BEHAVIOR OF R.C. FRAMED BUILDINGS WITH SHEAR WALLS

A DISSERTATION

Submitted in partial fulfillment of the requirements for the award of the Degree of

Master of Engineering
In
Structural Engineering

By

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ABSTRACT

The increase in population and the paucity of land, particularly in urban areas has compelled designers to go in for sky-scrapers. For low to medium rise buildings, the analysis and design with respect to lateral forces has generally been a process of checking the vertical load system for its ability to resist lateral forces. However, for tall building the vertical load resisting system is inefficient in resisting lateral forces. Hence in tall building of reinforced concrete, the lateral loads are commonly resisted by specially provided shear walls. For building having of 15 to 40 storeys, the shear wall-frame system has been found to be the most acceptable one.

The present study pertains to lateral load analysis of some shear wall frame systems. The analysis has been carried out by using STAAD Pro.2004. to determine the forces, deflections and point of contraflexure in various structural components, viz., beams, columns , link beams and shear wall .
Two examples have been solved using STAAD Pro.2004.

The first one is an unsymmetrical multistory building with two cases; firstly shear walls extend upto top and secondly where they are not provided in the top storeys and the effect of change in thickness of shear walls have also been considered.

The second building is symmetrical with four cases, i.e., symmetrical shear wall and secondly one of the shear walls removed to introduce unsymmetry.
In this example, the curtailment of shear walls has been considered for sysmmetrical building.

The shears, bending moments and deflections have been studied in all cases in each example and also point of contraflexure is compared in two examples. On the basis of this study, some useful conclusions are drawn.

DEPARTMENT OF CIVIL AND STRUCTURAL ENGINEERING

DELHI COLLEGE OF ENGINEERING

CERTIFICATE

This is to certify that the dissertation entitled "Behavior of R.C.Framed Building With Shear Walls" submitted by Asif Ali, to the Department of Civil and Structural Engineering of Delhi college of engineering (DCE), in the partial fulfillment of the requirement for the award of , MASTER Of ENGINEERING. This is a bonafide work carried out by Asif Ali under our guidance and supervision. To the best of our knowledge it has reach the standard fulfilling the requirement of the degree.

Jully, 2004

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CHAPTER-1 INTRODUCTION

1.1 GENERAL

The increase in population and the paucity in land, particularly in an urban area has compelled designers to go in for sky-scrapers. In the past, buildings were restricted to 6-7 stories because of conventional methods of Construction. In recent years, the demand for taller buildings has created the necessity to search for new structural systems. For low to medium rise structures, the analysis and design with respect to lateral forces has generally been a process of checking the vertical load resisting system for its ability to resist lateral forces. However, for tall buildings the vertical load resisting system cannot resist lateral forces efficiently. From economic, structural strength, and stiffness considerations, it is essential that the lateral force resisting system be carefully considered in the initial design stage and incorporated as an important feature of the total design. Therefore, in order to make the structure economically viable, various structural systems have been introduced in multistoried buildings depending upon the number of storeys as shown in Fig.1.1

1.2 REINFORCED CONCRETE BUILDINGS

Various kinds of structures, such as buildings, bridges, piers, dams, waterways, water tanks and tunnels are constructed in reinforced concrete. Among them, building is the most common reinforced concrete construction, planned for residential, commercial or institutional uses. Although the building is a three-dimensional structure, it is usually analyzed as assemblage of two dimensional sub systems. The various kinds of loads may be considered for the analysis of the

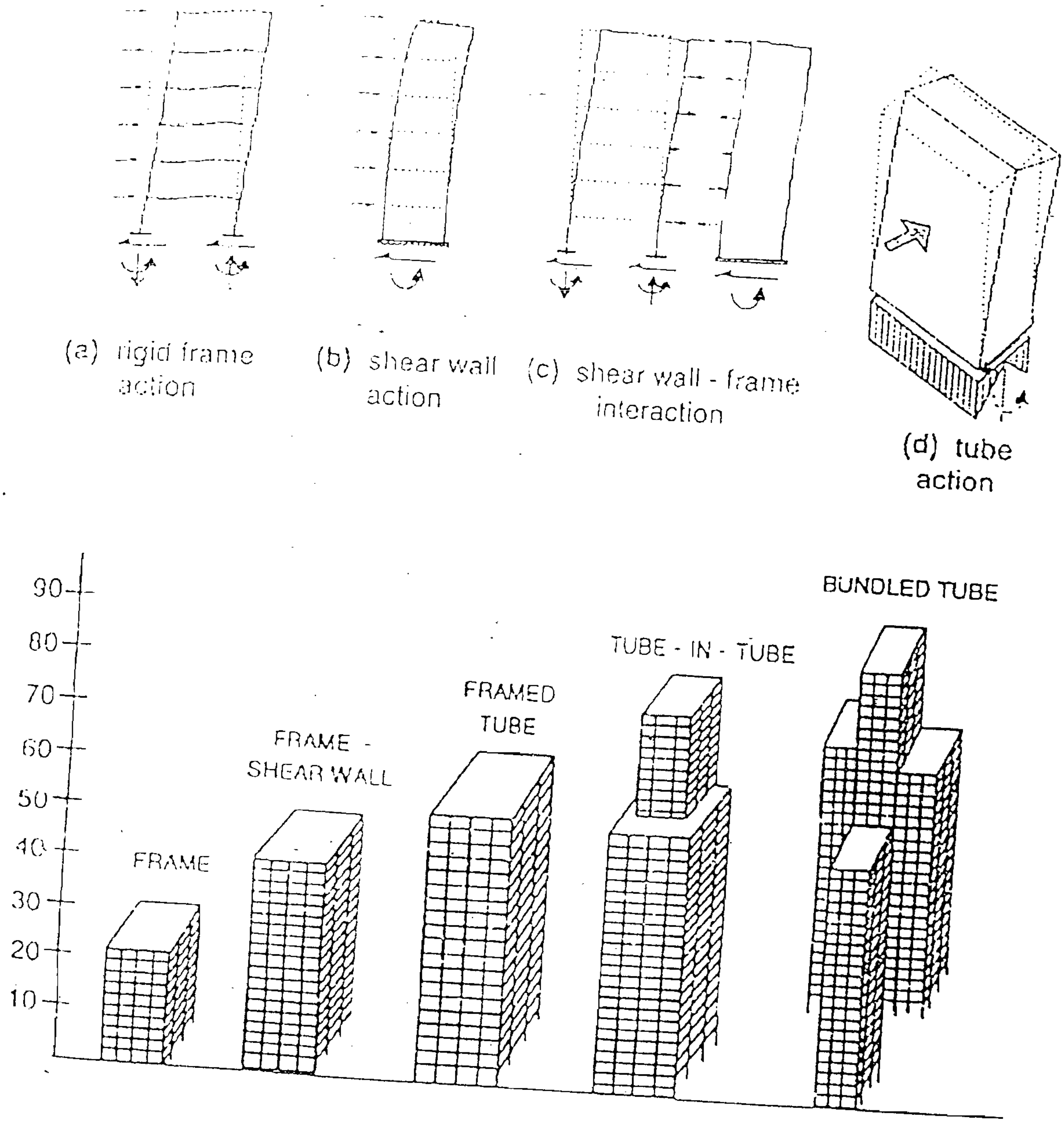


FIG .1.1-Structural Concept Vs No Of Storeys.

buildings (such as dead load, live load, wind load, snow load and earth quake load). The load transfer mechanism of the buildings is divided into gravity loads and lateral loads.

1.3 LOADS

The loads applied to the structures are the main cause of stresses and deformations. Other causes may be support settlements and temperature changes. As discussed above, the various loads may be subdivided into dead load, live load, wind load, earthquake load and snow load.

1.3.1 Dead Loads

Dead loads consist of the weight of the walls, frame, and other structural components and the machinery, which is permanently installed. Even though the calculation of the dead load of the members with known dimensions is routine task, computation of dead load may not be easy during design; this is because at the design stage the final dimensions of element may not be known. In practice the designer starts with a set of blue print with tentative dimensions established on the basis of similar project or estimated by an experienced designer. These dimensions may be used in calculation of dead load. If in the course of design some of the dimensions are modified, the adjustment must be made to reflect these changes, unless the change is insignificant. For the purposes of design, magnitude of dead loads of different building materials are given in IS: 875-1978, Part-1.

1.3.2 Live Loads

The live loads are the movable or moving loads, therefore, their magnitude and/or location may be variable, live loads may be due to people, furniture etc. For design purposes their magnitudes are specified in IS: 875-1978, Part-2.

1.3.3 Snow Loads

Roof snow loads depends on the geographic location and conditions of the building. Actual load due to snow will depend upon the shape of the roof and its capacity to retain the snow. For design purposes their magnitude is specified in IS: 875-1978, Part-3.

1.3.4 Wind Loads

Wind loads are particularly significant on tall buildings, however on small buildings the uplift caused by the wind may be severe enough to cause damage to the building. The various magnitudes of wind loads are specified in the IS: 875-1978 Part-4 according to the location and type of structures.

1.3.5 Earth Quake Loads

The earthquake force depends on the seismicity of the site and properties of the structure. The main forces are generally in the horizontal direction. The various magnitudes of earthquake loads are specified in IS:1893-1984 according to the zones in which the site lies.

For special loads & different load combinations IS: 875-1978 Part-5 should be referred.

For convenience, the structural system is separated into two load transfer mechanisms, viz. Gravity load resisting and lateral load resisting, although, in effect, these two systems are complementary and interactive. As an integrated system, the structure must resist and transmit all the effects of gravity loads and lateral loads acting on it to the foundation and below. While studying the load resisting mechanism it is convenient to divide the structural system of building into horizontal framing (floor) system and vertical framing system.

1.4. STRUCTURAL SYSTEMS

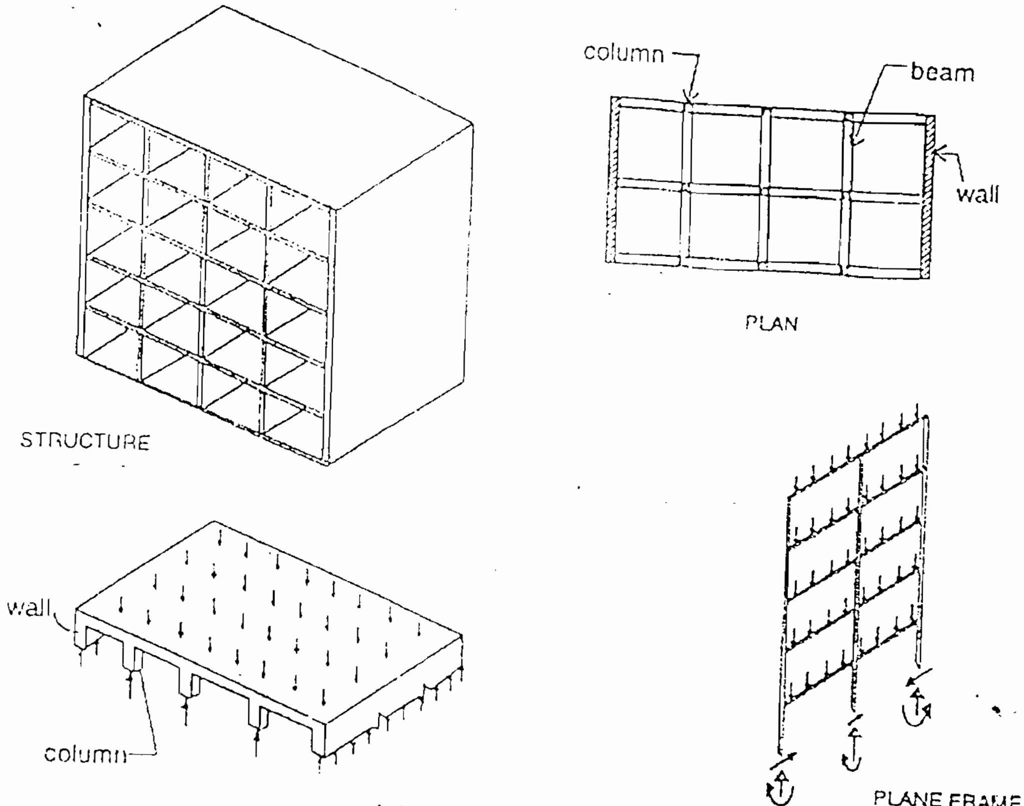
Structure consists of structural elements (load carrying, such as beams, column, slabs etc.) and non-structural element (such as partition, false ceiling, doors). The structural elements put together constitute the 'structural system'. Its function is to resist effectively, the action of gravitational and environmental loads and to transmit the resulting forces to the supporting ground, without significantly disturbing the geometry, integrity and serviceability of the structure. There are two types structural systems, which are used in buildings as describe below. Fig. 1.2

1.4.1 Horizontal Framing System (Floors)

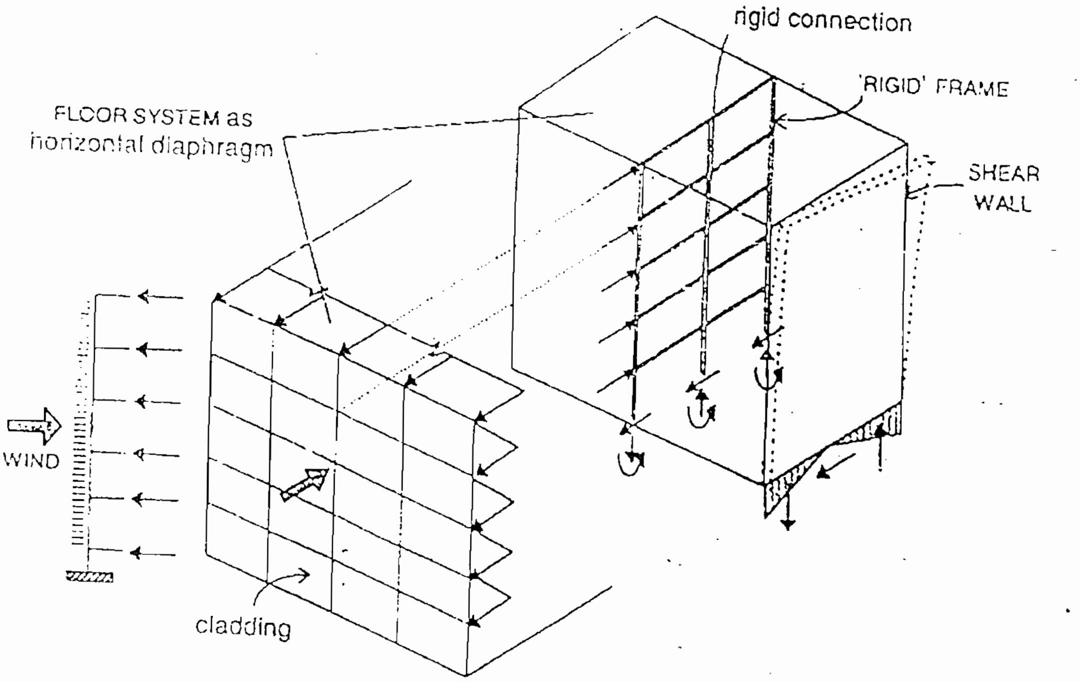
The horizontal (floor) system resists the gravity loads (dead load and live load) acting on it and transfers them to the vertical framing system. The horizontal frame elements are subjected primarily to flexure and transverse shear, where as vertical frame elements are generally subjected to axial compression, often coupled with flexure and shear. The floor systems usually consist of one of the following

1.4.1.1 Wall Supported Slab System

In this system, the floor slabs are supported generally on masonry walls. The slabs are cast in panels, which may be continuous over several wall supports and are called one way continuous or two-way continuous slabs, depending on whether the bending is predominant along one direction or two directions. Hogging moment is induced in the slab in the region adjacent to the continuous support. If the slab panel is not extended over the support, it is called simply



(a) vertical load transfer



(b) lateral load transfer

Fig. 1.2- Load Transfer Mechanism.

supported slab. Furthermore, the twisting moments are also introduced at corners, which are restrained against lifting. Fig. 1.3

1.4.1.2 Beam Supported Slab

The system is similar to wall supported system except that the floor slabs are supported on beams instead of walls. The slabs are cast monolithically with the beams in rigid pattern. The system is adopted in framed structures and the vertical loads are transmitted to the columns through beams. The span of these slab panels generally ranges from 3 to 8 meters. Fig. 1.4

1.4.1.3 Ribbed Slab System

This a special type of 'grid floor' slab-system in which the 'slab' (called topping) is very thin (50-100 mm) and the 'beams' called ribs are very slender and closely spaced (less than 1.5 m apart). The ribs have a thickness of not less than 65 mm and a depth, which is three to four times the thickness. The ribs may be designed in one way or two-way pattern and are generally cast in-situ, although pre-cast construction is also possible.

Two-way ribbed slabs are sometimes called waffle slabs. Along the outer edges, the ribbed slab system is generally supported on stiff edge beams or walls. In the wall-supported systems, the thickness of the rib resting on the wall is usually increased to match the wall thickness for better bearing. Waffle slabs, used in large-span construction, may rest directly on columns. In such case, the slab is made solid in the neighborhood of the column.

1.4.1.4 Flat Slab

The flat slab are plates which are stiffened near the column supports by means of 'drop panels' or 'column capital' or both, which are generally concealed under drop ceilings. The flat slab system is suitable for high loads and larger span,

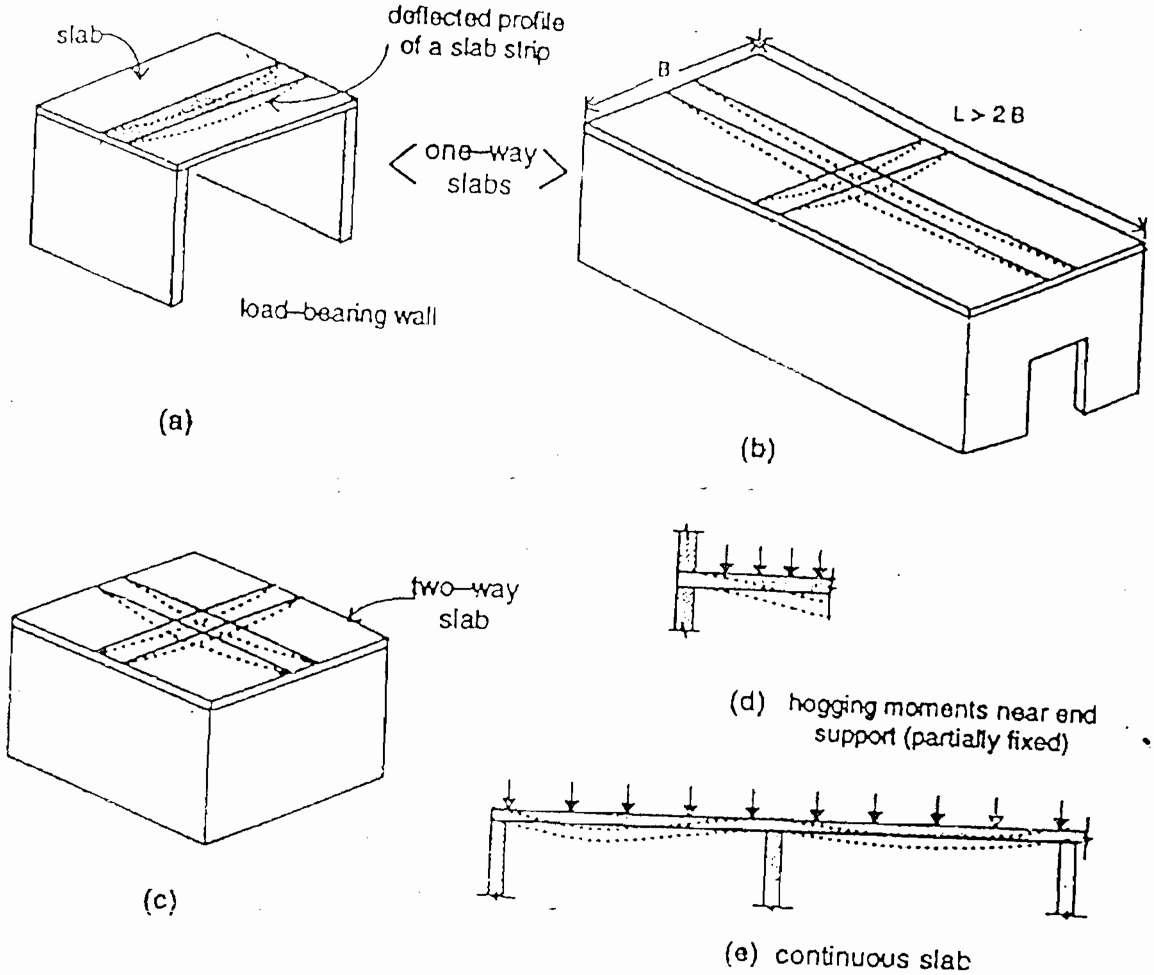


Fig. 1.3- Wall- Supported Slab Sysyem.

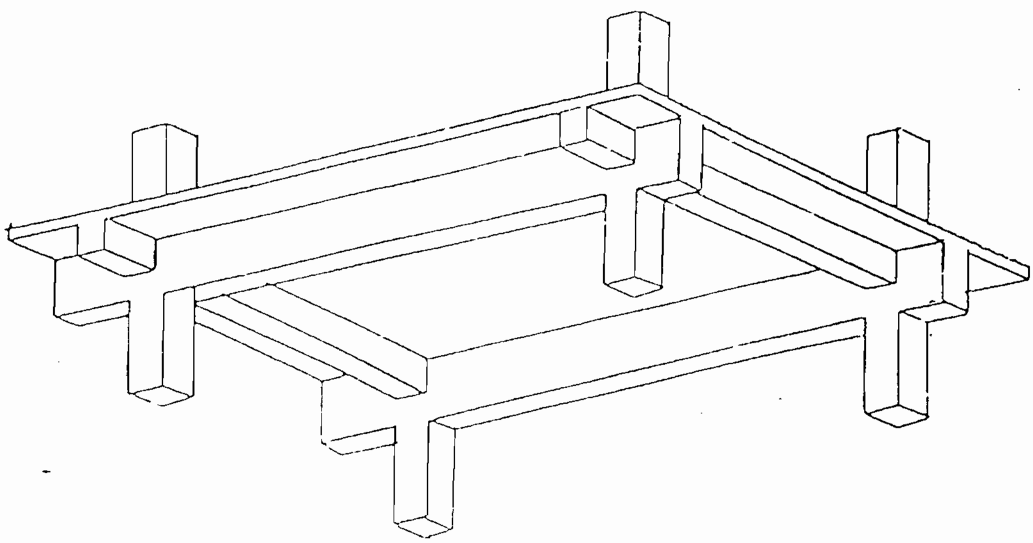


Fig. 1.4 -Beam- Supported Slab.

because of its enhanced capacity in resisting shear and hogging moments near the supports. Fig. 1.5

1.4.2 VERTICAL FRAMING SYSTEMS

The vertical framing system resists the gravity loads and lateral loads of the floor system and transmits them to the foundation and ground below. As the height of building increases, the effect of lateral loads become predominant and the selection of the structural system is primarily governed by it. Vertical framing system consists of one of the following:

1.4.2.1 Frames

Frames are often called rigid frames because the ends of the various members framing in to a joint are rigidly connected. A frame is a structural system composed of linear elements, which derives its behavior characteristics from the rigidity of the joints, which transfer moments. A typical framed building, shown in Fig.1.6 (a) and (b), is essentially three dimensional in character. The ability of frames to resist lateral loads is mainly due to the rigidity of the beam-column junction; therefore frames are used as lateral load resisting system in buildings with up to 15 or 20 storeys.

1.4.2.2 Shear Walls

A shear wall may be defined as a structural wall, which has high in plane stiffness. Shear wall is a two or three dimensional vertical structure, solid or perforated. Due to its high in plane stiffness, a shear wall is a very efficient lateral load-resisting element. It has much larger stiffness compared to a column. Total stiffness of shear walls in a building may even be 30 to 100 times the stiffness of all the frame combined. As lateral loads resisted by various resisting elements (frames and shear wall) are functions of their stiffness, shear

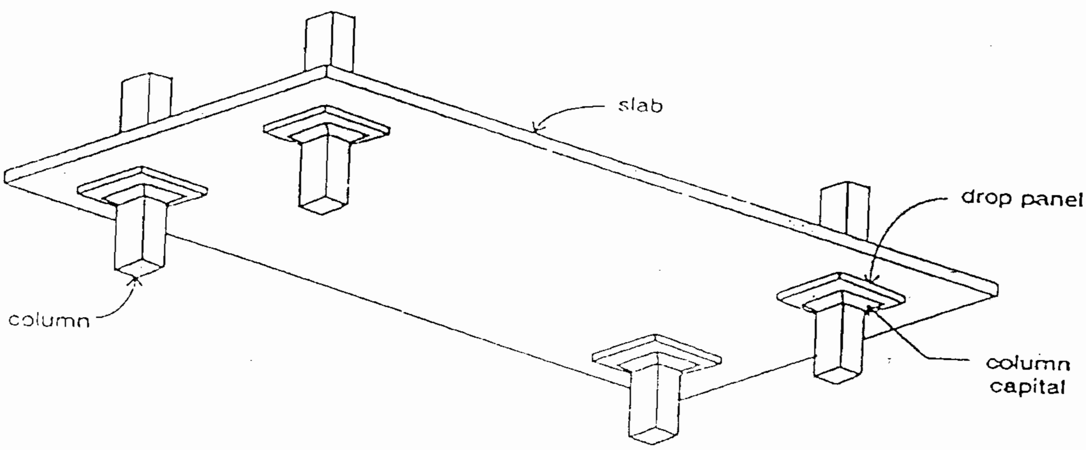


Fig. 1.5 -Flat Slab.

wall will resist a much larger component of the lateral load and in many cases, almost the entire lateral load in a tall building.(Fig. 1.7).

These walls are solid walls, which extend over the full height of the building. The walls are very stiff, having considerable depth in the direction of lateral load, they resist loads by bending like a vertical cantilever beam, fixed at the base. When the various shear walls and coexisting frames in building are linked at different floor levels by means of the floor systems, (which distribute the lateral loads to these different systems appropriately), then combined system is known as framed shear wall. The interaction between shear wall and the frame is structurally advantageous, as the wall restrains the frame deformation in the lower storeys, while the frame restrains the wall deformation in the upper storeys. Frame shear-wall systems are generally considered in buildings up to about 40 storeys.

1.4.2.2.1 FORMS OF SHEAR WALL CONSTRUCTION

1.4.2.2.2. Cross wall construction

Such form of construction consists of a series of parallel shear walls along both principal axes of the building as shown in Fig, 1.8(a). The walls act both as lateral load resisting elements as well as supports for vertical loads (i.e. bearing walls). The simplest example of cross wall construction is the load bearing brick building. However as the brick walls have little or no tensile strength and small shearing resistance, such buildings generally are not constructed for more than four storey height. If these walls are replaced by R.C.C. Walls, the construction becomes very efficient for very tall building having large strength and stiffness.

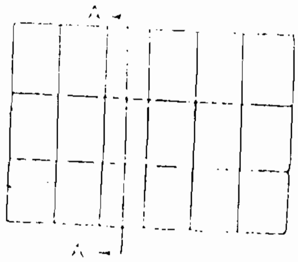


Fig. 1.6(a) –Typical Framing Plan Of Multistorey Building Composed Of Beams and Column.

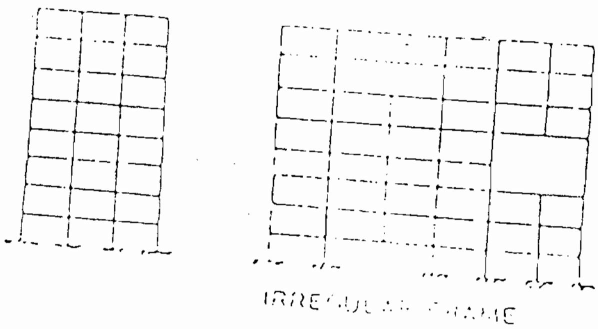
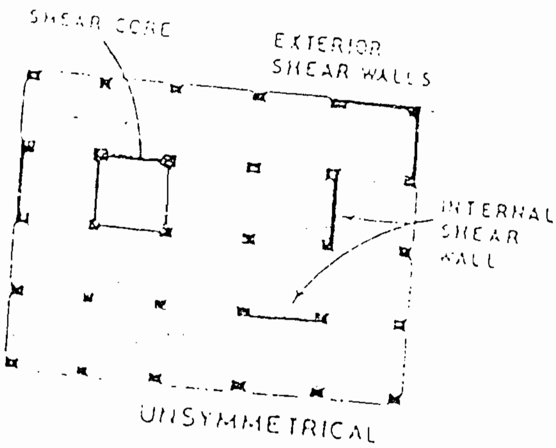


Fig. 1.6(b) –Elevation Of Typical Frame at Section A-A.



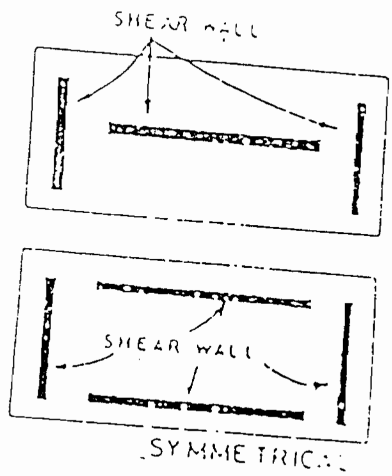


Fig. 1.7 – Typical Framing Plan Of Multistorey Building Composed Of Shear wall.

1.4.2.2.3 Shear Walls Acting With Frames

For office buildings, where large open spaces are required or for such buildings where position of particulars cannot be decided permanently, cross wall construction cannot be adopted. For such buildings, for heights above 12 to 15 storeys when frames alone become inadequate, shear walls acting along with frames are used to increase the rigidity for lateral load resistance. Such a form of construction remains efficient for up to about 40 storeys height. Since in India construction of buildings above 40 storeys is not likely in near future, this form of construction shall remain the most important form of shear wall construction for most of the tall buildings. (Refer Fig. 1.8(b)).

1.4.2.2.4 Coupled Shear Walls

A shear wall pierced by several openings one over the other is called a coupled shear wall. Two shear walls connected by floor beams or floor slabs are also called coupled shear walls. The connecting beams are called link beams. An external wall, if constructed as a shear wall, may be pierced by openings due to windows provided in the building, or a cross wall running right across the building, if constructed as a shear wall, may be pierced by doors or floor height openings due to a passage or a corridor as shown in Fig. 1.8(c).

1.4.2.2.5 Column Supported Shear Wall

When it is necessary for architectural reasons to discontinue shear walls at floor level it becomes necessary to carry the wall to the ground on widely spaced columns. In such column supported shear walls, the discontinuity in geometry at the lowest level should be specially taken care of in the design. Fig.1.8 (d).

