ANALYSIS & DESIGN OF TUBULAR STRUCTURES

A DISSERTATION

Submitted in partial fulfilment of the requirements for the award of the degree

of

MASTER OF TECHNOLOGY

in

CIVIL ENGINEERING

(With Specialization in Building Science and Technology)

By

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FEBRUARY, 2002

CANDIDATE'S DECLARATION

I hereby declare that the work presented in the dissertation report entitled, **"ANALYSIS & DESIGN OF TUBULAR STRUCTURES"**, submitted in partial fulfillment of the requirements for the award of the degree of **MASTER OF TECHNOLOGY** in **CIVIL ENGINEERING** with specialization in **Building Science & Technology** is an authentic record of my own work carried out under the humble guidance of **Prof. V.K. Gupta**, Professor, Structural Engineering Section, Department of Civil Engineering, Indian Institute of Technology, Roorkee.

I have not submitted the matter embodied in this dissertation report for the award of any other degree or diploma.

SKhourasiya

Date: 21st Feb., 2002

Place: Roorkee

(SATISH KUMAR CHOURASIYA)

CERTIFICATE

This is to certify that the above statement made by the candidate is correct to the best of my knowledge and belief.

VKGupta 21/2

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Professor,
Department of Civil Engineering,
Indian Institute of Technology, Roorkee.

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Last but not the least, I find myself fortunate enough to express my deep sense of gratitude to my parents and my other family members, who have forever been a constant moral support and strength to me.

Place: Roorkee

Dated: 21st Feb. 2002

SKChourasiya

(SATISH KUMAR CHOURASIYA)

ABSTRACT

Tubular buildings are the new structural concept in the development in high-rise buildings. This system is more efficient and economic in the use of material over a wide range of building heights than others. Out of many types of tubular buildings, two main types are:

1. Framed tube buildings
2. Tube-in-Tube buildings

In both types of structures, outer perimeter of columns is designed to resist lateral effects while inner columns & floors are assumed to take gravity loads.

In this dissertation, analysis & design of these buildings have been done using STAAD-pro 2001, a software package for the analysis and design of civil engineering structures. For the purpose of dissertation, the data of one 30-storeys and one 40-storeys building of both types have been taken from the literature. Then, their modeling has been done on STAAD-pro 2001. Finally, the buildings have analyzed followed by design with specifications of IS-codes.

Thus, by varying the building height (in terms of no. of stories), a parametric study has been done to study the effect on following parameters:

- Shear Lag Factor: - Shear Lag Effect is the phenomena of increase in axial stress in corner columns and reduction in axial stress in central columns in framed tube buildings due to the effect of wind loading.
- Storey Drift (% variation) :- at each fifth storey.
- Axial forces in columns at the same & different storey levels.
- Bending Moment in columns
- Axial stresses in columns.
- Variation in % of steel required for the design of:
    1 Beams
    2 Columns

The design of some critical elements has been done and an attempt has been done to determine the % variation in steel required in the design of both types of structures & then a comparative study has been done.

Design of a critical beam has also been done manually according to IS 456: 2000. Then, these hand calculations have been compared to computer results of design for the verification purpose.

Finally, graphs & tables have been drawn in between different parameters for different conditions using Microsoft Excel to make the study more effective and thus the results are obtained. A comparison has been made using different parameters in framed tube building and tube-in-tube building.

The study is limited to Static analysis. Fixed support conditions are assumed for all columns. All the structures have been designed for seismic zone III and correspondingly basic wind speed has been taken as 33 m/s. Aspect ratio of all buildings have been kept constant and its value is 3:5.

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CHAPTER 1

INTRODUCTION

1.1 GENERAL

The increasing rate of population, rapid industrialization and consequent shortage of land especially in metro cities has turned designers for construction in vertical direction.

In the past, conventional methods of construction were available, which restricted the buildings up to seven or eight stories. These low to medium- rise structures are normally designed for gravity loads, and then checked for their ability to resist lateral loads. However, for tall buildings the gravity load system cannot resist horizontal forces efficiently. Therefore, there was a need of such a type of structural system that can fulfill the requirements of resisting all types of load cases with economic point of view. [14]

A recent development in the structural design is the concept of tubular behavior introduced by Fazlur R. Khan. At present, four of the world's five tallest buildings are tubular system. They are the Hancock Building, the Sears Tower and the Standard Oil Building in Chicago, and was the World Trade Center in New York. Tubular systems are so efficient that in most cases the amount of structural material used per square meter of floor is comparable to that used in conventionally framed buildings half the size. [13]

1.2 VERTICAL STRUCTURAL SYSTEM [14]

Vertical structural system is the skeleton of structural members in a vertical plane formed by joining them together with suitable connections so that it can bear gravity and lateral loads acting on the structure and transfer them safely and economically to the ground.

1.2.1 Why More Efficient Vertical Structural System Required?

As the height of building increases with no corresponding increase in plan, width or depth, lateral forces resulting from wind and seismic effects become dominant consideration. Drift of the building needs to be strictly controlled both for the comfort of the occupant and to control secondary structural effects.

1.3 DEVELOPMENT OF FRAMED TUBE STRUCTURE [10], [8]

In the beginning the principle of masonry bearing wall structure was utilized for high-rise construction. But the high weight of the superstructure along with the inflexibility in plan made it inefficient for high-rise construction. Then the rigid frame system involving rigidly jointed beams and columns in a rectangular grid form was adopted. The idea that infilling the rectangular frame can provide stiffness to the system gave birth to shear wall system. The shear wall frame system increases lateral load bearing efficiency and reduced shear wall requirements. The logical extension of this system was the tubular system.

Structural systems based on tubular concept are most widely used today and likely to be used in future.

Even though initial forms were rectilinear, the rigidity and efficiency of the system for wind forces and the adaptability to create different shapes has been responsible for their broad upheaval.

F. R. Khan with De Witt Chestnut Apartment Building in Chicago introduced the tubular systems in the mid 1960's. Closely spaced perimeter columns and deep spandrel beams giving appearance of a punched tube formed the framed tube. This resulted in a system that behaved like a cantilever fixed at ground when subjected to lateral loads due to wind or earthquake. The interior floor framing was concrete flat plate with random arrangements of columns to suit apartment layout. The interior columns were primarily meant for only vertical load transfer.

There have been many example of this tubular system application for office and apartment buildings with columns spacing ranging from 4 ft to 15 ft. Some versions have included an interior shear wall tube for additional stiffness as a tube in tube system. The need for vertical modulation in a logical fashion has created a new type of tubular structure based on clustering and bundling smaller tubes each of which can resist rise to different height.

1.4 DEVELOPMENT OF TUBE-IN-TUBE SYSTEMS [13]

The tube-in-tube approach has been used in the 38-story Brunswick Building in Chicago, and the 52-story One Shell Plaza Building in Houston.

Taking the tube-in-tube concept one step further, the designers of a 60-story office building in Tokyo (Fig. 1.1) used a triple tube. In this system, the exterior tube alone resist wind loads, but all three tubes, connected by the floor systems interact in resisting earthquake loads, which is a significant design factor in Japan.

1.5 COMPARISON OF HIGH-RISE STRUCTURAL SYSTEMS [8], [13]

Fig. 1.2 illustrates different types of high-rise structural concepts to be suitable for certain building heights. Steel & concrete systems are presented separately. The chart is organized according to structural efficiency (i.e. optimization) as measured by the weight per sq. foot; that is, the weight of the total building structure divided by the total square footage of gross floor area.

Fig. 1.3 reveals the drastic increase in the amount of material needed for resistance of lateral forces for a five-bay rigid steel frame building. With respect to gravity loads, the weight of the structure increases almost linearly with the number of stories. However the amount of material needed for resistance of lateral forces increases at a drastically accelerating rate. The example shows the infeasibility of using the rigid frame principle with about 55 lbs/ft^2 (2.63 kN/m^2) for a 90-story building, instead of the tubular system with only 34 lbs/ft^2 (1.63 kN/m^2) (e.g. Standard Oil Building, Chicago). The selection of a particular structural system for a certain building height approaches that condition, as indicated by the broken line in Fig. 1.3.

Weight-to-area ratios for some typical high-rise buildings are given in the following table.

TABLE 1.1 SOME IMPORTANT HIGH-RISE BUILDINGS

Year	Stories	Height/Width	kN/m^2	Building
1930	102	9.3	2.02	Empire State Building, New York
1968	100	7.9	1.42	John Hancock Center, Chicago
1972	110	6.9	1.77	World Trade Center, New York

The frame-shear wall system of the Empire State Building is far from an optimum solution, as indicated by 2.02 kN/m^2 in contrast to the 1.42 kN/m^2 of the tubular John Hancock Center.

1.6 OBJECTIVES AND SCOPE OF THE PRESENT DISSERTATION

The present study deals with the analysis & design of symmetric framed tube structure by STAAD-pro 2001 and compares the same against the results of analysis & design of tube-in-tube structure. The verification of the results has been done by the manual design of a beam.

In addition to this, parametric study has been conducted to access the shear lag effect in both types of structure. The effect of increasing height of the building has also been seen on different parameters. For simplicity fixed support conditions are assumed at the base of the structure. The study is limited to static analysis. Dynamic analysis of the structure though essential for such heights has not been included in the present scope. Three-D Space analysis has been done using STAAD-pro 2001.

In this dissertation, an attempt has been made to determine the % variation in steel required in the design of both types of structures & then a comparative study has been done.

1.7 ORGANIZATION OF THESIS

Chapter 1 deals with the necessity of efficient vertical structural system and the historical development of framed tube and tube-in-tube structures along with a comparison of different types of structural concept.

Chapter 2 deals with the structural behavior of the framed tube and tube-in-tube structures along with theory of shear lag phenomena in framed tube buildings.

Chapter 3 deals with the data used for the analysis & design of different types of buildings. A detail about parametric study & assumption made for the analysis and design have also included in this chapter.

Chapter 4 deals with the input file and output file of software package STAAD-pro 2001 for analysis & design of different heights of buildings..

In Chapter 5, Results obtained are presented in the tabular form along with the graphical representation and discussion on the effect of different parameter due to building height.

Chapter 6 presents the conclusions arrived at the end of the study on the basis of parametric study and a remark on the optimum choice of tubular structure.

