_{12} = \frac{P_1 L_2}{2} \quad M_{13} = \frac{P_1 L_3}{2} \quad M_{14} = \frac{P_1 L_4}{2} \quad M_{15} = \frac{P_1 L_5}{2}$$

Under top story

$$M_{21} = \frac{P_2 L_1}{2} \quad M_{22} = \frac{P_2 L_2}{2} \quad M_{23} = \frac{P_2 L_3}{2} \quad M_{24} = \frac{P_2 L_4}{2} \quad M_{25} = \frac{P_2 L_5}{2}$$

*In general

$$M_{nj} = \frac{P_n L_j}{2} \dots \dots \dots (3.13)$$

Where:

F= Lateral Forces L= Span Length h= Height of story M= moments

P= Axial forces in columns H= shear forces in columns V= Shear forces in beams

i=Number of columns j=Number of beams n=Number of stories

3.4.2 Components of program:

3.4.2.1 Input data file:

In this file input the dimensions of the frame to be analyzed (floor height, lateral force, spans length)

3.4.2.2 Output data file:

In this file which is to show all of the values of shear forces and moments of the frame.

3.4.2.2.1 Shear forces calculation file:

This is file show all of the value of shear forces in beams and columns in the frame.

3.4.2.2.2 Bending Moments calculation file:

This is file show all of the value of moments in beams and columns in the frame.

3.4.3 Verification of program:

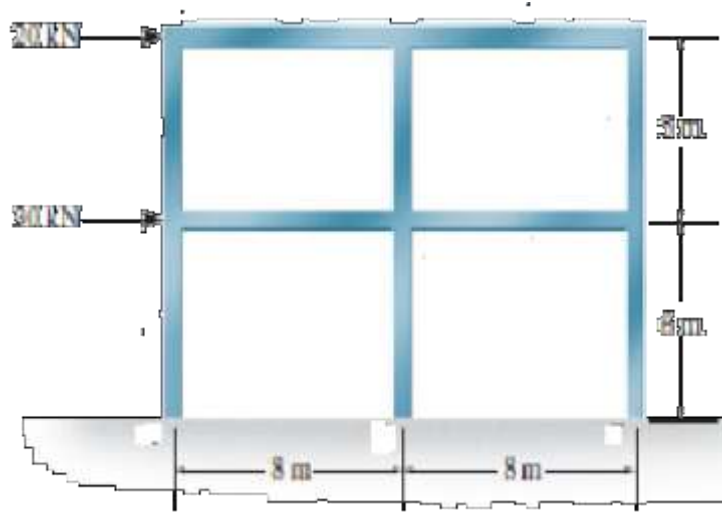


Fig. (3.6) Frame with lateral loads

Results of Axial Forces

Table (3.1) Results of Axial Forces.

Storey No.	Manual calculation (kN)	Program calculation (kN)	Variation%
Top	3.125	3.125	0
Ground	15.625	15.625	0

Results of bending moments

Table (3.2) Results of bending moments.

	manual calculation			Program calculation			Variation%	
Storey No.	Columns kN.m		Beams kN.m	Columns kN.m		Beams kN.m	Columns kN.m	Beams kN.m
	External	Internal	Internal	External	Internal	Internal	EX, In	Internal
Top	12.5	25	12.5	12.5	25	12.5	0	0
Ground	37.5	75	50	37.5	75	50	0	0

Results of Shear Forces

Table (3.3) Result of Shear Forces.

	manual calculation			Program calculation			Variation%	
Storey No.	Column (kN)		Beam (kN)	Column (kN)		Beam (kN)	Columns (kN)	Beams (kN)
	External	Internal	Internal	External	Internal	Internal	Ex, In	Internal
Top	5	10	3.125	5	10	3.125	0	0
Ground	12.5	25	12.5	12.5	25	12.5	0	0

ANALYSIS OF RIGID FRAME USING PORTAL METHOD (PMP)

sudan UNIVERSITY
M.SC STRUCTURE

PRESENTED BY:

rabaa mohammed mohammed ahmed

SUPERVISOR: dr. osama ahmed dawood

INPUT:

Numbre of story	Height of Story h (m)	lateral load (KN)	Span(L) Length(m)
2	5	20	8
1	6	30	8

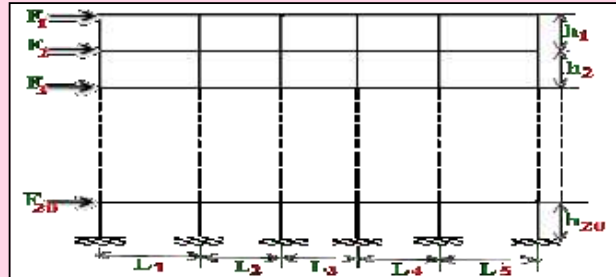


Fig. (3.7) Input in verification Example

OUTPUT:

SHEAR FORCES

Story	P (KN)	SF (KN)	Shear in each storey						total length
			H1	H2	H3	H4	H5	H6	
2	3.125	3.125	5	10	5	0	0	0	16
1	15.625	12.5	12.5	25	12.5	0	0	0	

Fig. (3.8) Output of Shear forces (KN) for verification Example

MOMENTS KN.m

Beams					
Story	M1	M2	M3	M4	M5
2	12.5	12.5	0	0	0
1	50	50	0	0	0

Columns				
M1	M2	M3	M4	M5
12.5	25	12.5	0	0
37.5	75	37.5	0	0

Fig. (3.9) Output for moments (KN.m) for verification Example.

3. 5 Developed Program for Cantilever method:

Developed cantilever program by used the assumption shown in section (3.3.2) and drive some equation to analyzed shown in section (3.5.1) .

3.5.1 Equations for developed program:

The derivation of general equations shown below from Figs (3.10) (3.11)

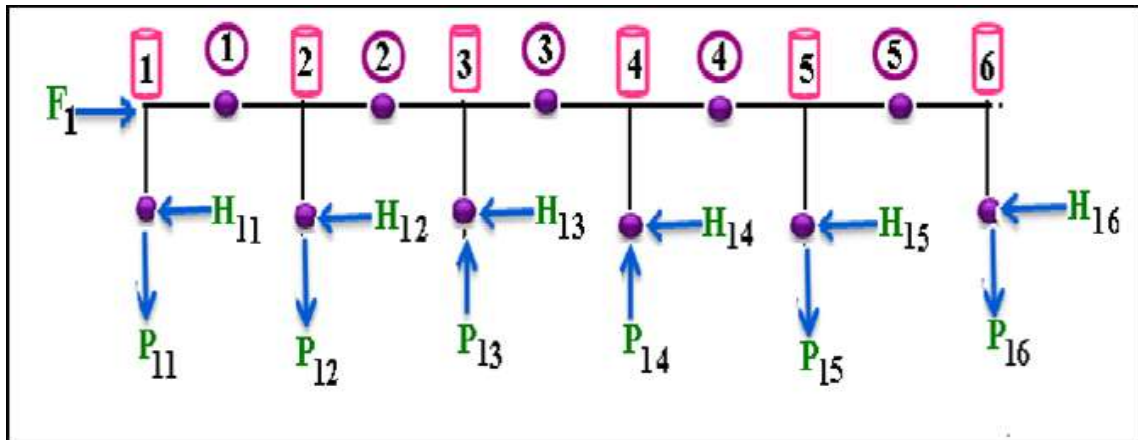


Fig (3.10) Top Story

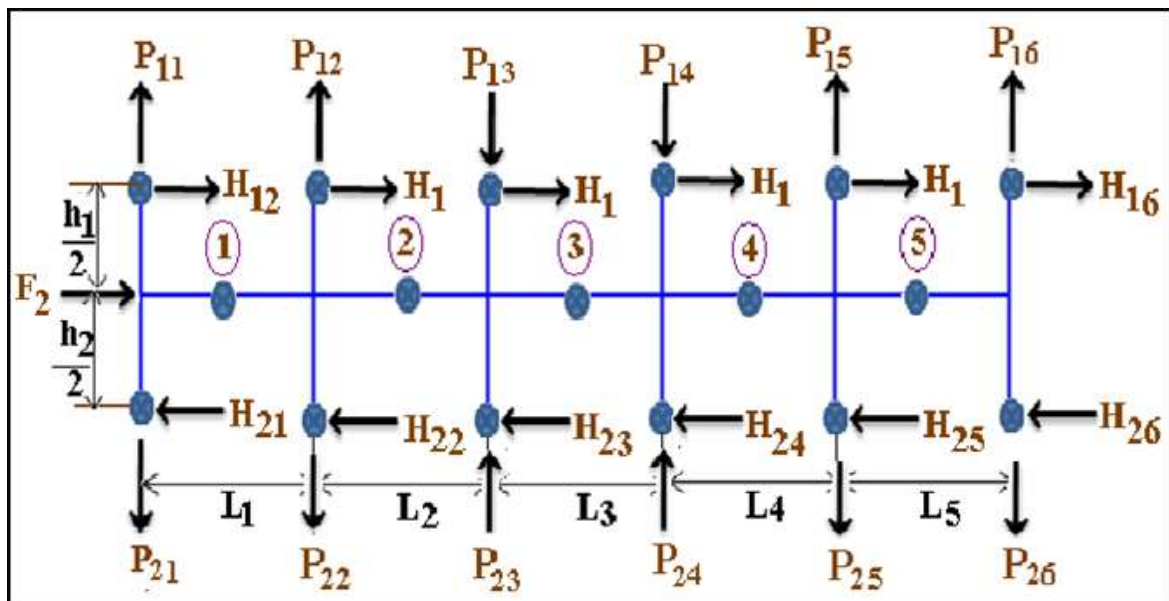


Fig. (3.11) Under Top Story

3.5.1.1 Axial forces in columns and shear forces in beams:

Top Story:

$$\ddot{X} = \sum \frac{A(0) + A(L_1) + \dots + A(L_1 + L_2 + L_3 + L_4 + \dots L_n)}{\Sigma A} \dots \dots \dots (3.14)$$

$$\begin{aligned} \beta_1 &= \frac{\ddot{X} - (0)}{\ddot{X}} & \beta_2 &= \frac{\ddot{X} - (L_1)}{\ddot{X}} & \beta_3 &= \frac{\ddot{X} - (L_1 + L_2)}{\ddot{X}} \\ \beta_4 &= \frac{\ddot{X} - (L_1 + L_2 + L_3)}{\ddot{X}} & \beta_5 &= \frac{\ddot{X} - (L_1 + L_2 + L_3 + L_4)}{\ddot{X}} \\ \beta_6 &= \frac{\ddot{X} - (L_1 + L_2 + L_3 + L_4 + L_5)}{\ddot{X}} \\ \beta_n &= \frac{\ddot{X} - (L_1 + L_2 + L_3 + L_4 + \dots \dots + L_n)}{\ddot{X}} \dots \dots \dots (3.15) \end{aligned}$$

Assume:

$$S_1 = L_1 \quad S_2 = L_1 + L_2 \quad S_3 = L_1 + L_2 + L_3 \quad S_4 = L_1 + L_2 + L_3 + L_4$$

$$S_5 = L_1 + L_2 + L_3 + L_4 + L_5$$

$$S_n = L_1 + L_2 + L_3 + L_4 + L_5 + \dots + L_n$$

Where

$$\begin{aligned} \beta_3 &= 0, \beta S = \beta_1 * S_1 \\ \beta_4 &= 0, \beta S = \beta_1 * S_2 + \beta_2 * L_2 \dots \dots \dots (3.16) \\ \beta_5 &= 0, \beta S = \beta_1 * S_3 + \beta_2 * (L_2 + L_3) + \beta_3 * L_3 \\ \beta_6 &= 0, \beta S = \beta_1 * S_4 + \beta_2 * (L_2 + L_3 + L_4) + \beta_3 * (L_3 + L_4) + \beta_4 * L_4 \\ \beta_7 &= 0, \beta S = \beta_1 S_5 + \beta_5 (L_2 + L_3 + L_4 + L_5) + \beta_3 (L_3 + L_4 + L_5) + \beta_4 (L_4 + L_5) + \beta_5 * L_5 \end{aligned}$$

$$\beta_n = 0 \quad \beta S = \beta_1 S_{n-2} + \beta_2 (L_2 + L_3 + \dots + L_{n-2}) + \beta_3 (L_3 + \dots + L_{n-2}) + \dots + \beta_{n-2} L_{n-2}$$

$$\sum M_x = 0$$

$$F_1 * \frac{h_1}{2} = P_1 * \beta S \quad \therefore P_1 = \frac{\left[\frac{F_1 h_1}{2} \right]}{\beta S} \dots \dots \dots (3.17)$$

Axial force in column

$$\begin{aligned} P_{11} &= P_1 * \beta_1 & P_{12} &= P_1 * \beta_2 & P_{13} &= P_1 * \beta_3 & P_{14} &= P_1 * \beta_4 \\ P_{15} &= P_1 * \beta_5 & P_{16} &= P_1 * \beta_6 \dots \dots \dots \end{aligned} \quad (3.18)$$

Shear forces in beams

$$\begin{aligned} \therefore V_{11} &= P_{11} & V_{12} &= P_{11} + P_{12} & V_{13} &= P_{11} + P_{12} + P_{13} \\ V_{14} &= P_{11} + P_{12} + P_{13} + P_{14} & V_{15} &= P_{11} + P_{12} + P_{13} + P_{14} + P_{15} \dots \dots \end{aligned} \quad (3.19)$$

***Under top story**

Axial force in column:

$$\sum M_x = 0$$

$$F_1 \left[h_1 + \frac{h_2}{2} \right] + F_2 \left[\frac{h_2}{2} \right] = P_2 * \beta S \quad \Rightarrow P_2 = \frac{\left[\frac{F_1 [2h_1 + h_2] + F_2 h_2}{2} \right]}{\beta S} \dots \dots \dots (3.20)$$

$$\begin{aligned} P_{21} &= P_2 * \beta_1 & P_{22} &= P_2 * \beta_2 & P_{23} &= P_2 * \beta_3 & P_{24} &= P_2 * \beta_4 \\ P_{25} &= P_2 * \beta_5 & P_{26} &= P_2 * \beta_6 \dots \dots \dots \end{aligned} \quad (3.21)$$

Shear forces in beams

$$\therefore V_{21} = P_{21} - P_{11} \quad V_{22} = (P_{21} + P_{22}) - (P_{11} + P_{12})$$

$$V_{23} = (P_{21} + P_{22} + P_{23}) - (P_{11} + P_{12} + P_{13})$$

$$V_{24} = (P_{21} + P_{22} + P_{23} + P_{24}) - (P_{11} + P_{12} + P_{13} + P_{14})$$

$$V_{25} = (P_{21} + P_{22} + P_{23} + P_{24} + P_{25}) - (P_{11} + P_{12} + P_{13} + P_{14} + P_{15}) \dots\dots\dots (3.22)$$

*In general

Axial force in column

$$P_n = K_{n1}F_1 + K_{n2}F_2 + K_{n3}F_3 + \dots + K_{nn}F_n \dots\dots\dots(3.23)$$

$$P_1 = K_{11}F_1 \quad P_2 = K_{21}F_1 + K_{22}F_2 \quad P_3 = K_{31}F_1 + K_{32}F_2 + K_{33}F_3$$

Where

$$K_{11} = \frac{\left[\frac{h_1}{2}\right]}{\beta S} \quad K_{21} = \left[\frac{[2h_1 + h_2]}{2}\right] / \beta S \quad , K_{22} = \left[\frac{h_2}{2}\right] / \beta S$$

$$K_{31} = \frac{\left[\frac{[2h_1 + 2h_2 + h_3]}{2}\right]}{\beta S} \quad , K_{32} = \frac{\left[\frac{[2h_2 + h_3]}{2}\right]}{\beta S} \quad , K_{33} = \left[\frac{h_3}{2}\right] / \beta S$$

$$K_{n1} = \frac{\left[\frac{[2h_1 + 2h_2 + 2h_3 + \dots + 2h_{n-1} + h_n]}{2}\right]}{\beta S} \quad , K_{n2} = \frac{\left[\frac{[2h_2 + 2h_3 + \dots + 2h_{n-1} + h_n]}{2}\right]}{\beta S} \quad , K_{nn} = \frac{\left[\frac{h_n}{2}\right]}{\beta S}$$

$$P_{n1} = P_n * \beta_1 \quad P_{n2} = P_n * \beta_2 \quad P_{n3} = P_n * \beta_3$$

$$P_{n4} = P_n * \beta_4 \quad P_{n5} = P_n * \beta_5 \quad P_{n6} = P_n * \beta_6$$

$$P_{ni} = P_n * \beta_i \quad \dots\dots\dots (3.24)$$

Shear Forces:

$$V_{ni} = (P_{n1} + P_{n2} + \dots + P_{ni}) - (P_{n-1\ 1} + P_{n-1\ 2} + \dots + P_{n-1\ i}) \dots\dots\dots (3.25)$$

3.5.1.2 Shear forces in columns

*Top story

$$\Sigma M_1 = 0$$

$$H_{11} * \frac{h_1}{2} = \frac{P_{11}L_1}{2} \Rightarrow H_{11} = \frac{P_{11}L_1}{h_1} \dots \dots \dots (3.26)$$

$$\Sigma M_2 = 0$$

$$\gg (H_{11} + H_{12}) \frac{h_1}{2} = (P_{11} \left[L_1 + \frac{L_2}{2} \right] + [P_{12} \frac{L_2}{2}])$$

$$\therefore H_{12} = \frac{(P_{11}[L_1 + L_2] + [P_{12} * L_2])}{h_1} \dots \dots \dots (3.27)$$

$$\Sigma M_3 = 0$$

$$(H_{11} + H_{12} + H_{13}) \frac{h_1}{2} = (P_{11} \left[L_1 + L_2 + \frac{L_3}{2} \right] + P_{12} \left[L_2 + \frac{L_3}{2} \right] + [P_{13} \frac{L_3}{2}])$$

$$\therefore H_{13} = \frac{(P_{11}[L_2 + L_3] + P_{12}[L_2 + L_3] + [P_{13}L_3])}{h_1} \dots \dots \dots (3.28)$$

Similarly

$$\therefore H_{14} = \frac{(P_{11}[L_3 + L_4] + P_{12}[L_3 + L_4] + P_{13}[L_3 + L_4] + [P_{14}L_4])}{h_1}$$

$$\therefore H_{15} = \frac{(P_{11}[L_4 + L_5] + P_{12}[L_4 + L_5] - P_{13}[L_4 + L_5] - P_{14}[L_4 + L_5] + [P_{15}L_5])}{h_1}$$

$$\therefore H_{16} = \frac{P_{16}L_5}{h_1}$$

***Under top story**

$$\Sigma M_1 = 0$$

$$H_{21} * \frac{h_2}{2} + H_{11} * \frac{h_1}{2} = (P_{21} - P_{11}) \frac{L_1}{2}$$

$$\therefore H_{21} = \frac{[(P_{21} - P_{11})L_1 - P_{11}L_1]}{h_2} \dots\dots\dots (3.29)$$

$$\sum M_2 = 0$$

$$[H_{21} + H_{22}] * \frac{h_2}{2} + [H_{11} + H_{12}] * \frac{h_1}{2} = (P_{21} - P_{11}) \left(L_1 + \frac{L_2}{2} \right) + (P_{22} - P_{12}) \left(\frac{L_2}{2} \right)$$

$$\therefore H_{22} = \frac{[(P_{21} - P_{11})[L_1 + L_2] + [P_{22} - P_{12}]L_2 - P_{11}[L_1 + L_2] - [P_{12}L_2]}{h_2} \dots\dots (3.30)$$

Similarly

$$\therefore H_{23} = \frac{[(P_{21} - P_{11})[L_2 + L_3] + [P_{22} - P_{12}][L_2 + L_3] + [P_{23} - P_{13}]L_3 - P_{11}[L_2 + L_3] - P_{12}[L_2 + L_3] - [P_{13}L_3]}{h_2}$$

$$\therefore H_{24} = \frac{[(P_{21} - P_{11})[L_3 + L_4] + [P_{22} - P_{12}][L_3 + L_4] + [P_{23} - P_{13}][L_3 + L_4] + [P_{24} - P_{14}]L_4 - P_{11}[L_3 + L_4] - P_{12}[L_3 + L_4] - P_{13}[L_3 + L_4] - [P_{14}L_4]}{h_2}$$

$$\therefore H_{25} = \frac{[(P_{21} - P_{11})[L_4 + L_5] + [P_{22} - P_{12}][L_4 + L_5] + [P_{23} - P_{13}][L_4 + L_5] + [P_{24} - P_{14}][L_4 + L_5] + [P_{25} - P_{15}]L_5 - P_{11}[L_4 + L_5] - P_{12}[L_4 + L_5] - P_{13}[L_4 + L_5] - P_{14}[L_4 + L_5] - [P_{15}L_5]}{h_2}$$

$$\therefore H_{26} = \frac{[(P_{26} - P_{16})L_5 - P_{16}L_5]}{h_2}$$

*In general:

$$\therefore H_{ni} = \frac{\begin{bmatrix} (P_{n1} - P_{n-11})[2L_1 + 2L_2 + 2L_3 + 2L_4 + \dots + 2L_{i-1} + L_i] + \\ [P_{n2} - P_{n-12}][2L_2 + 2L_3 + 2L_4 + \dots + 2L_{i-1} + L_i] \\ + [P_{n3} - P_{n-13}][2L_3 + 2L_4 + \dots + 2L_{i-1} + L_i] + \dots \dots + \\ [P_{ni-1} - P_{n-1(i-1)}][2L_{i-1} + L_i] + [P_{ni} - P_{n-1i}]L_i \\ -(H_{n-11} + H_{n-12} + H_{n-13} + \dots \dots + H_{n-1(i-1)} + H_{n-1i})h_{n-1} \end{bmatrix}}{h_n} - (H_{n1} + H_{n2} + H_{n3} + H_{n4} + \dots \dots + H_{ni-1}) \dots \dots \dots (3.31)$$

3.5.1.3 Bending Moments in columns

***Top story**

$$M_{11} = \frac{H_{11}h_1}{2} \quad M_{12} = \frac{H_{12}h_1}{2} \quad M_{13} = \frac{H_{13}h_1}{2}$$

$$M_{14} = \frac{H_{14}h_1}{2} \quad M_{15} = \frac{H_{15}h_1}{2} \quad M_{16} = \frac{H_{16}h_1}{2}$$

***Under Top story**

$$M_{21} = \frac{H_{21}h_2}{2} \quad M_{22} = \frac{H_{22}h_2}{2} \quad M_{23} = \frac{H_{23}h_2}{2}$$

$$M_{24} = \frac{H_{24}h_2}{2} \quad M_{25} = \frac{H_{25}h_2}{2} \quad M_{26} = \frac{H_{26}h_2}{2}$$

***In general**

$$M_{ni} = \frac{H_{ni}h_n}{2} \dots \dots \dots (3.32)$$

3.5.1.4 Bending Moments in beams

Top story

$$M_{11} = \frac{V_{11}L_1}{2} \quad M_{12} = \frac{V_{12}L_2}{2} \quad M_{13} = \frac{V_{13}L_3}{2} \quad M_{14} = \frac{V_{14}L_4}{2} \quad M_{15} = \frac{V_{15}L_5}{2}$$

Under top story

$$M_{21} = \frac{V_{21}L_1}{2} \quad M_{22} = \frac{V_{22}L_2}{2} \quad M_{23} = \frac{V_{23}L_3}{2} \quad M_{24} = \frac{V_{24}L_4}{2} \quad M_{25} = \frac{V_{25}L_5}{2}$$

***In general**

$$M_{nj} = \frac{V_{nj}L_j}{2} \dots\dots\dots (3.33)$$

3.5.2 Components of program

3.5.2.1 Input data file

In this file input the dimensions of the frame to be analyzed (floor height, lateral force, spans length

3.5.2.2 Output data file

In this file which is to show all of the values of shear forces and moments of the frame.

3.5.2.2.1 Shear forces calculation file

This is file show all of the value of shear forces in beams and columns in the frame.

3.5.2.2.2 Bending Moments calculation file

This is file show all of the value of moments in beams and columns in the frame.

3.5.3 Verification of example:

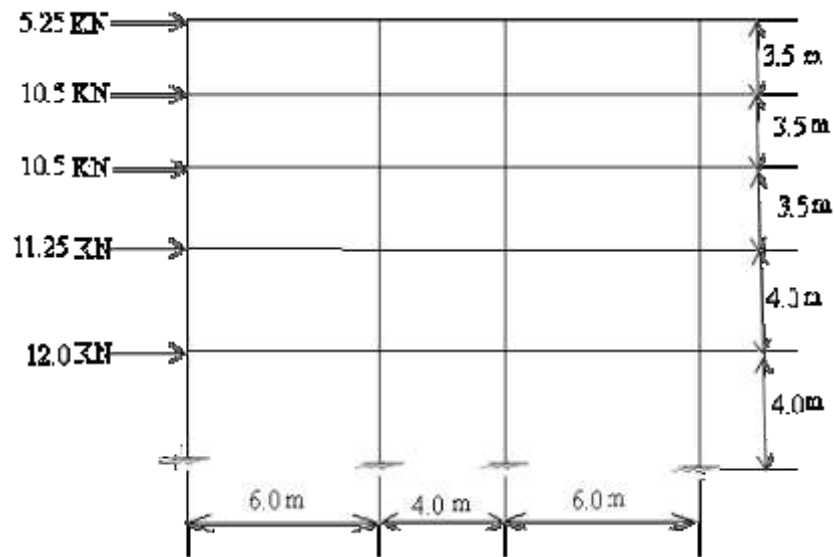


Fig. (3.12) Frame with lateral loads [Mosley, 1990]

Table (3.4) Result of Axial Forces in Columns.

storey No.	Manual calculation		Program calculation		Variation%	
	External (kN)	Internal (kN)	External (kN)	Internal (kN)	External (kN)	Internal (kN)
4	0.54	0.135	0.54	0.135	0	0
3	2.7	0.68	2.7	0.68	0	0
2	7.0	1.8	7.0	1.8	0	0
1	14.1	3.5	14.1	3.5	0	0
Ground	24.4	6.10	24.4	6.10	0	0

Table (3.5) Results of Shear forces in Columns.

	Manual calculation		Program calculation		Variation%	
storey No.	External (kN)	Internal (kN)	External (kN)	Internal (kN)	External (kN)	Internal (kN)
4	0.93	1.70	0.93	1.70	0	0
3	2.78	5.10	2.78	5.10	0	0
2	4.63	8.49	4.63	8.49	0	0
1	6.62	12.13	6.62	12.13	0	0
Ground	8.74	16.01	8.74	16.01	0	0

Table (3.6) Results of Shear Forces in Beams.

	Manual		CM		Variation%	
N. of story	External (kN)	Internal (kN)	External (kN)	Internal (kN)	External (kN)	Internal (kN)
4	0.54	0.675	0.54	0.675	0	0
3	2.16	2.70	2.16	2.70	0	0
2	4.32	5.40	4.32	5.40	0	0
1	7.11	8.89	7.11	8.89	0	0
Ground	10.24	12.79	10.24	12.79	0	0

Table (3.7) Results of Bending Moments in Columns.

	Manual calculation		Program calculation		Variation%	
storey No.	External (kN)	Internal (kN)	External (kN)	Internal (kN)	External (kN)	Internal (kN)
4	1.6	3.0	1.6	3.0	0	0
3	4.9	8.9	4.9	8.9	0	0
2	8.1	14.9	8.1	14.9	0	0
1	13.2	24.3	13.2	24.3	0	0
Ground	17.5	32.0	17.5	32.0	0	0

Table (3.6) Results of Bending Moments in Beams.

	Manual		CM		Variation%	
N. of story	External (kN)	Internal (kN)	External (kN)	Internal (kN)	External (kN)	Internal (kN)
4	1.6	1.4	1.6	1.4	0	0
3	6.5	5.4	6.5	5.4	0	0
2	13.0	10.8	13.0	10.8	0	0
1	21.3	17.8	21.3	17.8	0	0
Ground	30.7	25.6	30.7	25.6	0	0

ANALYSIS OF RIGID FRAME USING CANTILEVER METHOD (CMF)

Sudan UNIVERSITY
M.SC STRUCTURE
PRESENTED BY:
Rabaa Mohammed

SUPERVISOR: Dr. Osama Dawoud

INPUT:

Numbre of storey	Height of Story h (m)	lateral(Fy load (kN)	Span(L) Length(ft)
4	3.5	5.25	6
3	3.5	10.5	4
2	3.5	10.5	6
1	4	11.25	
Graound	4	12	

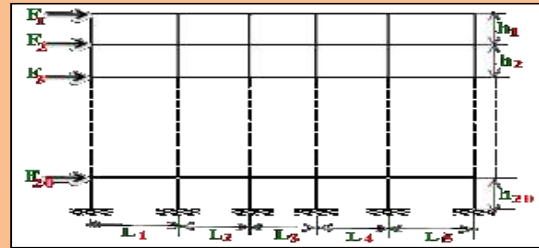


Fig. (3.13) Input in Verification Example

OUTPUT

Axial forces in columns

Number o Storey	1	2	3	4	5	6
4	0.540441	0.13511	-0.13511	-0.54044	0	0
3	2.702206	0.675551	-0.67555	-2.70221	0	0
2	7.025735	1.756434	-1.75643	-7.02574	0	0
1	14.13971	3.534926	-3.53493	-14.1397	0	0
Graound	24.375	6.09375	-6.09375	-24.375	0	0

Fig. (3.14) Axial Forces (kN) in Columns for Verification Example

Shear forces in beams

Number of Storey	1	2	3	4	5
4	0.540441	0.675551	0.540441	0	0
3	2.161765	2.702206	2.161765	0	0
2	4.323529	5.404412	4.323529	0	0
1	7.113971	8.892463	7.113971	0	0
Graound	10.23529	12.79412	10.23529	0	0

Fig. (3.15) Shear Forces (kN) in Beams for Verification Example

Shear forces in columns

Number of Storey	1	2	3	4	5	6
4	0.926471	1.698529	1.698529	0.926471	0	0
3	2.779412	5.095588	5.095588	2.779412	0	0
2	4.632353	8.492647	8.492647	4.632353	0	0
1	6.617647	12.13235	12.13235	6.617647	0	0
Graound	8.735294	16.01471	16.01471	8.735294	0	0

Fig. (3.16) Shear Forces (kN) in Columns for Verification Example

Moments in columns

Number of Storey	1	2	3	4	5	6
4	1.621324	2.972426	2.972426	1.621324	0	0
3	4.863971	8.917279	8.917279	4.863971	0	0
2	8.106618	14.86213	14.86213	8.106618	0	0
1	13.23529	24.26471	24.26471	13.23529	0	0
Graound	17.47059	32.02941	32.02941	17.47059	0	0

Moments in beams

1	2	3	4	5
1.621324	1.351103	1.621324	0	0
6.485294	5.404412	6.485294	0	0
12.97059	10.80882	12.97059	0	0
21.34191	17.78493	21.34191	0	0
30.70588	25.58824	30.70588	0	0

Fig. (3.17) Moments (kN.m) in Columns and Beams for Verification Example

Chapter four

Analysis and Discussion

Of Results

4.1 Introduction:

This chapter deals with the application of the different methods of structural analysis of tall buildings. The proposed building has been analyzed using simplified methods (portal and cantilever methods) and the finite element based on program (ETABS). Results from the three methods were analyzed, compared and discussed.

4.2 Description of Selected Problem:

The case study chosen is a building of a plan area of $25 \times 25m^2$, divided into 5 panels 5 meters each, both ways as shown in Fig. (4.1.a). The plan of 25 story high residential building, with Rigid frame system which will be investigated is shown in Fig. (4.1.b). The proposed building of 25 stories height and 3.2 floor height was modeled using ETABS program and the simplified methods. The models were subjected to wind loads.

Wind loads were calculated using principles outlined in the British Standard BS6399 the basic wind speed was taken as 45 m/s.

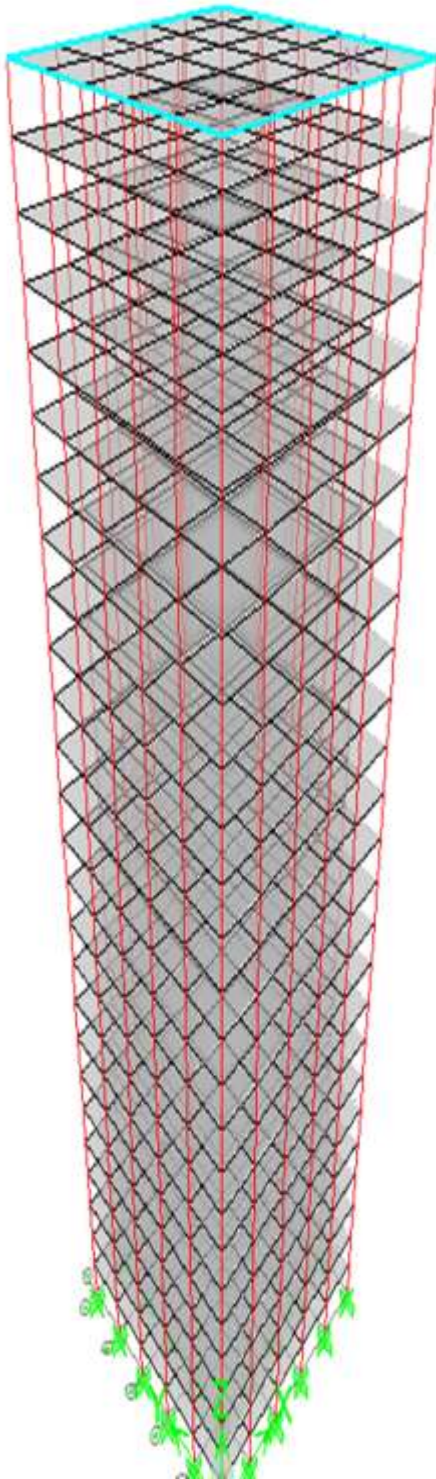


Fig. (4.1.b): 3D view of the building

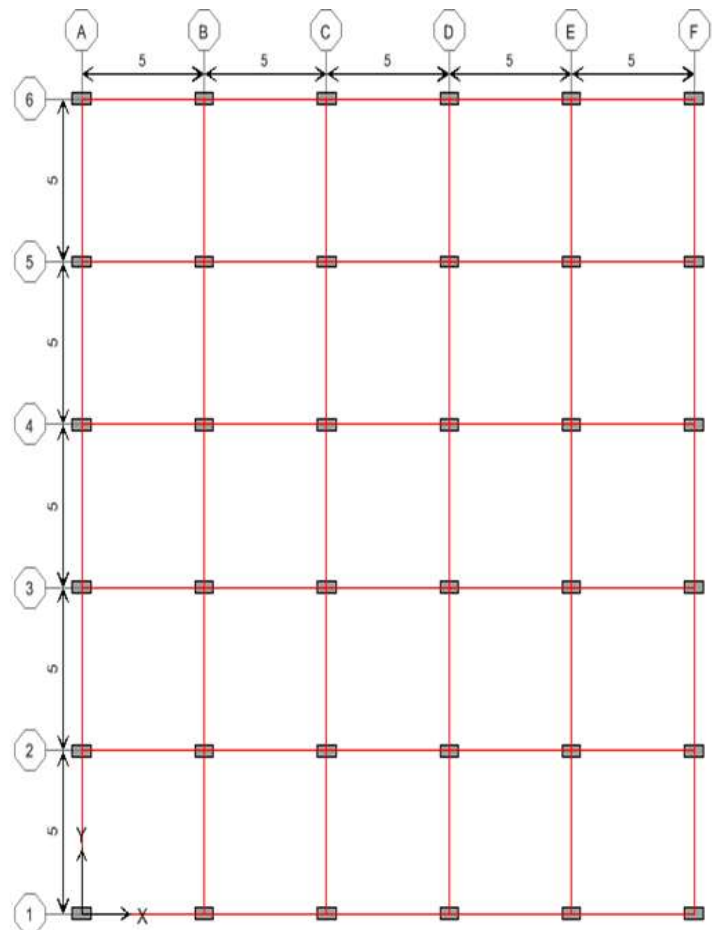


Fig. (4.1.a): Plan view of the building

4.3 Wind loads calculation:

Stage 1:

Determine the dynamic augmentation factor (Cr)

$H=83.2$ m

Building type factor (K_b) (A.1)

$Cr=$ (A.2)

Stage 2:-

Check Limits of applicability

$Cr<0.25$ & $H < 300m$ (OK)

Stage 3:

Determine the basic hourly mean wind speed (V_b) = 45m/s

Stage 4:-

Determine a site wind speed (V_s)

$$V_s = S_a \times S_d \times S_s \times S_p \dots\dots\dots (4. 1)$$

S_a ≡is an altitude factor =1

S_d ≡ is a direction factor =0.78 (A. 3)

S_s ≡ is a seasonal factor =1

S_p ≡ is a probability factor =1

Stage 5:

Terrain categories effective height

Reference height: $H_r=83.2$ m

The effective height $H_e = H_r$

Stage 6:

Choice method:

The standard wind load method.

