



ADDIS ABABA UNIVERSITY
SCHOOL OF GRADUATE STUDIES
FACULTY OF TECHNOLOGY
DEPARTMENT OF CIVIL ENGINEERING

**EVALUATION OF ANALYSIS AND DESIGN OF FRAMED TUBE AND
TUBE-IN-TUBE STRUCTURAL SYSTEMS FOR HIGH RISE
BUILDINGS**

A thesis submitted to the school of Graduate Studies of Addis Ababa University
in Partial fulfilment of the Requirements for the Degree of Master of Science in
Structural Engineering

By

Aster Yilma
November 2005



ADDIS ABABA UNIVERSITY
SCHOOL OF GRADUATE STUDIES
FACULTY OF TECHNOLOGY
DEPARTMENT OF CIVIL ENGINEERING

**EVALUATION OF ANALYSIS AND DESIGN OF FRAMED TUBE AND
TUBE-IN-TUBE STRUCTURAL SYSTEMS FOR HIGH RISE
BUILDINGS**

By
Aster Yilma

Approved by Board of Examiners

Chairperson, Department Graduate
Committee

Dr. -Ing. Girma Zerayohannes
Advisor

Dr. Asnake Adamu
External Examiner

Dr. -Ing. Adil Zekaria
Internal Examiner

TABLE OF CONTENTS

ACKNOWLEDGEMENT

ABSTRACT

1. Introduction.....	1
1.1 Background.....	1
1.2 Purpose of the study.....	2
2. Historical Background and Design Criteria of High-Rise Buildings and their Structural Systems.....	3
2.1 Historical background of high rise buildings and their structural systems.....	3
2.2 Design criteria of high-rise buildings.....	7
2.2.1 General design philosophy.....	7
2.2.2 Loading.....	7
2.2.2.1 Construction loads.....	8
2.2.3 Strength and Stability.....	8
2.2.3.1 Stability of high-rise buildings.....	8
2.2.3.2 P-delta effect.....	9
2.2.4 Stiffness and drift limitations.....	10
2.2.4.1 Stiffness of a tall building.....	10
2.2.4.2 Drift limitations.....	10
2.2.5 The dynamic comfort criteria.....	11
2.2.6 Creep, Shrinkage and temperature effects.....	11
2.2.7 Foundation settlement and soil-structure interaction.....	12
3. Tubular Structures for Tall Buildings.....	14
3.1 Framed-tube buildings.....	14

3.2	Tube in tube or Hull-core structures.....	14
3.3	Bundled-tube structures.....	15
3.4	Braced-tube structures.....	16
3.5	Composite Tube.....	17
4.	Behaviour of Framed Tube Buildings.....	18
4.1	Structural behaviour of framed tube buildings.....	18
4.2	The Shear lag effect in a framed tube building.....	20
4.3	Seismic behaviour of framed tube buildings.....	21
4.4	Tube in tube structural systems to improve stiffness of framed tube systems.....	23
5.	Approximate Methods of Analysing and Design of Framed Tube Structures.....	24
5.1	Initial preliminary design approach.....	24
5.2	Preliminary design using influence curves.....	25
5.3	Reduction modelling approach.....	31
6.	Analysis of Sample Framed Tube & Shear Wall Frame Buildings.....	33
6.1	General.....	33
6.2	Sample analysis.....	34
6.2.1	Preliminary design of framed-tube buildings using influence curves & reduction modelling technique	34
6.2.2	Analysis of sample framed tube and shear-wall frame buildings using ETABS.....	38
6.2.2.1	General.....	38
6.2.2.2	Framed-tube building 1.....	40
6.2.2.3	Shear-wall frame building 1.....	44
6.2.2.4	Framed-tube building 2	46
6.2.2.5	Shear-wall frame building 2.....	49
6.2.2.6	Framed-tube building 3.....	51

6.2.2.7 Shear-wall frame building 3.....	54
6.2.2.8 Framed-tube building 4.....	56
6.2.2.9 Shear-wall frame building 4.....	59
6.2.2.10 Framed-tube building 5.....	61
6.2.2.11 Shear wall-frame building 5.....	64
6.2.2.12 Framed-tube building 6	66
6.2.2.13 Shear-wall frame building 6.....	69
6.2.2.14 Tube in tube building 7.....	71
6.3 Discussion.....	73
6.4 Evaluation of analysis results.....	74
6.4.1 Shear lag effect in the sample framed tube buildings.....	74
6.4.2 P-delta effect.....	75
6.4.3 Stiffness degradation.....	76
6.5 Comparison of framed tube & shear wall frame buildings based on Concrete requirement	78
7. Conclusions and Recommendations.....	79
References.....	80

ACKNOWLEDGEMENT

This study was conducted under the supervision of my advisor, Dr.-Ing. Girma Zerayohannes, of Civil Engineering Department, Addis Ababa University. I am very grateful to my advisor Dr.-Ing. Girma Zerayohannes, for his useful suggestions and professional advice during my research work.

My thanks are also extended to Dr.-Ing. Adil Zekaria for his constructive opinions and advises.

Finally, I would like to thank C.D.S.Co., for letting me to pursue my graduate study and sponsoring me.

Aster Yilma

ABSTRACT

The socio-economic situations in large cities of Ethiopia are creating a strong need to the construction of tall buildings. To satisfy this need an economical and efficient design of high-rise buildings should be adopted. As it is well known, the structural system plays a great role in the design of high-rise buildings.

Framed-tube and tube in tube structural systems are systems, which have received wide acceptance among designers all over the world for tall buildings.

In this research work a comprehensive literature survey on historical background and design criteria of high-rise buildings and their structural system is conducted. The literature survey covers also tubular structures and their behaviour, approximate methods of analysing and design of framed tube buildings.

After the literature survey, analysis of sample framed tube, shear-wall frame and tube in tube buildings is carried out, in order to compare the effectiveness of the structural systems. Then, analysis results are evaluated, based on the design criteria p-delta effect, drift and stiffness degradation. Comparative analysis between framed tube and frame-shear wall buildings based on their concrete requirement is also made.

Finally, conclusions and recommendations are forwarded for the use of framed-tube and tube-in tube structural systems for tall buildings. Essentially, architects have to familiarise themselves with these structural systems, as they have influence on the architectural expressions of the building.

DECLARATION

I, the undersigned, declare that this thesis is my original work, has not been presented for a degree in any other University and that all sources of materials used for the thesis have been duly acknowledged.

Name	Aster Yilma
Signature	_____
Place	Addis Ababa University Faculty of Technology
Date of submission	November 2005

1.0 INTRODUCTION

1.1 Background

The need for high-rise buildings is increasing in the country. Land in the rapidly developing cities is becoming scarce and this is encouraging the commercial utilisation and the construction of high-rise buildings. The great demand for office space around business section of cities necessitates construction of high-rise or tall buildings.

In order to fulfil these needs an economical and efficient design of high-rise buildings has to be introduced. In the design of high-rise buildings, the structural system plays an important role and has to be well studied and designed. The structural systems, which are used for high-rise buildings are moment resisting frame systems, shear wall frame systems, framed tube systems, tube in tube systems and others. The Framed tube structural system is one of the most significant modern developments in high-rise structural form [12]. Framed tube and Tube-in-tube structural systems will be taken and studied in detail in this research work.

The intention of this thesis is therefore, to investigate and evaluate the application of framed tube and tube-in-tube structural systems for tall buildings. This is going to be done in this thesis work, by carrying out a comprehensive literature survey and analysis of sample buildings.

1.2 Purpose of the study

The main objective of this thesis is to investigate the effectiveness of framed tube and tube in tube structural systems for tall buildings. The investigation is to be carried out by conducting a comprehensive literature survey and by making comparative analysis between framed tube, tube in tube and the conventional shear wall frame structural systems.

The other objective of this research work is to familiarize framed tube and tube in tube structural systems to the professional structural designer.

2.0 HISTORICAL BACKGROUND AND DESIGN CRITERIA OF HIGHRISE BUILDINGS AND THEIR STRUCTURAL SYSTEMS

2.1 Historical Background of High-Rise Building Structural Systems

Reinforced concrete construction began in the early 1900s. But at that time reinforced concrete buildings were limited to only a few stories in height [1,4], since the structural system employed was the traditional beam-column frame system. This traditional beam-column system made the construction of taller buildings very expensive and economically impractical.

In the early 1950s, new structural systems (shear wall frame systems) are introduced and the use of reinforced concrete in apartment and office buildings as high as 30 stories made possible [4]. Buildings taller than 30 stories were still uneconomical, since the shear walls, which were mostly located in the core of the building, were small in size, to give sufficient stiffness to resist lateral loads, i.e., the overall dimensions of the shear walls were too small to economically provide the stability and stiffness for buildings over 30 stories. On the other hand the socio economic situations and an increasing demand for space in the growing U.S. cities created a strong need to the construction of tall buildings.

For the reasons stated above and other modern architectural requirements, such as the need for a column free space of 10 to 12 m, a new structural system had to be found. It was at this critical time in the development of the modern urban environment that one of the most famous structural engineers of the 20th century, Dr. Fazlur Rahman Khan, entered the profession [4,5,10]. He recognised that the structural systems used for high-rise construction were not a par with the modern scale of architecture, and he took on the challenge of advancing state-of-the-art structural engineering. He then brought forth series of progressive ideas for efficient high-rise construction in the 1960s and 70s, which were shown in his own efficient designs [10]. FR khan discovered structural systems appropriate for each new architectural requirement.

Eventhough, Khan developed structural systems for particular project needs, he based his innovations on fundamental structural principles that allowed their wide application.

The earliest contributions of Dr. Fazlur Rahman Khans to the field of structural engineering are the shear wall frame interaction system, the framed-tube structure, and the tube-in-tube

structure. These new structural systems brought significant improvement in structural efficiency and made the construction of tall buildings economically feasible.

The framed tube structural system was first used in 1964 in the construction of the Dewitt chestnut Apartments (fig. 2-1) in Chicago [4,10,13]. 43-story tower was designed by Fazlur Rahman Khan and his colleagues at skidmore, Owings and Merrill (SOM).

Because of its great structural efficiency, the tubed frame structural system became a standard method in high-rise design. The 83-story standard oil building in Chicago and the 110-story world trade center in New York are other examples of framed tube buildings. These buildings have interior cores. But, since the cores are not designed to resist lateral loads they act as hollow tubes [3]



Fig. 2-1 The 43-story Dewitt Chestnut apartment [4]

The concept tube-in-tube structural system was first implemented in 1962 in the construction of the 38-story Brunswick building (fig. 2-2). This concept was soon applied to many other structures including the 52-story shell oil plaza in Houston (fig. 2-3)

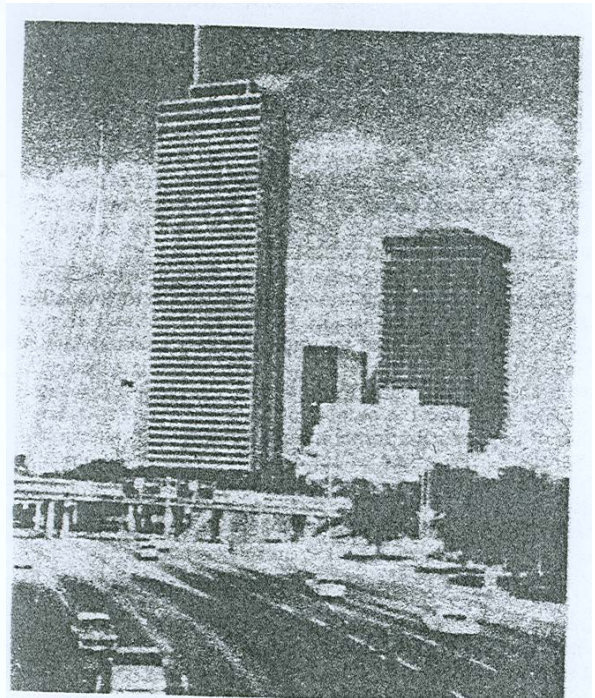


Fig 2-2 The Brunswick building in Chicago

Fig. 2-3 The 52-Story Shell oil plaza building in Houston

A trussed-tube structural system was developed and implemented for Chicago's 100-story John Hancock Center (Fig. 2.4). This structural system is an exceptionally efficient structural system with graceful form. The other very efficient structural system, which is introduced by Dr Fazlur Rahman Khan, is the bundled tube. The Chicago's 110-story sears tower (Fig. 2.5) is one example of bundled tube buildings [10]. This building was structurally efficient and economic.

The structural systems described above which are introduced by Dr. Fazlur Rahman Khan, have found wide application out of USA because of their structural efficiency and economic advantage.



Fig 2-4 The 100-story John Hancock center



Fig. 2-5 The Chicago's 110-story Sears tower

2.2 Design Criteria of High Rise Buildings

The important criterion, which must be considered in the structural design of a tall building, is described in [1,3,6] briefly. The criterion, which have particular consequences for high-rise buildings are discussed below based on the above-mentioned literatures.

2.2.1 General Design Philosophy

The limit states design philosophy is the universally accepted philosophy, which is based on semi probabilistic approach for both structural properties and loading conditions [1]. The ultimate Limit State is one of the two fundamental types of limit states, which must be considered in the design of tall buildings. Especially ultimate limit state caused by instability becomes a critical issue in such types of buildings and should be given due consideration. To ensure safety against this type of failure an appropriate safety factor must be introduced.

The other Limit State is the serviceability Limit State involving the appearance, efficiency and durability of the building throughout its design life. This can be achieved by controlling excessive deflection and crack width.

As it is well known, the above design principles apply also to low-rise buildings. But in the following, criteria that apply particularly to the design of high-rise or tall buildings will be discussed.

2.2.2 Loading

2.2.2.1 General

According to Wolfgang Schueller [3], the source of loads acting on a structure can be divided into two basic categories: these are geophysical and man made loads. The geophysical forces are the result of continuous changes in nature. The gravitational, meteorological and seismological forces belong to this category. The gravitational loads consist of dead and live load, while wind, temperature, humidity, rain snow and ice belong to meteorological loads, which vary with time and location. The man-made sources of loading may be movement of people and equipment, caused by blast and impact, shocks generated by cars, elevators, machines etc. Seismological forces are lateral forces, which result from ground motion.

Therefore, the building must be designed for gravity and lateral forces. The design forces for a particular building depend on the size and shape of the building and its geographic location.

The estimation of dead and live loads and probability of the simultaneous occurrence of different combinations of gravitational load with either wind or earthquake forces are based on the limit state design.

2.2.2.2 Construction loads

Construction loads can be defined as those are applied to members or frames of a building during construction process. Thus, the members may be subjected to loads larger than design loads [1,3,4]. These loads should be given important consideration in the design of the structural elements.

2.2.3 Strength and Stability

2.2.3.1 General

The main design requirement, for the ultimate Limit State is that the building should have adequate strength to resist and to remain stable throughout the lifetime of the building. To achieve this, analysis of the forces and stresses for the most critical load combinations has to be carried out and additional moments due to P-Delta effects have to be included. Critical members should be studied well, as their failure could initiate a progressive collapse of part of or the entire building [1]. Additional stresses resulting from restrained differential movements caused by creep, shrinkage or, temperatures have to be included in the design.

Furthermore, the most fundamental condition of equilibrium of the building should be checked, i.e. the building should also be checked against overturning to ensure that the applied lateral forces will not cause of the entire building to topple as a rigid body about one edge of the base.

2.2.3.2 Stability of High-Rise Buildings

As it is stated in [1,4], the increasing height and greater structural efficiency of tall buildings have resulted in having less or smaller reserves of stiffness and stability. Hence, the effects of this reduction in stability have to be checked very carefully during the design process of the building.

There are two stability considerations in the design of buildings, the stability of the structure as a whole is the first and the second is the stability of individual members that make up the

building. Both stability conditions must be examined thoroughly. The design for stability of individual columns of high-rise buildings is the same as for low-rise buildings and is not presented in this section. The emphasis in this section is the stability of the building as a whole or with whole stories of the building. The possibility of collapse due to the total gravity load that may cause overall buckling is very remote [1].

Instead, the more serious stability consideration is related to the second-order effects of gravity loading on lateral displacement caused by horizontal loading or acting on initial misalignments in the building. This P-Delta effect may initiate collapse in an extreme case, but it is usually either small and may be neglected or of moderate magnitude, in which case they can be accommodated by small increases in the size of the members [1]. However, it is wise to investigate whether P-Delta effects are significant or not in the design of high-rise buildings. Hence, the discussion in the next section is on P-Delta effects

2.2.3.3 P-Delta effects

As it is stated in the literatures, when horizontal loading acts in a building and causes it to drift, the resulting eccentricity of the gravity loading from the axes of the walls and columns produces additional external moments, which causes the structure to drift further. The additional drift induces additional internal moments sufficient to equilibrate the gravity load moments. This effect of the gravity loading P acting in the horizontal displacement δ is known as the P-Delta effect.

In case of lateral flexibility combined with exceptionally heavy gravity loading, the additional P-delta external moments may exceed the internal moments that the structure is capable of mobilising by drift, in which case the structure would collapse due to instability [1]. Although, second-order P-delta additional deflections and moments are small for typical high-rise buildings, they might be large enough to require consideration in flexible structures. The additional displacement due to this effect might cause unacceptable total deflection, in which case the structure has to be stiffened. Therefore in the design of high-rise buildings it is advisable to assess whether P-delta effect is significant. Different methods of P-delta analysis are discussed briefly in [1]. The first method is applying a constant amplification factor to all the results of a first-order analysis. This method is a very approximate method. The second method involves an iteration of the first-order analysis, in which the primary lateral force augmented by increments. The other iterative method is in which the gravity loads are applied to the laterally deflecting structure.

The last method is the direct method for rigid frame structures in which iterations are avoided and a direct second-order adjustment of the displacements and moments are made. In this last method, the stiffness matrix analysis of the structure includes both the first-order and second-order effects.

For the example buildings in this thesis work, P-delta effects are considered using the P-delta effect options of the analysis programme called ETABS (Extended Three Dimensional Analysis of Building Systems). This analysis program performs the P-delta effects based on iteration of the first order results [15].

2.2.4 Stiffness and drift limitations

2.2.4.1 Stiffness of a tall building

It is well known that the provision of adequate lateral stiffness is the main consideration in the design of tall buildings in order to ensure their satisfactory performance under service conditions. Regarding the ultimate limit state, the lateral deflection must be limited, so that second order P-delta effect due to gravity loading are kept as small as possible and collapse of the building will be prevented. The lateral deflection of a building can be controlled by providing sufficient lateral stiffness. Limiting the lateral deflection is also a criterion of serviceability limit state design. The serviceability limit state criteria is to keep the lateral deflections at a low level, so that the non-structural elements can function properly to prevent excessive cracking and consequent loss of stiffness.

Furthermore, any redistribution of loads to non-load bearing partitions must be avoided and the building must be sufficiently stiff to prevent motion of the building, as it affects comfort of the occupants, prevent delicate work being undertaken or affect sensitive equipment [1,3,4]. Sufficient lateral stiffness can be provided through the use of an efficient structural system.

As it is discussed in [1 and 4], the lateral stiffness of a building can be estimated by the simple parameter called drift index. The drift index is defined as the ratio of the maximum deflection at the top of the building to the total height. The control of lateral deflection is very important in modern high-rise buildings.

2.2.4.2 Drift Limitations

Deciding an appropriate value of drift index is a major design decision. The determination of a maximum permissible drift or lateral sway is based on the need to limit the adverse effects of the drift in the building. But, there are no widely accepted values of drift index or any firm guidance to determine it [1,4]. So, the designer will have to decide on an appropriate value, based on the building usage, the type of design criteria employed, the form of construction, the materials employed, the lateral load considered and past experience of similar buildings with good performance.

Design drift index limits between 0.001 to 0.005 [1] have been used in different countries. This can be further elaborated as, a maximum lateral top deflection of between 0.1 and 0.5m would be allowed for a 33-story or 100m high building or, a relative deflection of 3 to 15mm for a story having 3m height. As it is indicated in [1], a sound engineering judgement is required when deciding the drift index limit of a given high rise building. It is recommended in [1] that, for conventional structure, the value of drift index should lie between 0.0015 to 0.003 and sufficient stiffness has to be provided so that the top deflection does not exceed the given value under extreme load conditions. It is further to be noted that as the height of the building increases the drift index values should be decreased, in order to keep the top story deflection to a lower level. But it has to be realised that the drift criteria apply basically to quasistatic conditions. So, in addition to the drift criteria, the dynamic comfort criteria have to be considered in the design of high-rise buildings.