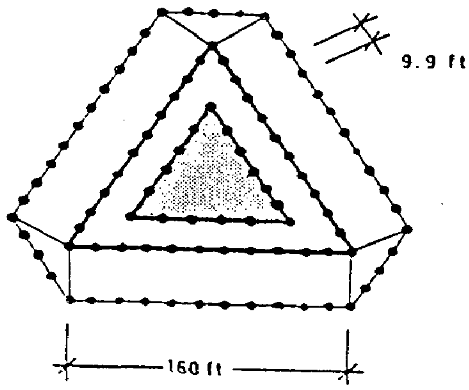


Fig. 1.1

A triple tube 60-storey office building in Tokyo

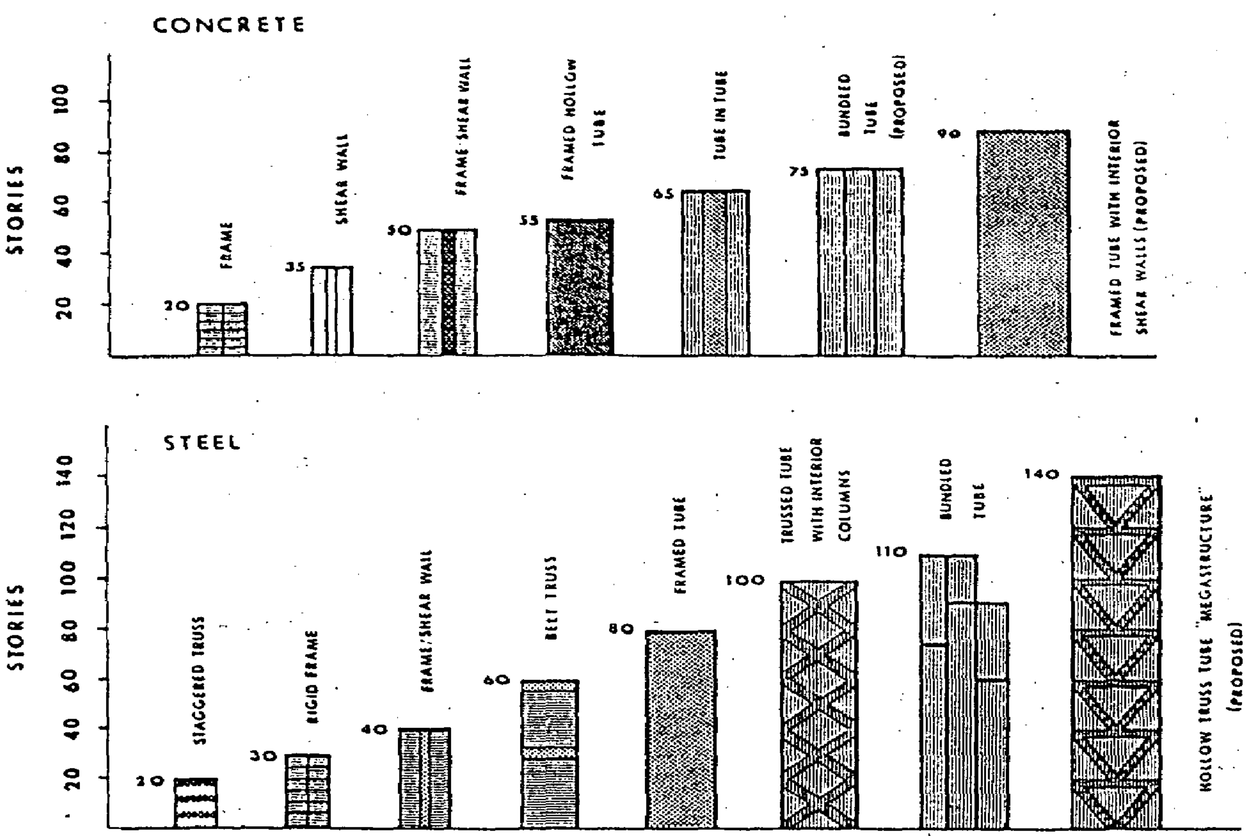


Fig. 1.2

High-rise structural concepts Vs No. of stories

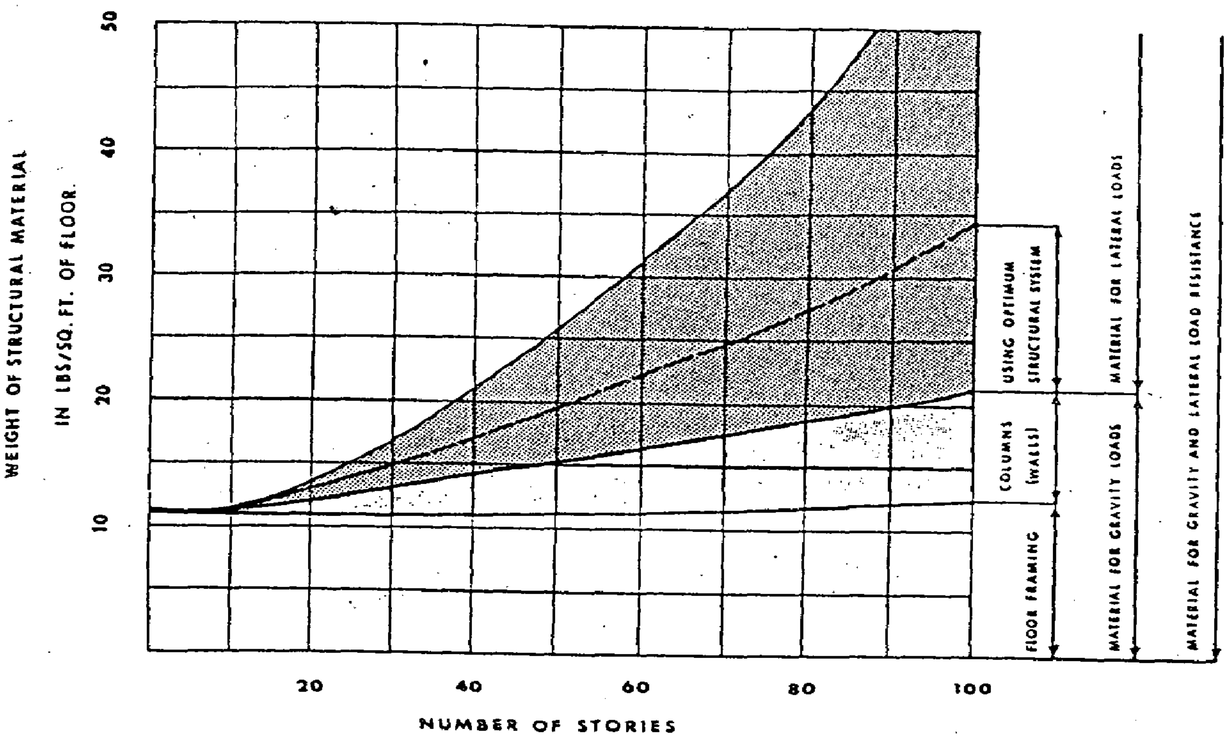


Fig. 1.3

Plot between Weight of the structural material Vs No. of Stories

CHAPTER 2
LITERATURE REVIEW

2.1 FRAMED TUBE STRUCTURES [9]

The framed tube structure is now widely accepted as an economic solution for tall structures of both steel and concrete. In its basic form, the system consist of closely spaced exterior columns tied at each floor level by spandrel beams to produce a system of four orthogonal rigidly jointed frame panels forming a rectangular tube system fig.2.1.

In many structures, the exterior tube is designed to resist the entire wind loading. The system has the advantage that it is compatible with traditional architectural arrangement for windows and it can be used for both commercial and structural requirements.

2.2 BEHAVIOUR OF THE FRAMED TUBE STRUCTURES [2]

In these structures frames parallel to the wind acts as the "webs" of the perforated tube cantilever, while the frames normal to the wind act as the "flanges". Vertical gravitational forces are resisted partly by the exterior frames and partly by some inner structure such as interior columns or an interior core, using the floor system that spans between the different vertical elements.

Though the structure has a tube like appearance, the behavior is more complex than a plain tube, and the stiffness is less. In addition to cantilever bending action, which produces tensile and compressive stresses on opposite faces of the tube, the side frames that are parallel to the lateral load undergoes the usual plane frame shearing action in each storey. This basic action is complicated by the fact that flexibility of spandrel beams produces shear lag that has the effect of increasing the stresses in the corner columns and reducing those in the inner columns. The later effect will produce warping of the floor slabs and consequent deformations of interior partitions and secondary structures making ideal beam theory no longer valid and stress variations much more complex. As a result it is important to predict accurately the structural behavior of the system in order to produce an efficient design. [Fig. 2.2]

2.3 TUBE-IN-TUBE STRUCTURES [7]

The term "tube-in-tube" is largely self-explanatory in that a second ring of columns, the ring surrounding the central service core of the building, are used as an inner framed or braced tube. The purpose of the second tube is to increase resistance to overturning and to increase lateral stiffness. The tubes need not be of the same character: that is, one tube could be framed, while the other could be braced. The system has been used for very tall buildings in both steel and concrete. Since outer-framed tube, "hull" is connected together with an internal elevator and service "core", hence this system is also termed as hull-core structure.

2.4 BEHAVIOUR OF TUBE-IN-TUBE STRUCTURES [2]

In the tube-in-tube structure, the inner tube bends with the same horizontal deflection as the outer tube, owing to the high inplane stiffness of the floor slab, and carries a proportionate share of the lateral load. When the core is symmetric, adding one-quarter of it in the same planer model may include it, connected by pin-ended axially rigid links to the web-frame system.

If the core acts as a simple cantilever, it may be modeled as a single equivalent column, as shown in fig. 2.3(b). If it is perforated, it may be treated as a wall with openings. Provided that the internal core can be modeled by an equivalent plane structure, it may always be linked to the outer framed-tube model to obtain the distribution of lateral forces on each component.

If the core cannot be treated as a plane element, or if the outer framed tube is not symmetrical, a three dimensional analysis must again be performed. The nodes of the interior core must either be constrained by a "rigid floor" option to deflect horizontally with the nodes of the exterior frame, or be connected to them by a fictitious horizontal frame of axially stiff links. Either of these techniques will simulate the rigid –plane actions of the floor slabs.

2.5 SHEAR LAG IN FRAMED TUBE STRUCTURES [12]

When the frame is loaded laterally especially at lower floors the axial force in the corner column is much larger than force in the central column of the flange frame. On the other hand forces in the web frames instead of growing smaller towards the centre linearly grow smaller much faster. This phenomenon is known as shear lag. [Fig. 2.4]

The ratio of stress at the centre column to stress at corner column is called shear lag factor. Shear lag factor of 0.7 is considered satisfactory in practice.

The tubular structures involve a range of related structural forms: framed tube, tube-in-tube, bundled (cellular) tube, braced tube and composite tube system. The original development was the framed tube, which under the action of the wind loading, suffers a considerable degree of shear lag in the normal-to-wind panels i.e. flange frames. Here one point should be noted that shear lag effect is seen only in framed tube structures due to the action of lateral loads especially wind load and there is no contribution of gravity loads in the shear lag phenomena.

The more efficient & improved models e.g. Tube-in-tube, bundled tube and braced tube have no problem of shear lag effect and they produce a more uniform axial stress distribution in the columns of the "normal" panels to the wind.

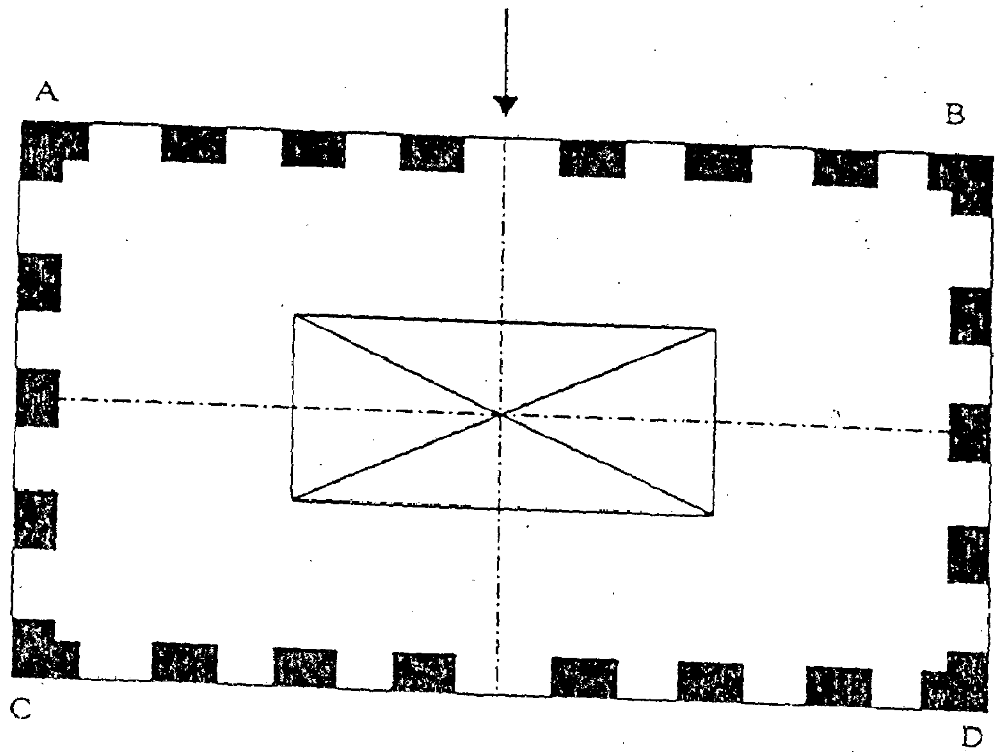


FIG 2.1 PLAN VIEW OF FRAMED TUBE MODEL

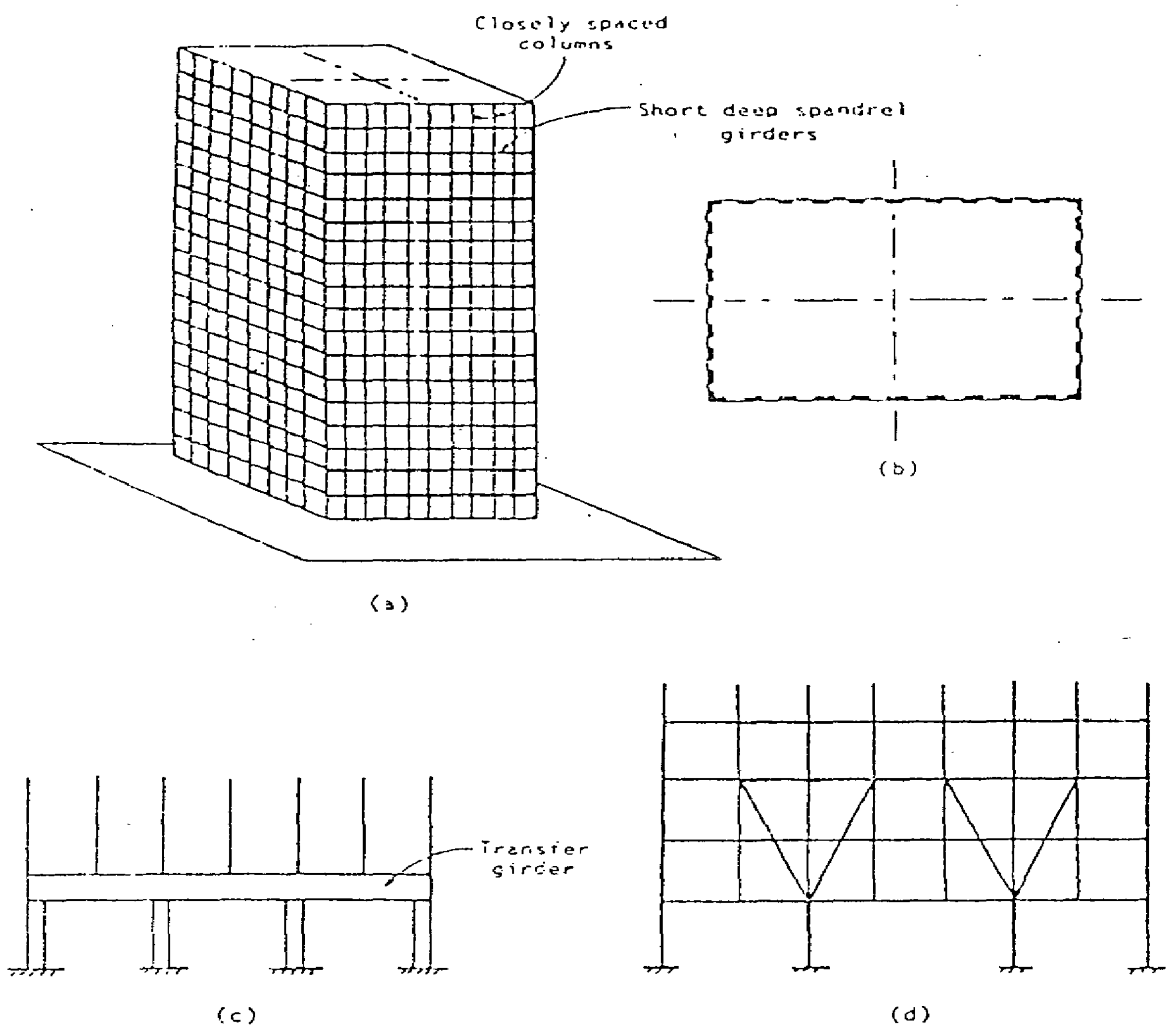


Fig. 2.2 Framed-tube structure. (a) Elevation; (b) structural plan; (c) transfer girder at base; (d) column collection at base.