1.4.2.2.6 Core Type Shear walls

In some buildings, the elevators and other service areas can be grouped in a vertical core, which may serve as devices to withstand lateral loads. Unsymmetry produces twisting and if twisting is not present these walls act as simple shear walls. Cores with designed lintels at regular intervals as in elevator shafts have also good resistance against torsion. Fig.1.8 (e).

1.4.2.3 Tubes

In this system, closely spaced-columns are located along the periphery of the building. These columns are interconnected by deep spandrel beams, located on the exterior surface of the building. The entire system behaves like a perforated box. The system has a high rigidity against lateral loads. The system can be of a framed tube, tube in tube, stiffened tube or a bundle tube type.

1.5 NECESSITY OF SHEAR WALL

For all high rise buildings, the problems of providing adequate stiffness and preventing large displacements are as important as providing adequate strength. Thus a shear wall system has two distinct advantages over frame system.

1. It provides adequate strength to resist large lateral loads without excessive additional cost.
2. It provides adequate stiffness to resist lateral displacements to Permissible limits, thus reducing the risk of non-structural damage and discomfort in tall buildings.

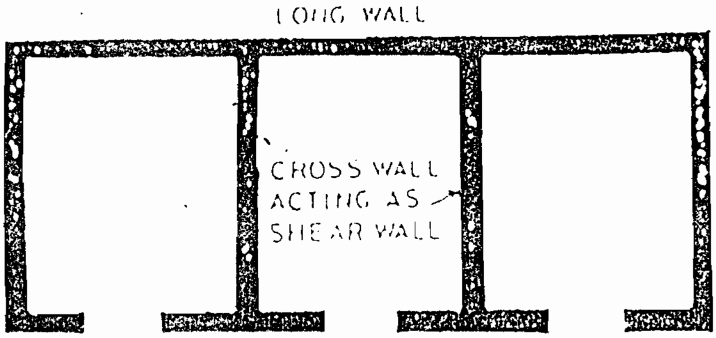


Fig. 1.8(a)-Cross Wall Construction.

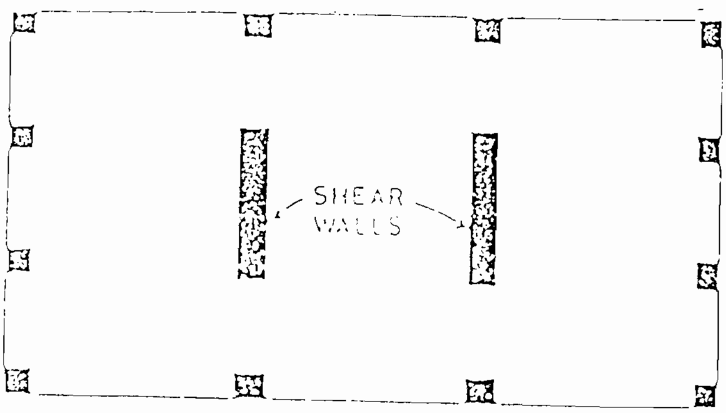
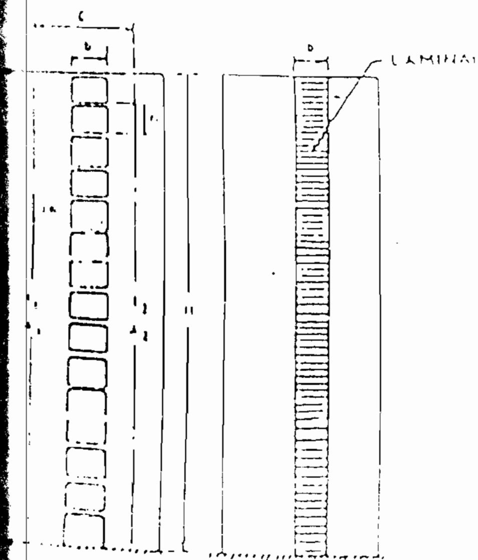


Fig. 1.8(b)-Shear Wall Acting With Plan.



g. 1.8(c)-Coupled Shear Walls.

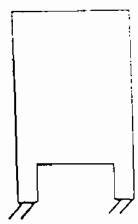
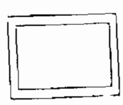


Fig. 1.8(d)-Column Supported Shear Walls.



Plan

Fig. 1.8(e)-Core Type Shear Walls.

1.6 IMPORTANT FEATURES IN PLANNING AND DESIGN OF SHEAR WALLS

(i) Shear walls should be located such that they also act as functional walls and do not interfere with the architecture of the building. Enclosures around the lift and stair case are the most commonly used systems for shear walls.

(ii) Shear walls should be placed along both the axes so that lateral stiffness can be provided in both directions, particularly in case of square type of plan.

(iii) Shear walls should be placed symmetrically about the axes to avoid torsion in the building,

(iv) Shear walls should be designed to take gravity loads also so that under lateral loads, large amount of tension may not develop.

(v) Shear walls, when provided, should be continued right upto foundation level.

1.7 FRAME - SHEAR WALL INTERACTION:

The basic difference between the lateral load behavior of a shear wall and a frame taken separately is in their modes of deformation. Under lateral loads, a rigid frame deforms in a shear mode as shown Fig.1.9(a). The floors essentially remain level and storey displacements are proportional to storey shear. On the other hand, a shear wall deflects in a bending mode as shown in fig. 1.9(b). The interaction of a shear wall and a frame is a special case of indeterminacy, in which basically two different components are tied together

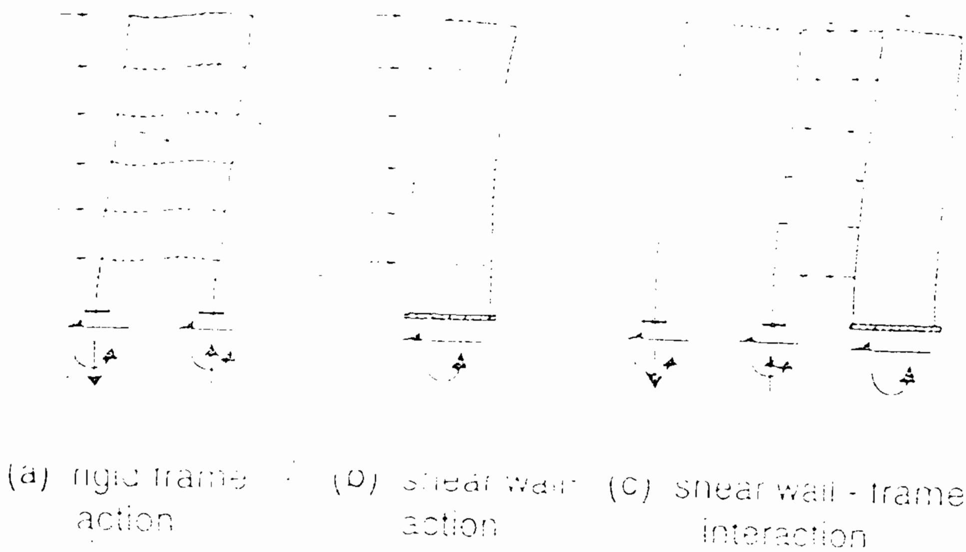


Fig. 1.9-Interaction Of Shear Walls and Frame

to produce one structure. Due to different deformation characteristics of the frame and shear wall under lateral loads, the frame tends to pull back the shear wall in the upper portion of the building and push it forward in the lower portion as shown in Fig. 1.9(c). As a result, the frame participates more effectively in the upper portion of the building where the lateral load stresses are less and the shear wall carries most of the shear in the lower portion of the building where the frame generally cannot afford to carry high lateral loads.

1.8 METHODS OF ANALYSIS

There are several methods used in the analysis of structures, to find the forces and displacements in the various members of the structure under any loading (Static or dynamic). However, these methods can be broadly classified into force methods and displacement methods.

1.8.1 Force Methods

These methods are also known as flexibility methods or compatibility methods. In these methods, the given structure is made statically determinate by choosing a number of actions (i.e. forces and moments) as redundant actions. To evaluate the redundant actions, as many equations are formed, as is the number of redundant actions. Once the redundant actions are evaluated, the structure is made statically determinate by removal of the redundant actions hence the structure can be analyzed like a statically determinate structure. The various force methods are:

- Moment Distribution Method.
- Unit Load Method.
- Energy Methods.
- Flexibility Matrix Method.

1.8.2 Displacement Methods

These methods are also known as stiffness methods or equilibrium methods. In these methods, the displacements at the joints are taken as unknowns. Using the equations of static equilibrium for the joints, as many equations are formed, as is the number of unknown joint displacements. Knowing the displacements, actions at the joints (called member end action, there being two ends for each member) are computed. Knowing member end actions, each member and therefore the complete structure can be analyzed. The various displacement methods are:

- Kani's Method
- Slope Deflection Method.
- Stiffness Matrix Method.
- Finite Element Method.

1.9 OBJECTIVE AND SCOPE OF WORK

A shear wall frame building is a three dimensional structural system consisting of frames, shear walls and floors. A typical floor plan of a shear wall frame building is shown in fig. 1.6.

The following are the objectives of this thesis:

(i) To study the behavior of multi storey building frame

(ii) To study the behavior of shear wall.

(iii) To study the combined behavior of building frame and shear wall

(iv) To analyze the multi storey building frame with shear wall provide up to top storey by using STADD. Pro 2004.

(v) To find out the possibility of curtailment of shear wall.

(vi) To study the nature of point of contraflexure curve

(vii) To study the behavior of frame after curtailment of shear wall.

The lateral load resistance of tall wall-frame building structure comprising a combination of moment-resisting frames and shear wall, the shear wall can reduced in size or terminated entirely at intermediate heights, which make the structure economical

CHAPTER - 2

LITERATURE REVIEW

2.1 GENERAL

Shear walls are specially designed structural walls incorporated in buildings to resist lateral forces that are produced in the plane of the wall due to wind, earthquake and other forces. The term 'shear wall' is rather misleading as such walls behave more like flexural members. They are usually provided in tall buildings and have been found of immense use to avoid total collapse of buildings under seismic forces. It is always advisable to incorporate them in buildings built in regions likely to experience earthquake of large intensity or high winds. Shear walls for wind are designed as simple concrete walls. The design of these walls for seismic forces requires special considerations, as they should be safe under repeated loads. Shear walls for wind or earthquakes are generally made of concrete or masonry. They are usually provided between columns, in stairwells, lift wells, toilets, utility shafts, etc. Tall buildings with flat slabs should invariably have shear walls. Such systems compared to slabs with beams have very little resistance even to moderate lateral loads. Initially shear walls were used in reinforced concrete buildings to resist wind forces. These came into general practice only as late as 1940. With the introduction of shear walls, concrete construction can be used for tall buildings. Earlier tall buildings were made only of steel, as bracings to take lateral loads could be easily provided in steel construction. However, since recent observations have shown consistently the excellent performance of buildings with shear walls even under seismic forces, such walls are now extensively used for all earthquake resistant designs. Surveys of buildings after earthquakes have consistently shown that the loss of life due to complete collapse was minimal in buildings with some sort of shear wall. However the most important property of shear walls for seismic design, as different from design for wind, is that it should have good ductility under reversible and repeated over loads. In planning shear walls, we should try to reduce the bending tensile stresses due to lateral loads as much as possible by loading them with as much gravity forces as it can safely take. They should be also laid symmetrically to avoid torsional stresses.

2.2 REVIEW OF LITERATURE

2.2.1 Slender Shear Walls

Slender Shear Walls can be defined as vertical cantilevers, with various cross sections such as: rectangular (a vertical plate), I, box, and other elevator walls. The shear walls support the vertical load, in addition to their function to stiffen the frames in their resistance to lateral loads due to winds, earthquake or blast. Although interior and exterior concrete walls have been used to stiffen structures as long as reinforced concrete itself has been in use, the modern concept of shear walls designed as vertical slender cantilevers were first utilized in 1948 in housing projects in New York City and in Chicago in buildings designed for wind forces, to augment the lateral resistance of the frames.

In non earthquake areas of the United States and Canada concrete buildings with more than 15 to 20 stories are usually designed with shear walls, primarily to improve their stiffness. Economically, the inclusion of shear walls seems to be the most expensive way to increase overall rigidity of concrete buildings, since they serve the triple function: to support gravity loads, to provide lateral resistance, and to function as a wall

Analysis for lateral loads of buildings containing shear was carried out initially, in the 1960s,by assigning all the lateral loads to the shear walls, since it was felt that the very big difference in stiffness between the shear walls and the frame would cause the shear walls to accept all total lateral loads. This inaccurate assumption may have been conservative for the computation of shear wall moments; It is, however, not conservative for frame, and particularly in the upper parts of the buildings.

The formal procedures for shear wall- frame interaction were first introduced in the early sixties. The concept of the interaction, with the resulting internal forces, which substantially increase the overall stiffness of the combined system.

Most of the recent prominent high-rise reinforced concrete buildings were built without any additional cost for the lateral resistance, which was mostly accommodated within the 1 percent increase in the allowable stresses when lateral loads are considered. The high lateral rigidity was achieved as a result of the shear wall-frame interaction.

2.2.2 Performance in Earthquakes.

To judge the merits of shear walls for earthquake resistance, an examination of their performance in earthquakes should be made, and particularly the comparative behavior of frame buildings and buildings containing shear walls during the earth-quakes of the last 10 years, starting with the earthquake of Managua, and going back in chronological order to the other earthquakes.

2.2.3 Managua earthquake, 1972

The earthquake of Managua of Dec. 23, 1972 seems to be the most significant of the recent earthquakes, since side by side were two buildings representing the two different structural systems. The building on the left is the 15-story Banco Central which is principally a frame building, while the building on the right is the Banco De-America, an 18-story shear wall-frame interactive system.

The Banco Central was designed in the early 1960s; the structural system of the tower is a one-bay frame with two small reinforced concrete cores and an in filled wall at the end of the building. These stiff elements may have done the structure more harm than good by introducing torsion due to their off-center location.

The building was subjected to violent shaking, as could be observed from the damage of the nonstructural elements, both in side and outside. Except for the 4th floor steel roof over the auditorium which fell off its support, the only structural damage observed in the building was some ripping between the off-center core and the slab. The structural frame of the tower it-self showed little distress. There was evidence of yielding at some junctions between the columns and the beams; however, the structure had enough ductility (whether designed for it or not) to sustain the large distortions. Inside, however, the building was in shambles over most of the stories, due to the extremely violent shaking experienced by the building.

In contrast, the building across the corner, the 18-story Banco De America, exhibited an entirely different performance, both inside and outside. The plan of the building contains four centrally located cores, arranged symmetrically within the peripheral column system. The cores are interconnected by two rows of stocky beams, many of which are penetrated by ducts. A general comment: the number of shear walls in this building is substantially higher than is usually provided in wind-resisting buildings of comparable height and plan size.

The interconnecting beams between the cores suffered repairable shear damage through most of the height of the building. Observing the large amount of flexural reinforcement in the beams, it should be recognized that it is almost impossible to provide enough shear capacity (whether the beams are penetrated by ducts or not) to be able to develop flexural hinging at the ends of the beams.

There was very little evidence throughout the height of building of any violent shaking. All the furniture was in place and there was no discernible non-structural damage.

A comparison of the above two buildings shows that both were well designed and well constructed, within several years of each other. Both buildings were designed according to the United States west coast standards in force at the time of their design. Although both buildings were subjected to the same earthquake motion, one had very severe architectural damage; the other could be reoccupied immediately while the repair proceeded. There was only one difference between the two buildings-a healthy system of shear walls which restricted the inter story distortions, thus providing damage control.

Another pair of comparative buildings from the Managua earthquake. Both of these 5-story buildings were located in areas of high damage. The 5-story Insurance Building outwardly appeared so badly damaged that people did not trust themselves to enter it for several days. Later inspection showed that, although the interior masonry walls and the interior partitions were badly damaged, there was a little structural damage to the building moment-resistant frame.

In contrast, the 5-story Enaluf building which has a relatively large reinforced concrete core in addition to the frame, went through the earthquake exceptionally well. The structural distress was light horizontal cracking in several of the exterior first story columns, and a shear wall on the ground floor had some damage adjacent to a duct penetration. Otherwise, neither structural nor serious nonstructural distress appeared in the building.

2.2.4. Performance of other Modern Time Building in Managua

There were also a number of well-designed moment-resistance frame buildings in the 15-story range. They were all subjected too intense shaking and were distortion as evidenced by no severe damage to their non-structural components. All of them exhibit sufficient ductility, since there was almost no structural distress. They all performed according to the present code philosophy little or no structural distress, however, quite a bit of non-structural damage.

The 8-story Supreme Justice building had minimal structural distress, but the inside was in shambles.

The 8-story Social Security Building outwardly showed no distress except for the collapsed roof -over the elevator machine room, and little structural distress on the inside. However, the nonstructural damage was considerable and the staircases were full of partition debris and could not have been used to evacuate people had the earthquake occurred during the day time.

The 8-story Telecommunications Building had post-tensioned beams on the column lines spanning across the entire building. There was an off-center core at the far end of the building. Substantial ripping apart between the core and the building was in evidence; however, the post - tensioned beams of the tower showed (with one exception) no structural distress. Inside, the nonstructural damage was substantial.

Summarizing the behavior of this group of frame buildings, it is evident that the buildings had enough ductility, whether intentionally designed for it or not, to sustain the large distortions to which they were subjected during the earthquake. The high degree of economic damage these buildings suffered was apparently not so much from the ground shaking as from an inadequate design philosophy—a philosophy in which we design the structure for large distortions, but we do not detail the rest of the building (which in many cases comprises up to 80 percent of the value) to accommodate the large distortions without damage.

Of particular interest was the performance of the National Theatre, built several years ago in the style of the Lincoln Center in New York.

The structure contains a U-shaped reinforced concrete shear well around the auditorium within another U-shaped shear wall of columns and beams in filled with 18-in.thick solid masonry around the lobby and stage. Except for a few marble statues, which were thrown off their pedestals, there was no evidence that the building had just gone through an earthquake. Neither structural nor nonstructural damage could be found.

2.2.5. San Fernando earthquake, 1971

The Indian Hill Medical Center is an example of an acceptable earthquake performance of a shear wall-frame type building. The building was restored and put back into operation within a short period after the earthquake. The structure of the Indian Hill Medical Center consists of beam column frames supplemented by shear walls. The shear walls exhibited some diagonal cracking and other local distress; they were repaired by increasing their thickness.

On the compound of the Veteran's Administration Hospital where some buildings of 1920 vintage collapsed, several auxiliary buildings built as reinforced concrete boxes went through the earthquake without structural damage; even the chimney of the central boiler plant had only some sliding at a cold joint. Unfortunately, these cases of excellent behavior somehow escaped the attention of the profession, which was busy examining the collapses.

2.2.6 Caracas, Venezuela earthquake in 1967

A multistory building with a very flexible skeleton of the type prevalent in Caracas. There were no shear walls used in the Caracas building. The skeletons were filled with brittle and weak hollow clay tile infill walls. During earthquake the buildings were subjected to large distortion and the weak partitions exploded. It is obvious that such damage should have been expected from building with flexible skeletons under going large distortions and filled with brittle partitions.

The 17-story Plaza One Building was the only complete shear wall building in caracas. It was located within an area of extremely high damage. One of its neighbors 10-story buildings, collapsed while the other surrounding structures suffered severe damage. The Plaza One Building went through the earthquake without any damage whatsoever. The building has shear walls in both directions.

2.2.7 Skopje, Yugoslavia earthquake 1963

Many of the residential buildings in Skopje up to 10 Storys high had two shear walls, all across the width, flanking the central stairway. The shear walls were usually non-reinforced of poor quality concrete. Nevertheless, the rigidity of the shear walls did not permit interstory distortions, and consequently, there was no damage. In some instances slip of the cold joints between successive story lifts was observed.

The 14-story Party Headquarters, which was the only building in Skopje with a structure similar to our shear wall frame systems. Being the party headquarters, it was deigned more carefully and constructed with greater attention to quality. Three non-reinforced shear walls in center were of good quality concrete. Although it is known that the building underwent severe shaking during the earthquake (according to witnesses who were thrown from one end of the room to the other), there was neither structural nor nonstructural damage in the building, with exception of the elevators, which did not function after the earthquake.

2.3 BEHAVIOR OF SHEAR WALLS

The preceding description of performance in past earthquakes showed many examples of good behavior of buildings containing shear walls. On the opposite side of the spectrum there are to distinct categories of misbehavior of shear wall buildings during earthquakes:

(a) Interrupted shear walls, and (b) brittle-linkage between coupled shear walls.

The Olive View Hospital (Sandarmando) is the example of interrupted shear walls. The upper four stories contained shear walls; however, they were undesirable in the II floor due to the architectural layout, and were, therefore omitted. The building distress during the earthquake by more than 2 feet within the ground story. In retrospect, it seems that in this building the ductility available in the upper stories was of little benefit because the presence of shear walls did not allow any distortions; while the extremely high amount of ductility available in the columns of the ground story did not really do very much good in keeping the building intact, except possibly preventing total collapse of the ground story.

2.4 DISCUSSION

If we look back and review what we have learned from the previous earthquakes, we believe that we were extremely eager during the last decade to verify the viability of the concept of the ductile moment-resistant frame. Consequently, after each of the past earthquakes, we have introduced some improvement to the ductile moment-resistant frame, i.e., after Caracas.

We increased substantially the overturning moment after other earthquakes, other details were modified. However, we believe that we have not thoroughly examined the basic concept of our earthquake design philosophy as related to reinforced concrete.

For, we believe if we were to make a thorough examination, would probably find that ductile moment-resisting frame without shear walls is a relatively poor structural system for residential and office building which contain a lot of nonstructural elements that are not designed and detailed to accommodate the large earthquake distortions of the moment-resistant frame.

A brief look at the history shows that the ductile moment resistant frame evolved in the1950s out of the moment-resistant frame, which at that time, was the only system for multi-story buildings for both steel and concrete. By adding ductility to the then available system, we created a convenient solution to the problem of earthquake resistance. However, in the mean time, better and more efficient structural systems for multistory structures (both in steel and concrete) were developed for wind resistance. Actually, the last moment-resistant frame utilized in the east for a very tall building was the 60-story steel framed Chase Manhattan Bank in New York, built in the early 1960s. At that time, this 60-story building utilized about 45 lb of steel per square foot of floor area (220kg/sqm.). Today, 60-story buildings are built with about 20 lb of steel per square foot (98kg/sqm), while buildings in the 100-story range are built with slightly over 30 lb of structural steel per square foot of floor area (146kg/sqm).

The emphasis on ductility as the key to survival of moment-resisting open frames led to the adoption of concrete structures reflecting characteristics more common to steel buildings, rather than taking advantage of the strength, stiffness, and ductility inherent in the natural forms to which concrete lends itself, such as shear walls. Recent experience has demonstrated that the twin requirements of safety and damage control can be better met by structures possessing adequate stiffness, such as shear Walls can most economically provide, when coupled with sufficient ductility or energy-absorption capacity. This is especially desirable for apartment and office buildings, where considerable nonstructural damage can result from excessive interstory displacements during an earthquake. When sufficient lateral stiffness is built into a structure by the introduction of ductile shear walls, so that large lateral displacements are prevented, it is doubtful if the connected frame in a frame-shear wall building will ever undergo the distortions which would call the ductility, which we now design into them.

It seems that for uses like parking, garages, stadium bridges, and the like, the ductile moment-resistant frame without shear wall is an excellent earthquake resistant system. However, for apartment and other buildings in which 80 percent of the value is nonstructural we should have more damage control than the ductile moment frame can provide.

2.5 BEHAVIOR OF WALL-FRAME STRUCTURE

2.5.1 INTRODUCTION

Wall-frame structures consist of a combination of shear walls and moment-resisting frames, which act jointly in resisting both gravity and horizontal loading. It is common in high-rise wall-frame structures to reduce in size and number, or to eliminate entirely, the shear walls in the upper part of the building where fewer elevator shafts are required. The behavior of such "curtailed" wall-frame structures are the topic of this paper.

It will be shown that the elimination or reduction in size of the shear Walls in the upper part of a wall-frame building structure is not necessarily detrimental to the deflection of the building. Providing the wall curtailment is made above a certain level, which depends on relative stiffness of the walls and frames, there will be an insignificant change in the top deflection and the forces in the structure

In wall-frame structures that are plan-symmetrical about the axis of loading, and therefore do not twist, the high in-plane stiffness of the floor slab causes the lateral deflections of the walls and frames to be effectively identical.

When a wall-frame structure is loaded laterally, the lower part of the structure deflects in a flexural configuration, i.e., concavity downwind, and

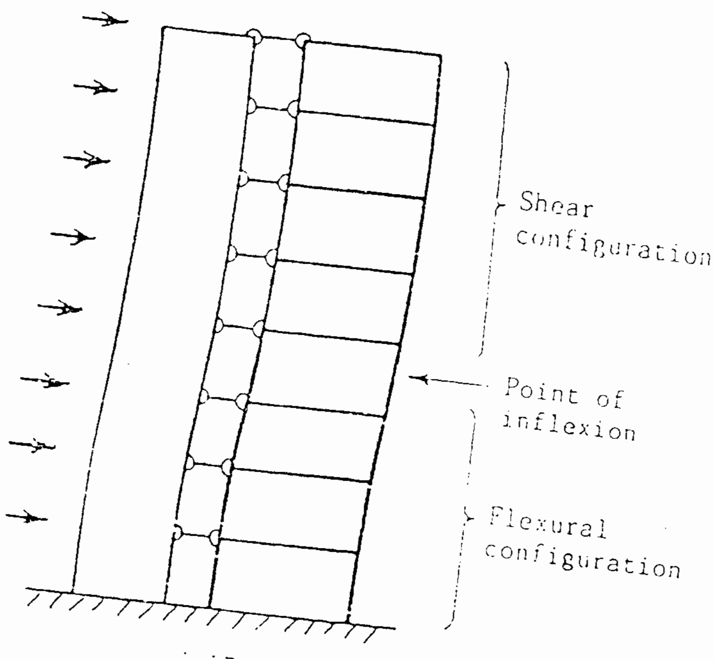


Fig.2.1 -Typical Deflected Shape of wall -frame structure

the upper part in a shear configuration, i.e., concavity upwind, with a point of inflection at the transition. (Fig2.1).The greater the racking shear rigidity of the frames relative to the flexural rigidity of the walls ,the lower the level of the point of inflection.

When a wall-frame structure deflects laterally under horizontal loading, horizontal interaction forces occur between the walls and frames. Typical distributions of these interaction forces for the planar model of a uniform wall-frame structure, and the resulting shear forces and bending moment carried by the walls and frames are shown in Figs.2.2 (a),*(b)*, and (c), respectively.

In real wall-frame structures, with the walls and frames in parallel planes, the horizontal interaction forces are transferred between the walls and the frames through horizontal shear in the floor slabs, and they vary incrementally from floor to floor through the height. At the top level, a relatively large concentrated interaction force acts between the wall and the frame, with a sense corresponding to the frame restraining the wall from deflecting.

2.6 BEHAVIOR OP CURTAILED WALL-FRAME STRUCTURE

The behavior of wall frame structure curtailed walls is not obvious. An understanding is made easier, however, by first reviewing known behavior of the corresponding full-height wall-frame structure.

Referring to the distribution of bending moment for the full-height-wall structure (fig.2.2(c); the wall moment in the region above the point of inflection where bending moment is opposite in sense to the external load moment, while the moment in the frame (which is carried mainly by axial forces in the columns) is actually greater than the external load moment. Therefore, if the wall were curtailed anywhere in region above the point

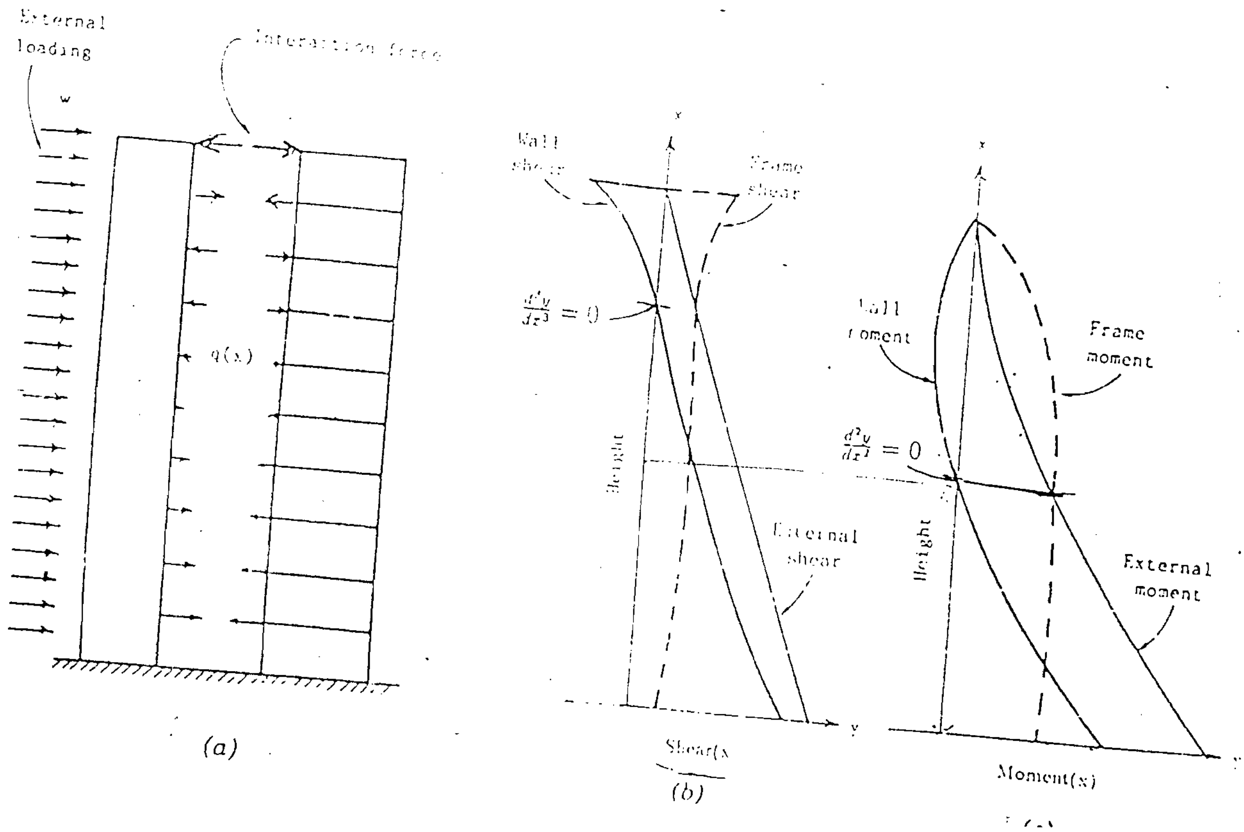


Fig.2.2 -Forces in Uniform Wall - frame structure:
(a)Horizontal interaction between wall and frame
(b)Typical Distribution of shear in wall and frame
(c)Typical Distribution of moment in wall and frame

inflection, the moment carried by the frame would be reduced to become equal to the external moment.