Stage 7:-

Standard effective wind speeds (V_e):

$$V_e = V_s \times S_b \dots\dots\dots (4.2)$$

$S_b \equiv$ is the terrain and building factor obtained (A.4)

Stage 8:-

Dynamic pressure (q_s):

$$q_s = 0.613 V_e^2 (p_a) \dots\dots\dots (4.3)$$

Stage 9:-

Pressure coefficients (C_p):- Standard

9.1 External pressure coefficients C_{pe} wind word=0.85 (A.5)

9.2 External pressure coefficients C_{pe} lee word= -0.5 (A.5)

9.3 Internal pressure coefficients $C_{pi}=-0.3$ (A.6)

Stage 10:-

10.1 \ External surface pressures

$$P_e = q_s \times C_{pe} \times C_{ae} \dots\dots\dots (4.4)$$

$C_{ae} \equiv$ is the size effect factor for external pressures (A.7)

Dependent on the diagonal dimension (A.8)

10.2 internal surface pressures

$$P_i = q_s \times C_{pi} \times C_{ai} \dots\dots\dots (4.5)$$

$C_{ai} \equiv$ is the size effect factor for Internal pressures (A.9)

Dependent on the diagonal dimension a

Stage 11:-

11.1 Net surface pressures (P)

$$P = P_e - P_i \dots\dots\dots (4.6)$$

11.2 Surface loads

The net load P on an area of a building surface or element is given by:

$$P = p \times A \dots\dots\dots (4.7)$$

Where:

$P \equiv$ is the net pressure across the surface

$A \equiv$ is the loaded area

Table (4.1): Dynamic Pressure Calculation

<i>Floor No.</i>	V_b	S_a	S_d	S_s	S_p	$V_s = S_a * S_d$ $* S_s * S_p$	S_b	$V_e = V_s * S_b$	V_e^2	$q_s = 0.613$ V_e^2	$q_s/1000$
base	45	1	0.78	1	1	35.1	1.3	45.63	2082	1276	1.27
1st	45	1	0.78	1	1	35.1	1.5	52.65	2772	1699	1.69
2nd	45	1	0.78	1	1	35.1	1.6	56.16	3153	1933	1.93
3rd	45	1	0.78	1	1	35.1	1.6	56.16	3153	1933	1.93
4th	45	1	0.78	1	1	35.1	1.74	61.074	3730	2286	2.28
5th	45	1	0.78	1	1	35.1	1.7	59.67	3560	2182	2.18
6th	45	1	0.78	1	1	35.1	1.8	63.18	3991	2446	2.44
7th floor	45	1	0.78	1	1	35.1	1.8	63.18	3991	2446	2.44
8th floor	45	1	0.78	1	1	35.1	1.8	63.18	3991	2446	2.44
9th floor	45	1	0.78	1	1	35.1	1.8	63.18	3991	2446	2.44
10th floor	45	1	0.78	1	1	35.1	1.8	63.18	3991	2446	2.44
11th floor	45	1	0.78	1	1	35.1	1.9	66.69	4447	2726	2.72
12th floor	45	1	0.78	1	1	35.1	1.9	66.69	4447	2726	2.72
13th floor	45	1	0.78	1	1	35.1	1.9	66.69	4447	2726	2.72
14th floor	45	1	0.78	1	1	35.1	1.9	66.69	4447	2726	2.72
15th floor	45	1	0.78	1	1	35.1	1.9	66.69	4447	2726	2.72
16th floor	45	1	0.78	1	1	35.1	1.9	66.69	4447	2726	2.72
17th floor	45	1	0.78	1	1	35.1	1.9	66.69	4447	2726	2.72
18th floor	45	1	0.78	1	1	35.1	1.9	66.69	4447	2726	2.72
19th floor	45	1	0.78	1	1	35.1	1.9	66.69	4447	2726	2.72
20th floor	45	1	0.78	1	1	35.1	2	70.2	4928	3020	3.02
21st floor	45	1	0.78	1	1	35.1	2	70.2	4928	3020	3.02
22nd floor	45	1	0.78	1	1	35.1	2	70.2	4928	3020	3.02
23rd floor	45	1	0.78	1	1	35.1	2	70.2	4928	3020	3.02
24th floor	45	1	0.78	1	1	35.1	2	70.2	4928	3020	3.02
25th floor	45	1	0.78	1	1	35.1	2	70.2	4928	3020	3.02

Table (4.2): External & Internal Pressures Calculation

External surface pressures windward				internal pressure				External surface pressures leeward			
$q_s/1000$	C_{pe}	C_a	P_e	$q_s/1000$	C_{pi}	C_a	P_i	$q_s/1000$	C_{pe}	C_a	P_e
1.34	0.85	0.77	0.8	1.34	-0.3	1	-0.4	1.34	-0.5	1	-0.67
1.74	0.85	0.77	1.13	1.74	-0.3	1	-0.5	1.74	-0.5	1	-0.87
2	0.85	0.77	1.3	2	-0.3	1	-0.6	2.00	-0.5	1	-1
2.16	0.85	0.77	1.43	2.18	-0.3	1	-0.6	2.16	-0.5	1	-1.08
2.28	0.85	0.77	1.41	2.21	-0.3	1	-0.6	2.28	-0.5	1	-1.14
2.38	0.85	0.77	1.6	2.32	-0.3	1	-0.71	2.38	-0.5	1	-1.19
2.46	0.85	0.77	1.63	2.46	-0.3	1	-0.73	2.46	-0.5	1	-1.23
2.54	0.85	0.77	1.62	2.52	-0.3	1	-0.76	2.54	-0.5	1	-1.27
2.6	0.85	0.77	1.72	2.68	-0.3	1	-0.78	2.6	-0.5	1	-1.3
2.61	0.85	0.77	1.78	2.68	-0.3	1	-0.79	2.65	-0.5	1	-1.32
2.7	0.85	0.77	1.72	2.71	-0.3	1	-0.81	2.7	-0.5	1	-1.35
2.72	0.85	0.77	1.83	2.76	-0.3	1	-0.82	2.75	-0.5	1	-1.37
2.8	0.85	0.77	1.85	2.84	-0.3	1	-0.84	2.8	-0.5	1	-1.4
2.88	0.85	0.77	1.82	2.84	-0.3	1	-0.85	2.85	-0.5	1	-1.42
2.85	0.85	0.77	1.83	2.82	-0.3	1	-0.86	2.88	-0.5	1	-1.44
2.92	0.85	0.77	1.7	2.99	-0.3	1	-0.87	2.92	-0.5	1	-1.46
2.96	0.85	0.77	1.8	2.99	-0.3	1	-0.88	2.93	-0.5	1	-1.46
2.92	0.85	0.77	1.95	2.93	-0.3	1	-0.88	2.96	-0.5	1	-1.48
2.94	0.85	0.77	1.92	2.93	-0.3	1	-0.89	2.98	-0.5	1	-1.49
3.04	0.85	0.77	1.99	3.05	-0.3	1	-0.9	3.01	-0.5	1	-1.5
3.04	0.85	0.77	1.96	3.03	-0.3	1	-0.91	3.04	-0.5	1	-1.52
3.03	0.85	0.77	2.09	3.09	-0.3	1	-0.91	3.06	-0.5	1	-1.53
3.09	0.85	0.77	2.08	3.08	-0.3	1	-0.92	3.09	-0.5	1	-1.54
3.12	0.85	0.77	2.04	3.13	-0.3	1	-0.93	3.11	-0.5	1	-1.55
3.16	0.85	0.77	2.09	3.12	-0.3	1	-0.94	3.14	-0.5	1	-1.57
3.11	0.85	0.77	2.05	3.11	-0.3	1	-0.95	3.16	-0.5	1	-1.58

Table (4.3): Forces in Floors

FLOOR NO.	$P = P_e - P_i$ (KN/m ²)	$F_x = p * 3.2 * 5$ (kN)	$F_x = p * 3.2 * 2.5$ (kN)
base	1.15	18.40	9.20
1st floor	1.48	23.79	11.89
2nd floor	1.71	27.39	13.69
3rd floor	1.84	29.56	14.78
4th floor	1.95	31.26	15.63
5th floor	2.04	32.64	16.32
6th floor	2.10	33.67	16.83
7th floor	2.17	34.73	17.36
8th floor	2.22	35.62	17.81
9th floor	2.26	36.30	18.15
10th floor	2.31	36.96	18.48
11th floor	2.35	37.66	18.83
12th floor	2.39	38.36	19.18
13th floor	2.44	39.06	19.53
14th floor	2.46	39.50	19.75
15th floor	2.49	39.92	19.96
16th floor	2.51	40.18	20.09
17th floor	2.53	40.52	20.26
18th floor	2.55	40.87	20.43
19th floor	2.57	41.21	20.60
20th floor	2.59	41.56	20.78
21st floor	2.61	41.88	20.94
22nd floor	2.64	42.25	21.12
23rd floor	2.66	42.61	21.30
24th floor	2.68	42.97	21.48
25th floor	2.70	21.65	10.82

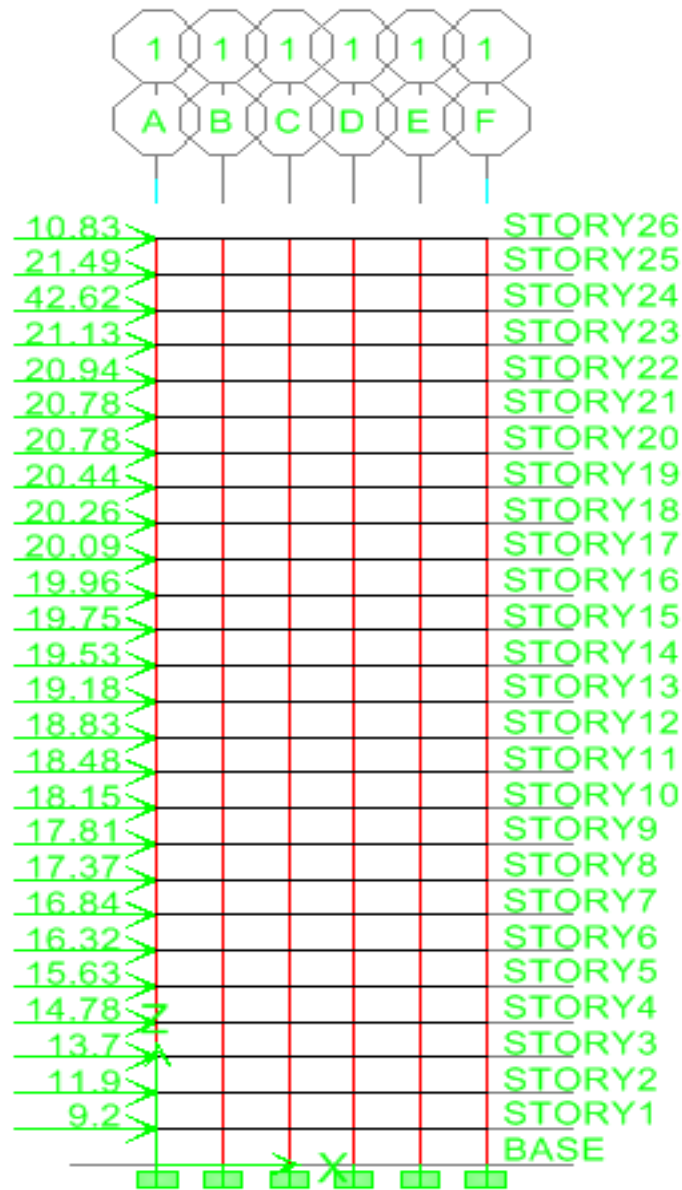


Fig. (4.2) Tall building subjected to lateral loads.

4.4 Analysis of the model

The building was divided in to five different levels, each level is higher than the lower level with five floors. Analysis of the building was carried out for each different heights starting from the lowest height level ten. Result of the analysis was considered at a selected constant point in the building. At the selected point, axial forces, shear forces and bending moments were calculated and plotted.

4.5 Results of analysis for 25 story using simplified methods

4.5.1 Portal Method Results:

ANALYSIS OF RIGID FRAME USING PORTAL METHOD (PMP)		SUDAN UNIVERSITY M.SC STRUCTURE PRESENTED BY: Rabaa Mohammed Mohammed Ahmed
SUPERVISOR:	Dr. Osama Mohammed Ahmed Dawoud	

INPUT:

Numbre of story	Height of Story h (m)	lateral(Fx load (KN)	Span(L) Length(m)
25	3.2	10.82696	5
24	3.2	21.48509	5
23	3.2	21.30851	5
22	3.2	21.12849	5
21	3.2	20.94092	5
20	3.2	20.78317	
19	3.2	20.60951	
18	3.2	20.43658	
17	3.2	20.26438	
16	3.2	20.09291	
15	3.2	19.96276	
14	3.2	19.75215	
13	3.2	19.53062	
12	3.2	19.18075	
11	3.2	18.83404	
10	3.2	18.48073	
9	3.2	18.15011	
8	3.2	17.81289	
7	3.2	17.36503	
6	3.2	16.83884	
5	3.2	16.32076	
4	3.2	15.63059	
3	3.2	14.7801	
2	3.2	13.69995	
1	3.2	11.89653	
GROUND	3.2	9.20109	

Fig. (4.3) Input of lateral force in the model

OUTPUT:

SHEAR FORCES

Story	P (KN)	SF (KN)	Shear in each storey H (KN)						total length
			H1	H2	H3	H4	H5	H6	
25	0.692926	0.692926	1.082696	2.165393	2.165393	2.165393	2.165393	1.082696	25
24	3.453822	2.760897	3.231205	6.46241	6.46241	6.46241	6.46241	3.231205	
23	8.95351	5.499687	5.362056	10.72411	10.72411	10.72411	10.72411	5.362056	
22	17.16917	8.215656	7.474906	14.94981	14.94981	14.94981	14.94981	7.474906	
21	28.07726	10.9081	9.568997	19.13799	19.13799	19.13799	19.13799	9.568997	
20	41.6557	13.57844	11.64731	23.29463	23.29463	23.29463	23.29463	11.64731	
19	57.88327	16.22757	13.70827	27.41653	27.41653	27.41653	27.41653	13.70827	
18	76.7378	18.85452	15.75192	31.50385	31.50385	31.50385	31.50385	15.75192	
17	98.19718	21.45938	17.77836	35.55673	35.55673	35.55673	35.55673	17.77836	
16	122.2394	24.04225	19.78765	39.57531	39.57531	39.57531	39.57531	19.78765	
15	148.8452	26.60581	21.78393	43.56786	43.56786	43.56786	43.56786	21.78393	
14	177.9928	29.14757	23.75914	47.51829	47.51829	47.51829	47.51829	23.75914	
13	209.6545	31.66166	25.71221	51.42441	51.42441	51.42441	51.42441	25.71221	
12	243.7937	34.13919	27.63028	55.26056	55.26056	55.26056	55.26056	27.63028	
11	280.3658	36.57214	29.51369	59.02737	59.02737	59.02737	59.02737	29.51369	
10	319.3261	38.96028	31.36176	62.72352	62.72352	62.72352	62.72352	31.36176	
9	360.6307	41.30466	33.17677	66.35354	66.35354	66.35354	66.35354	33.17677	
8	404.237	43.60629	34.95806	69.91612	69.91612	69.91612	69.91612	34.95806	
7	450.0947	45.85768	36.69456	73.38912	73.38912	73.38912	73.38912	36.69456	
6	498.1414	48.04672	38.37844	76.75689	76.75689	76.75689	76.75689	38.37844	
5	548.3104	50.16894	40.01052	80.02104	80.02104	80.02104	80.02104	40.01052	
4	600.5242	52.21382	41.57358	83.14716	83.14716	83.14716	83.14716	41.57358	
3	654.6843	54.16011	43.05159	86.10318	86.10318	86.10318	86.10318	43.05159	
2	710.6671	55.98283	44.42158	88.84317	88.84317	88.84317	88.84317	44.42158	
1	768.2881	57.62101	45.61124	91.22247	91.22247	91.22247	91.22247	45.61124	
GROUND	827.2594	58.97125	46.53135	93.06269	93.06269	93.06269	93.06269	46.53135	

Fig. (4.4) Results of shear forces (kN) for 25-storey using portal frame method.

MOMENTS KN.m

Beams

Story	M1	M2	M3	M4	M5
25	1.73231	1.73231	1.73231	1.73231	1.73231
24	6.90224	6.90224	6.90224	6.90224	6.90224
23	13.7492	13.7492	13.7492	13.7492	13.7492
22	20.5391	20.5391	20.5391	20.5391	20.5391
21	27.2702	27.2702	27.2702	27.2702	27.2702
20	33.9461	33.9461	33.9461	33.9461	33.9461
19	40.5689	40.5689	40.5689	40.5689	40.5689
18	47.1363	47.1363	47.1363	47.1363	47.1363
17	53.6485	53.6485	53.6485	53.6485	53.6485
16	60.1056	60.1056	60.1056	60.1056	60.1056
15	66.5145	66.5145	66.5145	66.5145	66.5145
14	72.8689	72.8689	72.8689	72.8689	72.8689
13	79.1542	79.1542	79.1542	79.1542	79.1542
12	85.348	85.348	85.348	85.348	85.348
11	91.4303	91.4303	91.4303	91.4303	91.4303
10	97.4007	97.4007	97.4007	97.4007	97.4007
9	103.262	103.262	103.262	103.262	103.262
8	109.016	109.016	109.016	109.016	109.016
7	114.644	114.644	114.644	114.644	114.644
6	120.117	120.117	120.117	120.117	120.117
5	125.422	125.422	125.422	125.422	125.422
4	130.535	130.535	130.535	130.535	130.535
3	135.4	135.4	135.4	135.4	135.4
2	139.957	139.957	139.957	139.957	139.957
1	144.053	144.053	144.053	144.053	144.053
GROUND	147.428	147.428	147.428	147.428	147.428

Columns

M1	M2	M3	M4	M5
1.73231	3.46463	3.46463	3.46463	3.46463
5.16993	10.3399	10.3399	10.3399	10.3399
8.57929	17.1586	17.1586	17.1586	17.1586
11.9598	23.9197	23.9197	23.9197	23.9197
15.3104	30.6208	30.6208	30.6208	30.6208
18.6357	37.2714	37.2714	37.2714	37.2714
21.9332	43.8665	43.8665	43.8665	43.8665
25.2031	50.4062	50.4062	50.4062	50.4062
28.4454	56.8908	56.8908	56.8908	56.8908
31.6602	63.3205	63.3205	63.3205	63.3205
34.8543	69.7086	69.7086	69.7086	69.7086
38.0146	76.0293	76.0293	76.0293	76.0293
41.1395	82.2791	82.2791	82.2791	82.2791
44.2084	88.4169	88.4169	88.4169	88.4169
47.2219	94.4438	94.4438	94.4438	94.4438
50.1788	100.358	100.358	100.358	100.358
53.0828	106.166	106.166	106.166	106.166
55.9329	111.866	111.866	111.866	111.866
58.7113	117.423	117.423	117.423	117.423
61.4055	122.811	122.811	122.811	122.811
64.0168	128.034	128.034	128.034	128.034
66.5177	133.035	133.035	133.035	133.035
68.8825	137.765	137.765	137.765	137.765
71.0745	142.149	142.149	142.149	142.149
72.978	145.956	145.956	145.956	145.956
74.4502	148.9	148.9	148.9	148.9

Fig. (4.5) Results of moments (kN.m) in Columns and Beams for 25storey.

4.5.2 Cantilever method (CM)

ANALYSIS OF RIGID FRAME USED
CANTILEVER METHOD (CM)

Sudan UNIVERSITY
M.SC STRUCTURE
PRESENTED BY:
Rabaa Mohammed

SUPERVISOR:

Dr. Osama Dawoud

INPUT:

Numbre of storey	Height of Story h (m)	lateral(Fy load (KN)	Span(L) Length(ft)
25	3.2	10.82696	5
24	3.2	21.48509	5
23	3.2	21.30851	5
22	3.2	21.12849	5
21	3.2	20.94092	5
20	3.2	20.78317	
19	3.2	20.60951	
18	3.2	20.43658	
17	3.2	20.26438	
16	3.2	20.09291	
15	3.2	19.96276	
14	3.2	19.75215	
13	3.2	19.53062	
12	3.2	19.18075	
11	3.2	18.83404	
10	3.2	18.48073	
9	3.2	18.15011	
8	3.2	17.81289	
7	3.2	17.36503	
6	3.2	16.83884	
5	3.2	16.32076	
4	3.2	15.63059	
3	3.2	14.7801	
2	3.2	13.69995	
1	3.2	11.89653	
GROUND	3.2	9.20109	

Fig. (4.6) Input of lateral force in the model

OUTPUT

Axial forces in columns

Number o Storey	1	2	3	4	5	6
25	0.524944	0.314966	0.104989	-0.10499	-0.31497	-0.52494
24	2.616532	1.569919	0.523306	-0.52331	-1.56992	-2.61653
23	6.782962	4.069777	1.356592	-1.35659	-4.06978	-6.78296
22	13.00694	7.804166	2.601389	-2.60139	-7.80417	-13.0069
21	21.27065	12.76239	4.254131	-4.25413	-12.7624	-21.2707
20	31.55735	18.93441	6.31147	-6.31147	-18.9344	-31.5574
19	43.85097	26.31058	8.770193	-8.77019	-26.3106	-43.851
18	58.13469	34.88082	11.62694	-11.6269	-34.8808	-58.1347
17	74.3918	44.63508	14.87836	-14.8784	-44.6351	-74.3918
16	92.60563	55.56338	18.52113	-18.5211	-55.5634	-92.6056
15	112.7615	67.65693	22.55231	-22.5523	-67.6569	-112.762
14	134.843	80.90582	26.96861	-26.9686	-80.9058	-134.843
13	158.8291	95.29749	31.76583	-31.7658	-95.2975	-158.829
12	184.6922	110.8153	36.93843	-36.9384	-110.815	-184.692
11	212.3983	127.439	42.47967	-42.4797	-127.439	-212.398
10	241.9137	145.1482	48.38274	-48.3827	-145.148	-241.914
9	273.2051	163.9231	54.64102	-54.641	-163.923	-273.205
8	306.2402	183.7441	61.24804	-61.248	-183.744	-306.24
7	340.9808	204.5885	68.19617	-68.1962	-204.589	-340.981
6	377.3799	226.4279	75.47597	-75.476	-226.428	-377.38
5	415.3866	249.232	83.07733	-83.0773	-249.232	-415.387
4	454.9426	272.9655	90.98851	-90.9885	-272.966	-454.943
3	495.973	297.5838	99.19459	-99.1946	-297.584	-495.973
2	538.3842	323.0305	107.6768	-107.677	-323.031	-538.384
1	582.0365	349.2219	116.4073	-116.407	-349.222	-582.036
GROUND	626.7117	376.027	125.3423	-125.342	-376.027	-626.712

Fig. (4.7) Results of Axial forces (kN) in columns for 25storey.

Shear forces in beams

Number of Storey	1	2	3	4	5
1	0.524944	0.83991	0.944899	0.83991	0.524944
2	2.091589	3.346542	3.764859	3.346542	2.091589
3	4.16643	6.666288	7.499574	6.666288	4.16643
4	6.223982	9.958371	11.20317	9.958371	6.223982
5	8.263711	13.22194	14.87468	13.22194	8.263711
6	10.2867	16.45871	18.51605	16.45871	10.2867
7	12.29361	19.66978	22.12851	19.66978	12.29361
8	14.28373	22.85397	25.71071	22.85397	14.28373
9	16.25711	26.01137	29.2628	26.01137	16.25711
10	18.21383	29.14212	32.78489	29.14212	18.21383
11	20.15592	32.24947	36.28065	32.24947	20.15592
12	22.08149	35.33038	39.74668	35.33038	22.08149
13	23.98611	38.37778	43.175	38.37778	23.98611
14	25.86302	41.38084	46.55344	41.38084	25.86302
15	27.70617	44.32986	49.8711	44.32986	27.70617
16	29.51537	47.22459	53.12766	47.22459	29.51537
17	31.29141	50.06625	56.32453	50.06625	31.29141
18	33.03507	52.85611	59.46312	52.85611	33.03507
19	34.74066	55.58506	62.53319	55.58506	34.74066
20	36.39903	58.23845	65.51826	58.23845	36.39903
21	38.00677	60.81083	68.41219	60.81083	38.00677
22	39.55593	63.28948	71.20067	63.28948	39.55593
23	41.03038	65.64862	73.85469	65.64862	41.03038
24	42.41124	67.85798	76.34022	67.85798	42.41124
25	43.65228	69.84364	78.5741	69.84364	43.65228
26	44.67519	71.48031	80.41535	71.48031	44.67519

Fig. (4.8) Results of shear forces (kN) in beams for 25 storey

Shear forces in columns

Number of Storey	1	2	3	4	5	6
1	0.820225	2.132584	2.788763	2.788763	2.132584	0.820225
2	2.447883	6.364495	8.322801	8.322801	6.364495	2.447883
3	4.062164	10.56163	13.81136	13.81136	10.56163	4.062164
4	5.662807	14.7233	19.25354	19.25354	14.7233	5.662807
5	7.24924	18.84803	24.64742	24.64742	18.84803	7.24924
6	8.823723	22.94168	30.00066	30.00066	22.94168	8.823723
7	10.38505	27.00113	35.30917	35.30917	27.00113	10.38505
8	11.93328	31.02652	40.57314	40.57314	31.02652	11.93328
9	13.46846	35.01799	45.79275	45.79275	35.01799	13.46846
10	14.99065	38.97568	50.9682	50.9682	38.97568	14.99065
11	16.50298	42.90774	56.11012	56.11012	42.90774	16.50298
12	17.99935	46.79832	61.1978	61.1978	46.79832	17.99935
13	19.47894	50.64526	66.22841	66.22841	50.64526	19.47894
14	20.93203	54.42328	71.16891	71.16891	54.42328	20.93203
15	22.35885	58.13302	76.0201	76.0201	58.13302	22.35885
16	23.75891	61.77316	80.78029	80.78029	61.77316	23.75891
17	25.13392	65.34818	85.45531	85.45531	65.34818	25.13392
18	26.48338	68.85678	90.04348	90.04348	68.85678	26.48338
19	27.79891	72.27716	94.51629	94.51629	72.27716	27.79891
20	29.07458	75.59391	98.85357	98.85357	75.59391	29.07458
21	30.311	78.8086	103.0574	103.0574	78.8086	30.311
22	31.49514	81.88735	107.0835	107.0835	81.88735	31.49514
23	32.61484	84.79859	110.8905	110.8905	84.79859	32.61484
24	33.65272	87.49706	114.4192	114.4192	87.49706	33.65272
25	34.55397	89.84032	117.4835	117.4835	89.84032	34.55397
26	35.25102	91.65265	119.8535	119.8535	91.65265	35.25102

Fig. (4.9) Results of shear forces (kN) in columns for 25-storey

Moments in columns

Number of Storey	1	2	3	4	5	6
25	1.312359	3.412134	4.462021	4.462021	3.412134	1.312359
24	3.916612	10.18319	13.31648	13.31648	10.18319	3.916612
23	6.499462	16.8986	22.09817	22.09817	16.8986	6.499462
22	9.060492	23.55728	30.80567	30.80567	23.55728	9.060492
21	11.59878	30.15684	39.43587	39.43587	30.15684	11.59878
20	14.11796	36.70669	48.00105	48.00105	36.70669	14.11796
19	16.61608	43.20181	56.49467	56.49467	43.20181	16.61608
18	19.09324	49.64243	64.91702	64.91702	49.64243	19.09324
17	21.54953	56.02878	73.2684	73.2684	56.02878	21.54953
16	23.98503	62.36109	81.54912	81.54912	62.36109	23.98503
15	26.40476	68.65238	89.77619	89.77619	68.65238	26.40476
14	28.79896	74.8773	97.91648	97.91648	74.8773	28.79896
13	31.16631	81.03241	105.9655	105.9655	81.03241	31.16631
12	33.49125	87.07725	113.8702	113.8702	87.07725	33.49125
11	35.77416	93.01283	121.6322	121.6322	93.01283	35.77416
10	38.01425	98.83705	129.2485	129.2485	98.83705	38.01425
9	40.21427	104.5571	136.7285	136.7285	104.5571	40.21427
8	42.3734	110.1708	144.0696	144.0696	110.1708	42.3734
7	44.47826	115.6435	151.2261	151.2261	115.6435	44.47826
6	46.51933	120.9503	158.1657	158.1657	120.9503	46.51933
5	48.4976	126.0938	164.8918	164.8918	126.0938	48.4976
4	50.39222	131.0198	171.3335	171.3335	131.0198	50.39222
3	52.18374	135.6777	177.4247	177.4247	135.6777	52.18374
2	53.84434	139.9953	183.0708	183.0708	139.9953	53.84434
1	55.28635	143.7445	187.9736	187.9736	143.7445	55.28635
GROUND	56.40163	146.6442	191.7655	191.7655	146.6442	56.40163

Moments in beams

1	2	3	4	5
1.312359	2.099775	2.362247	2.099775	1.312359
5.228971	8.366354	9.412148	8.366354	5.228971
10.41607	16.66572	18.74893	16.66572	10.41607
15.55995	24.89593	28.00792	24.89593	15.55995
20.65928	33.05484	37.1867	33.05484	20.65928
25.71674	41.14679	46.29013	41.14679	25.71674
30.73404	49.17446	55.32127	49.17446	30.73404
35.70932	57.13491	64.27678	57.13491	35.70932
40.64277	65.02843	73.15699	65.02843	40.64277
45.53457	72.8553	81.96222	72.8553	45.53457
50.3898	80.62368	90.70164	80.62368	50.3898
55.20373	88.32596	99.36671	88.32596	55.20373
59.96527	95.94444	107.9375	95.94444	59.96527
64.65756	103.4521	116.3836	103.4521	64.65756
69.26541	110.8247	124.6777	110.8247	69.26541
73.78842	118.0615	132.8191	118.0615	73.78842
78.22852	125.1656	140.8113	125.1656	78.22852
82.58767	132.1403	148.6578	132.1403	82.58767
86.85166	138.9627	156.333	138.9627	86.85166
90.99758	145.5961	163.7956	145.5961	90.99758
95.01693	152.0271	171.0305	152.0271	95.01693
98.88982	158.2237	178.0017	158.2237	98.88982
102.576	164.1215	184.6367	164.1215	102.576
106.0281	169.6449	190.8506	169.6449	106.0281
109.1307	174.6091	196.4352	174.6091	109.1307
111.688	178.7008	201.0384	178.7008	111.688

Fig (4.10) Results of bending moments (kN.m) in beams & columns for 25 Story.

4.6 Results of Finite element against portal against cantilever methods

4.6.1 Results of Axial Forces in Columns:

Table 4.4 Results of Axial Forces in columns:

	<i>Method of analysis</i>			<i>Difference %</i>		
<i>N. of Stories</i>	<i>Finite element (F.E.M)</i>	<i>Portal (PMP)</i>	<i>Cantilever (CMP)</i>	<i>(F.E.M), (PMP)</i>	<i>(F.E.M), (CMP)</i>	<i>(PMP), (CMP)</i>
Ground	651.31	827.2594	626.7117	-27.0147	3.776748	24.24242
1	593.28	768.2881	582.0365	-29.4984	1.895147	24.24242
2	537.9	710.6671	538.3842	-32.1188	-0.09002	24.24242
3	487.34	654.6843	495.973	-34.3383	-1.77144	24.24242
4	440.84	600.5242	454.9426	-36.2227	-3.19902	24.24242
5	397.74	548.3104	415.3866	-37.8565	-4.43673	24.24242
6	357.57	498.1414	377.3799	-39.313	-5.54014	24.24242
7	320.01	450.0947	340.9808	-40.6502	-6.55318	24.24242
8	284.82	404.237	306.2402	-41.9272	-7.5206	24.24242
9	251.85	360.6307	273.2051	-43.1927	-8.4793	24.24242
10	220.97	319.3261	241.9137	-44.5111	-9.47808	24.24242
11	192.1	280.3658	212.3983	-45.9478	-10.5665	24.24242
12	165.19	243.7937	184.6922	-47.5838	-11.8059	24.24242
13	140.22	209.6545	158.8291	-49.5182	-13.2714	24.24242
14	117.2	177.9928	134.843	-51.871	-15.0538	24.24242
15	96.11	148.8452	112.7615	-54.8697	-17.3255	24.24242
16	76.96	122.2394	92.60563	-58.835	-20.3296	24.24242
17	59.77	98.19718	74.3918	-64.2918	-24.4634	24.24242
18	44.54	76.7378	58.13469	-72.2896	-30.5224	24.24242
19	31.31	57.88327	43.85097	-84.8715	-40.0542	24.24242
20	20.12	41.6557	31.55735	-107.036	-56.8457	24.24242
21	11.04	28.07726	21.27065	-154.323	-92.669	24.24242
22	4.15	17.16917	13.00694	-313.715	-213.42	24.24242
23	-0.45	8.95351	6.782962	2089.669	1607.325	24.24242
24	-2.57	3.453822	2.616532	234.39	201.8106	24.24242
25	-1.85	0.692926	0.524944	137.4554	128.3753	24.24242

4.6.2 Results of Shear Forces in External Columns:

Table 4.5 Results of Shear Forces in External Columns.