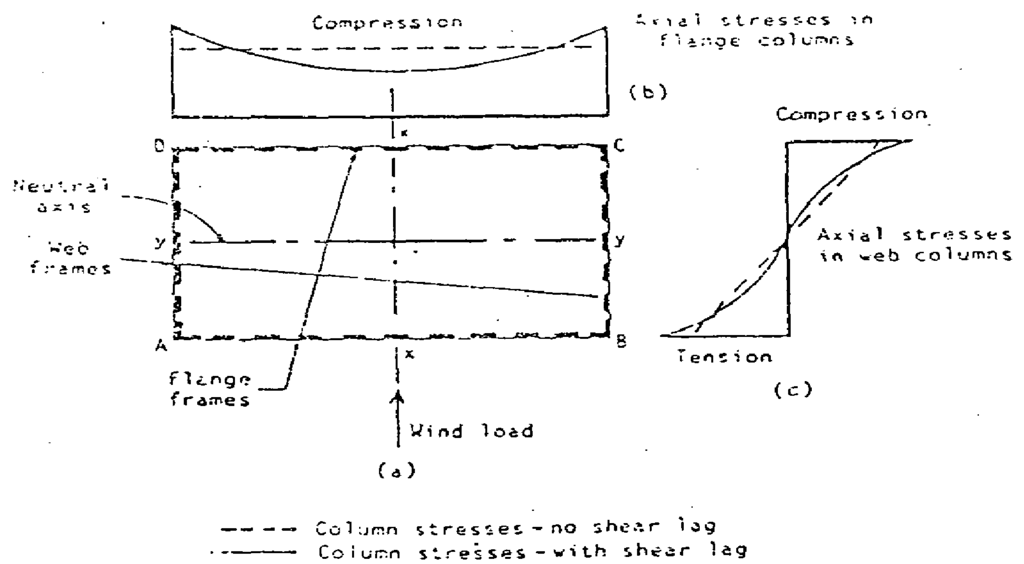


Fig. 2.4 Axial Stress Distribution in columns of laterally loaded framed tube

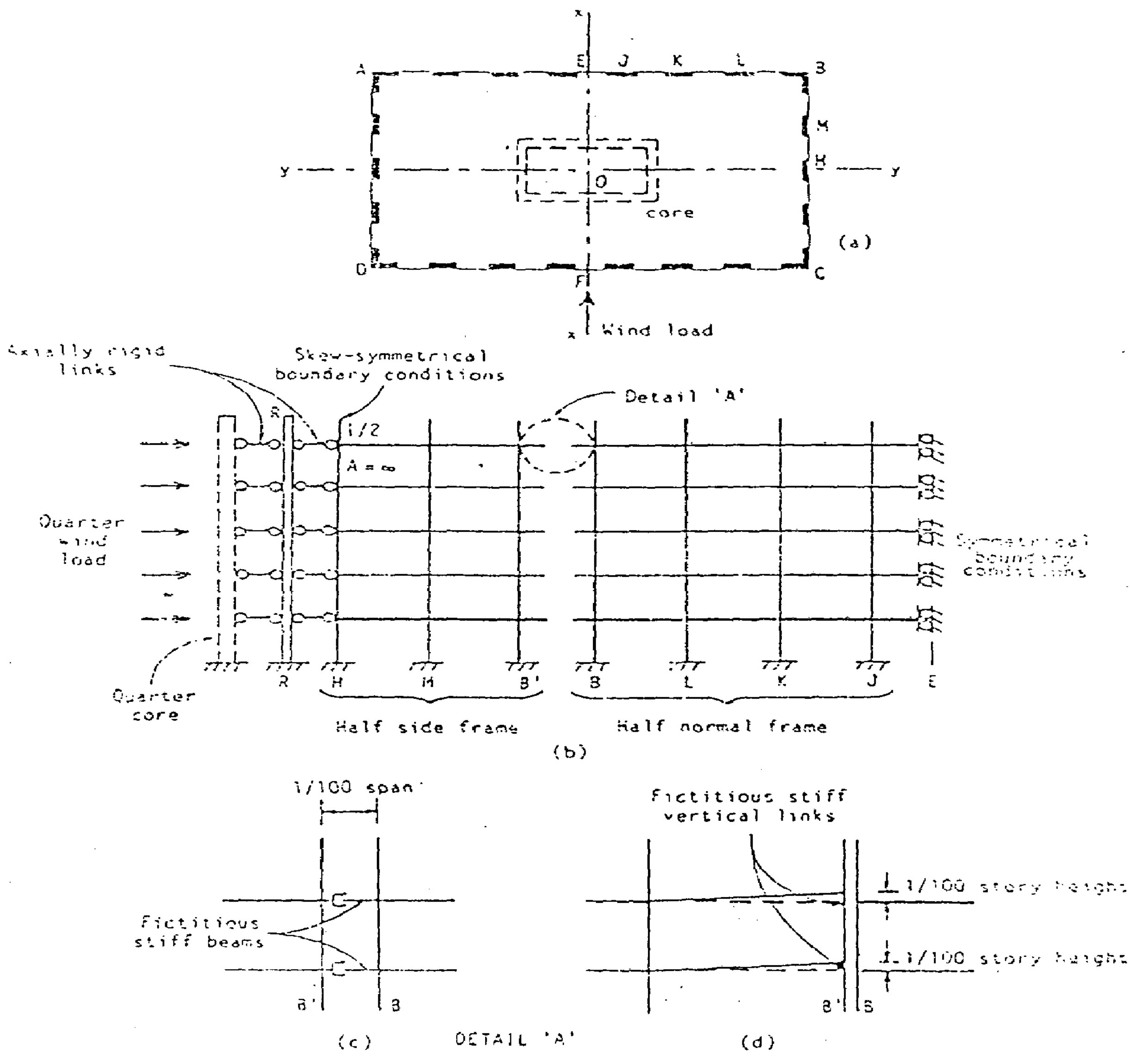


FIG 2.3 Tube-in-Tube Structure (a) Structural Plan; (b) Equivalent Planer Model; (c, d) Detail of alternative joint connection in planer model

CHAPTER 3

COMPUTATIONAL DETAILS

3.1 DESCRIPTION OF THE PROBLEM [1], [7], [10]

Problem of a 40 storeyed building has been chosen for the dissertation. The analysis ensures reasonable assumptions of no. of stories, storey height, sizes of columns and beams, bay width etc. before the investigation.

The plan of the building and the sectional view of the columns and the beams are given in Fig. 3.1. In the present case the number of stories has been taken as 40 and the storey height has been kept as 3.96m as reported in the literature.

The exterior framed tube dimensions are kept unchanged. The modulus of elasticity and the shear modulus of concrete have been kept same throughout the height of the building i.e. 3500 Mpa and 1521 Mpa respectively.

The dimensions are as follows:

TABLE 3.1 Cross-Sectional dimensions of beams & columns

Floors	h	b'	d	b
0-10	0.9	0.81	0.84	0.51
10-20	0.9	0.81	0.84	0.51
20-30	0.9	0.51	0.84	0.41
30-40	0.9	0.41	0.84	0.30

All dimensions are in meters.

h, b', b, d are the dimensions as shown in fig 3.1.

The other data are as follows:

Office live load	:	2.4 kN/m^2
Partition load	:	0.958kN/m^2
Seismic zone	:	III
Basic wind speed	:	33 m/s
Depth of slab	:	150 mm
Thickness of external panel wall	:	230 mm

Density of concrete	:	25 kN/m^3
Poisson's Ratio	:	0.15
Grade of concrete used	:	M30
Grade of steel used	:	Fe415
Density of masonry wall	:	20kN/m^3

For better comparison of the results the plan area of all the buildings have been kept same. The dimensions of plan of each building are 60.96m x 36.576m (200 ft x 120 ft) Thus, the aspect ratios of all the buildings have equal value and it is 3:5. The center-to-center column spacing is kept as 10 ft (3.048m).

The beam & column dimensions for two different building heights are given below:

TABLE 3.2 Elements Cross –Sections at Different Floor Levels

	30 Storeys Building		40 Storeys Building	
Floor level	**Columns**	**Beams**	**Columns**	**Beams**
0-10	0.90 x 0.81	0.84 x 0.51	0.90 x 0.81	0.84 x 0.51
10-20	0.90 x 0.51	0.84 x 0.41	0.90 x 0.81	0.84 x 0.51
20-30	0.90 x 0.41	0.84 x 0.30	0.90 x 0.51	0.84 x 0.41
30-40	-----------	-----------	0.90 x 0.41	0.84 x 0.30

3.2 PARAMETER VARIATION

The only single parameter varied is the Height of the Building in terms of the no. of storeys. For this purpose, 30-storeyed & 40-storeyed buildings of each type have been taken for the analysis. Each storey height has been kept 3.96m, hence the height of 30-storeyed building is 118.8m and that of 40-storeyed building is 158.4m.

3.3 PARAMETERS INVESTIGATED

In this dissertation, the effect of building height has been seen on Shear Lag Effect, a phenomena in high-rise framed tube buildings. The second important parameter investigated considered is Storey Drift in both types of buildings. Then, variation of Bending Moment and Axial Force distribution have been seen in columns of both flange frame & web frame of both types of buildings. These parameters have been observed by both ways viz. in the corner columns of a frame in all the storey levels (height wise) and in columns from one corner to other corner at the lowest storey level. Effect on Axial Stresses in columns also has been seen. Then, finally the amount of steel required for the design of beams & columns is taken into consideration.

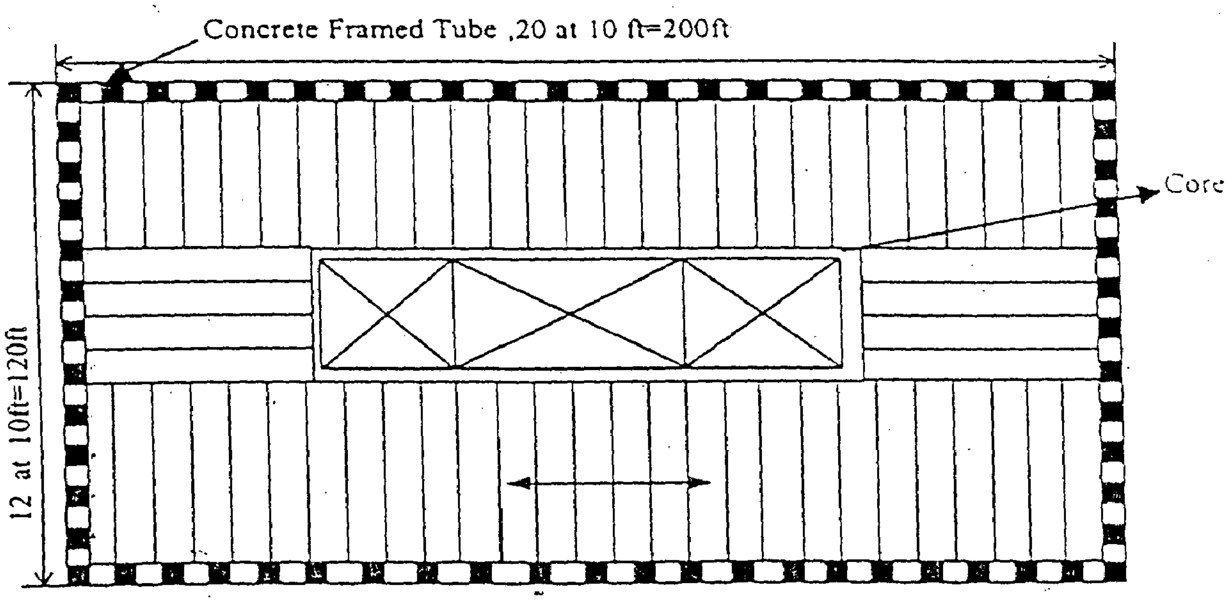
3.4 ASSUMPTIONS:

Following assumption are made for the analysis and design of tube-in-tube and framed tube buildings:

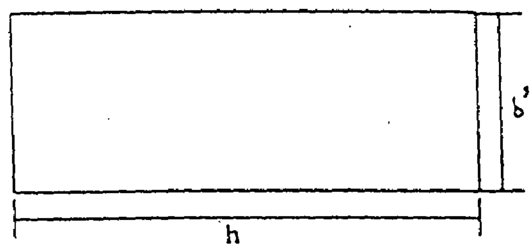
1. The entire flooring is assumed to be made of concrete and the floor slabs are assumed to be simply supported so that the entire lateral load due to earthquake, acting on the building is resisted only by the perimeter tube that is normally the case.
2. The interior columns and the shear core is assumed to resist vertical loads only.
3. The columns of external perimeter tube in each storey are designed to resist storey shear.
4. At any joint, column flexural strength is greater than the beam flexural strength.
5. The material of the structure and the structural components are linearly elastic. So, the analysis is limited to the elastic analysis.
6. Only the primary structural components participate in the overall behavior. The effects of the secondary structural components and nonstructural components are assumed to be negligible and conservative. But the effect of masonry infills has been taken into account.
7. Floor slabs are assumed to be rigid in plane. Thus the number of unknown displacements to be determined in the analysis is greatly reduced.

8. Component stiffnesses of relatively small magnitude are assumed negligible. These often include, for example, the transverse bending stiffness of slabs and torsional stiffness of columns, beams and walls.

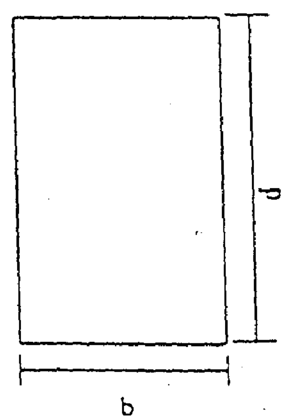
9. Deformations that are relatively small and of little influence are neglected. These include the shear and axial deformations of the beams, the axial deformations of columns and shear deformations of floor slabs.



(a) FLOOR PLAN



(b) CROSS SECTION OF A COLUMN



(c) CROSS SECTION OF A BEAM

FIG 3.1 A 40-storey Framed Tube Building

CHAPTER 4

INPUT AND OUTPUT FILES OF STAAD-pro 2001

4.1 GENERAL

In this chapter, input and output files are presented. Input files of STAAD-pro 2001 of a 30-storeyed framed tube building and of a 40-storeyed tube-in-tube building are shown. Joint coordinates, member incidences and element incidences are shown just for example. In both the input files, very limited commands of STAAD-pro 2001 have been shown. Since they are just for illustration purpose, in actual practice an input file is run for many times with different commands as per the requirements. In fact, all joint coordinates, member incidences and element incidences cannot be shown in these files, since they will consume a large space.

In output files, design of only two beams and two columns are presented of a 30-storeyed framed tube building. Analysis part is not shown in this file since it has been presented in tabular form in tables 5.1 to 5.10 with a different manner along with necessary plots.

4.2 INPUT FILE FROM STAAD-PRO 2001 OF A 30 - STOREY FRAME TUBE BUILDING

```
STAAD SPACE
START JOB INFORMATION
JOB NAME DISSERTATION
JOB NO 1
ENGINEER DATE 07-Nov-01
END JOB INFORMATION
INPUT WIDTH 79
UNIT METER KN
JOINT COORDINATES
1 0 0 0; 2 0 3.96001 0; 3 12.2 3.96001 0; 4 12.2 0 0; 5 3.05 3.96001 0;    etc.

MEMBER INCIDENCES
1 1 2; 2 2 5; 3 3 4; 4 5 6; 5 6 7; 6 7 3; 7 3 10; 8 8 9; 9 10 11; 10 11 12;    etc.

ELEMENT INCIDENCES SHELL
231 2 3 30 29; 232 3 8 35 30; 233 8 13 40 35; 234 13 18 45 40; 235 18 23 50 45; etc.

MEMBER PROPERTY INDIAN
```

5094 5096 TO 5099 5101 TO 5104 5106 TO 5109 5111 TO 5114 5116 TO 5139 5141 -
6160 TO 6162 6164 TO 6167 6169 **PRIS YD 0.84 ZD 0.3** etc
3620 TO 3622 3624 TO 3627 3629 **PRIS YD 0.84 ZD 0.41** etc
1085 TO 1087 1089 **PRIS YD 0.84 ZD 0.51**

7460 7465 7470 7475 7480 7505 7507 7512 7517 7522 7527 7561 TO 7631 -
7632 PRIS YD 0.91 ZD 0.81

ELEMENT PROPERTY
231 TO 242 7633 TO 7980 **THICKNESS 0.15**

CONSTANTS
E CONCRETE MEMB 231 TO 242 7633 TO 7980
E CONCRETE MEMB 1 TO 230 243 TO 7632
DENSITY CONCRETE MEMB 1 TO 230 243 TO 7632
POISSON CONCRETE MEMB 1 TO 230 243 TO 7632
DENSITY CONCRETE MEMB 231 TO 242 7633 TO 7980
POISSON CONCRETE MEMB 231 TO 242 7633 TO 7980

SUPPORTS
1 4 9 14 19 24 28 31 36 41 46 51 55 58 63 68 73 78 82 85 90 95 100 105 121 -
122 TO 123 127 TO 132 136 TO 153 157 TO 159 163 TO 204 **FIXED**

DEFINE **1893 LOAD**
ZONE 0.04 I 1.5 K 1 B 1
SELFWEIGHT
MEMBER WEIGHT

DEFINE **WIND LOAD**
TYPE 1
INT 1.87 HEIG 118.8
EXP 1 JOINT 2 29 56 83 109 TO 111 154 TO 156 160 TO 162 205 226 247 268 .. etc.