Similarly, for the distribution of the shear force in the full height wall structure [fig.2.2 (b), the shear in the wall above the point of zero shear, where S.F. =0, opposite in sense to the external load shear, while the shear in the frame exceeds the external shear. Therefore, if the wall were curtailed anywhere in that uppermost region, the shear in the frame would be reduced to become equal lo the external load shear.

An inspection of Fig.2.2 (b) and 2.2(c) shows that if the wall were curtailed between the points of zero shear and inflection, the shear in the frame above the curtailment level would be increased by a small amount while the moment in the frame above that level would be reduced. If the wall were curtailed below the point of contra flexure, both the shear and the moment in the frame would increase.

On the basis of the preceding discussion, it is evident that the levels of zero moment and zero shear in the wall of the full-height-wall structure should be taken as levels of reference in assessing the effects of curtailing the wall on the shear and moment distribution.

The effect of wall curtailment on the top deflection of the whole structure may be regarded in another way as the effect on the top deflection of the frame due to changes in the distributions of shear and moment in the frame caused by curtailment of the wall. Any significant modification to those force distributions could lead to significant changes in the top deflection.

Since curtailment of the wall between the two reference levels should produce changes in the force distributions that are either beneficial, or of little detriment to the frame, the resulting change in top deflection of the frame should also be small.

Considering that the effects of curtailment vary according to the level of curtailment, there should be an optimum level of curtailment that produces minimum changes in the force distributions, and consequently in the top deflection. In order lo investigate the effects of curtailment in more detail and to assess the behavior of curtailed wall-frame structures, a mathematical solution for a continuum model of the curtailed structure is developed.

CHAPTER -3

Modeling of Wall- Frame Structure

3.1 Creating a New Structure

The structure type is to be defined by choosing from among *Space, Plane, Floor* and *Truss.* A *Space* type is one where the structure, the loading or both, cause the structure to deform in all 3 global axes (X, Y and Z).

We choose meter as the length unit and KN as the force unit in which we will start to build the model. The units can be changed later if necessary, at any stage of the model creation.

In the next dialog box, we choose the tools to be used to initially construct the model. Add Beams, Add Plates or Add Solids are respectively, the starting points for constructing beams, plates or solids. Open *Structure Wizard* provides access to a library of structural templates, which the program comes equipped with. Those template models can be extracted and modified parametrically to arrive at our model geometry or some of its parts.

3.2 STEP INVOLVED IN MODELING

In order to generate the model graphically, the structure geometry consists of joint numbers, their coordinates, member numbers, the member connectivity information, surface element etc.

(i) The joint coordinate's specification is used to specify the x, y, and z coordinates of the joints.

(ii) The member incidence specification is used for specifying member connectivity or defined the member by joints they are connected to.

(iii) Select the beam element, click translation repeat and specified the number of stories, height of the individual story. After completing translation repeats, the 3-D model is obtained.

(iv) All member properties of provided using the prismatic option YD and ZD stand for depth and width. All properties are calculated automatically from their dimension unless different sets of value of the properties are defined.

(v) The constant command initiates input for material constant, like E 'modulus of elasticity', Poisson's ratio, density etc. the constant specification is use to specify material properties. In this case, the default value has been used.

(vi) The supports of the structure are defined through supports specification. Here all the support is fixed.

(vii) Shear wall are modeled in STAAD Pro using the new basic element type surface. The surface element is in many ways similar to the plate element, particularly with respect to the modeling task. (e.g. adding, selecting, deleting).

In order to facilitate rapid modeling of complex walls and slabs, a new type of entity has been introduced - Surface. At the modeling level it corresponds to the entire structural part, such as a wall, floor slab or bridge deck. At the analysis level, it is first decomposed into a number of triangular plate elements. Thus the Surface is a super element for modeling purposes (it is composed from a number of plate elements). Consequently, the user has the convenience of specifying only one large structural component per wall or slab, yet may maintain full control over the computational accuracy by setting the desired number of finite element divisions. In the current release of the program, the Surface element is limited to a rectangular shape and no openings within its boundaries are permitted. In the future, however, these restrictions will be eliminated.

Surface element adds a number of important enhancements to the finite element analysis reporting. It is now possible to obtain in-plane bending moments as well as stresses along any arbitrary line cutting the surface.

3.3.Description

The surface definition must comprise a minimum of four nodal points forming corners of a rectangle. However, any number of additional nodes may be incorporated into the surface boundaries provided the nodes are collinear on edges they belong to. In addition, the user specifies the number of edge divisions that will be the basis for triangular mesh generation. A single command per wall is used for this purpose. The program will subdivide all edges into the requested number of fragments and each of these fragments will become an edge of a triangular plate element. However, if the original surface edges have additional nodal points between the corners, all node-to-node lengths of the surface edge will be divided into the same number of fragments. Surface elements may be loaded by uniformly distributed loads in any global direction or by loads normal to the plane. The program includes a new support generation function that allows a quick assignment of support specifications to multiple nodal points.

3.4. Surface Element Specification.

Surface, as a new entity, requires new syntax to define its geometry and properties. In most cases it is very similar to the plate element syntax, which the user is familiar with.

First, the number of divisions for each node-to-node segment of the surface boundary is indicated by the **SET DIVISION** command. Then, the joint connectivity is specified by the **SURFACE INCIDENCES** command to define boundaries of the surface. The joints to be included in boundary definition do not include joints generated by the **SET DIVISION** as those are only used by internal routines of the program.

(viii) A structural model of the multi story building is created first. We will need surface elements stacked on top of each other to allow for the connection of the wall to the slab. The first part of shear wall will extend from foundation to the top floor level.

This will assign fixed support to the all nodes between the node number 2 and node number 5 including nodes created internally by program conjunction with surface meshing. Shear walls can model without surface element to introduce shear walls in the form member element.

(ix) Insert the shear walls in the form member element.

(x) Once the elements are created we will proceed to assign thickness and material property to them

(xi) Three load cases are to be created for this structure, dead load(including self weight), live load and earthquake load. In load case one and two in this problem; a floor load generation is performed. In a floor load generation a pressure load (force/unit area) is converted by a program into specific point and distributed forces on the member located in that region. The Y- range specifications are used to define the area of the structure on which the pressure is acting. The floor load specification is used to specify the value of that pressure.

(xii) Generation of earthquake loading the analysis of building for earthquake loads is done accordance with IS: 1893-1984. The building analyzed by seismic coefficients method.

The first step in the seismic analysis is to determine join weight, this can be done by applying a pinned support to the all joints of space frame and Then analyze the building for the combination of dead load + 0.5 live load, and after analysis we get the reaction at all joints in form of joint weights.

(xiii) By using these joint weights STAAD Pro. It self calculates the base shear.

(xiv) Define zone factor, importance factor, soil factor and performance factor, and make different load combinations for different loads

(xv) Perform P delta analysis; this command instructs the program to proceed with analysis. The analysis type is P-Delta indicating that second order effects are to be calculated.