2.2.5 The dynamic comfort criteria

If a tall building is subjected to oscillatory movements due to lateral or torsional deflections under the action of lateral loads such as wind loads, it may cause psychological or physiological effects to the occupant of the building [1,3,4]. It is stated in [1] that, in several buildings in the 40 to 50-story range in New York City, excessive lateral sway and noise are observed. Employees are regularly excused from work during high windstorms, since the situation is very difficult for them to work at their desk. People feel the movement and sense the twisting of the building, and some of them have experienced motion sickness caused by building sway. Due to the above-mentioned problems a building may become undesirable or even unrentable.

Therefore, the reduction of such noticeable motion to an acceptable level is an important design criterion of tall buildings. As it is presented in [1,4], acceleration is the predominant parameter in determining human response to vibration. Other factors like period, amplitude, past experience, etc. can be also influential.

Universally accepted standards or reliable limiting discomfort to the occupants are not yet available. But as it is noted in [1,4], some curves that give various limits for human behaviour in terms of acceleration and period and tentative acceleration limitations are available.

2.2.6 Creep, Shrinkage and Temperature Effects

Material volume changes may result from shrinkage, creep and temperature effects. When the response to the change in volume of the building is prevented loads may result. Axial and rotational stresses are then induced in the building where these volume changes are prevented. Volume changes can be controlled by allowing free movement by using expansion joints [3].

In very tall concrete buildings, the time-dependent shortening of columns and shear walls is of great concern due to its cumulative nature. As the number of stories increases, the cumulative shortening of columns also increases. There might also exist a significant differential shortening. The cumulative distortion may cause damage to non-structural elements and overstressing the slabs in the upper levels. A large heavily reinforced column [4] attracts additional loads from the adjacent shear wall, which has higher creep and shrinkage due to a lower percentage of reinforcement and a lower volume-to-surface ratio.

Another example from [1] is an 80- story-high concrete column, which might be expected to shorten by about 4.6 cm due to creep and 6cm due to shrinkage. Therefore, the influence of creep, shrinkage and temperature effects should be taken into consideration in the design of tall or high-rise buildings.

2.2.7 Foundation Settlement and Soil-Structure Interaction

The principles of foundation design for tall buildings are not different from that of low-rise buildings. It is the influence of any foundation deformation on the building's structural behaviour and on the soil-structure interactive forces that needs special consideration.

The loads transmitted by the column in a tall building can be very heavy due to its height. In a site where the underlying soil is rock or other strong stable subgrade, the loads from the superstructure can be carried down to the stiff load-bearing layers using appropriate type of

foundation. Differential settlement is negligible in such conditions. But in areas where soil conditions are poor, shear failure or excessive differential settlement can be encountered. Thus, to prevent such problems, loading on foundation elements must be limited. The foundation should also be checked, to make sure that there is safety against uplift of foundation.

Foundation deformations have two major influences on a building [1]. The first is that the influence of the relative displacement on the forces in the horizontal elements. The second influence of foundation deformation on buildings occurs when an overall rotational settlement of the entire foundation occurs. This type of movement increases the maximum drift and a destabilising effect may be induced on the structure as a whole by increasing the P-delta effect. In the soil-structure interaction, the static and dynamic behaviours have to be considered. The static soil structure can be studied using simplified models of subgrade behaviour and finite element methods. It is further noted in [1], that both interaction between soil and structure, and any amplification caused by a coincidence of the natural frequencies of building and foundation have to be included in the study of the dynamic behaviour of soil structure interaction. The possibility of liquefaction during an earthquake should also be checked. Therefore, the soil structure interaction must be considered in tall buildings, especially under seismic actions.

3.0 TUBULAR STRUCTURES FOR TALL BUILDINGS

3.1 Framed-Tube Buildings

The framed tube system in its simplest form consists of closely spaced exterior columns tied at each floor level with relatively deep spandrel beams, thereby creating the effect of a hollow concrete tube. This structural system is referred to as „ framed tube“; since it simulates a hollow tube using perimeter of closely spaced frame elements.

As in many structures of this form, the exterior tube is designed to resist the entire lateral loading. The frames parallel to the wind act 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.

Tubular structures are most commonly square or rectangular in shape, but circular, triangular and trapezoidal shaped cross– sections can also be employed.

The center to center spacing of the exterior columns in the system is generally from 2m to 4m. The spandrel beams interconnecting the closely spaced columns generally vary from 60cm to 120cm in depth with widths from 25cm to 90cm.

The closely spaced column configuration makes access difficult to the public area at the base. This difficulty can be avoided by using a large transfer girder to collect the vertical loads from the closely spaced columns and distribute them to a smaller number of larger more widely spaced columns at the base.

3.2 Tube-in-Tube or Hull-Core structures

A tube-in-tube system is a variation of the framed tube created by the shear walls, and the outer tube consisting of the closely spaced column system. The tube in tube system has the advantage of both the framed tube structures and the shear wall type structures. The shear deflection of the columns of the framed tube is considerably reduced by the shear wall inner tube; thereby enhancing the structural characteristics of the exterior framed tube. The tube-in-tube system is a refined and unique version of the shear wall-frame interaction type structures [4].

3.3 Bundled –Tube Structures

These structural systems consist of tubes, which are bundled together. As in the single-tube structure, the frames in the direction of lateral loading serve as “Webs” of the vertical cantilever, with the normal frames acting as “flanges”.

It is stated in [5], that the introduction of the internal webs greatly reduces the shear lag in the flanges. Due to this, the columns of bundled-tube structures are more evenly stressed than in the single-tube structure and their contribution to the lateral stiffness is greater.

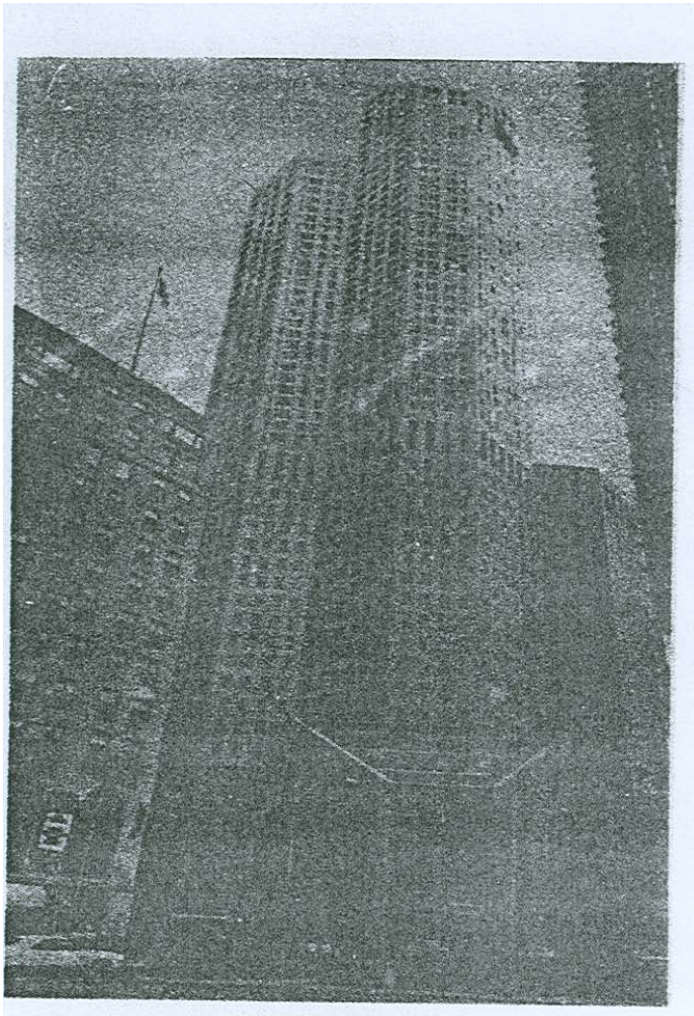


Fig. 3.1 Bundled-tube [1]

3.4 Braced –Tube System

The efficiency of the framed-tube system can be improved by adding bracings. The addition of diagonal bracing to the faces of the tube increases its potential for use to greater heights as well as allowing greater spacing between the columns. In concrete structures the bracing is formed by a diagonal pattern or concrete window-size panels, poured integrally with the frame [5].

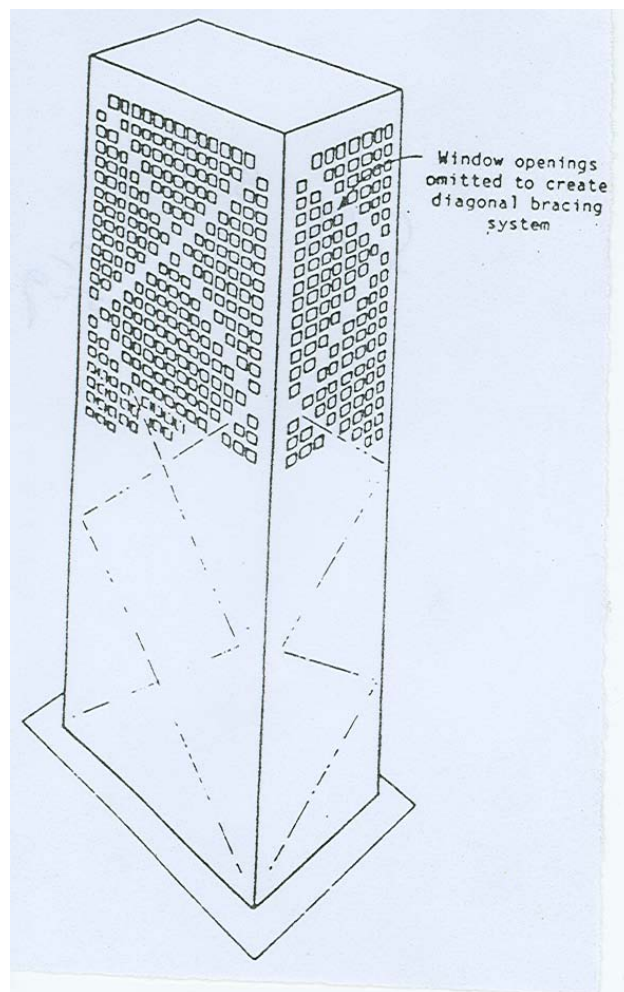


Fig 3.2. Concrete braced tube [1]

3.5 Composite Tube Systems

In this system the entire building is designed by composite construction. This approach combines the fast erection and high strength of steel construction with the fireproofing, insulating, lateral rigidity, and moldability of the concrete curtain wall [3].

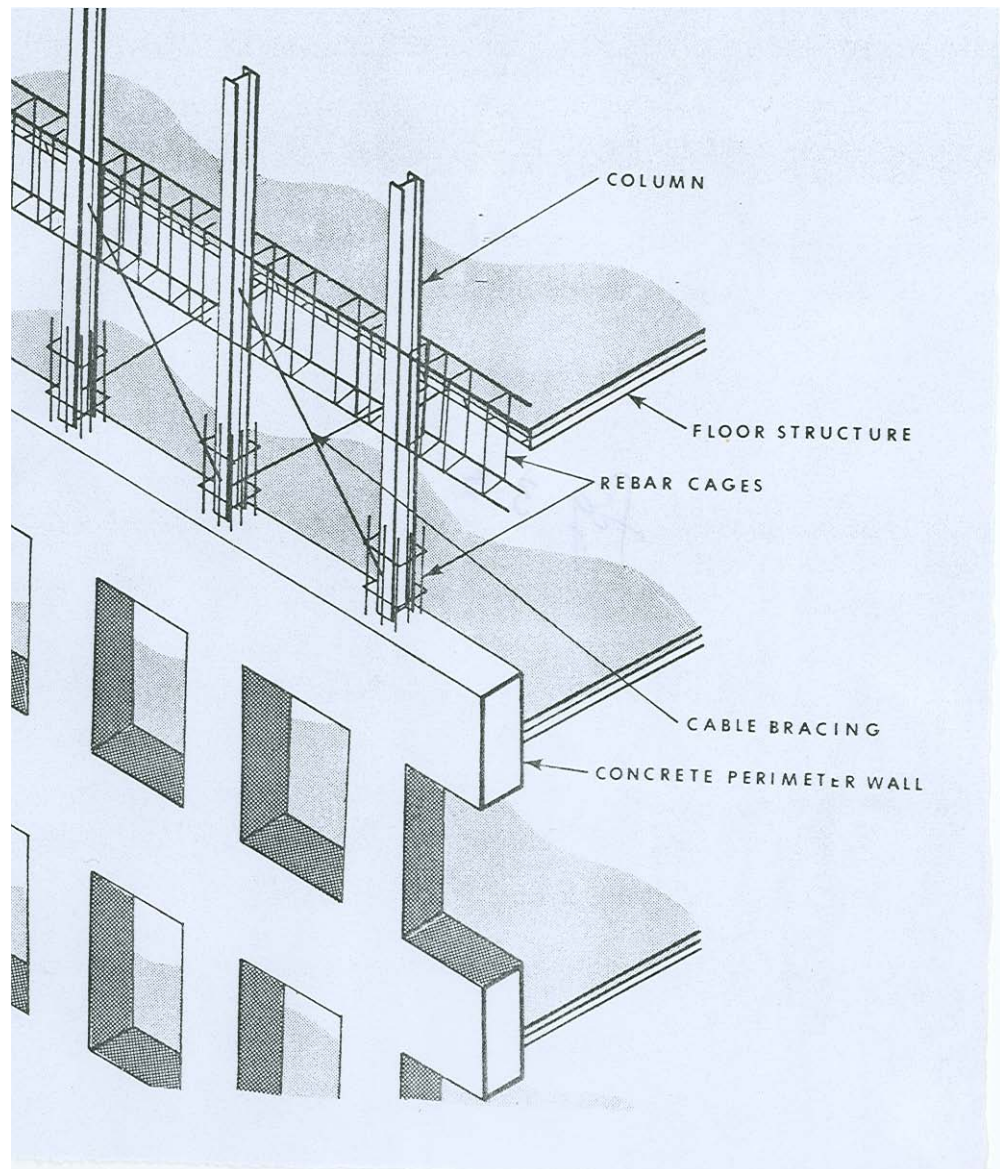


Fig. 3.3 Schematic sketch of a Tubular Composite Building [3]

4.0 BEHAVIOUR OF FRAMED TUBE STRUCTURAL SYSTEM

4.1 Structural Behaviour of Framed Tube Buildings

As previously mentioned, framed tube systems are structural systems having closely spaced columns all along the perimeter and interconnected by perimeter beams and floor slabs.

Although the structure has a tube-like form, its behaviour is much more complex than that of a plain unperforated tube, and the stiffness is also considerably less. It would be ideal in the design of framed tube systems if the exterior walls were to act as a unit, responding to lateral loads in pure cantilever bending. If this were the case, all columns that made up the tube would be either in direct axial tension or in compression [3].

However the true behaviour of the tube lies somewhere between that of a pure cantilever and a pure frame, i.e., this system combines the behaviour of a true cantilever, such as a shear wall, with that of a beam-column-frame.

The bending under the action of lateral forces is resisted by the tube form causing compression and tension in the columns on opposite sides of the neutral axis as indicated by the broken line in Fig. 4.1. The shear from the lateral load is resisted by bending in columns and beams primarily in the two sides of the building parallel to the direction of the lateral load.

In other words, two distinct actions take place in framed tube structural systems. The first action is frame behaviour of the web of the perforated tube and the second is tube behaviour of the front and back of the perforated tube (flanges).

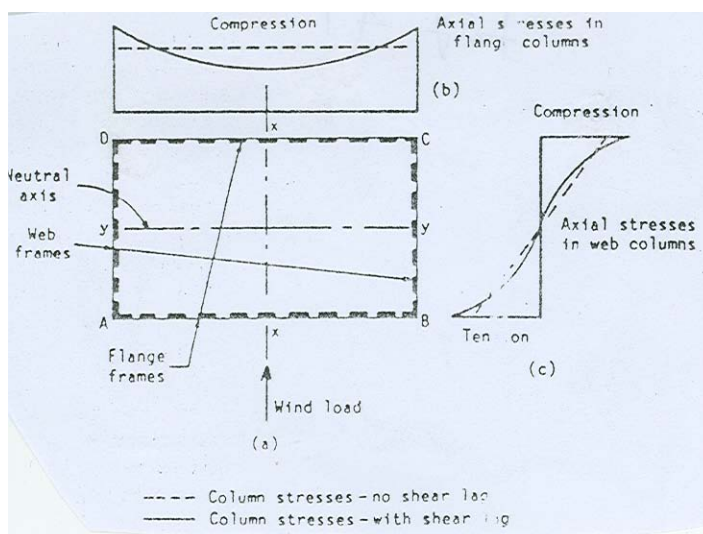


Fig. 4.1 Axial stress distribution in columns of laterally loaded framed tube [1]

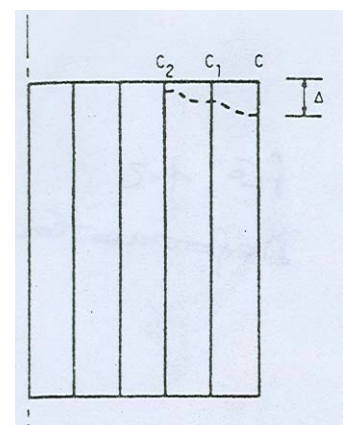


Fig. 4.2 Deformation of flange frame causing shear lag [1]

It is recognised that, the webs of the hollow tube, that is, the two sides parallel to the direction of the lateral force, are not truly solid webs but are grid frames. Thus, the effect of loss of efficiency due to flexibility of this web-frame has to be considered.

The flexibility of the spandrel beams or the web-frames produces a shear lag, which increases the stresses in the corner columns and reduces those in the inner columns of both the flange panels (AB & DC), and the web panels (AD & BC) as shown by the solid lines in Fig. 4.1. This behaviour may readily be understood by considering the basic mode of action involved in resisting lateral forces. The primary resistance comes from the side web panels, which deform so that the columns A and B are in tension and D and C are in compression. The interaction between the web and flange frames gives an intermediate type of deflection mode, which results from the combination of shear mode due to frame behaviour of the webs and bending mode due to tube behaviour of the flanges [13].

The principal interaction between the web and flange frames occurs through the vertical displacements of the corner columns. These displacements correspond to vertical shear in the girders of the flange frames, which mobilises the axial forces in the flange columns.

The cause of shear lag is discussed very briefly using an example in [1] as follows. When column C in Fig. 4.2 for example, suffers a compressive deformation, it will tend to compress the adjacent column C_1 , since the two are connected by the spandrel beams. The compressive deformations will not be the same, since the flexible connecting spandrel beam will bend, and the axial deformation of the adjacent column will be less, by an amount depending on the stiffness of the connecting beam. Pure tubular behaviour would theoretically need connecting beams having infinite stiffness. The deformation of column C_1 will in turn induce compressive deformations of the next inner column C_2 , but the deformation will again be less. Thus each successive interior column will suffer a smaller deformation and hence a lower stress than the outer ones. As the external applied moment must be resisted by the internal couple produced by the compressive and tensile forces on opposite sides of the neutral axis of the building, the stresses in the corner columns will be greater than those from pure tubular action, and those in the inner columns will be less [1].

The difference between the pure tubular stress distributions and framed-tube is shown in Fig. 4.1. Since the column stresses in a framed tube system are distributed less effectively than in a proper tube, the moment of resistance and the flexural rigidity are reduced. Therefore, the shear lag in framed tube buildings are caused by the flexibility of the spandrel beams as mentioned above

4.2 The Shear Lag Effect in a Framed-Tube Building

As stated in the previous section the effect of shear lag on the tube action results in non-linear pressure distribution along the column envelope, the columns at the corners of the building are forced to take a higher share of the load than the column in between [3]. Furthermore, the total deflection of the building does not resemble a true cantilever tube, as shear mode deformation becomes more significant.

Thus the shear problem severely affects the efficiency of tubular systems, and all later developments of tubular designs try to overcome it [3]. The occurrence of negative shear lag in framed tube buildings under lateral loading is reported in the literature. But explanations of its origin and comprehensive studies of it are presented in [7] by Y.Singh and A K. Nagpal. The paper presented in this Journal of Structural Engineering separates column axial force distribution in a story into two modes: mode-1 contributing to the positive shear lag and mode-2 to the negative lag. The net shear lag, positive or negative, is the resultant of the axial force distributions in these modes.

In this section origin and effects of shear lag and the structural parameters governing the shear-lag behaviour will be discussed based on the paper presented in [7].