<i>N. of Stories</i>	<i>Method of analysis</i>			<i>Difference %</i>		
	<i>Finite element (F.E.M)</i>	<i>Cantilever (CMP)</i>	<i>Portal (PMP)</i>	<i>(F.E.M), (PMP)</i>	<i>(F.E.M), (CMP)</i>	<i>(PMP), (CMP)</i>
Ground	66.55	35.25102	46.53135	30.08062	47.03077	24.24242
1	53.45	34.55397	45.61124	14.6656	35.35273	24.24242
2	48.95	33.65272	44.42158	9.251104	31.25084	24.24242
3	45.46	32.61484	43.05159	5.297867	28.25596	24.24242
4	42.5	31.49514	41.57358	2.179814	25.8938	24.24242
5	39.88	30.311	40.01052	-0.32728	23.99448	24.24242
6	37.51	29.07458	38.37844	-2.31524	22.48846	24.24242
7	35.33	27.79891	36.69456	-3.86233	21.31642	24.24242
8	33.26	26.48338	34.95806	-5.10541	20.37469	24.24242
9	31.27	25.13392	33.17677	-6.09776	19.62291	24.24242
10	29.35	23.75891	31.36176	-6.85437	19.04972	24.24242
11	27.47	22.35885	29.51369	-7.4397	18.60629	24.24242
12	25.6	20.93203	27.63028	-7.93079	18.23425	24.24242
13	23.73	19.47894	25.71221	-8.35317	17.91427	24.24242
14	21.83	17.99935	23.75914	-8.83713	17.54763	24.24242
15	19.95	16.50298	21.78393	-9.19263	17.27831	24.24242
16	18.04	14.99065	19.78765	-9.68766	16.90329	24.24242
17	16.14	13.46846	17.77836	-10.1509	16.55231	24.24242
18	14.2	11.93328	15.75192	-10.929	15.96285	24.24242
19	12.24	10.38505	13.70827	-11.9956	15.15482	24.24242
20	10.24	8.823723	11.64731	-13.7433	13.83083	24.24242
21	8.18	7.24924	9.568997	-16.9804	11.37848	24.24242
22	6.06	5.662807	7.474906	-23.3483	6.554334	24.24242
23	3.84	4.062164	5.362056	-39.6369	-5.78552	24.24242
24	1.5	2.447883	3.231205	-115.414	-63.1922	24.24242
25	-2.07	0.820225	1.082696	152.3042	139.6244	24.24242

4.6.3 Results of Shear Forces in Internal Columns:

Table 4.6 Results of Shear Forces in Internal Columns.

	<i>Method of analysis</i>			<i>Difference %</i>		
<i>N. of Stories</i>	<i>Finite element (F.E.M)</i>	<i>Portal (PMP)</i>	<i>Cantilever (CMP)</i>	<i>(F.E.M), (PMP)</i>	<i>(F.E.M), (CMP)</i>	<i>(PMP), (CMP)</i>
Ground	84.6	93	91.6	9.032258	7.641921	24.24242
1	88.4	91.2	89.8	3.070175	1.55902	24.24242
2	85.9	88.8	87.4	3.265766	1.716247	24.24242
3	82.9	86.1	84.7	3.716609	2.125148	24.24242
4	79.6	83	81.8	4.096386	2.689487	24.24242
5	76.3	80	78.8	4.625	3.172589	24.24242
6	72.8	76.7	75.5	5.084746	3.576159	24.24242
7	69.4	73	72.2	4.931507	3.878116	24.24242
8	65.9	69.9	68.8	5.722461	4.215116	24.24242
9	62.3	66	65.3	5.606061	4.594181	24.24242
10	58.7	62.7	61.7	6.379585	4.862237	24.24242
11	55.1	59	58.1	6.610169	5.163511	24.24242
12	51.5	55	54.4	6.363636	5.330882	24.24242
13	47.7	51	50.6	6.470588	5.731225	24.24242
14	44	47.5	46.7	7.368421	5.781585	24.24242
15	40.2	43.5	42.9	7.586207	6.293706	24.24242
16	36.4	39.5	38.9	7.848101	6.426735	24.24242
17	32.6	35.5	35	8.169014	6.857143	24.24242
18	28.7	31.5	31	8.888889	7.419355	24.24242
19	24.8	27.4	27	9.489051	8.148148	24.24242
20	20.9	23	22.9	9.130435	8.733624	24.24242
21	16.9	19.1	18.8	11.51832	10.10638	24.24242
22	12.9	14.9	14.7	13.42282	12.2449	24.24242
23	8.8	10.7	10.5	17.75701	16.19048	24.24242
24	4.6	6.4	6.3	28.125	26.98413	24.24242
25	0.08	2.1	2.1	96.19048	96.19048	24.24242

4.6.4 Results of Bending Moments in External Columns:

Table 4.7 Results of bending moment in External Columns.

	<i>Method of analysis</i>			<i>Difference %</i>		
<i>N. of Stories</i>	<i>Finite element (F.E.M)</i>	<i>Portal (PMP)</i>	<i>Cantilever (CMP)</i>	<i>(F.E.M), (PMP)</i>	<i>(F.E.M), (CMP)</i>	<i>(PMP), (CMP)</i>
Ground	-3.781	74.45015	56.40163	46.11504	59.17806	24.24242
1	0.983	72.97798	55.28635	21.44881	40.49153	24.24242
2	4.807	71.07453	53.84434	15.10143	35.6829	24.24242
3	8.378	68.88254	52.18374	10.82473	32.44298	24.24242
4	11.824	66.51773	50.39222	7.318202	29.78652	24.24242
5	15.196	64.01683	48.4976	4.392555	27.57012	24.24242
6	18.508	61.40551	46.51933	1.939457	25.71171	24.24242
7	21.778	58.7113	44.47826	-0.09769	24.16842	24.24242
8	25.022	55.93289	42.3734	-1.86471	22.82977	24.24242
9	28.245	53.08283	40.21427	-3.40274	21.66459	24.24242
10	31.489	50.17881	38.01425	-4.73994	20.65156	24.24242
11	34.731	47.2219	35.77416	-5.95234	19.73308	24.24242
12	38.006	44.20845	33.49125	-7.10968	18.8563	24.24242
13	41.274	41.13953	31.16631	-8.24483	17.99634	24.24242
14	44.569	38.01463	28.79896	-9.45447	17.07995	24.24242
15	47.908	34.85429	26.40476	-10.6872	16.14607	24.24242
16	51.336	31.66025	23.98503	-12.0915	15.08219	24.24242
17	54.909	28.44538	21.54953	-13.6815	13.87767	24.24242
18	58.654	25.20308	19.09324	-15.7272	12.32785	24.24242
19	62.62	21.93323	16.61608	-18.5067	10.22218	24.24242
20	66.958	18.6357	14.11796	-22.6356	7.094256	24.24242
21	71.77	15.3104	11.59878	-29.4858	1.90473	24.24242
22	77.244	11.95985	9.060492	-42.753	-8.14624	24.24242
23	83.717	8.57929	6.499462	-78.4749	-35.2083	24.24242
24	92.905	5.169928	3.916612	-425.934	-298.435	24.24242
25	138.165	1.732314	1.312359	145.8163	134.7093	24.24242

4.6.5 Results of Bending Moments in Internal Columns:

Table 4.8 Results of bending moment in Internal Columns.

	<i>Method of analysis</i>			<i>Difference %</i>		
<i>N. of Stories</i>	<i>Finite element (F.E.M)</i>	<i>Portal (PMP)</i>	<i>Cantilever (CMP)</i>	<i>(F.E.M), (PMP)</i>	<i>(F.E.M), (CMP)</i>	<i>(PMP), (CMP)</i>
Ground	179	148	146	-20.9459	-22.6027	24.24242
1	151	145	143	-4.13793	-5.59441	24.24242
2	141	142	139	0.704225	-1.43885	24.24242
3	134	137	135	2.189781	0.740741	24.24242
4	129	133	131	3.007519	1.526718	24.24242
5	123	128	126	3.90625	2.380952	24.24242
6	117	122	120	4.098361	2.5	24.24242
7	111	117	115	5.128205	3.478261	24.24242
8	105	111	110	5.405405	4.545455	24.24242
9	99	106	104	6.603774	4.807692	24.24242
10	93	100	98	7	5.102041	24.24242
11	87	94	93	7.446809	6.451613	24.24242
12	81	88	87	7.954545	6.896552	24.24242
13	75	82	81	8.536585	7.407407	24.24242
14	69	76	74	9.210526	6.756757	24.24242
15	62	69	68	10.14493	8.823529	24.24242
16	56	63	62	11.11111	9.677419	24.24242
17	50	56	56	10.71429	10.71429	24.24242
18	44	50	49	12	10.20408	24.24242
19	37	43	43	13.95349	13.95349	24.24242
20	31	37	36	16.21622	13.88889	24.24242
21	24	30	30	20	20	24.24242
22	18	23	23	21.73913	21.73913	24.24242
23	11	17	16	35.29412	31.25	24.24242
24	5.3	10	10	47	47	24.24242
25	-0.9	3.4	3.4	126.4706	126.4706	24.24242

4.6.6 Results of Shear Forces in External Beams:

Table 4.9 Results of Shear Forces in External beams.

	<i>Method of analysis</i>			<i>Difference %</i>		
<i>N. of Stories</i>	<i>Finite element (F.E.M)</i>	<i>Portal (PMP)</i>	<i>Cantilever (CMP)</i>	<i>(F.E.M), (PMP)</i>	<i>(F.E.M), (CMP)</i>	<i>(PMP), (CMP)</i>
Ground	58.03	58.97125	44.67519	-1.62201	23.01363	24.24242
1	55.39	57.62101	43.65228	-4.02781	21.19105	24.24242
2	50.55	55.98283	42.41124	-10.7474	16.10042	24.24242
3	46.5	54.16011	41.03038	-16.4734	11.76261	24.24242
4	43.1	52.21382	39.55593	-21.1458	8.222907	24.24242
5	40.16	50.16894	38.00677	-24.9227	5.361626	24.24242
6	37.56	48.04672	36.39903	-27.9199	3.090966	24.24242
7	35.19	45.85768	34.74066	-30.3145	1.276887	24.24242
8	32.98	43.60629	33.03507	-32.2204	-0.16697	24.24242
9	30.88	41.30466	31.29141	-33.7586	-1.33228	24.24242
10	28.87	38.96028	29.51537	-34.9508	-2.23542	24.24242
11	26.91	36.57214	27.70617	-35.9054	-2.95862	24.24242
12	24.97	34.13919	25.86302	-36.7208	-3.57639	24.24242
13	23.03	31.66166	23.98611	-37.4801	-4.15158	24.24242
14	21.09	29.14757	22.08149	-38.2056	-4.70123	24.24242
15	19.15	26.60581	20.15592	-38.9337	-5.25284	24.24242
16	17.2	24.04225	18.21383	-39.7805	-5.89434	24.24242
17	15.23	21.45938	16.25711	-40.9021	-6.74398	24.24242
18	13.23	18.85452	14.28373	-42.5134	-7.96469	24.24242
19	11.19	16.22757	12.29361	-45.0185	-9.86251	24.24242
20	9.08	13.57844	10.2867	-49.5423	-13.2896	24.24242
21	6.89	10.9081	8.263711	-58.3178	-19.9377	24.24242
22	4.6	8.215656	6.223982	-78.6012	-35.3039	24.24242
23	2.13	5.499687	4.16643	-158.201	-95.607	24.24242
24	-0.72	2.760897	2.091589	483.4579	390.4984	24.24242
25	-1.85	0.692926	0.524944	137.4554	128.3753	24.24242

4.6.7 Results of Shear Forces in Internal Beams:

Table 4.10 Results of Shear Forces Internal beams.

	<i>Method of analysis</i>			<i>Difference %</i>		
<i>N. of Stories</i>	<i>Finite element (F.E.M)</i>	<i>Portal (PMP)</i>	<i>Cantilever (CMP)</i>	<i>(F.E.M), (PMP)</i>	<i>(F.E.M), (CMP)</i>	<i>(PMP), (CMP)</i>
Ground	48.8	58	71.4	-18.8525	31.65266	24.24242
1	56.5	57	69.8	-0.88496	19.05444	24.24242
2	57.3	55	67.8	4.013962	15.48673	24.24242
3	56.5	54	65.6	4.424779	13.87195	24.24242
4	55.3	52	63.2	5.96745	12.5	24.24242
5	53.7	50	60.8	6.89013	11.67763	24.24242
6	51.9	48	58.2	7.514451	10.82474	24.24242
7	50	45	55.5	10	9.90991	24.24242
8	47.8	43	52.8	10.04184	9.469697	24.24242
9	45.5	41	50	9.89011	9	24.24242
10	43.2	38	47.2	12.03704	8.474576	24.24242
11	40.7	36	44.3	11.54791	8.126411	24.24242
12	38.1	34	41.3	10.76115	7.748184	24.24242
13	35.5	31	38.3	12.67606	7.310705	24.24242
14	32.8	29	35.3	11.58537	7.082153	24.24242
15	30	26	32.2	13.33333	6.832298	24.24242
16	27.3	24	29.1	12.08791	6.185567	24.24242
17	24.5	21	26	14.28571	5.769231	24.24242
18	21.7	18	22.8	17.05069	4.824561	24.24242
19	18.8	16	19.6	14.89362	4.081633	24.24242
20	16	13	16.4	18.75	2.439024	24.24242
21	13.2	10	13.2	24.24242	0	24.24242
22	10.4	8	9.9	23.07692	-5.05051	24.24242
23	7.6	5	6.6	34.21053	-15.1515	24.24242
24	5	2	3.3	60	-51.5152	24.24242
25	2.6	0.6	0.8	76.92308	-225	24.24242

4.6.8 Results of Bending Moments in External Beams

Table 4.11 Results of bending moment in external beams

<i>N. of Stories</i>	<i>Method of analysis</i>			<i>Difference %</i>		
	<i>Finite element (F.E.M)</i>	<i>Portal (PMP)</i>	<i>Cantilever (CMP)</i>	<i>(F.E.M), (PMP)</i>	<i>(F.E.M), (CMP)</i>	<i>(PMP), (CMP)</i>
Ground	167.687	147.4281	111.688	12.08136	33.39497	24.24242
1	161.867	144.0525	109.1307	11.00563	32.58002	24.24242
2	150.162	139.9571	106.0281	6.795942	29.39086	24.24242
3	140.009	135.4003	102.576	3.291738	26.73617	24.24242
4	131.179	130.5346	98.88982	0.491268	24.6146	24.24242
5	123.281	125.4223	95.01693	-1.73696	22.92654	24.24242
6	116.061	120.1168	90.99758	-3.49455	21.59504	24.24242
7	109.313	114.6442	86.85166	-4.877	20.54773	24.24242
8	102.866	109.0157	82.58767	-5.97838	19.71335	24.24242
9	96.644	103.2616	78.22852	-6.84744	19.05497	24.24242
10	90.582	97.40071	73.78842	-7.52766	18.53965	24.24242
11	84.599	91.43035	69.26541	-8.07497	18.12502	24.24242
12	78.637	85.34798	64.65756	-8.53413	17.77718	24.24242
13	72.647	79.15416	59.96527	-8.95723	17.45664	24.24242
14	66.628	72.86892	55.20373	-9.36681	17.14636	24.24242
15	60.585	66.51453	50.3898	-9.78713	16.82793	24.24242
16	54.512	60.10563	45.53457	-10.2613	16.46873	24.24242
17	48.393	53.64846	40.64277	-10.86	16.01518	24.24242
18	42.185	47.1363	35.70932	-11.7371	15.35067	24.24242
19	35.863	40.56893	30.73404	-13.122	14.30155	24.24242
20	29.39	33.9461	25.71674	-15.5022	12.49833	24.24242
21	22.727	27.27024	20.65928	-19.9905	9.098093	24.24242
22	15.809	20.53914	15.55995	-29.9205	1.575343	24.24242
23	8.469	13.74922	10.41607	-62.3476	-22.9906	24.24242
24	0.041	6.902242	5.228971	-16734.7	-12653.6	24.24242
25	-2.848	1.732314	1.312359	160.8256	146.08	24.24242

4.6.9 Results of Bending Moments in Internal Beams

Table 4.12 Results of bending moment in internal beams

	<i>Method of analysis</i>			<i>Difference %</i>		
<i>N. of Stories</i>	<i>Finite element (F.E.M)</i>	<i>Portal (PMP)</i>	<i>Cantilever (CMP)</i>	<i>(F.E.M), (PMP)</i>	<i>(F.E.M), (CMP)</i>	<i>(PMP), (CMP)</i>
Ground	134	147.4	178.7	-10	-33.3582	24.24242
1	151.2	144	174.6	4.761905	-15.4762	24.24242
2	155	139.9	169.6	9.741935	-9.41935	24.24242
3	155.1	135.4	164.1	12.70148	-5.80271	24.24242
4	153.1	130.5	158.2	14.76159	-3.33116	24.24242
5	149.6	125.4	152	16.17647	-1.60428	24.24242
6	145	120.1	145.5	17.17241	-0.34483	24.24242
7	139.6	114.6	138.9	17.90831	0.501433	24.24242
8	133.7	109	132.1	18.4742	1.196709	24.24242
9	127.3	103.2	125.1	18.93166	1.728201	24.24242
10	120.5	97.4	118	19.17012	2.074689	24.24242
11	113.5	91.4	110.8	19.47137	2.378855	24.24242
12	106.2	85.3	103.4	19.67985	2.636535	24.24242
13	98.7	79.1	95.9	19.85816	2.836879	24.24242
14	91	72.8	88.3	20	2.967033	24.24242
15	83.3	66.5	80.6	20.16807	3.241297	24.24242
16	75.5	60.1	72.8	20.39735	3.576159	24.24242
17	67.6	53.6	65	20.71006	3.846154	24.24242
18	59.6	47.1	57.1	20.97315	4.194631	24.24242
19	51.7	40.5	49.1	21.66344	5.029014	24.24242
20	43.8	33.9	41.1	22.60274	6.164384	24.24242
21	35.9	27.2	33	24.23398	8.077994	24.24242
22	28.2	20.5	24.8	27.30496	12.05674	24.24242
23	20.7	13.7	16.6	33.81643	19.80676	24.24242
24	13.6	6.9	8.3	49.26471	38.97059	24.24242
25	8.3	1.7	2	79.51807	75.90361	24.24242

4.7 Discussion of Results:

Results of simplified methods of analysis and Computer analysis for the three models under wind loads only are shown present in Figures (4.11) to (4.45).

4.7.1 Discussion of Axial Forces Results in External Columns:

Comparisons between results of simplified methods and FEM for axial forces in columns presented and compared as shown in Fig. (4.11) - (4.14).

Considering the FEM as the reference for comparison, it is clear that results of the simplified methods are approximately accurate results and can be used to estimate the axial forces on columns for stories.

- 1- Analysis results of the 25 storeys were shown in Fig. (4.11), it was shown that at level storey No.10, the difference between the axial forces at the reference point calculated using portal method and FEM is about 28%. The difference between the FEM and cantilever method is about 6%. At storey level No. 15, the difference calculated using portal method and FEM is about 36%.The difference between the FEM and cantilever method is about 15%. At storey level story no. 20, the difference calculated using portal method and FEM is about 56%. The difference between the FEM and cantilever method is about 42%. At storey level No.25, the difference calculated using portal method and FEM is about 229%. The difference between the FEM and cantilever method is about 269%.
- 2- Analysis results of the 20 storeys were shown in Fig. (4.12), it was shown that at level storey No.10, the difference between the axial forces at the reference point calculated using portal method and FEM is about 33%. The difference between the FEM and cantilever method is about 11%. At storey level No. 15, the difference calculated using portal method and FEM is about 54%.The difference

between the FEM and cantilever method is about 40%. At storey level story no. 20, the difference calculated using portal method and FEM is about 441%.

- 3- Analysis results of the 15 storeys were shown in Fig. (4.13), it was shown that at level storey No.10, the difference between the axial forces at the reference point calculated using portal method and FEM is about 44%. The difference between the FEM and cantilever method is about 26%. At storey level No. 15, the difference calculated using portal method and FEM is about 350%.The difference between the FEM and cantilever method is about 475%.
- 4- Analysis results of the 10 storeys were shown in Fig. (4.14), it was shown that at level storey No.5, the difference between the axial forces at the reference point calculated using portal method and FEM is about 26%. The difference between the FEM and cantilever method is about 4%. At storey level No. 10, the difference calculated using portal method and FEM is about 260%.The difference between the FEM and cantilever method is about 300%.

These differences above may be due to the values of axial forces are small which induce large percentages

The Cantilever method appears to give approximately accurate Results, slightly underestimated, for the top stories for all heights, and slightly over-estimated at bottom story's for columns for all heights. Though it shows little negative difference in the columns.

The Portal method appears to give approximately accurate Results, slightly over-estimated, for the story's for all heights in the columns.

As the building height increases, the Cantilever method appears to be giving more accurate results, and hence can be used to estimate the axial forces for tall buildings for preliminary design.

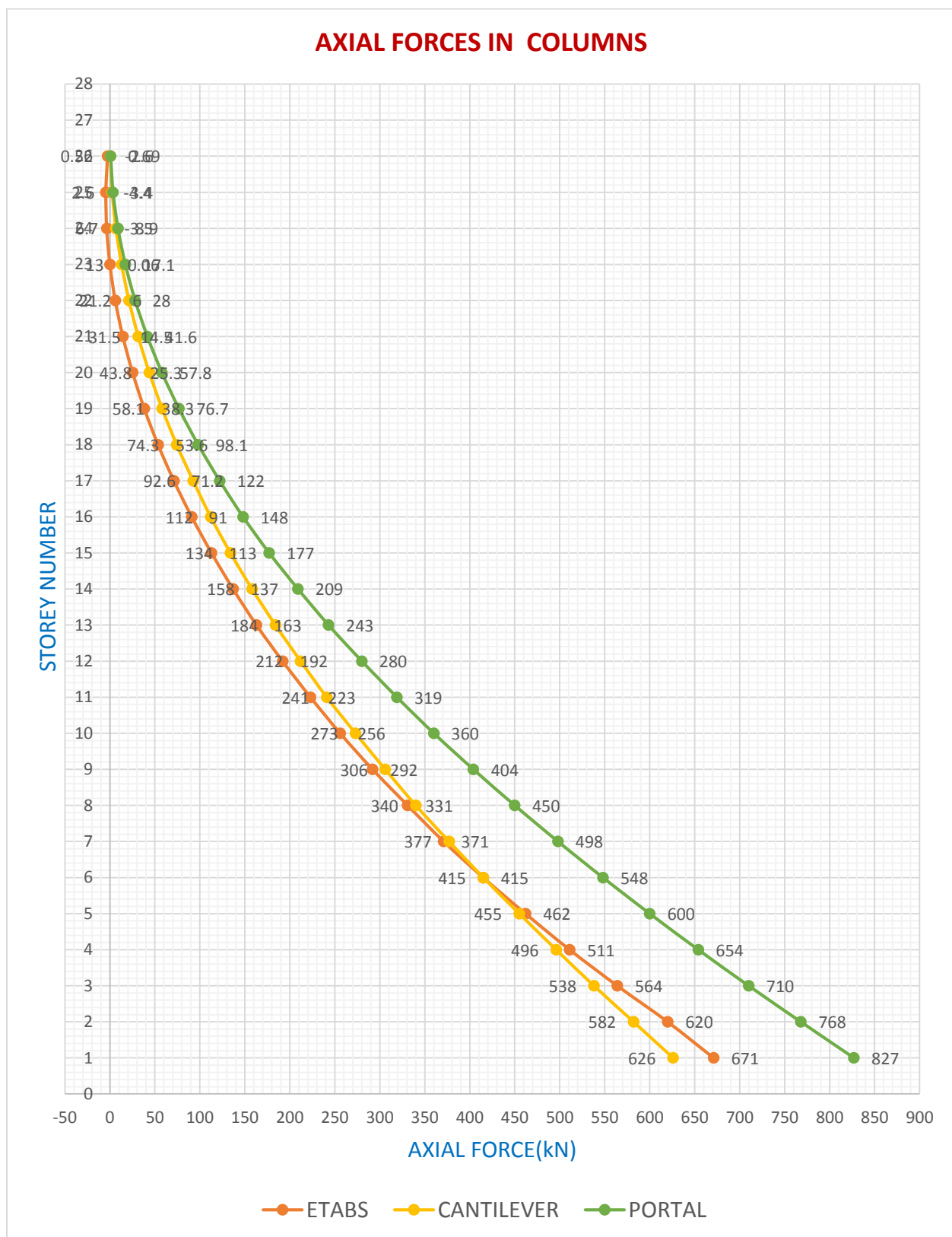


Fig. (4.11): Axial Forces in External Columns due to Wind Loads on 25 storey by Portal, Cantilever and ETABS

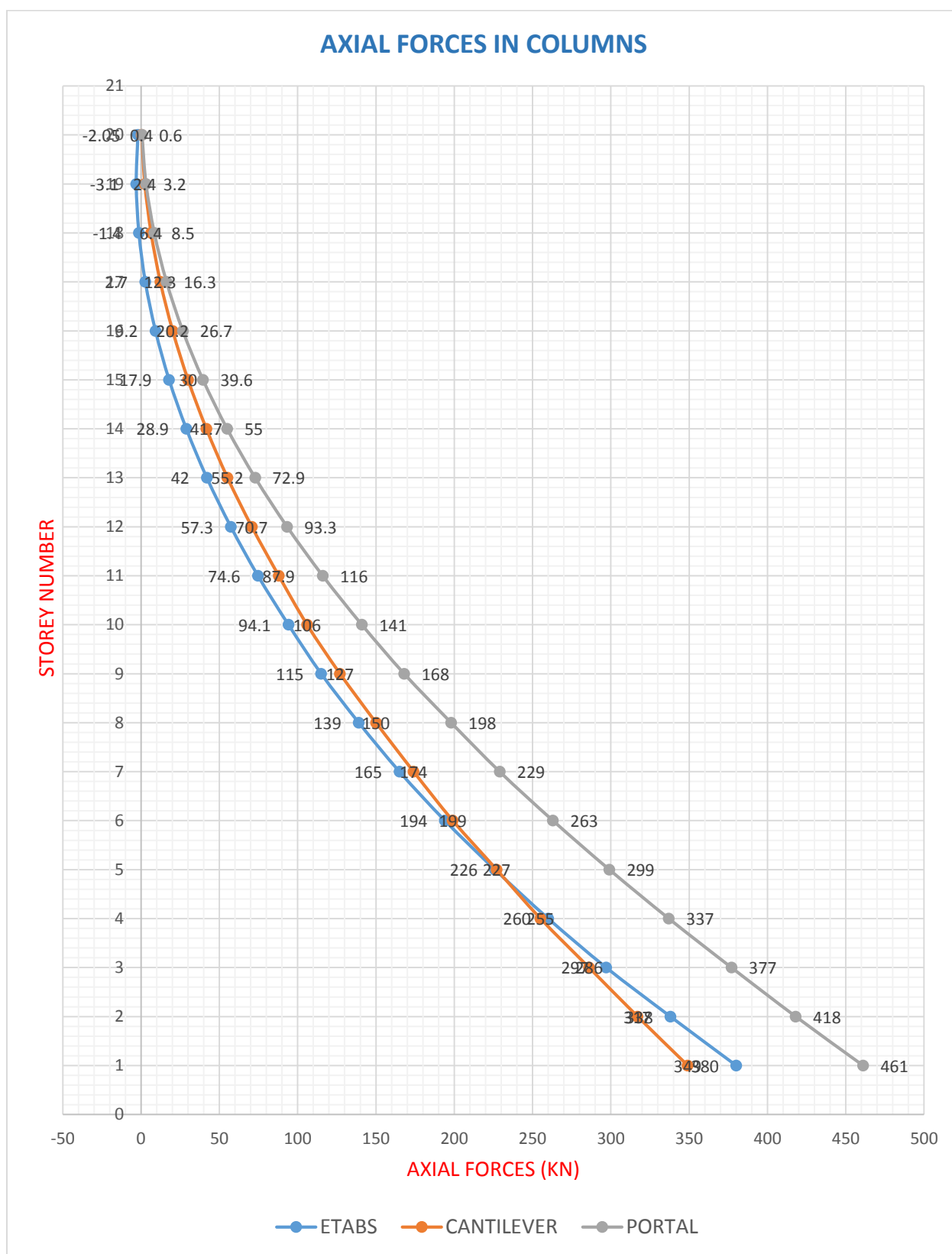


Fig (4.12): Axial Forces in External Columns due to Wind Loads on 20 storey by Portal, Cantilever and ETABS

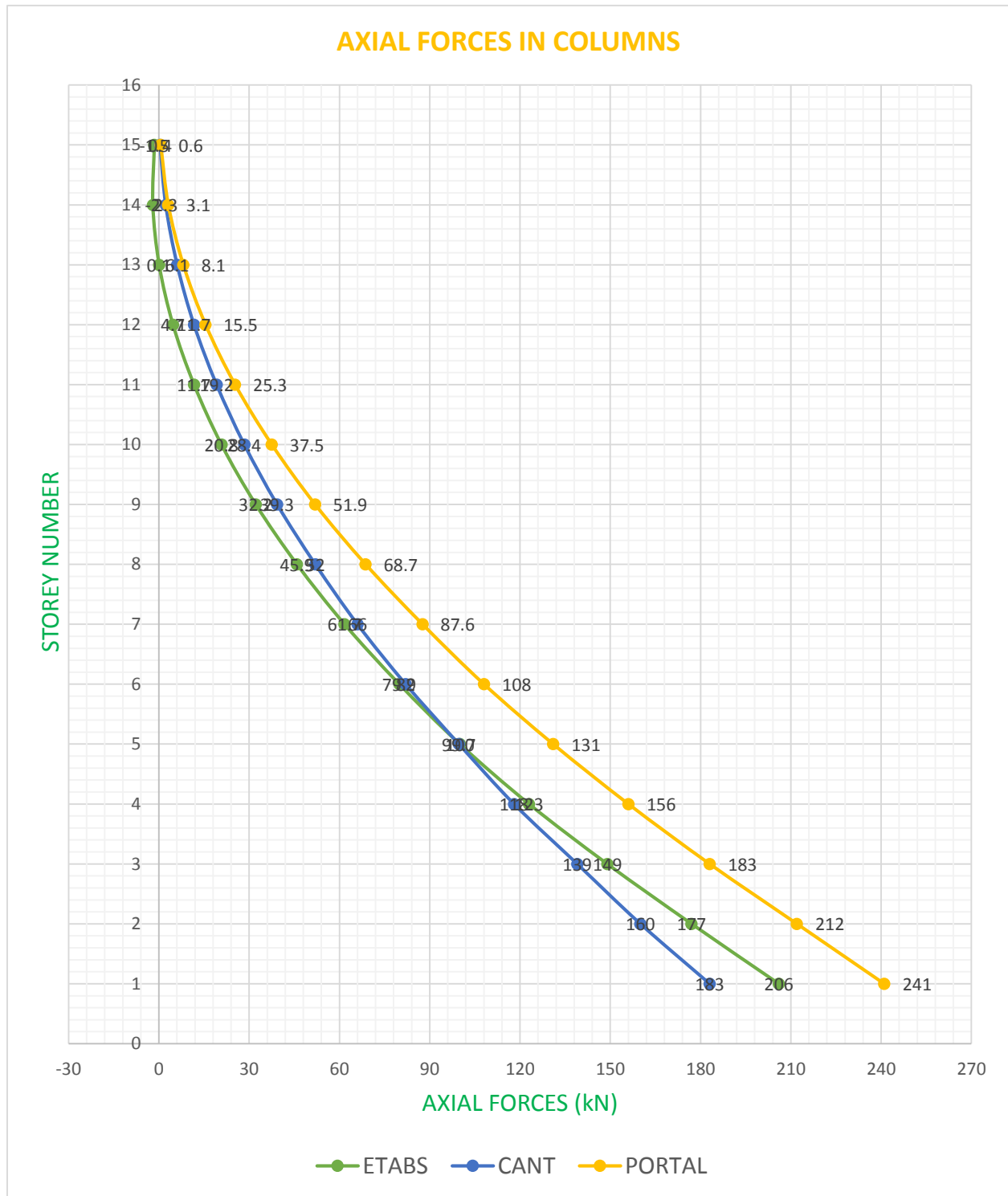


Fig (4.13): Axial Forces in External Columns due to Wind Loads on 15 storey by Portal, Cantilever and ETABS

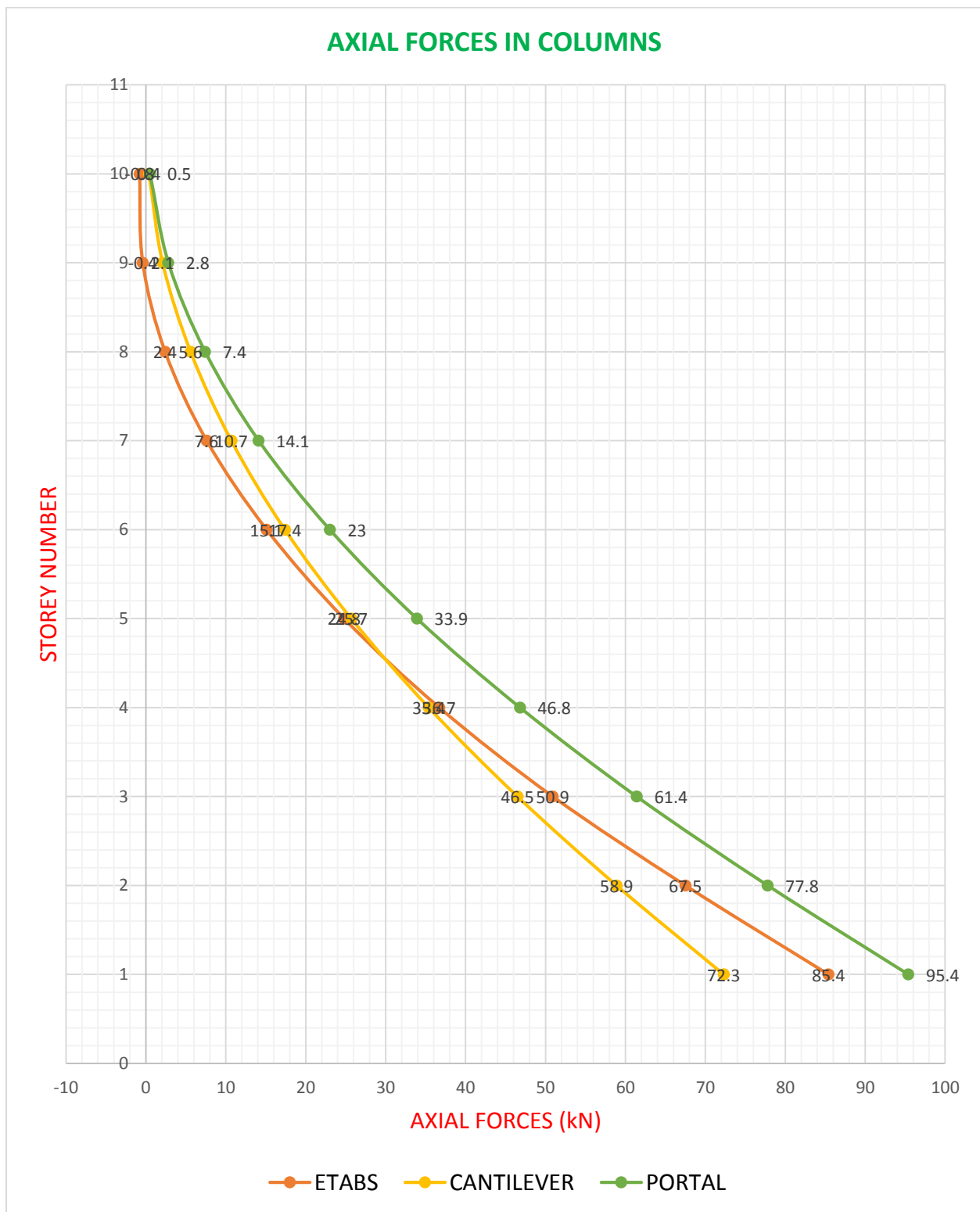


Fig (4.14): Axial Forces in External Columns due to Wind Loads on 10 storey by Portal, Cantilever and ETABS

4.7.2 Discussion of Shear Force Results on Columns:

Comparisons of shear forces results in column between simplified methods and FEM were presented and compared as shown in Figures (4.15) - (4.22).