4.1 INPUT FILE OF EXAMPLE 2 FOR CASE

```
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START JOB INFORMATION
ENGINEER DATE 15-Mar-04
END JOB INFORMATION
INPUT WIDTH 79
UNIT METER KN
JOINT COORDINATES
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9 11 0 6; 10 16 0 6; 11 22 0 6; 12 27 0 6; 13 0 0 11; 14 5 0 11; 15 11 0 11;
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765 WEIGHT 222.954
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768 WEIGHT 144.623
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783 WEIGHT 147.639
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787 WEIGHT 164.716
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792 WEIGHT 223.834
793 WEIGHT 223.078
794 WEIGHT 223.078
795 WEIGHT 223.835
796 WEIGHT 144.572
797 WEIGHT 144.572
798 WEIGHT 223.834
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814 WEIGHT 166.056
815 WEIGHT 164.947
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840 WEIGHT 148.538
841 WEIGHT 78.525
842 WEIGHT 155.235
843 WEIGHT 152.006
844 WEIGHT 152.006
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863 WEIGHT 155.235
864 WEIGHT 78.525
865 WEIGHT 113.201
866 WEIGHT 113.201
867 WEIGHT 113.201
868 WEIGHT 113.201
LOAD 1 SISX LOAD
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LOAD 2 SISZ LOAD
1893 LOAD Z

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LOAD 3 DEAD LOAD
FLOOR LOAD
YRANGE 0 91 FLOAD -4.75
SELFWEIGHT Y -1
LOAD 4 LIVE LOAD
FLOOR LOAD
YRANGE 0 91 FLOAD -4
LOAD COMB 5 1.5(DL+SISX)
3 1.5 1 1.5
*LOAD COMB 5 1.5(dl+sisx)
*3 1.5 1 1.5
*LOAD COMB 3 dl+.5ll
*1 1.0 2 0.5
PDELTA ANALYSIS
LOAD LIST 5
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CHAPTER -5

RESULTS AND DISCUSSION

5.1 GENERAL

Two examples have been presented in this chapter and it is analyzed by STAAD. Pro. 2004. In this chapter, the effects on bending, shear force and deflection in a shear wall and frame, when there is no shear wall and variation in point of contraflexure for symmetrical as well as unsymmetrical building and when symmetrical building is made unsymmetrical by removing a shear wall, have been studied through these example.

5.2 EXAMIPLES

5.2.1 EXAMIPLE I

5.2.1.1 Details of the Building

Fig. 5.1 shows a typical floor plan of the building. In cross direction, building consists of three types of frame. Frame 'A' consists of no shear wall, frame 'C' consists of shear wall in the middle and frame 'D' consists of one shear wall a the end. Various cross-sectional parameters and material

Properties of the structure are as follows -

Column size - 0.45m x 0.75m Storey height - 3.0m

Beam size - 0.30m x 0.50m

Shear wall thickness - 0.15m

5.2.1.2. Analysis and Results

Case -1: When shear wall is right upto top storey

Out these frame, the result of frame D is shown. The shear force, bending moment and deflection carried by shear wall is given in table 5.1 to 5.15. The variation of point of cotraflexure is given in table 5.16 and the variation of bending moment and shear force in shear walls with change in thickness is given in table 5.17. The variation of point of cotraflexure is shown in Fig. 5.3 and the variation of bending moment and shear force for different storeys is shown in Fig. 5.4 to 5.33

Case - 2: When there is no shear wall in top storeys.

The same example has been analyzed when there is no shear wall in top storeys.The shear carried by frames at each floor level in two cases is given in Table 5.18. The shear force in shear wall at each floor level in two cases is given in Table 5.19. The variation of shear force at various floor levels for the two cases is also shown in Fig. 5.34(a) and (b). The bending moment in shear wall at each floor level in two cases is also given in Table 5.19. The variation of bending moment at various floor levels for two cases is also shown in Fig.5.35.(a) and (b). The horizontal deflection at each floor level in two cases is also shown in Table 5.20.

5.2.2 Example 2

5.2.2.1 Details of the Building

Fig. 5.1(a) shows a typical floor plan of the building. In this structure, there are two types of frames. Frame A. has five bays with no shear walls, and frame B has three bays with two shear walls. Various cross-sectional parameters and material

Properties of the structure are as follows -

Column size - 0.45m x 0.75m Storey height - 3.0m

Beam size - 0.30m x 0.50m Shear wall thickness - 0.15m

5.2.2.2. Analysis and Results

Case -1: When all the shear walls are there maintaining symmetry.

Out these frame, the result of frame C is shown.

Case-2: When the building made unsymmetrical by removing one shear wall.

The same example is analyzed by removing the shear wall along frame 'B' between 4 and 5 which makes the building unsymmetricalas shown in Fig.5.1(b)

The shear force in shear wall at each floor level in case 1 and 2 is given in table 5.21. The variation of shear force at various floor levels for the cases 1 and 2 is shown in Fig.5.36 (a) and 5.36(b).

The bending moment in shear wall at each floor level in case 1 and 2 is given in table 5.21. The variation of bending moment at various floor levels for the cases 1 and 2 is shown in Fig. 5.37(a) and 5.37(b).

The horizontal deflection at each floor level in two cases is given in

The horizontal deflection at each floor level in two cases is given in table 5.21.

Case -3: When shear wall is right up to top *storey*

Out these frame, the result of frame C is shown. The shear force, bending moment and deflection carried by shear wall for different storeys is given in table 5.22 to 5.36. The variation of point of cotraflexure is given in table 5.37. The variation of bending moment and shear force for different storeys is shown in Fig. 5.38 to 5.67. The variation of point of cotraflexure is shown in Fig. 5.68.

Case - 4 : When there is no shear wall in top storeys.

The same example has been analyzed when there is no shear wall in top storeys.The shear carried by frames at each floor level in two cases is given in Table 5.38. The shear force in shear wall at each floor level in two cases is given in Table 5.39. The variation of shear force at various floor levels for the two cases is also shown in Fig. 5.69(a) and (b). The bending moment in shear wall at each floor level in two cases is also given in Table 5.39. The variation of bending moment at various floor levels for two cases is also shown in Fig.5.70(a) and (b). The horizontal deflection at each floor level in two cases is also shown in Table 5.40.

5.3 DISCUSSION OF RESULTS

The three-dimensional frame shear wall is analyzed by using STAAD Pro. 2004. The results presented in this chapter earlier pertain to two buildings. One is an unsymmetrical multistory building with two cases where shear walls extend upto the top and secondly where they are not provided in top stories and effect of change in thickness of shear walls have been considered. In each case, shear taken by one of the frames, namely, frame D is analyze by using the STAAD Pro. 2004. The second building is symmetrical with four cases. In then the first case, four shear wall provided symmetrically on two column lines. In the second case, one of shear walls is removed tc introduce unsymmetry. In this example, the curtailment of shear walls has been considered for symmetrical building. The frame 'C' is analyzed by STAAD Pro. 2004. These results are discussed below.

5.3.1 EFFECT OF REMOVAL OF SHEAR WALL IN TOP STOREYS

When shear wall is removed in Top stories, there is a marginal increase in shear in the shear wall at lower stories. There is also an increase in shear resisted by the frame as shown in Table 5.18.and 5.38. When the shear wall is removed.

The variation of bending moment in shear wall in the two cases of example 1 and example 2 (Fig. 5.35(a),(b) Fig. 5.70(a),(b)) shows that there will be a marginal increase in bending moment in the shear wall at lower levels when it is removed from the top stories.

In the two cases, it is seen that the horizontal deflections at various levels do not differ much. The percentage difference at the top most level is only 3.5% in this particular case, which is very small. Hence it can be said that the removal of shear wall from the top stories of these frame resulted in nominal increase in horizontal deflection.

5.3.2 EFFECT ON SHEAR WALL WITH CHANGE IN THICKNESS.

When the thickness of shear wall changes then bending moment and shear force of shear walls also changes as shown in Table 5.17. The point of contraflexure moves upward with the increase in thickness and the bending moment and shear force resisting capacity of shear walls also increases but resisting capacity of frame decreases

5.3.3 EFFECT ON POINT OF CONTRAFLEXURE WITH INCREASE IN NO. STOREYS ON BOTH EXAMPLES

The point of contraflexure in both cases move upward with the increase in No. of storeys i.e. more number of storeys can curtailed. The graph in symmetrical as well in unsymmetrical building is linear as . There is also an increase in shear ,bending moment resisted by frame and shear wall as shown in table , when the number of storeys is increases.

5.3.4 EFFECT OF UNSYMMETRY

When the symmetrical building is made unsymmetrical, there will be some increase (3.6%) in shear through out the height of the shear wall as the load coming on this particular frame increases. The bending moment in the second case increases with respect to the first case as shown in Fig. 5.37(a) and 5.37(b). There is some increase in bending moment through out the height of the shear wall in the second case.

In the two cases, the horizontal deflections at various floor levels do not differ much. The percentage difference at the topmost level is small.

5.4. CONCLUSION