LOAD 1 1893 LOAD IN X-DIRECTION
1893 LOAD X
LOAD 2 WIND LOAD IN X-DIRECTION
WIND LOAD X 1 TYPE 1
LOAD 3 DL
SELFWEIGHT Y -1
LOAD 4 WALL LOAD
LOAD 5 LL
ELEMENT LOAD
231 TO 242 7633 TO 7980 PR GY -3.358

LOAD COMB 6 1893(X)+DL+WALL LOAD+LL
1 1.2 3 1.2 4 1.2 5 1.2

LOAD COMB 7 WL(X)+DL+WALL LOAD+LL
2 1.2 3 1.2 4 1.2 5 1.2
LOAD COMB 8 DL+WALL LOAD+LL
3 1.5 4 1.5 5 1.5

PERFORM ANALYSIS
PRINT MEMBER FORCES LIST 1 48 80 127 129 134 139 144 149 183 TO 206
PRINT MEMBER STRESSES LIST 1 48 80 127 129 134 139 144 149 183 TO 206

PRINT **STORY DRIFT**
PRINT ELEMENT FORCE LIST 231 TO 242

START **CONCRETE DESIGN**
CODE **INDIAN**
RATIO 6 MEMB 1 48 80 127 129 134 139 144 149 183 TO 206
MAXMAIN 32 MEMB 1 TO 206
MAXSEC 12 MEMB 1 TO 206
MINMAIN 10 MEMB 1 TO 206
MINSEC 6 MEMB 1 TO 206

DESIGN BEAM 27 74 106 128 130 TO 133 135 TO 138 140 TO 143 145 TO 148 150 -
151 TO 155 177 TO 182
DESIGN COLUMN 1 48 80 127 129 134 139 144 149 183 TO 206
DESIGN ELEMENT 231 TO 242
CONCRETE TAKE
END CONCRETE DESIGN
FINISH

4.3 INPUT FILE FROM STAAD-PRO 2001 OF A 40 - STOREY TUBE-IN-TUBE BUILDING

STAAD SPACE
START JOB INFORMATION
JOB NAME DISSERTATION
JOB NO 1
ENGINEER DATE 07-Nov-01
END JOB INFORMATION
INPUT WIDTH 79
UNIT METER KN
JOINT COORDINATES
1 0 0 0; 2 0 3.96001 0; 3 12.2 3.96001 0; 4 12.2 0 0; 5 3.05 3.96001 0;.......etc
MEMBER INCIDENCES
1 1 2; 2 2 5; 3 3 4; 4 5 6; 5 6 7; 6 7 3; 7 3 10; 8 8 9; 9 10 11; 10 11 12;.......etc
ELEMENT INCIDENCES SHELL

```
231 2 3 30 29; 232 3 8 35 30; 233 8 13 40 35; 234 13 18 45 40; 235 18 23 50 45;....etc
MEMBER PROPERTY INDIAN
6914 6916 TO 6919 6921 TO 6924 6926 TO 6929 6931 TO 6934 6936 TO 6959 6961 -
........
7884 TO 7886 7888 TO 7891 7893 PRIS YD 0.84 ZD 0.3
4614 4616 TO 4619 4621 TO 4624 4626 TO 4629 4631 TO 4634 4636 TO 4659 4661 -
......
5584 TO 5586 5588 TO 5591 5593 PRIS YD 0.84 ZD 0.41
2 4 TO 7 9 TO 12 14 TO 17 19 TO 22 24 TO 47 49 51 TO 54 56 TO 59 61 TO 64 -......
994 PRIS YD 0.84 ZD 0.51.
1 3 8 13 18 23 48 50 55 60 65 70 80 82 87 92 97 102 127 129 134 139 144 -........
2439  PRIS YD 0.91 ZD 0.81
ELEMENT PROPERTY
231 TO 242 9213 TO 9680 THICKNESS 0.15
UNIT INCHES KIP
CONSTANTS
E 3150 MEMB 1 TO 9680
ALPHA 6.5e-006 MEMB 1 TO 9680
DENSITY CONCRETE MEMB 1 TO 230 243 TO 9212
POISSON CONCRETE MEMB 1 TO 230 243 TO 9212
DENSITY CONCRETE MEMB 231 TO 242 9213 TO 9680
POISSON CONCRETE MEMB 231 TO 242 9213 TO 9680
UNIT METER KN
SUPPORTS
1 4 9 14 19 24 28 31 36 41 46 51 55 58 63 68 73 78 82 85 90 95 100 105 121 -
122 TO 123 127 TO 132 136 TO 153 157 TO 159 163 TO 180 FIXED
DEFINE 1893 LOAD
ZONE 0.04 I 1.5 K 1 B 1
SELFWEIGHT
MEMBER WEIGHT
6914 6916 TO 6919 6921 TO 6924 6926 TO 6929 6931 TO 6934 6936 TO 6959 6961 -
.......
7884 TO 7886 7888 TO 7891 7893 UNI -6.3
4614 4616 TO 4619 4621 TO 4624 4626 TO 4629 4631 TO 4634 4636 TO 4659 4661 -
........
5584 TO 5586 5588 TO 5591 5593 UNI -8.61
2 4 TO 7 9 TO 12 14 TO 17 19 TO 22 24 TO 47 49 51 TO 54 56 TO 59 61 TO 64 -.......
994 UNI -10.71
DEFINE WIND LOAD
TYPE 1
INT 1.87 HEIG 158.4
EXP 1 JOINT 2 29 56 83 109 TO 111 154 TO 156 160 TO 162 181 202 223 244 265 -
........
LOAD 1 1893 LOAD IN X-DIRECTION
1893 LOAD X
LOAD 2 WIND LOAD IN X-DIRECTION
```

```
WIND LOAD X 1 TYPE 1
LOAD 3 SLFWEIGHT
Y -1
LOAD 4 WALL LOAD
MEMBER LOAD
1 TO 28 32 36 40 44 48 TO 107 111 115 119 123 127 TO 164 171 TO 230 243 -.........
3589 TO 3626 3633 TO 3720 3724 3728 UNI GY -18.216
LOAD 5 LL
ELEMENT LOAD
231 TO 242 9213 TO 9680 PR GY -3.358
LOAD COMB 6 1893(X)+DL+WALL LOAD+LL
1 0.75 3 0.75 4 0.75 5 0.75
LOAD COMB 7 WL(X)+DL+WALL LOAD+LL
2 0.75 3 0.75 4 0.75 5 0.75
LOAD COMB 8 DL+WALL LOAD+LL
3 0.75 4 0.75 5 0.75
PERFORM ANALYSIS
PRINT MEMBER FORCES LIST 1 933 2083 3233 4383 5533 6683 7833 8983
PRINT MEMBER FORCES LIST 223 1155 2305 3455 4605 5755 6905 8055 9205
FINISH
```

4.4 OUT-PUT FILE OF STAAD-pro 2001

An output file is shown as follows. It shows the design of only some critical members starting from concrete design. Indian code provisions have been followed in the design as indicated in the output file. (For example, maximum 6% longitudinal steel has been allowed in the design of columns as shown in the output file.)

```
****************************************************
*                                                  *
*           STAAD.Pro                              *
*           Version  2001    Build 1004            *
*           Proprietary Program of                 *
*           RESEARCH ENGINEERS, Intl.              *
*           Date=    NOV 12, 2001                  *
*           Time=    18:39:48                      *
*                                                  *
*      USER ID: CIVIL ENGG. DEPTT. U.O.R           *
****************************************************

4122. START CONCRETE DESIGN
4123. CODE INDIAN
4124. RATIO 6 MEMB 1 3 8 13 18 23 48 70 80 102 127 129 134 139 144
4125. MAXMAIN 32 MEMB 1 TO 230
4126. MAXSEC 12 MEMB 1 TO 230
```

4127. MINMAIN 10 MEMB 1 TO 230
4128. MINSEC 6 MEMB 1 TO 230

4129. DESIGN BEAM 2 4 TO 7 9 TO 12 14 TO 17 19 TO 22 24 TO 27 44 74

==

B E A M N O. 2 D E S I G N R E S U L T S

M30 Fe415 (Main) Fe415 (Sec.)

LENGTH: 3050.0 mm SIZE: 510.0 mm X 840.0 mm COVER: 25.0 mm

SUMMARY OF REINF. AREA (Sq.mm)

SECTION	0.0 mm	762.5 mm	1525.0 mm	2287.5 mm	3050.0 mm
TOP REINF.	845.06 (Sq. mm)	845.06 (Sq. mm)	845.06 (Sq. mm)	845.06 (Sq. mm)	1804.36 (Sq. mm)
BOTTOM REINF.	2077.65 (Sq. mm)	1045.27 (Sq. mm)	845.06 (Sq. mm)	845.06 (Sq. mm)	845.06 (Sq. mm)

SUMMARY OF PROVIDED REINF. AREA

SECTION	0.0 mm	762.5 mm	1525.0 mm	2287.5 mm	3050.0 mm
TOP REINF.	8-12í 1 layer(s)	8-12í 1 layer(s)	8-12í 1 layer(s)	8-12í 1 layer(s)	16-12í 2layer
BOTTOM REINF.	19-12í 2 layer(s)	10-12í 1 layer(s)	8-12í 1 layer(s)	8-12í 1 layer(s)	8-12í 1 layer(s)
SHEAR REINF.	2 legged 10í @ 300 mm c/c	2 legged 10í @ 225 mm c/c	2 legged 10í @ 225 mm c/c	2 legged 10í @ 225 mm c/c	2 legged 10í @ 300 mm c/c

Provide 2-12í along each face of the beam (Side face reinf.)

==

B E A M N O. 74 D E S I G N R E S U L T S

M30 Fe415 (Main) Fe415 (Sec.)

LENGTH: 3050.0 mm SIZE: 300.0 mm X 840.0 mm COVER: 25.0 mm

SUMMARY OF REINF. AREA (Sq.mm)

SECTION	0.0 mm	762.5 mm	1525.0 mm	2287.5 mm	3050.0 mm
TOP REINF.	842.97 (Sq. mm)	842.97 (Sq. mm)	842.97 (Sq. mm)	950.50 (Sq. mm)	1965.92 (Sq. mm)

BOTTOM REINF.	1976.81 (Sq. mm)	966.65 (Sq. mm)	842.97 (Sq. mm)	842.97 (Sq. mm)	842.97 (Sq. mm)

SUMMARY OF PROVIDED REINF. AREA

SECTION	0.0 mm	762.5 mm	1525.0 mm	2287.5 mm	3050.0 mm
TOP REINF.	5-16í 1 layer(s)	5-16í 1 layer(s)	5-16í 1 layer(s)	5-16í 1 layer(s)	10-16í 1 layer(s)
BOTTOM REINF.	10-16í 1 layer(s)	5-16í 1 layer(s)	5-16í 1 layer(s)	5-16í 1 layer(s)	5-16í 1layer(s)
SHEAR REINF.	2 legged 10í @ 275 mm c/c	2 legged 10í @ 200 mm c/c	2 legged 10í @ 200 mm c/c	2 legged 10í @ 200 mm c/c	2 legged 10í @ 275 mm c/c

Provide 2-12í along each face of the beam (Side face reinf.)

===

********************END OF BEAM DESIGN RESULTS********************

4132. DESIGN COLUMN 1 3 8 13 18 23 48 70 80 102 127 129 134 139 144 149

===

C O L U M N   N O.   1   D E S I G N   R E S U L T S

M30 Fe415 (Main) Fe415 (Sec.)

LENGTH: 3960.0 mm CROSS SECTION: 810.0 mm X 910.0 mm COVER: 40.0 mm

** GUIDING LOAD CASE: 3 END JOINT: 1 SHORT COLUMN

REQD. STEEL AREA : 17100.72 Sq.mm.
MAIN REINFORCEMENT : Provide 56 - 20 dia. (2.39%, 17592.92 Sq.mm.)
(Equally distributed)
TIE REINFORCEMENT : Provide 8 mm dia. rectangular ties @ 320 mm c/c

SECTION CAPACITY (KNS-MET)

```
Puz :  15042.59   Muzl :     878.19   Muyl :     771.95

INTERACTION RATIO: 0.94 (as per Cl. 38.6, IS456)

======================================================================

======================================================================

        C O L U M N   N O.    3   D E S I G N   R E S U L T S

     M30                      Fe415 (Main)                Fe415 (Sec.)

 LENGTH: 3960.0 mm  CROSS SECTION: 810.0 mm X  910.0 mm COVER: 40.0 mm

  ** GUIDING LOAD CASE:   2 END JOINT:    4  SHORT COLUMN


 REQD. STEEL AREA     :  8845.20 Sq.mm.
 MAIN REINFORCEMENT : Provide  44 - 16 dia. (1.20%,   8846.72 Sq.mm.)
                       (Equally distributed)
 TIE REINFORCEMENT   : Provide  8 mm dia. rectangular ties @ 255 mm c/c

  SECTION CAPACITY (KNS-MET)
  --------------------------
  Puz :  12584.51   Muzl :    688.95   Muyl :    606.94

  INTERACTION RATIO: 0.97 (as per Cl. 38.6, IS456)

======================================================================

  ********************END OF COLUMN DESIGN RESULTS********************

 4135. END CONCRETE DESIGN
 4136. FINISH

            *************** END OF STAAD.Pro ***************
```

4.5 MANUAL DESIGN OF A BEAM

Manual design (hand calculations) of a beam is given in this section so that the accuracy of computer results (output) can be checked against the manual work.

PROPERTIES OF THE BEAM:

Size	300 X 840 mm
Length	3050.00 mm
Grade of concrete used	M 30
Grade of steel used	Fe 415

σ_{cbc} = c = Compressive stresses developed in the concrete due to bending =10N/mm^2 for M 30 grade of con.

σ_{st} = t = Tensile stresses developed in steel = 230N/mm^2 for Fe 415 grade of steel.

Cover (top as well as bottom) = d' = 25 mm

Maximum moment developed ($M_{max.}$) = 337.401 kNm.