Considering the FEM as the reference for comparison, it is clear that results of the simplified methods are approximately accurate results, and can be used to estimate the shear forces on columns for stories.

- 1- Analysis results of the 25 storeys were shown in Fig. (4.15), it was shown that at level storey No.10, the difference between the shear forces in external columns at the reference point calculated using portal method and FEM was about 12%. The difference between the FEM and cantilever method was about 13%. At storey level No. 15, the difference calculated using portal method and FEM was about 19%.The difference between the FEM and cantilever method was about 5.7%. At storey level story no. 20, the difference calculated using portal method and FEM was about 27%. The difference between the FEM and cantilever method was about -3%. At storey level No.25, the difference calculated using portal method and FEM was about 97%. The difference between the FEM and cantilever method was about -3328%.
- 2- Analysis results of the 25 storeys were shown in Fig. (4.19), it was shown that at level storey No.10, the difference between the shear forces in internal columns at the reference point calculated using portal method and FEM was about 5%. The difference between the FEM and cantilever method was about 4%. At storey level No. 15, the difference calculated using portal method and FEM was about 7%.The difference between the FEM and cantilever method was about 5%. At storey level story no. 20, the difference calculated using portal method and FEM was about 9%. The difference between the FEM and cantilever method was about 8%. At storey level No.25, the difference calculated using portal method and

FEM was about 28%. The difference between the FEM and cantilever method was about 26%.

- 3- Analysis results of the 20 storeys were shown in Fig. (4.16), it was shown that at level storey No.10, the difference between the shear forces in external columns at the reference point calculated using portal method and FEM was about 7%. The difference between the FEM and cantilever method was about 17%. At storey level No. 15, the difference calculated using portal method and FEM was about 15%. The difference between the FEM and cantilever method was about 10%. At storey level story no. 20, the difference calculated using portal method and FEM was about 350%. The difference between the FEM and cantilever method was about 131%.
- 4- Analysis results of the 20 storeys were shown in Fig. (4.20), it was shown that at level storey No.10, the difference between the shear forces in internal columns at the reference point calculated using portal method and FEM was about 8%. The difference between the FEM and cantilever method was about 7%. At storey level No. 15, the difference calculated using portal method and FEM was about 10%. The difference between the FEM and cantilever method was about 9%. At storey level story no. 20, the difference calculated using portal method and FEM was about 70%. The difference between the FEM and cantilever method was about 70%.
- 5- Analysis results of the 15 storeys were shown in Fig. (4.17), it was shown that at level storey No.10, the difference between the shear forces in external columns at the reference point calculated using portal method and FEM was about 11%. The difference between the FEM and cantilever method was about 333%. At storey level No. 15, the difference calculated using portal method and FEM was about 133%.
- 6- Analysis results of the 15 storeys were shown in Fig. (4.21), it was shown that at level storey No.10, the difference between the shear forces in internal columns

at the reference point calculated using portal method and FEM was about 8%. The difference between the FEM and cantilever method was about 6%. At storey level No. 15, the difference calculated using portal method and FEM was about 63%. The difference between the FEM and cantilever method was about 63%.

- 7- Analysis results of the 10 storeys were shown in Fig. (4.18), it was shown that at level storey No.5, the difference between the shear forces in external columns at the reference point calculated using portal method and FEM was about 5 %. The difference between the FEM and cantilever method was about 28%. At storey level No. 10, the difference calculated using portal method and FEM was about 190%. The difference between the FEM and cantilever method was about 160%.
- 8- Analysis results of the 10 storeys were shown in Fig. (4.22), it was shown that at level storey No.5, the difference between the shear forces in internal columns at the reference point calculated using portal method and FEM was about 6 %. The difference between the FEM and cantilever method was about 5%. At storey level No. 10, the difference calculated using portal method and FEM was about 38%. The difference between the FEM and cantilever method was about 35%.

Some differences above may be due to the values of shear forces were small which induce large percentages.

In an over-all view, the Cantilever method gives approximately accurate results, slightly over-estimated, for the internal columns for all heights, and slightly under-estimated at bottom stories for outer columns for all heights.

The Portal method gives over-estimated shear force results in internal columns, and slightly under-estimated at bottom stories, slightly over-estimated at top story's for outer columns for all heights.

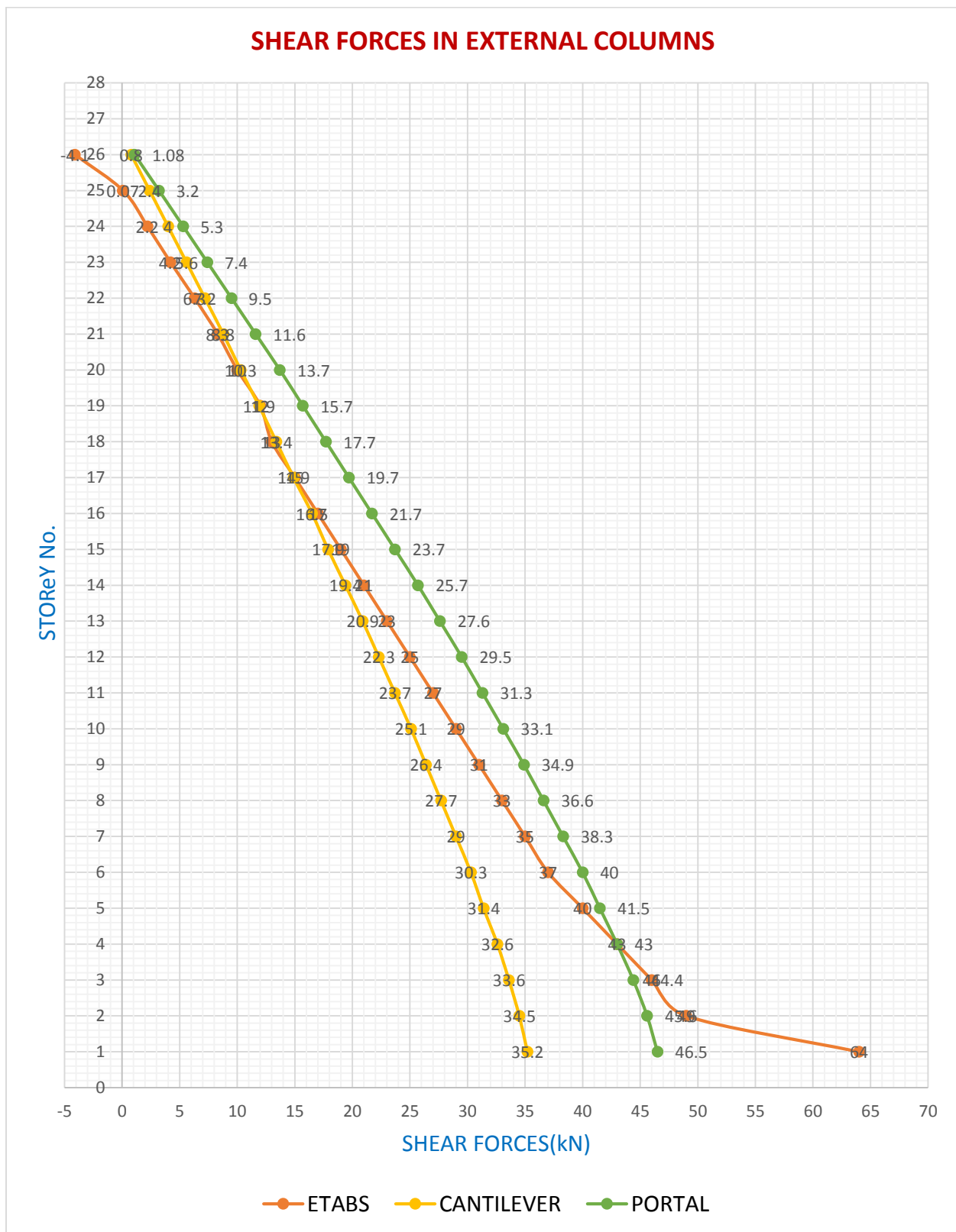


Fig. (4.15): Shear Forces in External Columns duo to Wind Loads on 25Storey

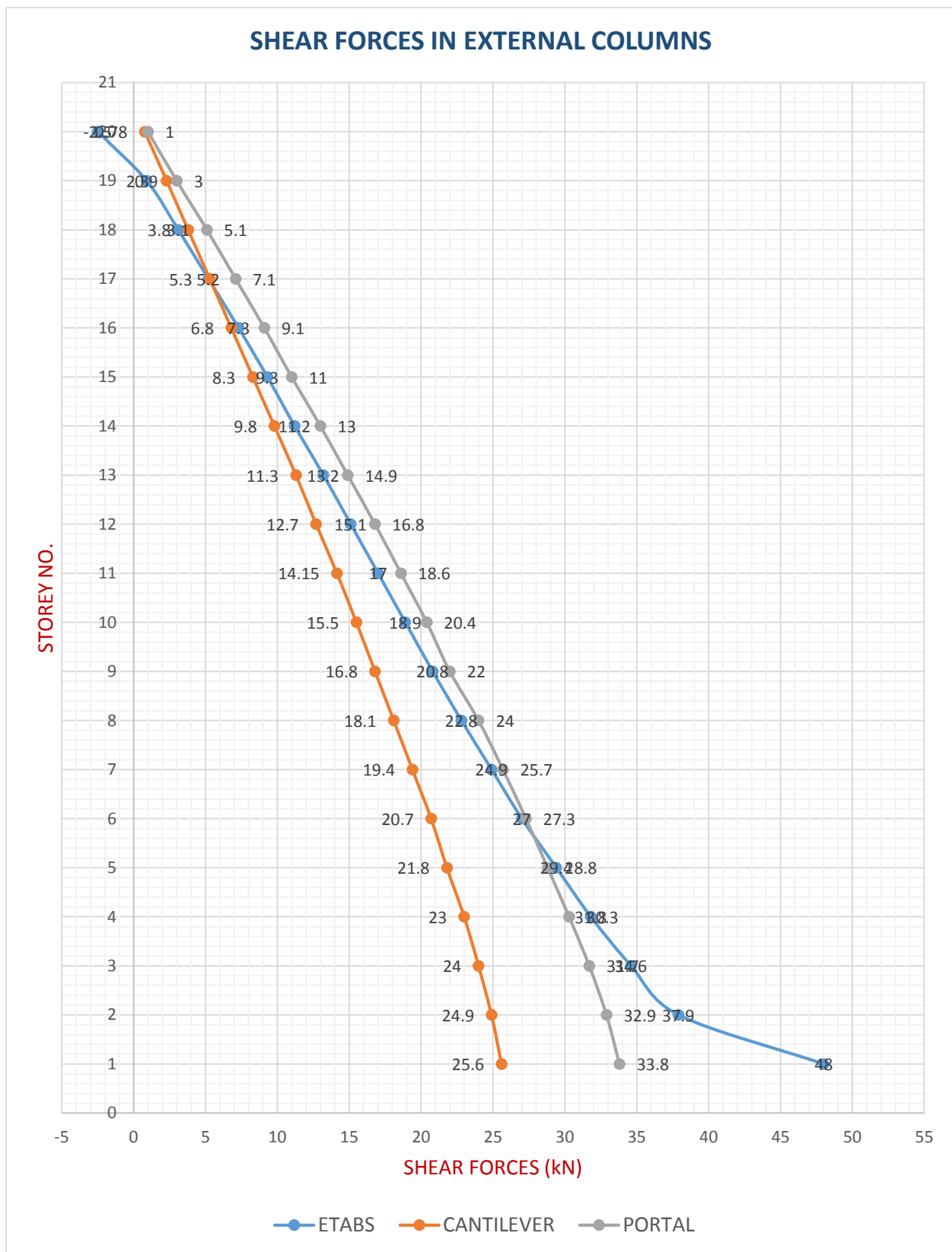


Fig (4.16): Shear Forces in External Columns due to Wind Loads on 20 Storey

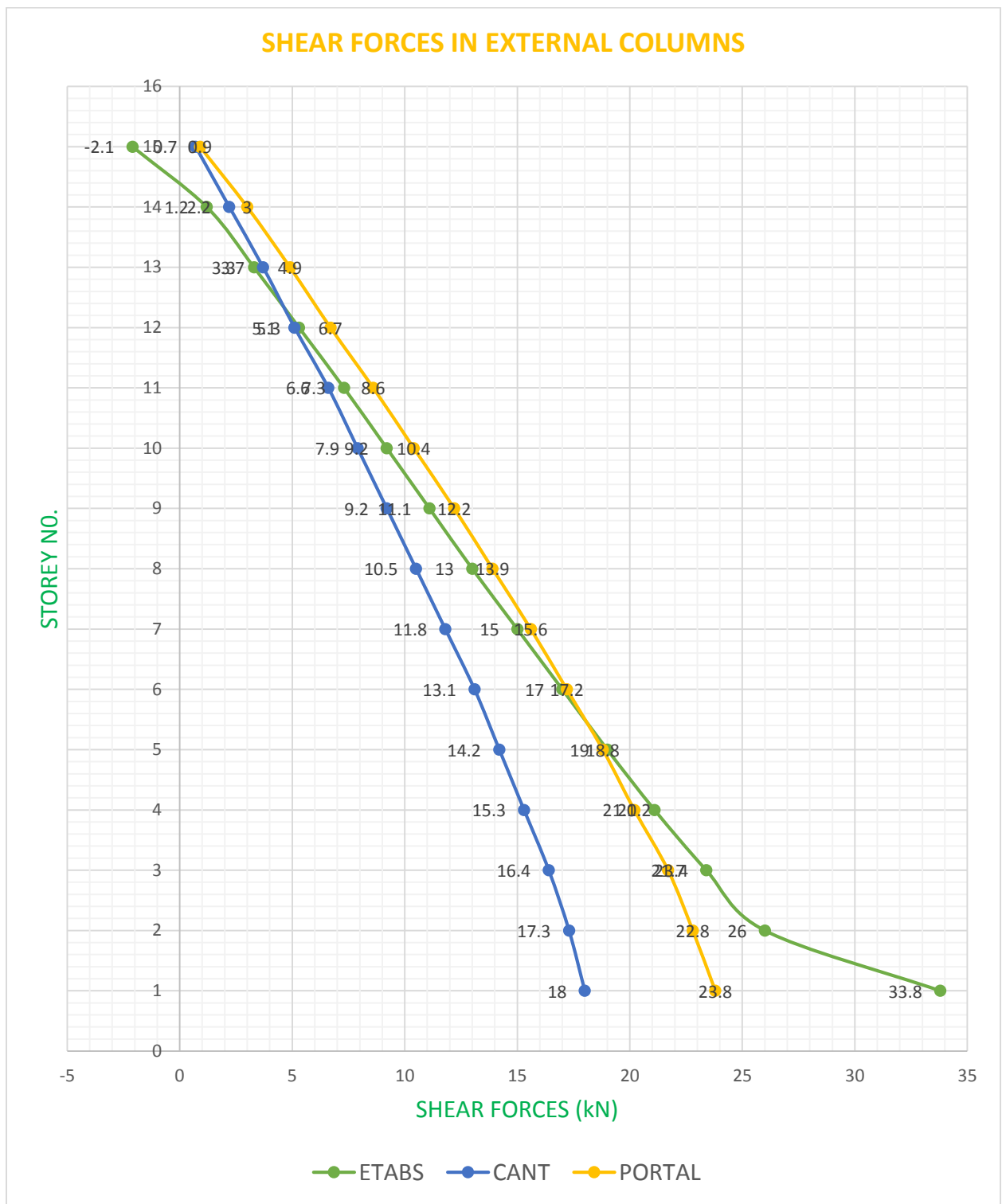


Fig. (4.17): Shear Forces in External Columns due to Wind Loads on 15 Storey

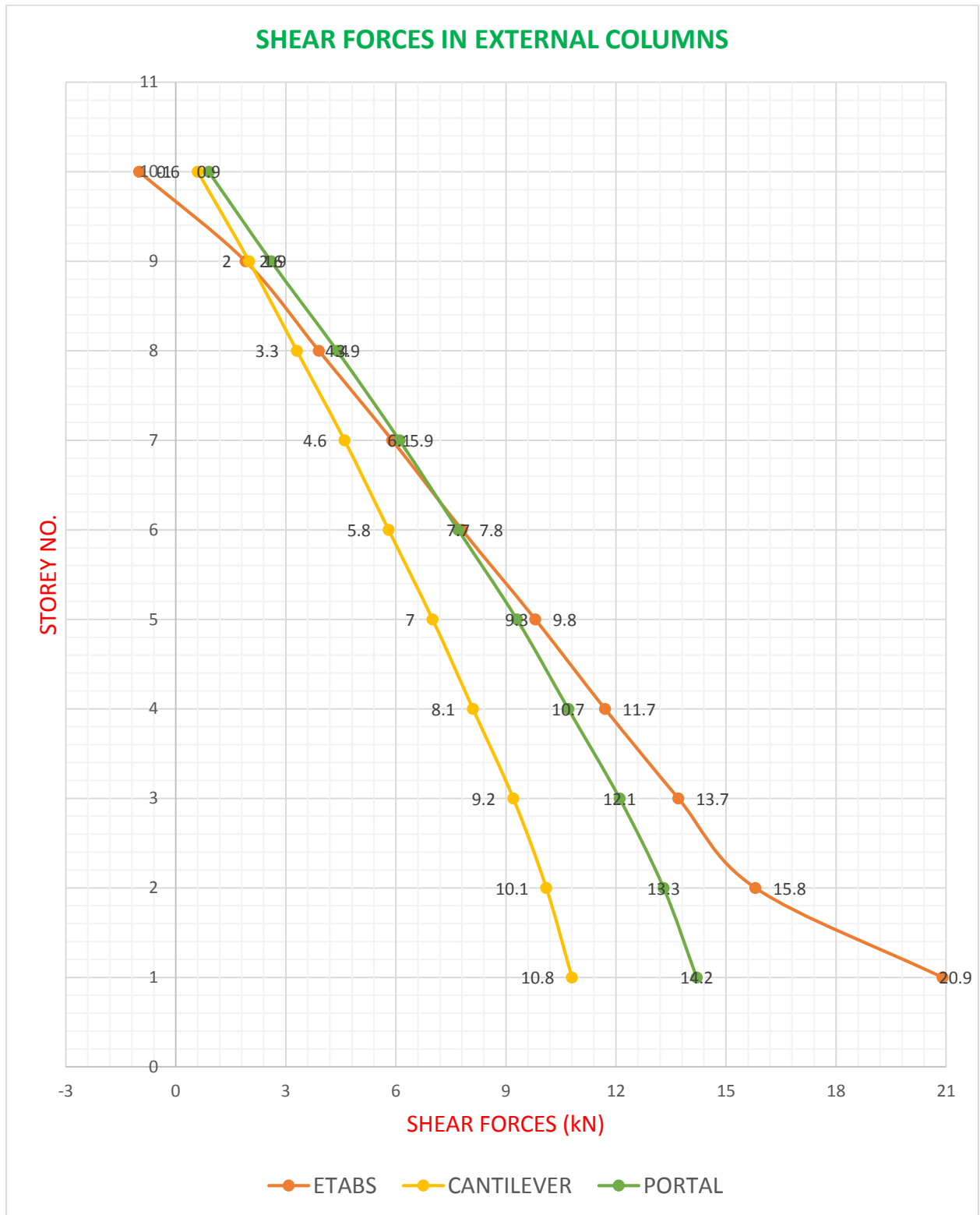


Fig (4.18): Shear Forces in External Columns duo to Wind Loads on 10 Storey



Fig (4.19): Shear Forces in Internal Columns due to Wind Loads on 25 Storey

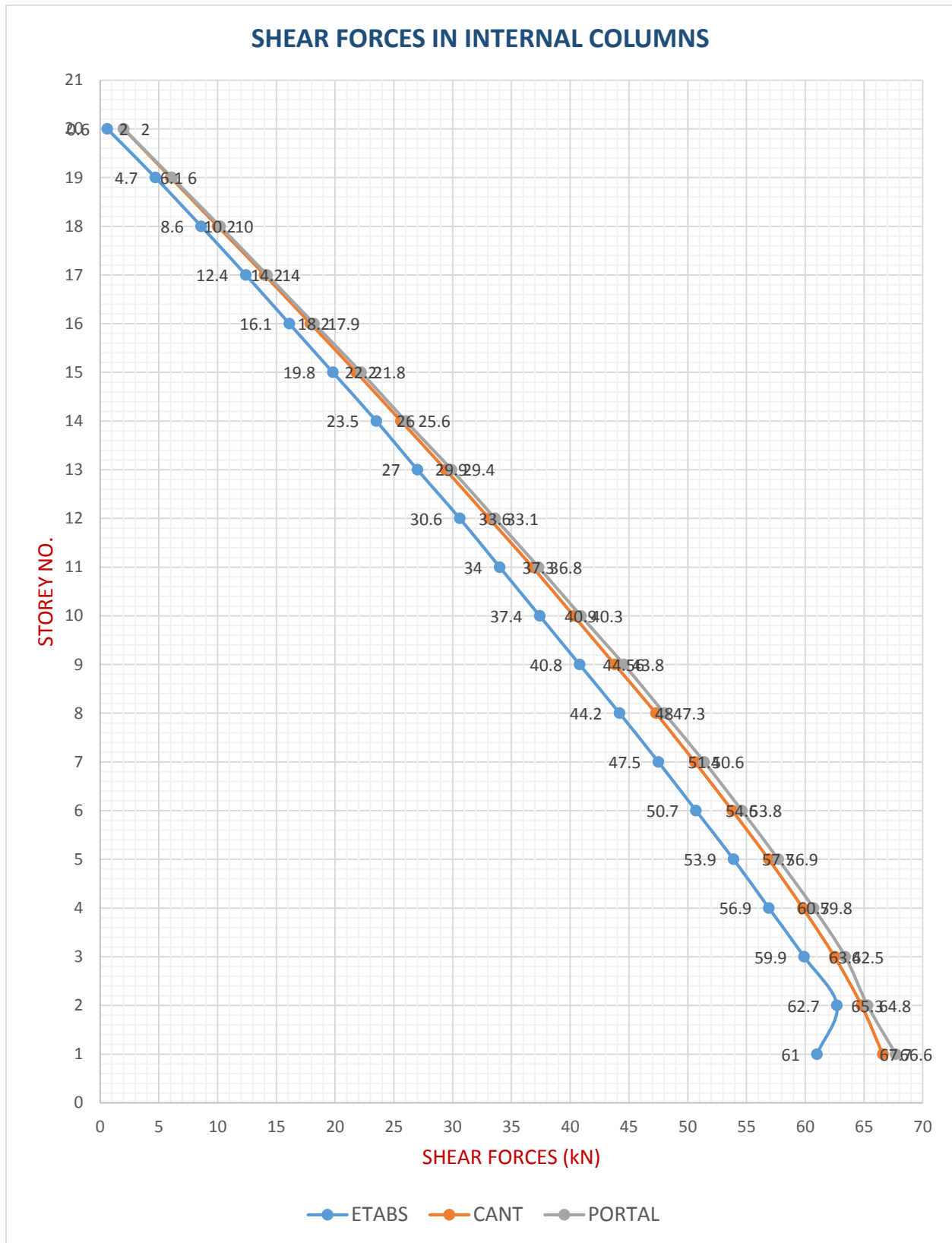


Fig. (4.20): Shear Forces in Internal Columns due to Wind Loads on 20 Storey



Fig. (4.21): Shear Forces in Internal Columns due to Wind Loads on 15 Storey

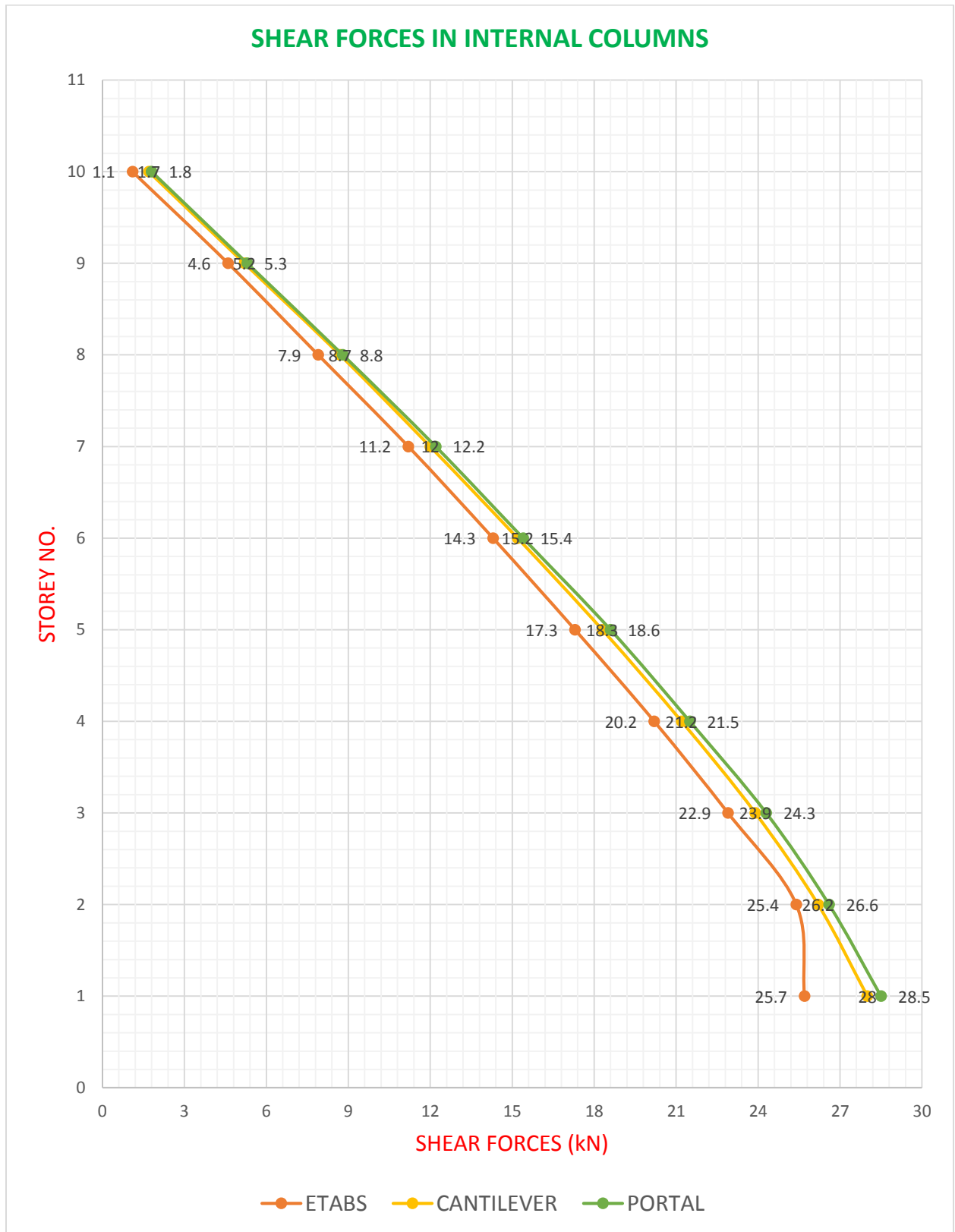


Fig (4.22): Shear Forces in Internal Columns due to Wind Loads on 10 Storey

4.7.3 Discussion of Bending Moments Results in columns:

Comparisons of bending moments results in column between simplified methods and FEM were presented and compared as shown in Figures (4.23) - (4.30).

Considering the FEM as the reference for comparison, it is clear that results of the simplified methods are approximately accurate results, and can be used to estimate the bending moments on columns for stories.

- 1- Analysis results of the 25 storeys were shown in Fig. (4.23), it was shown that at level storey No.10, the difference between the bending moments in external columns at the reference point calculated using portal method and FEM was about 15%. The difference between the FEM and cantilever method was about 11%. At storey level No. 15, the difference calculated using portal method and FEM was about 23%.The difference between the FEM and cantilever method was about 3.4%. At storey level story no. 20, the difference calculated using portal method and FEM was about 38%. The difference between the FEM and cantilever method was about -23%. At storey level No.25, the difference calculated using portal method and FEM was about 158%. The difference between the FEM and cantilever method was about 203%.
- 2- Analysis results of the 25 storeys were shown in Fig. (4.27), it was shown that at level storey No.10, the difference between the bending moments in internal columns at the reference point calculated using portal method and FEM was about 6.6%. The difference between the FEM and cantilever method was about 4.8%. At storey level No. 15, the difference calculated using portal method and FEM was about 9.2%.The difference between the FEM and cantilever method was about 6.7%. At storey level story no. 20, the difference calculated using portal method and FEM was about 13.9%. The difference between the FEM and cantilever method was about 13.9%. At storey level No.25, the difference

calculated using portal method and FEM was about 47%. The difference between the FEM and cantilever method was about 47%.

- 3- Analysis results of the 20 storeys were shown in Fig. (4.24), it was shown that at level storey No.10, the difference between the bending moments in external columns at the reference point calculated using portal method and FEM was about 9.7%. The difference between the FEM and cantilever method was about 16%. At storey level No. 15, the difference calculated using portal method and FEM was about 23%.The difference between the FEM and cantilever method was about 1.4%. At storey level story no. 20, the difference calculated using portal method and FEM was about 375%. The difference between the FEM and cantilever method was about 129%.
- 4- Analysis results of the 20 storeys were shown in Fig. (4.28), it was shown that at level storey No.10, the difference between the bending moments in internal columns at the reference point calculated using portal method and FEM was about 8.7%. The difference between the FEM and cantilever method was about 7.4%. At storey level No. 15, the difference calculated using portal method and FEM was about 12.4%.The difference between the FEM and cantilever method was about 11%. At storey level story no. 20, the difference calculated using portal method and FEM was about 84.3%. The difference between the FEM and cantilever method was about 84.3%.
- 5- Analysis results of the 15 storeys were shown in Fig. (4.25), it was shown that at level storey No.10, the difference between the bending moments in external columns at the reference point calculated using portal method and FEM was about 19%. The difference between the FEM and cantilever method was about 5%. At storey level No. 15, the difference calculated using portal method and FEM was about 343%.The difference between the FEM and cantilever method was about 130%.

- 6- Analysis results of the 15 storeys were shown in Fig. (4.29), it was shown that at level storey No.10, the difference between the bending moments in internal columns at the reference point calculated using portal method and FEM was about 10%. The difference between the FEM and cantilever method was about 9%. At storey level No. 15, the difference calculated using portal method and FEM was about 80%.The difference between the FEM and cantilever method was about 80%.
- 7- Analysis results of the 10 storeys were shown in Fig. (4.26), it was shown that at level storey No.5, the difference between the bending moments in external columns at the reference point calculated using portal method and FEM was about 25%. The difference between the FEM and cantilever method was about 1.3%. At storey level No. 10, the difference calculated using portal method and FEM was about 155%.The difference between the FEM and cantilever method was about 170%.
- 8- Analysis results of the 10 storeys were shown in Fig. (4.30), it was shown that at level storey No.5, the difference between the bending moments in internal columns at the reference point calculated using portal method and FEM was about 7%. The difference between the FEM and cantilever method was about 6%. At storey level No. 10, the difference calculated using portal method and FEM was about 55%.The difference between the FEM and cantilever method was about 53%.

Some differences above may be due to the values of bending moments were small which induce large percentages

The Cantilever method appears to give results approximately similar to the program results in the internal columns, and the top story in external columns, but appears to under-estimate for the bottom story in the external columns. Though it shows little negative difference in the internal columns.

The Portal method appears to give results approximately similar to the program results in the internal columns, and the top and middle story in external columns, but appears under-estimate bending moments for the bottom story in the external columns.

As the building height increases, the Cantilever method appears to be giving more accurate results, and hence can be used to estimate the bending moments for tall buildings for preliminary design.