In the present study, the 3-dimensional frame-shear wall structure has been analyzed by using STAAD Pro 2004.The forces in these structural components found by this program. Two examples for different cases have been solved by using STAAD Pro 2004.

From the results of examples solved, the following conclusions are drawn.

1. It can be concluded from the above study, that 85 % of the lateral load will be resisted by shear walls.

2. The reversal in the direction of bending moment at top levels of the shear wall indicates that shear wall has a tendency to behave as a propped cantilever.

3. It may be advantageous to curtail the shear wall at top levels as shear wall takes most of the shear in lower portion of the building. Whereas frame participates effectively in the upper portion.

4. When shear wall is removed in top stories, there will be nominal increase in horizontal deflection at each floor level.

5. When shear wall is removed in top storeys, there will be nominal increase in bending moment and shear each floor level.

6. The variation of point of contraflexure for both symmetrical and unsymmetrical buildings is linear.

7. The bending moment and shear force resisted by the shear walls increases but that of frame decreases when thickness of shear wall increases.

8. When symmetrical building is made unsymmetrical, there will be some increase in shear force and bending moment in shear wall through out the height of the shear wall.

TABLE 5.1

B.M., S.F.,and Deflection of 16 storey of Example 1 of case 1

Floor Level	B.M.(KN-m)	S.F.(KN)	Deflection(cm)
1	3303	260	0.001
2	2584	236	0.003
3	1968	201	0.005
4	1476	173	0.008
5	1078	149	0.011
6	753	129	0.014
7	484	111	0.018
8	259	95	0.021
9	71	79	0.025
10	-165	63	0.028
11	-273	48	0.031
12	-346	28	0.035
13	-381	7	0.038
14	-373	14	0.041
15	-316	-40	0.044
16	-207	-74	0.046

TABLE 5.2

B.M., S.F.,and Deflection of 17 storey building of Example 1of case 1			
Floor Level	B.M.	S.F.	Deflection
	(KN-m)	(KN)	(cm)
1	3324	259	0.001
2	2609	234	0.003
3	1998	199	0.005
4	1512	171	0.008
5	1119	147	0.011
6	799	127	0.014
7	535	110	0.018
8	314	94	0.021
9	128	80	0.025
10	-112	65	0.029
11	-225	51	0.032
12	-310	35	0.036
13	-364	18	0.039
14	-383	-7	0.042
15	-381	-20	0.045
16	-310	-44	0.048
17	-199	-74	0.051

TABLE 5.3

B.M., S.F.,and Deflection of 18 storey of Example 1of case 1			
Floor Level	B.M.	S.F.	Deflection
	(KN-m)	(KN)	(cm)
1	3343	257	0.001
2	2632	233	0.003
3	2026	198	0.005
4	1544	170	0.008
5	1156	146	0.011
6	841	126	0.015
7	582	109	0.018
8	365	94	0.022
9	182	80	0.026
10	59	67	0.029
11	-176	54	0.033
12	-268	40	0.037
13	-335	26	0.04
14	-373	9	0.044
15	-374	-7	0.047
16	-360	-26	0.05
17	-303	-48	0.053
18	-191	-74	0.056

TABLE 5.4

B.M., S.F.,and Deflection of 19 storey of Example 1of case 1

Floor Level	B.M.	S.F.	Deflection
	(KN-m)	(KN)	(cm)
1	3359	255	0.001
2	2653	231	0.003
3	2050	196	0.005
4	1573	168	0.008
5	1190	144	0.011
6	879	125	0.015
7	624	108	0.019
8	412	93	0.022
9	233	80	0.026
10	79	68	0.03
11	-127	57	0.034
12	-223	44	0.038
13	-297	31	0.041
14	-350	17	0.045
15	-376	2	0.048
16	-378	-13	0.051
17	-356	-30	0.055
18	-294	-50	0.058
19	-183	-73	0.06

TABLE 5.5

B.M., S.F.,and Deflection of 20 storey of Example 1 of case 1

Floor Level	B.M.	S.F.	Deflection
	(KN-m)	(KN)	(cm)
1	3373	254	0.001
2	2671	230	0.003
3	2073	195	0.005
4	1599	166	0.008
5	1220	143	0.011
6	914	123	0.015
7	664	107	0.019
8	456	92	0.023
9	280	80	0.027
10	129	69	0.031
11	-79	58	0.035
12	-176	47	0.039
13	-256	36	0.042
14	-317	24	0.046
15	-357	10	0.05
16	-374	-3	0.053
17	-378	-19	0.056
18	-349	-34	0.059
19	-285	-53	0.063
20	-174	-73	0.066

TABLE 5.6

B.M., S.F.,and Deflection of 21 storey of Example 1 of case 1

Floor Level	B.M.	S.F.	Deflection
	(KN-m)	(KN)	(cm)
1	3385	253	0.001
2	2687	228	0.003
3	2093	193	0.005
4	1623	165	0.008
5	1248	141	0.012
6	946	122	0.015
7	700	106	0.019
8	496	92	0.023
9	324	80	0.027
10	176	69	0.031
11	48	59	0.035
12	-130	49	0.039
13	-212	40	0.043
14	-279	29	0.047
15	-329	17	0.051
16	-360	5	0.055
17	-379	-9	0.058
18	-376	-24	0.061
19	-342	-38	0.065
20	-275	-55	0.068
21	-166	-73	0.071

TABLE 5.7

B.M., S.F.,and Deflection of 22 storey of Example 1of case 1

Floor Level	B.M.	S.F.	Deflection
	(KN-m)	(KN)	(cm)
1	3396	251	0.001
2	2702	227	0.003
3	2111	192	0.005
4	1646	164	0.008
5	1274	140	0.012
6	976	121	0.015
7	735	104	0.019
8	535	90	0.023
9	364	79	0.028
10	220	69	0.032
11	94	59	0.036
12	-84	50	0.04
13	-168	41	0.044
14	-238	32	0.048
15	-295	22	0.052
16	-336	11	0.056
17	-369	-0.82	0.06
18	-380	-14	0.063
19	-372	-27	0.067
20	-339	-41	0.07
21	-365	-56	0.073
22	-158	-71	0.076

TABLE 5.8

B.M., S.F.,and Deflection of 23 storey of Example 1 of case 1			
Floor Level	B.M.	S.F.	Deflection
	(KN-m)	(KN)	(cm)
1	3455	254	0.001
2	2754	229	0.003
3	2158	193	0.006
4	1689	165	0.009
5	1315	141	0.012
6	1016	121	0.016
7	774	105	0.02
8	575	92	0.024
9	408	80	0.028
10	265	70	0.033
11	140	61	0.0371
12	31	53	0.041
13	-128	45	0.046
14	-201	36	0.05
15	-262	27	0.054
16	-311	17	0.058
17	-345	7	0.062
18	-375	-5	0.066
19	-386	-18	0.069
20	-372	-31	0.073
21	-330	-44	0.076
22	-260	-59	0.08
23	-164	-72	0.083

TABLE 5.9

B.M., S.F.,and Deflection of 24 storey of Example 1 of case 1			
Floor Level	B.M.	S.F.	Deflection
	(KN-m)	(KN)	(cm)
1	3629	265	0.001
2	2896	239	0.003
3	2274	202	0.006
4	1786	172	0.009
5	1398	147	0.013
6	1088	127	0.017
7	838	110	0.021
8	634	96	0.025
9	463	84	0.03
10	317	74	0.035
11	190	65	0.039
12	78	57	0.044
13	-94	49	0.049
14	-171	41	0.053
15	-238	33	0.058
16	-294	24	0.062
17	-338	15	0.066
18	-368	4	0.07
19	-397	-8	0.074
20	-404	-21	0.078
21	-386	-34	0.082
22	-340	-46	0.086
23	-266	-61	0.089
24	-155	-74	0.093

TABLE 5.10

B.M., S.F.,and Deflection of 25 storey of Example 1 of case 1

Floor Level	B.M.	S.F.	Deflection
	(KN-m)	(KN)	(cm)
1	3804	277	0.001
2	3040	250	0.003
3	2392	211	0.006
4	1884	180	0.01
5	1481	154	0.013
6	1161	132	0.018
7	904	115	0.022
8	693	100	0.027
9	519	88	0.031
10	370	78	0.036
11	241	69	0.041
12	127	62	0.046
13	25	54	0.051
14	-137	46	0.056
15	-208	39	0.061
16	-270	31	0.066
17	-323	22	0.071
18	-363	12	0.075
19	-399	2	0.079
20	-398	-10	0.084
21	-421	-23	0.088
22	-399	-36	0.092
23	-349	-49	0.096
24	-271	-63	0.099
25	-158	-76	0.103

TABLE 5.11

B.M., S.F.,and Deflection of 26 storey of Example 1 of case 1

Floor Level	B.M.	S.F.	Deflection
	(KN-m)	(KN)	(cm)
1	3981	289	0.001
2	3184	260	0.004
3	2510	220	0.006
4	1982	187	0.01
5	1565	160	0.014
6	1235	138	0.018
7	969	119	0.023
8	754	105	0.028
9	575	93	0.033
10	424	82	0.038
11	293	73	0.044
12	178	65	0.049
13	73	58	0.054
14	-101	51	0.06
15	-175	44	0.065
16	-242	37	0.07
17	-300	29	0.075
18	-348	20	0.08
19	-386	10	0.085
20	-417	-0.85	0.089
21	-438	-13	0.094
22	-437	-26	0.098
23	-412	-39	0.102
24	-358	-51	0.106
25	-276	-66	0.11
26	-160	-77	0.114

TABLE 5.12

B.M., S.F.,and Deflection of 27 storey of Example 1 of case 1			
Floor Level	B.M.	S.F.	Deflection
	(KN-m)	(KN)	(cm)
1	4158	301	0.002
2	3329	271	0.004
3	2630	228	0.007
4	2002	194	0.01
5	1650	166	0.015
6	1309	143	0.019
7	1036	124	0.024
8	815	109	0.029
9	632	96	0.035
10	479	86	0.04
11	346	77	0.046
12	229	69	0.051
13	123	62	0.057
14	27	55	0.063
15	-139	49	0.068
16	-209	42	0.074
17	-272	35	0.079
18	-327	27	0.085
19	-372	18	0.09
20	-406	8	0.095
21	-478	-3	0.1
22	-456	-15	0.104
23	-452	-28	0.109
24	-423	-41	0.113
25	-366	-53	0.116
26	-281	-68	0.122
27	-162	-79	0.126

TABLE 5.13

B.M., S.F.,and Deflection of 28 storey of Example 1 of case 1			
Floor Level	B.M.	S.F.	Deflection
	(KN-m)	(KN)	(cm)
1	4336	312	0.002
2	3476	281	0.004
3	2750	237	0.007
4	2183	202	0.011
5	1736	173	0.015
6	1384	149	0.02
7	1103	129	0.025
8	876	113	0.031
9	690	100	0.036
10	533	90	0.042
11	399	81	0.048
12	281	73	0.054
13	174	66	0.06
14	76	60	0.066
15	-101	53	0.072
16	-173	47	0.078
17	-239	40	0.084
18	-299	33	0.09
19	-351	25	0.095
20	-394	16	0.101
21	-426	6	0.106
22	-466	-5	0.111
23	-473	-17	0.116
24	-466	-30	0.121
25	-434	-43	0.126
26	-374	-55	0.13
27	-286	-70	0.135
28	-164	-81	0.139

TABLE 5.14

B.M., S.F.,and Deflection of 29 storey of Example 1 of case 1			
Floor Level	B.M.	S.F.	Deflection
	(KN-m)	(KN)	(cm)
1	4516	324	0.002
2	3623	292	0.004
3	2871	246	0.007
4	2284	209	0.011
5	1823	179	0.016
6	1459	154	0.021
7	1171	134	0.026
8	938	118	0.032
9	748	104	0.038
10	589	94	0.044
11	453	84	0.05
12	333	76	0.057
13	226	69	0.063
14	127	63	0.069
15	36	57	0.076
16	-135	51	0.082
17	-204	45	0.088
18	-267	38	0.094
19	-324	31	0.1
20	-374	23	0.106
21	-415	14	0.112
22	-444	5	0.118
23	-476	-7	0.123
24	-489	-19	0.128
25	-479	-32	0.134
26	-445	-45	0.138
27	-381	-57	0.143
28	-291	-71	0.148
29	-165	-82	0.153

TABLE 5.15

B.M., S.F.,and Deflection of 30 storey of Example 1 of case 1			
Floor Level	B.M.	S.F.	Deflection
	(KN-m)	(KN)	(cm)
1	4697	336	0.002
2	3772	302	0.004
3	2993	255	0.008
4	2386	217	0.012
5	1910	185	0.017
6	1536	160	0.022
7	1239	139	0.028
8	1001	122	0.033
9	807	108	0.04
10	645	97	0.046
11	507	88	0.053
12	386	80	0.059
13	278	73	0.066
14	179	67	0.073
15	87	61	0.079
16	-95	55	0.086
17	-165	49	0.093
18	-231	43	0.099
19	-292	37	0.106
20	-347	30	0.112
21	-395	22	0.119
22	-434	13	0.125
23	-461	3	0.13
24	-494	-8	0.136
25	-504	-21	0.142
26	-492	-34	0.147
27	-454	-47	0.152
28	-388	-59	0.157
29	-295	-73	0.162
30	-167	-84	0.167

TABLE 5.16

Variation of inflection point with Number of Storeys for example 1 of case 1	
Floor Level	**Inflection Point(m)**
15	24.24
16	24.91
17	25.601
18	27.4
19	28.16
20	28.87
21	30.812
22	31.59
23	33.59
24	34.369
25	36.46
26	37.26
27	39.48
28	40.29
29	42.63
30	43.43

TABLE 5.17

Variation of Bending Moment & Shear Force in Shear wall with Change in thickness

	D=0.175		D=0.2		D=0.225		D=0.25		D=0.275	
F.L.	B.M (KN-m)	S.F (KN)	B.M (KN-m)	S.F (KN)	B.M (KN-m)	S.F (KN)	B.M (KN-m)	S.F (KN)	B.M (KN-m)	S.F (KN)
1	3641	270	3961	279	4262	287	4548	294	4822	301
2	2883	247	3176	256	3451	265	3718	273	3964	280
3	2234	213	2489	223	2733	232	2968	241	3195	248
4	1702	185	1920	195	2131	204	2336	213	2536	221
5	1263	160	1444	170	1622	108	1797	188	1938	196
6	898	139	1044	149	1190	158	1334	166	1477	174