DESIGN CALCULATIONS:

Modular ratio	$m = 280 / (3\ \sigma_{cbc}) = 9.33$
Neutral axis constant	$N = mc / (mc + t) = 0.2886$
Lever arm constant	$j = 1 - (N / 3) = 0.9038$
Moment of Resistance constant	$R = (1 / 2)\ N\ c\ j = 1.304\ N/mm^2$
Now, width of beam	b = 300 mm
Effective depth of beam	d = overall depth (D) – bottom cover
	= 840 – 25 = 815 mm
Depth of critical neutral axis	n_c = Nd = 235.21 mm from top fiber

Now, Moment of resistance for balanced section, say

$M_1 = M_{bal} = Rbd^2 = 259.845$ kNm

Since, $M_{max} > M_{bal}$, hence beam section will be doubly reinforced

$\therefore$ Unbalanced moment (M_2) = $M_{max} - M_1$ = 77.556 kNm

Now, Area of tensile steel required for the balanced section corresponding to M_1

i.e., $A_{st1} = M_1 / (t\ j\ d) = 1533.75\ mm^2$

And, Area of the additional tensile steel required to develop the moment M_2

i.e., $A_{st2} = M_2\ / t\ (d - d') = 426.84\ mm^2$

$\therefore$ Total area of tensile steel required Ast = $A_{st1} + A_{st2} = 1960.59\ mm^2$

And area of steel in compression is given by

$A_{sc} = m\ A_{st2}\ (d - n_c) / (1.5\ m - 1)\ (n_c - d') = 845.26\ mm^2$

CHAPTER 5
RESULTS AND DISCUSSION

In this chapter the results obtained have presented in tabular form. Graphs of some necessary results have also been plotted for better explanation & to discuss about the results. The results shown are only for the buildings having aspect ratio 3:5.

5.1 Notations used in the tables:

AF	-	Axial force
BM	-	Bending Moment
ST. NO.	-	Storey Number
DL	-	Dead Load
LL	-	Live Load
WL	-	Wind Load
GL	-	Gravity Load
SL	-	Seismic Load

* Dead Load takes masonry load into account.

5.2 Loads and Load Combinations:

1. SL in x-direction (normal to flange frame)
2. WL in x-direction (normal to flange frame)
3. DL
4. LL
5. 1.2 (SL+DL+LL)
6. 1.2 (WL+DL+LL)
7. 1.5 (DL+LL)

5.3 Effect on Axial Force:

Table 5.1 gives the AF variation in corner columns with storey nos. due to Seismic and Gravity load (load case 5) in both types of buildings of 30-storey. The corresponding plot is shown in fig. 5.1. It is clear from the observation that in framed tube structure, the increase in AF over tube-in-tube structure varies from 8% to 10% in vertical direction.

5.4 Effect on BM:

Table 5.2 gives variation of BM in corner column with increasing story level in both types of building of 30-story due to wind and gravity loads. The corresponding graph is shown in fig. 5.2. It is clear from the graph that in lower 15 story (i.e. upto half height of the building), the Bending Moment in columns is higher in Framed tube structure than in tube-in-tube structure. From ground floor to 15th floor the increase in the BM in corner columns in framed tube buildings decreases from 46% to 2.4% as compared to tube-in-tube building. But from the level of 15th story to top story, BM in corner columns in the tube-in-tube buildings is higher than those in framed tube building.

Table 5.4 contains AF variation in corner columns with storey nos. for seismic & gravity loads (load case 5) in both types of structures of 40-storey. The corresponding graph is shown in fig. 5.4. In this case, columns of framed tube structure have 2.5% to 40% higher axial forces than tube-in-tube structure. But in 35th to 39th storey the variation is more than 50%. As compared to the same result of 30-storey buildings, variation is too high.

Table 5.3 shows the variation of BM in corner columns with story nos. in both types of buildings for seismic & gravity loads. The ratios of Bending Moments in columns are also shown in the table. Its graphical representation is shown in fig. 5.3. It is evident from the table that the ratio of BM in columns of framed tube building to that in tube-in-tube building vary from 3 to 4 upto 12th story and it increases from 4 to 6 as story level increases from 12th to 22nd. At 30th story level this ratio is less than 10 but above this storey level it is greater than 10. Here a point to note is that the variation of BM in columns of both structure have same pattern. This pattern of distribution of BM is same in 30-story and 40-story buildings.

Table 5.5 shows the AF distribution in columns of flange frame of framed tube building due to wind loading only. The values of AF are given in the table with

TABLE - 5.1

AF variation in corner columns with storey numbers

FRAMED	ST. NO.	T-IN-T	% VAR.
7594	1	7058	7.6
7283	2	6753	7.85
6993	3	6467	8.13
6717	4	6198	8.4
6452	5	5940	8.62
6194	6	5691	8.8
5941	7	5448	9
5690	8	5210	9.2
5442	9	4976	9.4
5198	10	4745	9.5
4955	11	4516	9.7
4697	12	4277	9.8
4439	13	4040	9.8
4183	14	3805	9.9
3929	15	3571	10
3675	16	3339	10.1
3422	17	3108	10.1
3170	18	2880	10.1
2920	19	2654	10
2676	20	2432	10
2434	21	2213	9.9
2172	22	1978	9.8
1912	23	1746	9.5
1658	24	1517	9.6
1405	25	1291	8.1
1155	26	1069	8.1
906	27	849	6.7
659	28	631	4.4
411	29	416	-1.2
192	30	210	-9.3

TABLE - 5.2

Variation of BM in corner columns with storey no.

FRAMED	ST. NO.	T-IN-T	% VAR.
314	1	231	35.9
184.8	2	126	46.6
155	3	110	40.9
139.7	4	102	36.9
127.4	5	96.7	31.7
116.5	6	91.1	27.8
106.7	7	86.1	24
98.1	8	81.6	21
90.8	9	77.8	16.7
88.7	10	75	18.3
91.3	11	82	11.3
83.2	12	78	6.7
78.1	13	72.6	7.6
72	14	70.3	2.4
67.8	15	68.5	-1
64.2	16	67	-4.2
61.1	17	65.9	-7.9
62.5	18	68.1	-10.2
50.14	19	58	-15.6
55	20	59	-7.3
65.7	21	72.8	-10.8
52.5	22	65.5	-24.8
44.8	23	56.5	-26
33.2	24	51	-54.5
17	25	39.9	-135
-6.21	26	22	-455
-39.7	27	-2.9	
-71.2	28	-26	
-94.1	29	-37.4	
-38.7	30	-35.3	

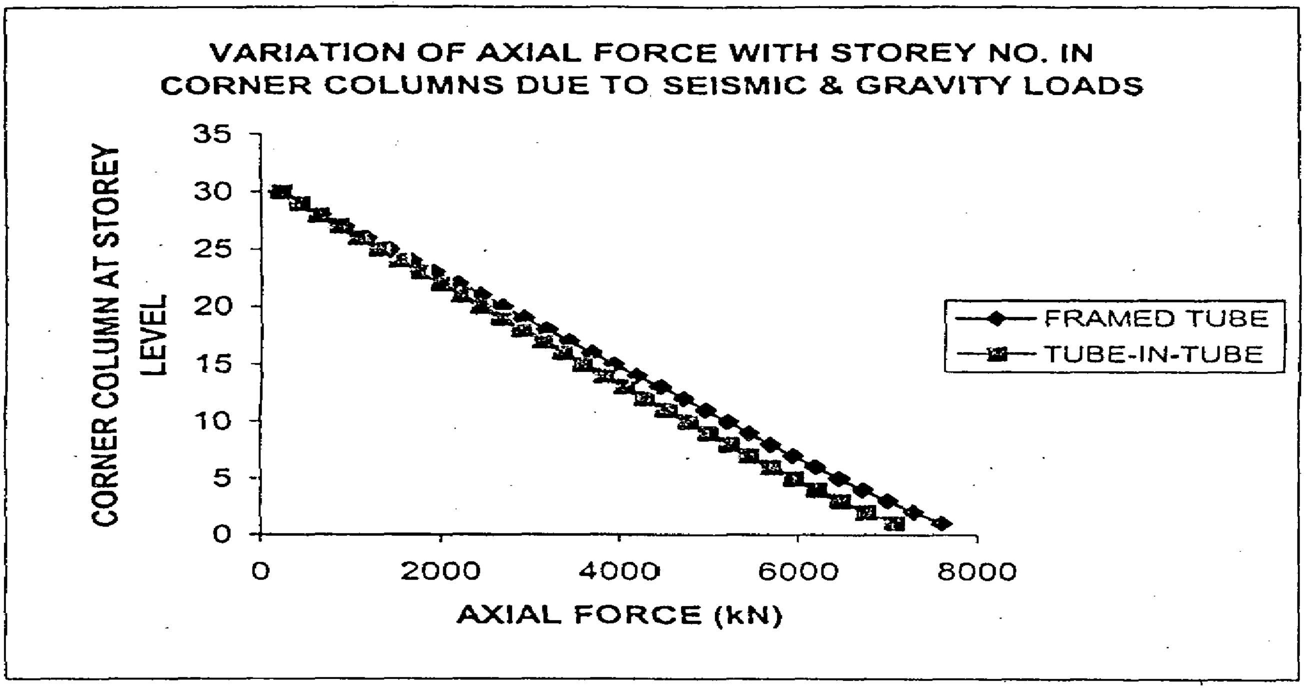

VARIATION OF AXIAL FORCE WITH STOREY NO. IN CORNER COLUMNS DUE TO SEISMIC & GRAVITY LOADS
CORNER COLUMN AT STOREY LEVEL
AXIAL FORCE (kN)
FRAMED TUBE
TUBE-IN-TUBE
0
2000
4000
6000
8000
35
30
25
20
15
10
5
0

FIG.- 5.1

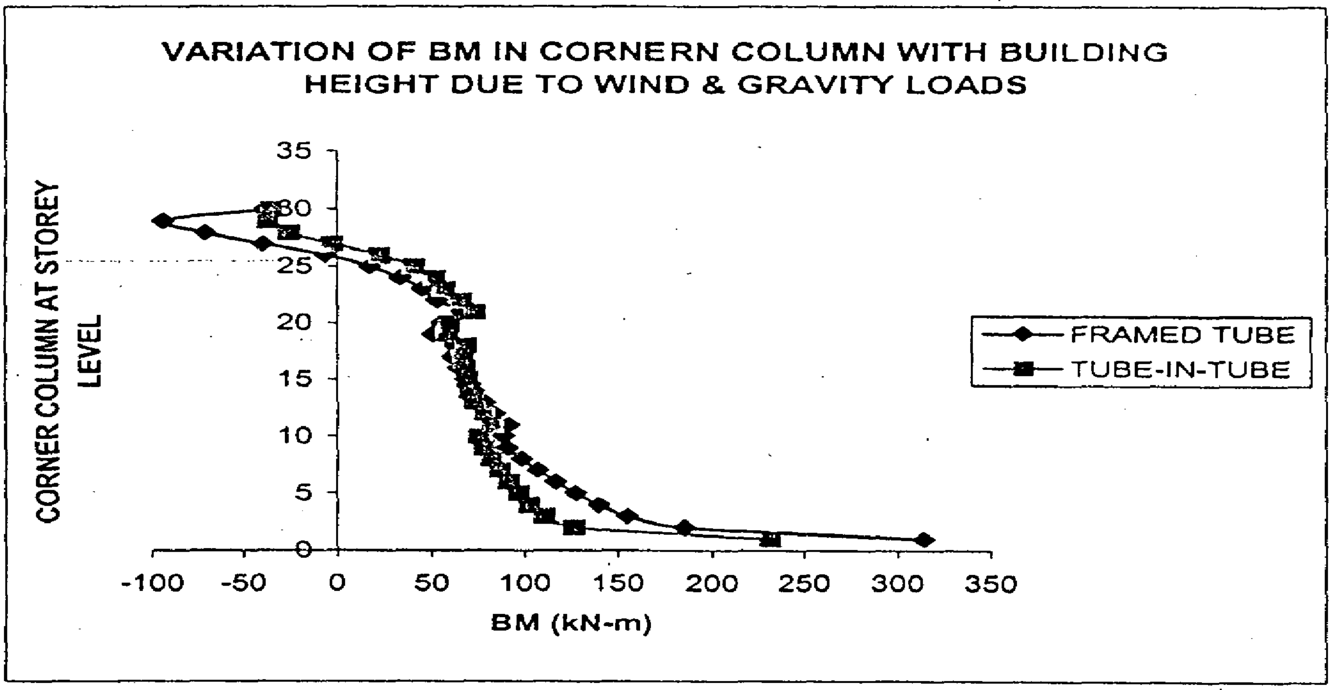

VARIATION OF BM IN CORNERN COLUMN WITH BUILDING HEIGHT DUE TO WIND & GRAVITY LOADS
CORNER COLUMN AT STOREY LEVEL
BM (kN-m)
FRAMED TUBE
TUBE-IN-TUBE
-100
-50
0
50
100
150
200
250
300
350
35
30
25
20
15
10
5
0

FIG. - 5.2

TABLE - 5.3
Variation of BM in corner columns with storey no

T-IN-T	ST. NO.	FRAMED	F/T RAT.
153.7	1	478	3.1
86.9	2	315	3.6
84	3	298	3.5
83.7	4	287	3.4
82.6	5	280	3.4
80.6	6	273	3.4
78	7	268	3.4
75	8	263	3.5
71.3	9	259	3.6
68.3	10	256	3.8
64.9	11	252	3.9
61.5	12	249	4
58.2	13	246	4.2
54.8	14	242	4.4
51.6	15	238	4.6
48.4	16	234	4.8
45.4	17	230	5.1
42.6	18	225	5.2
40.1	19	221	5.5
39.3	20	219	5.6
41	21	230	5.6
35.4	22	211	6
32.7	23	202	6.2
29.8	24	194	6.5
27.1	25	187	6.9
24.3	26	178	7.4
21.3	27	171	8.1
18.4	28	162	8.8
15.8	29	152	9.6
15.3	30	147	9.6
14.5	31	149	10.3
9.7	32	119	12.3
6.26	33	106	16.8
2.03	34	91	44.8
-1.98	35	78	
-6.2	36	63	
-10.8	37	41	
-16.2	38	30	
-20.4	39	-1.56	
-9.9	40	10.6	

TABLE - 5.4
AF variation in corner columns with storey numbers

T-I-T	COL. NO.	FRAMED	% VAR.
9862	1	9881	0.2
9546	2	9788	2.5
9250	3	9720	5.1
8970	4	9634	7.4
8702	5	9525	9.5
8443	6	9392	11.2
8191	7	9240	12.8
7941	8	9071	14.2
7695	9	8889	15.5
7448	10	8694	16.7
7202	11	8488	17.8
6956	12	8272	18.9
6709	13	8048	19.9
6462	14	7815	20.9
6214	15	7573	21.8
5966	16	7325	22.8
5717	17	7070	23.6
5468	18	6807	24.5
5221	19	6540	25.3
4975	20	6271	26
4731	21	6008	26.9
4477	22	5738	28.2
4223	23	5474	29.6
3973	24	5210	31.1
3724	25	4940	32.6
3478	26	4665	34.1
3234	27	4385	35.6
2992	28	4099	36.9
2754	29	3808	38.3
2520	30	3518	39.6
2291	31	3231	41
2046	32	2930	43.2
1804	33	2633	45.9
1567	34	2333	48.8
1333	35	2023	51.7
1102	36	1702	54.4
874	37	1368	56.5
649	38	1018	56.8
426	39	647	51.8
212	40	297	40.1