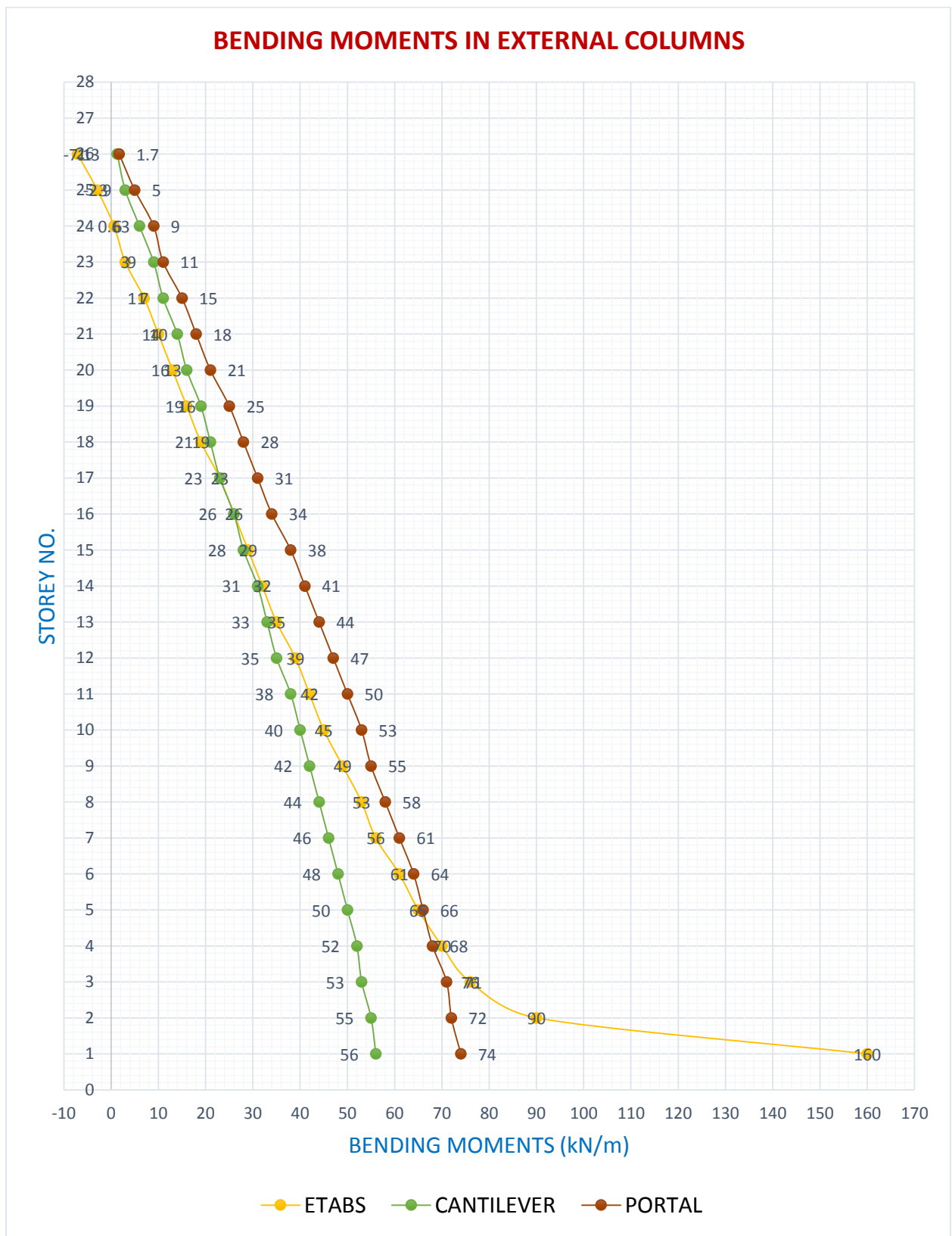


Fig (4.23): Bending Moments in External Columns duo to Wind Loads on 25 Storey

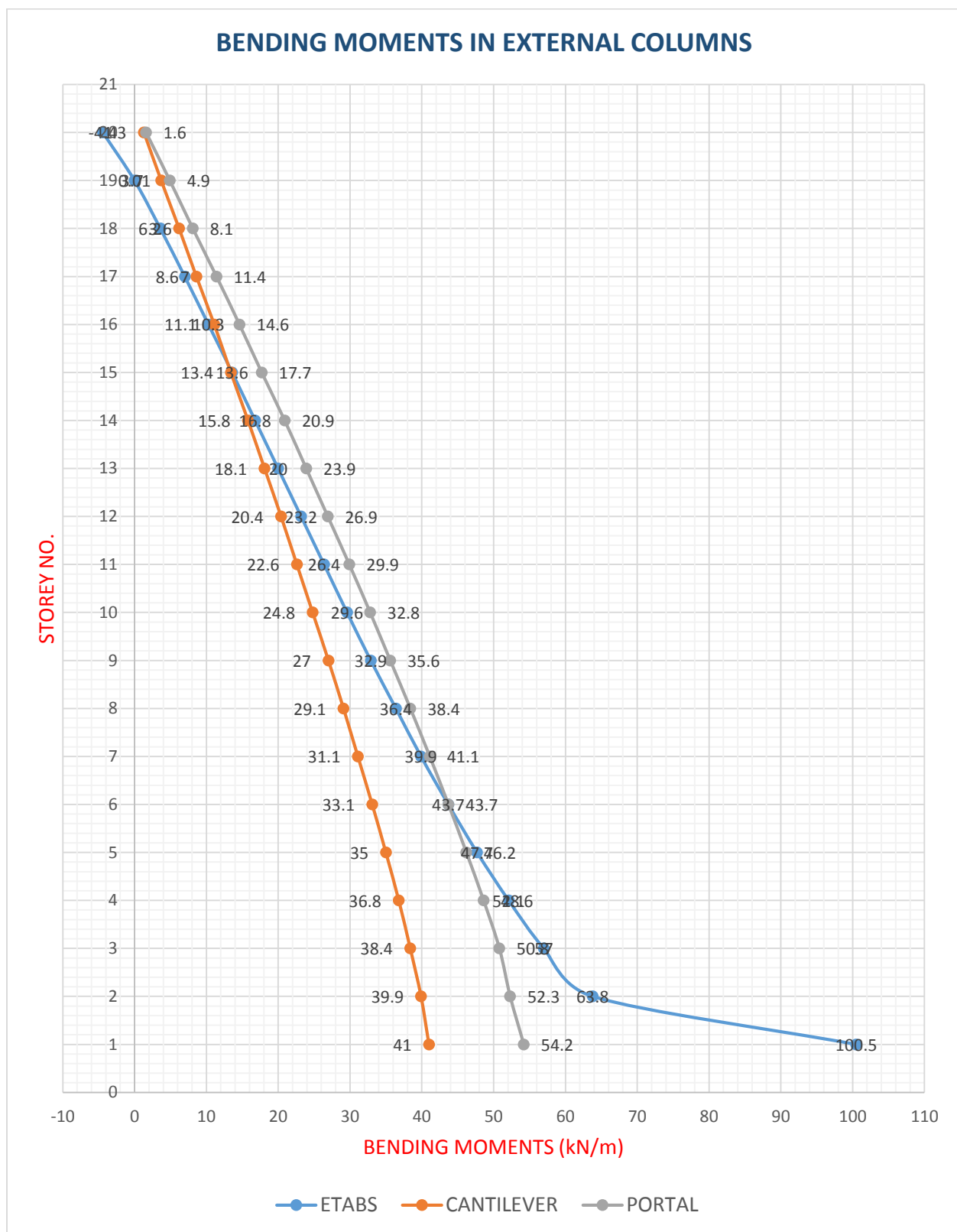


Fig (4.24): Bending Moments in External Columns due to Wind Loads on 20 Storey

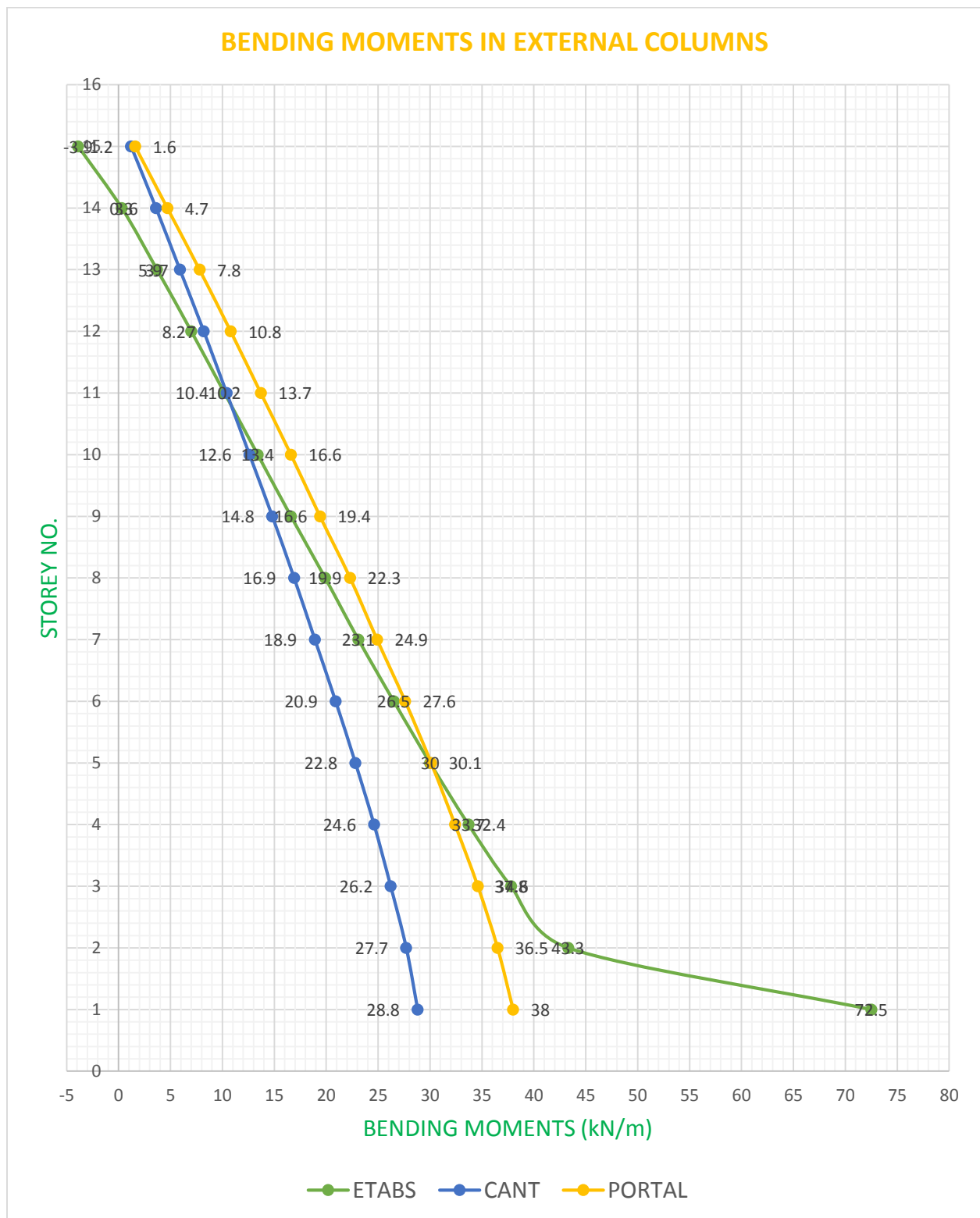


Fig (4.25): Bending Moments in External Columns due to Wind Loads on 15 Storey

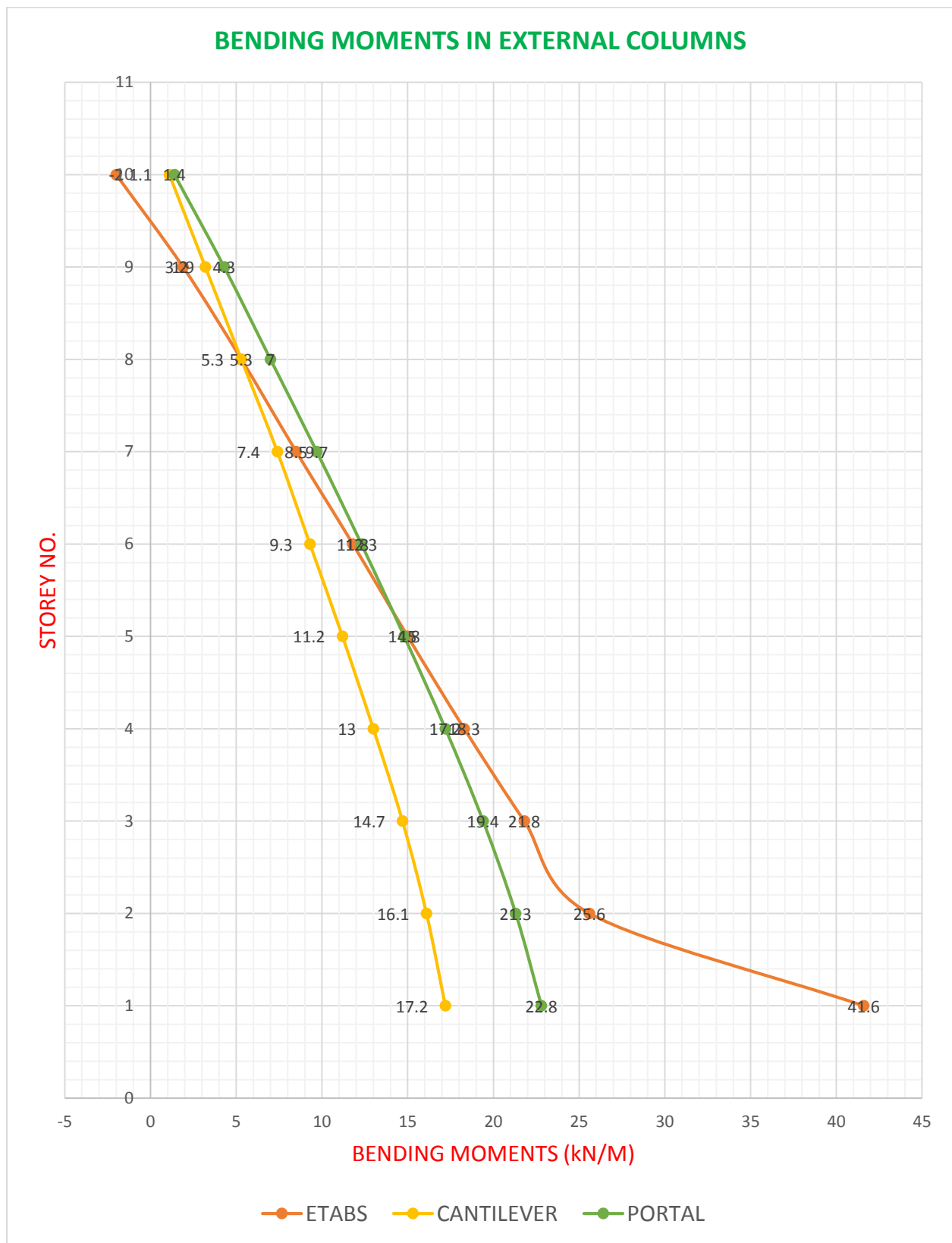


Fig (4.26): Bending Moments in External Columns due to Wind Loads on 10 Storey

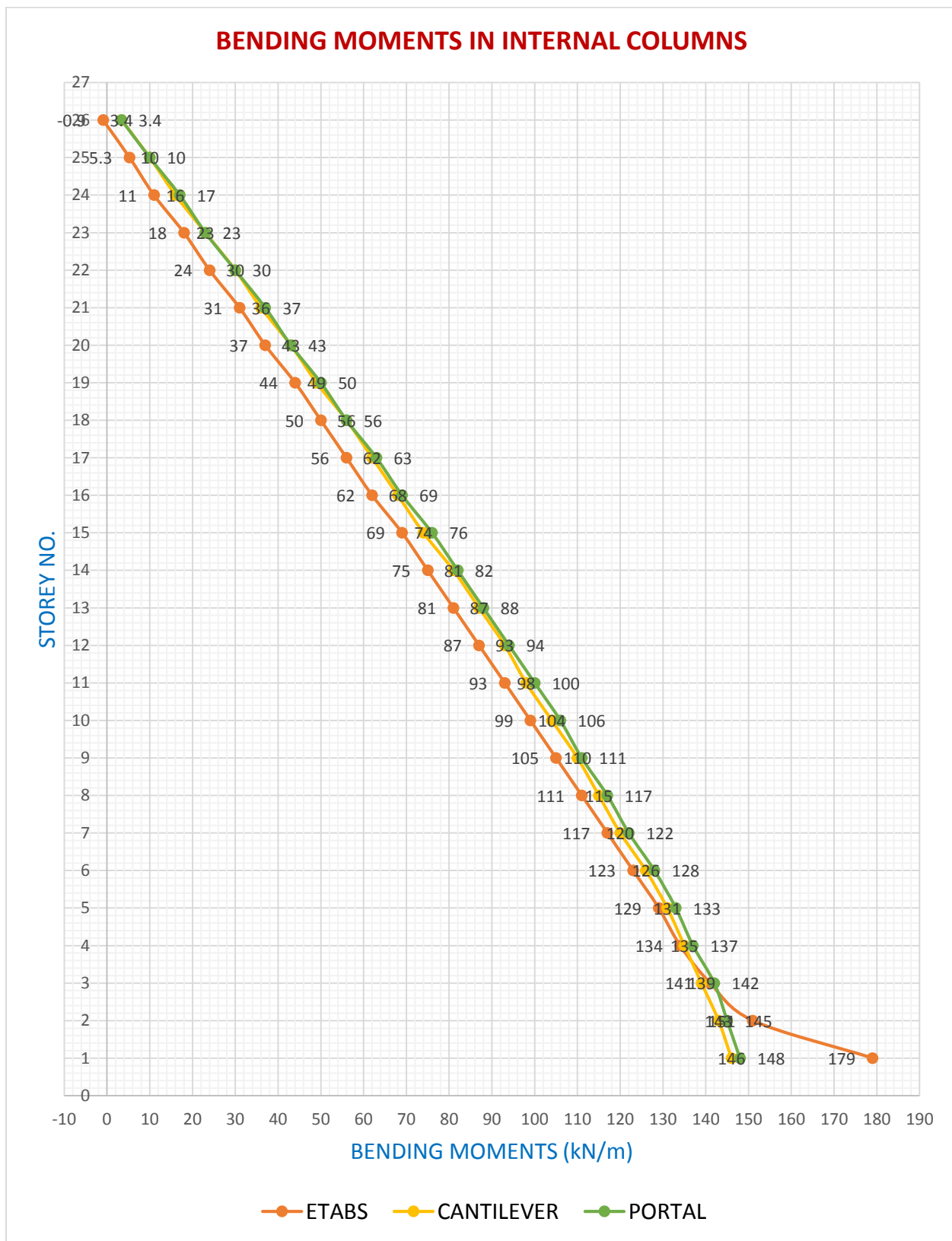


Fig. (4.27): Bending Moments in Internal Columns duo to Wind Loads on 25 Storey

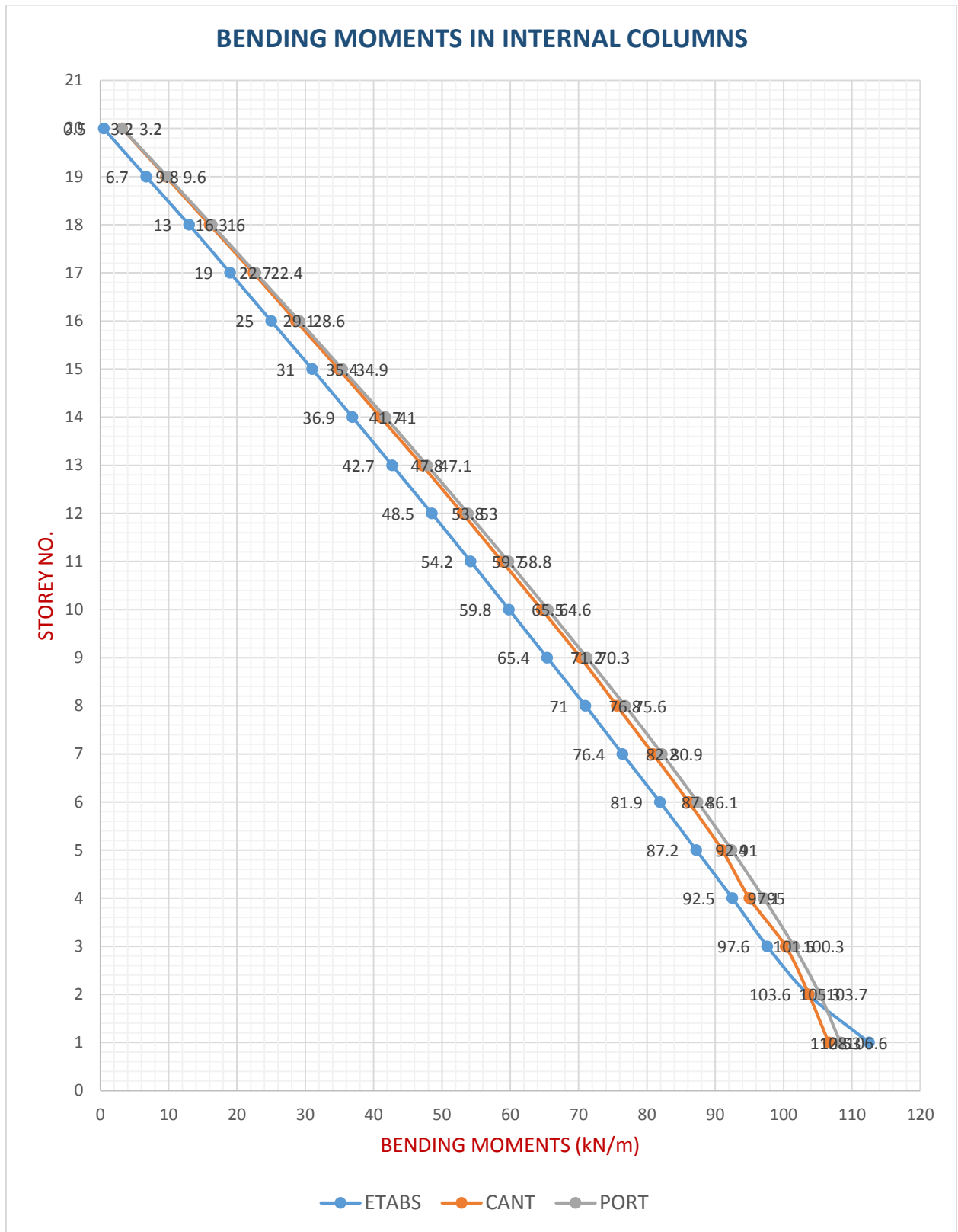


Fig. (4.28): Bending Moments in Internal Columns due to Wind Loads on 20 Storey



Fig. (4.29): Bending Moments in Internal Columns duo to Wind Loads on 15 Storey

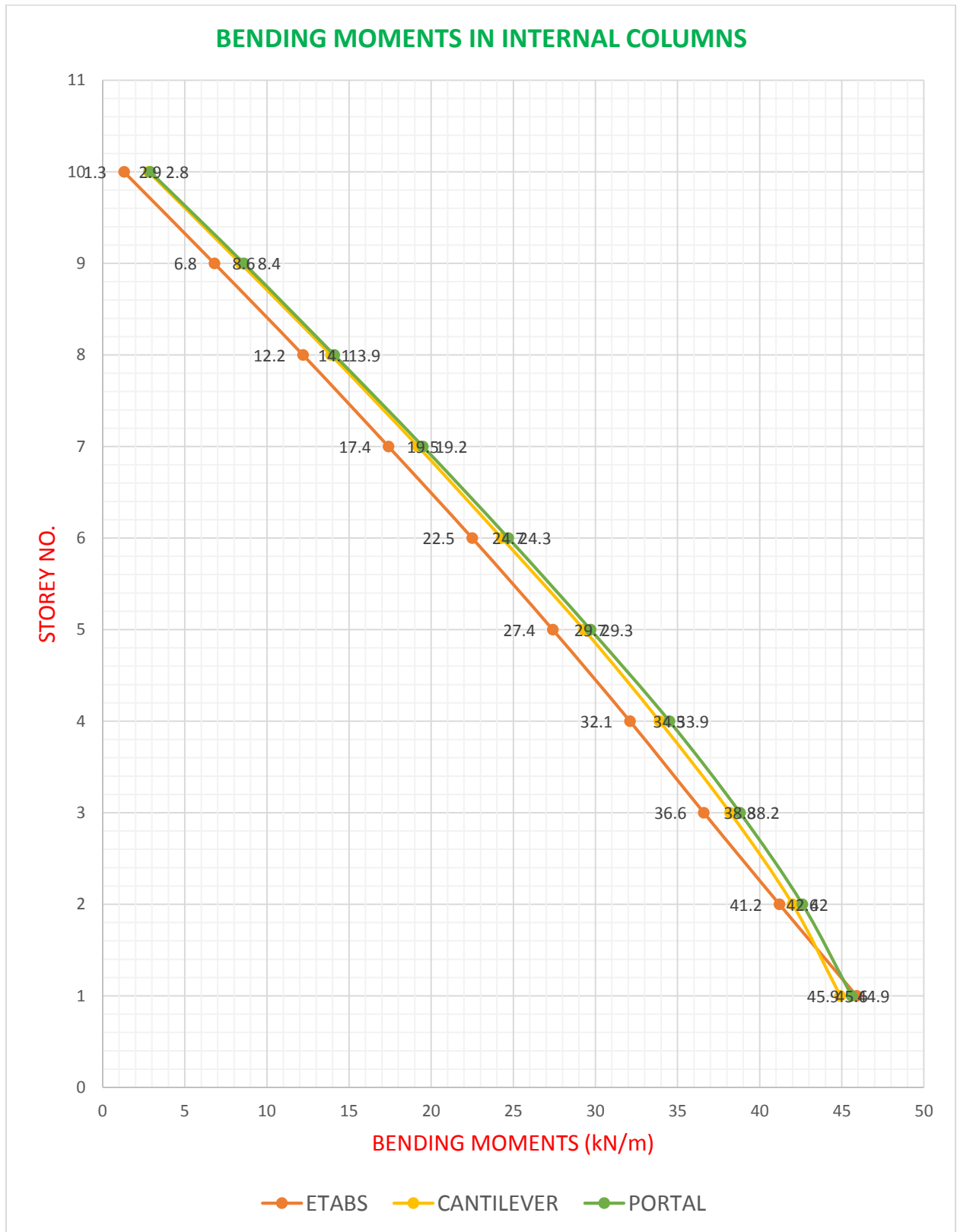


Fig. (4.30): Bending Moments in Internal Columns duo to Wind Loads on 10 Storey

4.7.4 Discussion of Shear Forces Results in Beams:

Comparisons of shear forces results in beams between simplified methods and FEM were presented and compared as shown in Figures (4.31) - (4.38).

Considering the FEM as the reference for comparison, it is clear that results of the simplified methods are approximately accurate results, and can be used to estimate the shear forces on columns for stories.

- 1- Analysis results of the 25 storeys were shown in Fig. (4.31), it was shown that at level storey No.10, the difference between the shear forces in external beams at the reference point calculated using portal method and FEM was about 19%. The difference between the FEM and cantilever method was about 6%. At storey level No. 15, the difference calculated using portal method and FEM was about 27%.The difference between the FEM and cantilever method was about -4%. At storey level story no. 20, the difference calculated using portal method and FEM was about 37%. The difference between the FEM and cantilever method was about -20%. At storey level No.25, the difference calculated using portal method and FEM was about 190%. The difference between the FEM and cantilever method was about 211%.
- 2- Analysis results of the 25 storeys were shown in Fig. (4.35), it was shown that at level storey No.10, the difference between the shear forces in internal beams at the reference point calculated using portal method and FEM was about 9.8%. The difference between the FEM and cantilever method was about 9%. At storey level No. 15, the difference calculated using portal method and FEM was about 11%.The difference between the FEM and cantilever method was about 7%. At storey level story no. 20, the difference calculated using portal method and FEM was about 14%. The difference between the FEM and cantilever method was about 4%. At storey level No.25, the difference calculated using portal method

and FEM was about 60%. The difference between the FEM and cantilever method was about -51%.

- 3- Analysis results of the 20 storeys were shown in Fig. (4.32), it was shown that at level storey No.10, the difference between the shear forces in external beams at the reference point calculated using portal method and FEM was about 22%. The difference between the FEM and cantilever method was about 2.5%. At storey level No. 15, the difference calculated using portal method and FEM was about 28%.The difference between the FEM and cantilever method was about -11%. At storey level story no. 20, the difference calculated using portal method and FEM was about 441%. The difference between the FEM and cantilever method was about 119%.
- 4- Analysis results of the 20 storeys were shown in Fig. (4.36), it was shown that at level storey No.10, the difference between the shear forces in internal beams at the reference point calculated using portal method and FEM was about 10%. The difference between the FEM and cantilever method was about 7%. At storey level No. 15, the difference calculated using portal method and FEM was about 18%.The difference between the FEM and cantilever method was about 4%. At storey level story no. 20, the difference calculated using portal method and FEM was about 70%. The difference between the FEM and cantilever method was about -185%.
- 5- Analysis results of the 15 storeys were shown in Fig. (4.33), it was shown that at level storey No.10, the difference between the shear forces in external beams at the reference point calculated using portal method and FEM was about 25%. The difference between the FEM and cantilever method was about -1%. At storey level No. 15, the difference calculated using portal method and FEM was about 350%.The difference between the FEM and cantilever method was about 126%.
- 6- Analysis results of the 15 storeys were shown in Fig. (4.37), it was shown that at level storey No.10, the difference between the shear forces in internal beams at

the reference point calculated using portal method and FEM was about 10%. The difference between the FEM and cantilever method was about 6%. At storey level No. 15, the difference calculated using portal method and FEM was about 66%. The difference between the FEM and cantilever method was about 61%.

- 7- Analysis results of the 10 storeys were shown in Fig. (4.34), it was shown that at level storey No.5, the difference between the shear forces in external beams at the reference point calculated using portal method and FEM was about 11%. The difference between the FEM and cantilever method was about 14%. At storey level No. 10, the difference calculated using portal method and FEM was about 260%. The difference between the FEM and cantilever method was about 150%.
- 8- Analysis results of the 10 storeys were shown in Fig. (4.38), it was shown that at level storey No.5, the difference between the shear forces in internal beams at the reference point calculated using portal method and FEM was about 6%. The difference between the FEM and cantilever method was about 12%. At storey level No. 10, the difference calculated using portal method and FEM was about 64%. The difference between the FEM and cantilever method was about -100%.

Some differences above may be due to the values of shear forces were small which induce large percentages

The Cantilever Method appears too slightly under-estimate the shear forces on the girders at all levels for the top story's, but appears to over-estimate for the bottom stories.

The Portal method gives over-estimated shear forces results on the girders at all levels for all heights.

As the building height increases, the Cantilever method appears to be giving more accurate results, and hence can be used to estimate the shear forces for tall buildings for preliminary design.