Origin of negative shear lag in framed-tube building as mentioned above is explained by identifying the two modes of behaviour. Overall behaviour of the building depends on the net effect of the two modes. In this paper it is demonstrated that negative shear lag originates from positive shear lag and counteracts it. Furthermore, it is stated that, negative shear lag cannot occur, if positive shear lag is absent.

Shear-lag behaviour of a framed-tube building depends on the structural properties of the building and the type of loading. The behaviour of a framed-tube building in the two modes was studied in the paper by considering a 40-story building as an example.

The non-dimensional structural parameters, stiffness factor S_f ($S_f=12EI_bH/EA_cL^3$), Stiffness ratio S_r ($S_r=EI_cL/EI_bH$), Ratio of number of stories to the number of bays in half-range, ρ and Aspect ratio A_r (ratio of flange dimension to web dimension) are identified for the purpose of the study.

Where L =beam span, H =story height, I_b =moment of inertia of beam, I_c =moment of inertia of column, A_c = cross-sectional area of column

The ratio f of axial force in the corner column to that in the central column of the flange was chosen as the measure of shear lag. A value of f greater than unity represents positive shear lag, and that less than unity represents negative shear lag. The level at which f is unity

represents the level of shear-lag reversal. In this study, it is observed that, increase in stiffness ratio S_r implies an increase in the restraint provided by the columns to the rotation of beams resulting in increased shearing rigidity of a bay and therefore decrease in shear lag. As the stiffness factor increases, shearing rigidity of a bay increases, resulting in a relatively more uniform distribution of column axial forces, i.e., reduced positive and negative shear lags. Variation of f for different values of ρ was also investigated, and it is seen that a decrease of shear lag, positive as well as negative, with an increase in ρ . A large number of stories results in a greater cumulative shear stiffness of beams of a bay, and hence in reduced shear lag. In addition, with smaller number of bays, the number of steps in which axial forces in columns are reduced from corner column to center column is smaller, resulting in reduced shear lag.

It was seen also that aspect ratio A_r does not have significant effect on the shear-lag behaviour of the flange frame. The shear-lag behaviour of the building under three types of loading, 1) uniform load throughout the height of the building; 2) a point load at the top; and 3) triangularly varying load along the height was also studied in the example. From the study it is observed that shear-lag reversal takes place under all the loading types, though negative shear lag in case of a point load is limited to a small area near the top of the building. For uniform and triangular loading, the shear-lag behaviour of the building was almost similar, and shear-lag reversal took place at about one-quarter to one-third of the height of the building.

The shear lag effect in framed-tube buildings varies along height and changes its direction at a certain level, positive shear lag occurs in the bottom portion of the building while negative shear lag occurs in the top portion of the building. Shear lag, positive as well as negative, is more pronounced for lower values of structural parameters S_f , S_r and ρ .

4.3 Seismic Behaviour of Framed-Tube Buildings

In this section the performance of framed-tube buildings in seismic zones will be discussed based on a paper presented in 11th WCEE (World Conference on Earthquake Engineering).

The performance characteristics of tall framed tube buildings in seismic zones was carried out by MIR M. Ali and presented in 11th WCEE (World Conference on Earthquake Engineering) [6]. In this paper it was confirmed that tall-framed tube buildings perform well as moment-resisting space frames under lateral seismic loads due to their ductile three-dimensional behaviour.

As it is mentioned in the previous sections, the tubular behaviour of a framed tube that results in axial stress in the columns does not develop 100% efficiency (i .e, as for an ideal tube) due to shear lag. An approximate method of dynamic analysis of tubed frames using a continuum approach was developed by Chang and Foutch (1983) [6]. A measure of tubular efficiency of a building can be found by determining the shear-wracking portion of the lateral deflection [6]. The shear-wracking portion of the lateral deflection results from flexibility of spandrel beams, i.e. due to bending of beams and columns. It is stated in the above-mentioned paper that a good tubular characteristic is indicated, if the shear wracking portion is less than 20 to 40 % of the deflection.

Another measure of the tubular characteristics is to compare the minimum axial stress in the interior columns and that in corner columns belonging to the flange frame. It is known that the flexibility of the spandrels increases the axial stresses in the corner columns and, thereby, increases the shear lag effect. There are some important parameters to be considered for the analysis and design of framed tube buildings under seismic loading.

These parameters are the bending stiffness K_c and K_b of columns and beams, which are defined as:

$K_c = I_c/H$; $K_b = I_b/L$, where I_c = moments of inertia of the column

I_b = moments of inertia of the spandrel beams

H = height of the column, L = the beam length

The shear stiffness of the spandrel beams is define as,

$$S_b = 12EI_b/L^3$$

and the axial stiffness of column is defined as

$$S_c = A_c E/H \quad \text{where } A_c = \text{cross-sectional area of the column}$$

E = modulus of elasticity of the material

According to [6], framed-tube behaviour is controlled by the parameters S_r and S_f . Where S_r and S_f are defined as:

$$\text{Stiffness Ratio, } S_r = K_c/K_b,$$

$$\text{Stiffness Factor, } S_f = S_b/S_c$$

It is stated in [6] that as S_f increases, the axial stress in the corner column decreases. This will result in a decrease of the shear lag effect. Thus, from the above equation of S_f one can see that in order to minimise shear lag, S_f should increase. This can be done by increasing I_b and reducing A_c . It is noted in the paper that, the most effective means to increase S_f is to increase

I_b , since A_c is usually influenced by vertical column force due to gravity loads and, hence, preselected on that basis.

The seismic performance of a framed tube building is investigated in [6] by taking an example of a tube-in-tube ultra high-rise building. A full 3D model was adopted for the computer analysis. First, a wind load and a static earthquake analysis were performed, to obtain a preliminary set of member sizes. After evaluating the structure on the basis of the results of the approximate analysis, i.e., as to whether any further adjustment of member sizes are required or not, a dynamic analysis was conducted on the full 3D model using a computer program. From the result it was observed that the building under consideration has a large fundamental period. And the acceleration was low for such a long period of vibration. The building is not obviously excited as much as for a shorter building with a smaller period. For the given example, the lateral deflection of the structure for the maximum probable earthquake was found to be similar to that obtained in the approximate analysis.

The example demonstrated that a framed tube building is generally suitable for seismic zones because of the inherent ductile moment-resisting nature of this structural system.

4.4 Tube-In-Tube Structural System to Improve Stiffness of Framed Tube System

As it is mentioned in section 2.2 a tube in tube system is a variation of the framed tube system. The tube-in-tube system has the advantage of both the framed tube structure and the shear wall type structure. The shear wall inner tube greatly enhances the structural characteristics of the exterior framed tube by reducing the shear deflection of the columns of the framed tube [4]. The stiffness of a framed tube system is very much improved by using the core to resist lateral loads [1,3,4]. It is concluded in [5] that in terms of lateral stiffness, the framed tube system is only about 25 percent as efficient as true rigid tube system and 50 percent as efficient as a tube-in-tube system. It is stated also in the same literature that for buildings up to about 120 meters the framed tube is an optimum structure.

5.0 APPROXIMATE METHODS OF ANALYSING AND DESIGN OF FRAMED TUBE STRUCTURE

5.1 Initial Preliminary Design Approach

The cantilever tube type behaviour becomes significant when the overturning of the entire building due to lateral load is considered. However, the loss of efficiency due to the flexibility of the web-frame, which causes shear lag, has to be taken into consideration. The effective configuration of the tube could be reduced to two equivalent channels (Fig. 5.1) resisting the total overturning moments, for a very preliminary estimate of the overall resistance, as well as the deflection of the building [4].

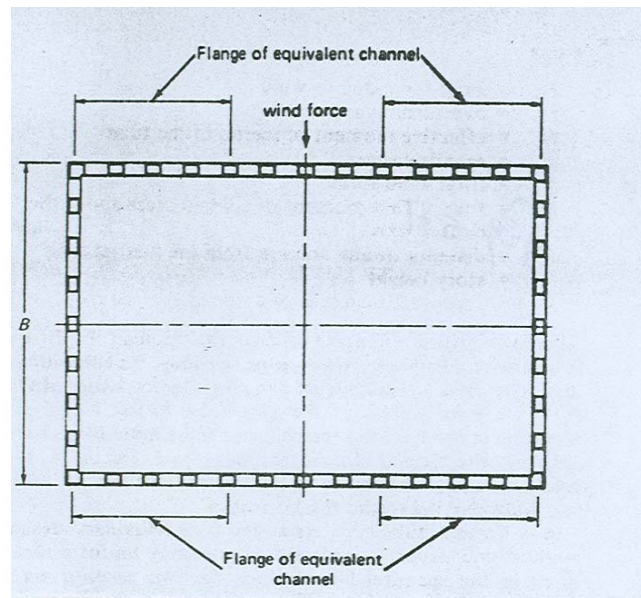


Fig. 5.1 Framed tube plan [4]

It is stated in [4] that for preliminary designs channel flanges should not be more than half the depth of the web or more than about 10% of the height of the building. It is further noted that these approximate rules have given conservative values of shear and moment as compared to the actual forces in the exterior columns obtained by the exact analysis performed by a generalised computer program.

In this section, the preliminary design approach of framed tube buildings will be discussed based on Dr. Fazlur R. Khan's writings in [4]. The overturning moment resisted by the two equivalent channels will produce axial forces in the columns of the channels and shear forces in the connecting spandrels. The preliminary estimate of the axial forces in the columns and the shears in the connecting spandrel can be expressed as:

$$P_w = M \cdot C \cdot A_c / I_e, \quad V_s = V_w \cdot Q \cdot h / I_e$$

Where, P_w = axial force due to wind

M = overturning moment

I_e = effective moment of inertia of the tube

V_s = spandrel shear

V_w = total wind shear

Q = sum of first moment of column areas about the neutral axis

C = distance of any column from the neutral axis

H = story height

A_c = cross-sectional area of the column

The preliminary design method will indicate a uniform shear force in the spandrel beams along the walls parallel to the wind. The preliminary moments in the spandrel beams are derived from these shears. The axial forces due to overturning should be added to the known dead and live loads in the design of the closely spaced columns.

5.2 Preliminary Design Using Influence Curves

Dr. Fazlur Rahman K. has developed influence curves for a relatively accurate preliminary design of framed tube buildings. This method can be used to achieve a more accurate design than the equivalent channel method for framed tube buildings of any proportion and height within practical range. These curves have been developed on the basis of a number of computer runs on a 10-story equivalent framed tube with variable non-dimensional parameters, representing ratios of shear stiffness, S_b , of the spandrel beam to the axial stiffness, A_c , of the columns, and a linearly varying ratio of bending stiffnesses of columns to spandrels. According to [4], the structural properties affecting the tube action and which are important in the use of influence curves are:

1. Bending stiffness: K_c for column = I_c/H

K_b for spandrel beam = I_b/L

2. Shear stiffness of the spandrel beams (the force required to displace one end of the spandrel a unit distance at right angles to the axis of the beam):

$$S_b = 12EI_b/L^3$$

3. Axial stiffness of the column (the axial force required to shorten the column a unit distance along the axis of the column):

$$S_c = A_c E / H$$

Where I_c = moment of inertia of the column
 I_b = moment of inertia of the spandrel beam
 A_c = cross-sectional area of the column
 H = height of column
 L = effective span of the spandrel beam
 E = modulus of elasticity

As it is stated in section 4.3 the controlling parameters of framed tubes are :

Stiffness factor, $S_f = S_b/S_c$

Aspect ratio, $R = \text{flange dimension}/\text{web dimension}$

The influence curves shown in Fig. 5.3 to 5.11 are plotted after analysing framed tubes for uniform lateral load and for the ranges of variables: aspect ratio values of 0.5, 0.666, 1, 1.5, and 2.0; Stiffness factor values of 0.1, 1.0, 10.0 and a linearly varying stiffness ratio of 0.75 at roof to 0.5 at ground level [4]. The design curves are developed based on the configuration shown in Fig. 5.2, by analysing the equivalent framed tube.