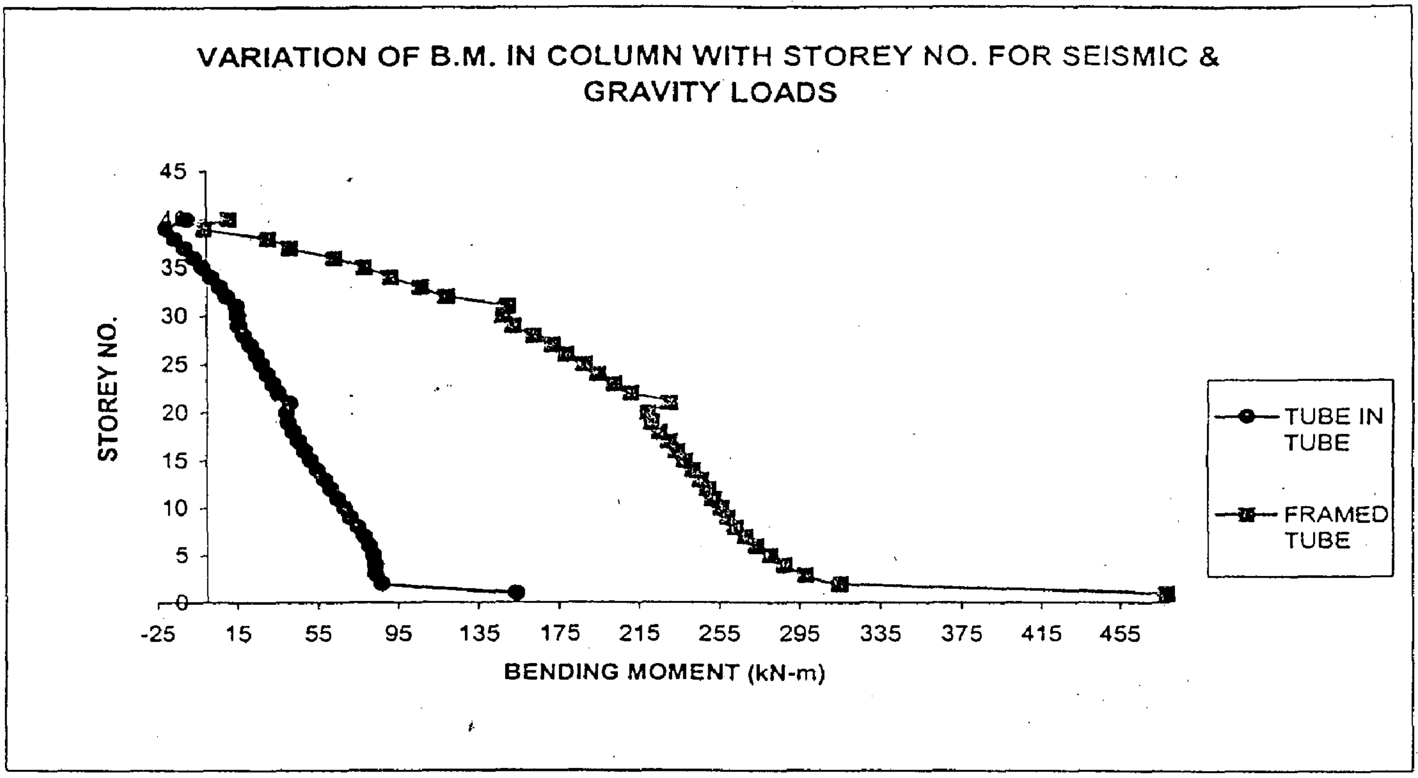

VARIATION OF B.M. IN COLUMN WITH STOREY NO. FOR SEISMIC & GRAVITY LOADS
STOREY NO.
45
40
35
30
25
20
15
10
5
0
-25
15
55
95
135
175
215
255
295
335
375
415
455
BENDING MOMENT (kN-m)
TUBE IN TUBE
FRAMED TUBE

FIG. - 5.3

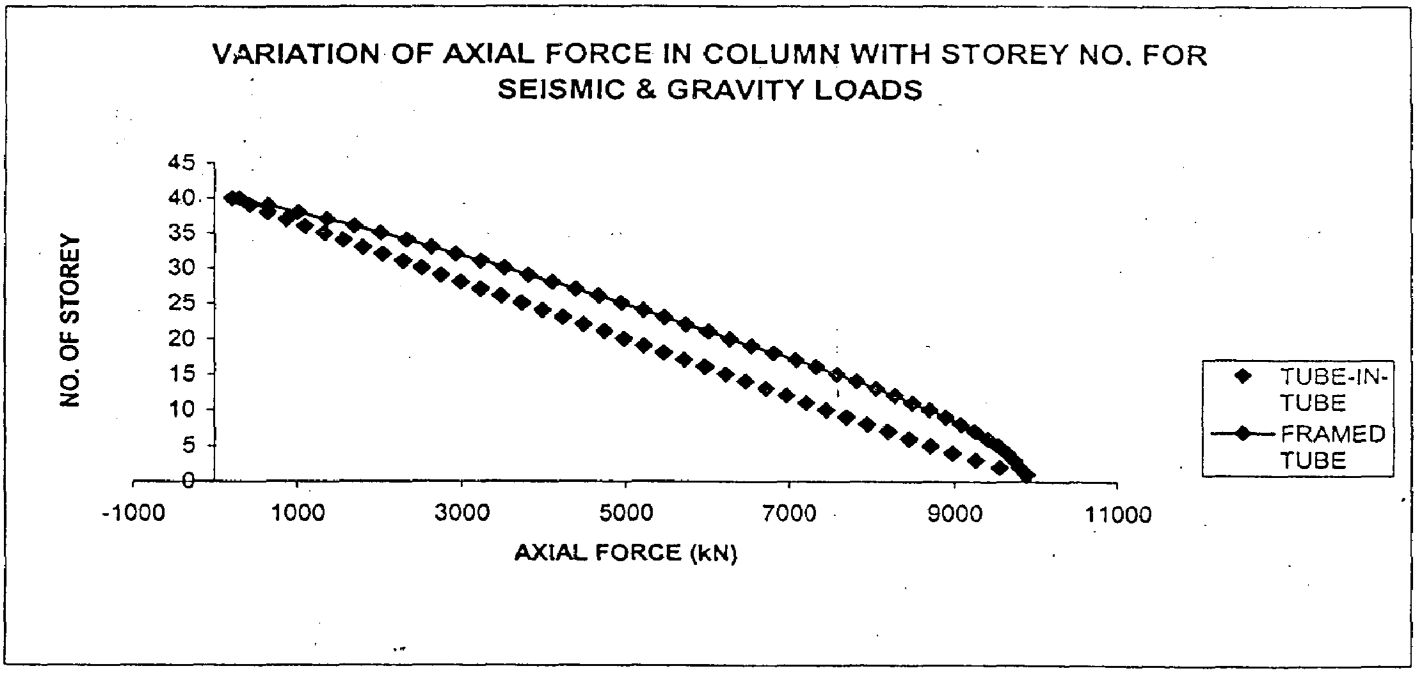

VARIATION OF AXIAL FORCE IN COLUMN WITH STOREY NO. FOR SEISMIC & GRAVITY LOADS
NO. OF STOREY
45
40
35
30
25
20
15
10
5
0
-1000
1000
3000
5000
7000
9000
11000
AXIAL FORCE (kN)
TUBE-IN-TUBE
FRAMED TUBE

FIG. - 5.4

increasing storey level for 30-storey & 40-storey buildings. It is evident from the table that as the storey level increases, axial force in columns decreases. Another important point to be noted is that axial forces in corner columns have higher values than those in central columns. The data of this table have been used to determine shear lag factor, which are shown in tables 5.9 & 5.10.

5.5 Effect of Storey Height on Shear Lag Factor:

Table 5.6 gives AS distribution in columns of first story level of flange frame due to wind load in a 40-storey framed tube building. These results are plotted in Fig. 5.6. Table 5.7 gives the AS distribution for 20^{th} storey level under similar conditions. AS distribution at 20^{th} storey level is plotted in fig. 5.7. By observing these two graphs, it is found that the curve for distribution of AF against the columns for any storey level tends to flatten as the storey level increases. It means that the ratio in AS in corner columns to AS in central columns tends to 1 as the storey level increases. In other words effect of shear lag decreases with increasing storey levels.

Table 5.8 gives AS distribution in columns of flange frame of 40^{th} storey level of framed tube building due to wind load. The result is plotted in fig. 5.8. It is clear from the graph that axial forces increase towards central column. Hence, AS in corner column is less than AS in central column, thus shear lag factor becomes greater than 1 for this storey. This phenomenon is known as negative shear lag effect.

Table 5.9 contains shear lag factor with storey no. in columns of flange frame of a 30-storey framed tube building due to seismic load only. Shear lag factor, is nothing but the ratio of axial stresses in central column to that in corner column. The graphical representation of shear lag factor variation is shown in fig. 5.9. Table 5.10 gives variation in shear lag factor with storey height in flange frame of 40-storey framed tube building for wind loading only. Graphical representations of these results are shown in fig. 5.10. These graphs show that as the SLF attains a value greater than 1 from any storey level, the negative shear lag effect starts from that storey level and goes on increasing towards upper stories. Shear lag effect has been obtained predominantly in the lower storey levels. The minimum value of shear lag factor at lowest storey level indicates the maximum

TABLE 5.5 AXIAL FORCE (kN) DISTRIBUTION IN FLANGE FRAME OF FRAMED TUBE BUILDING DUE TO WIND LOADING

ST. LEV.	30 STOREY BLDG.		40 STOREY BLGD.	
	CEN. COL	COR.COL	CEN. COL	COR.COL
1	491.6	2504	3507.6	6001.5
5	426	1622	2980.4	4370.1
10	318	983.9	2289	3001.2
15	214	481.9	1917.4	2370.1
20	119	132	1666.41	2015
25	50.5	40.07	1518.7	1610.49
30	4.9	3.42	1322.4	1248.72
35			926.05	773
40			748.3	571.3

TABLE 5.6

COL. NO.	AF
1	6001.5
2	4856.4
3	3918
4	3702.1
5	3690.2
6	3599.8
7	3507.6
8	3596
9	3688
10	3708.1
11	3920.2
12	4851.2
13	6000.6

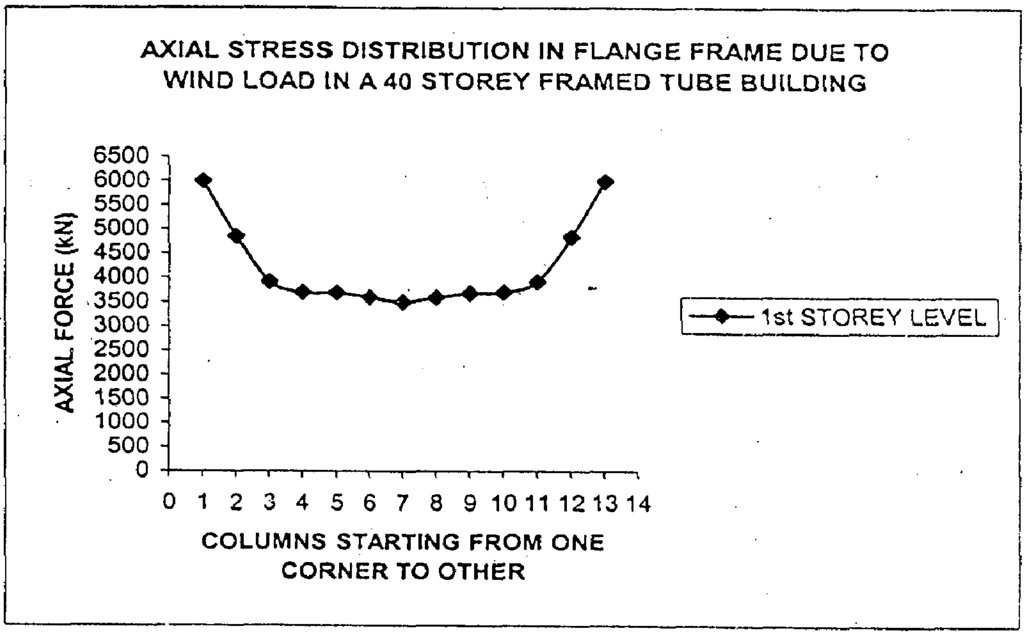


FIG. - 5.6

TABLE - 5.7

COL. NO.	AF(kN)
1	2015
2	2001.1
3	1905.9
4	1840.3
5	1761.5
6	1686.9
7	1666.41
8	1685.5
9	1762.6
10	1840
11	1905.1
12	1999.2
13	2015