7	593	120	706	129	821	138	937	145	1053	153
8	337	103	419	111	506	119	596	126	687	133
9	127	86	177	93	239	100	305	106	373	112
10	-137	69	-102	75	17	81	62	36	110	92
11	-264	51	-248	56	-225	60	-197	65	-164	69
12	-352	31	-350	35	-342	38	-328	42	-310	46
13	-397	10	-405	12	-407	14	-404	17	-397	20
14	-393	-13	-407	-12	-415	-11	-419	-9	-420	-7
15	-332	-40	-344	-40	-352	-40	-357	-39	-360	-38
16	-218	-76	-266	-77	-232	-78	-237	-79	-260	-80

TABLE 5.18

Comparision of Shear Force,Bending Moment & Deflection in Frame at Various Floor Levels of 25 storey for example 1 in two cases

Floor Level	S.F.(KN)	B.M.(KN-m)	Deflection(cm)	S.F.(KN)	B.M.(KN-m)	Deflection(cm)
1	9.74	30.9	0	10.3	30.1	0
2	14.7	32	0.0528	17.3	34.5	0.059
3	22	41.4	0.1793	26.7	47.3	0.189
4	26.2	46	0.3978	33.4	55.7	0.3698
5	28.8	48.6	0.5612	38.9	62.5	0.5873
6	30.3	49.6	0.8048	43.6	68.4	0.831
7	30.8	49.5	1.0713	47.9	79	1.0932
8	30.7	48.7	1.3548	52.3	79.6	1.3678
9	30.2	47.2	1.6507	57	85.9	1.6503
10	29.3	48.3	1.9553	62.4	93	1.9368
11	28.2	43.2	2.2653	69	102	2.2244
12	26.8	40.7	2.5782	74.4	111	2.5107
13	25.4	38.1	2.8913	104	137	2.7937
14	23.7	-35.9	3.2027	12	-103	3.0735
15	22	-33.6	3.5702	6.95	-63	3.418
16	20.2	-31.1	3.8121	4.01	-54	3.88
17	18.2	-28.8	4.1065	1.95	-49	4.2972
18	16.2	-28.7	4.3919	0.738	-38.8	4.7267
19	14.2	-22.8	4.667	0.188	-32.8	5.1265
20	12.2	-19.9	4.9305	-0.028	-29.8	5.494
21	10.3	-17	5.1818	-0.157	-28.4	5.829
22	8.64	-14.3	5.9205	-0.345	-20.8	6.1304
23	7.32	-12.1	5.6469	-0.654	-14.2	6.3966
24	6.16	-9.87	5.8619	-1.13	-10.8	6.6266
25	7.81	-13.5	6.0675	-1.9	-2.02	6.8234
			6.2665			6.9977

Comparision of Shear Force,Bending Moment & Deflection in Frame at Various Floor Levels of 15 storey for example 1 in two cases

Floor Level	S.F.(KN)	B.M.(KN-m)	Deflection(cm)	S.F.(KN)	B.M.(KN-m)	Deflection(cm)
1	8.94	27.1	0	9.47	27.1	0
2	13.3	27.6	0.0672	15.3	29.9	0.0705
3	19.8	36	0.2205	23.5	40.8	0.2285
4	23.4	39.7	0.4369	28.9	47.3	0.4791
5	25.4	41.3	0.6995	33	52.1	0.7146
6	26.1	41.4	0.9944	36.2	57.7	1.0107
7	25.9	40.1	1.3102	38.8	58.8	1.3263
8	25	37.9	1.6378	41.5	64	1.6528
9	23.5	-35.5	1.9695	42.9	-61.1	1.9836
10	21.7	-33.2	2.306	50.2	-62.6	2.3151
11	19.6	-34.4	2.6219	10.1	-64	2.6384
12	17.4	-27.3	2.934	4.72	-93	2.9135
13	15.4	-24.4	3.2332	1.24	-23	3.2461
14	13.1	-20.4	3.5185	-0.575	-6.86	3.5465
15	16.4	-28.7	3.7927	-1.12	-6.79	3.7997
			4.0557			4.0238

Comparison of Shear Force, Bending Moment & Deflection in Frame at Various Floor Levels of 20 storey for example 1 in two cases						
Floor Level	S.F.(KN)	B.M.(KN-m)	Deflection(cm)	S.F.(KN)	B.M.(KN-m)	Deflection(cm)
1	8.84	27.7	0	9.85	28.1	0
2	13.4	28.5	0.0472	16.7	32.5	0.0544
3	20.2	37.3	0.1549	26	45.1	0.1727
4	24	41.5	0.3076	32.6	53.4	0.3354
5	26.4	43.9	0.4939	38.1	60.4	0.5291
6	27.6	44.6	0.305	42.8	66.2	0.7436
7	27.9	44.2	0.9336	47.2	71.8	0.9713
8	27.6	43	1.1742	51.6	77.5	1.444
9	26.8	41.1	1.422	50.6	84	1.6816
10	25.6	38.2	1.6733	60.3	90	1.9154
11	24.1	-36.2	1.9247	81.3	-85.8	2.1457
12	22.3	-34	2.1734	13.8	-90.4	2.4205
13	20.4	-31.4	2.6529	6.77	-136	2.743
14	18.4	-28.7	2.8797	4.2	-104	3.0953
15	16.3	-25.7	3.0958	4.22	-83.4	3.3129
16	14.2	-22.7	3.3003	2.4	-50	3.5486
17	12.3	-19.8	3.4928	-0.338	-40	3.7566
18	10.7	-17.3	3.6741	-0.452	-28	3.9394
19	9.14	-14.3	3.8458	-0.352	-29	4.0992
20	11.6	-20.2	4.0101	-0.977	-30	4.2416

Comparison of Shear Force, Bending Moment & Deflection in Frame at Various Floor Levels of 30 storey for example 1 in two cases						
Floor Level	S.F.(KN)	B.M.(KN-m)	Deflection(cm)	S.F.(KN)	B.M.(KN-m)	Deflection(cm)
1	12	38	0.067	12.5	38.2	0.0697
2	18	39.5	0.2146	20.1	41.8	0.2276
3	26.4	50.4	0.4298	30.2	55.3	0.4496
4	31.3	55.7	0.6958	37.1	63.4	0.7212
5	34.4	58.7	1.0012	42.3	69.6	1.0297
6	36	59.9	1.3373	46.3	74.4	1.3661
7	36.7	59.8	1.6975	49.7	78.4	1.7237
8	36.7	58.9	2.0763	52.9	82.1	2.0971
9	36.2	57.5	2.4697	56	86	2.4818
10	35.4	55.7	2.8792	59.5	90.9	2.8744
11	34.4	53.6	3.2869	63.5	95.6	3.2715
12	33.2	51.3	3.7052	68.2	102	3.6704
13	31.9	48.9	4.1269	73.9	110	4.0684
14	30.5	46.4	4.55	81.3	131	4.3639
15	29	43.8	4.9725	87.2	75.9	4.8535
16	27.4	-41.2	5.3926	12.1	-92	5.2393
17	25.8	-39.1	5.8085	18.1	-73.2	5.6948
18	24.1	-36.8	6.2185	9.09	-69.4	6.2138
19	22.3	-34.4	6.6211	3.14	-62.9	6.7155
20	20.4	-31.8	7.0146	0.523	-59.8	7.1891
21	18.4	-29.1	7.3976	-0.212	-54.3	7.624
22	16.3	-26.1	7.7687	-0.232	-49.7	8.0416
23	14.2	-23.1	8.1269	-0.116	-45.8	8.4397
24	12.1	-20	8.9713	-0.053	-39.3	8.8186
25	10	-16.9	8.8013	-0.068	-32.2	9.1768
26	8.11	-13.9	9.1169	-0.153	-28.3	9.5128
27	6.41	-11.2	9.1487	-0.315	-22.9	9.8249
28	5.09	-8.91	9.7081	-0.582	-15.3	10.1125
29	4.08	-6.9	9.9879	-0.947	-10.9	10.377
30	5.02	-8.74	10.2999	-1.08	-17.23	10.624

TABLE 5.18

TABLE 5.19

Comparision of Bending Moment & Shear Force in Shear Wall at various floor levels in two cases of example1					
Floor Level	B.M(KN-m)	S.F(KN)	B.M(KN-m)	S.F(KN)	Deflection(cm)
1	3175	253	3285	260	0.001
2	2475	230	2575	236	0.003
3	1876	196	1966	201	0.005
4	1396	169	1486	173	0.007
5	1006	145	1114	148	0.01
6	685	125	795	127	0.013
7	420	107	535	108	0.017
8	199	90	350	88	0.02
9	17	73	51	65	0.023
10	-203	55	30	51	0.026
11	-296	36			0.03
12	-350	15			0.033
13	-357	-7			0.035
14	-304	-34			0.038
15	-204	-70			0.041

TABLE 5.20

Horizontal deflections at various floor levels in two cases of example 1		
	Case 1SW upto 15th Storey	Case 2 SW upto 10th Storey
Floor Level	Deflection(cm)	Deflection(cm)
1	0	0
2	0.0672	0.0705
3	0.2205	0.2285
4	0.4369	0.4791
5	0.6995	0.7146
6	0.9944	1.0107
7	1.3102	1.3263
8	1.6378	1.6528
9	1.9695	1.9836
10	2.306	2.3151
11	2.6219	2.6384
12	2.934	2.9135
13	3.2332	3.2461
14	3.5185	3.5465
15	3.7927	3.7997
16	4.0557	4.0238

TABLE 5.21

Comparision of Bending Moment, Shear Force & Deflection in Shear wall at various Floor Levels of example 2 in two cases						
	sysmetrical			Un sysmetrical		
Fl. level	B.M	S.F	Deflection	B.M	S.F	Deflection
	(KN-m)	(KN)	(cm)	(KN-m)	(KN)	(cm)
1	4876	325	0	5056	342	0
2	3957	307	0.001	4117	319	0.00136
3	3135	277	0.002	3265	288	0.0027
4	2434	251	0.004	2524	261	0.004136
5	1830	228	0.007	1900	237	0.00726
6	1309	207	0.01	1359	216	0.01035
7	860	187	0.013	890	196	0.0135
8	475	166	0.016	490	173	0.01658
9	149	145	0.019	154	151	0.01973
10	-285	121	0.022	-286	126	0.02284
11	-480	95	0.025	-495	100	0.0259
12	-604	65	0.028	-629	68	0.02905
13	-647	30	0.031	-672	32	0.03219
14	-596	8	0.034	-616	10	0.0353
15	-317	-74	0.036	-327	-79	0.03738
			0.039			0.0403

TABLE 5.22

Bending Moment, Shear Force & Deflection of 16 storey of case 3 of example 2			
Fl. Level	B.M.(KN-m)	S.F.(KN)	Deflection(cm)
1	4918	324	0.001
2	4005	305	0.002
3	3190	275	0.005
4	2496	249	0.007
5	1901	227	0.01
6	1387	206	0.013
7	944	187	0.016
8	562	168	0.019
9	235	149	0.023
10	-213	129	0.026
11	-427	107	0.029
12	-581	82	0.032
13	-670	53	0.035
14	-683	21	0.038
15	-611	-14	0.04
16	-322	-73	0.043

TABLE 5.23

Bending Moment And Shear Force

of 17 storey of case 3 of example 2

Floor Level	B.M. (KN-m)	S.F. (KN)	Deflection (cm)
1	4955	322	0.001
2	4047	303	0.003
3	3238	273	0.005
4	2552	247	0.007
5	1963	225	0.01
6	1458	205	0.013
7	1021	187	0.016
8	643	169	0.02
9	318	152	0.023
10	39	135	0.026
11	-364	116	0.03
12	-538	94	0.033
13	-658	71	0.036
14	-718	44	0.039
15	-708	14	0.042
16	-618	-19	0.044
17	-324	-75	0.047

TABLE 5.24

Bending Moment And Shear Force

of 18 storey of case 3 of example 2

Fl. Level	B.M. (KN-m)	S.F. (KN)	Deflection (cm)
1	4987	320	0.001
2	4004	301	0.003
3	3281	271	0.005
4	2601	245	0.007
5	2020	223	0.01
6	1521	203	0.013
7	1091	186	0.017
8	719	169	0.02
9	396	154	0.024
10	118	138	0.027
11	-296	122	0.031
12	-483	104	0.034
13	-624	84	0.037
14	-716	61	0.04
15	-752	36	0.043
16	-724	-7	0.046
17	-621	-23	0.049
18	-323	-77	0.051

TABLE 5.25

Bending Moment And Shear Force of 19 storey of case 3 of example 2			
Fl. Level	B.M.	S.F.	Deflection
	KN-m	KN	cm
1	5014	319	0.001
2	4116	299	0.003
3	3319	269	0.005
4	2645	243	0.007
5	2070	221	0.01
6	1578	202	0.014
7	1155	185	0.017
8	788	169	0.021
9	470	154	0.024
10	194	140	0.028
11	-226	126	0.031
12	-441	110	0.035
13	-556	93	0.038
14	-690	74	0.041
15	-759	53	0.044
16	-776	29	0.047
17	-773	2	0.05
18	-616	-26	0.053
19	-322	-78	0.055

TABLE 5.26

Bending Moment And Shear Force of 20 storeyof case 3 of example 2			
Fl. Level	B.M.	S.F.	Deflection
	KN-m	KN	CM
1	5038	317	0.001
2	4145	297	0.003
3	3353	267	0.003
4	2684	241	0.008
5	2116	219	0.011
6	1630	200	0.014
7	1213	183	0.017
8	853	168	0.021
9	539	155	0.025
10	267	141	0.028
11	29	128	0.032
12	-335	115	0.035
13	-519	100	0.039
14	-649	84	0.042
15	-740	66	0.046
16	-790	46	0.049
17	-791	24	0.052
18	-736	-1	0.054
19	-507	-29	0.057
20	-318	-78	0.06

TABLE 5.27

Bending Moment, Shear Force & Deflection of 21 storey of case 3 of example 2

Fl. Level	B.M. (KN-m)	S.F. (KN)	Deflection (cm)
1	5059	316	0.001
2	4170	296	0.003
3	3383	265	0.005
4	2720	240	0.008
5	2157	218	0.011
6	1667	199	0.014
7	1266	182	0.018
8	912	167	0.021
9	604	154	0.025
10	335	142	0.029
11	100	130	0.033
12	-290	118	0.036
13	-458	105	0.04
14	-597	91	0.043
15	-705	76	0.047
16	-777	60	0.05
17	-811	40	0.053
18	-800	19	0.056
19	-734	-5	0.059
20	-505	-31	0.061
21	-315	-78	0.064

TABLE 5.28

Bending Moment, Shear Force & Deflection of 22 storey of case 3 of example 2

Fl. Level.	B.M. (KN-m)	S.F. (KN)	Deflection (cm)
1	5077	314	0.001
2	4192	294	0.003
3	3410	264	0.005
4	2752	238	0.008
5	2194	216	0.011
6	1720	197	0.014
7	1315	181	0.018
8	966	166	0.022
9	664	154	0.025
10	339	142	0.029
11	168	131	0.033
12	-225	120	0.037
13	-396	109	0.041
14	-540	97	0.044
15	-658	84	0.048
16	-747	70	0.051
17	-804	53	0.055
18	-825	35	0.058
19	-803	15	0.061
20	-730	-8	0.063
21	-501	-33	0.066
22	-310	-78	0.068

TABLE 5.29

B. M, S. Force & Deflection of 23 storey of case 3 of example 2

Fl. Level	B.M. (KN-m)	S.F. (KN)	Deflection (cm)
1	5093	313	0.001
2	4213	293	0.003
3	3434	262	0.005
4	2781	237	0.008
5	2228	215	0.011
6	1759	196	0.015
7	1360	179	0.018
8	1016	165	0.022