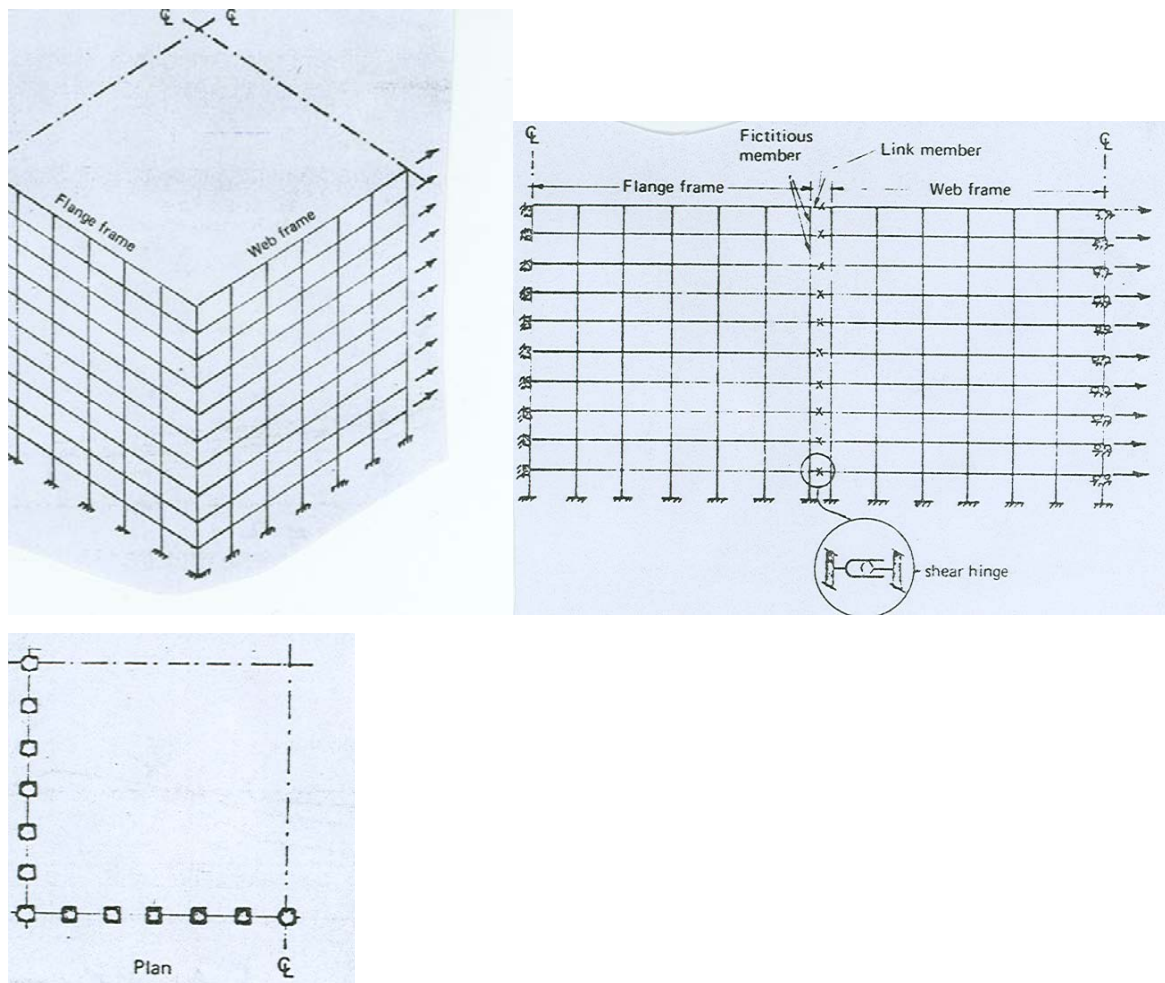


Fig 5.2 Substituting an equivalent plane frame for the framed tube

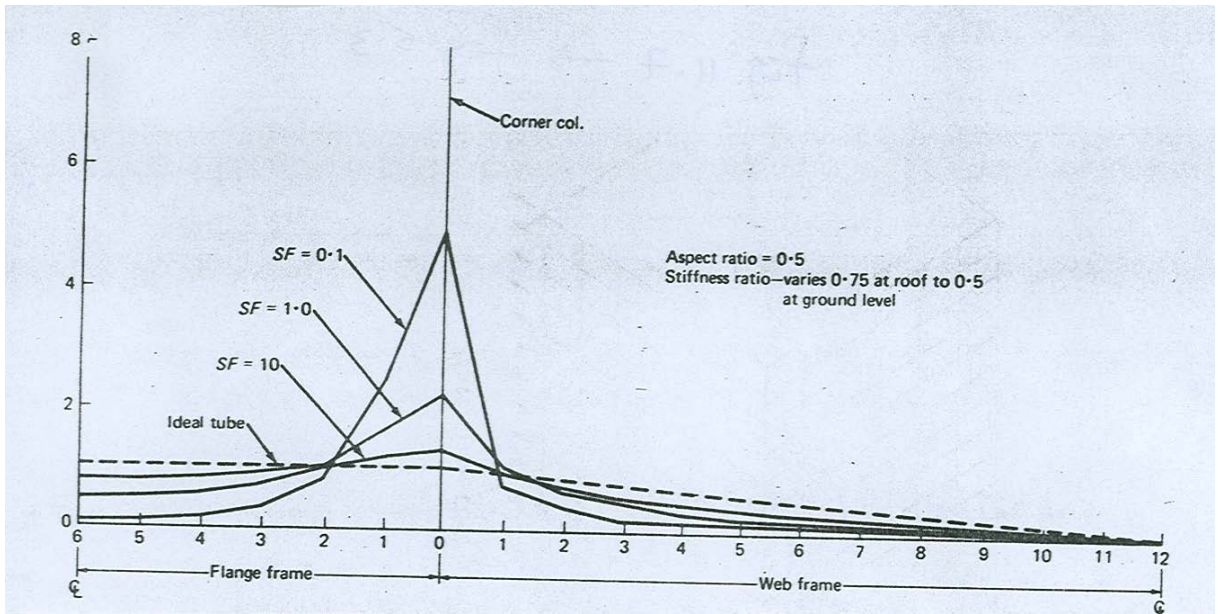


Fig 5.3 Column axial force coefficients-level 1, aspect ratio=0.5

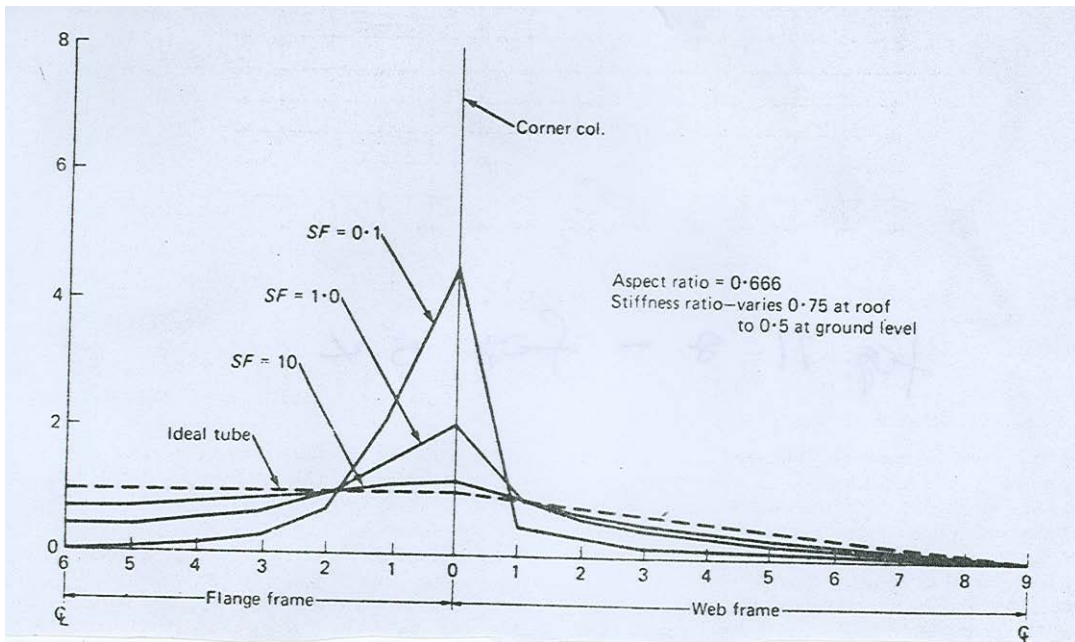


Fig 5.4 Column axial force coefficients-level 1, aspect ratio=0.666

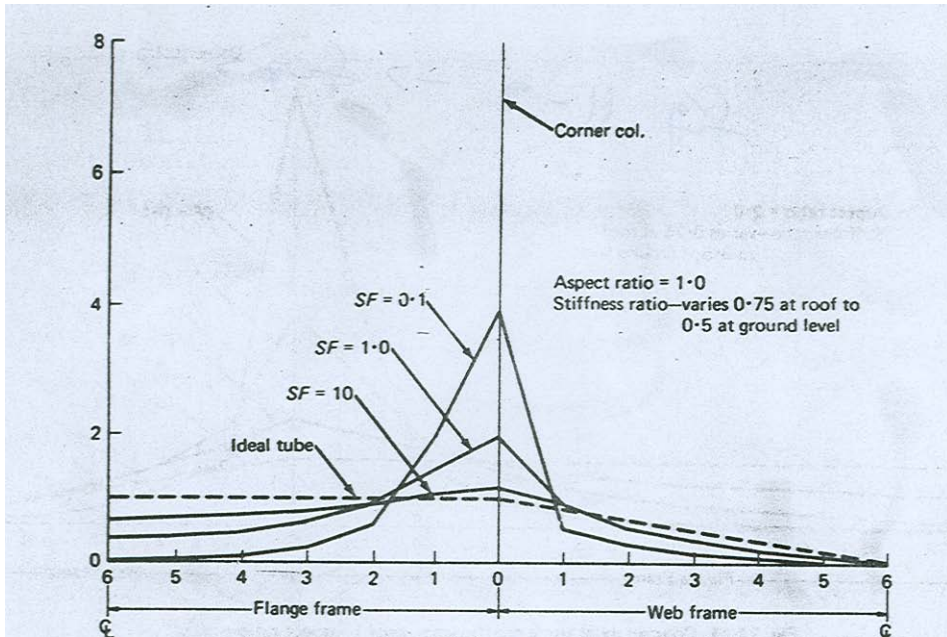


Fig 5.5 Column axial force coefficients-level 1, aspect ratio=1

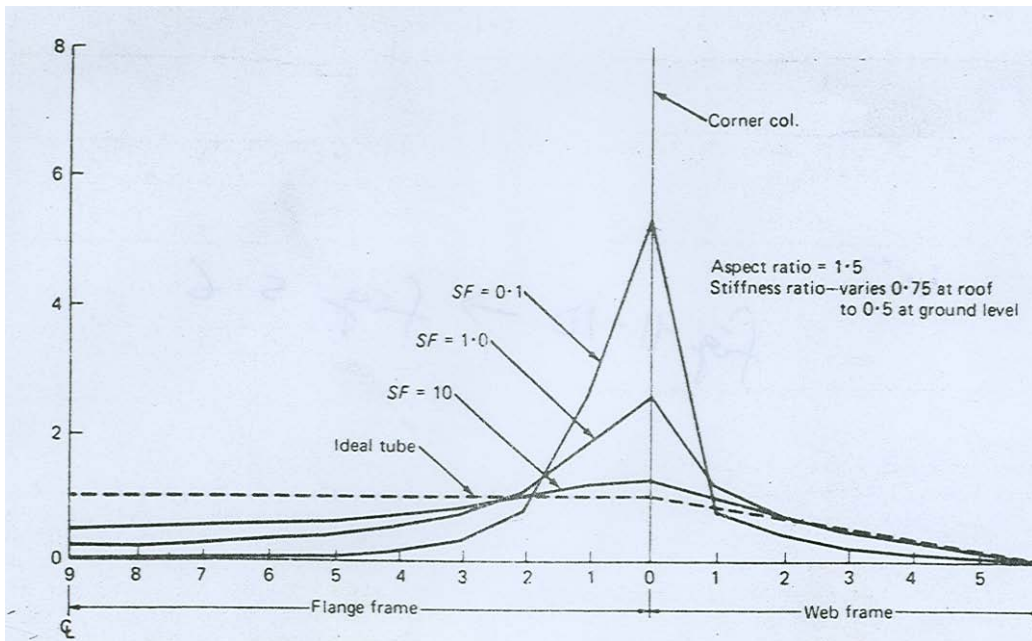


Fig 5.6 Column axial force coefficients-level 1, aspect ratio=1.5

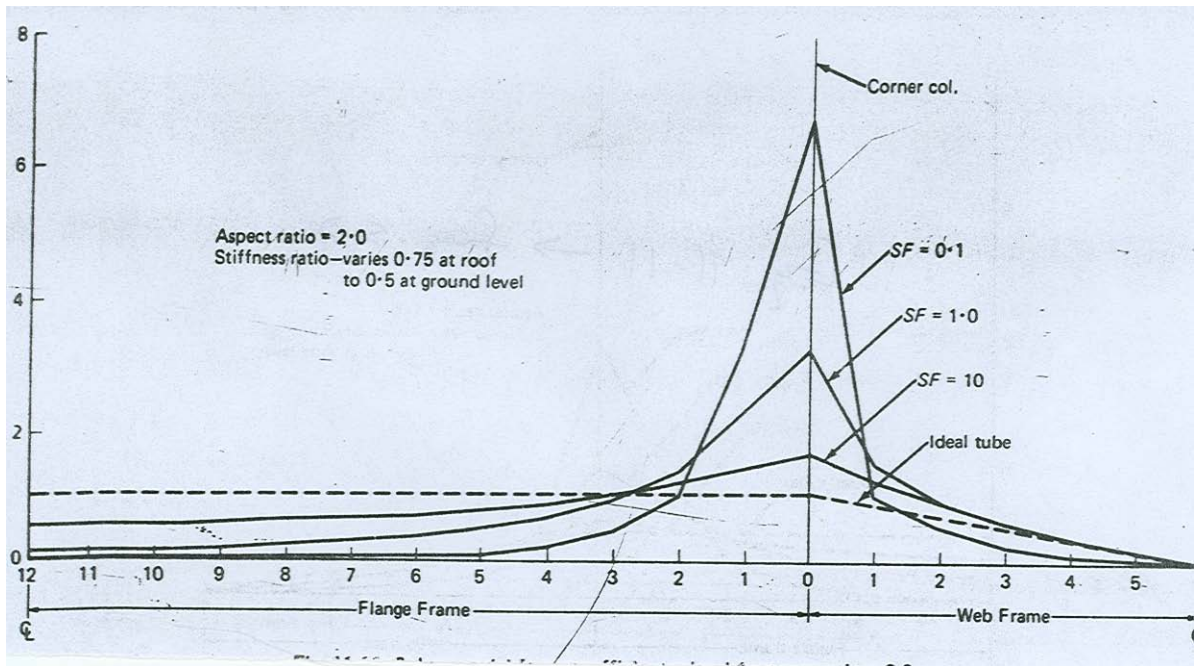


Fig 5.7 Column axial force coefficients-level 1, aspect ratio=2.0

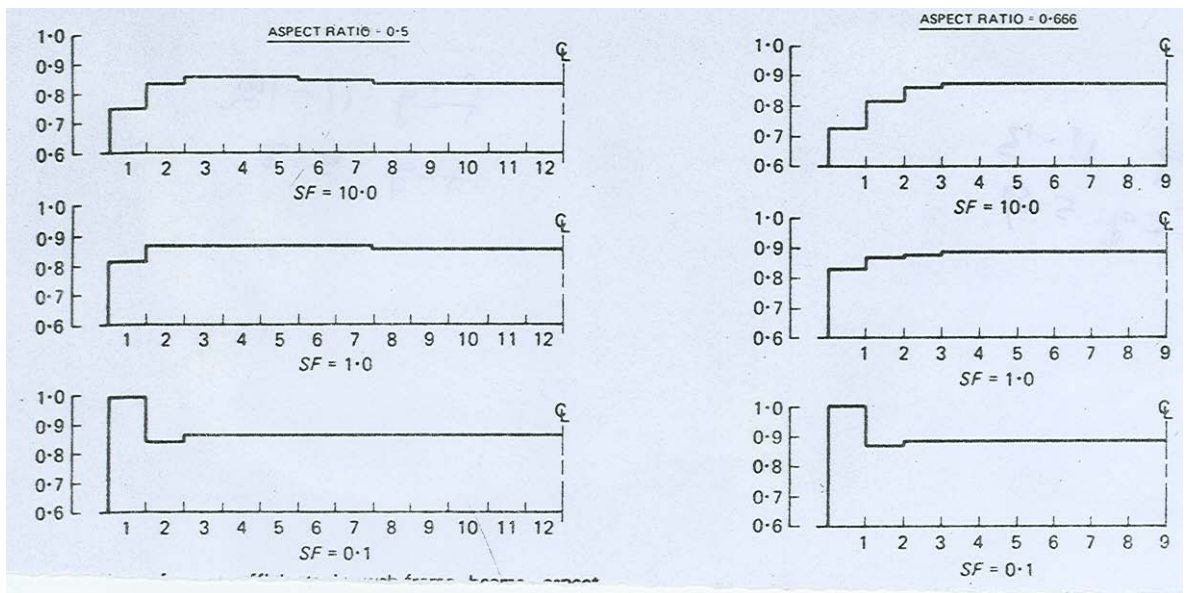


Fig 5.8 Shear force coefficients in web-frame beams, aspect ratio=0.5

Fig. 5.9 Shear force coefficients in web-frame beams, aspect ratio=0.666

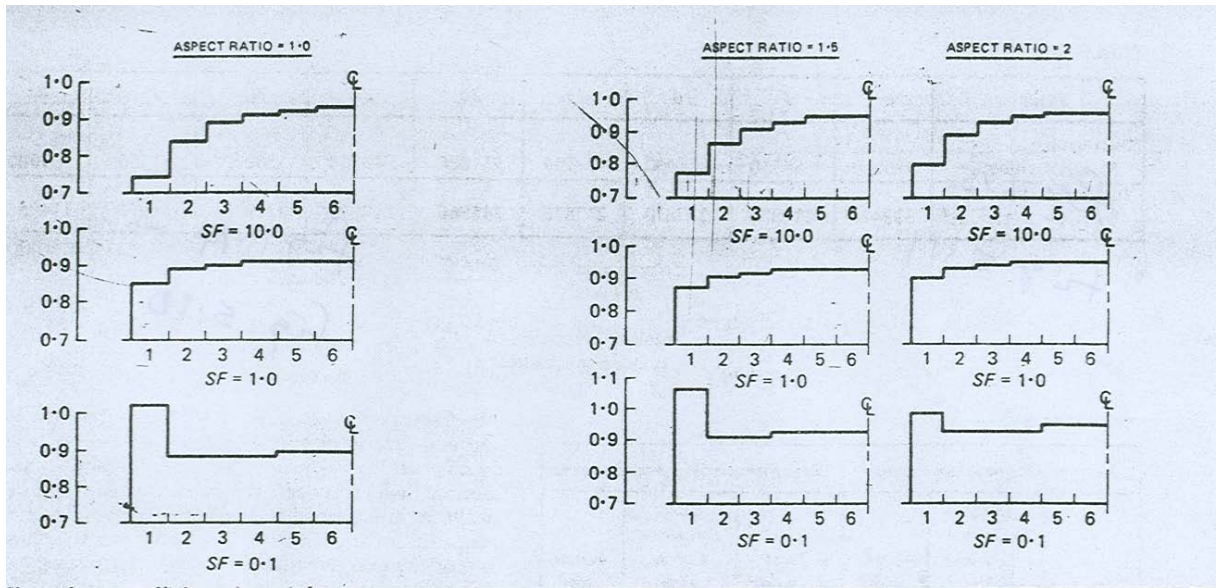


Fig. 5.10 Shear force coefficients in web-frame beams, aspect ratio=1.0 [4]

Fig. 5.11 Shear force coefficients in web-frame beams, aspect ratio=1.5 & 2

Using these curves, column axial force coefficients for flange and web frame columns and shear force coefficients for the web frame beams can be computed. The computed coefficients are related to unit values for the corresponding forces of the ideal tube.

Generally, the main uses of the influence curves are to determine the tubular characteristics of any given framed tube and compute the total deflection and bending moments and shears in the beams caused by the tubular nature of the entire structure.

5.3 Reduction Modelling Approach

It is stated in [4] that a reduction modelling approach can be employed to make the influence curves described in the previous section, usable for any number of stories as well as any number of columns around the perimeter of a building.

The solutions of the influence curves were obtained on a number of specific 10-story hypothetical framed tubes. But the tubular behaviour of any framed tube of any number of stories can be simulated by converting it into an equivalent framed tube of the same height, having a fewer or greater number of stories. For any given aspect ratio, R , of a building, the variables that affect the tubular stress distribution are the shear stiffness S_b of the spandrel beams, and the axial stiffness, S_c , of the columns.

According to Dr. Fazlur R. Khan, any framed tube of N stories can be reduced to a 10-story equivalent tube of the same height by considering a transformation of the actual stiffness factor to a 10-story equivalent stiffness factor of S'_{f10} , where

$$S'_{f10} = S_f * (N/10)^2$$

Therefore, any given actual framed tube can first be converted to an equivalent 10-story framed tube using the reduction model technique and analysed by using the influence curves. The use of the influence curves is illustrated by taking an example building in [4], and is presented here in short.

In the example a hypothetical framed tube 50 stories high and 39.6x54.9m in plan with columns 3m center to center and 3.96m floor to floor height is considered. Before using the influence curves, the basic parameters of the structure, such as aspect ratio, average stiffness factor, average stiffness ratio and frame deflection are computed. In addition, the deflection and forces in columns and beams for an ideal tube of the same aspect ratio must be known before using the influence curves.

For this example stiffness factor =10 and aspect ratio = 1.5 is used. The column axial forces in first story columns were computed using the influence curve in Fig. 5-6, and the beam shears in the first story model were computed using the curve in Fig. 5.11. In this example it is noted that the deflection of the total system is the sum of the frame deflection plus the ideal tube deflection times the magnification factor. The magnification factor is defined as the ratio of the sum of the ideal tube column forces to the actual column forces of the flange columns.

The results obtained using the approximate method in this example were compared with an exact computer analysis, and they show that the use of influence curves gives a result with an accuracy of 90 to 95%.

5.4 Approximate Analysis of Tube-In-Tube Structural System

It is necessary to define the factors contributing to the total lateral deflection of tube-in-tube structural system, before making an approximate analysis for preliminary design of such system [4]. When the tube-in-tube structural system is subjected to lateral loads, it will primarily act as a shear wall-frame interactive system. The interaction takes place between the interior shear wall tube and the walls of the framed tube parallel to the direction of the lateral force. The overturning moments in the exterior tube will cause additional deflection due to column shortening.

It is stated in [4], that for approximate analysis of the shear wall-frame interaction type behaviour of this system, one may use the influence charts developed for interaction of shear walls and frames.

6.0 ANALYSIS OF EXAMPLE FRAMED TUBE, TUBE IN TUBE AND SHEAR WALL-FRAME BUILDINGS

6.1 General

The aim of analysing the sample buildings in this section is to demonstrate the economic advantage of using framed tube structural systems for tall buildings. As it is mentioned in the previous sections framed tube or tube in tube structural system are advantageous over the conventional shear-wall frame system in many ways. By using framed tube or tube-in-tube structural systems, in a concrete building, a 10 to 12m wide column free space can be obtained. This is a very important architectural requirement nowadays, in which the demand for column free wide space in office buildings is increasing.

The closely spaced column system has the additional advantage of replacing the vertical mullions for the support of the glass windows. Thus, the elimination of the traditional curtain wall with its metallic mullions was in itself the justification for choosing these structural systems [4].

In this section analysis of hypothetical framed tube, tube-in-tube and shear-wall frame buildings is carried out. The sample buildings consist of 6 framed tube, 1 tube in tube and 6 shear-wall frame buildings.

It is well known that as a building becomes taller the lateral sway controls the design of the structural system. Hence, the provision of adequate stiffness, particularly lateral stiffness, is a major consideration in the design of a tall building [1,4].

Accordingly, this section investigates the different structural systems, which are used in the sample buildings based on their stiffness for limiting lateral sway.

The simple parameter drift index, defined as the ratio of the maximum deflection at the top of a building to the total height, is used in the sample buildings to estimate their lateral stiffness. The inter story drift index is also considered to check the possibility of localized excessive deformation.

Although the establishment of a drift index depends on many factors, a value of 0,002 is adopted in all the example buildings for reasons of simplicity.

6.2 Sample Analysis

6.2.1 Preliminary Design of Framed Tube Buildings using Influence Curves & Reduction Modelling Technique

As it is discussed in sections 5.2 and 5.3, a relatively accurate preliminary design of framed tube buildings having any number of stories can be carried out, using the influence curves and reduction modelling technique developed by Dr. Fazlur Rahman Khan.

In this section a hypothetical framed tube 40 stories high and 39 x 54m in plan with columns at 3m center to center and 4m floor to floor height is considered. The aim of this example is to compare the approximate method with an exact computer analysis results.

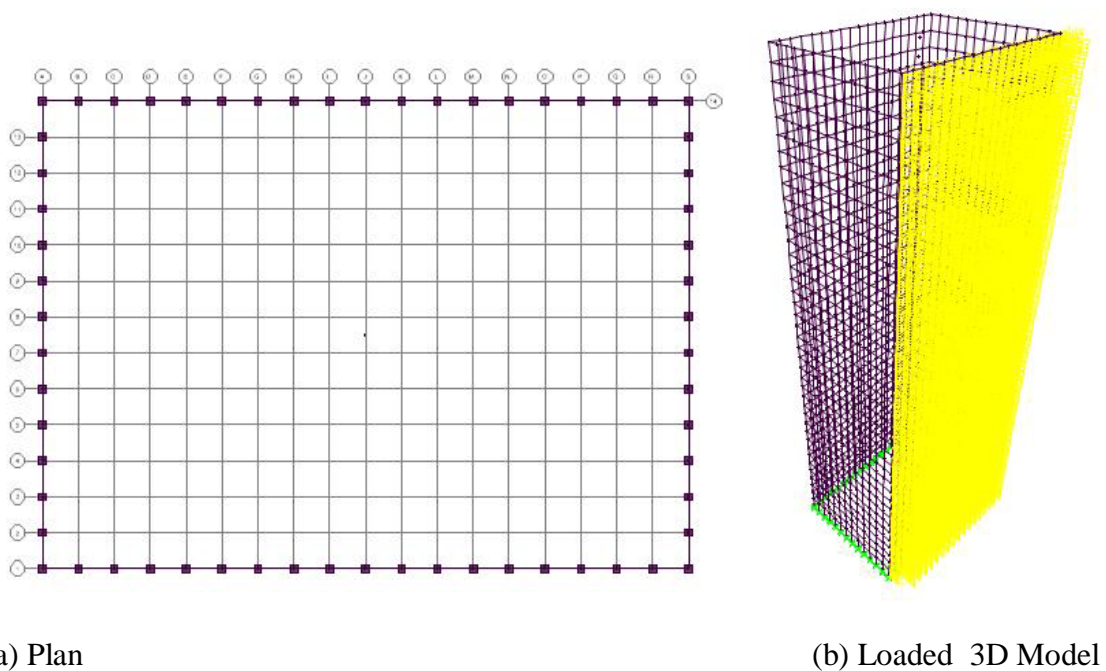


Fig. 6.1 Plan & loaded 3D Model of Example Framed Tube Building

The influence curves are plotted based on the analysis of framed tubes for uniform lateral load, hence the framed tube in this example will be analysed using a uniform wind pressure of $2.0\text{KN}/\text{M}^2$. It should be noted here that, this wind pressure is not an actual one, it is taken only to illustrate the approximate method under consideration.

Before using the influence curves, the basic parameters of the framed tube building are calculated. These basic parameters include aspect ratio, average stiffness factor, average

stiffness ratio and frame deflection. The deflection and forces in columns for an ideal tube of the same aspect ratio must be known.

Basic section properties and parameters are calculated and presented in table 6.1.

The moment of inertia of the building is calculated using the formula,

$$I = A_{ic} C_i^2$$

Where, A_{ci} = Area of column

C_i = distance of column to the neutral axis

Inserting values of A_{ic} and C_i into the above formula, the moment of inertia I of the building at different levels is computed and presented in table 6.2. The overturning moment of the building is also computed and presented in the same table.