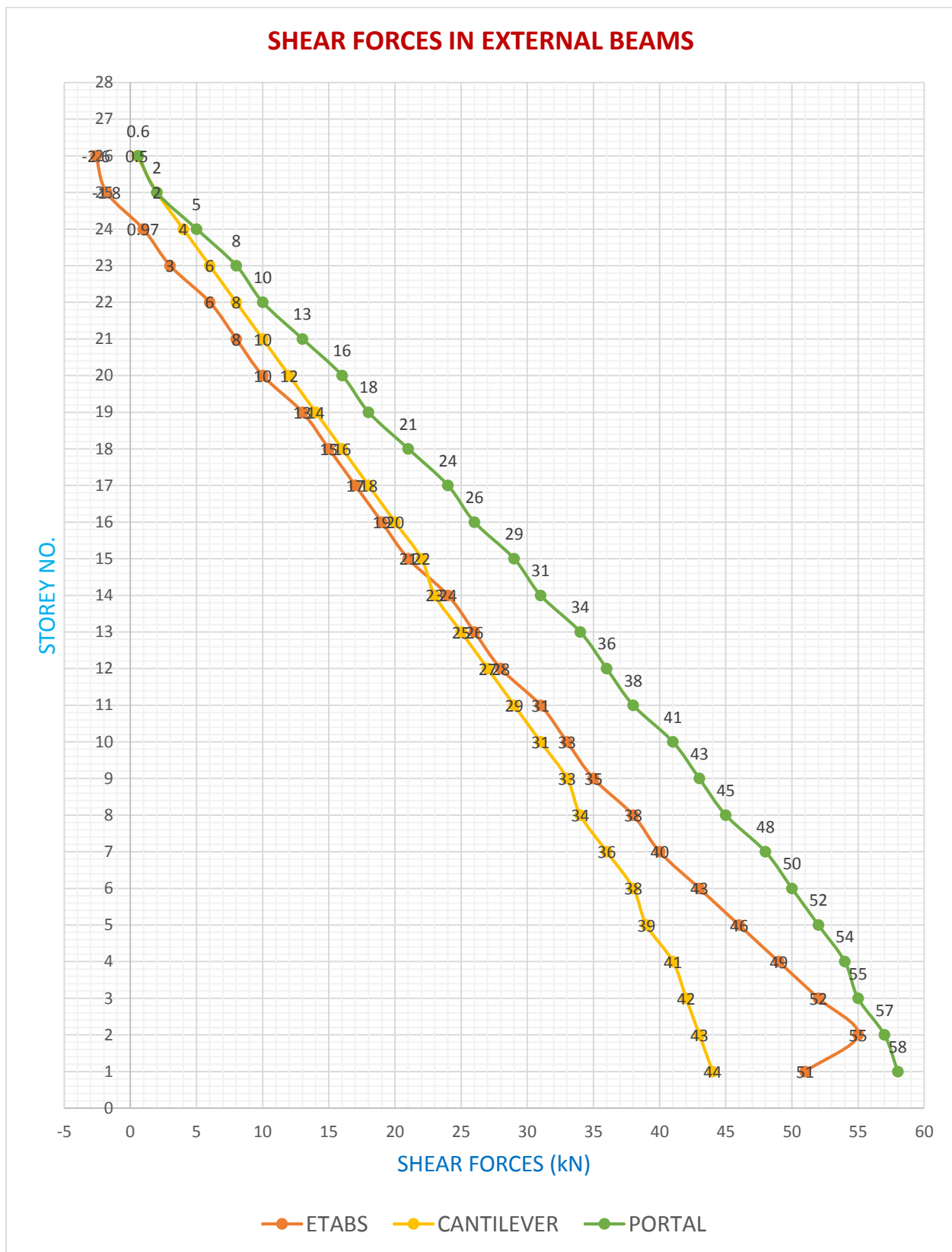


Fig. (4.31): Shear Forces in External Beams duo to Wind Loads on 25Story



Fig (4.32): Shear Forces in External Beams duo to Wind Loads on 20Storey

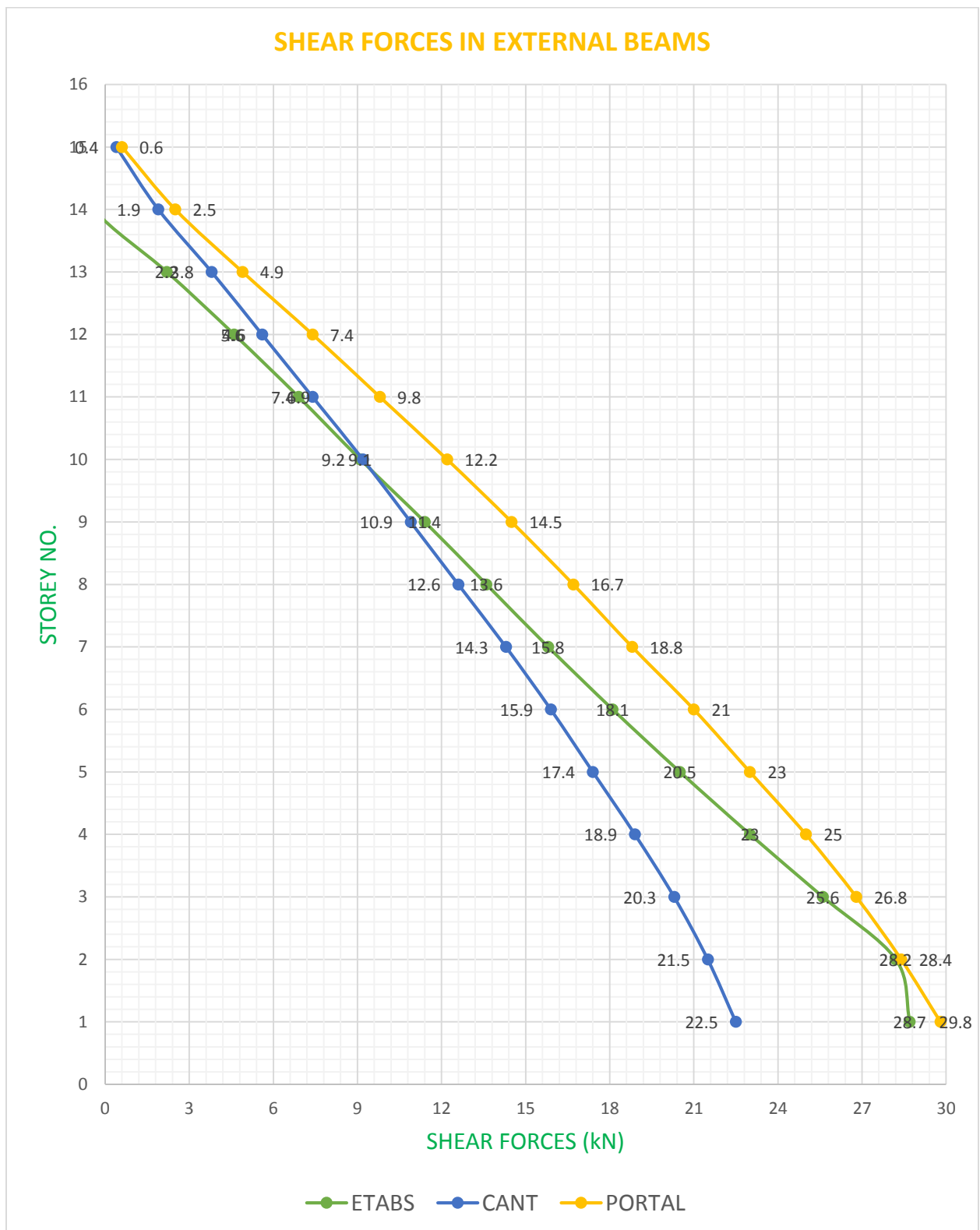


Fig. (4.33): Shear Forces in External Beams due to Wind Loads on 15Storey

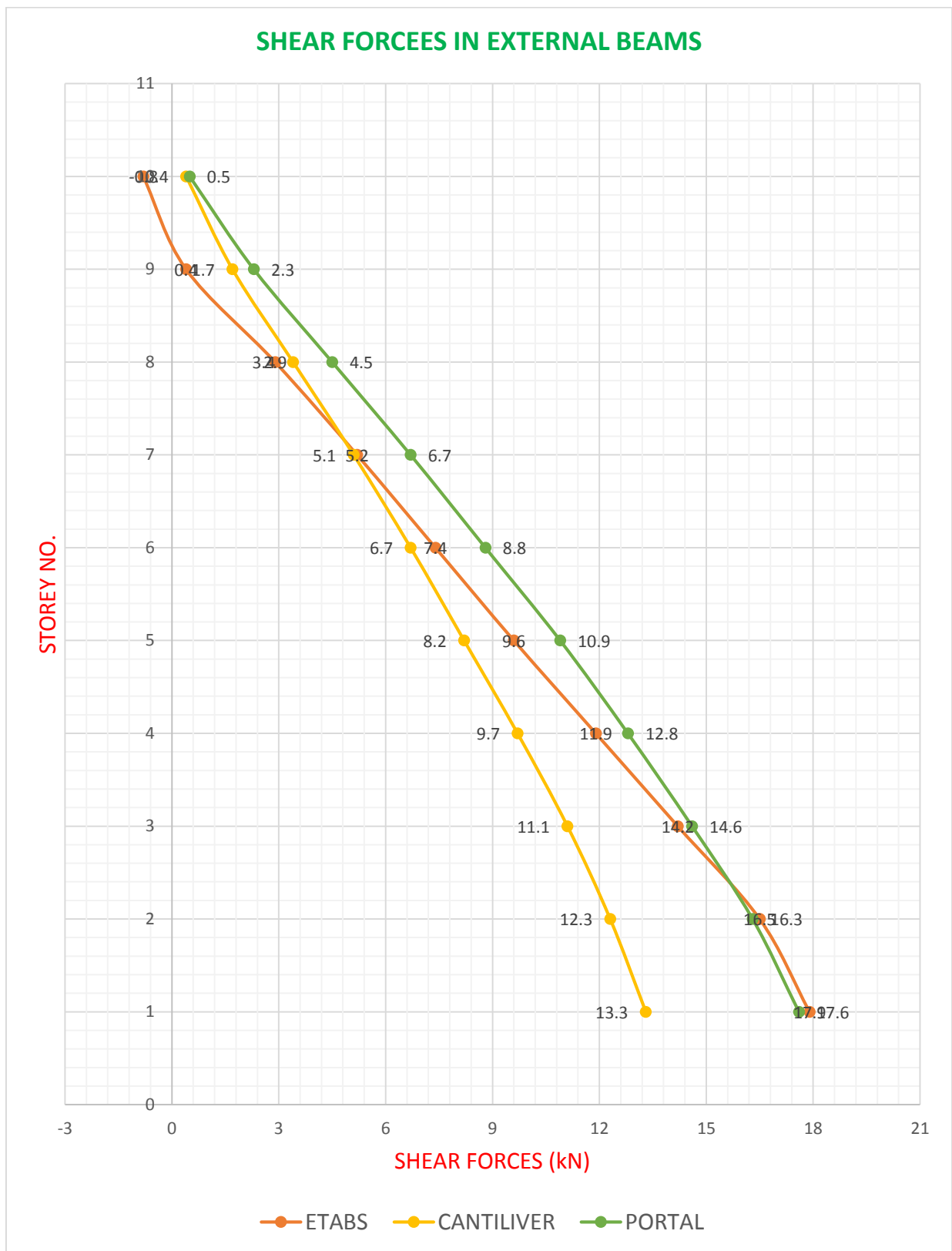


Fig (4.34): Shear Forces in External Beams due to Wind Loads on 10Storey

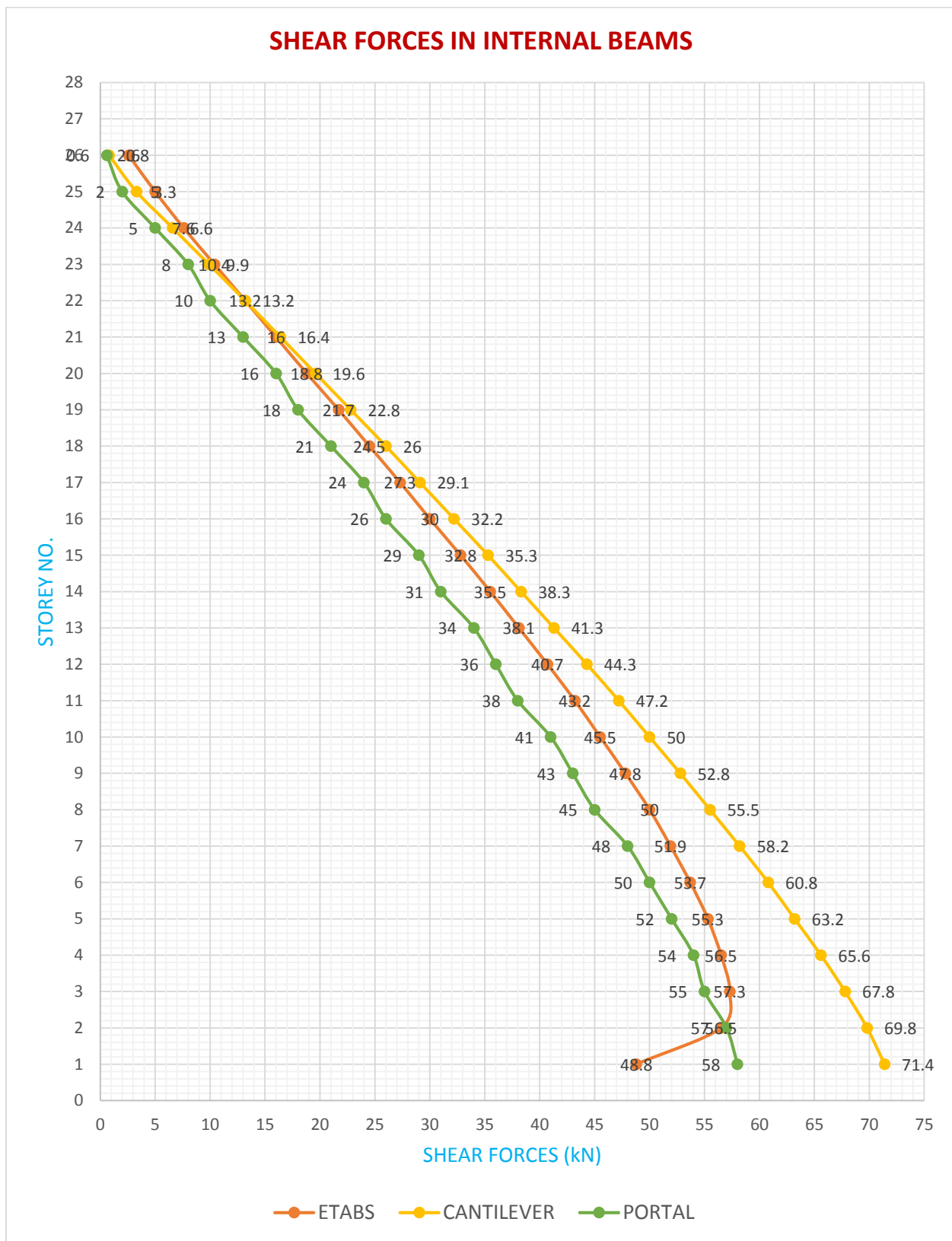


Fig (4.35): Shear Forces in Internal Beams due to Wind Loads on 25Storey

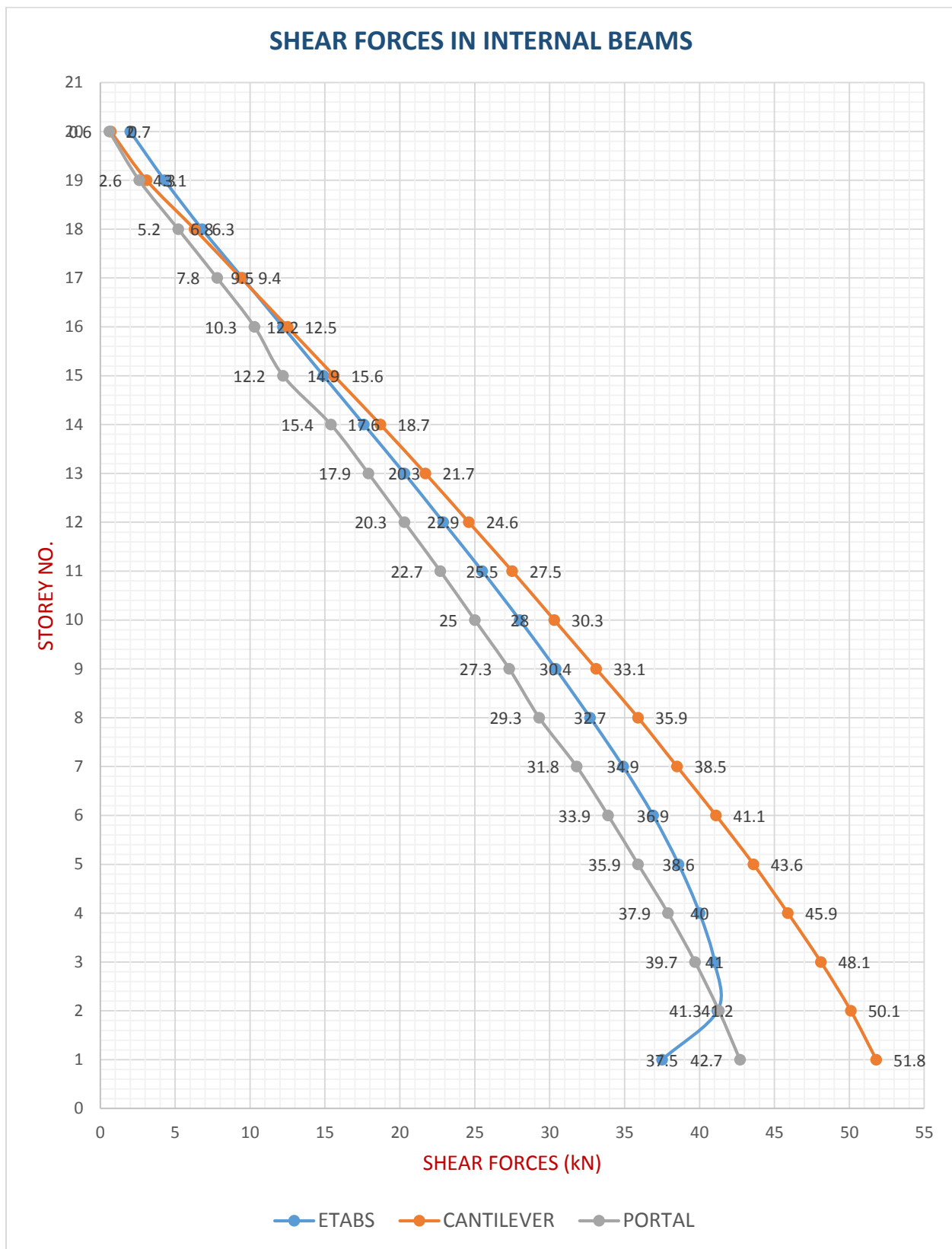


Fig. (4.36): Shear Forces in Internal Beams duo to Wind Loads on 20Storey

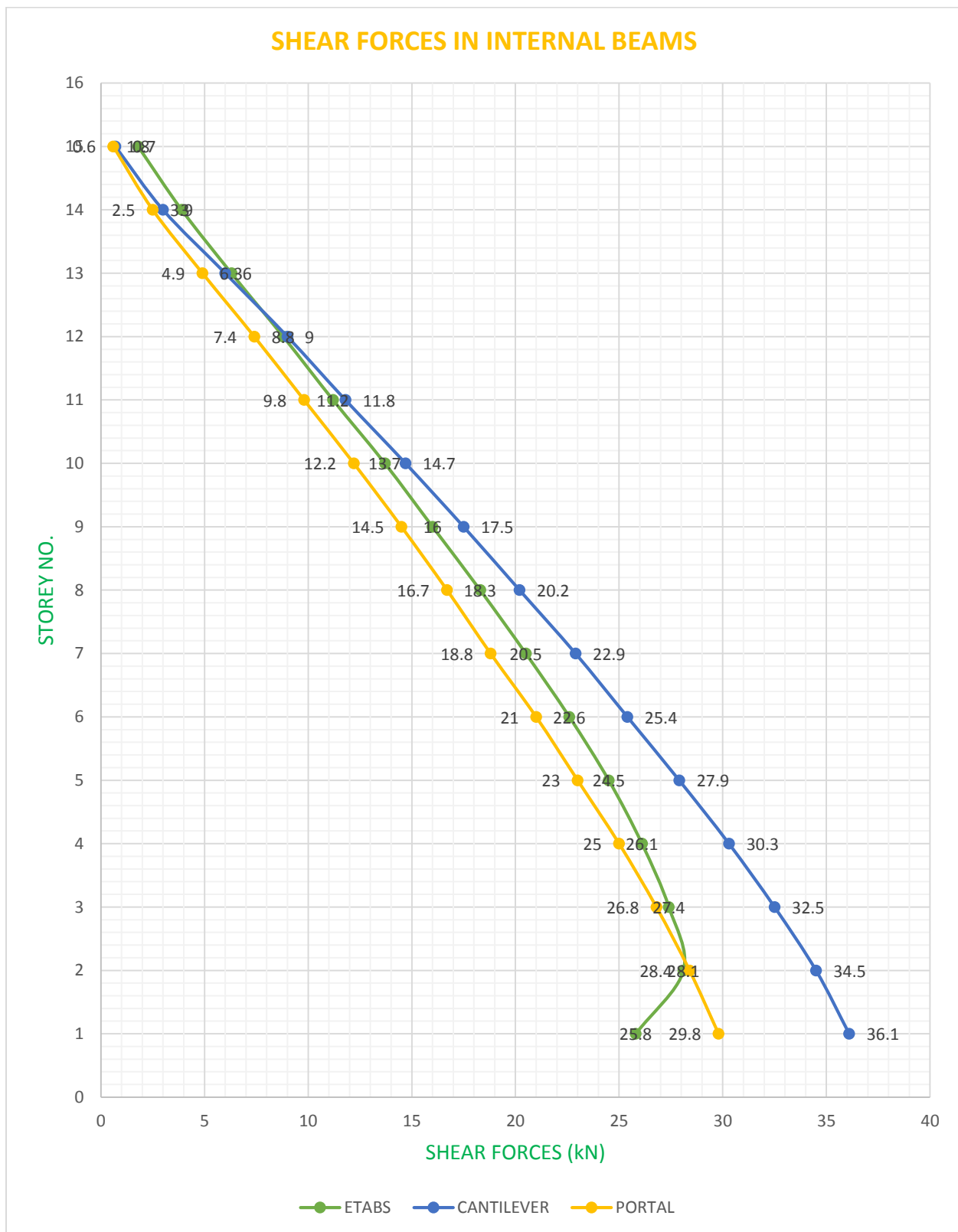


Fig (4.37): Shear Forces in Internal Beams duo to Wind Loads on 15Storey

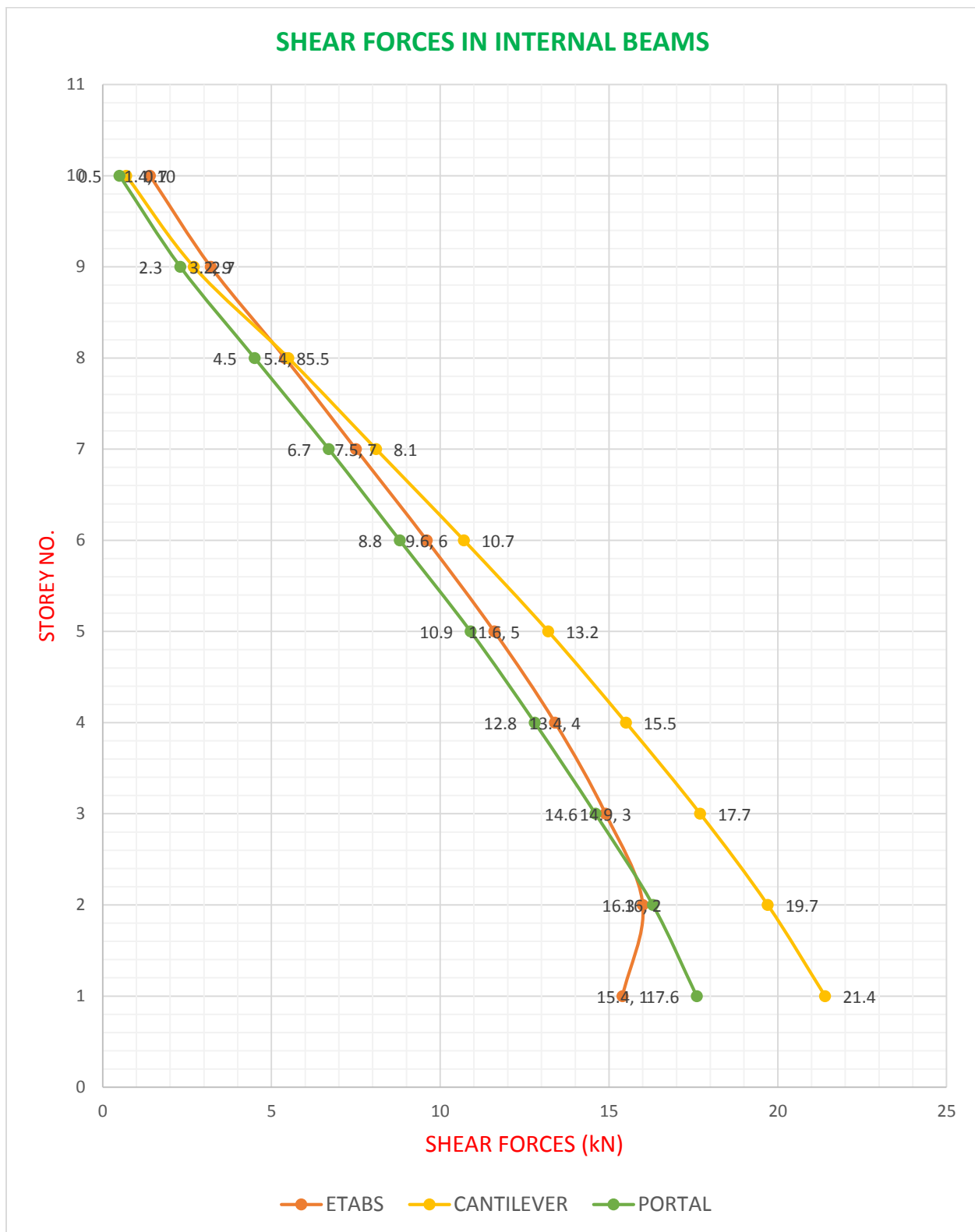


Fig. (4.38): Shear Forces in Internal Beams due to Wind Loads on 10Storey

4.7.4 Discussion of Bending Moments Results in Beams:

Comparisons of bending moment's results in beams between simplified methods and FEM were presented and compared as shown in Figures (4.39).

Considering the FEM as the reference for comparison, it is clear that results of the simplified methods are approximately accurate results, and can be used to estimate the bending moments on beams for stories.

- 1- Analysis results of the 25 storeys were shown in Fig. (4.39), it was shown that at level storey No.10, the difference between the bending moments in external beams at the reference point calculated using portal method and FEM was about 6%. The difference between the FEM and cantilever method was about 19%. At storey level No. 15, the difference calculated using portal method and FEM was about 8%.The difference between the FEM and cantilever method was about - 17%. At storey level story no. 20, the difference calculated using portal method and FEM was about 11%. The difference between the FEM and cantilever method was about 14%. At storey level No.25, the difference calculated using portal method and FEM was about 99%. The difference between the FEM and cantilever method was about 99%.
- 2- Analysis results of the 25 storeys were shown in Fig. (4.39), it was shown that at level storey No.10, the difference between the bending moments in internal beams at the reference point calculated using portal method and FEM was about 18%. The difference between the FEM and cantilever method was about 1.7%. At storey level No. 15, the difference calculated using portal method and FEM was about 20%.The difference between the FEM and cantilever method was about 3%. At storey level story no. 20, the difference calculated using portal method and FEM was about 21%. The difference between the FEM and cantilever method was about 5%. At storey level No.25, the difference calculated

using portal method and FEM was about 49%. The difference between the FEM and cantilever method was about 38%.

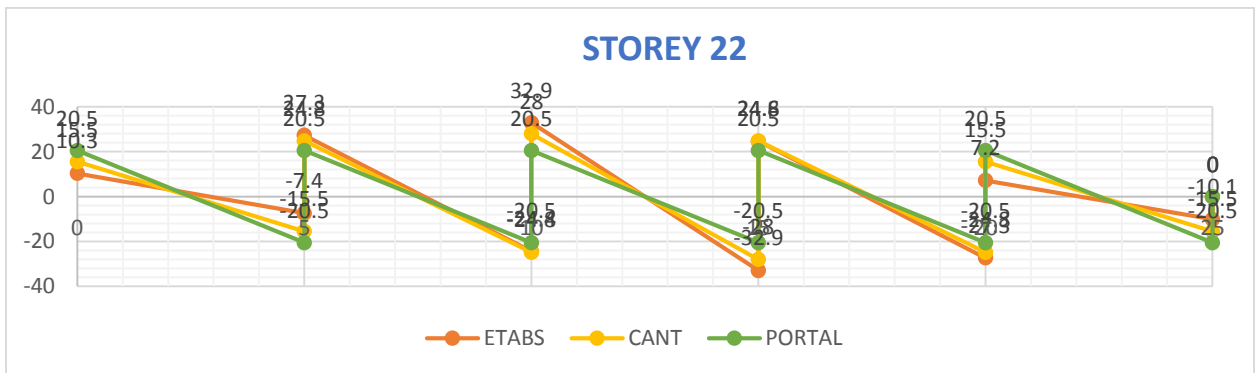
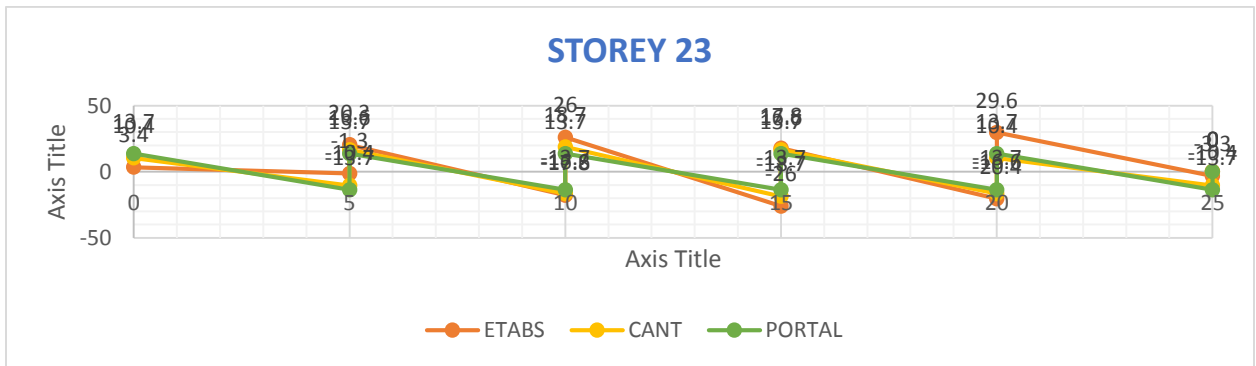
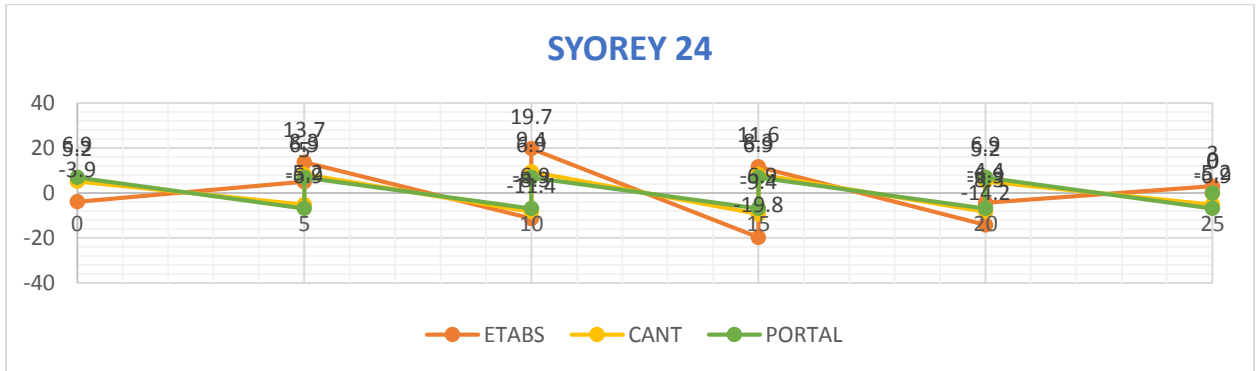
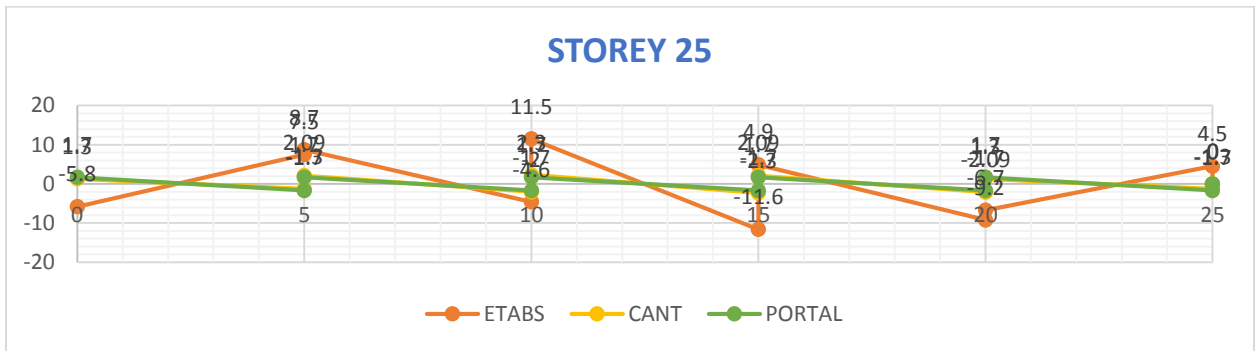
Some differences above may be due to the values of bending moments were small which induce large percentages.

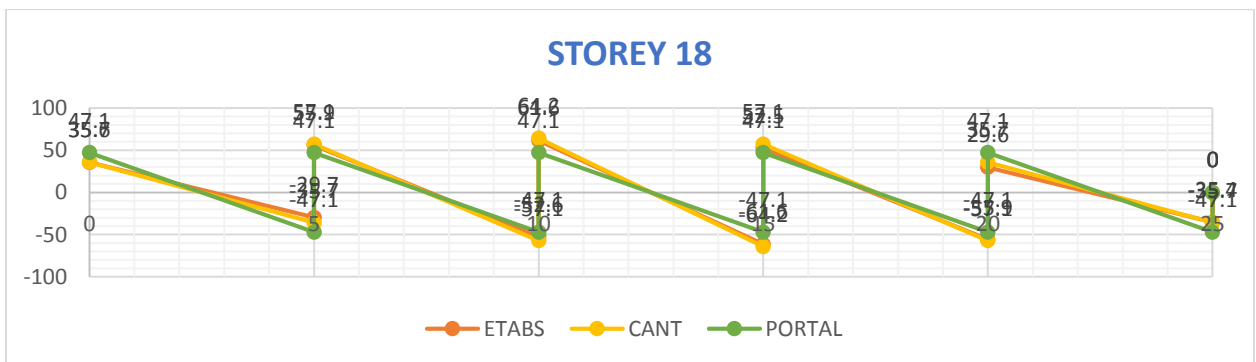
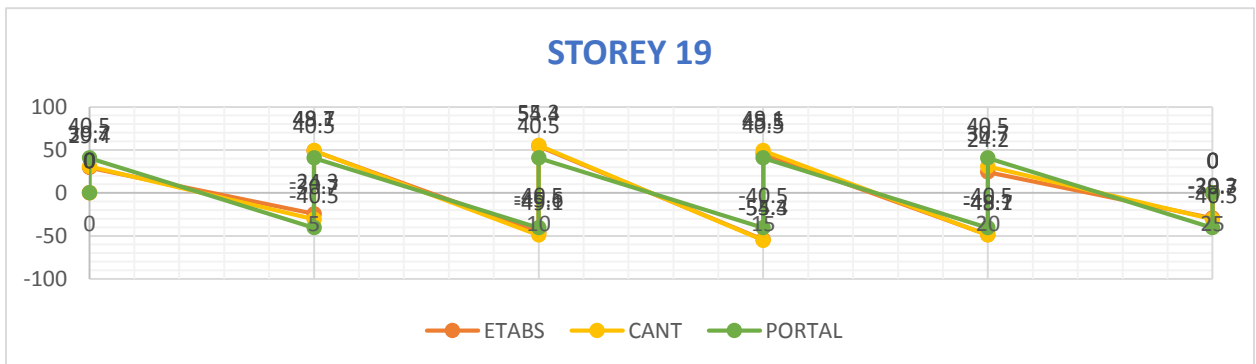
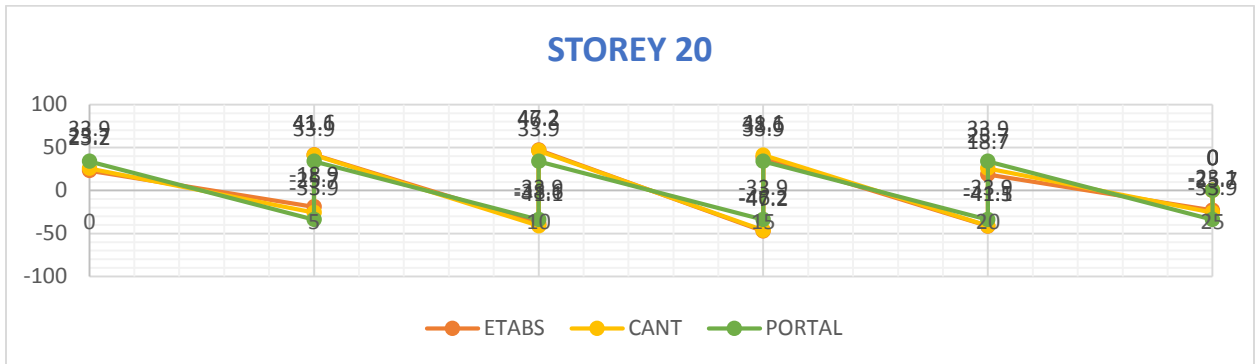
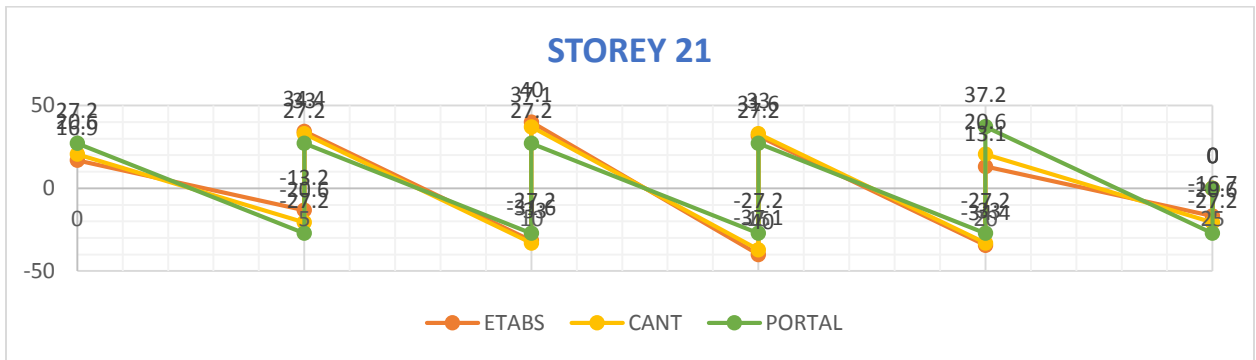
The cantilever method appears to slightly under-estimate the bending moments on the girders at all levels.