9	719	153	0.026
10	459	142	0.03
11	232	131	0.034
12	31	121	0.038
13	-333	111	0.042
14	-480	101	0.045
15	-605	90	0.049
16	-705	78	0.052
17	-779	64	0.056
18	-823	49	0.059
19	-833	31	0.062
20	-802	11	0.065
21	-723	-11	0.068
22	-496	-35	0.07
23	-305	-78	0.073

TABLE 5.30

B. M, S. Force & Deflection of 24 storey of case 3 of example 2

Floor Level	B.M. (KN-m)	S.F. (KN)	Deflection (cm)
1	5108	312	0.001
2	4231	292	0.003
3	3456	261	0.006
4	2807	235	0.008
5	2259	213	0.011
6	1795	194	0.015
7	1400	178	0.019
8	1062	164	0.022
9	770	152	0.026
10	515	141	0.03
11	292	131	0.034
12	94	122	0.038
13	-272	113	0.042
14	-420	104	0.046
15	-540	94	0.05
16	-655	84	0.054
17	-740	72	0.057
18	-802	59	0.061
19	-835	44	0.064
20	-836	27	0.067
21	-798	8	0.07
22	-715	-13	0.072
23	-490	-36	0.075
24	-300	-77	0.078

TABLE 5.31

B. M, S. Force & Deflection of 25 storey of case 3 of example 2			
Fl. Level	B.M.	S.F.	Deflection
	(KN-m)	(KN)	(cm)
1	5309	322	0.002
2	4402	301	0.003
3	3603	269	0.006
4	2934	242	0.009
5	2371	220	0.012
6	1895	200	0.016
7	1490	183	0.019
8	1144	169	0.023
9	847	157	0.027
10	588	146	0.032
11	360	136	0.036
12	159	127	0.04
13	-221	118	0.044
14	-374	110	0.049
15	-508	101	0.053
16	-624	91	0.057
17	-720	81	0.06
18	-796	69	0.064
19	-848	56	0.068
20	-873	42	0.071
21	-866	25	0.074
22	-821	6	0.077
23	-731	-15	0.08
24	-501	-38	0.083
25	-305	-80	0.085

TABLE 5.32

B. M, S. Force & Deflection of 26 storey of case 3 of example 2			
Fl. Level	B.M.	S.F.	Deflection
	(KN-m)	(KN)	(cm)
1	5548	335	0.002
2	4606	313	0.004
3	3778	279	0.006
4	3084	252	0.009
5	2500	228	0.013
6	2008	207	0.016
7	1591	190	0.02
8	1235	175	0.025
9	930	162	0.0029
10	665	151	0.033
11	432	141	0.038
12	227	132	0.042
13	44	124	0.047
14	-328	116	0.051
15	-468	107	0.056
16	-591	98	0.06
17	-696	89	0.064
18	-784	79	0.068
19	-857	67	0.072
20	-896	54	0.075
21	-914	39	0.079
22	-900	22	0.082
23	-848	38	0.085
24	-752	-17	0.088
25	-514	-39	0.091
26	-312	-81	0.094

TABLE 5.33

M, S. Force & Deflection of 27 storey of case 3 of example 2			
Fl. Level	B.M.	S.F.	Deflection
	(KN-m)	(KN)	(cm)
1	5788	347	0.002
2	4812	324	0.004
3	3952	290	0.006
4	3234	261	0.01
5	2630	236	0.013
6	2122	215	0.017
7	1692	196	0.021
8	1327	181	0.026
9	1013	168	0.03
10	742	156	0.035
11	505	147	0.04
12	295	138	0.045
13	109	129	0.049
14	-280	122	0.054
15	-424	114	0.059
16	-552	106	0.063
17	-665	97	0.068
18	-762	88	0.072
19	-842	78	0.076
20	-902	66	0.08
21	-940	53	0.084
22	-952	38	0.087
23	-931	21	0.091
24	-873	2	0.094
25	-770	-19	0.097
26	-526	-42	0.1
27	-319	-84	0.103

TABLE 5.34

B. M, S. Force & Deflection of 28 storey of case 3 of example 2			
Fl. Level	B.M.	S.F.	Deflection
	(KN-m)	(KN)	(cm)
1	6029	360	0.002
2	5017	336	0.004
3	4127	300	0.007
4	3384	270	0.01
5	2760	244	0.014
6	2236	222	0.018
7	1793	203	0.022
8	1418	187	0.027
9	1097	173	0.032
10	819	162	0.037
11	578	152	0.042
12	365	143	0.047
13	175	135	0.052
14	-229	127	0.057
15	-377	120	0.062
16	-510	112	0.067
17	-629	104	0.071
18	-733	96	0.076
19	-823	86	0.08
20	-896	76	0.085
21	-950	64	0.089
22	-981	51	0.093
23	-987	36	0.097
24	-961	19	0.1
25	-867	0.151	0.104
26	-788	-21	0.107
27	-538	-43	0.11
28	-325	-86	0.113

TABLE 5.35

B. M, S. Force & Deflection of 29 storey of case 3 of example 2

Fl. Level	B.M. (KN-m)	S.F. (KN)	Deflection (cm)
1	6270	372	0.002
2	5222	347	0.004
3	4302	310	0.007
4	3534	279	0.01
5	2890	252	0.014
6	2350	229	0.019
7	1894	209	0.023
8	1509	193	0.028
9	1180	179	0.033
10	897	167	0.038
11	651	157	0.044
12	434	148	0.049
13	241	140	0.054
14	67	132	0.059
15	-329	125	0.065
16	-465	118	0.07
17	-588	111	0.075
18	-698	103	0.08
19	-795	94	0.085
20	-878	85	0.089
21	-945	75	0.094
22	-993	63	0.098
23	-1020	56	0.102
24	-1019	35	0.106
25	-988	18	0.11
26	-918	-1.53	0.113
27	-706	-23	0.117
28	-549	-45	0.12
29	-331	-88	0.123

TABLE 5.36

B. M, S. Force & Deflection of 30 storey of case 3 of example 2

Fl. Level	B.M. (KN-m)	S.F. (KN)	Deflection (cm)
1	6511	385	0.002
2	5428	359	0.004
3	4478	320	0.007
4	3685	288	0.011
5	3021	260	0.015
6	2464	236	0.019
7	1996	216	0.024
8	1600	199	0.029
9	1263	184	0.034
10	974	172	0.04
11	724	162	0.045
12	554	152	0.051
13	308	144	0.057
14	132	137	0.062
15	-279	130	0.068
16	-418	123	0.073
17	-545	116	0.079
18	-659	109	0.084
19	-762	102	0.089
20	-852	93	0.094
21	-929	84	0.099
22	-991	74	0.104
23	-1034	62	0.108
24	-1055	48	0.112
25	-1050	33	0.116
26	-1013	16	0.12
27	-938	-3	0.124
28	-721	-24	0.127
29	-559	-42	0.131
30	-336	-90	0.134

TABLE 5.37

Variation of inflection point with Number of Storeys for example 2 of case 3

Floor Level	Inflection Point(m)
15	25.03
16	25.5
17	27.3
18	27.86
19	28.38
20	30.22
21	30.77
22	31.28
23	33.25
24	33.77
25	34.71
26	36.37
27	36.83
28	37.75
29	39.51
30	39.96

TABLE 5.38

Comparision of Shear Force,Bending Moment & Deflection in Frame at Various Floor Levels of 15 storey for example 2 in cases 3 and 4

Floor Level	S.F.(KN)	B.M.(KN-m)	Deflection(cm)	S.F.(KN)	B.M.(KN-m)	Deflection(cm)
1	21.5	37.3	0	22.5	38	0
2	39.8	65.4	0.602	42.8	69.2	0.609
3	51.7	81.4	0.1953	56.8	88.5	0.1973
4	61.8	95.7	0.3883	69.2	106	0.39
5	69.3	106	0.6264	79.2	120	0.631
6	74.8	113	0.8978	87.4	132	0.8995
7	78.5	118	1.1922	94	141	1.201
8	80.7	121	1.501	99.7	149	1.531
9	81.7	-123	1.8165	104	-141	1.845
10	81.7	-123	2.1326	112	-150	2.181
11	80.8	-122	2.4439	73.6	-156	2.4512
12	79.4	-120	2.4668	40	-136	2.478
13	78.2	-119	3.038	37.5	-162.5	3.059
14	73.1	-108	3.3169	28.5	-135.3	3.5053
15	97.5	-167	3.5859	33.7	-170	3.8449
16			3.834			4.0872

Comparision of Shear Force,Bending Moment & Deflection in Frame at Various Floor Levels of 20 storey for example 2 in cases 3 and 4

Floor Level	S.F.(KN)	B.M.(KN-m)	Deflection(cm)	S.F.(KN)	B.M.(KN-m)	Deflection(cm)
1	21.5	37.8	0	23	39	0
2	40.4	66.7	0.6515	44.7	72.1	0.661
3	52.8	83.6	0.2015	60.1	93.6	0.2218
4	63.6	98.9	0.4035	74	114	0.4304
5	71.8	110	0.6555	85.7	130	0.6803
6	78.3	119	0.9462	95.7	144	0.9895
7	83	126	1.2661	104	157	1.2731
8	86.4	130	1.6073	112	168	1.6823
9	88.6	133	1.9626	119	178	1.9824
10	89.8	135	2.326	126	187	2.3504
11	90.2	-136	2.6924	131	-169	2.7304
12	89.7	-137	3.057	147	-180	3.058
13	88.7	-135	3.416	77.2	-190	3.4501
14	87.1	-134	3.7662	37.6	-197	3.7923
15	85.2	-132	4.1048	37.3	-231	4.128
16	83	-129	4.4299	31.7	-180	4.5445
17	80.7	-122	4.9403	28.2	-152	4.9257
18	79.1	-120	5.0358	28.5	-139	5.2723
19	73.8	-160	5.3171	20.7	-165	5.5875
20	98.6	-168	5.5883	29.3	-170	5.8352
			5.9893			6.1348

TABLE 5.38

Comparision of Shear Force,Bending Moment & Deflection in Frame at Various Floor Levels of 25 storey for example 2 in cases 3 and 4						
Floor Level	S.F.(KN)	B.M.(KN-m)	Deflection(cm)	S.F.(KN)	B.M.(KN-m)	Deflection(cm)
1	39	21.9	0	90.8	24.1	0
2	68.5	91.3	0.0699	76.2	47.4	0.06494
3	86.3	54.2	0.2121	100	64.5	0.2129
4	102	65.5	0.9286	123	80.2	0.4366
5	115	74.4	0.6958	142	93.7	0.7132
6	125	81.4	1.0085	160	106	1.0105
7	132	86.8	1.3556	175	117	1.3759
8	138	90.9	1.729	181	126	1.749
9	142	93.9	2.1222	203	136	2.1801
10	144	95.9	2.5294	216	145	2.8694
11	146	97.1	2.9955	229	154	2.993
12	146	97.6	3.3663	241	161	3.37
13	-147	97.5	3.8871	-190	186	3.812
14	-147	96.8	4.2068	-205	86.7	4.2604
15	-144	95.6	4.6201	-219	38.1	4.6803
16	-142	94	5.025	-234	39.5	5.1032
17	-139	92.1	5.4192	-244	33.3	5.5192
18	-136	89.9	5.8007	-245	29.7	5.8505
19	-133	87.8	6.1675	-251	26.6	6.2836
20	-129	84.1	6.5196	-230.2	23.8	6.6834
21	-125	82.4	6.8552	-220	21.4	7.0614
22	-121	79.8	7.1744	-190	19.2	7.4191
23	-119	78.2	7.4782	-160	17.7	7.7581
24	-115	72.9	7.7678	-145	14.4	8.0814
25	-169	97.6	8.0471	-180	29.4	8.3964
			8.0385			8.7078

TABLE 5.38

Comparision of Shear Force,Bending Moment & Deflection in Frame at Various Floor Levels of 30 storey for example 2 in cases 3 and 4

Floor Level	S.F.(KN)	B.M.(KN-m)	Deflection(cm)	S.F.(KN)	B.M.(KN-m)	Deflection(cm)
1	26.8	45.4	0	29.6	47.3	0
2	52.4	77.3	0.0786	96	85.4	0.0796
3	72	98.1	0.2599	61.1	113	0.2618
4	89.6	117	0.5244	74.1	138	0.534
5	105	131	0.8576	84.4	160	0.8672
6	118	142	1.2465	92.5	179	1.2868
7	130	151	1.6802	98.9	196	1.7
8	141	158	2.1495	104	212	2.1998
9	151	163	2.6468	108	227	2.6588
10	161	167	3.6656	110	241	3.701
11	171	169	3.7044	112	255	3.7201
12	181	170	4.2462	113	269	4.2862
13	191	170	4.7989	114	298	4.805
14	200	170	5.3548	113	324	5.3948
15	230	-170	5.4105	113	-227	5.99
16	104	-170	6.463	112	-243	6.603
17	45.1	-168	7.0095	110	-258	7.00187
18	47.1	-166	7.5475	108	-273	7.601
19	39.9	-169	8.0746	106	-260	8.08
20	35.8	-160	8.5888	104	-248	8.603
21	32.1	-157	9.0882	101	-210	9.0892
22	28.8	-153	9.571	97.9	-198	9.801
23	25.8	-149	10.036	94.8	-183	10.1209
24	22.9	-144	10.4821	91.6	-178	10.6594
25	20.3	-139	10.9088	88.4	-168	11.127
26	17.8	-134	11.3151	85.3	-160	11.5723
27	15.7	-130	11.7039	82.3	-159	11.9965
28	14.4	-125	12.0141	80.4	-158	12.402
29	11.6	-122	12.4285	74.8	-130	12.7936
30	23.6	-131	12.772	100	-180	13.1822
			13.0954			13.5769

TABLE 5.39

Comparison of B.M., S.F., and Deflection at various floor level of example 2 in case 3 and 4					
Floor Level	**B.M(KN-m)**	**S.F.(KN)**	**B.M(KN-m)**	**S.F.(KN)**	**Deflection(cm)**
1	4870	325	4978	332	0.001
2	3957	307	4052	313	0.002
3	3135	277	3225	281	0.004
4	2434	251	2526	254	0.007
5	1830	228	1918	230	0.01
6	1309	207	1412	208	0.013
7	860	187	980	187	0.016
8	475	166	600	161	0.019
9	149	145	281	138	0.022
10	-285	121	130	119	0.025
11	-480	95			0.028
12	-604	65			0.031
13	-647	30			0.034
14	-596	8			0.036
15	-317	-74			0.039

TABLE 5.22

Bending Moment, Shear Force & Deflection of 16 storey of case 3 of example 2			
Fl. Level	**B.M.(KN-m)**	**S.F.(KN)**	**Deflection(cm)**
1	4918	324	0.001
2	4005	305	0.002
3	3190	275	0.005
4	2496	249	0.007
5	1901	227	0.01
6	1387	206	0.013
7	944	187	0.016
8	562	168	0.019
9	235	149	0.023
10	-213	129	0.026
11	-427	107	0.029
12	-581	82	0.032
13	-670	53	0.035
14	-683	21	0.038
15	-611	-14	0.04
16	-322	-73	0.043

TABLE 5.40

Horizontal Deflection at various Floor Levels in Case 3 & 4 of Example 2		
	Case 3 SW upto 15th Storey	Case 4 SW upto 15th Storey