Table 6.1 Section Properties of Example buildings

Floor	Column Properties		Spandrel Beam Properties		Stiffness Factor $S_f = S_b/S_c$	$S_{f10} = S_f(N/10)^2$	Stiffness Ratio = K_c/K_b
	Area cm^2	M. of I. cm^4	Area cm^2	M. of I. cm^4			
1-4	6400	3413333	9300	18619375	0.52	8.32	0.7
5-8	6400	3413333	9300	18619375	0.52	8.32	0.7
9-12	6400	3413333	9300	18619375	0.52	8.32	0.7
13-16	6400	3413333	9300	18619375	0.52	8.32	0.7
17-20	4900	2000833	6975	13964531	0.507	8.11	0.783
21-24	4900	2000833	6975	13964531	0.507	8.11	0.783
25-28	4900	2000833	6975	13964531	0.507	8.11	0.783
29-32	3600	1080000	5425	10861302	0.536	8.6	0.842
33-36	3600	1080000	5425	10861302	0.536	8.6	0.842
37-40	3600	1080000	5425	10861302	0.536	8.6	0.842

Where: Shear stiffness S_b , Axial stiffness S_c , and the 10-story equivalent stiffness factor S_{f10} are as defined in sections 5.2 and 5.3.

Average Stiffness Factor for ten-story model = 8.341

Average Stiffness Ratio = 0.77

Aspect Ratio = $5400/3900 = 1.4$

For this example use Stiffness Factor = 10

And Aspect Ratio = 1.5

Table 6.2 Moment of Inertia & Overturning Moment of Building

Floor	1	4	8	12	16	20	24	28	32	36
Overturning Moment, KN-M	1382400	1119744	884736	677376	497664	345600	221184	124416	55296	13824
Tubular Moment of Inertia, m ⁴	9856	8663	8663	8663	8663	6507	6507	5544	5544	5544

Column axial force at level 1, $(M \cdot C/I) \times A_C = 1382400 \cdot 19.5 \cdot 0.64 / 9856 = 1750 \text{KN}$

The actual forces in columns and deflection of the system can be calculated using the influence curves. For this building, Fig. 5.6, SF = 10 curve is used for computing the column axial forces in first story columns of a 10-story model. The above results are presented in table 6.3

Table 6.3 Column Axial Forces

Flange Column Axial Forces (KN)	
	Using Influence Curves
0	2275
1	2100
2	1750
3	1575
4	1400
5	1230
6	1134
7	1134
8	1134
9	1087
TOTAL	14819

The deflection of the total system is the sum of the frame deflection plus the ideal tube deflection times the magnification factor. Frame deflection for this example is approximately 5.2 cm. The cantilever deflection is computed as 11.2cm. The magnification factor is defined as the ratio of the sum of the ideal tube column forces to the actual column forces of the flange columns. For this example, the sum of the ideal tube column forces is, $10 \cdot 1750 = 17500 \text{KN}$ and the sum of the actual column forces is 14819KN. Therefore, the magnification factor will be 1.18.

Total deflection = $5.2 + (11.2 * 1.18) = 18.42\text{cm}$

Total deflection actual = 17.5cm. (Computer analysis result).

Total Column axial force = 14081KN (computer analysis result).

Comparing the values of the approximate method with an exact computer analysis shows that the use of influence curves gives results with an accuracy of about 90 % and is fairly reliable for preliminary design.

6.2.2 Analysis of Sample Framed Tube and Shear-wall Frame Buildings using ETABS (Extended Three Dimensional Analysis of Building Systems)

6.2.2.1 General

For the analysis of the sample buildings a lateral load due to earthquake and vertical loads are considered. It is assumed that the buildings are to be located in high seismic zone, which is zone 4 according to EBCS-8. A subsoil class A is also adopted to obtain the site coefficient S. Both equivalent static and dynamic analysis methods are used in the example buildings. For the dynamic analysis a design response spectrum is developed based on EBCS-8 as follows.

The seismic base shear can be obtained as,

$$F_b = S_d(T_1) W. \text{-----}[12]$$

Where $S_d(T_1)$ is ordinate of the design spectrum at period T_1 , and is defined by the following expression:

$$S_d(T_1) = \alpha \beta \gamma, \quad \alpha = \alpha_o I$$

α_o = The bedrock acceleration for the specific site

β = The design spectrum factor

S = Site coefficient for soil characteristics

T = Fundamental period of vibration in seconds

I = Importance factor

γ = Behaviour factor

For the example buildings under consideration the values of α_o , S, I and γ are taken to be as follows:

$$\alpha_o = 0.1 \text{-----}[12]$$

$$I = 1.4$$

Then α can be obtained from,

$$\alpha = \alpha_o I = 0.1 * 1.4 = 0.14$$

The behaviour factor γ can be obtained from:

$$\gamma = \gamma_o * K_D * K_R * K_W < 0.70$$

Where, γ_o = base value of the behaviour factor,

K_D = factor reflecting the ductility class

K_R = factor reflecting the structural regularity in elevation

K_W = factor reflecting the prevailing failure in structural system with walls

Values of γ_o , K_D , K_R and K_W , which are used in the analysis of the example buildings, are:

$$\gamma_o = 0.2, K_D = 2, K_R = 1, K_W = 1$$

Then γ is calculated as : $\gamma = 0.2 * 2 * 1 * 1 = 0.4$

Finally, $S_d(T_1)$ is obtained as:

$$S_d(T_1) = \alpha\beta\gamma = 0.4 * 0.14 * \beta = 0.0576 * \beta$$

The design spectrum is presented in table 6.4

Table 6.4 Design Spectrum

Normalized Response Spectra								
Soil Type A								
Time in Sec.	β	$\alpha\beta\gamma$	Time in Sec.	β	$\alpha\beta\gamma$	Time in Sec.	β	$\alpha\beta\gamma$
0	1	0,0560	1	1	0,0560	2	0,5	0,0280
0,05	1,75	0,0980	1,05	0,9524	0,0533	2,05	0,4878	0,0273
0,1	2,5	0,1400	1,1	0,9091	0,0509	2,1	0,4762	0,0267
0,15	2,5	0,1400	1,15	0,8696	0,0487	2,15	0,4651	0,0260
0,2	2,5	0,1400	1,2	0,8333	0,0467	2,2	0,4545	0,0255
0,25	2,5	0,1400	1,25	0,8	0,0448	2,25	0,4444	0,0249
0,3	2,5	0,1400	1,3	0,7692	0,0431	2,3	0,4348	0,0243
0,35	2,5	0,1400	1,35	0,7407	0,0415	2,35	0,4255	0,0238
0,4	2,5	0,1400	1,4	0,7143	0,0400	2,4	0,4167	0,0233
0,45	2,2222	0,1244	1,45	0,6897	0,0386	2,45	0,4082	0,0229
0,5	2	0,1120	1,5	0,6667	0,0373	2,5	0,4	0,0224
0,55	1,8182	0,1018	1,55	0,6452	0,0361	2,55	0,3922	0,0220
0,6	1,6667	0,0933	1,6	0,625	0,0350	2,6	0,3846	0,0215
0,65	1,5385	0,0862	1,65	0,6061	0,0339	2,65	0,3774	0,0211
0,7	1,4286	0,0800	1,7	0,5882	0,0329	2,7	0,3704	0,0207
0,75	1,3333	0,0747	1,75	0,5714	0,0320	2,75	0,3636	0,0204
0,8	1,25	0,0700	1,8	0,5556	0,0311	2,8	0,3571	0,0200
0,85	1,1765	0,0659	1,85	0,5405	0,0303	2,85	0,3509	0,0196
0,9	1,1111	0,0622	1,9	0,5263	0,0295	2,9	0,3448	0,0193
0,95	1,0526	0,0589	1,95	0,5128	0,0287	2,95	0,3390	0,0190
1	1	0,0560				3	0,3333	0,0187

The equivalent static analysis is carried out to compare the results with that of dynamic analysis method. The sample building plans, 3D models and the loads, to which they are subjected, are given in sections 6.2.2.2 to 6.2.2.14

6.2.2.2 Framed-Tube Building 1

In this example building a hypothetical framed tube (Fig 6.2), 25 stories high and 22mx22m in plan with column spacing 2.0m and 3.8m floor to floor height is considered. Using the expression on page 37 the base shear F_b is calculated as,

$$F_b = S_d(T_1)W$$

$$S_d(T_1) = \alpha\beta\gamma \quad \text{and} \quad \beta = 1.2 * S/T^{2/3}$$

Where T is the fundamental period of vibration of the structure for translational motion in the direction considered [12], and is approximated from $T = C_1 H^{3/4}$

For the building under consideration the natural period is obtained to be 2.28 sec.

Although the limit of the natural period to use the equivalent static method is 2 sec. or less, the analysis is carried out further to see the result and compare with that of the dynamic analysis result.

The design spectrum factor β is found from the above equation as follows:

$$\begin{aligned} \beta &= 1.2 * S/T^{2/3} \\ &= (1.2 * 1)/(2.28)^{2/3} \\ &= 0.693 < 2.5 \end{aligned}$$

The behaviour factor $\gamma = 0.4$ and the importance factor $\alpha = 0.14$ as calculated on page 38 and 39 respectively.

Then $S_d(T_1)$ will be:

$$S_d(T_1) = \alpha\beta\gamma = 0.12 * 0.693 * 0.6 = 0.05$$

For reasons of simplicity an average dead load of 10KN/m² is assumed. The total building weight will then be:

$$\text{Total building weight, } W = 25 * 10 * 22 * 22 = 121000 \text{KN}$$

The base shear force F_b is obtained using the previous expression, $F_b = S_d(T_1)W$.

$$F_b = 0.05 * 121000 = 6050 \text{KN}$$

The concentrated force at the top is determined as, $F_t = 0.07 * T * F_b = 965 \text{KN}$

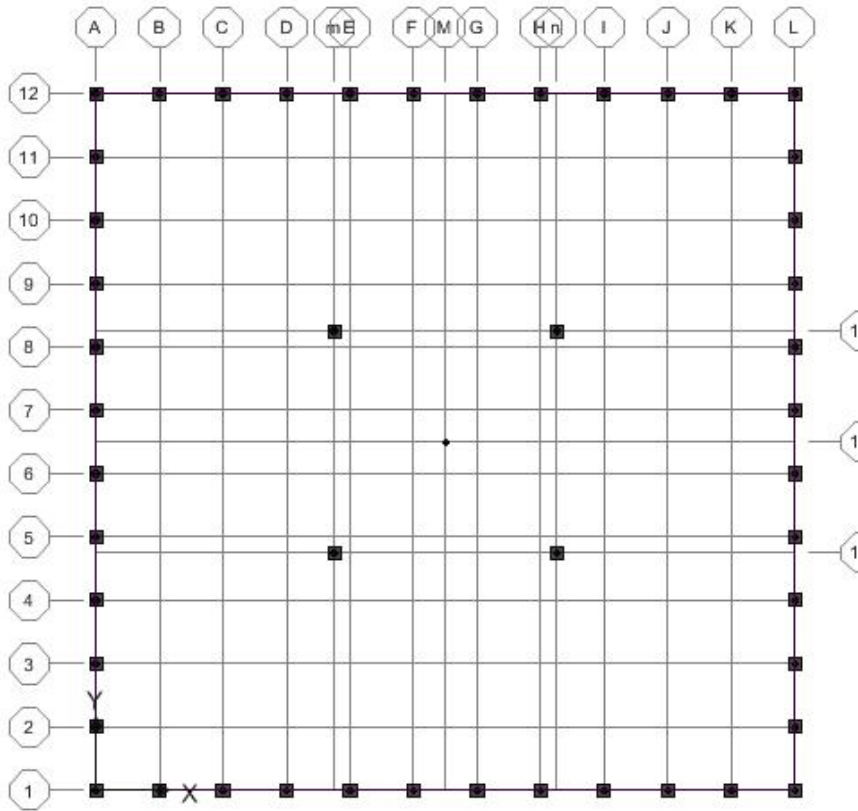
The remaining portion of the lateral force is distributed over the height of the building according to the following formula,

$$F_i = (F_b - F_t) * w_i h_i / \sum w_i h_i$$

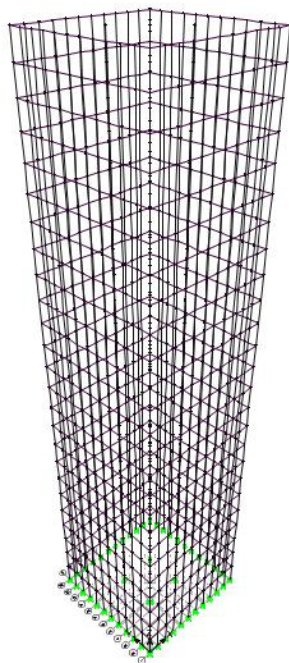
The distribution of the lateral force over the height of the building is presented in table 6.5.

Table 6.5 Distribution of lateral force over the height of the building

Floor level	$w_i h_i$	$F_i = (F_b - F_t) * w_i h_i / \sum w_i h_i$
25	45980000	1356
24	44140800	376
23	42301600	360
22	40462400	344
21	38623200	329
20	36784000	313
19	34944800	297
18	33105600	282
17	31266400	266
16	29427200	250
15	27588000	235
14	25748800	219
13	23909600	203
12	22070400	188
11	20231200	172
10	18392000	156
9	16552800	141
8	14713600	125
7	12874400	110
6	11035200	94
5	9196000	78
4	7356800	63
3	5517600	47
2	3678400	31
1	1839200	16
		6050KN

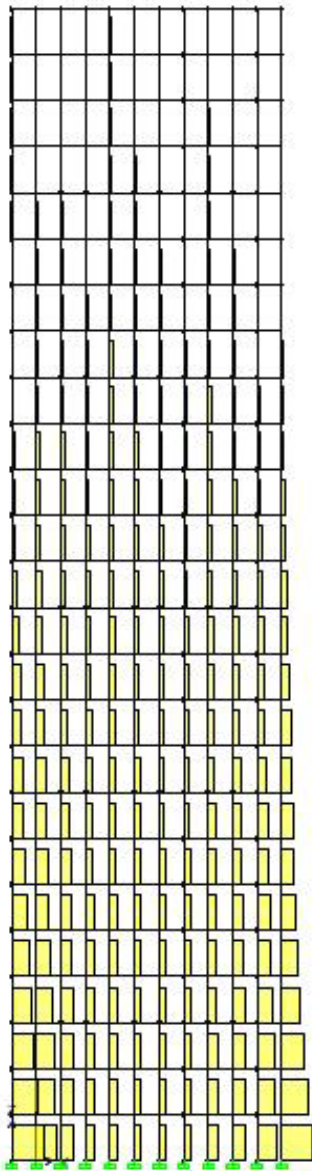


(a) Framed Tube Building 1 on Plan



(b) 3D Model

Fig. 6.2 Plan & 3D Model of framed building - 1



(a) column axial force in flange frame



(b) Column axial force in web frame

Fig. 6.3 Column axial force diag. of Framed-Tube building - 1

6.2.2.3 Shear Wall –Frame Building 1

In this example a hypothetical shear-wall frame building (Fig. 6.4 & 6.5), 25 stories high and 22m x 22m on plan and 3.8m floor to floor height is used. The plan dimension, number of stories as well as the floor height are kept the same as tubed frame 1, in order to compare the effectiveness of their structural systems. Both structural systems are subjected to the same lateral earthquake forces.

The permissible lateral deflection for both systems is: $0.002 \times 95 = 0.19\text{m}$

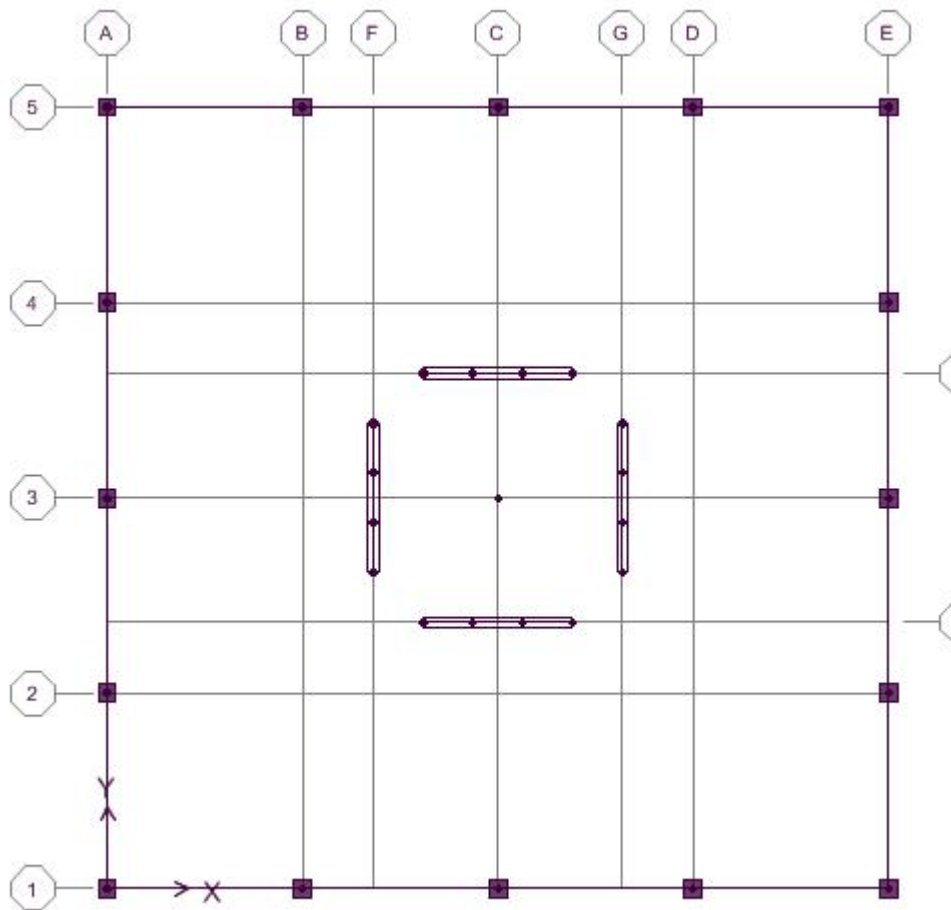
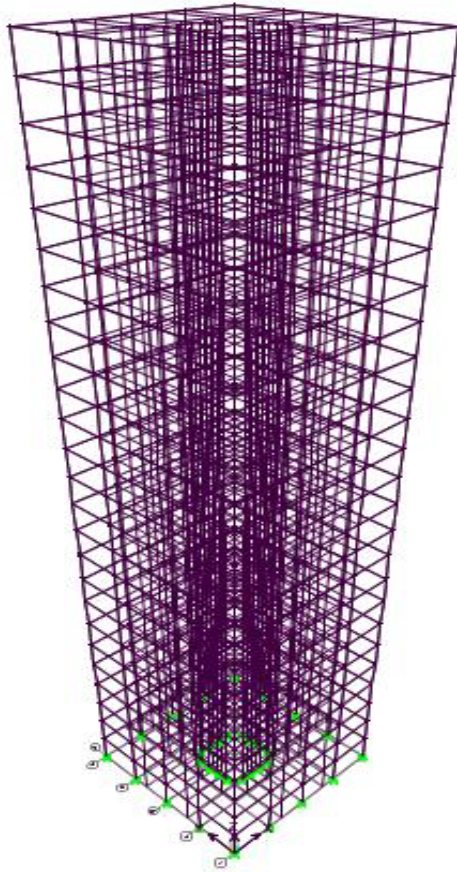


Fig 6.4 Shear-wall frame 1 on plan



(b) 3D-Model

Fig. 6.5. 3D Model of Shearwall-frame 1

6.2.2.4 Framed-Tube Building-2

Here a hypothetical framed tube 30 stories high and 33.6m x 33.6m in plan (Fig. 6.6 & 6.7) with column spacing 2.4m and 3.8m floor-to-floor height is considered.

The lateral load due to earthquake is calculated as follows.