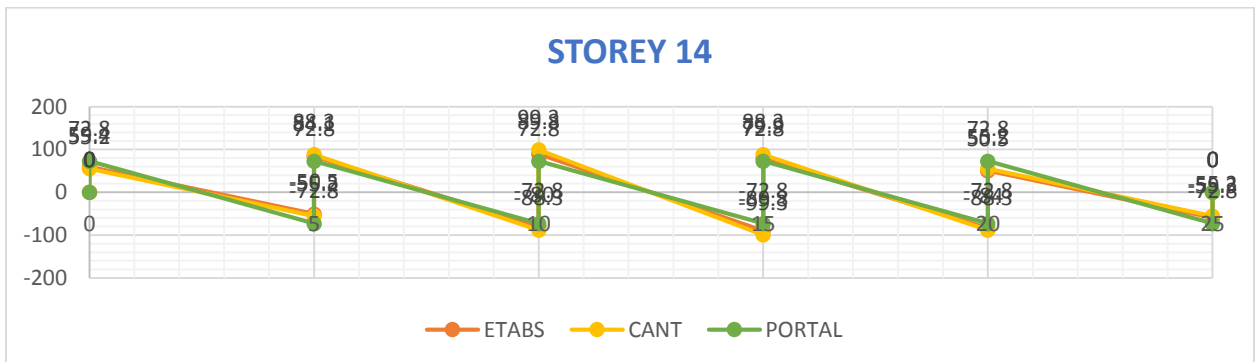
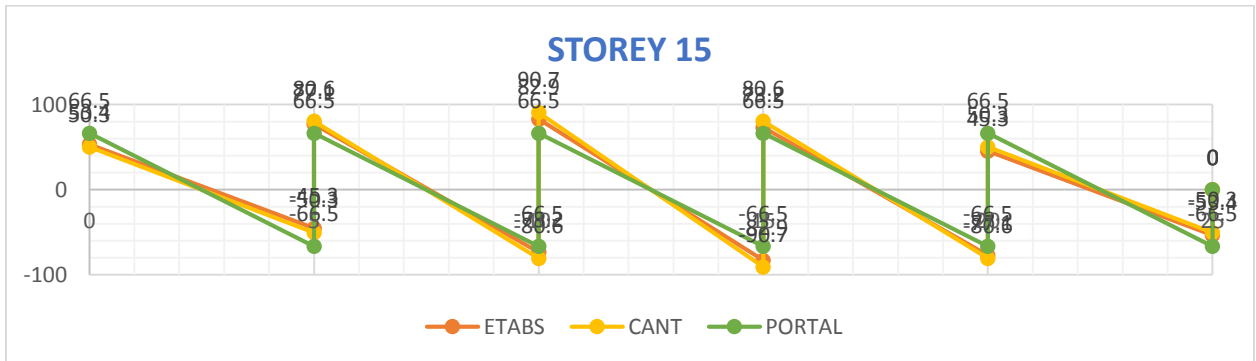
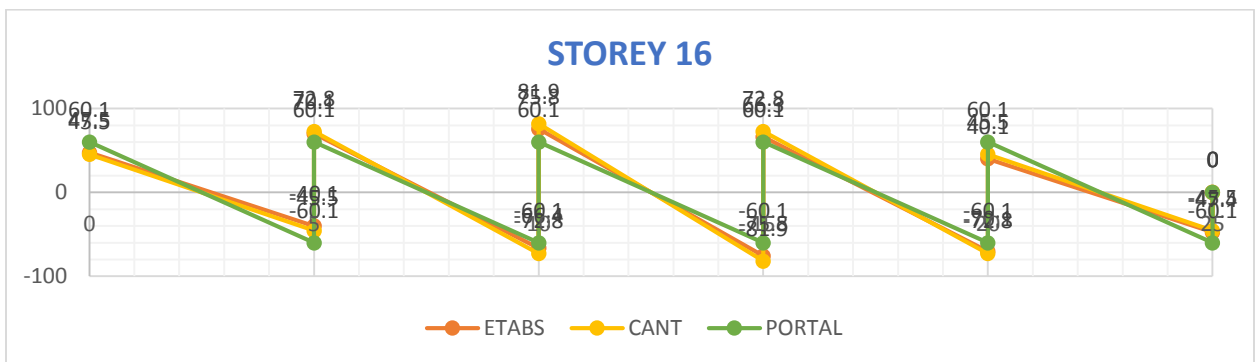
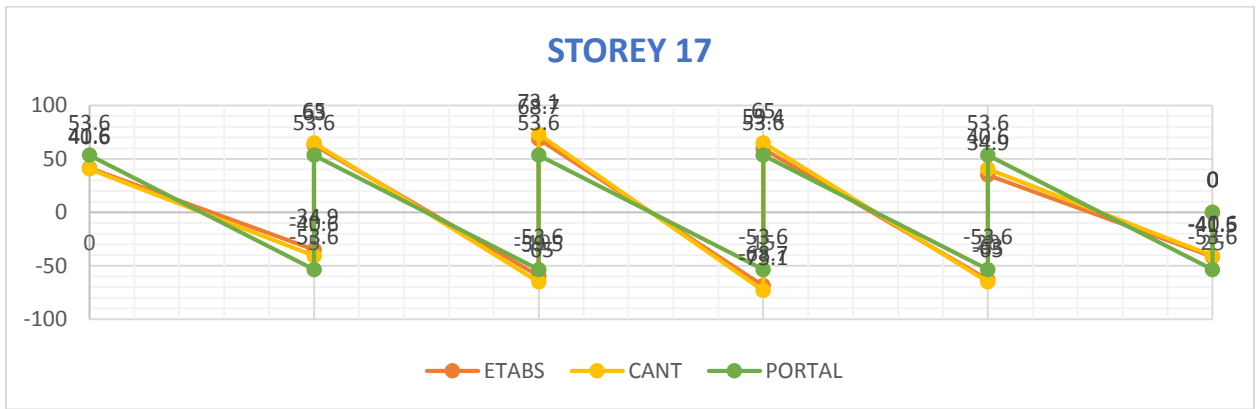
The portal method appears to slightly under estimate the girder moments at middle bays and over-estimates them at the both end bays for the all stories.

The actual bending moment's diagrams appear to be completely different from the diagrams estimated by both manual methods for the top two stories. This was justified by the shear deflection mode of the rigid frame, which tends to reverse the girder moments at the top stories, and this wasn't taken into account in the approximate methods.

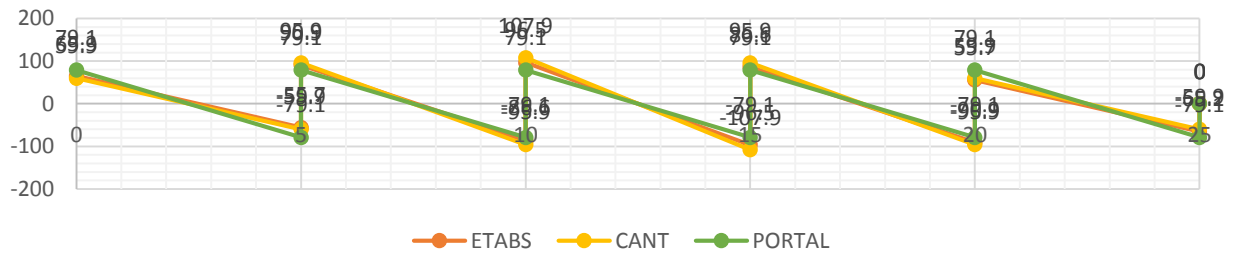
As the building height increases, the Cantilever method appears to be giving more accurate results, and hence can be used to estimate the bending moments for tall buildings for preliminary design.



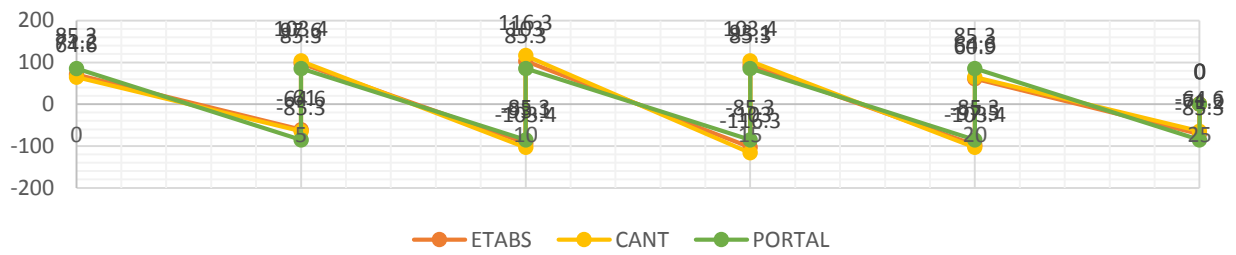




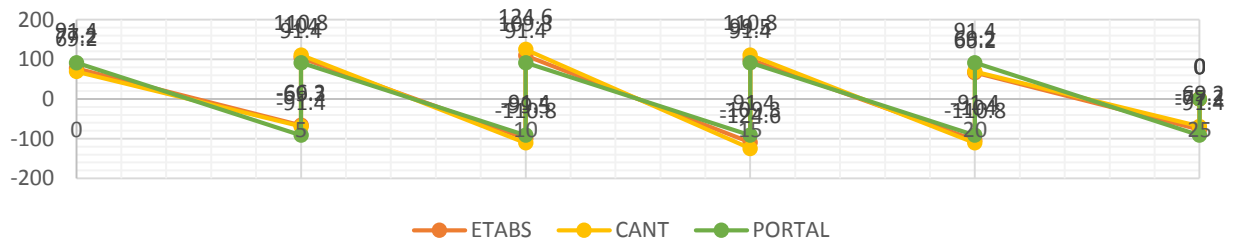
STOREY 13



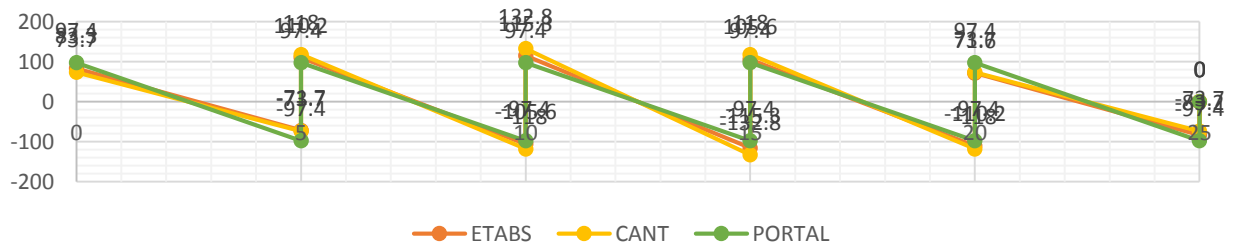
SYOREY 12

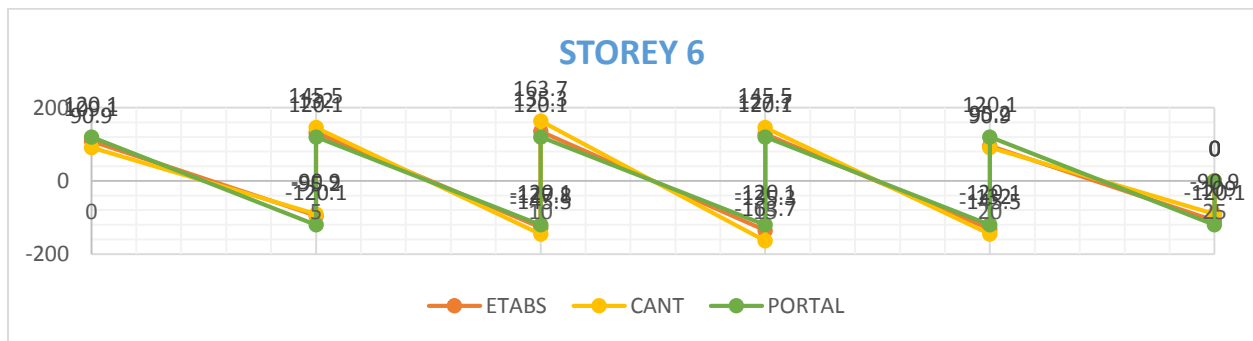
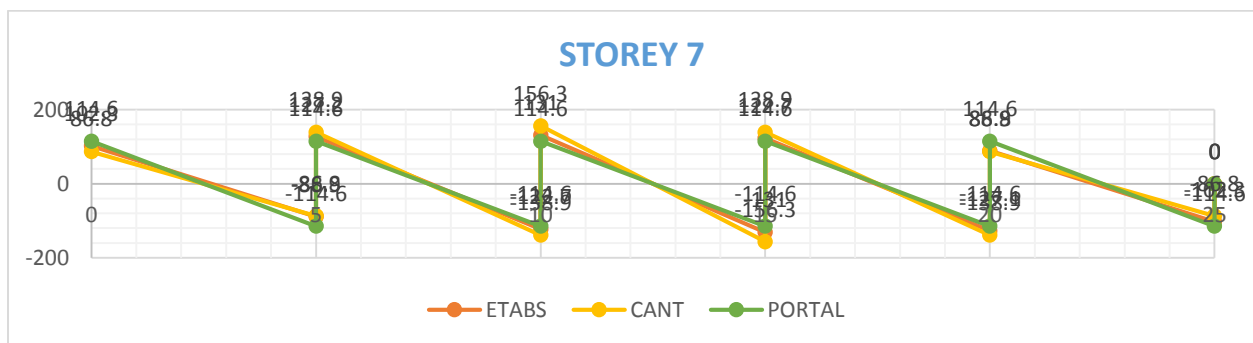
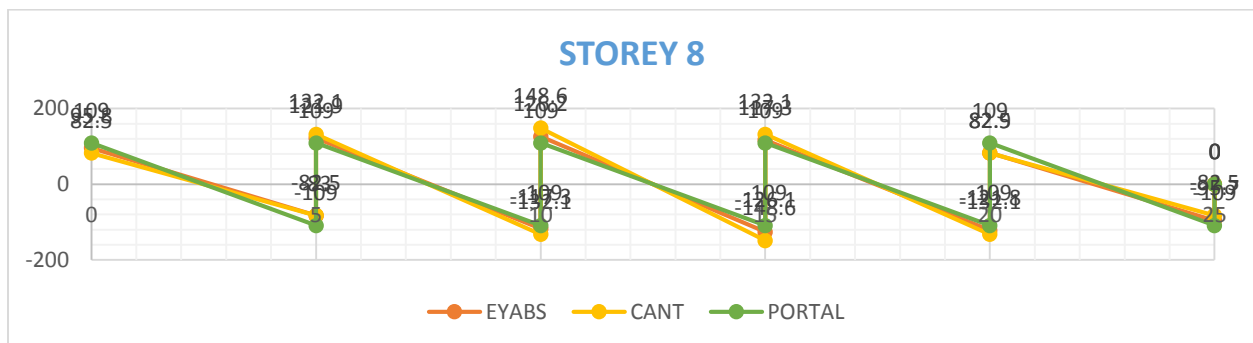
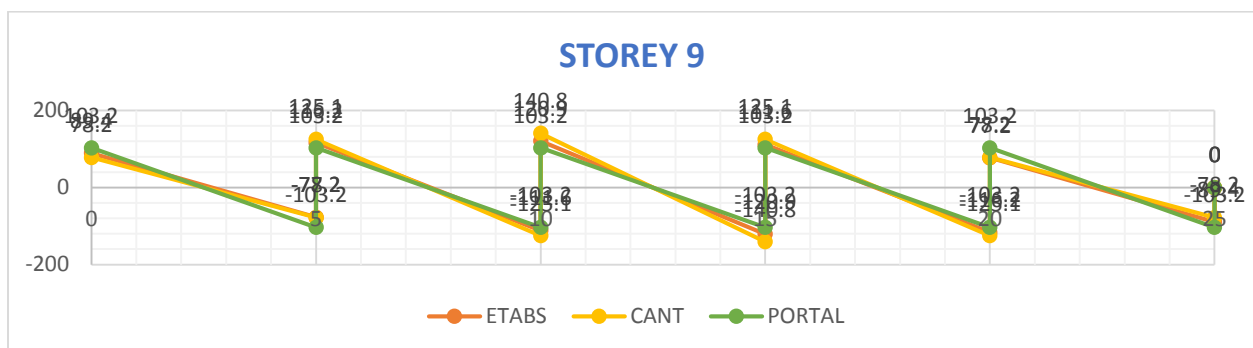


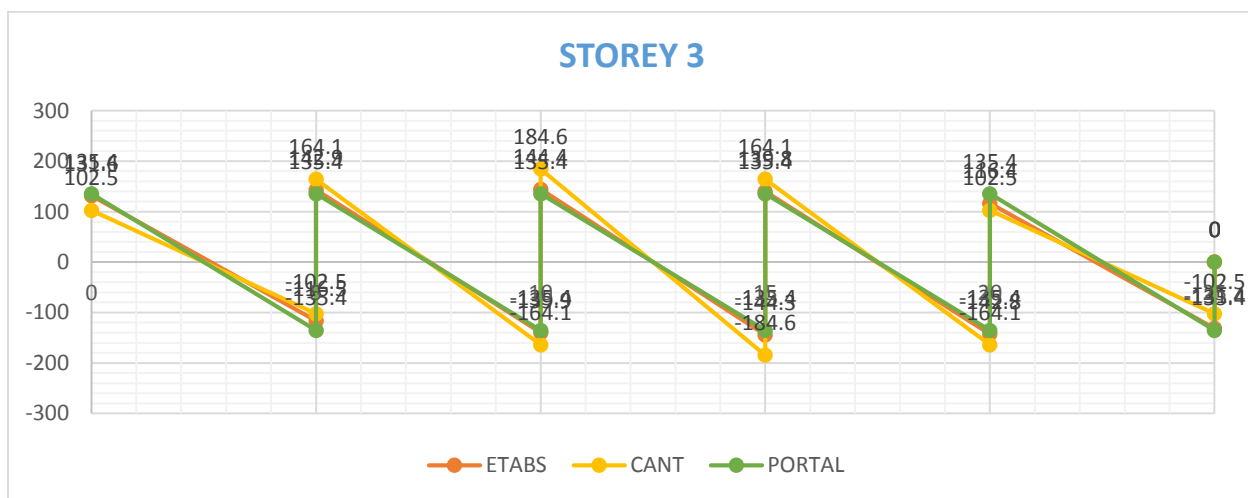
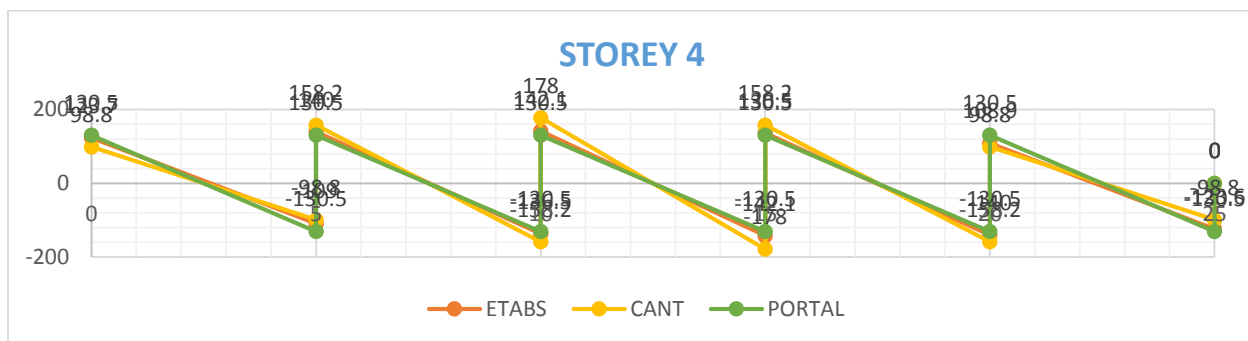
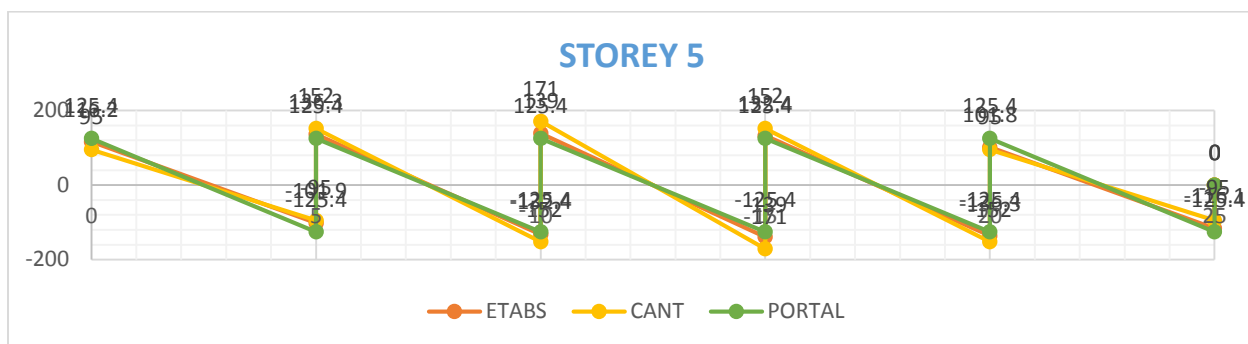
STOREY 11



STOREY 10







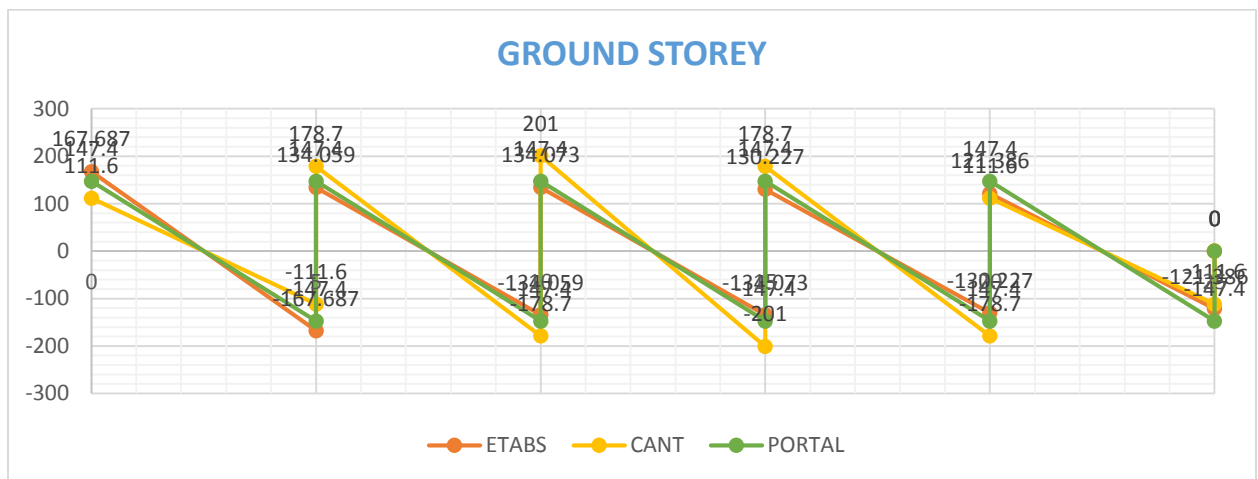
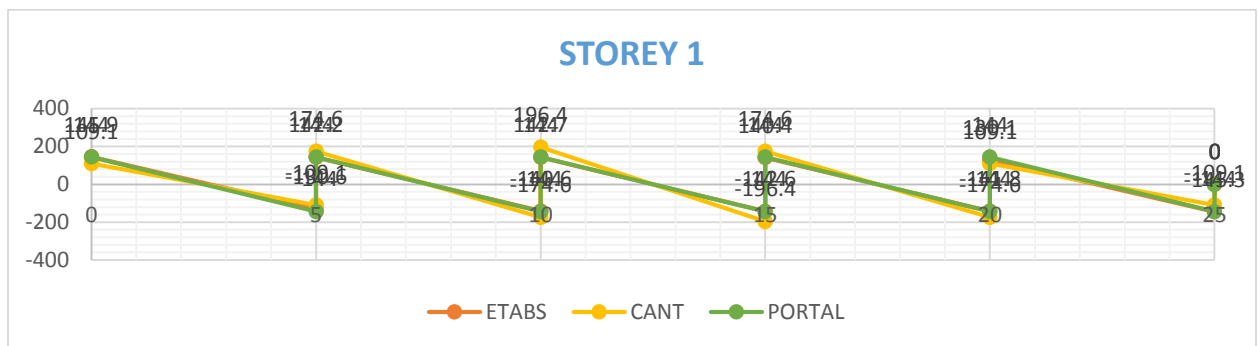
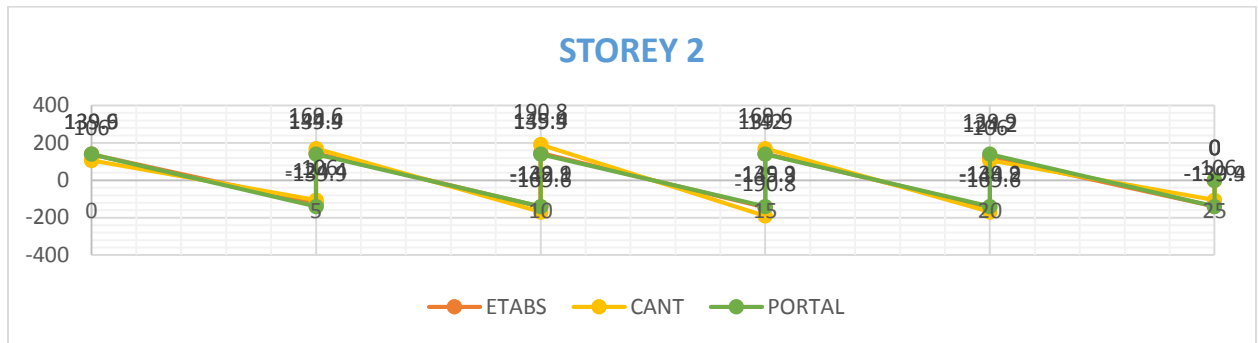


Fig. (4.39) Bending Moments in Beams due to Wind Loads on 25Storey

Chapter Five

Conclusions and Recommendations

5.1 Conclusions

Tall buildings are structures which are much affected by lateral forces due to increased ratio of height to horizontal dimension (slenderness).

1. The differences between portal and finite element method were due to assumption of portal method where the axial deformation ignored.
2. The differences between cantilever and finite element method were due to assumption of cantilever method where the axial deformation ignored.
3. The portal and cantilever methods are considered to be appropriate for the preliminary analysis of reasonably tall building frames.
4. The finite element method is more accurate than the portal and cantilever methods but it is not economical for everyday design works.
5. The portal and cantilever program methods using Excel sheet program are fast and give acceptable results within certain limits.
6. The axial forces in columns, considering the FEM as the reference for comparison, in an over-all view, the cantilever method appears to give approximately accurate results, slightly under-estimated, for the top storeys for all heights, and slightly over-estimated at bottom storeys for columns for all heights, though it shows little negative difference in the columns, while the portal method appears to give approximately accurate results, slightly over-estimated, for the storeys for all heights in the columns.
7. The shear forces in columns, in an over-all view, the cantilever method give approximately accurate results, slightly over-estimated, for the internal columns for all heights, and slightly under-estimated at bottom storeys for outer columns for all heights, while the portal method gives over-estimated shear force results

in internal columns, and slightly under-estimated at bottom storeys, slightly over-estimated at top storeys for outer columns for all heights

8. The bending moments in columns, in an over-all view, the cantilever method appears to give results approximately similar to the program results in the internal columns, and the top story in external columns, but appears to under-estimate for the bottom storey in the external columns, though it shows little negative difference in the internal columns, while the portal method appears to give results approximately similar to the program results in the internal columns, and the top and middle storey in external columns, but appears under-estimate bending moments for the bottom storey in the external columns
9. The shear forces in beams, in an over-all view, the cantilever method appears too slightly under-estimate the shear forces on the girders at all levels for the top storeys, but appears to over-estimate for the bottom storeys, while the portal method gives over-estimated shear forces results on the girders at all levels for all heights.
10. The bending moments of beams, in an over-all view, the cantilever method appears to slightly under-estimate the bending moments on the girders at all levels, while the portal method appears to slightly under estimate the girder moments at middle bays and over-estimates them at the both end bays for the all storeys.
11. When comparing results of two programs, it shows the cantilever method is more close to the results of finite element method but the values of bending moments and shear forces in external columns are closer than portal method.

5.2. Recommendations

Based on findings and conclusions of this study the following recommendations could be formulated:

1. The newly-developed programs for portal and cantilever methods can be extended to include the design of buildings elements.
2. More study is needed for tall building analysis due to gravity load used simplified methods to continue developing program.
3. It is recommendable to perform analysis for other structural forms like shear wall, tube frames, braced frame and compare of the

References

1. Abbas Mahgoub A.rahman Mohammed, 2009 ‘‘***Analysis and design of high-rise buildings to resist cyclic loads (Wind and Earthquakes Loads)***’’, (M.Sc. Thesis). Sudan University of Science and Technology.
2. Alaa Awad Ahmed, 2015, ‘‘***Nonlinear analysis of tall building under seismic load***’’, (M.SC. Thesis). Sudan University of Science and Technology.
3. Bryan Stafford Smith Alex Coull, 1991, ‘‘***Tall Building Structures Analysis and Design***’’, Awiley Interference Publication, United States of America.
4. Bungale S. Taranth, 2010, ‘‘***Reinforced Concrete Design of Tall Building***’’, Taylor & Francis Group, London & New York.
5. Bungale S.Taranth, 2005, ‘‘***Wind Load and Earthquake Resistant Building***’’, Marcelld, New York.
6. BS 8110-1:1997, Structural use of Concrete –Part 1: Code of practice for design and construction, British Standard Institution, London.
7. Bhavikatti, S. S, 2010, ‘‘***Finite Element Analysis***’’, New Age International (p) Pvt Ltd, Delhi.
8. ETABS (Computers & Structures, Inc, 1995).
9. Hamza Ismail Kidder, 2011, ‘‘***The Analysis and Design of High Rise Buildings for the Effects of Winds and Earthquakes***’’, (M.Sc. Thesis). Karary University.
- 10.Loannis Kourakis, 2007, ‘‘***Structural System and Tuned Mass Dampers of super-Tall Building case Study of Taipei***’’, Master of Engineering Thesis, Massachusetts Institute of technology in Civil and Environmental Engineering,.
- 11.Maha Ibrahim Mohammed Alhassan, 2012, ‘‘***Analysis of tall building due to lateral load used approximate method and finite element method used SAP 2000***’’, (M.SC. Thesis). Karary University

12. Osman Abdel raouf Ahmed Derar, 2011, ‘‘ *Static Analysis of Wind Effect on Tall Building in Sudan*’’, (M.Sc. Thesis). Karary University.
13. Will Pank, Maunsell, 2002, ‘‘*Tall Buildings and Sustainability Report*’’, Economic Development Office, London.
14. W.H. Mosley, J.H Bungey & R. Hulse, 1999, ‘‘ *Reinforced concrete Design*’’, Fifth Edition, PALGRAVE, Great Britain.

Appendix A

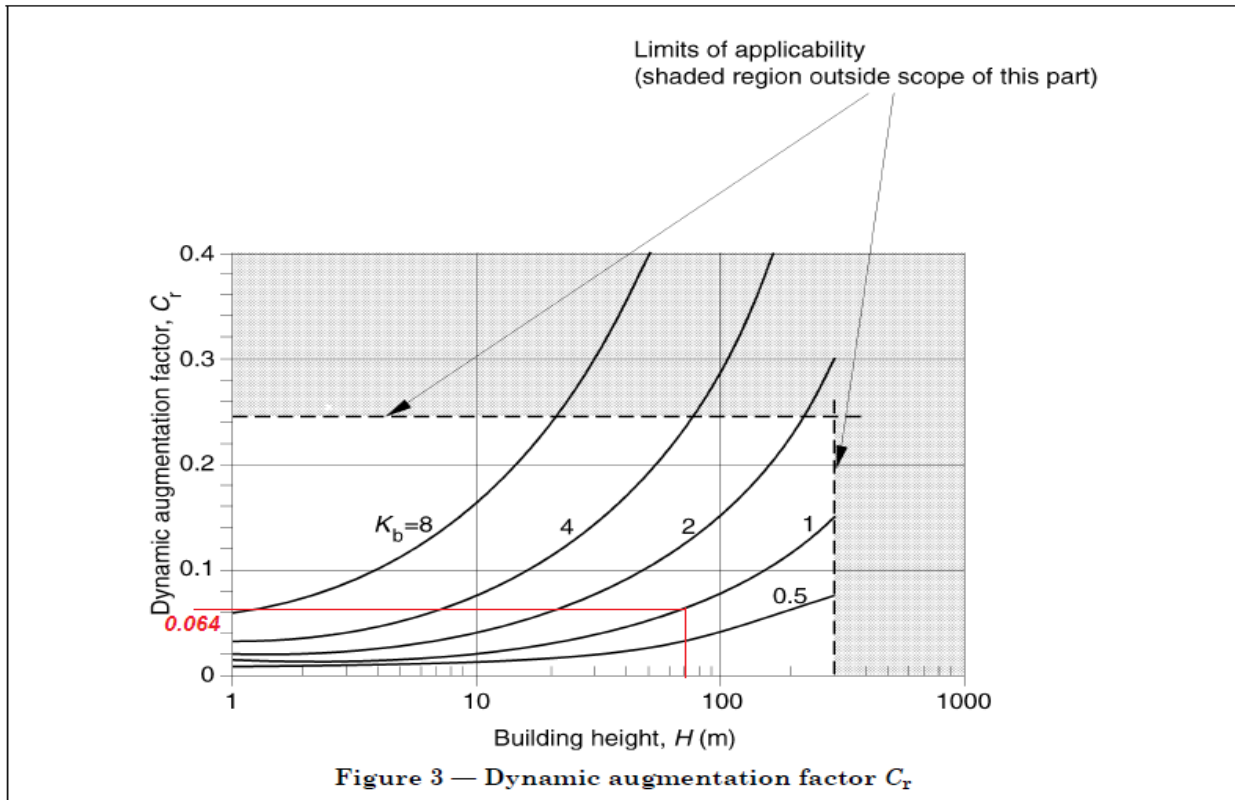
(A.1): Building-type factor K_b

Table 1 — Building-type factor K_b

Type of building	K_b
Welded steel unclad frames	8
Bolted steel and reinforced concrete unclad frames	4
Portal sheds and similar light structures with few internal walls	2
Framed buildings with structural walls around lifts and stairs only (e.g. office buildings of open plan or with partitioning)	1
Framed buildings with structural walls around lifts and stairs with additional masonry subdivision walls (e.g. apartment buildings), buildings of masonry construction and timber-framed housing	0.5

NOTE The values of the factors K_b and C_r have been derived for typical building structures with typical frequency and damping characteristics, under typical UK wind speeds, without accounting for topography or terrain roughness effects. More accurate values of these factors may be derived using Annex C when the building characteristics are not typical, or when the effects of topography and terrain roughness need to be taken into account.

(A.2): DYNAMIC AUGMENTATION FACTOR C_r



(A.3): DIRECTION FACTOR S_d

Table 3 — Values of direction factor S_d

Direction φ	Direction factor S_d
0° North	0.78
30°	0.73
60°	0.73
90° East	0.74
120°	0.73
150°	0.80
180° South	0.85
210°	0.93
240°	1.00
270° West	0.99
300°	0.91
330°	0.82
360° North	0.78
NOTE Interpolation may be used within this table.	

(A.4): FACTOR S_b

Table 4 — Factor S_b for standard method

Site in country or up to 2 km into town					Site in town, extending ≥ 2 km upwind from the site			
Effective height H_e	Closest distance to sea upwind km				Effective height H_e	Closest distance to sea upwind km		
m	≤ 0.1	2	10	≥ 100	m	2	10	≥ 100
≤ 2	1.48	1.40	1.35	1.26	≤ 2	1.18	1.15	1.07
5	1.65	1.62	1.57	1.45	5	1.50	1.45	1.36
10	1.78	1.78	1.73	1.62	10	1.73	1.69	1.58
15	1.85	1.85	1.82	1.71	15	1.85	1.82	1.71
20	1.90	1.90	1.89	1.77	20	1.90	1.89	1.77
30	1.96	1.96	1.96	1.85	30	1.96	1.96	1.85
50	2.04	2.04	2.04	1.95	50	2.04	2.04	1.95
100	2.12	2.12	2.12	2.07	100	2.12	2.12	2.07
NOTE 1 Interpolation may be used within each table.								
NOTE 2 The figures in this table have been derived from reference [5].								
NOTE 3 Values assume a diagonal dimension $a = 5$ m.								
NOTE 4 If $H_e > 100$ m use the directional method of Section 3.								