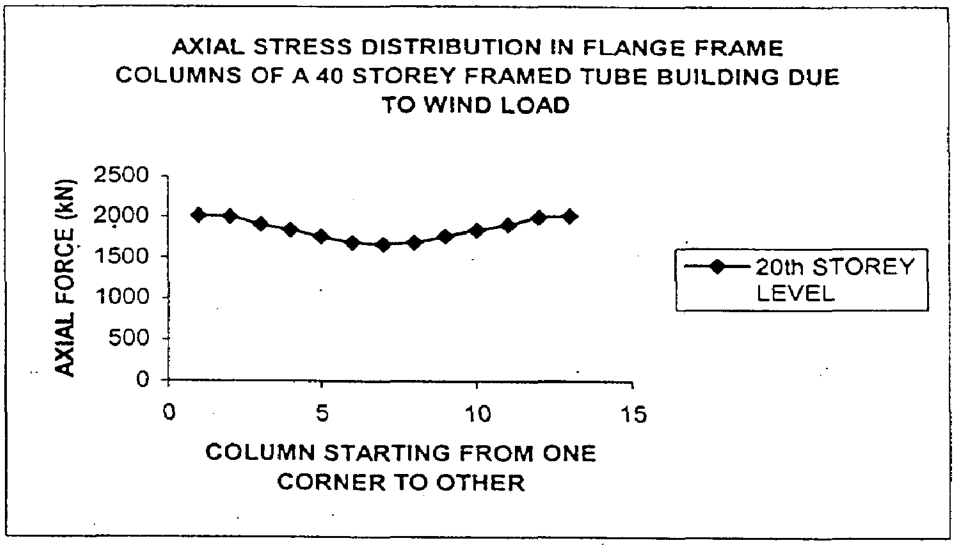


FIG.- 5.7

TABLE - 5.8

COL. NO.	AF (kN)
1	571.3
2	663.1
3	674.8
4	692.3
5	700.9
6	722.3
7	748.3
8	725
9	701.2
10	693
11	676
12	662.6
13	570.1

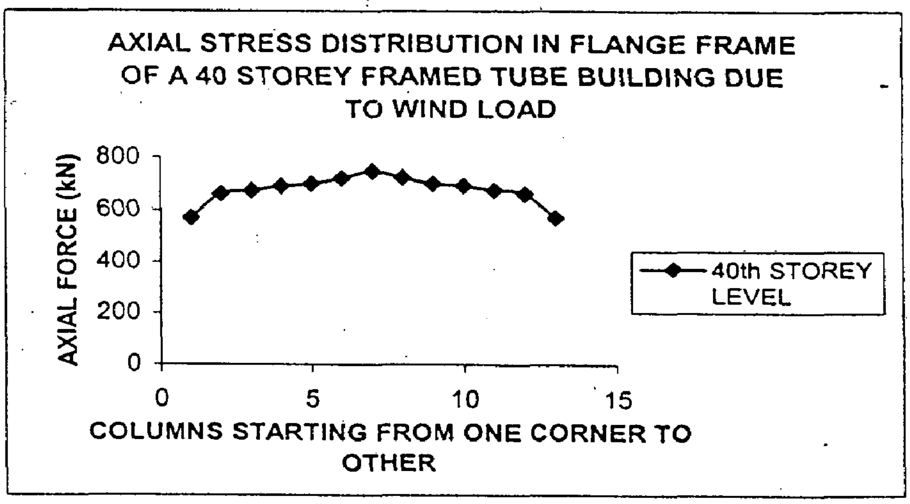


FIG. - 5.8

TABLE - 5.9

S.L.F.	ST. NO.
0.196	1
0.263	5
0.323	10
0.51	15
0.9	20
1.26	25
1.43	30

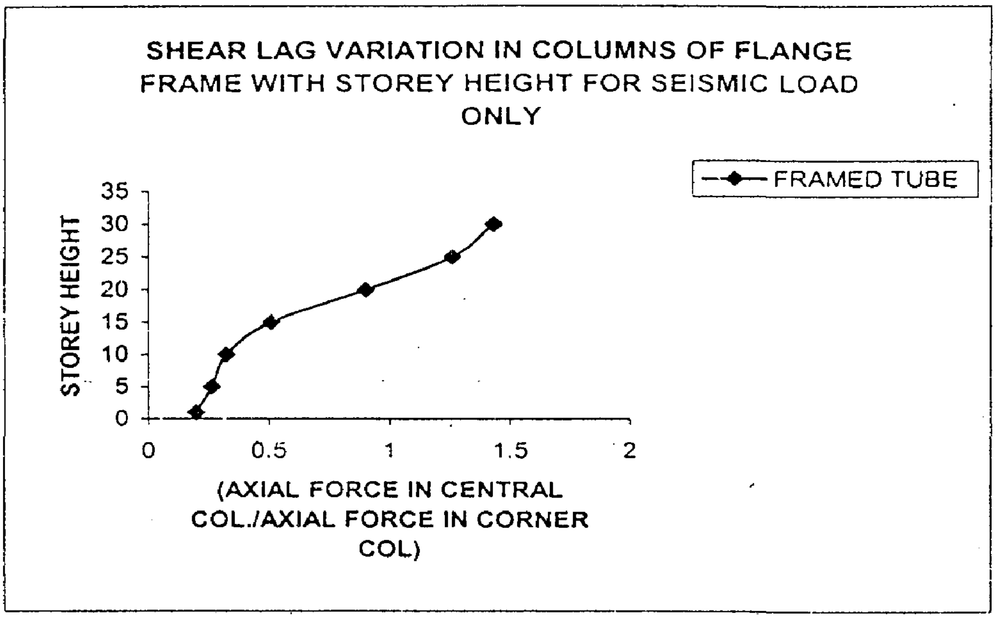


FIG. - 5.9

TABLE - 5.10

S.L.F.	ST. NO.
0.585	1
0.682	5
0.763	10
0.809	15
0.827	20
0.943	25
1.059	30
1.198	35
1.31	40

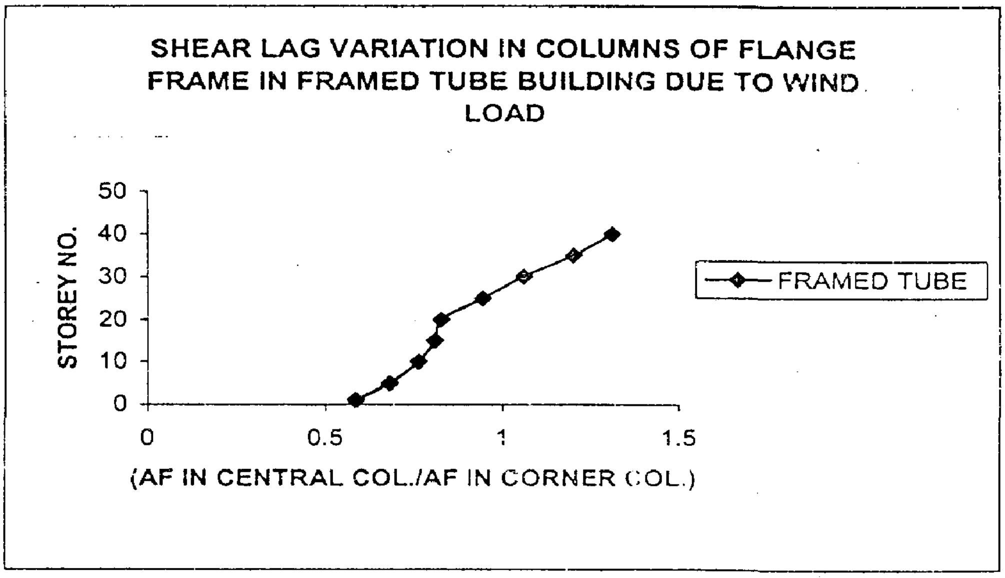


FIG. - 5.10

shear lag effect at that storey level. The maximum value of shear lag factor at the highest storey level is the significant value of maximum negative shear lag effect at this storey.

5.6 Effect of Storey Height on Storey Drift:

Table 5.11 gives storey drift of 30-storey buildings for wind loading only for both types of buildings. It is clear from the fig. 5.11 that storey drift in framed tube building at each storey is more than that in tube-in-tube building. In framed tube structure, the storey drift at each fifth storey is 44% to 47% more than that in tube-in-tube structure of same aspect ratio.

Table 5.12 gives storey drift of 40-storey building for wind loading in both types of buildings. Fig. 5.12 shows graphical representation of these results. In this case, similar results have been obtained in relation to % variation in storey drift i.e. 45%-47% higher in framed tube structure as compared to that in tube-in-tube building.

Table 5.13 shows storey drift in 30-storey & 40-storey framed tube buildings for wind load only. The corresponding graph is shown in fig. 5.13. It is evident from the graph that for the same type of framed tube building with different height, the storey drifts at each storey level in higher building are more than those in lower building. The % increase in storey drift, in 40-storey framed tube building over 30-storey building decreases from 92.7% to 32.4% as the storey height increases.

Table 5.14 shows storey drift in 30-storey & 40-storey tube-in-tube buildings for seismic load only, corresponding graph is shown in fig.5.14. In this case the variation in storey drift obtained is random, initially it decreases then it increases with storey height.

5.7 Effect on Steel Required in Columns:

Table 5.15 shows area of steel required for the design of columns at lowest storey in a web frame of 30-storeyed building. This table gives comparison of A_{st} required for two types of building. It is clear from the table that in most of the columns of framed tube buildings; A_{st} required is more than that in tube-in-tube building. Percentage increase in steel required varies from 2% to 36% in framed tube buildings. But in some columns, A_{st} required is same for both structures.

5.11 Storey drift of 30-storey buildings for wind load only

FRAMED	ST. NO.
1.38	5
3.02	10
4.9	15
6.84	20
8.97	25
10.36	30

T-IN-T	ST. NO.
0.94	5
2.05	10
3.32	15
4.64	20
6.14	25
7.17	30

% VAR.
46.8
47.3
47.6
47.4
46.1
44.5

5.12 Storey drift of 40-storey buildings for wind load only

FRAMED	ST. NO.
2.66	5
5.8	10
9.02	15
11.74	20
14.38	25
16.72	30
18.94	35
19.54	40

T-IN-T	ST. NO.
1.8	5
3.93	10
6.14	15
8.01	20
9.83	25
11.47	30
13	35
13.51	40

% VAR.
47.7
47.6
46.9
46.6
46.3
45.8
45.7
44.6

5.13 Storey drift in framed tube buildings for wind load only

S.D.(cm)	ST. NO.
2.66	5
5.8	10
9.02	15
11.74	20
14.38	25
16.72	30
18.94	35
19.54	40

S.D.(cm)	ST. NO.
1.38	5
3.02	10
4.9	15
6.84	20
8.97	25
10.36	30

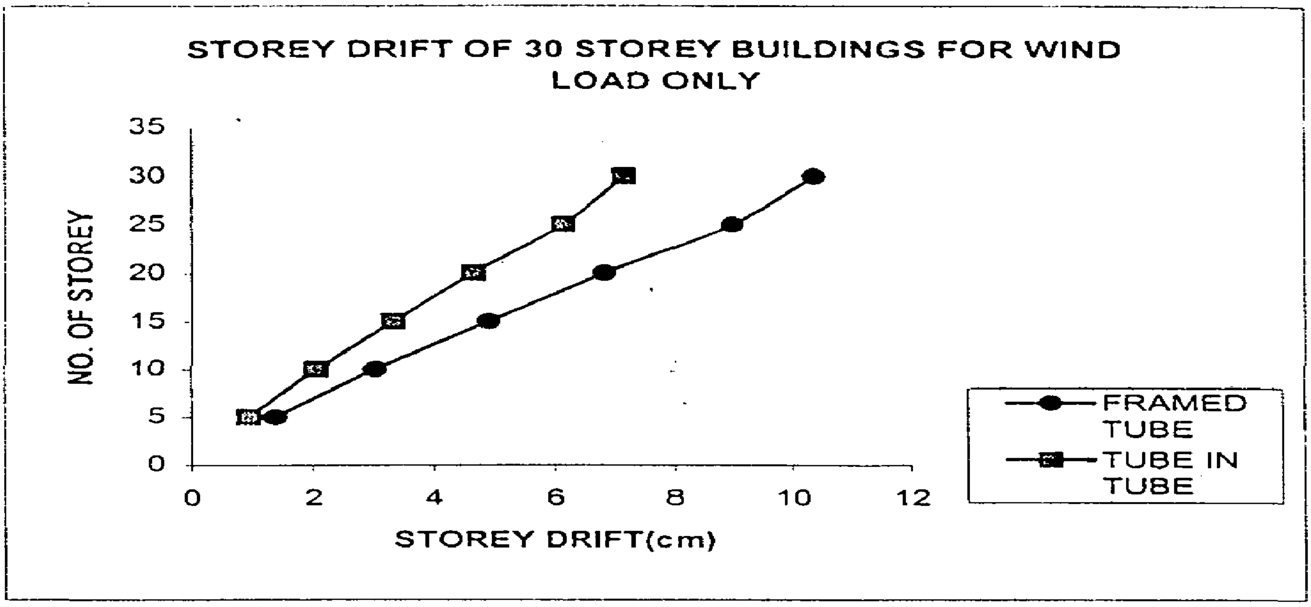
% VAR.
92.7
92.1
84.1
71.6
60.3
32.4

5.14 Storey drift of Tube-in-tube buildings for seismic load only

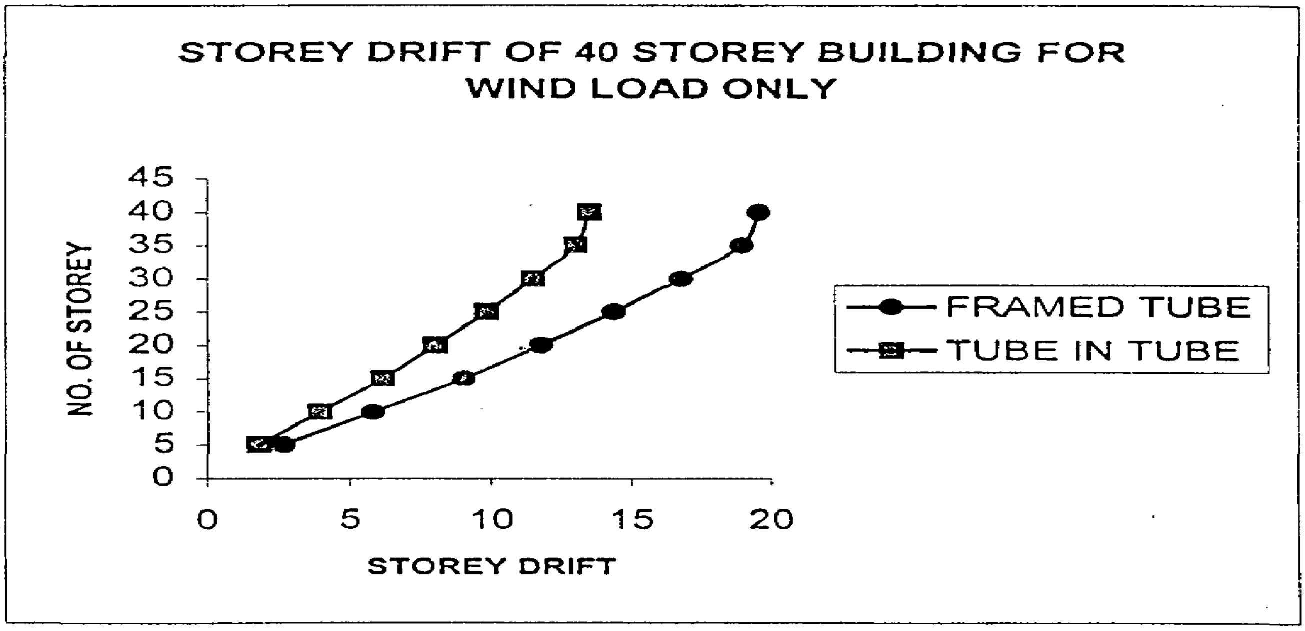
S.D.(cm)	ST. NO.
1.53	5
3.38	10
5.43	15
7.33	20
9.03	25
10.04	30

S.D.(cm)	ST.NO.
1.34	5
4.07	10
6.31	15
8.49	20
10.7	25
12.62	30
14.27	35
15.23	40