Total building weight, $W=30 \times 10 \times 33,6 \times 33,6$

$$=338688\text{KN} \text{ (assuming the building wt. to be } 10\text{KN/m}^2 \text{ per floor)}$$

$$F_b=S_d(T_1)W$$

For the building under consideration the natural period is obtained to be 2.28 sec

$\gamma=0,6$ from the previous calculation

$$\beta=1.2 \times S/T^{2/3} =0.6311$$

$$S_d(T_1) = \alpha\beta\gamma =0.12 \times 0.6 \times 0.6311=0.04544$$

Finally. the base shear will be:

$$F_b=0,0454 \times 338688=15390\text{KN}$$

This base shear is distributed over the height of the building as presented in table 6.6

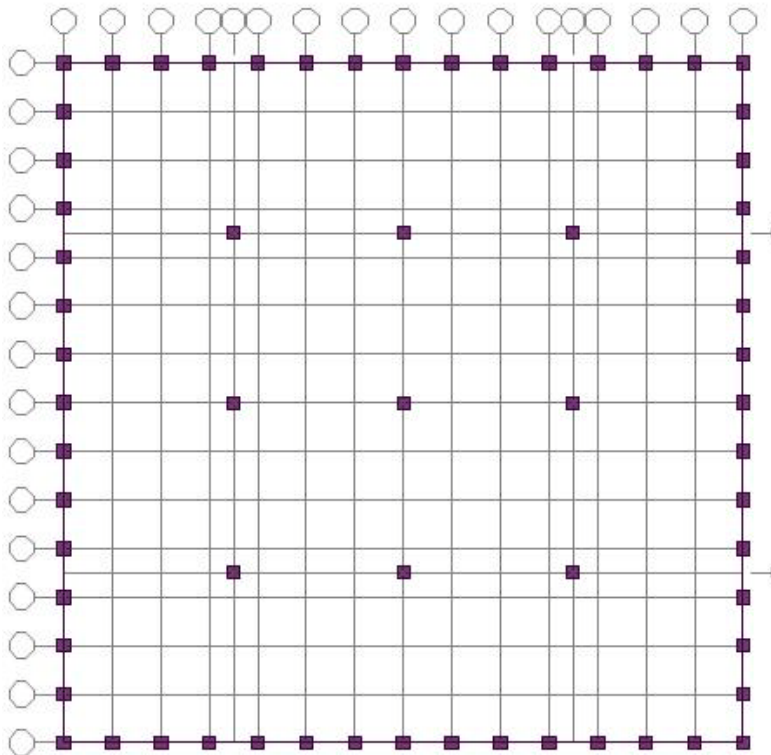
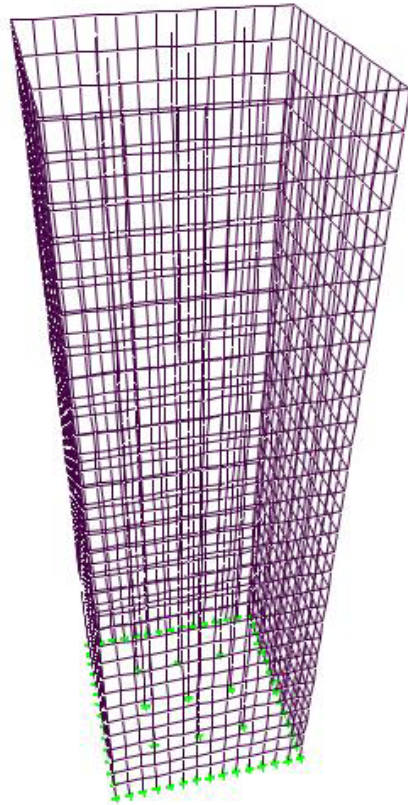


Fig. 6.6 Framed-Tube building 2 on plan

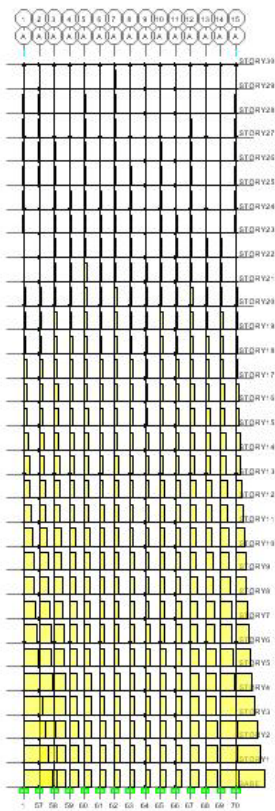
The permissible deflection of the top of this building is $0,002*114=0,228\text{m}$. The required stiffness is provided to this building during the analysis to limit the deflection below the permissible value.

Table 6.6 Distribution of lateral force over the height of the building

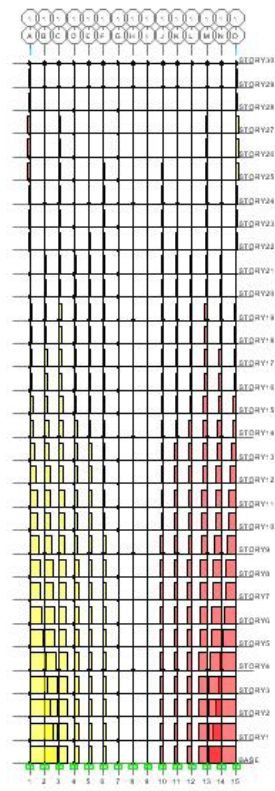
Floor level	$w_i h_i$	$F_i=(F_b-F_t)*w_i h_i / \sum w_i h_i$
30	1287014	3290
29	1244114	807
28	1201213	779
27	1158313	751
26	1115412	723
25	1072512	695
24	1029612	668
23	986711	640
22	943811	612
21	900910	584
20	858010	556
19	815109	528
18	772209	501
17	729308	473
16	686408	445
15	643507.00	417
14	600607	389
13	557706	362
12	514806	334
11	471905	306
10	429005	278
9	386104	250
8	343204	223
7	300303	195
6	257403	167
5	214502	139
4	171602	111
3	128701	83
2	85801	56
1	42901	28
	19948723	15390



(a) 3D Model



(b) column axial force in flange frame



(c) Column axial force in web frame

Fig. 6.7 3D Model and column axial force diag. of Framed Tube building 2

6.2.2.5 Shear-Wall Frame Building 2

In this example a hypothetical shear-wall frame building (Fig. 6.8 & 6.9), 30 stories high and 33,6m x 33,6m on plan and 3.8m floor to floor height is used. The plan dimensions, number of stories as well as the floor height are kept the same as tubed frame 2, in order to compare the effectiveness of their structural systems. Both structural systems are subjected to the same lateral earthquake forces.

The permissible lateral deflection for both systems is: $0.002 * 114 = 0.228\text{m}$

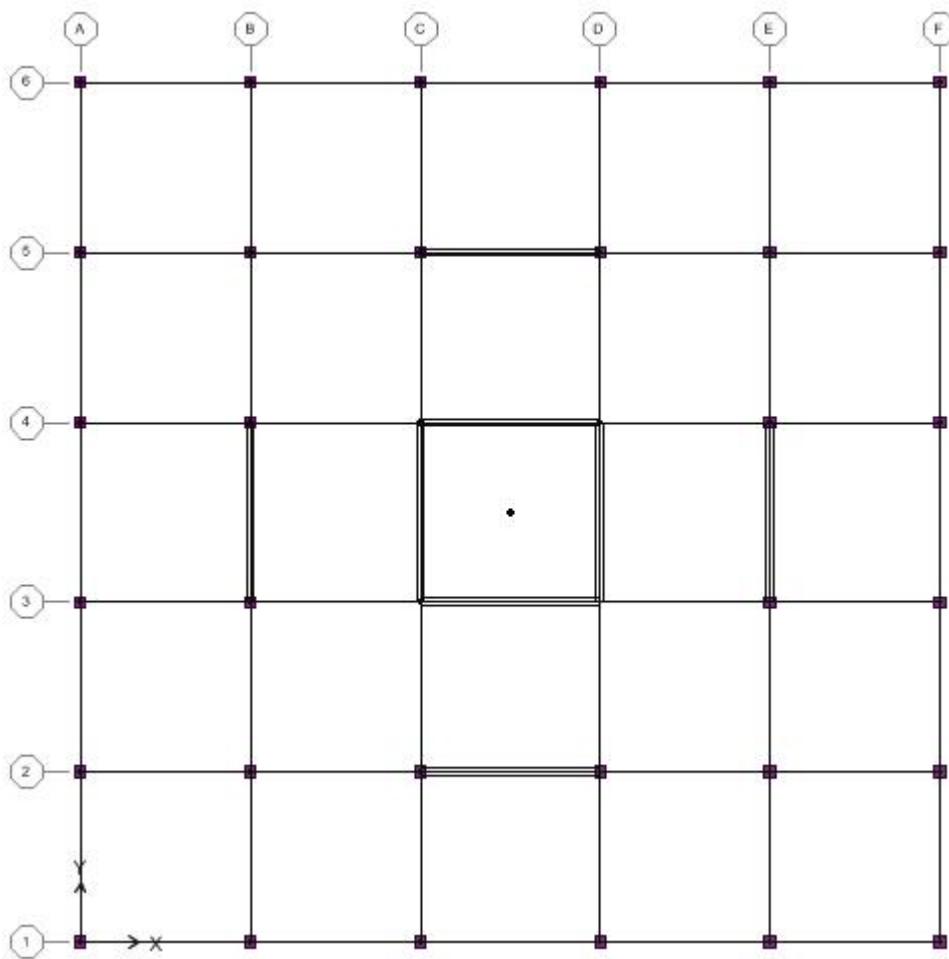


Fig. 6.8 Shear-wall frame building 2 on plan

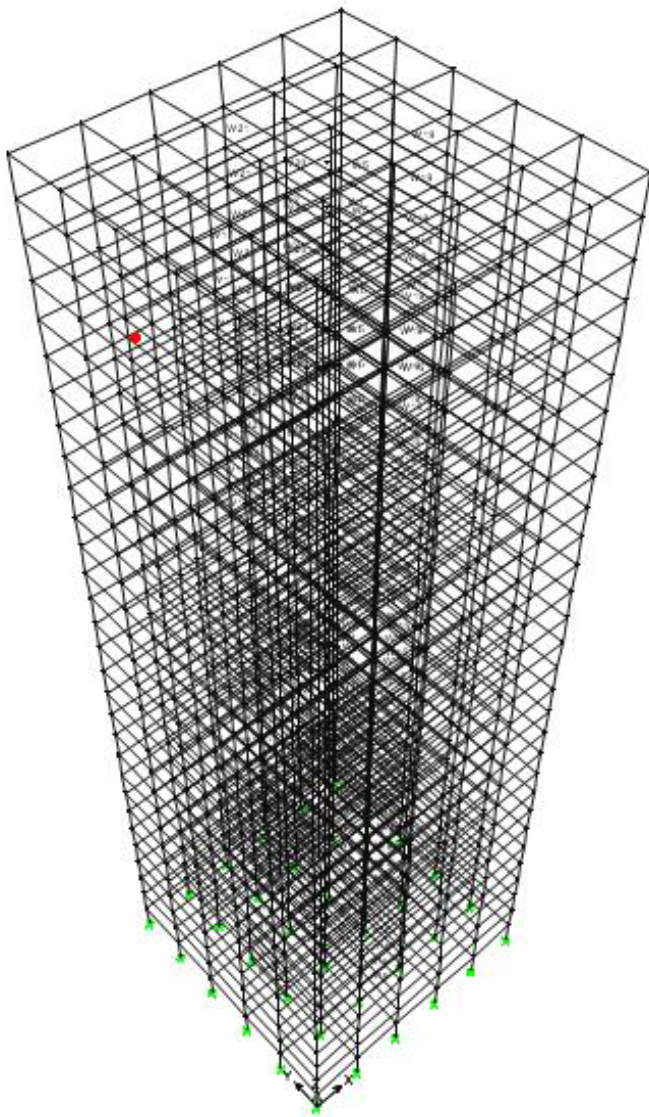


Fig. 6.9 3D Model of Shear-wall frame Building-2

6.2.2.6 Framed- Tube Building 3

In this example a hypothetical framed tube 35 stories high and 36mx24m in plan with column spacing 2m and 3.8m floor-to-floor height (Fig 6.10 & 11) is considered.

The lateral load due to earthquake is calculated as follows.

Total building weight, $W=35*10*36*24$

$$=302400\text{KN} \quad (\text{assuming the building wt. to be } 10\text{KN/m}^2 \text{ per floor})$$

$$F_b = S_d(T_1)W$$

The natural period of the building under consideration is 2.94sec.

$\gamma=0,6$ from the previous calculation

$$\beta=1.2*S/T^{2/3}$$

$$=0.5847$$

$$S_d(T_1) = \alpha\beta\gamma = 0.12*0.6*0.5847=0.0421$$

Finally. the base shear will be:

$$F_b=0,0421*302400=12731\text{KN}$$

This base shear is distributed over the height of the building and presented in table 6.7

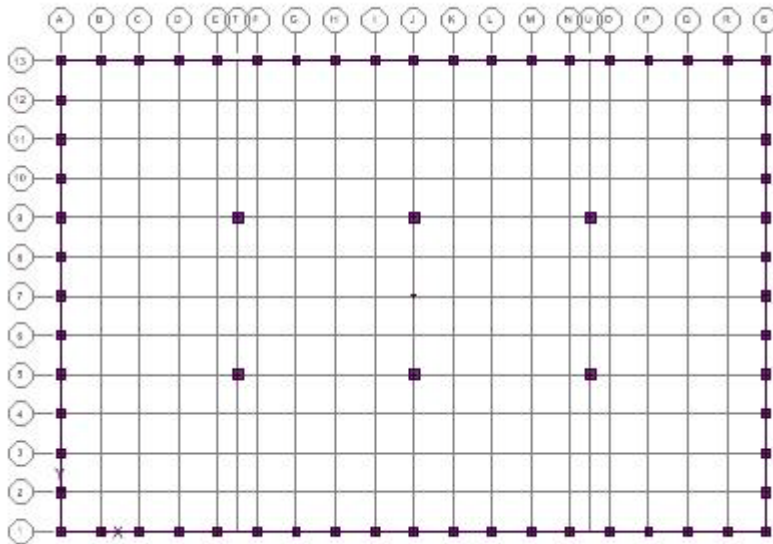
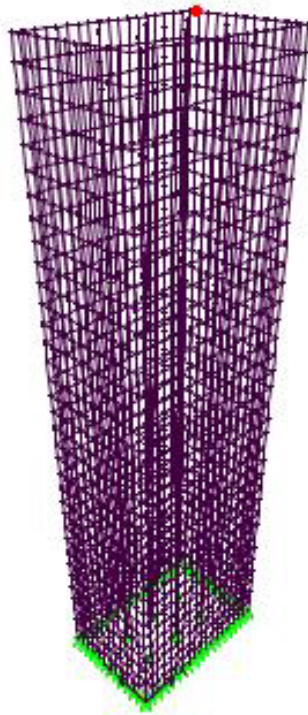


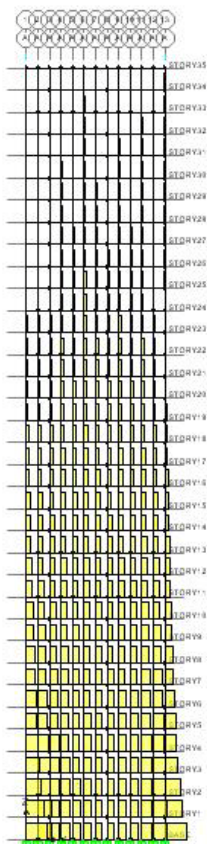
Fig. 6.10 Framed Tube building-3 on Plan

Table 6.7 Distribution of lateral force over the height of the building

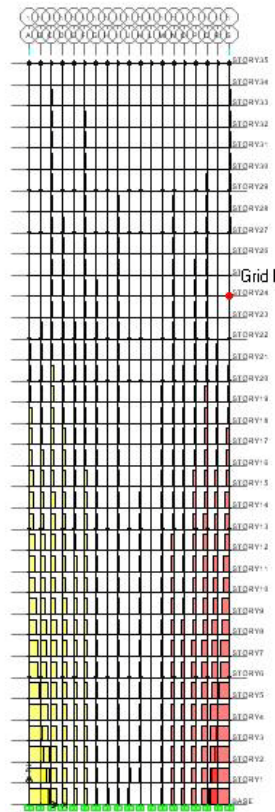
Floor level	$w_i h_i$	$F_i = (F_b - F_t) * w_i h_i / \sum w_i h_i$
35	1149120	3182
34	1116288	546
33	1083456	530
32	1050624	514
31	1017792	498
30	984960	481
29	952128	465
28	919296	449
27	886464	433
26	853632	417
25	820800	401
24	787968	385
23	755136	369
22	722304	353
21	689472	337
20	656640	321
19	623808	305
18	590976	289
17	558144	273
16	525312	257
15	492480	241
14	459648	225
13	426816	209
12	393984	193
11	361152	177
10	328320	160
9	295488	144
8	262656	128
7	229824	112
6	196992	96
5	164160	80
4	131328	64
3	98496	48
2	65664	32
1	32832	16
	20684160	12731



(a) 3D-Model



(b) column axial force in flange frame



(c) Column axial force in web frame

Fig. 6.11 3D Model of Framed tube 3

6.2.2.7 Shear-Wall Building 3

In this example a hypothetical shear-wall frame building (Fig. 6.12 & 13), 35 stories high and 36m x 24m on plan and 3.8m floor to floor height is used. The plan dimension, number of stories as well as the floor height are kept the same as tubed frame 3, in order to compare the effectiveness of their structural systems. Both structural systems are subjected to the same lateral earthquake forces.

The permissible lateral deflection for both systems is: $0.002 \times 133 = 0.266\text{m}$

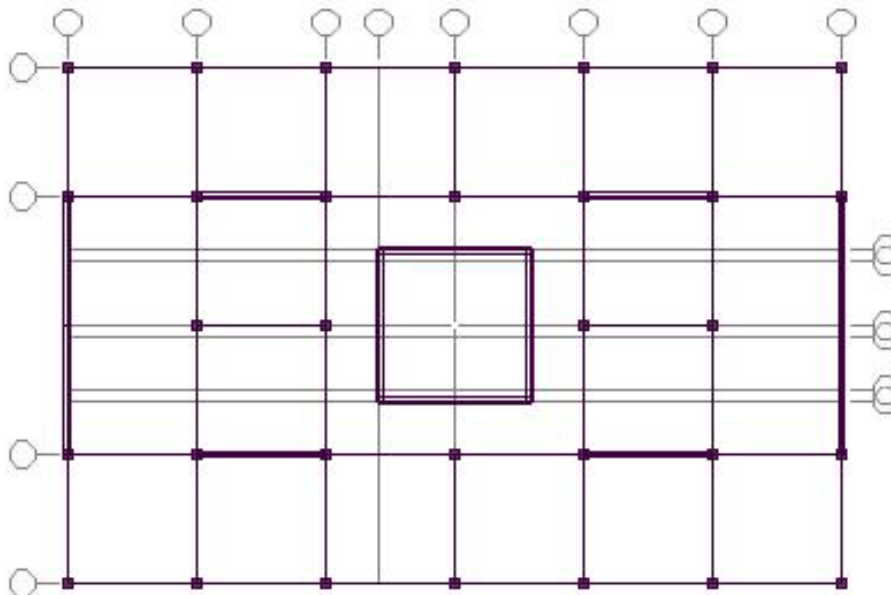


Fig. 6.12 Shear-wall frame building 3 on plan

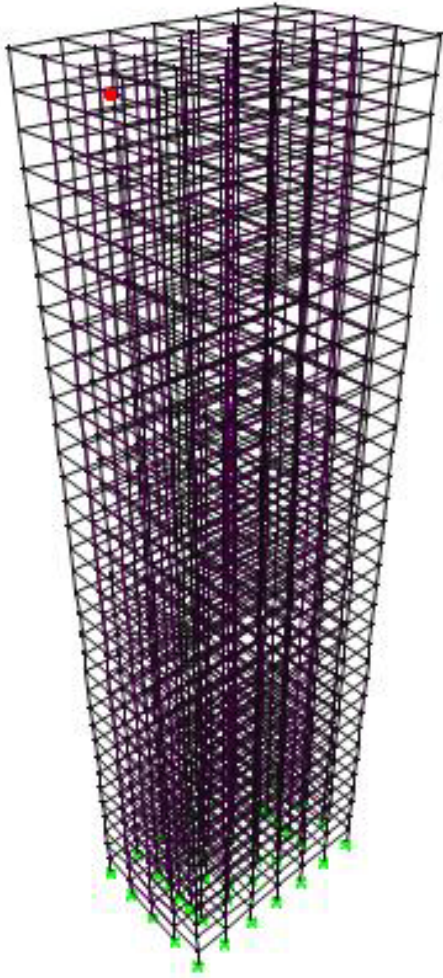


Fig. 6.13 3D Model of Shearwall-frame Building-3

6.2.2.8 Framed-Tube Building 4

In this example a hypothetical framed tube 40 stories high and 63mx63m in plan with column spacing 2.1m and 3.5m floor to floor height (Fig 6.14 and 6.15) is considered.

The lateral load due to earthquake is calculated as follows.

Total building weight, $W=40*10*63*63$

$$=1587600\text{KN} \quad (\text{assuming the building wt. to be } 10\text{KN/m}^2 \text{ per floor})$$

$$F_b=S_d(T_1)W$$

The natural period of the building is 3.0525sec.