(A.5): External pressure coefficients C_{pe} for vertical walls

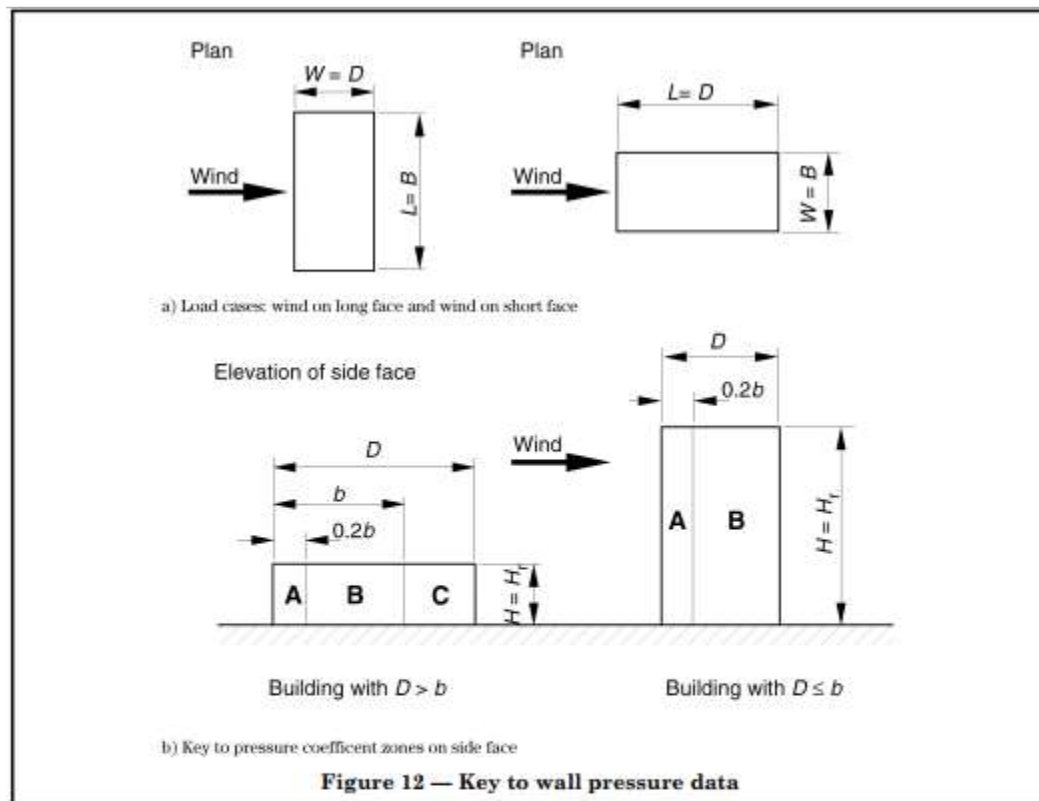


Table 5 — External pressure coefficients C_{pe} for vertical walls

Vertical wall face	Span ratio of building		Vertical wall face		Exposure case	
	$D/H \leq 1$	$D/H \geq 4$			Isolated	Funnelling
Windward (front)	+0.85	+0.6	Side	Zone A	-1.3	-1.6
Leeward (rear)	-0.5	-0.5		Zone B	-0.8	-0.9
				Zone C	-0.5	-0.9

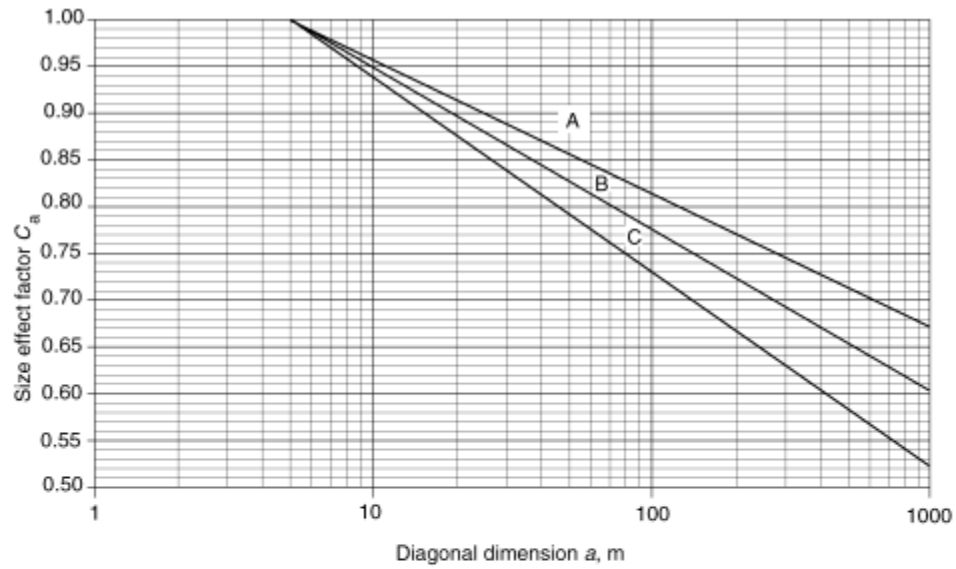
NOTE Interpolation may be used in the range $1 < D/H < 4$. See 2.4.1.4 for interpolation between isolated and funnelling.

(A.6): Internal pressure coefficients C_{pi}

Table 16 — Internal pressure coefficients C_{pi} for enclosed buildings

Type of walls	C_{pi}
Two opposite walls equally permeable; other faces impermeable	
— Wind normal to permeable face	+0.2
— Wind normal to impermeable face	-0.3
Four walls equally permeable; roof impermeable	-0.3

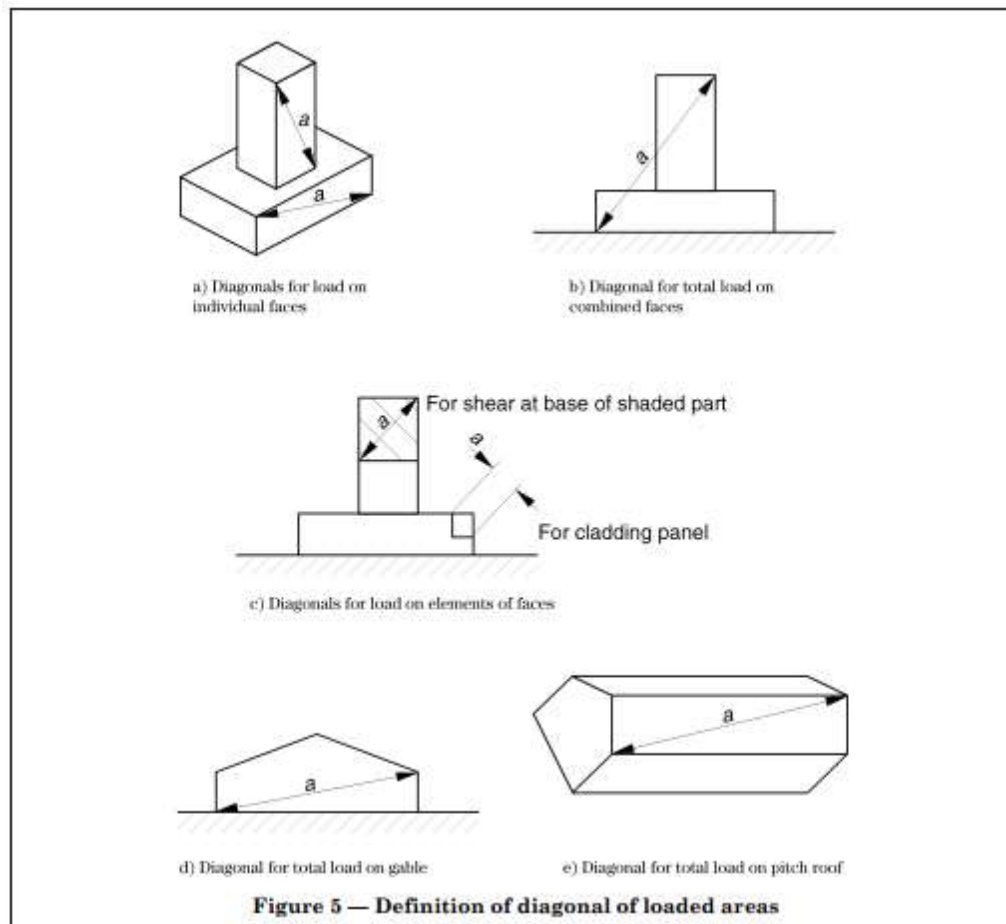
(A.7): SIZE EFFECT FACTOR C_a



Key to lines on Figure 4							
Effective height H_e m	Site in country: closest distance to sea (km)				Site in town: closest distance to sea (km)		
	0 to < 2	2 to < 10	10 to < 100	≥ 100	2 to < 10	10 to < 100	≥ 100
≤ 2	A	B	B	B	C	C	C
> 2 to 5	A	B	B	B	C	C	C
> 5 to 10	A	A	B	B	A	C	C
> 10 to 15	A	A	B	B	A	B	B
> 15 to 20	A	A	B	B	A	B	B
> 20 to 30	A	A	A	B	A	A	B
> 30 to 50	A	A	A	B	A	A	B
> 50	A	A	A	B	A	A	B

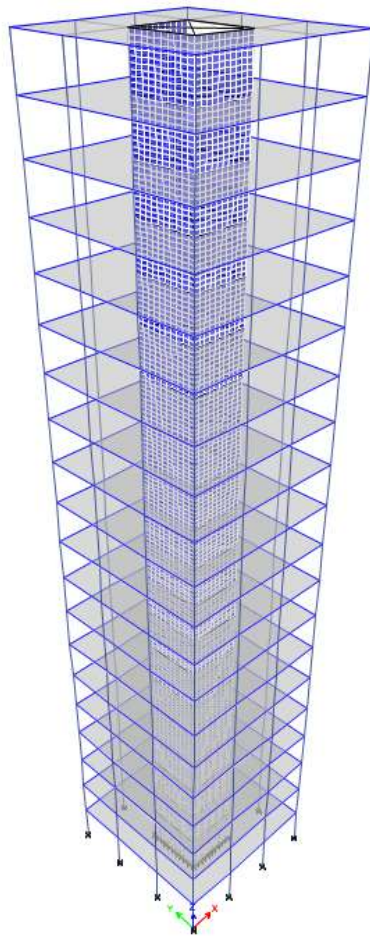
Figure 4 — Size effect factor C_a of standard method

(A.8): Definition Of Diagonal Of Loaded Areas



Appendix B

ETABS® 2013
Integrated Building Design Software



Project Report

1. Structure Data

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

1.1 Story Data

Table 1.1 - Story Data

Story	Height(m)	Elevation(m)	Similar To
STORY26	3.2	83.2	None
STORY25	3.2	80	STORY26
STORY24	3.2	76.8	STORY26
STORY23	3.2	73.6	STORY26
STORY22	3.2	70.4	STORY26
STORY21	3.2	67.2	STORY26
STORY20	3.2	64	STORY26
STORY19	3.2	60.8	STORY26
STORY18	3.2	57.6	STORY26
STORY17	3.2	54.4	STORY26
STORY16	3.2	51.2	STORY26
STORY15	3.2	48	STORY26
STORY14	3.2	44.8	STORY26

STORY13	3.2	41.6	STORY26
STORY12	3.2	38.4	STORY26
STORY11	3.2	35.2	STORY26
STORY10	3.2	32	STORY26
STORY9	3.2	28.8	STORY26
STORY8	3.2	25.6	STORY26
STORY7	3.2	22.4	STORY26
STORY6	3.2	19.2	STORY26
STORY5	3.2	16	STORY26
STORY4	3.2	12.8	STORY26
STORY3	3.2	9.6	STORY26
STORY2	3.2	6.4	STORY26
STORY1	3.2	3.2	STORY26
BASE	0	0	None

1.2 Grid Data

Table 1.2 - Grid Lines

SysName	GridDir	GridID	GridCoord	GridType	GridColor	GridHide	BubbleLoc	SortID
GLOBAL	X	A	0	Primary	Gray8Dark	No	Default	1
GLOBAL	X	B	5	Primary	Gray8Dark	No	Default	2
GLOBAL	X	C	10	Primary	Gray8Dark	No	Default	3
GLOBAL	X	D	15	Primary	Gray8Dark	No	Default	4
GLOBAL	X	E	20	Primary	Gray8Dark	No	Default	5
GLOBAL	X	F	25	Primary	Gray8Dark	No	Default	6
GLOBAL	Y	1	0	Primary	Gray8Dark	No	Switched	7

2 Properties

This chapter provides property information for materials, frame sections

2.1 Materials

Table 2.1 - Material Properties - Summary

Material	Type	Mass	Weight	Dir	Plane	E	Poisson	ThermCoeff	G	DesignType
CONC	Isotropic	2.40E+00	2.40E+01	All	All	24821128	0.2	9.90E-06	10342137	Conc
OTHER	Isotropic	7.83E+00	7.68E+01	All	All	2E+08	0.3	1.17E-05	76903069	None

2.2 Frame Sections

Table 2.2 - Frame Sections - Summary

Section Name	Material	From File	ThickBot	Area	TorsionConst
BEAM	CONC	No	Rectangular	1	0.3
COL	CONC	No	Rectangular	1	0.35

3. Analysis Results

This chapter provides analysis results.

3.1 Structure Results

Table 3.1 - Base Reactions

Story	Point	Load	FX	FY	FZ	MX	MY	MZ
BASE	1	WINDX	-83.64	0	221.38	0	-177.643	0
BASE	2	WINDX	-62.67	0	669.98	0	-157.207	0
BASE	3	WINDX	-84.88	0	62.95	0	-179.586	0
BASE	4	WINDX	-85.22	0	-62.54	0	-180.217	0
BASE	5	WINDX	-84.68	0	-220.94	0	-179.6	0
BASE	6	WINDX	-64.22	0	-670.83	0	-160.357	0
Summation	0, 0, Base	WINDX	-465.31	0	0	0	-21426	0

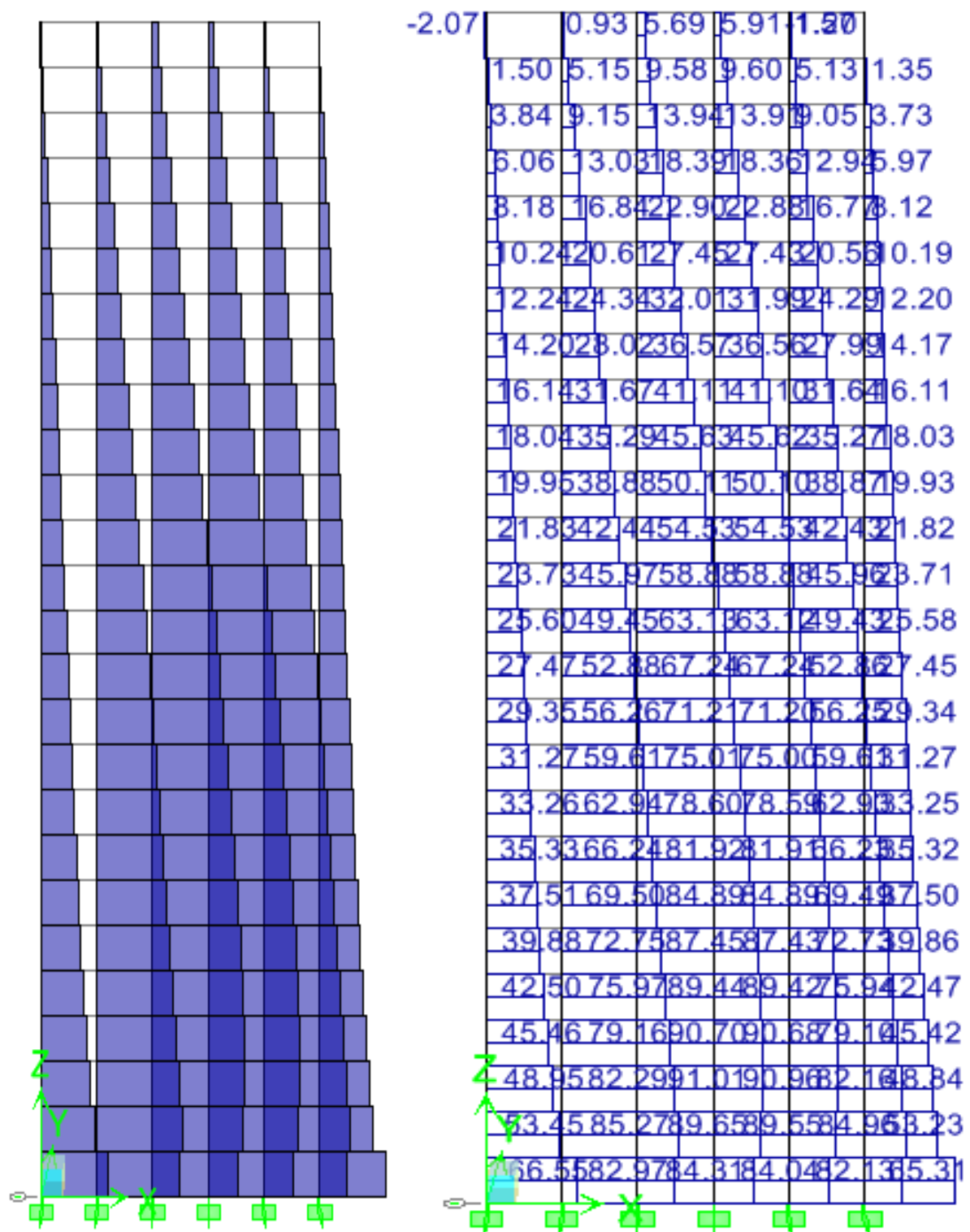
3.2 Story Results

Table 3.2 - Story Forces

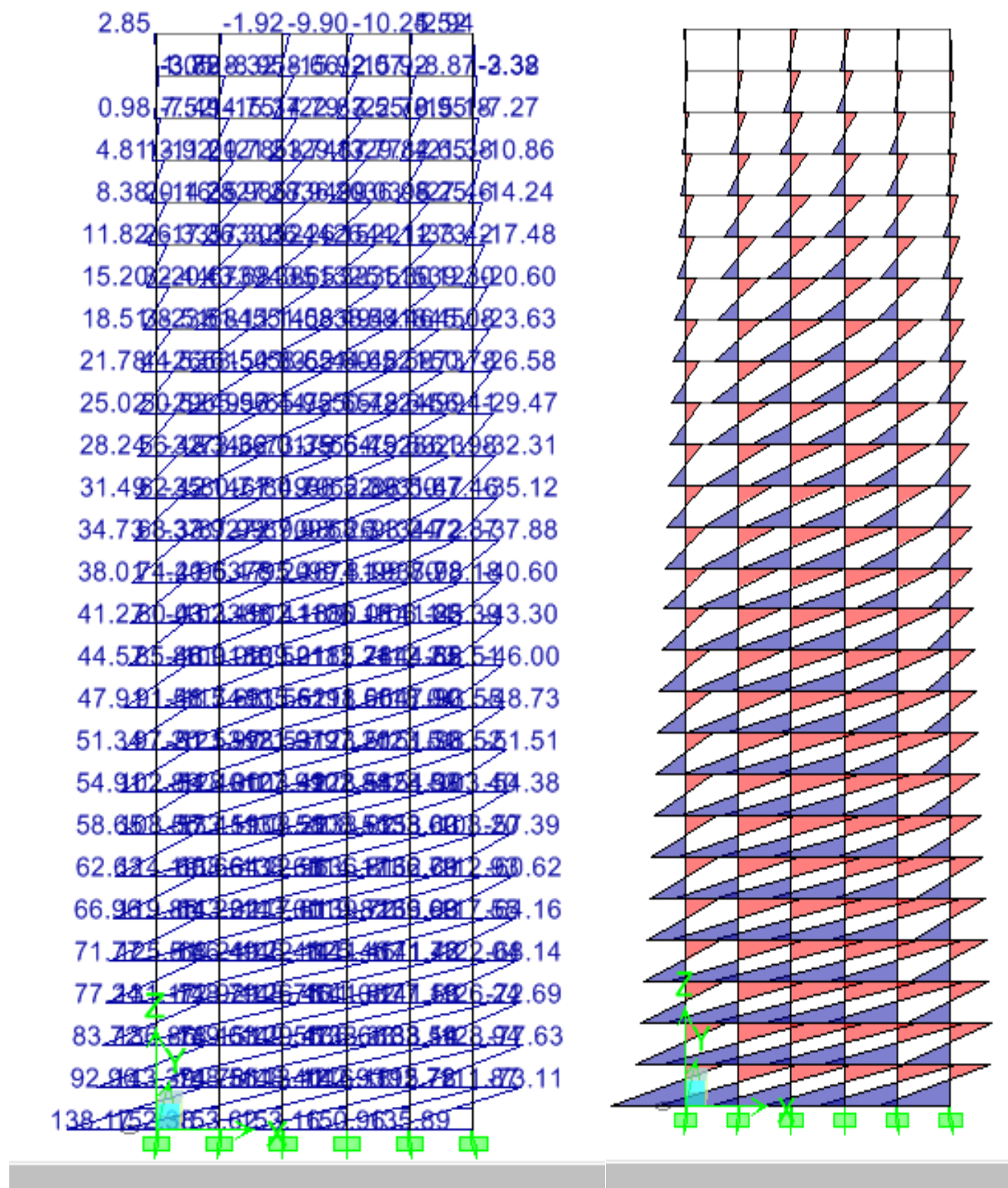
Story	Load	Loc	P	VX	VY	T	MX	MY
STORY26	WINDX	Top	0	-10.83	0	0	0	0
STORY26	WINDX	Bottom	0	-10.83	0	0	0	-34.646
STORY25	WINDX	Top	0	-32.31	0	0	0	-34.646
STORY25	WINDX	Bottom	0	-32.31	0	0	0	-138.045
STORY24	WINDX	Top	0	-53.62	0	0	0	-138.045
STORY24	WINDX	Bottom	0	-53.62	0	0	0	-309.631
STORY23	WINDX	Top	0	-74.75	0	0	0	-309.631
STORY23	WINDX	Bottom	0	-74.75	0	0	0	-548.828
STORY22	WINDX	Top	0	-95.69	0	0	0	-548.828
STORY22	WINDX	Bottom	0	-95.69	0	0	0	-855.036
STORY21	WINDX	Top	0	-116.47	0	0	0	-855.036
STORY21	WINDX	Bottom	0	-116.47	0	0	0	-1227.75
STORY20	WINDX	Top	0	-137.08	0	0	0	-1227.75
STORY20	WINDX	Bottom	0	-137.08	0	0	0	-1666.41
STORY19	WINDX	Top	0	-157.52	0	0	0	-1666.41
STORY19	WINDX	Bottom	0	-157.52	0	0	0	-2170.48
STORY18	WINDX	Top	0	-177.78	0	0	0	-2170.48

STORY18	WINDX	Bottom	0	-177.78	0	0	0	-2739.38
STORY17	WINDX	Top	0	-197.88	0	0	0	-2739.38
STORY17	WINDX	Bottom	0	-197.88	0	0	0	-3372.59
STORY16	WINDX	Top	0	-217.84	0	0	0	-3372.59
STORY16	WINDX	Bottom	0	-217.84	0	0	0	-4069.67
STORY15	WINDX	Top	0	-237.59	0	0	0	-4069.67
STORY15	WINDX	Bottom	0	-237.59	0	0	0	-4829.97
STORY14	WINDX	Top	0	-257.12	0	0	0	-4829.97
STORY14	WINDX	Bottom	0	-257.12	0	0	0	-5652.76
STORY13	WINDX	Top	0	-276.3	0	0	0	-5652.76
STORY13	WINDX	Bottom	0	-276.3	0	0	0	-6536.93
STORY12	WINDX	Top	0	-295.14	0	0	0	-6536.93
STORY12	WINDX	Bottom	0	-295.14	0	0	0	-7481.37
STORY11	WINDX	Top	0	-313.62	0	0	0	-7481.37
STORY11	WINDX	Bottom	0	-313.62	0	0	0	-8484.94
STORY10	WINDX	Top	0	-331.77	0	0	0	-8484.94
STORY10	WINDX	Bottom	0	-331.77	0	0	0	-9546.6
STORY9	WINDX	Top	0	-349.58	0	0	0	-9546.6
STORY9	WINDX	Bottom	0	-349.58	0	0	0	-10665.3
STORY8	WINDX	Top	0	-366.95	0	0	0	-10665.3
STORY8	WINDX	Bottom	0	-366.95	0	0	0	-11839.5

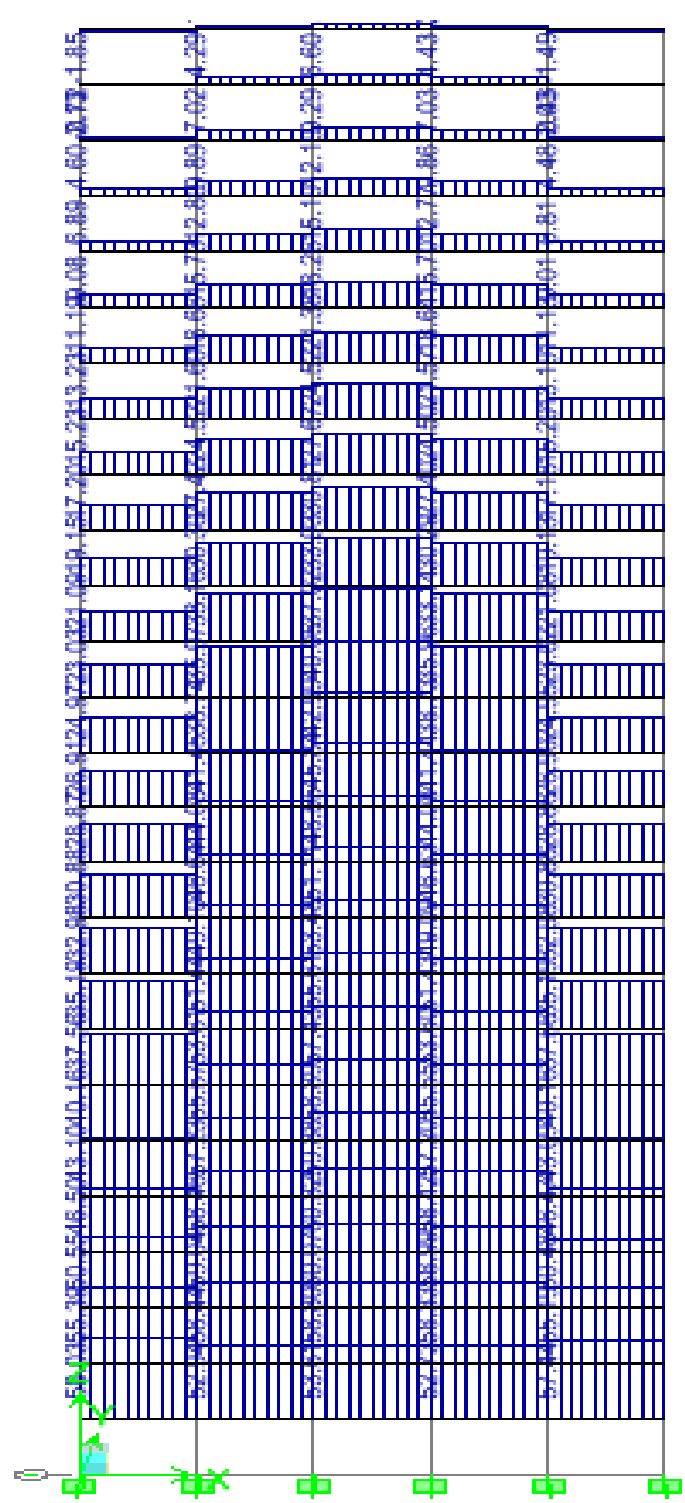
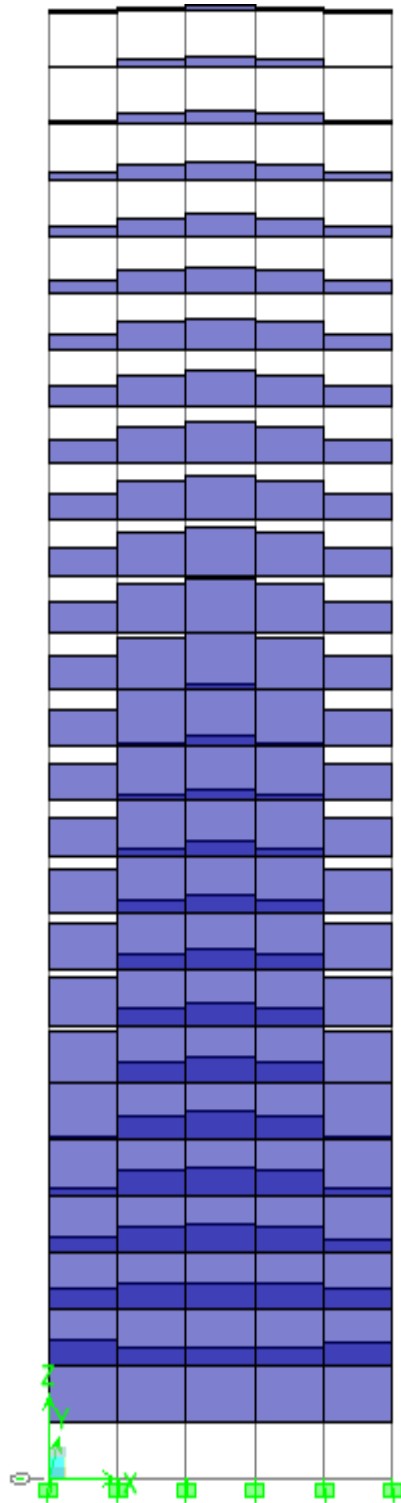
STORY7	WINDX	Top	0	-383.78	0	0	0	-11839.5
STORY7	WINDX	Bottom	0	-383.78	0	0	0	-13067.6
STORY6	WINDX	Top	0	-400.11	0	0	0	-13067.6
STORY6	WINDX	Bottom	0	-400.11	0	0	0	-14347.9
STORY5	WINDX	Top	0	-415.74	0	0	0	-14347.9
STORY5	WINDX	Bottom	0	-415.74	0	0	0	-15678.3
STORY4	WINDX	Top	0	-430.52	0	0	0	-15678.3
STORY4	WINDX	Bottom	0	-430.52	0	0	0	-17055.9
STORY3	WINDX	Top	0	-444.22	0	0	0	-17055.9
STORY3	WINDX	Bottom	0	-444.22	0	0	0	-18477.4
STORY2	WINDX	Top	0	-456.11	0	0	0	-18477.4
STORY2	WINDX	Bottom	0	-456.11	0	0	0	-19937
STORY1	WINDX	Top	0	-465.31	0	0	0	-19937
STORY1	WINDX	Bottom	0	-465.31	0	0	0	-21426



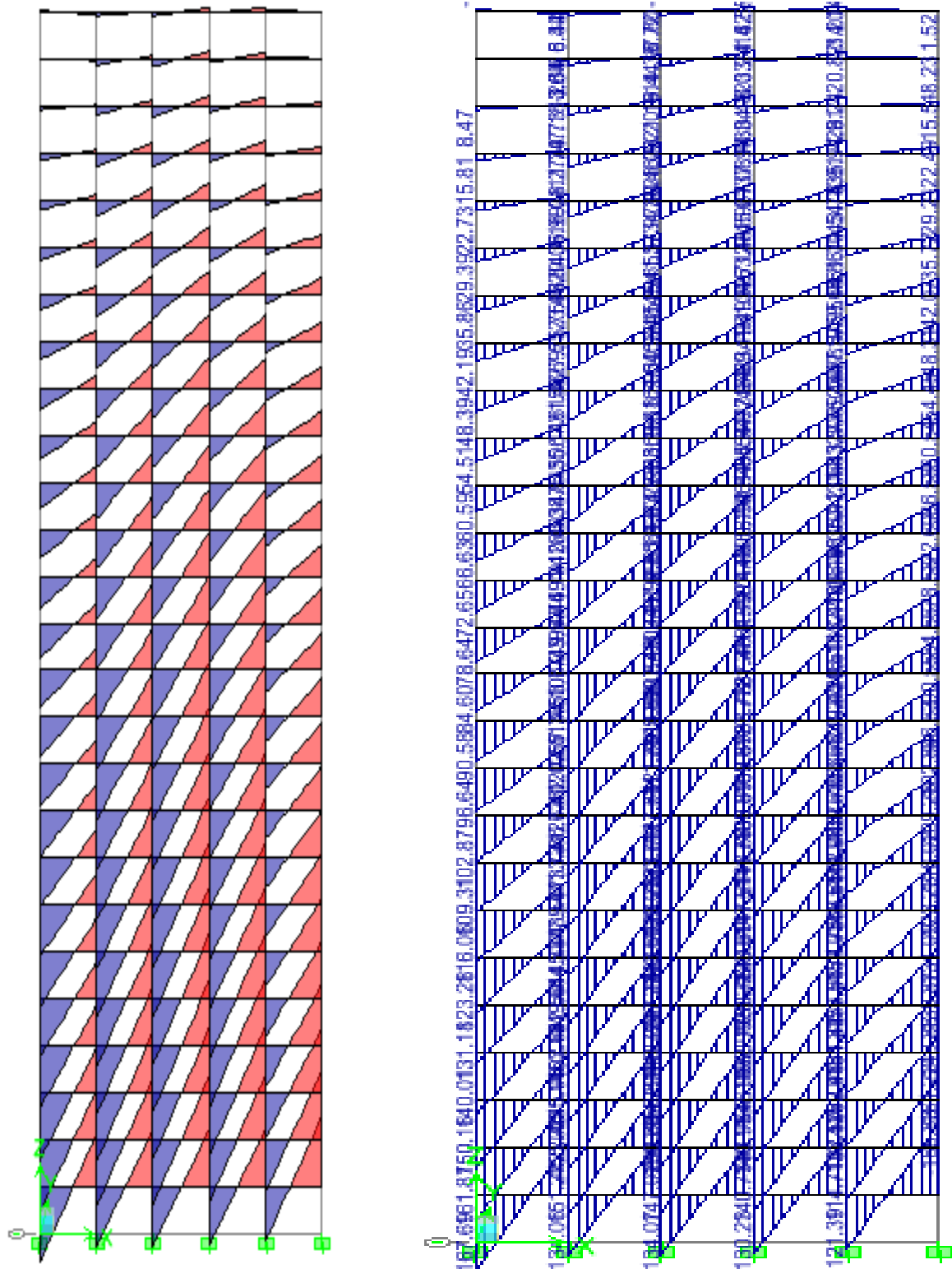
Shear Forces (kN) in Columns.



Bending Moments (kN/m) in Columns.



Shear Forces (kN) in Beams.



Bending Moments (kN/m) in Beams