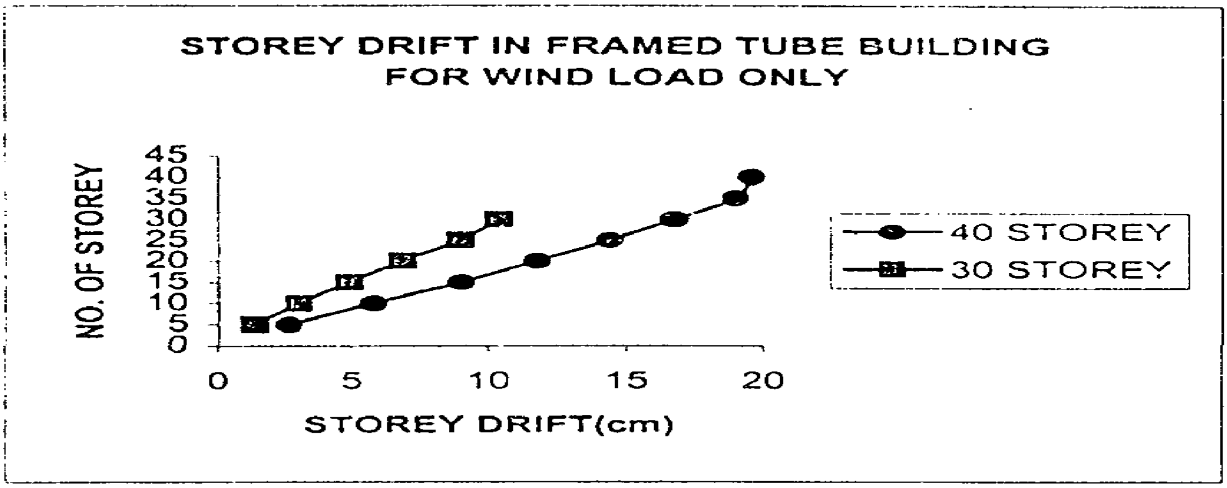
% VAR.
-14.2
20.41
16.2
15.8
18.5
25.7


STOREY DRIFT OF 30 STOREY BUILDINGS FOR WIND LOAD ONLY
NO. OF STOREY
35
30
25
20
15
10
5
0
0
2
4
6
8
10
12
STOREY DRIFT(cm)
FRAMED TUBE
TUBE IN TUBE

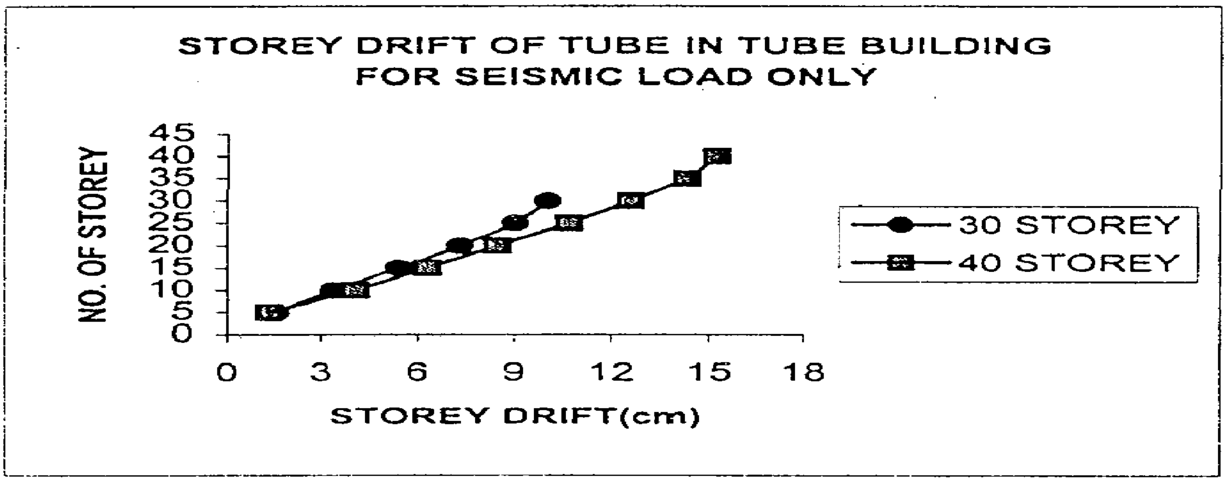
5.11


STOREY DRIFT OF 40 STOREY BUILDING FOR WIND LOAD ONLY
NO. OF STOREY
45
40
35
30
25
20
15
10
5
0
0
5
10
15
20
STOREY DRIFT
FRAMED TUBE
TUBE IN TUBE

5.12


STOREY DRIFT IN FRAMED TUBE BUILDING
FOR WIND LOAD ONLY
NO. OF STOREY
STOREY DRIFT(cm)
40 STOREY
30 STOREY

5.13


STOREY DRIFT OF TUBE IN TUBE BUILDING
FOR SEISMIC LOAD ONLY
NO. OF STOREY
STOREY DRIFT(cm)
30 STOREY
40 STOREY

5.14

Table 5.16 shows area of steel required for the design of columns at lowest storey in a flange frame of 30-storeyed building. This table gives comparison of Ast required for two types of building (-ve sign indicates that steel required is more in case of tube-in-tube building and it shows % increase of A_{st} in columns of tube-in-tube building over those in framed tube building.) In most of the columns, A_{st} required is same in both the structure except in corner columns.

Table 5.17 shows area of steel required for the design of columns at lowest storey in a flange frame of 40-storeyed building. In this table, comparison of A_{st} required for two types of building has been presented. A variation in A_{st} required has been seen in both types of buildings. Variation in % of A_{st} required in both structures is more in corner columns & less in central columns. In all columns, A_{st} required is more in framed tube building except one column.

Table 5.18 shows area of steel required for the design of columns at lowest storey in a web frame of 40-storeyed buildings. Percentage increase in A_{st} required in columns in framed tube-building decreases from one corner to center; then it becomes zero and then % increase in A_{st} required in columns in tube-in-tube building starts increasing, it increases towards another corner column.

5.8 Effect on Steel Required in Beams:

Table 5.19 to 5.22 shows the A_{st} required for the design of beams at the lowest floor level in flange frame and web frame of both types of buildings in 30-storey and 40-storey buildings. The required steel lies in the range of 840mm^2 to 846 mm^2 in all beams at mid span. This amount of steel is required at top face as well as at bottom face. The amount of steel required is same in all beams. It is because of the fact that an equal amount of moment generates in each beam.

A more realistic picture can be available if the parametric study is conducted for some more heights of buildings.

AREA OF STEEL REQUIRED (sq. mm) IN THE DESIGN OF COLUMNS AT LOWEST STOREY IN 30-STOREY BUILDINGS

TABLE - 5.15 IN WEB FRAME COLUMNS

COL. NO.	FRAMED	T-IN-T	% VAR.
1	8845	6486	36.4
2	6486	5896	10
3	5896	5896	0
4	5896	5896	0
5	5896	5896	0
6	5896	5896	0
7	5896	5896	0
8	6022	5896	2.1
9	7076	5896	20
10	5896	5896	0
11	5896	4797	22.9
12	5535	4992	10.9
13	7076	5710	23.9
14	6021	4989	20.7
15	5286	4784	10.5
16	5438	4960	9.6
17	6486	5636	15.1
18	5236	4835	8.3
19	4853	4525	7.2
20	4790	4537	5.6
21	5307	4989	6.4

TABLE - 5.16 IN FLANGE FRAME COL.

COL. NO.	FRAMED	T-IN-T	% VAR.
1	8845	9434	-6.7
2	5896	6486	-10
3	5897	5897	0
4	5896	5896	0
5	5896	5896	0
6	5896	5896	0
7	5896	5896	0
8	5896	5896	0
9	5896	5896	0
10	5896	5896	0
11	5896	5896	0
12	5896	5896	0
13	8845	6486	36.4

AREA OF STEEL REQUIRED (sq. mm) IN THE DESIGN OF COLUMNS AT LOWEST STOREY
IN 40-STOREY BUILDINGS

TABLE - 5.17 IN FLANGE FRAME COLM.

COL. NO.	FRAMED	T-IN-T	% VAR.
1	17100	12383	38.1
2	11793	7665	53.8
3	9434	6486	45.5
4	8255	6486	27.3
5	10024	10614	-5.9
6	8845	7665	15.4
7	8255	7665	7.7
8	8845	7665	15.4
9	10614	10614	0
10	8845	7076	25
11	8845	7076	25
12	11203	8845	26.7
13	17690	12972	36.4

TABLE - 5.18 IN WEB FRAME COLUMNS

COL. NO.	FRAMED	T-IN-T	% VAR.
1	17690	12972	36.4
2	12973	9435	37.5
3	11203	8255	35.7
4	10024	7665	30.8
5	11965	11203	6.8
6	8845	7665	15.4
7	8845	7076	25
8	8255	8255	0
9	12345	12508	-1.3
10	8255	8845	-7.1
11	7474	7665	-2.6
12	8255	8845	-7.1
13	12383	12884	-4
14	8845	9434	-6.7
15	7456	8255	-10.7
16	8253	8845	-7.1
17	11965	13018	-8.8
18	7573	8845	-16.5
19	6054	7665	-26.6
20	6734	7665	-13.8
21	11203	10614	5.5

CHAPTER 6
CONCLUSION

6.1 GENERAL

This chapter deals with the concluding remarks drawn from the results of all the analysis and design made for 30-storey & 40-storey framed tube & tube-in-tube buildings. The results have been presented in tabular form along with the graphical mode in previous chapter. This chapter contains only the conclusions drawn on the basis of discussion made in previous chapter. The conclusions are valid under the consideration that the aspect ratio of building is 3:5 and analysis is static.

6.2 CONCLUSIONS

The conclusions drawn on the basis of limited study are as follows:

- In a 30-storey framed tube structure, the increase in AF in corner columns, over tube-in-tube structure varies from 8% to 10% in vertical direction when subjected to Seismic and Gravity load. For the same load case in a 40-storey building, this variation lies between 2.5% to 50%. Thus, on increasing the building height, the % increase in AF in corner column in a framed tube building increases as compared to tube-in-tube building for same load case.
- The BM in corner columns in both types of buildings increases with building height. BM is found more than 3 times higher in case of framed tube building over that in tube-in-tube building in the case of 40-storey. But in case of 30-storey building, BM in columns above 15^{th} storey level is found increasing in tube-in-tube building in comparison to framed tube building. The graphs of variation of BM Vs building height for both the structures have similar pattern as shown in fig. 5.2 & fig. 5.3.
- Effect of shear lag decreases with increasing storey levels. As the SLF attains a value greater than 1 from a particular storey level, the negative shear lag effect starts from that storey level and goes on increasing towards upper stories. Shear lag effect has been found to be most predominant in the lowermost storey levels, which are heavily loaded. In upper storeys (including topmost storey) of the flange frame, axial stresses

in central column are greater than that in corner column. This phenomena is known as negative shear lag effect. The minimum value of shear lag factor at lowest storey level indicates the maximum shear lag effect at that storey level. The maximum value of shear lag factor at the highest storey level is indicated by the maximum negative shear lag effect at this storey.

- In framed tube structure, the storey drift at each fifth storey is varying from 44% to 47% more than that in tube-in-tube structure for wind loading only with the same aspect ratio.
- For the framed tube building with different height, the storey drifts at each storey level are more as compared to those in lower height building. The % increase in storey drift, in 40-storey framed tube building over 30-storey building decreases from 92.7% to 32.4% as the storey height increases.
- The variation in increase in storey drift obtained in 30-storey & 40-storey tube-in-tube building for seismic load only is random, initially it decreases then it increases with storey height.
- In most of the columns of web frame of 30-storey framed tube buildings; Ast required is more than that in tube-in-tube building. Percentage increase in steel required varies from 2% to 36% in framed tube buildings. But in some columns, Ast required is same for both structures. But in some central columns of flange frame at lowest storey level; in tube-in-tube structure steel required is more as compared to framed tube structure.
- In case of 40-storey building in flange frame columns variation in % of A_{st} required in both structures is more in corner columns & less in central columns. In all columns, A_{st} required is more in framed tube building. But in case of web frame; Percentage increase in A_{st} required in columns in framed tube-building decreases from one corner to center columns; then it becomes zero and then % increase in A_{st} required in columns in tube-in-tube building starts increasing and it increases towards another corner column.
- Ast required is equal for the design of beams at the lowest floor level in flange frame and web frame of both types of buildings in 30-storey and 40-storey buildings in all

beams at mid span. It is because of the fact that an equal amount of moment generates in each beam.

- Manual deign results of beam verify the design by STAAD-pro.

6.3 CHOICE OF SUITABLE STRUCTURE

Framed tube structures are the initial development in the field of tall buildings; Tube-in-tube buildings are the modified form of framed tube. Since, in the case of tube-in-tube buildings, the effect of shear lag is reduced to very large extent and these buildings are also safer from the storey drift point of view, hence for very high buildings (over 50-storey) tube-in-tube structures should be adopted. But due to the provision of an additional interior perimeter column tube in tube-in-tube buildings, they may be costly as compared to framed tube buildings. At the same time AF & BM generated in different members are less in tube-in-tube buildings. Hence the sectional properties of members will reduce and correspondingly steel may be less.

Hence the choice in the two types of structures should be governed to fulfil all structural, building service, economy and safety requirements.

REFERENCES

1. Beedle L. S., Second century of the sky scraper, Council on tall buildings and urban habitat, Van Nostrand Reinhold Company, New York, 1988.
2. Coull Alex & Smith B. S., Tall building structures: Analysis and Design, John Wiley & Sons Inc., New York, 1991.
3. Coull, A., and Bose, B., (1975) " Simplified analysis of framed tube structures". Journal of structural division ASCE, Vol. 101, pp 2223-2239.
4. IS: 456-2000, Code of practice for plain & reinforced concrete, BIS, New Delhi.
5. IS: 1893-1984, criteria for earthquake resistance design of structures. (Fourth Revision), BIS, New Delhi.
6. IS: 875-1987, code of practice for design loads (other than earthquake) for buildings and structures (Second revision), BIS, New Delhi.
7. Iyengar & Iqbal, Seismic design, John Wiley & Sons Inc., New York, 1982.
8. Khan, F. R., " Current trends in concrete high-rise buildings", Proceedings, Symposium on Tall Buildings, University of Southampton, Pergamon Press, 1967.
9. "Planning and design of tall buildings", Proceeding, International conference on planning & design of tall buildings, American Society of Civil Engineers, 1989.
10. Richard N. White and Charles G. Salman, Building structural design handbook, A Wiley-Interscience Publication, John Wiley & Sons, New York, 1986.
11. Ramaswamy S. D. & Tam C. T., Tall Buildings, Proceedings of the International Conference on Tall Buildings, 22-26 October, Singapore, 1988.
12. Singh, Y., and Nagpal, A. K., (1994) " Negative shear lag in framed tube buildings", Journal of Structural Div., ASCE, Vol. 120, pp 3105-3121.
13. Schueller, W., High-rise building structures, A Wiley-Interscience publication, 1977, New York.

14. Sandeep Chakraborty, Simplified analysis of tubular structures, M. Tech., Dissertation, I.I.T. Roorkee, 2001.

15. STAAD-pro 2001, A package for analysis and design of Civil Engineering structures, Research Engineers International Ltd.