$\gamma=0,6$ from the previous calculation

$$\beta=1.2*S/T^{2/3} =0.55$$

$$S_d(T_1) = \alpha\beta\gamma =0.12*0.6*0.55=0.04$$

Finally. the base shear will be:

$$F_b=0,04*1587600=62560\text{KN}$$

This base shear is distributed over the height of the building as presented in table 6.8

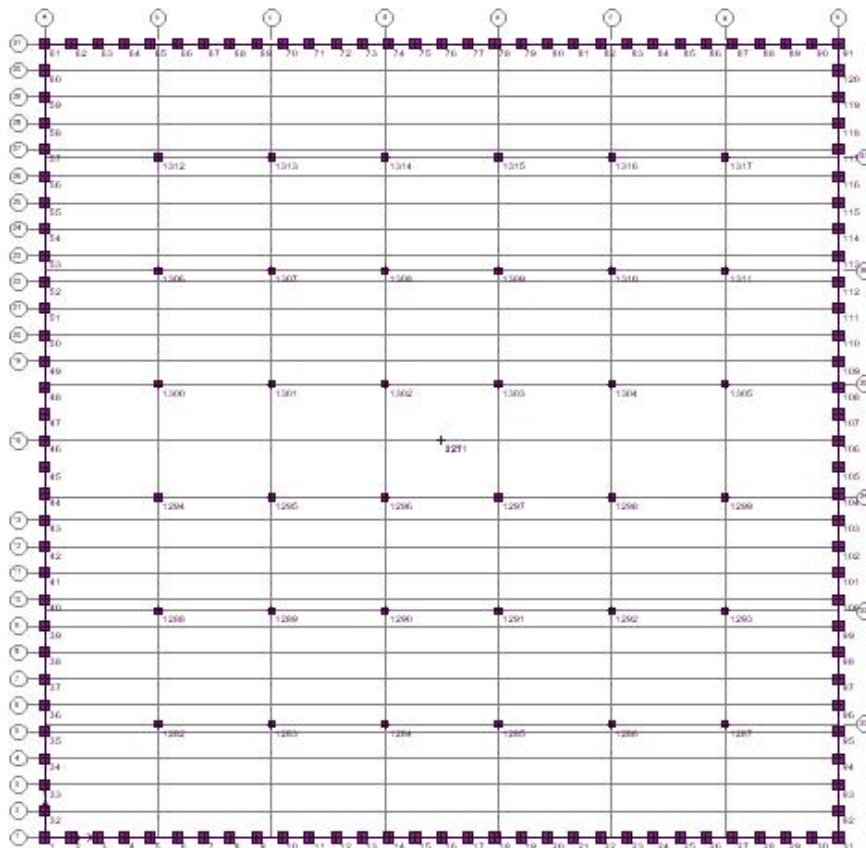
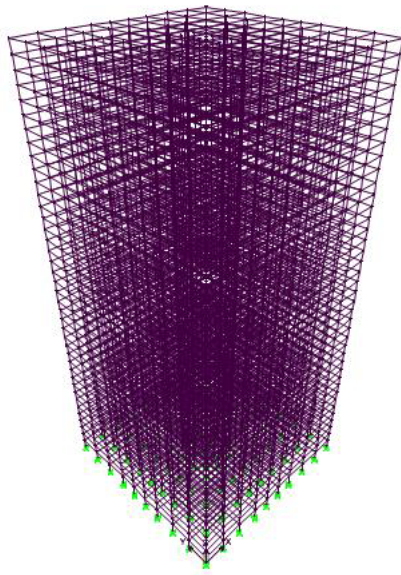


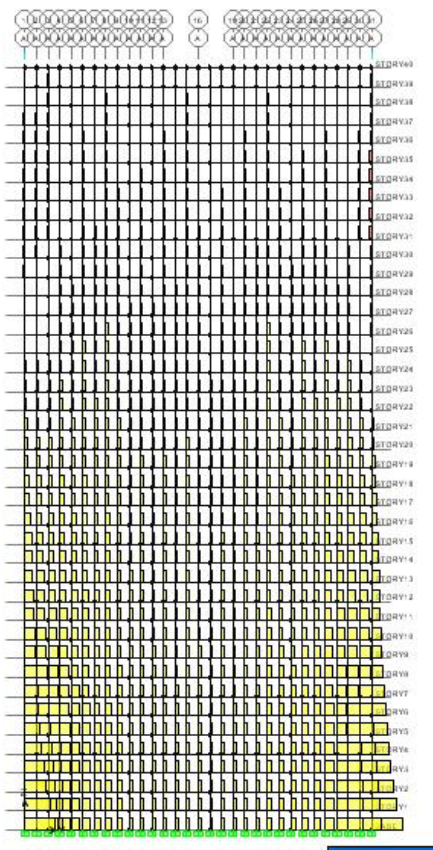
Fig. 6.14 Framed Tube building-4 on Plan

Table 6.8 Distribution of lateral force over the height of the building

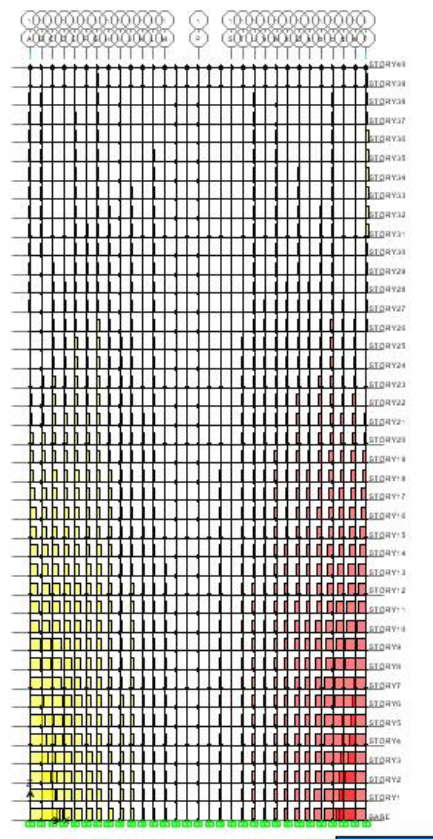
Floor level	$w_i h_i$	$F_i = (F_b - F_t) * w_i h_i / \sum w_i h_i$
40	5556600	15768
39	5417685	2340
38	5278770	2280
37	5139855	2220
36	5000940	2160
35	4862025	2100
34	4723110	2040
33	4584195	1980
32	4445280	1920
31	4306365	1860
30	4167450	1800
29	4028535	1740
28	3889620	1680
27	3750705	1620
26	3611790	1560
25	3472875	1500
24	3333960	1440
23	3195045	1380
22	3056130	1320
21	2917215	1260
20	2778300	1200
19	2639385	1140
18	2500470	1080
17	2361555	1020
16	2222640	960
15	2083725	900
14	1944810	840
13	1805895	780
12	1666980	720
11	1528065	660
10	1389150	600
9	1250235	540
8	1111320	480
7	972405	420
6	833490	360
5	694575	300
4	555660	240
3	416745	180
2	277830	120
1	138915	60
	113910300	62560



(a) 3D-Model



(b) column axial force in flange frame



(c) Column axial force in web frame

Fig. 6.15 3D Model and column axial force diag. of Framed-Tube 4

6.2.2.9 Shear-Wall Frame Building 4

In this example a hypothetical shear-wall frame building (Fig. 6.16 & 6.17), 40 stories high and 63m x 63m on plan and 3.5m floor to floor height is used. The plan dimension, number of stories as well as the floor height are kept the same as tubed frame 4, in order to compare the effectiveness of their structural systems. Both structural systems are subjected to the same lateral earthquake forces.

The permissible lateral deflection for both systems is: $0.002 \times 140 = 0.28\text{m}$

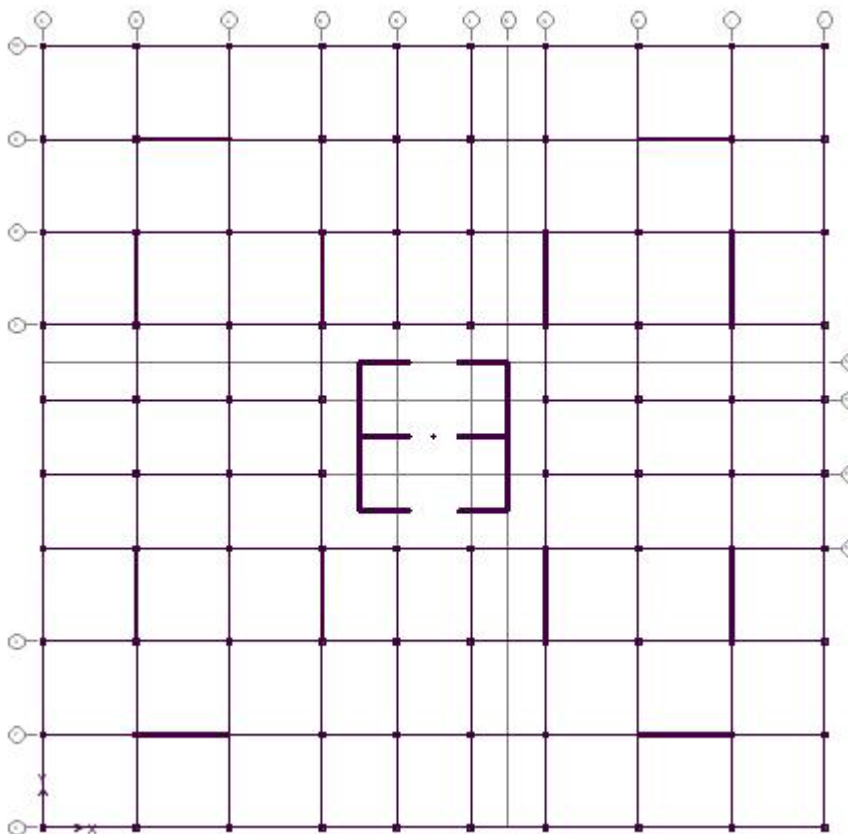


Fig. 6.16 Shear-wall frame building-4 on plan

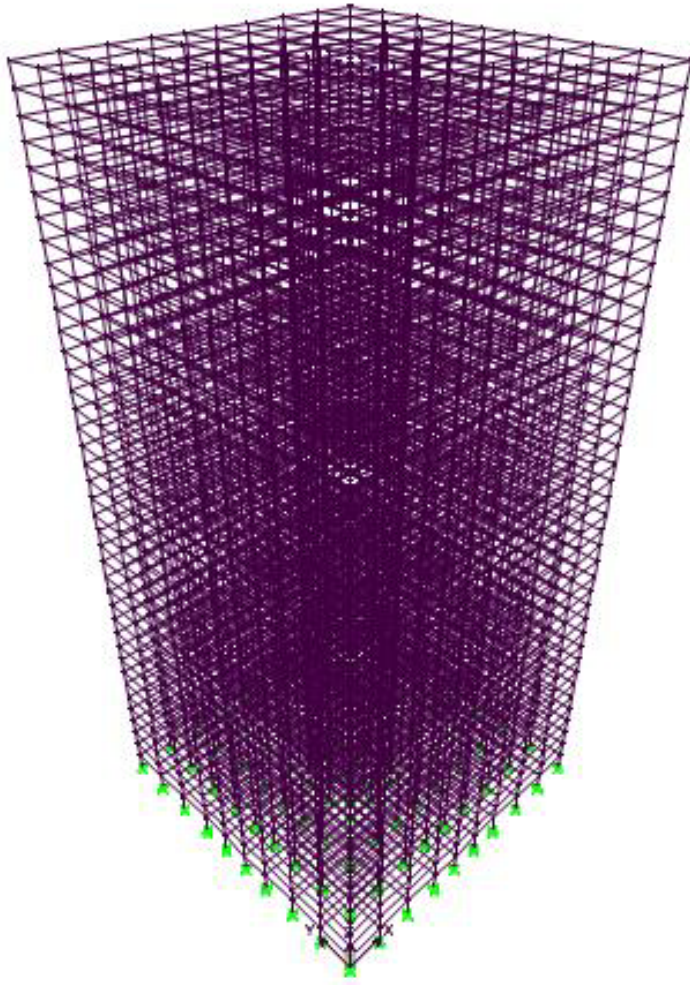


Fig. 6.17 3D Model of Shear wall-frame Building-4

6.2.2.10 Framed-Tube Building 5

In this example a hypothetical framed tube 43 stories high and 38.5m x 24.5m in plan with column spacing 1.75m and 3.7m floor to floor height (Fig 6.18 & 6.19) is considered.

The lateral load due to earthquake is calculated as follows.

Total building weight, $W=43 \times 10 \times 38,5 \times 24,5$

$$=405598\text{KN} \quad (\text{assuming the building wt. to be } 10\text{KN/m}^2)$$

$$F_b = S_d(T_1)W$$

The natural period of the building is found to be 3.36sec.

$\gamma=0,6$ from the previous calculation

$$\beta = 1.2 * S / T^{2/3}$$

$$=0.5276$$

$$S_d(T_1) = \alpha\beta\gamma = 0.12 * 0.6 * 0.5276 = 0.038$$

Finally. the base shear will be:

$$F_b = 0,038 * 405598 = 15407\text{KN}$$

This base shear is distributed over the height of the building as presented in table 6.9

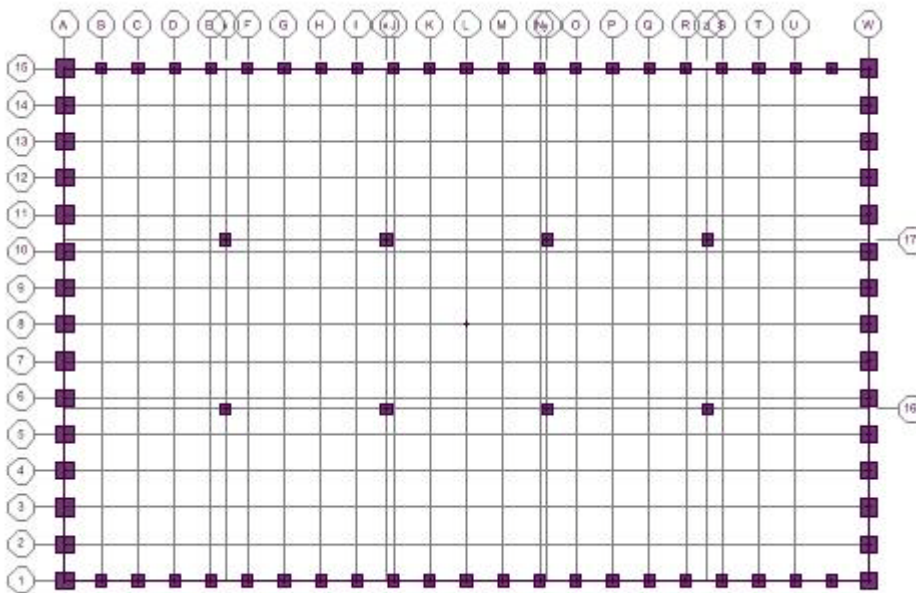
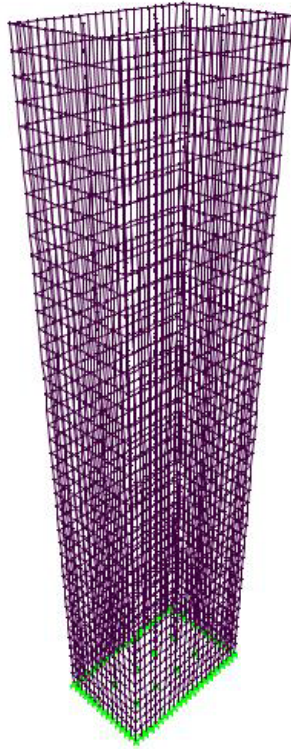


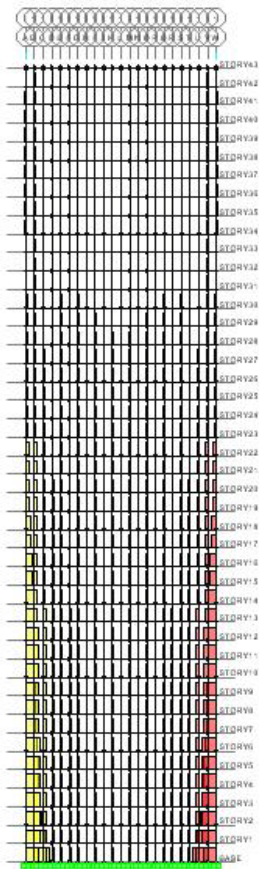
Fig. 6.18 Framed-Tube building-5 on Plan

Table 6.9 Distribution of lateral force over the height of the building

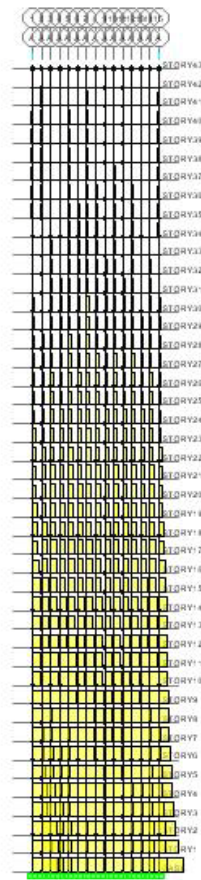
Floor level	$w_i h_i$	$F_i = (F_b - F_t) * w_i h_i / \sum w_i h_i$
43	1500710	4160
42	1465810	523
41	1430910	511
40	1396010	498
39	1361109	486
38	1326209	473
37	1291309	461
36	1256409	448
35	1221508	436
34	1186608	423
33	1151708	411
32	1116808	399
31	1081907	386
30	1047007	374
29	1012107	361
28	977207	349
27	942306	336
26	907406	324
25	872506	311
24	837606	299
23	802705	286
22	767805	274
21	732905	262
20	698005	249
19	663104	237
18	628204	224
17	593304	212
16	558404	199
15	523503	187
14	488603	174
13	453703	162
12	418803	149
11	383902	137
10	349002	125
9	314102	112
8	279202	100
7	244301	87
6	209401	75
5	174501	62
4	139601	50
3	104700	37
2	69800	25
1	34900	12
	33015620	15407



(a) 3D Model



(b) column axial force in flange frame



(c) Column axial force in web frame

Fig. 6.19 3D Model and column axial force diag. of Framed-tube building-5

6.2.2.11 Shear-Wall Frame Building 5

In this example a hypothetical shear-wall frame building (Fig. 6.20 & 6.21), 43 stories high and 38.5m x 24m on plan and 3.7m floor to floor height is used. The plan dimension, number of stories as well as the floor height are kept the same as tubed frame 5, in order to compare the effectiveness of their structural systems. Both structural systems are subjected to the same lateral earthquake forces.