FL. Level	Deflection(cm)	Deflection(cm)
1	0	0
2	0.602	0.609
3	0.1953	0.1973
4	0.3883	0.39
5	0.6264	0.631
6	0.8978	0.8995
7	1.1922	1.201
8	1.501	1.531
9	1.8165	1.845
10	2.1326	2.181
11	2.4439	2.4512
12	2.4668	2.478
13	3.038	3.059
14	3.3169	3.5053
15	3.5859	3.8449
16	3.834	4.0872

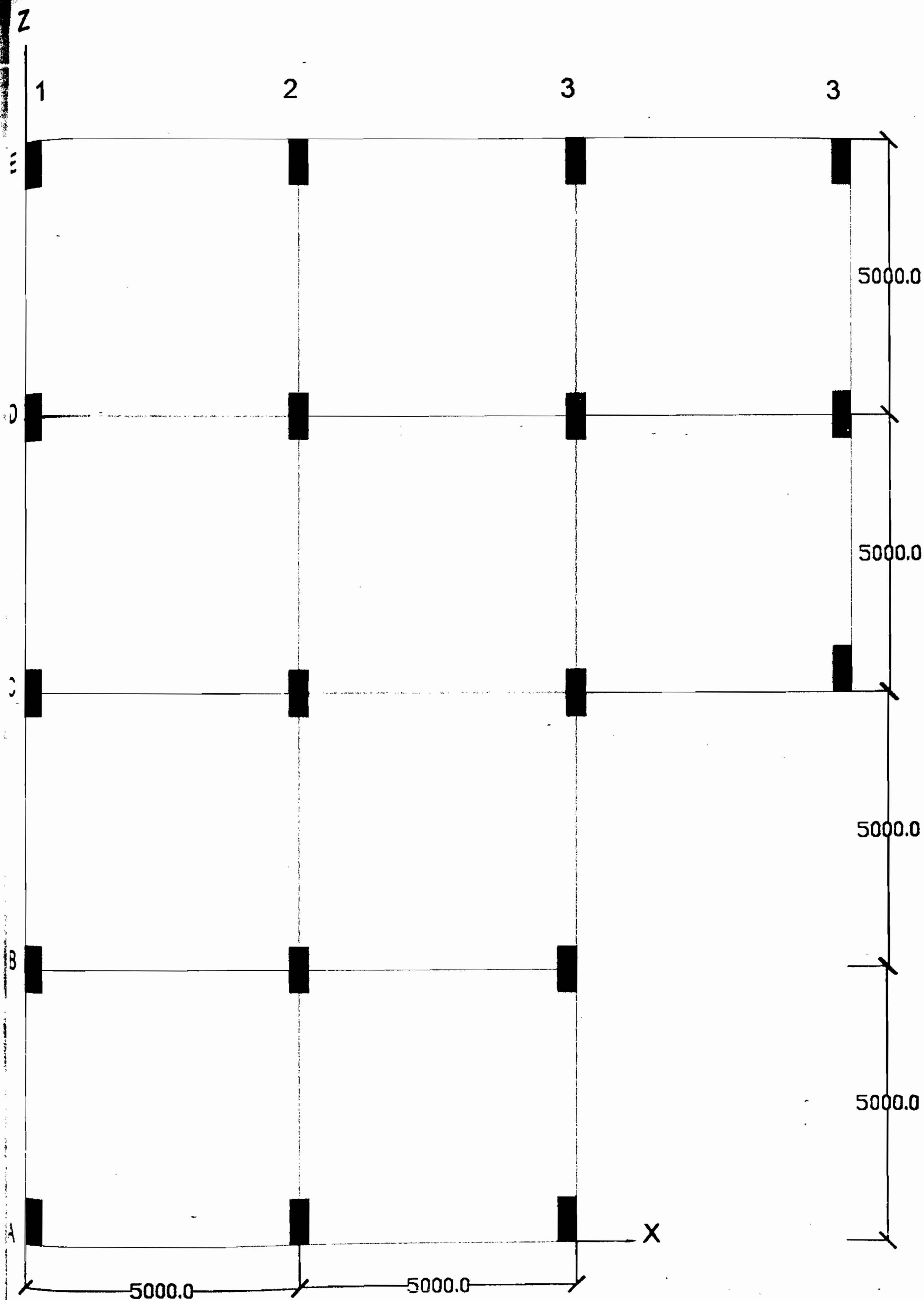


FIG.5.1 TYPICAL FLOOR PLAN OF UNSYMMETRICAL MULTI STOREYED BUILDING OF EXAMPLE 1

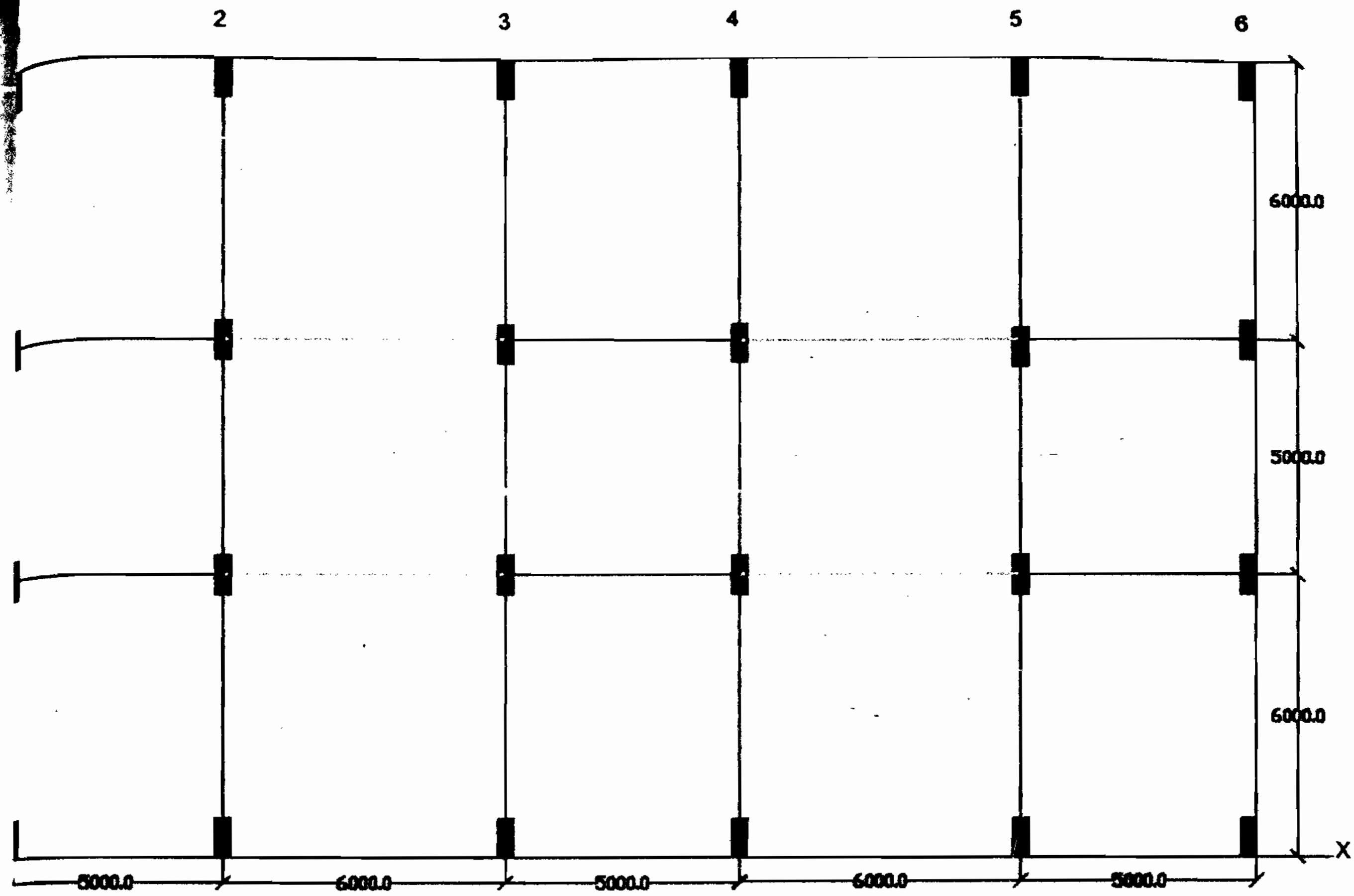


FIG.5.1(a) TYPICAL FLOOR PLAN OF SYMMETRICAL MULTI STOREYED BUILDING OF EXAMPLE 2 (CASE 1)

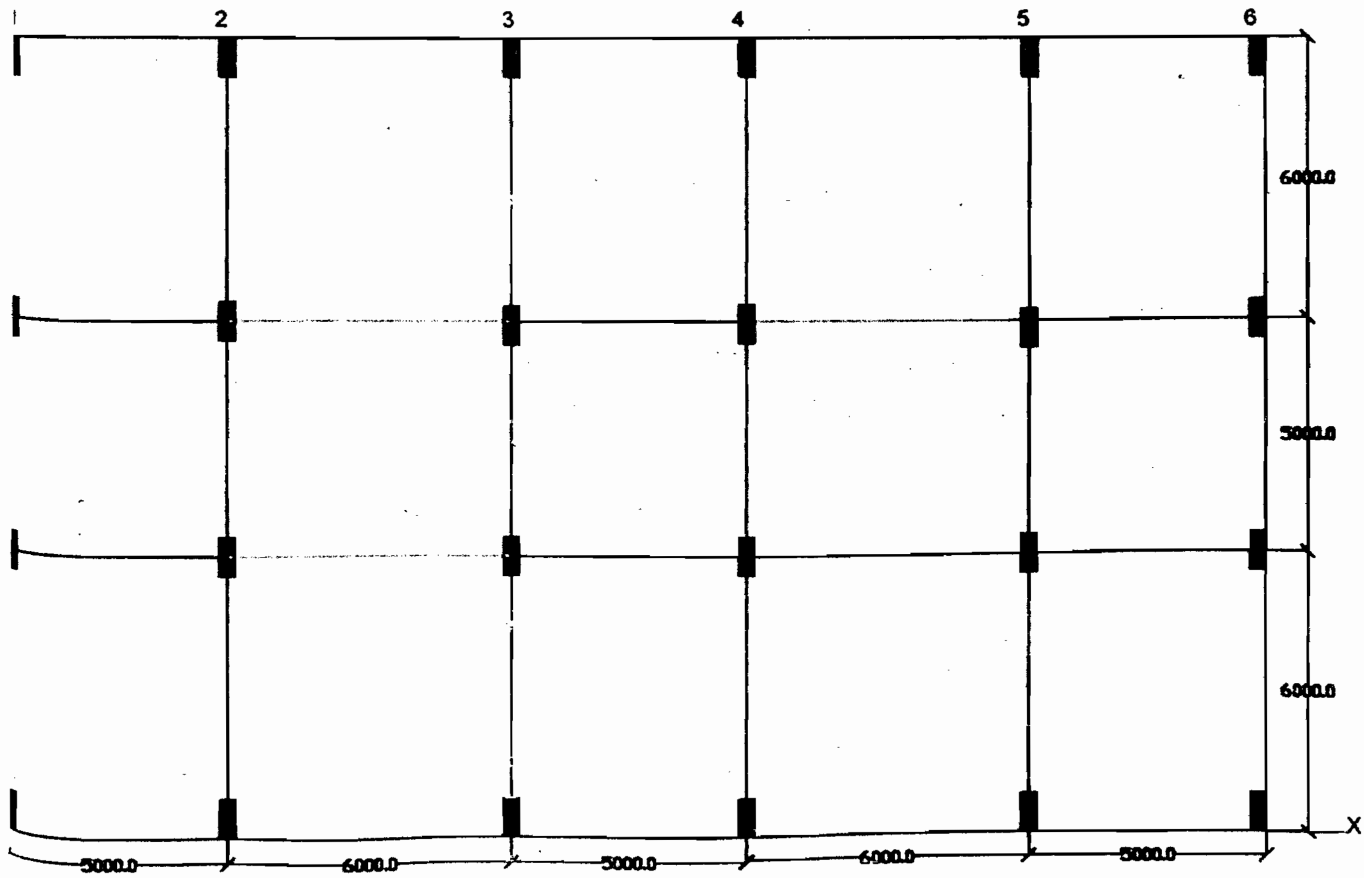


FIG.5.1(b) TYPICAL FLOOR PLAN OF SYMMETRICAL MULTI STOREYED BUILDING OF EXAMPLE 1 (CASE 2)

Fig: 5.3 Variation of Point of Infection for unsymmetrical Buildings

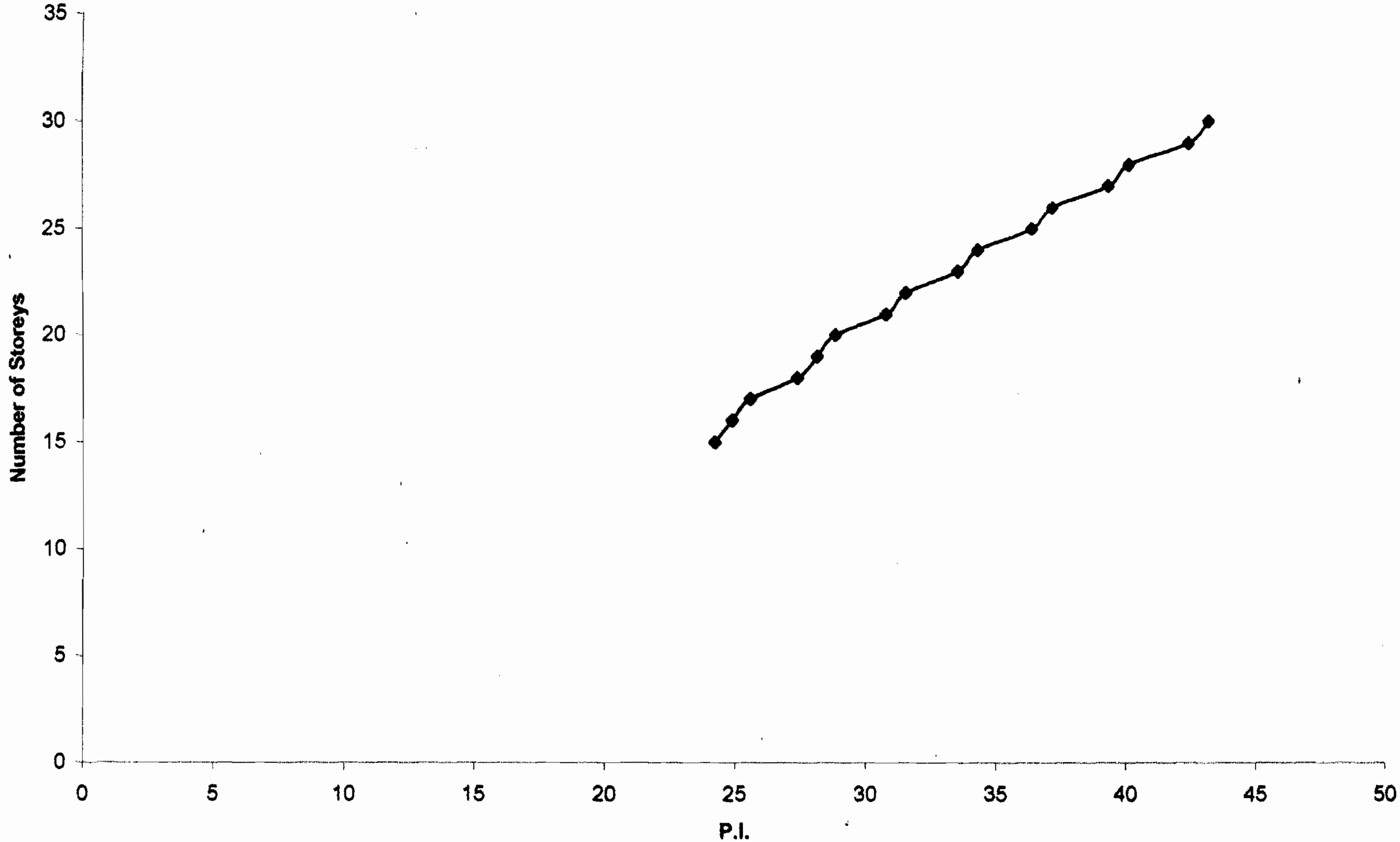


Fig:5.4 Variation in Bending Moment of16 Storey Building of example1 of case 1

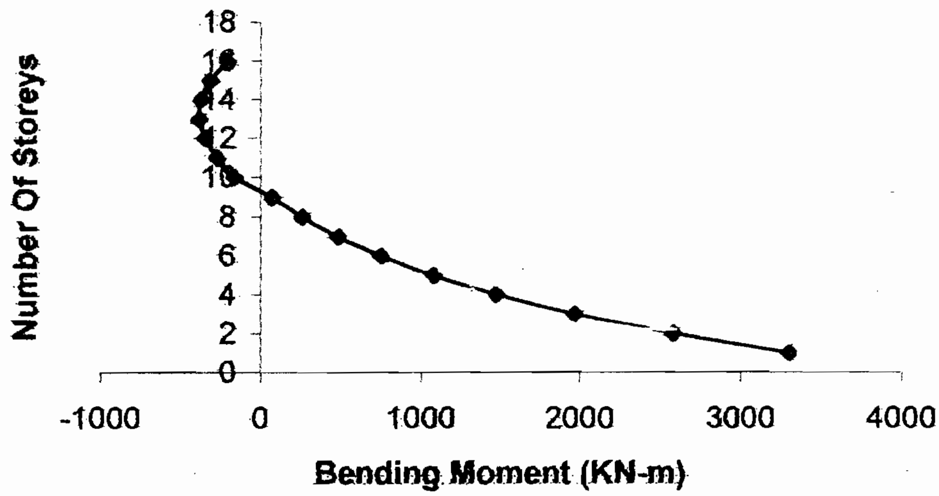


Fig:5.5 Variation in Bending Moment in shear wall of 17 storey of example 1 of case 1

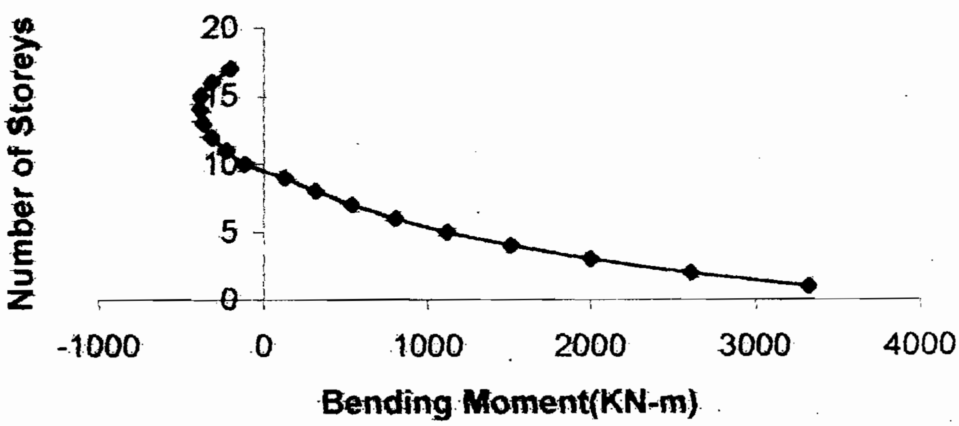


Fig: 5.6 Variation in Bending Moment of 18 storey of example 1 of case1

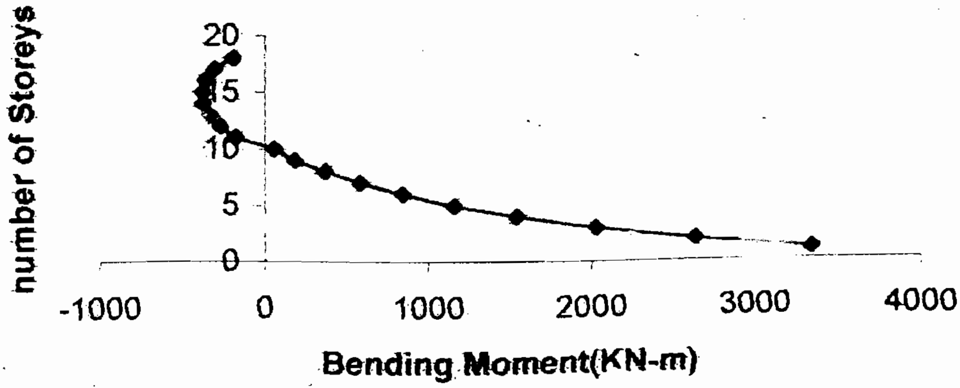


Fig:5.8 Variation in bending Moment of 20 Storey of example1 of case1

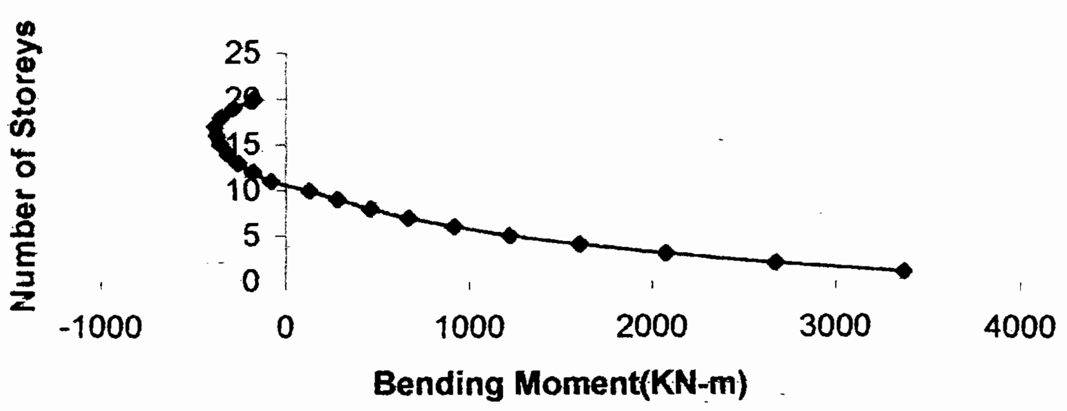


Fig:5.7 Variation in Bending Moment of 19 storey of example1 of case1

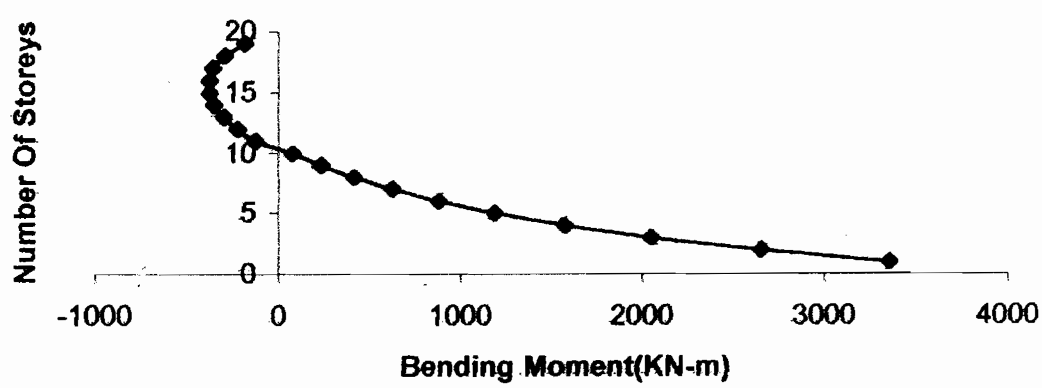


Fig:5.9 Variation in Bending Moment of 21storey of example1 of case 1

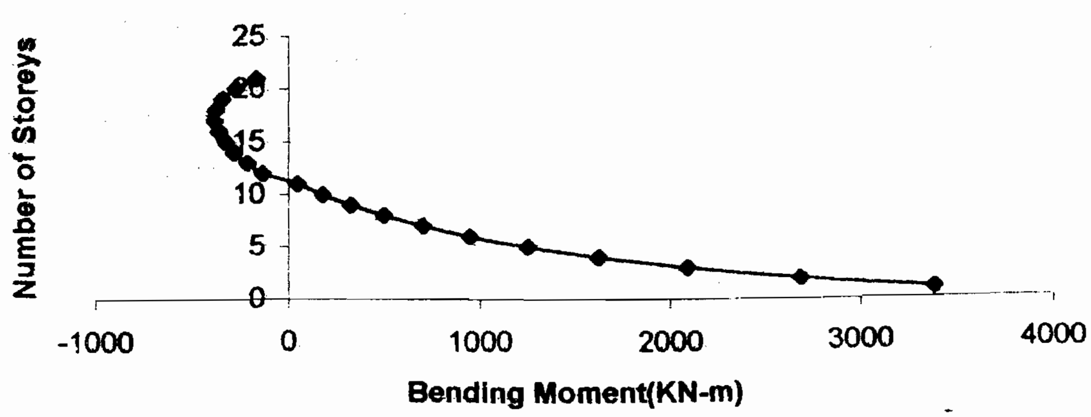


Fig 5.10 Variation in Bending Moment of 22 Storey of example1 of case 1

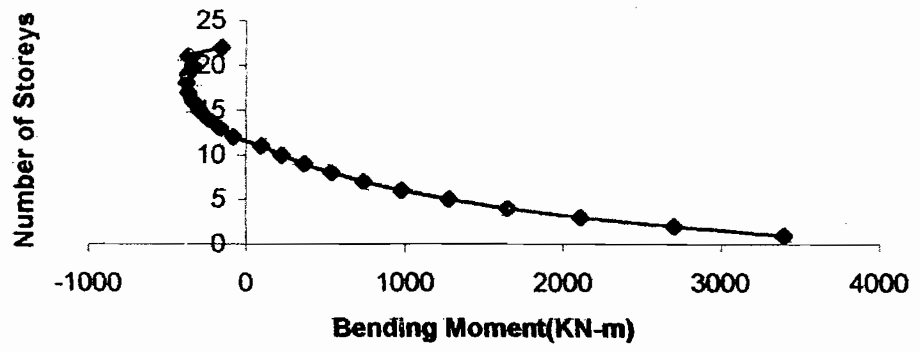


Fig-5.11 Variation in Bending Moment of 23 Storey of example1 of case 1

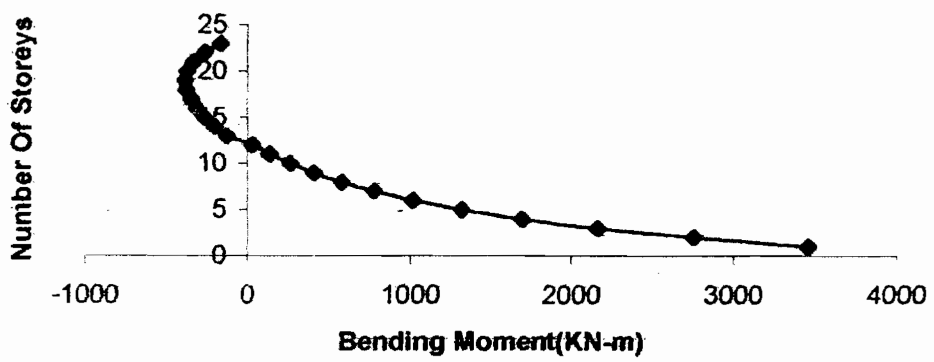


Fig-5.12 Variation in Bending Moment of 24 storey of example1 of case1

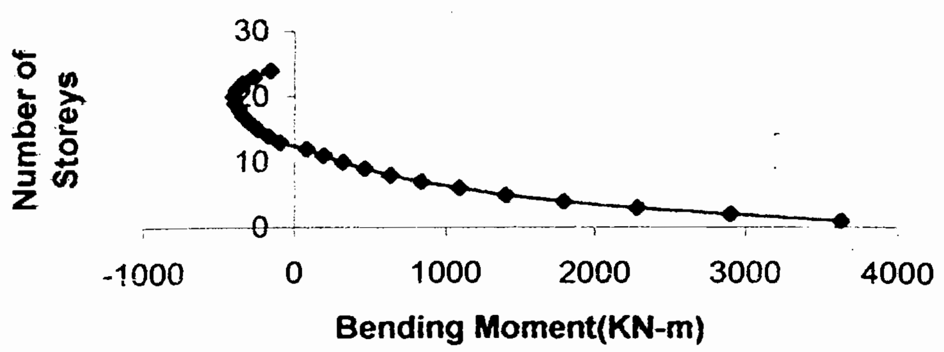
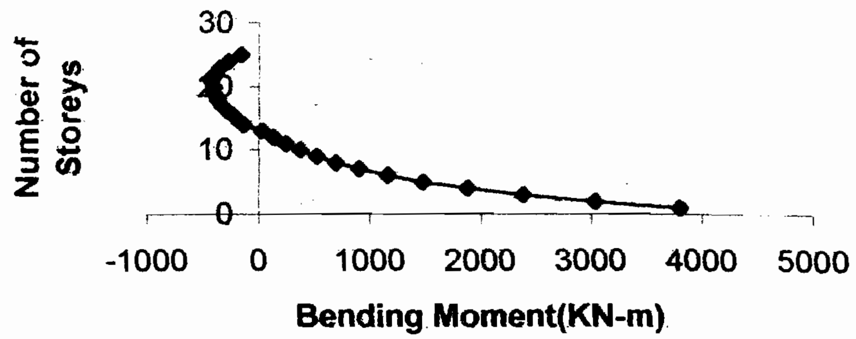


Fig-5.13 Variation in Bending Moment of 25 Storey of example1 of case 1



Fid-5.14 Variation in Bending Moment of 26 storey of example1 of case1

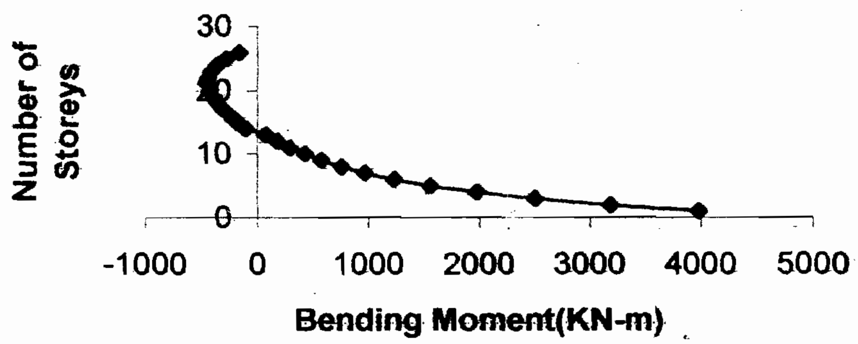


Fig-5.15 Variation in Bending Moment of 27 storey of example1 of case1

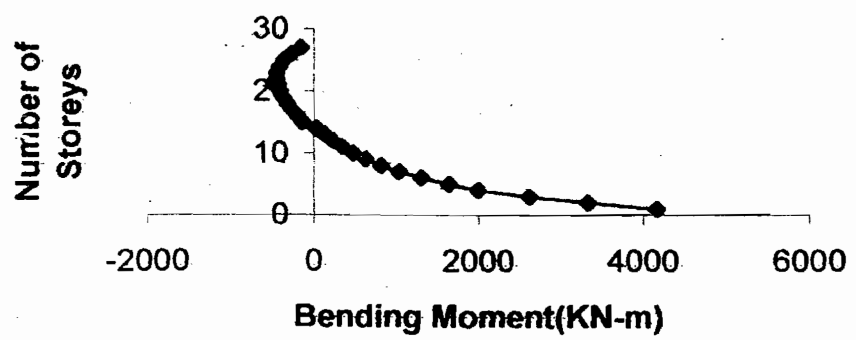


Fig:5.16 Variation in Bending Moment of 28 storey of example1 of case 1

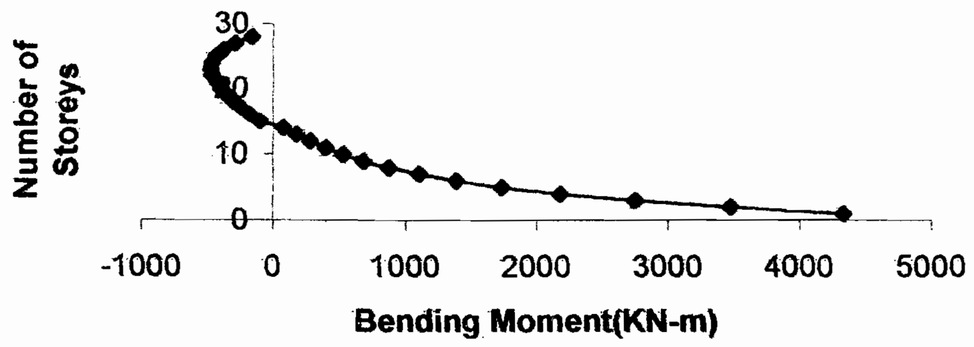


Fig:5.17 Variation in Bending Moment of 29 Storey of example1 of case 1

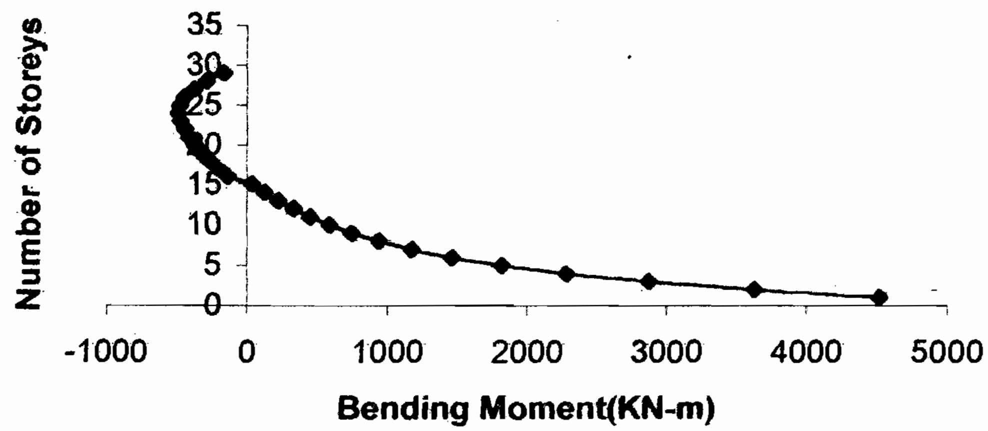
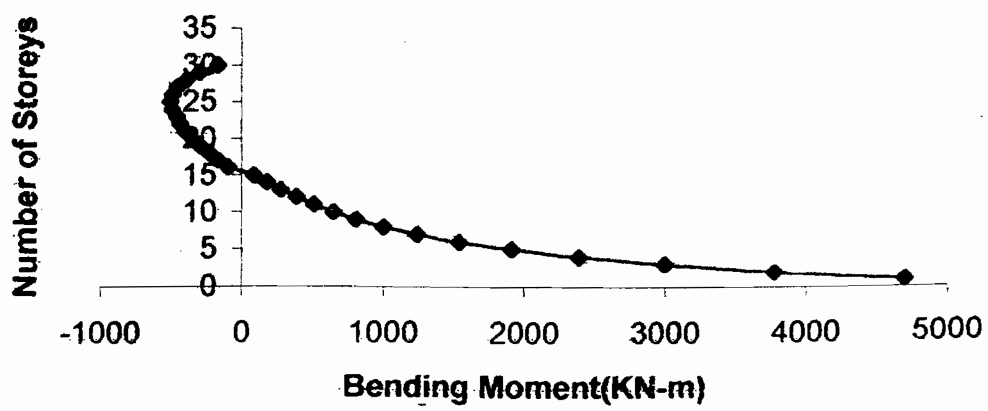


Fig:5.18 Variation in Bending Moment of 30 storey of example1 of case1



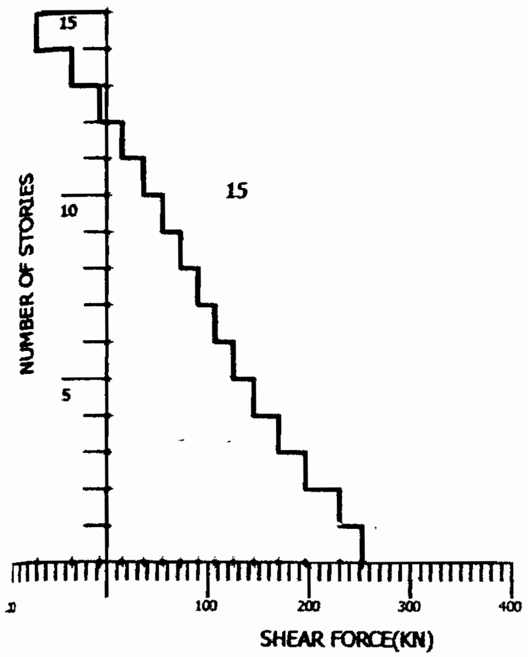


FIG.Variation of Shear Force in Shear Wall in Case 1 of Example 1

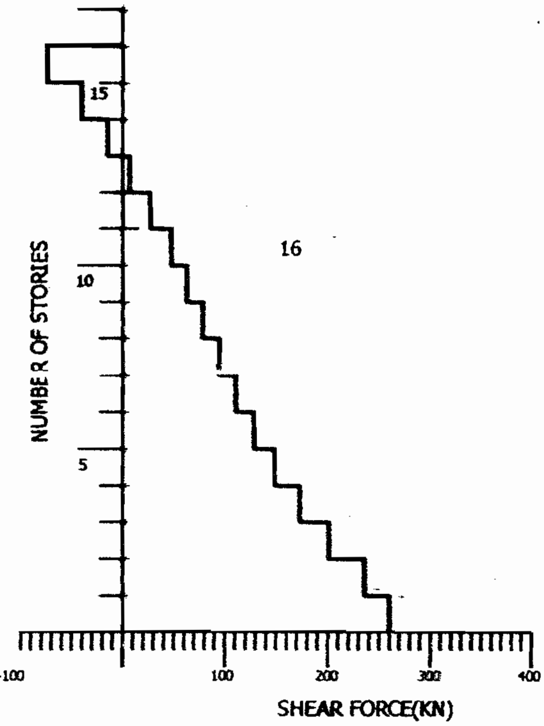
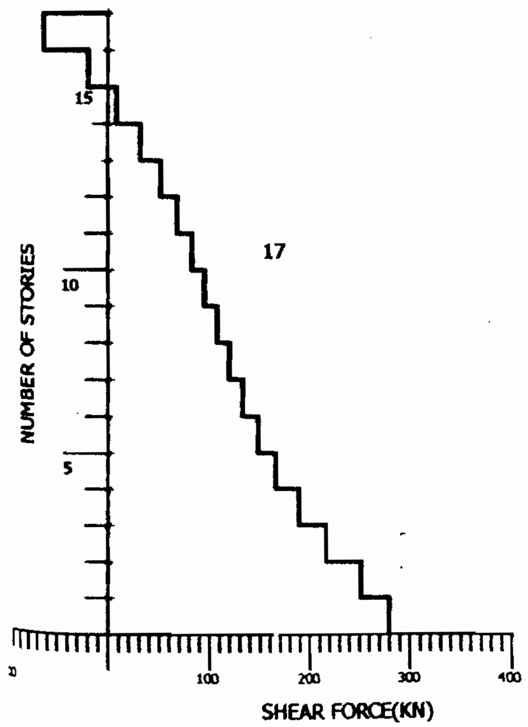


FIG.5.19. Variation of Shear Force in Shear Wall in Case 1 of Example 1



5.20. Variation of Shear Force in Shear Wall in Case Example 1

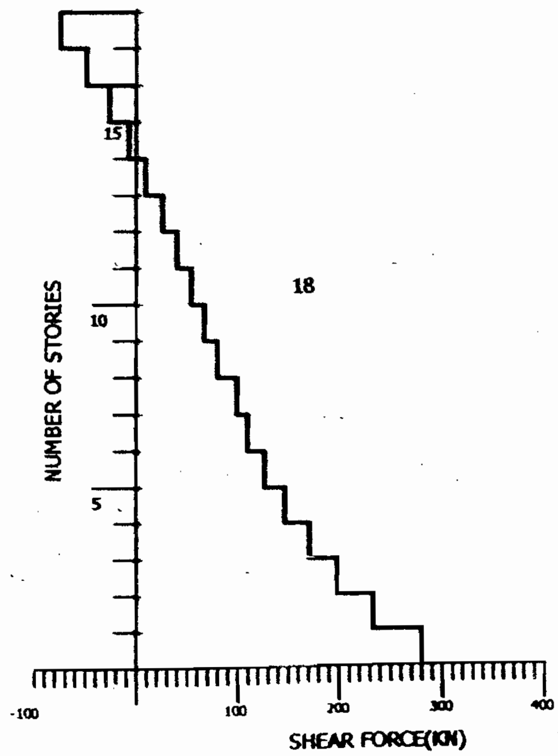
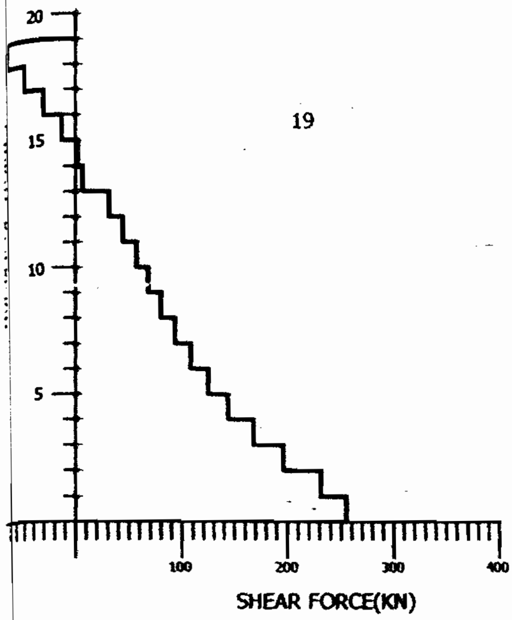


FIG.5.21.Variation of Shear Force in Shear Wall in Case 1 of Example 1



2. Variation of Shear Force in Shear Wall in Case
ample 1

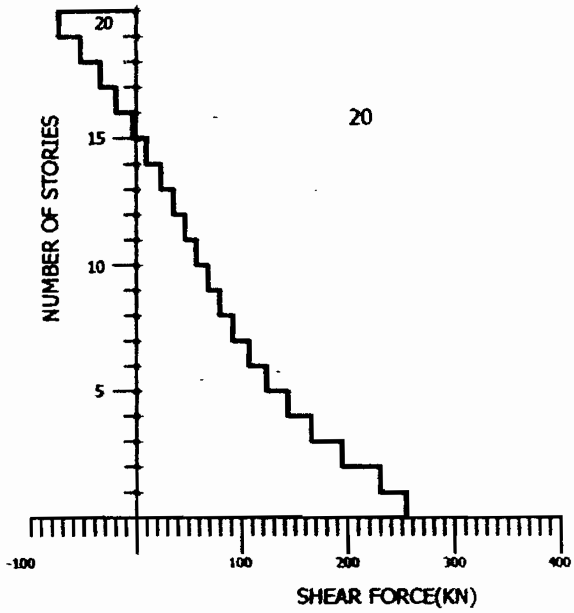
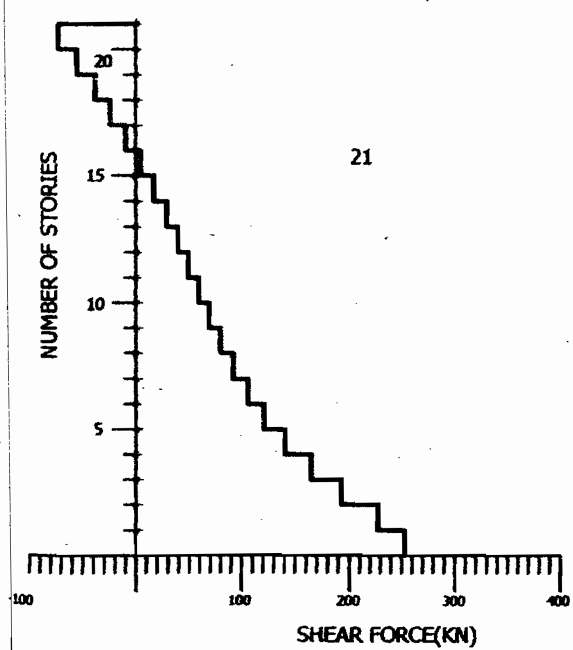


FIG.5.23. Variation of Shear Force in Shear Wall in Case 1 of Example 1



G.5.24.Variation of Shear Force in Shear Wall in Case
f Example 1

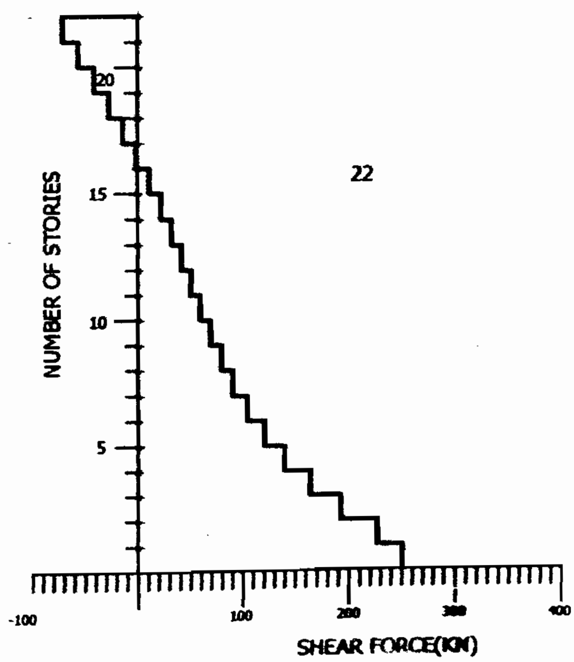


FIG.5.25. Variation of Shear Force in Shear Wall in Case 1 of Example 1

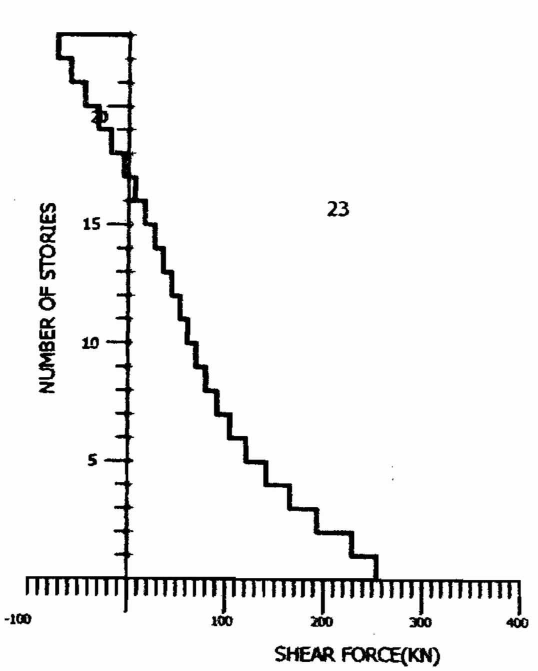


FIG.5.26. Variation of Shear Force in Shear Wall in Case 1 of Example 1

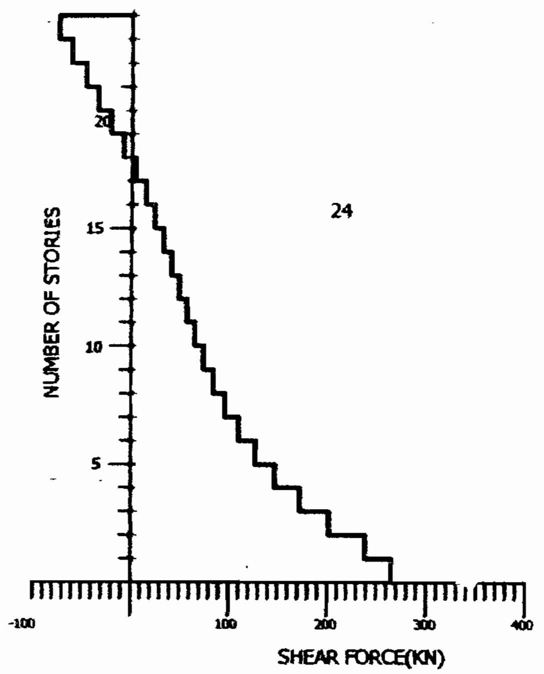


FIG.5.27.Variation of Shear Force in Shear Wall in Case 1 of Example 1

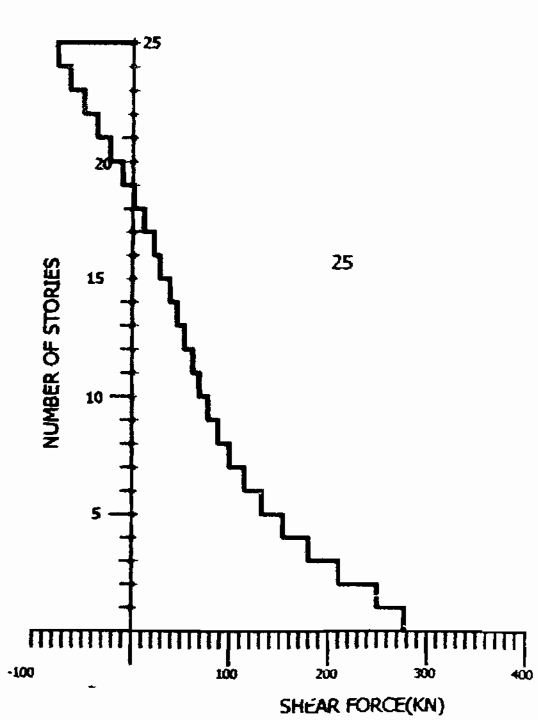


FIG.5.28. Variation of Shear Force in Shear Wall in Case 1 of Example 1

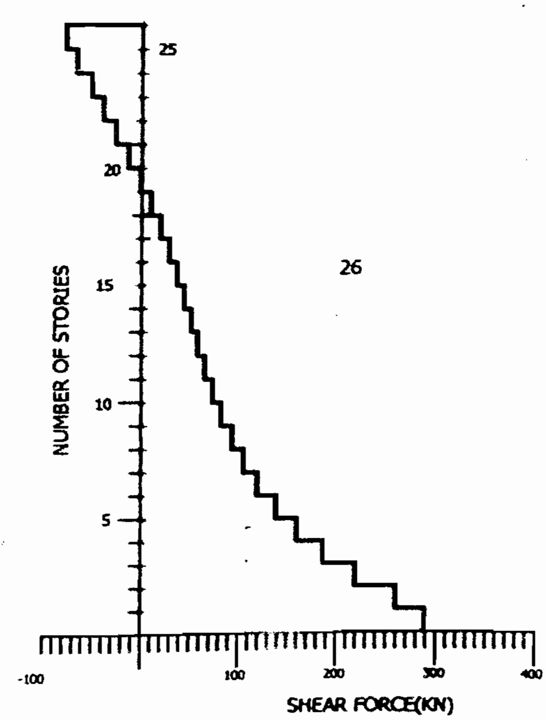


FIG.5.29. Variation of Shear Force in Shear Wall in Case 1 of Example 1

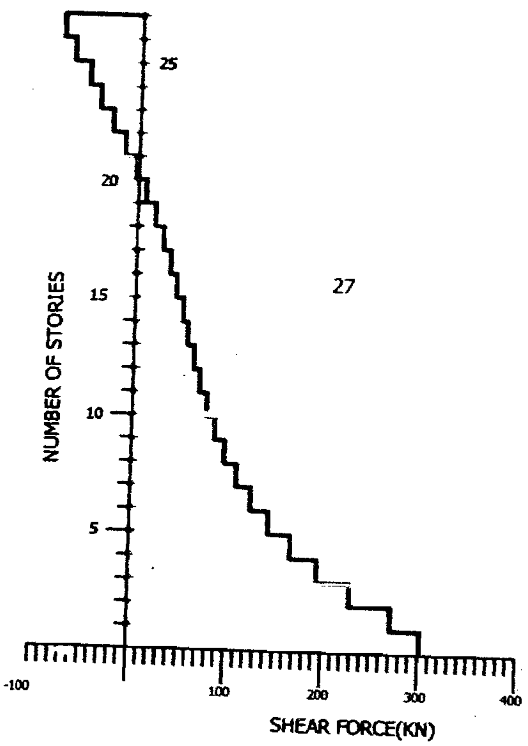


FIG.5.30. Variation of Shear Force in Shear Wall in Case 1 of Example 1

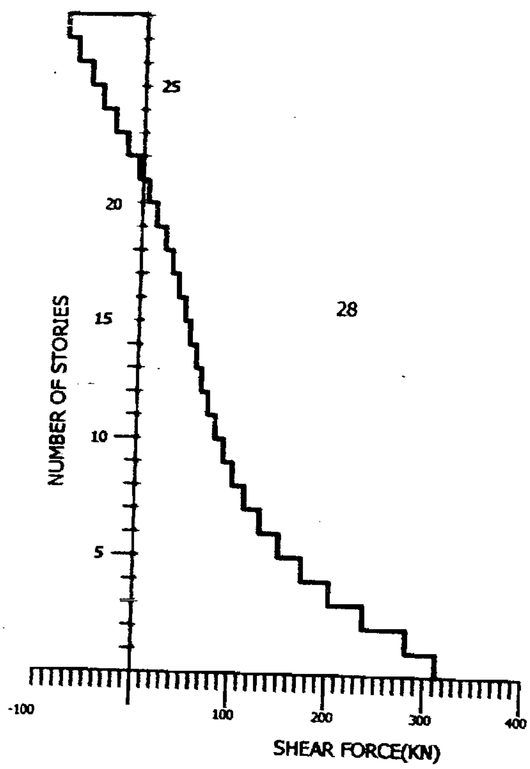


FIG.5.31. Variation of Shear Force in Shear Wall in Case 1 of Example 1

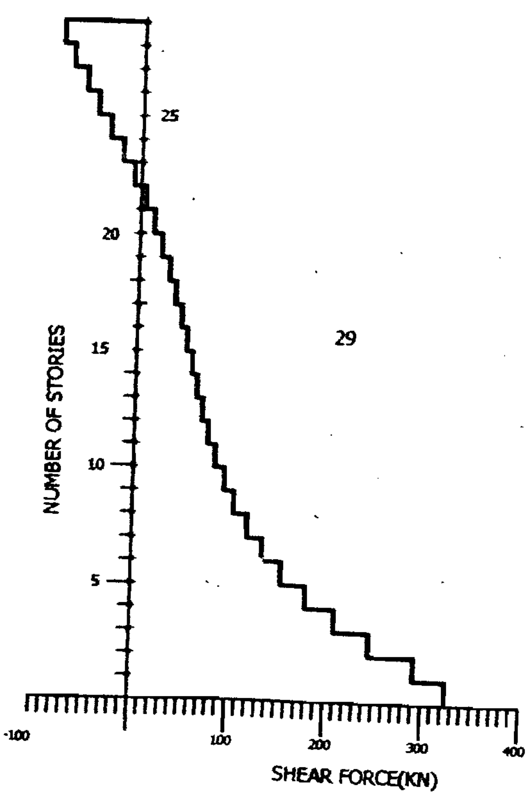


FIG.5.32. Variation of Shear Force in Shear Wall in Case 1 of Example 1

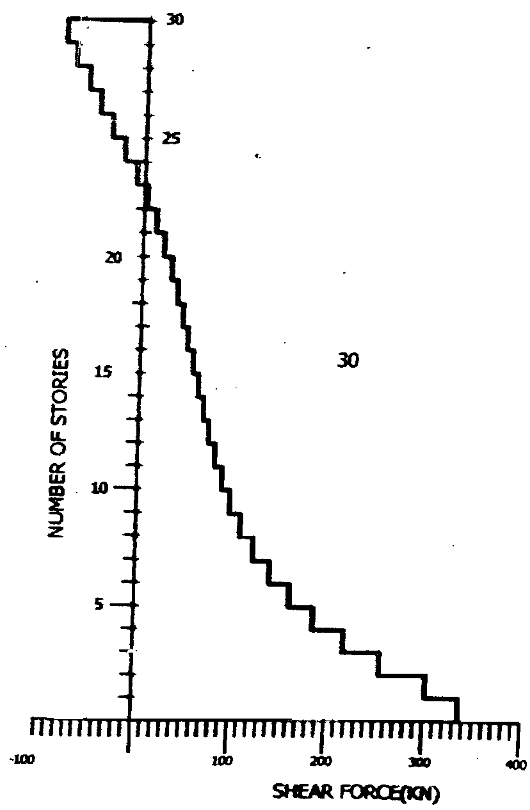


FIG.5.33. Variation of Shear Force in Shear Wall in Case 1 of Example 1

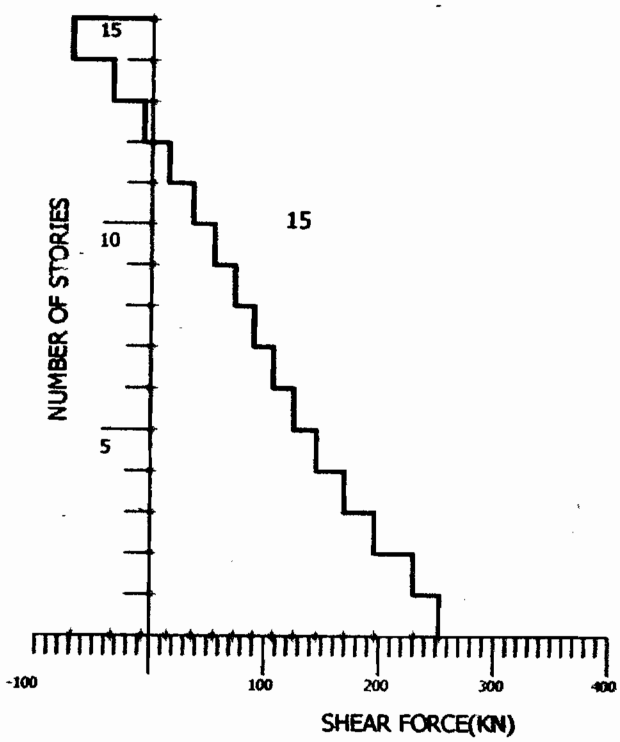


FIG.5.34(a) Variation of Shear Force in Shear Wall in Case 1 of Example 1(15 storeys)

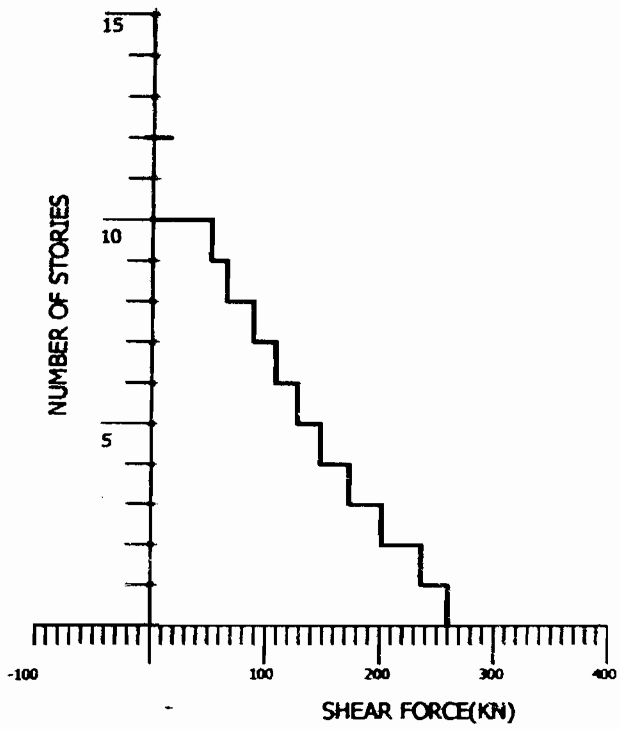


FIG.5.34(b) Variation of Shear Force in Shear Wall in Case 2 of Example 1

Fig: 5.35(a) Variation of bending Moment in Shear wall in case 1 of Example1 (15storey)

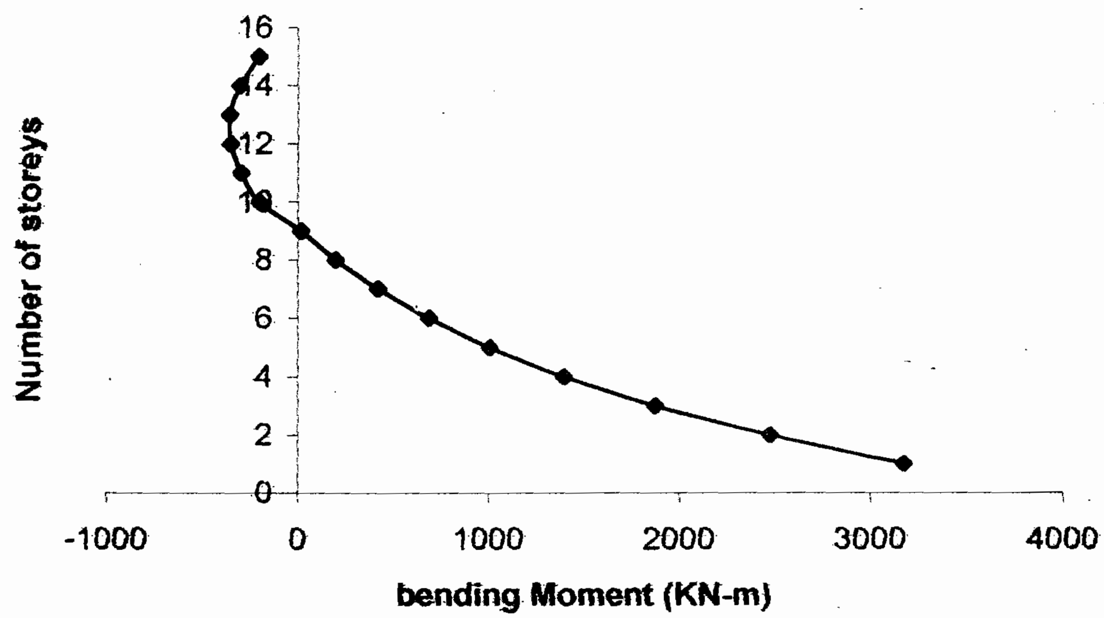
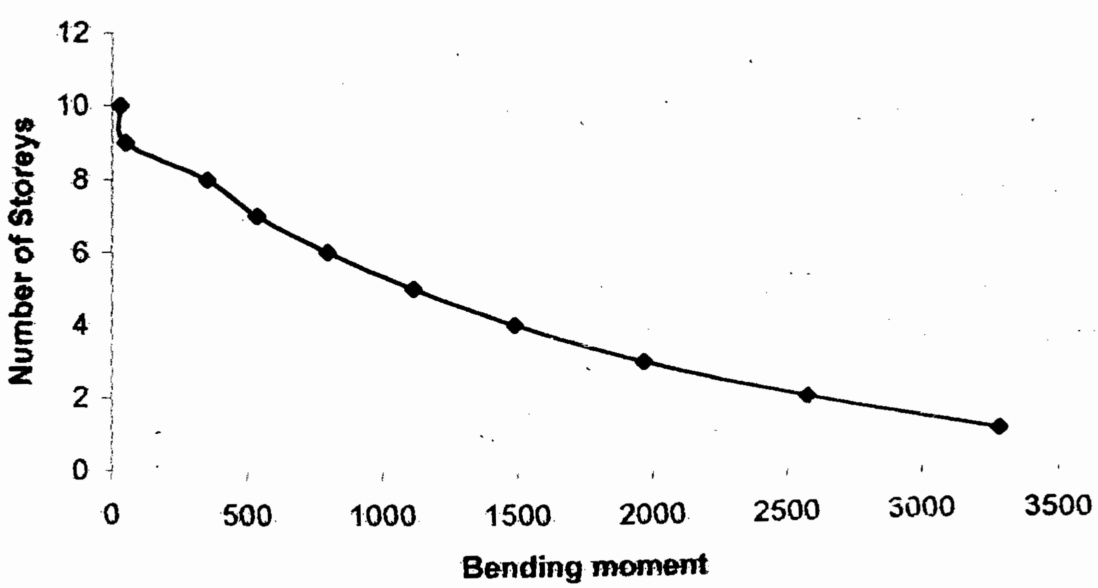


Fig:5.35(b) Variation of B.M.in shear wall in case 2 of example 1(15storey)



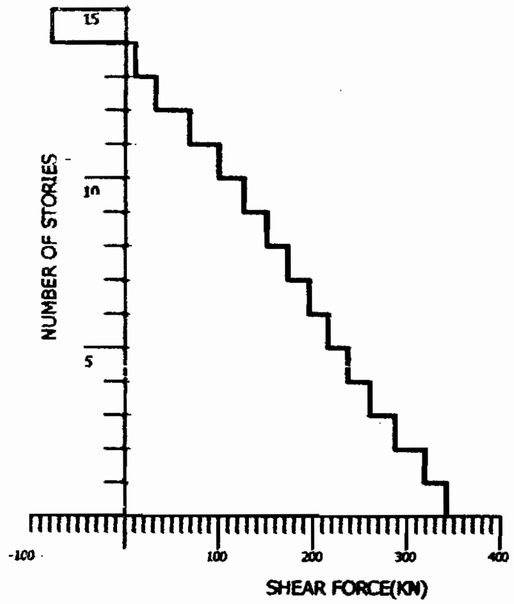


FIG.3.36(a)Variation of Shear Force in Shear Wall in Case 2 of Example 2

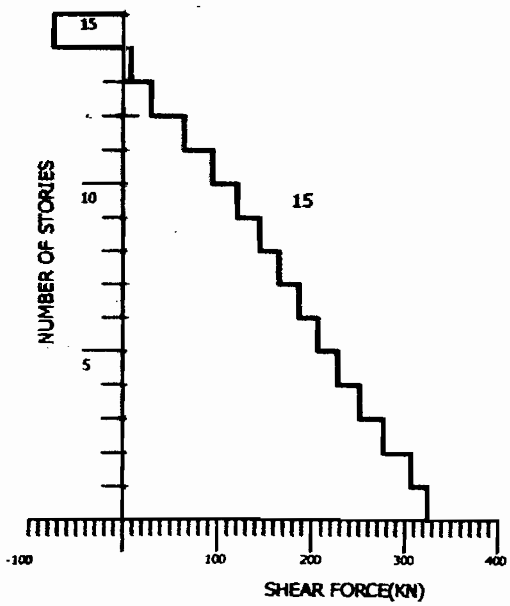


FIG.-3.36(b) Variation of Shear Force in Shear Wall in Case 1 of Example 2

Fig-5.37(a) Variation of bending Moment in Shear wall of 15 Storey in case 2 of Example 2

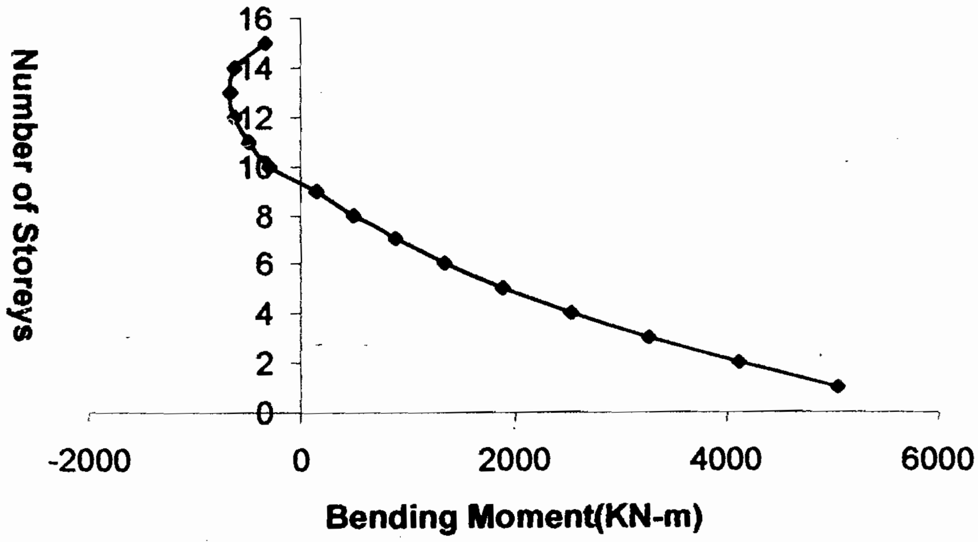


Fig-5.37(b) Variation of bending Moment in Shear wall of 15 Storey in case 1 of Example 2

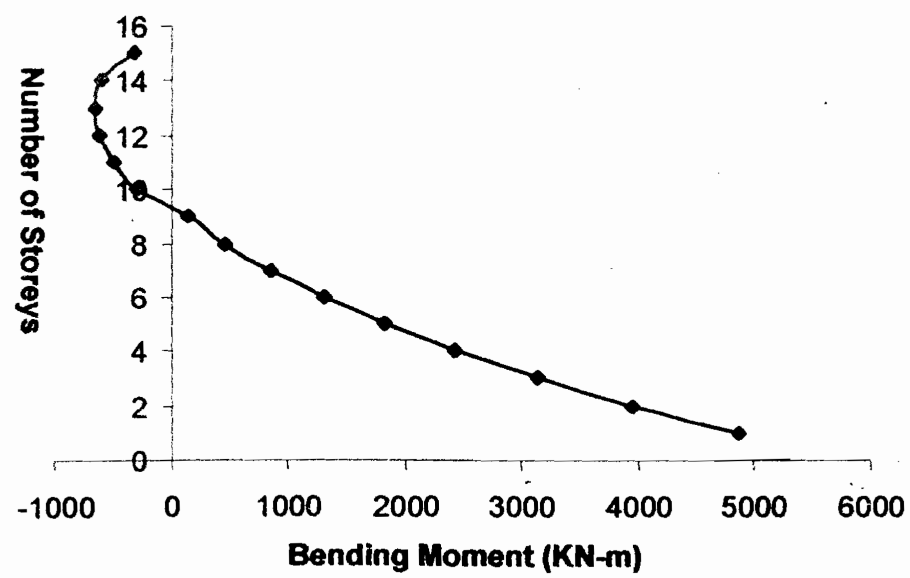


Fig: (a) Variation in Bending Moment in Shear wall of 15 storey of example2 Case 3

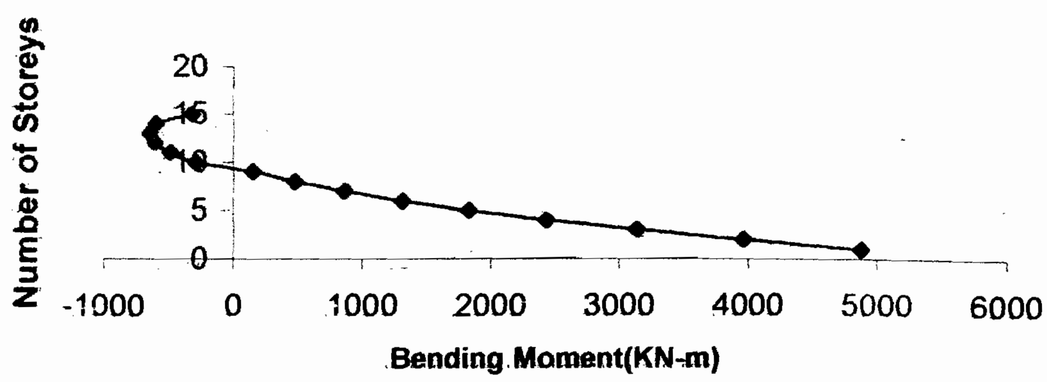


Fig:3.38 Variation in Bending Moment in Shear wall of 16 storey of example2 Case3

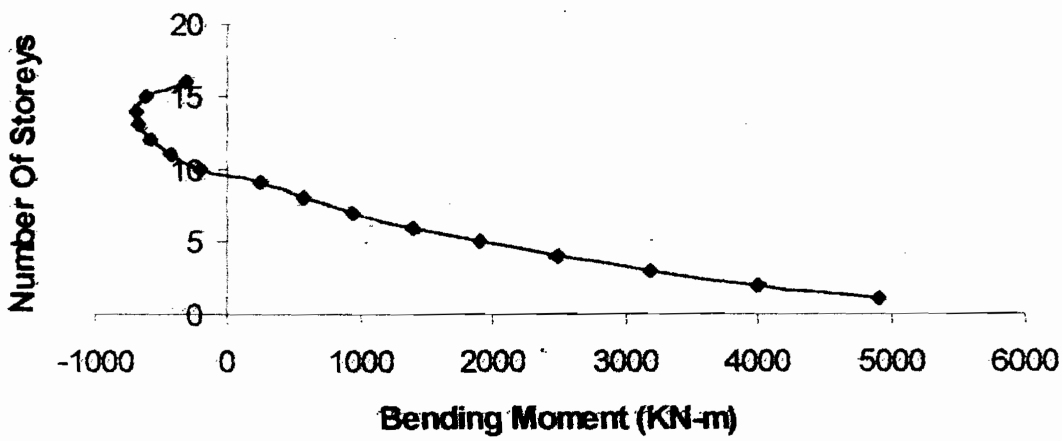


Fig: 5.39 Variation in Bending Moment in Shear wall of 17 storey of example 2 Case 3

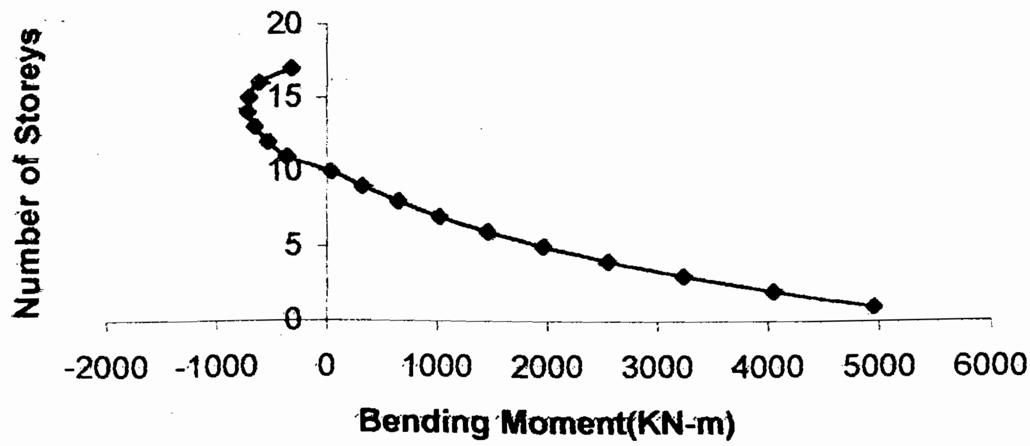


Fig: 5.41 Variation in Bending Moment in Shear wall of 19 storey of example 2 Case 3

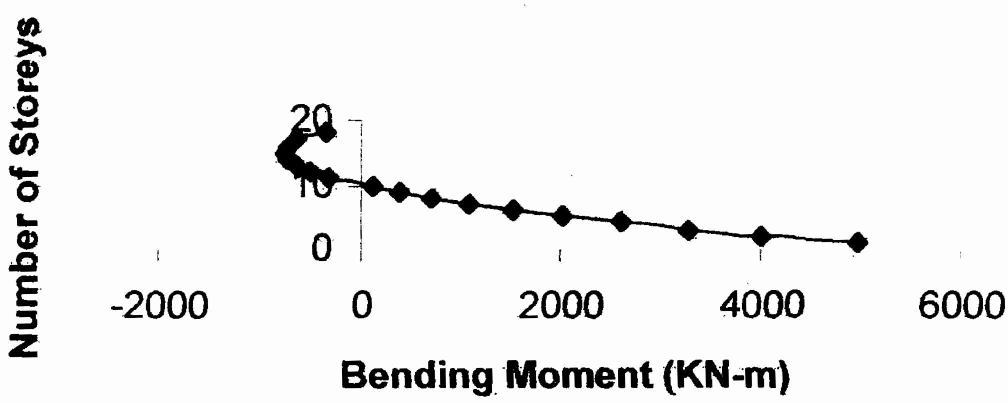


Fig: 5.42 Variation in Bending Moment in Shear wall of 20 storey of example 2 Case3

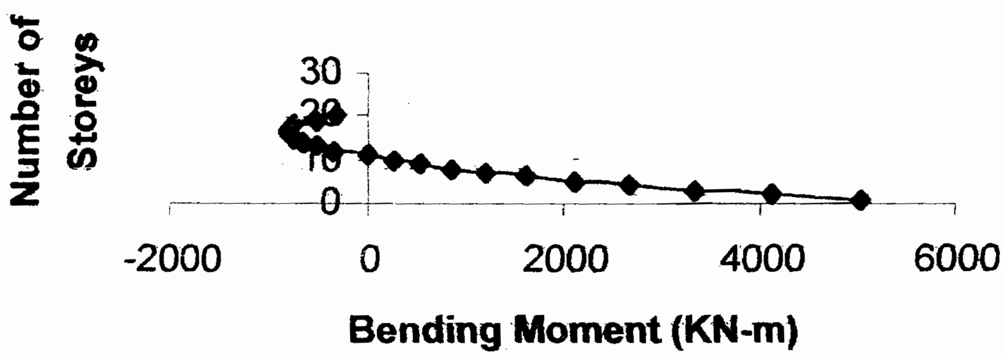


Fig: 5.43 Variation in Bending Moment in Shear wall of 21 storey of example 2 Case 3

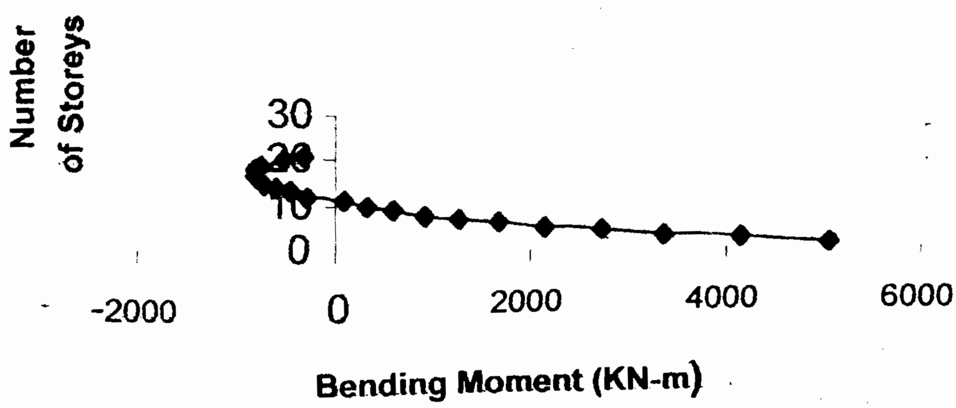


Fig: 5.44 Variations in Bending Moment in Shear wall of 22 storey of example 2 Case 3

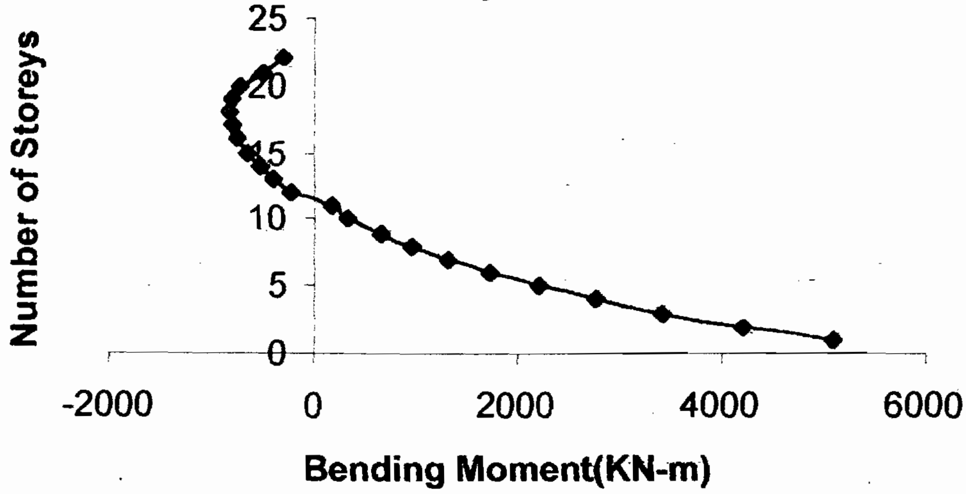


Fig: 5.45 Variation in Bending Moment in Shear wall Of 23 storey of example 2 Case 3

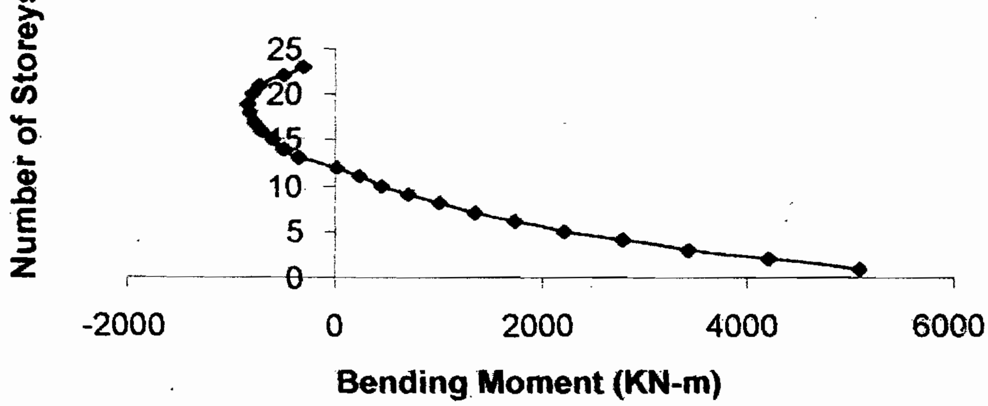


Fig: 5.46 Variations in Bending Moment in Shear wall of 24 storey of example 2 Case 3

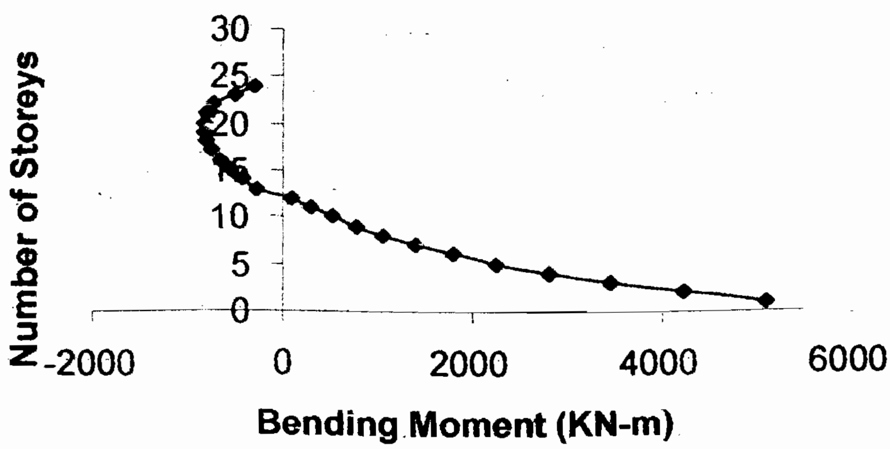


Fig: 5.47 Variation in Bending Moment in Shear wall of 25 storey of example 2 Case 3

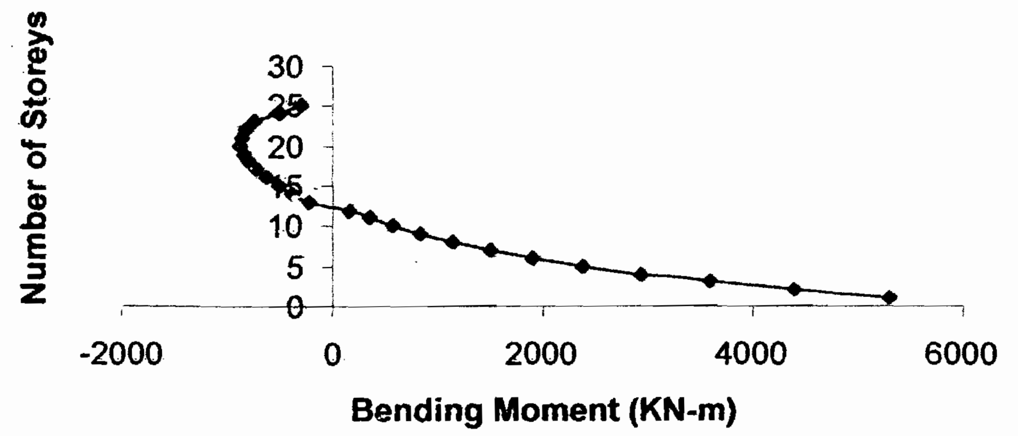


Fig: 5.48 Variation in Bending Moment in Shear wall of 26 storey of example 2 Case 3

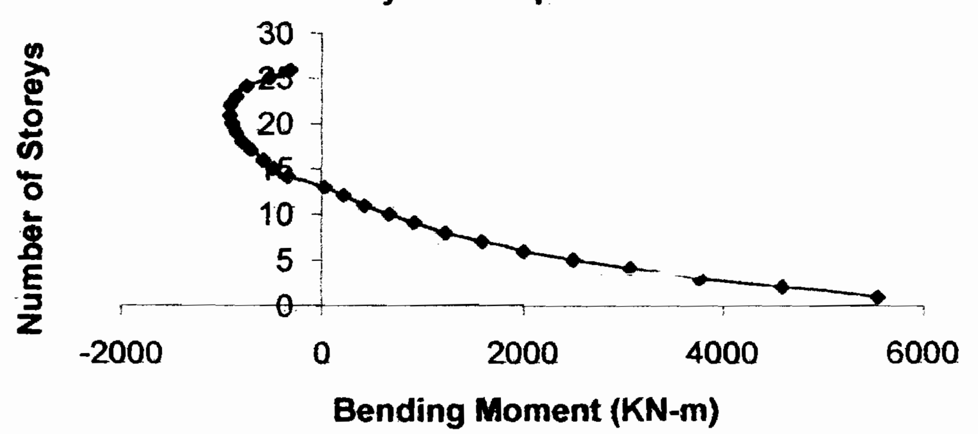


Fig: 5.49 Variation in Bending Moment in Shear wall of 27 storey of example 2 Case 3

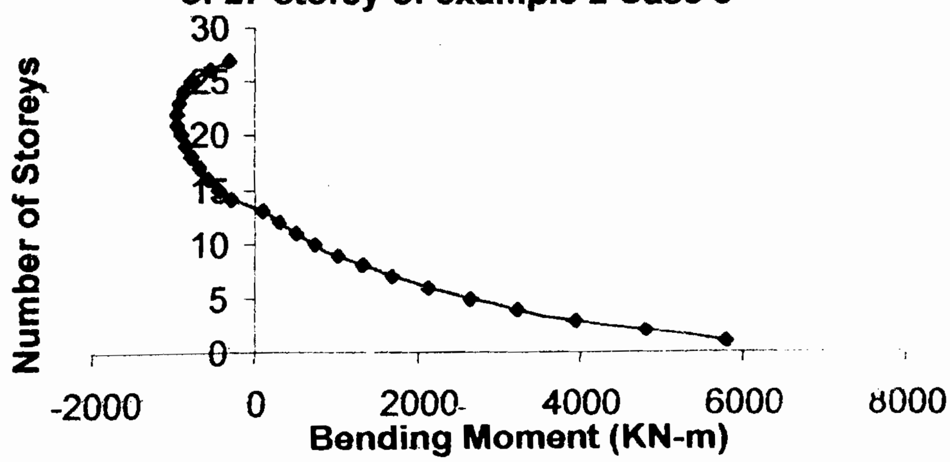


Fig: 5.50 Variation in Bending Moment in Shear wall of 28 storey of example 2 Case 3

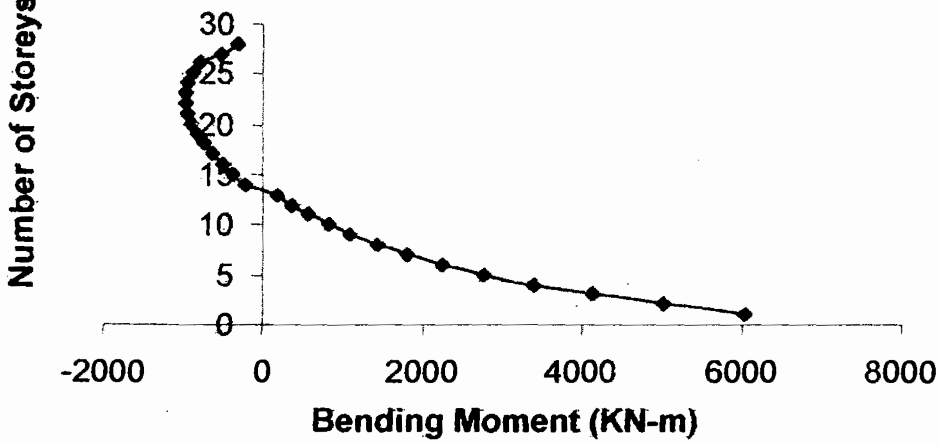


Fig: 5.51 Variation in Bending Moment in Shear wall of 29 storey of example 2 Case 3

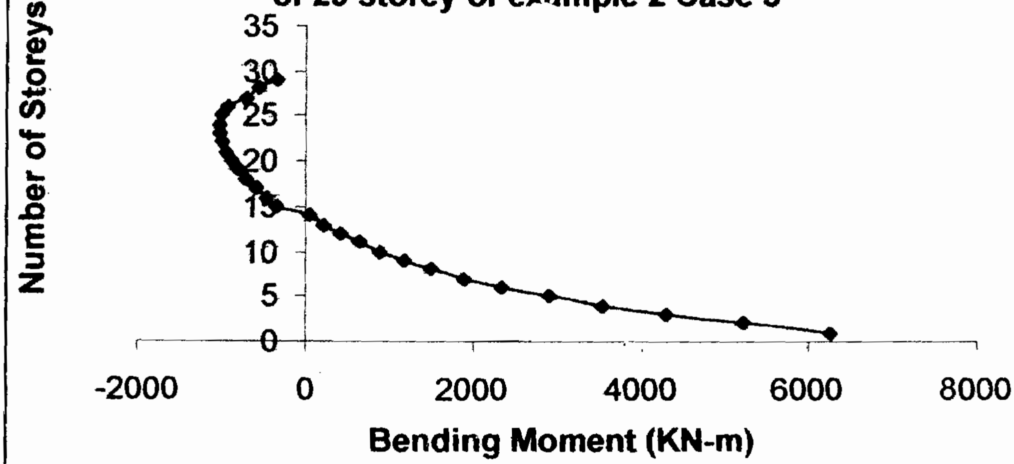
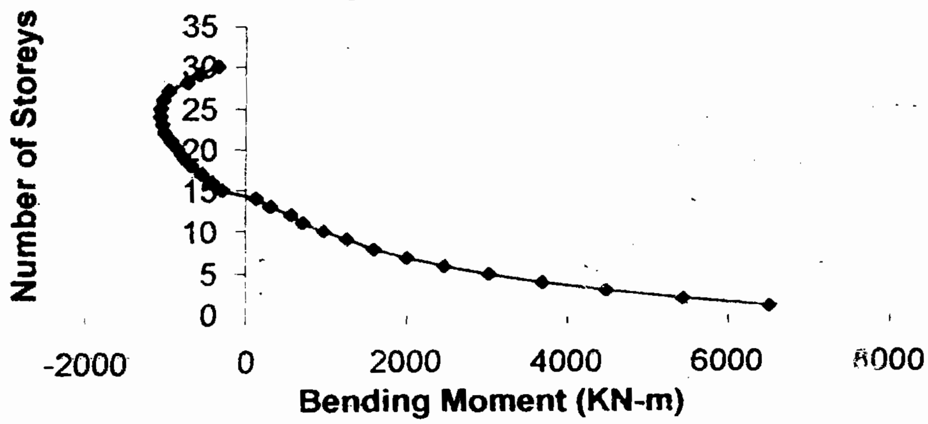


Fig: 5.52 Variation in Bending Moment in Shear wall of 30 storey of example 2 Case 3



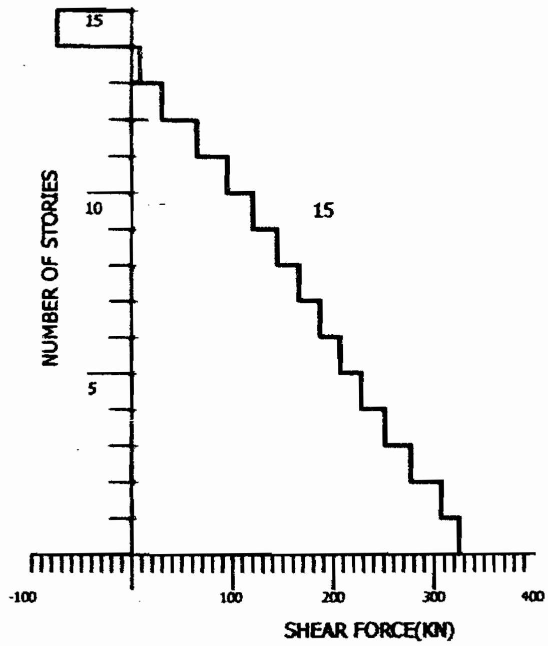


FIG. Variation of Shear Force in Shear Wall in Case 3 of Example 2

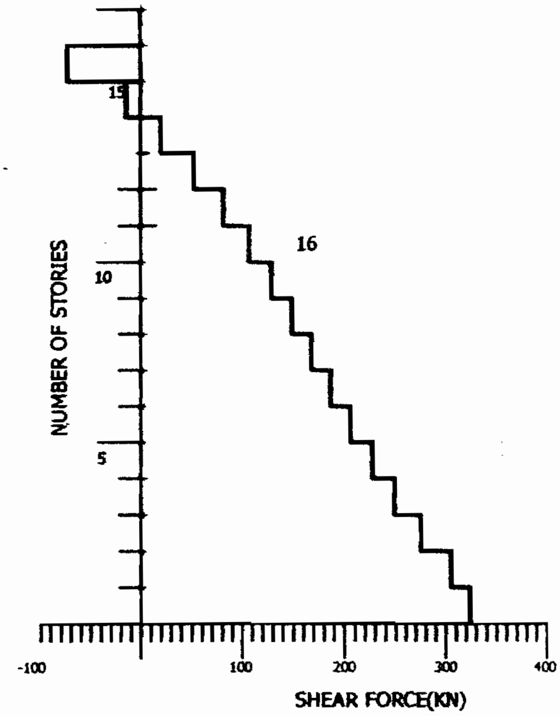


FIG.5.53 Variation of Shear Force in Shear Wall in Case 3 of Example 2

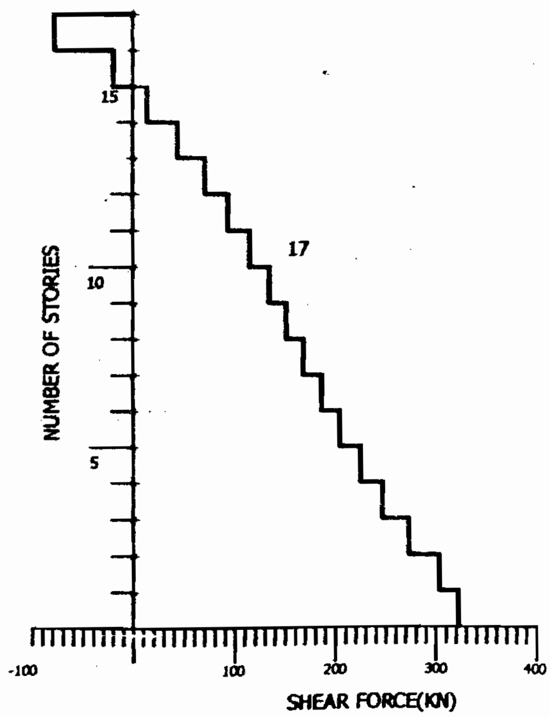


FIG.5.54 Variation of Shear Force in Shear Wall in Case 3 of Example 2

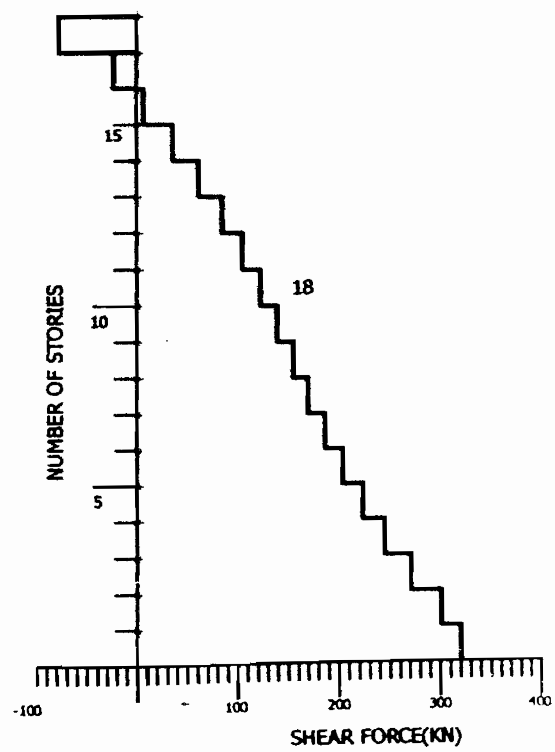


FIG. 5.55 Variation of Shear Force in Shear Wall in Case 3 of Example 2

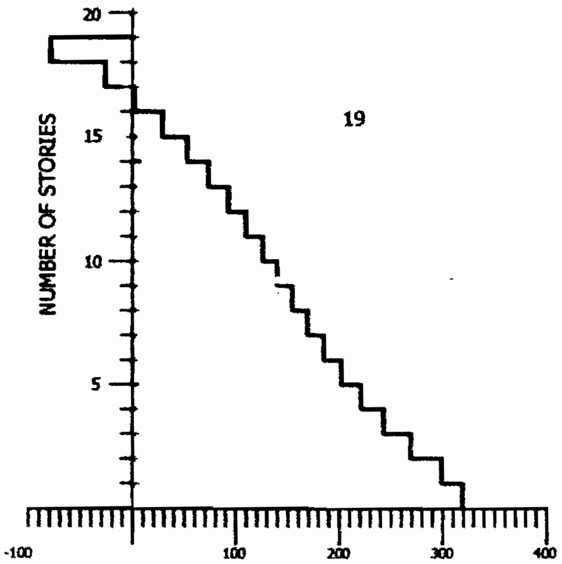


FIG.5.56 Variation of Shear Force in Shear Wall in Case 3 of Example 2

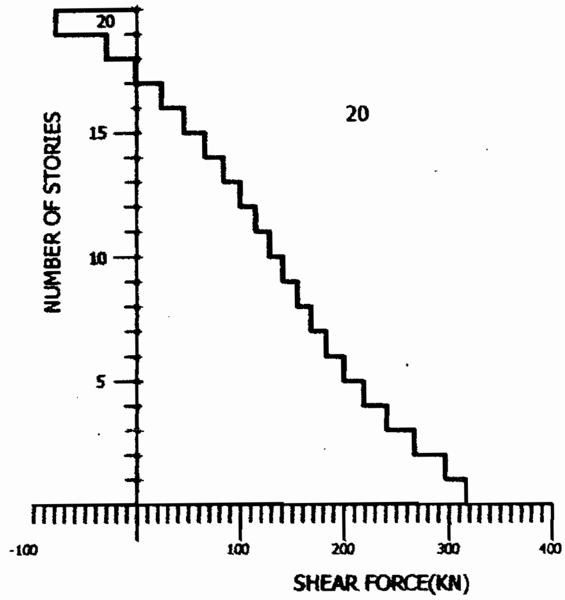


FIG.5.57 Variation of Shear Force in Shear Wall in Case 3 of Example 2

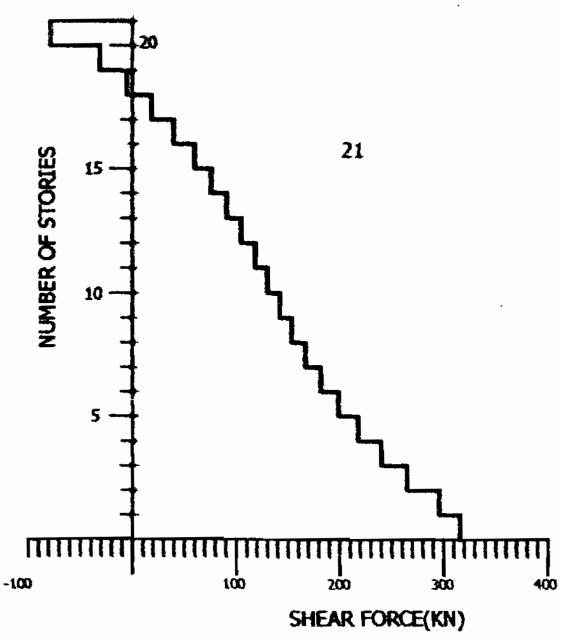


FIG.5.58 Variation of Shear Force in Shear Wall in Case 3 of Example 2

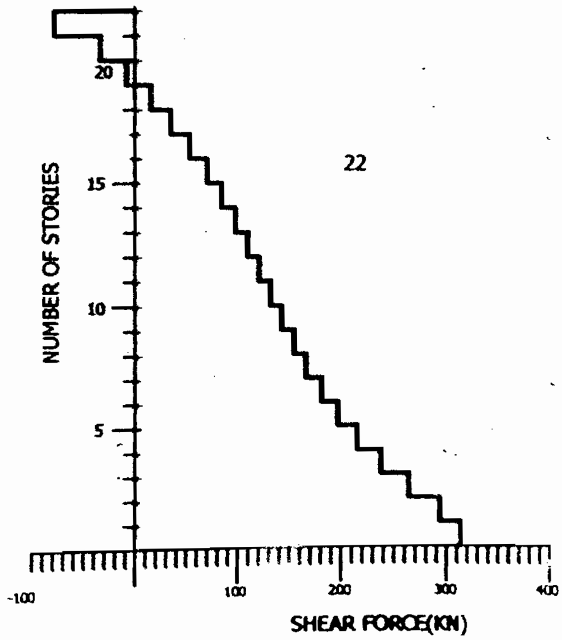


FIG.5.59 Variation of Shear Force in Shear Wall in Case 3 of Example 2

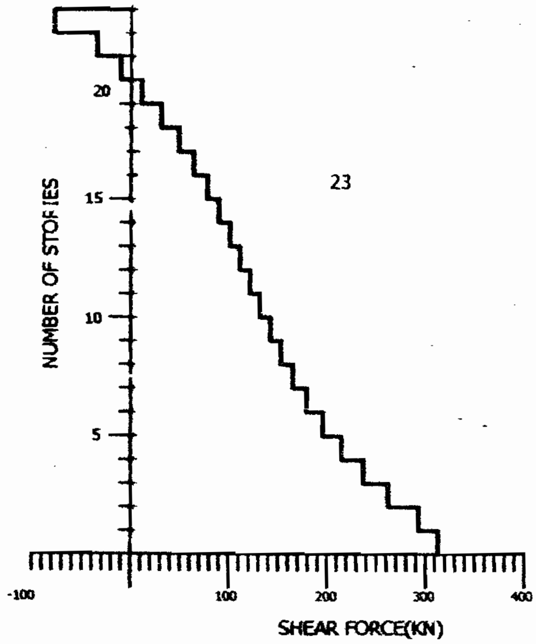


FIG. 5.60 Variation of Shear Force in Shear Wall in Case 1 of Example 2

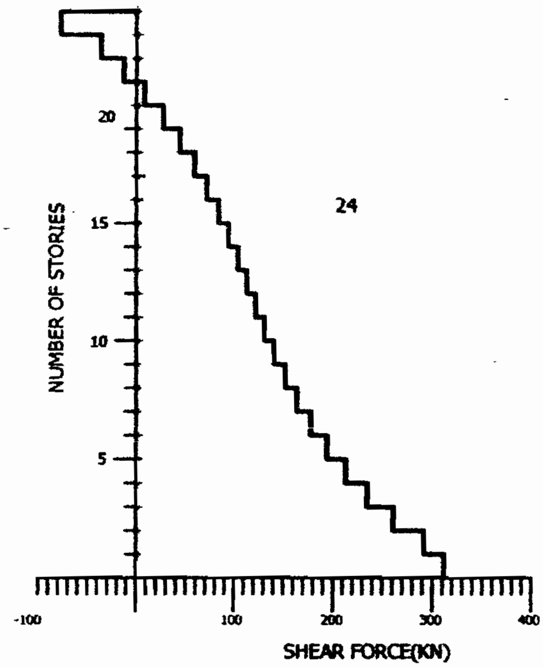


FIG.5.61 Variation of Shear Force in Shear Wall in Case 1 of Example 2

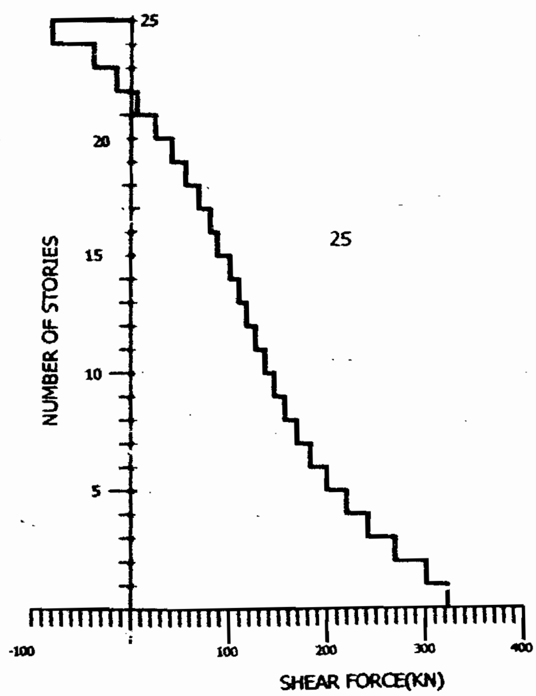


FIG.5.62 Variation of Shear Force in Shear Wall in Case 3 of Example 2

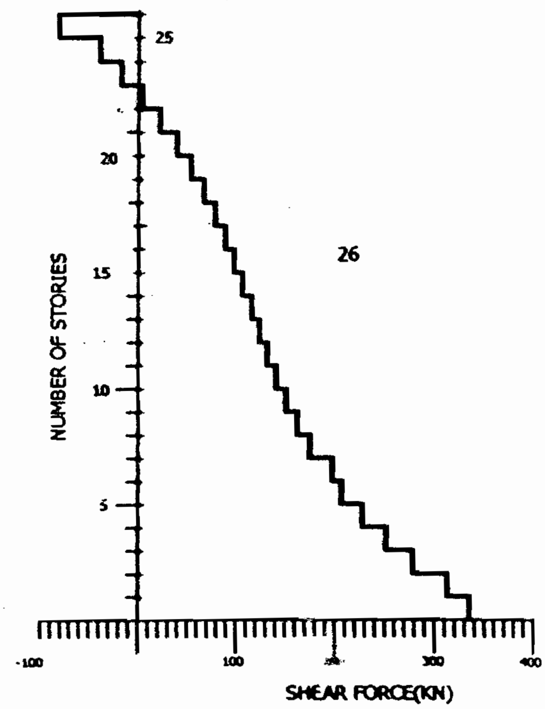


FIG.5.63 Variation of Shear Force in Shear Wall in Case 3 of Example 2

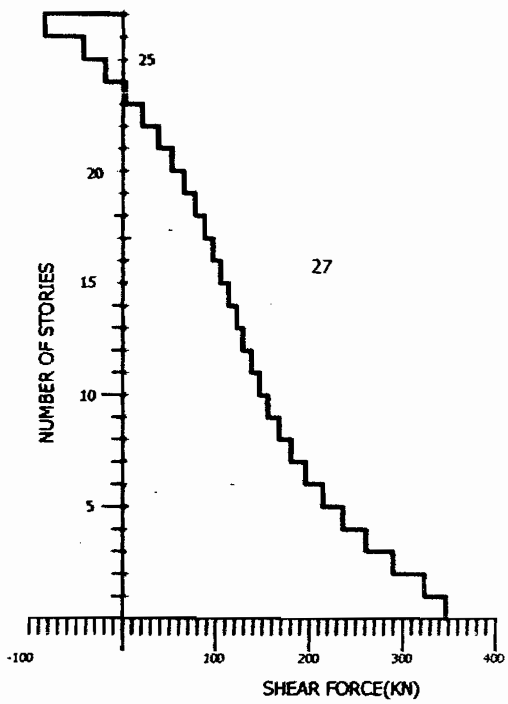


FIG.5.64 Variation of Shear Force in Shear Wall in Case 3 of Example 2

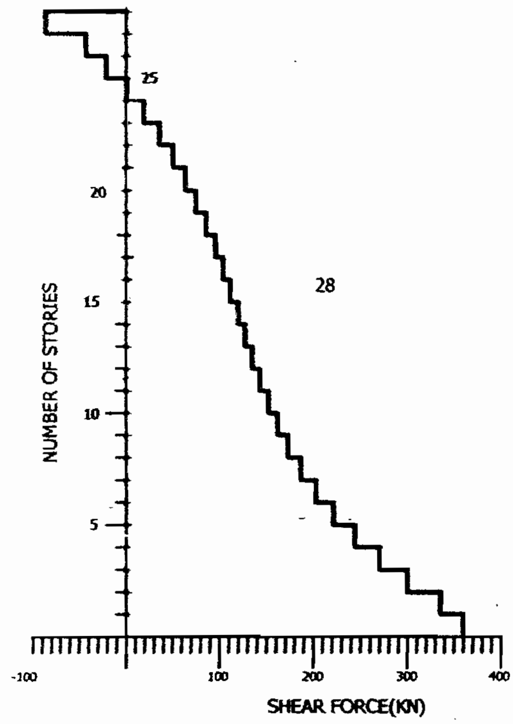


FIG.5.65 Variation of Shear Force in Shear Wall in Case 3 of Example 2

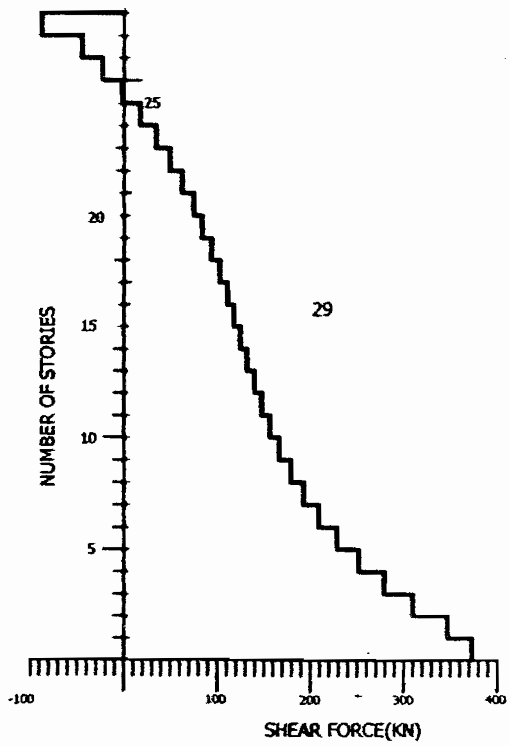


FIG.5.66 Variation of Shear Force in Shear Wall in Case 3 of Example 2

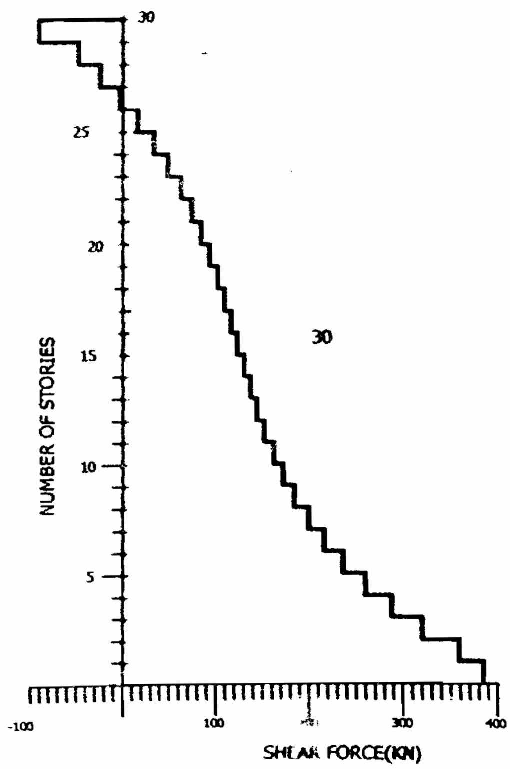


FIG-5.67 sys15 Variation of Shear Force in Shear Wall in Case 3 of Example 2

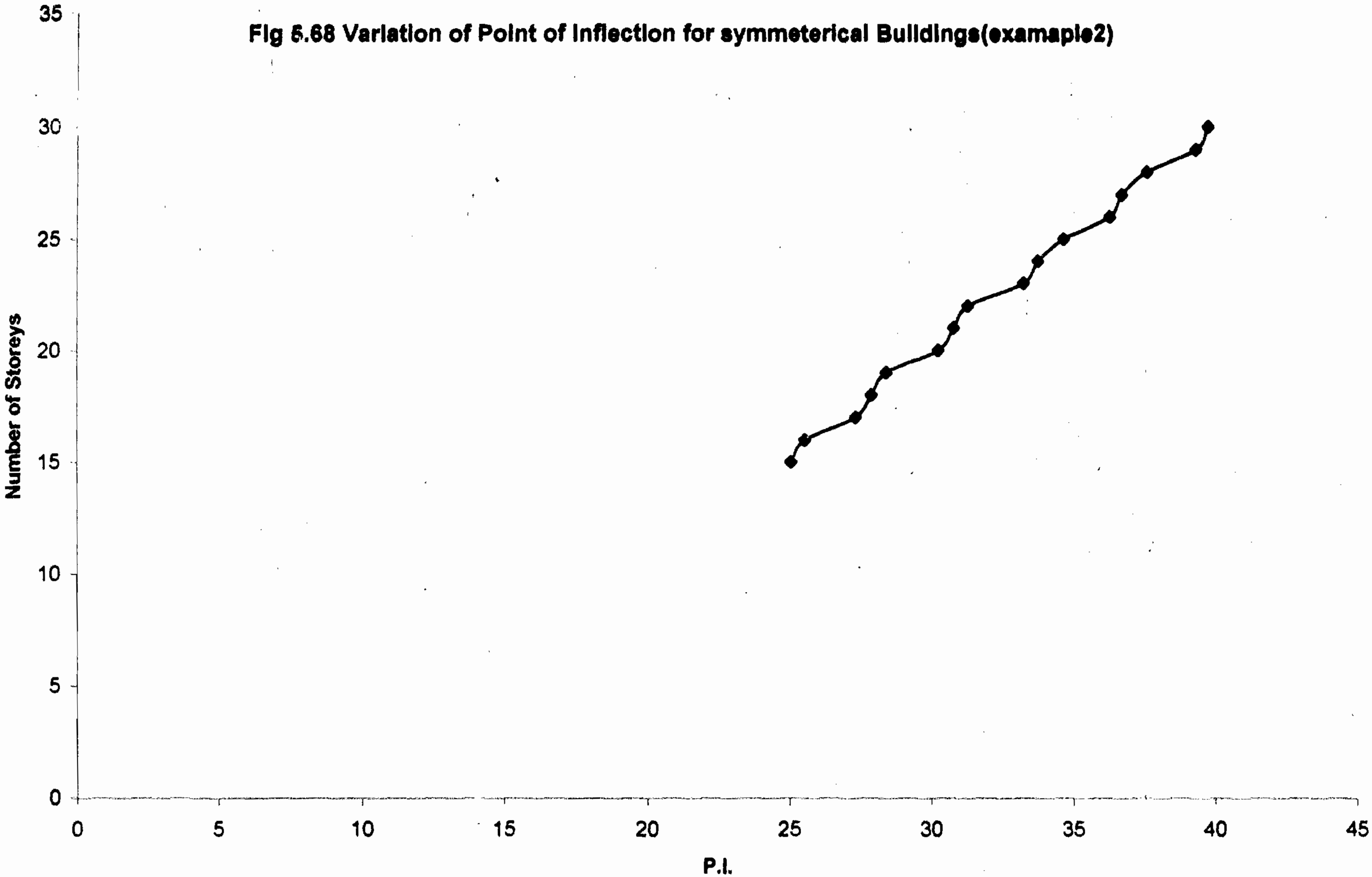


Fig 5.68 Variation of Point of Inflection for symmeterical Buildings(examaple2)

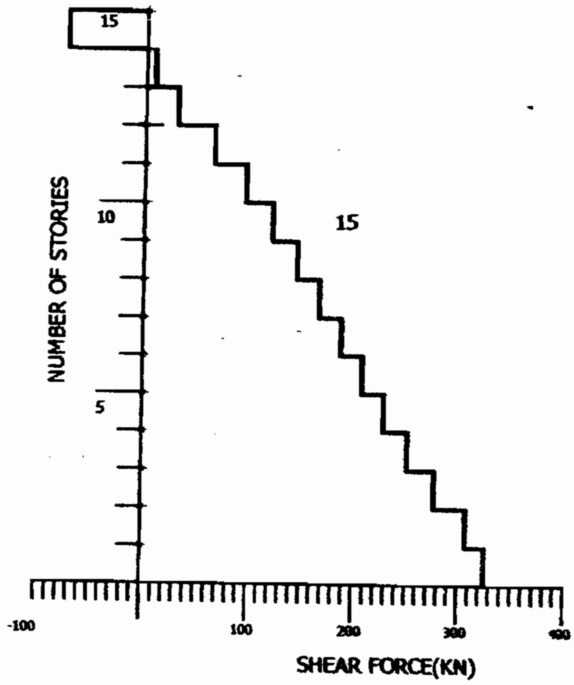


FIG.5.69(a) Variation of Shear Force in Shear Wall in Case 3 of Example 2

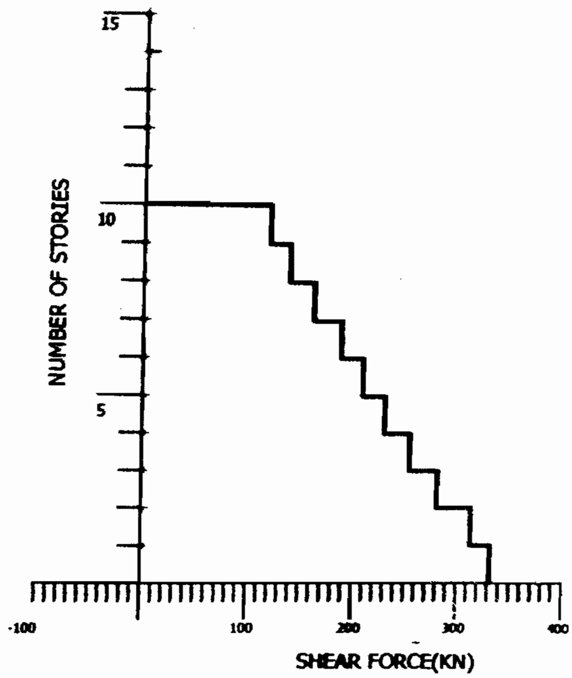
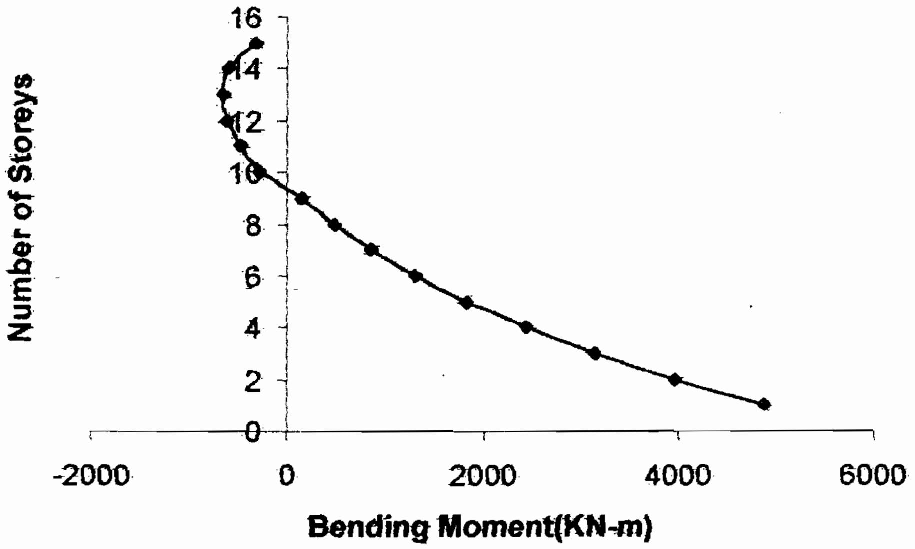
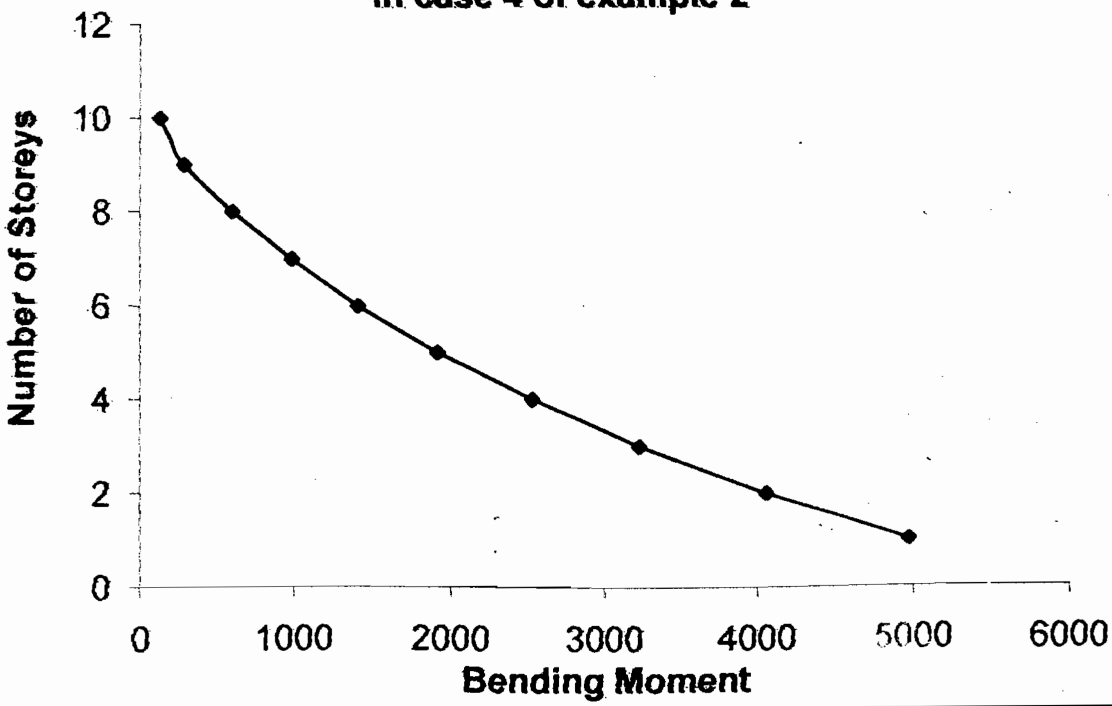


FIG.5.69(b)Variation of Shear Force in Shear Wall in Case 4 of Example 2

Fig.5.70(a)Variation of bending moment in Shear wall of 15 storey in case 3 of example 2



5.70(b)Variation of bending moment in Shear wall of 15 storey in case 4 of example 2



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