The permissible lateral deflection for both systems is: $0.002 * 159.1 = 0.32\text{m}$

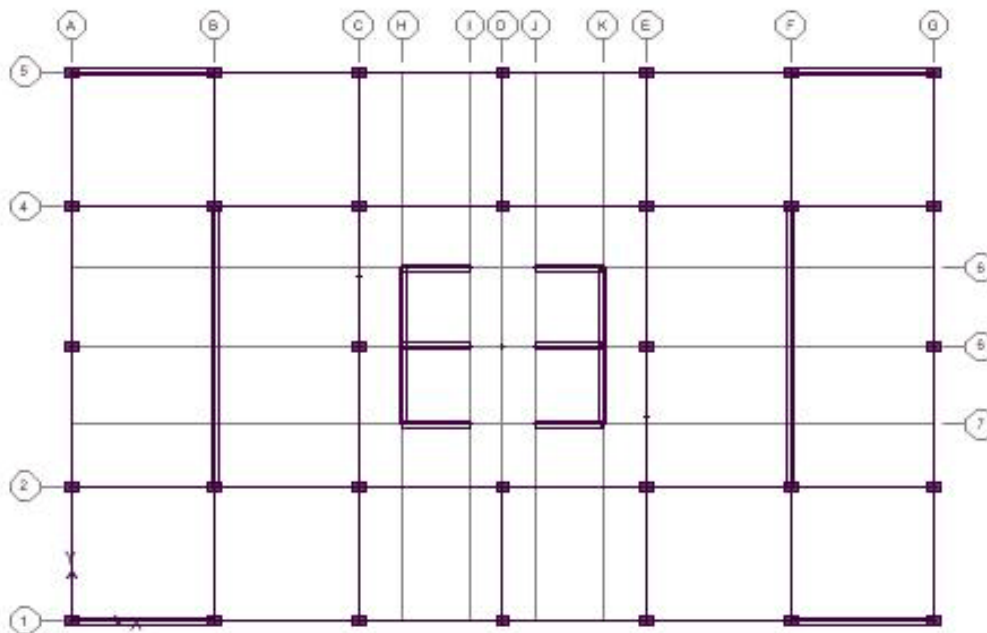


Fig. 6.20 Shear-wall frame building 5 on plan

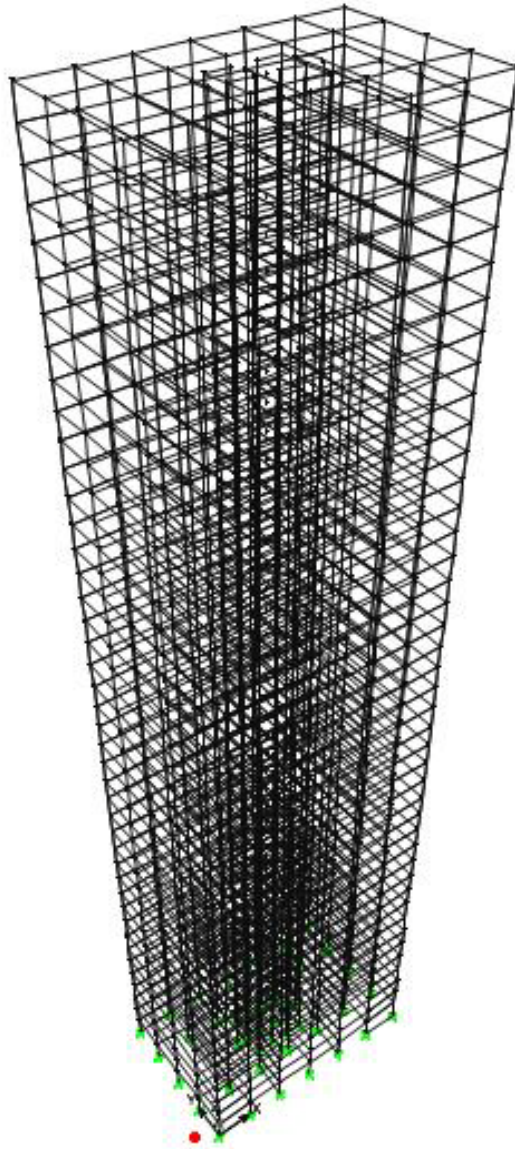


Fig. 6.21 3D Model of Shearwall-frame Building-5

6.2.2.12 Framed -Tube Building 6

In this example a hypothetical framed tube 50 stories high and 54mx39m in plan with column spacing 2m and 3.6m floor to floor height (Fig. 6.22 & 6.23) is considered.

The lateral load due to earthquake is calculated as follows.

Total building weight, $W=50*10*54*39$

$$=1053000\text{KN} \quad (\text{assuming the building wt. to be } 10\text{KN/m}^2 \text{ per floor})$$

$$F_b=S_d(T_1)W$$

The natural period of the building is obtained to be 3.68sec.

$\gamma=0,6$ from the previous calculation

$$\beta=1.2*S/T^{2/3}$$

$$=0.503$$

$$S_d(T_1) = \alpha\beta\gamma = 0.12*0.6*0.503=0.0362$$

Finally. the base shear will be:

$$F_b=0,0362*1053000=38119\text{KN}$$

This base shear is distributed over the height of the building as presented in table 6.10

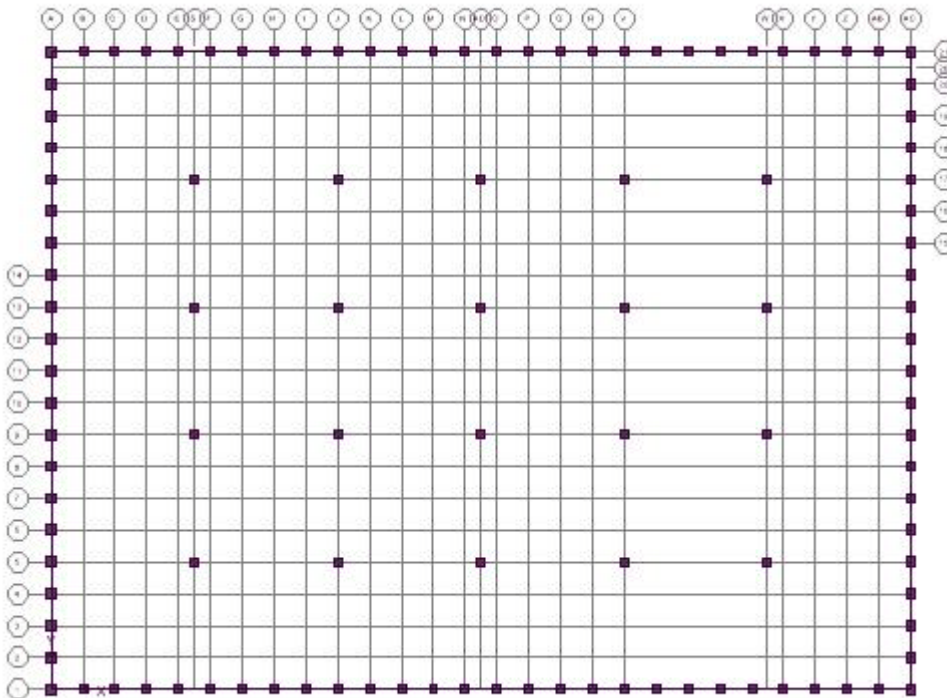


Fig. 6.22 Framed-Tube building-6 on Plan

Table 6.10 Distribution of lateral force over the height of the building

Floor level	$W_i h_i$	$F_i = (F_b - F_t) * W_i h_i / \sum W_i h_i$
50	3790800	10929
49	3714984	1088
48	3639168	1065
47	3563352	1043
46	3487536	1021
45	3411720	999
44	3335904	977
43	3260088	954
42	3184272	932
41	3108456	910
40	3032640	888
39	2956824	866
38	2881008	843
37	2805192	821
36	2729376	799
35	2653560	777
34	2577744	755
33	2501928	732
32	2426112	710
31	2350296	688
30	2274480	666
29	2198664	644
28	2122848	621
27	2047032	599
26	1971216	577
25	1895400	555
24	1819584	533
23	1743768	511
22	1667952	488
21	1592136	466
20	1516320	444
19	1440504	422
18	1364688	400
17	1288872	377
16	1213056	355
15	1137240	333
14	1061424	311
13	985608	289
12	909792	266
11	833976	244
10	758160	222
9	682344	200
8	606528	178
7	530712	155
6	454896	133
5	379080	111
4	303264	89
3	227448	67
2	151632	44
1	75816	22
	96665400	38119

6.2.2.13 Shear-Wall Frame Building 6

In this example a hypothetical shear-wall frame building (Fig. 6.24 & 6.25), 50 stories high and 54m x 39m on plan and 3.6m floor to floor height is used. The plan dimension, number of stories as well as the floor height are kept the same as tubed frame 6, in order to compare the effectiveness of their structural systems. Both structural systems are subjected to the same lateral earthquake forces.

The permissible lateral deflection for both systems is: $0.002 \times 180 = 0.36\text{m}$

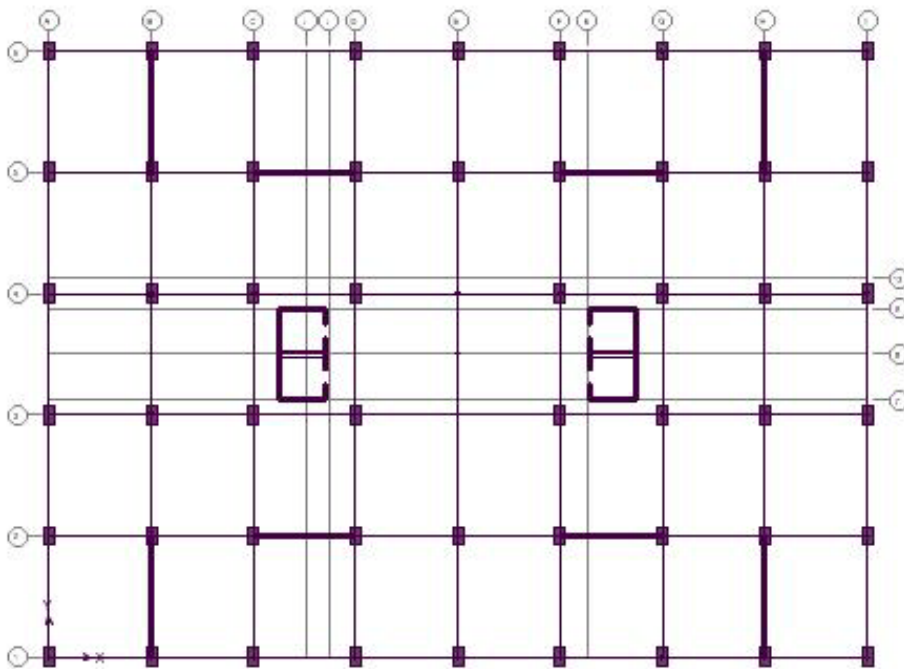


Fig. 6.24 Shear-wall frame building-6 on plan

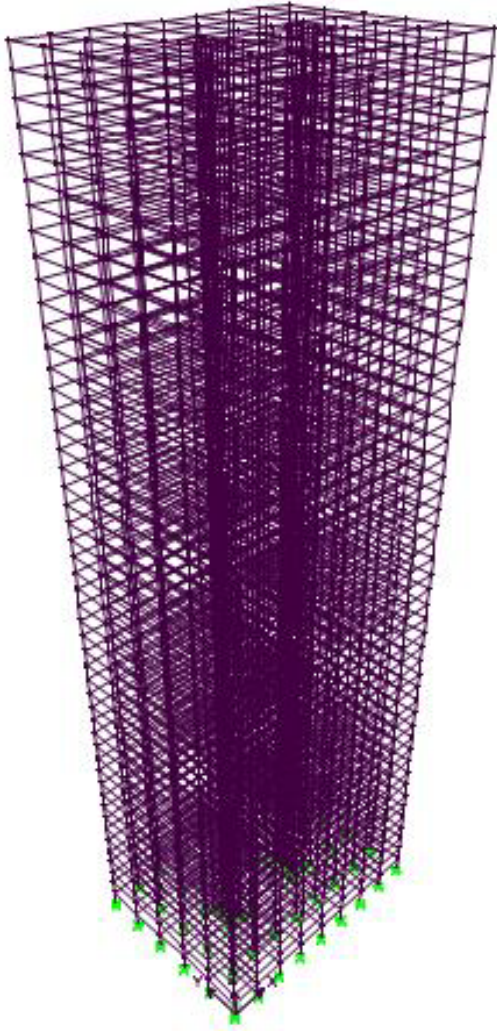


Fig. 6.25 3D Model of Shear wall-frame Building-6

6.2.2.14 Tube-in-tube building-7

In this example a hypothetical tube in tube building (Fig. 6.26 & 6.27), 50 stories high and 54m x 39m on plan and 3.6m floor to floor height is used. The plan dimension, number of stories as well as the floor height are kept the same as tubed frame 6, in order to compare the effectiveness of their structural systems. Both structural systems are subjected to the same lateral earthquake forces.

The permissible lateral deflection for both systems is: $0.002 * 180 = 0.36\text{m}$

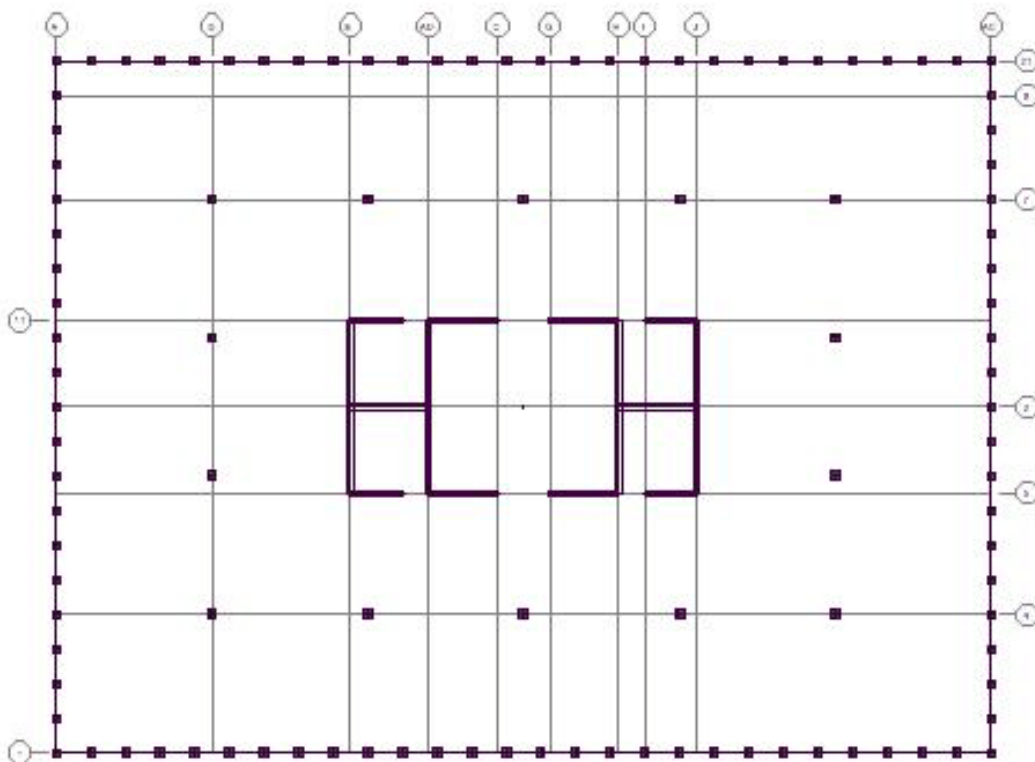
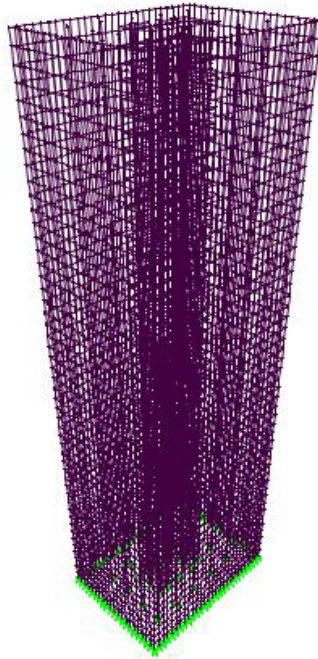
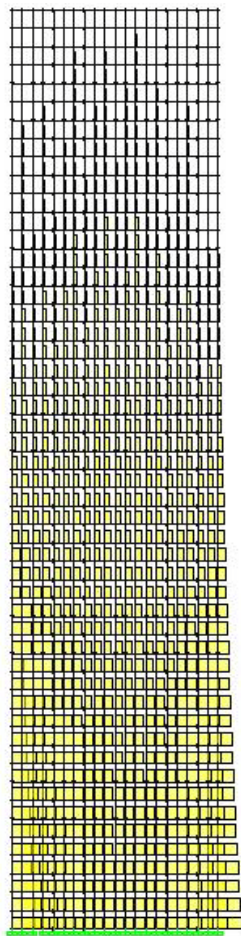


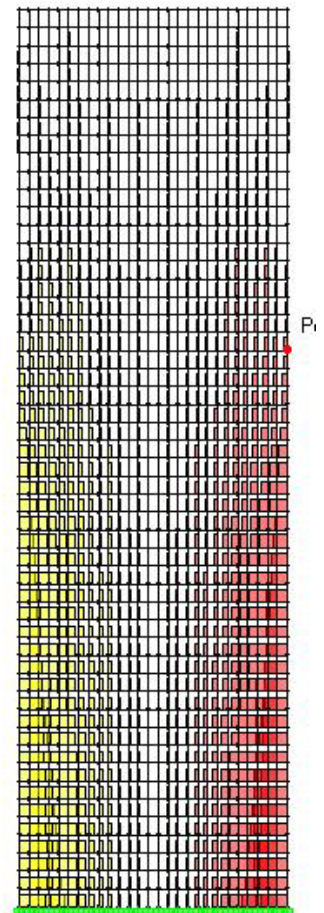
Fig. 6.26 Tube-in-tube building-7 on Plan



(a) 3D-Model



(b) column axial force in flange frame



(c) Column axial force in web frame

Fig. 6.27 3D Model & axial force diag. of tube-in-tube building 7

6.3 Discussion

The example buildings are analysed for lateral EQ & vertical loads as previously mentioned. The Equivalent static & dynamic analyses are carried out according to the following procedure.

The permissible lateral deflection was determined first for each building based on its given drift index and then, the required stiffness is provided to the system to limit the lateral deflection of the top of the building below the permissible value. This is done for each pair structural systems. If we take framed tube building-1 for example, it has the same aspect ratio and building height with shear-wall frame building-1. The difference between the two is the type of their structural system.

So, the aim of the analysis was to keep the lateral deflection of the two buildings similar, i.e below the permissible value. In this way one can compare the concrete required for each structural systems having the same aspect ratio and building height. For the shear-wall frame building structural system, different arrangements of shear walls have been tried and the final one, which is more practical and relatively economical is adopted.

After carrying out the analysis, the concrete requirement of framed tube and shear-wall frame buildings is investigated. The result shows that framed tube buildings require less amount of concrete than shear-wall frame buildings. It is also observed that the economic advantage becomes more significant as the building gets taller. Comparison of the framed tube and shear-wall frame buildings based on their concrete requirement is shown in table 6.14.

Comparative analysis based on stiffness degradation and P-delta effect is also made and presented in tables 6.11 to 6.14.

From the analysis it is also observed that the fundamental period obtained from the dynamic analysis is greater than that obtained from the equivalent static analysis and it becomes longer as the building height increases.

6.4 Evaluation of Analysis Results

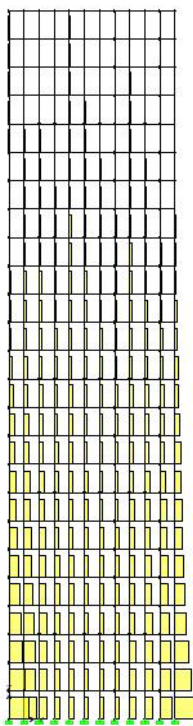
6.4.1 Shear Lag Effect in the Sample Framed Tube Buildings

As stated in the literatures, the shear lag effect results in non-linear pressure distribution along the column envelope, i.e. the columns at the corners of the building are forced to take a higher share of the load than the column in between.

This behaviour of the shear lag effect is observed also in the sample framed tube buildings.

The column axial forces are higher in the corner columns and their value becomes gradually smaller from the corner to the center of the sample building frames.

The column axial force diagrams of each framed tube building are shown in figures 6.3, 6.7, 6.11, 6.15, 6.19, 6.23 & 6.27. The column axial force diagram of framed tube-1 is shown below for illustration



(a) column axial force in flange frame



(b) Column axial force in web frame

Fig. 6.28 Shear lag effect in frames

6.4.2 P-delta Effect

The P-delta effect is also considered in the analysis of the sample buildings. Each sample building is analysed for two cases, i.e with and without P-delta effect. From the analysis result it is observed that, for a building having the same cross-sectional properties and subjected to the same lateral earthquake forces there is a considerable deflection difference.

The deflection of each building with and without P-delta effect is presented in table 6.11

Table 6.11 Deflections of sample buildings (in metre)

Building Type	Deflection with P-delta effect	Deflection w/out P-delta effect	Permissible deflection
Tubed Frame-1	0.1826	0.1504	0.19
Shear-wall frame bldg-1	0.1869	0.1636	0.19
Tubed Frame-2	0.227	0.211	0.228
Shear-wall frame bldg-2	0.2174	0.1874	0.228
Tubed Frame-3	0.266	0.2209	0.266
Shear-wall frame bldg-3	0.258	0.2124	0.266
Tubed Frame-4	0.282	0.2611	0.28
Shear-wall frame bldg-4	0.2726	0.2253	0.28
Tubed Frame-5	0.321	0.3055	0.32
Shear-wall frame bldg-5	0.3105	0.2557	0.32
Tubed Frame-6	0.3573	0.2983	0.36
Shear-wall frame bldg-6	0.3542	0.2721	0.36

From the above table it can be observed that the P-delta is critical for both type of structural systems.

6.4.3 Stiffness Degradation

The stiffness degradation due to cracking in the sample buildings has been taken into consideration during the analysis of the sample buildings. In order to show the effects of stiffness degradation, the cross-sectional properties of the sample building are presented in table 6.12 & 6.13

Each building is analysed two times keeping the same drift index, i.e with and without consideration of stiffness degradation

Table 6.12 Cross-Sectional Properties of Sample Buildings

Building Type	Column Cross-sections		Beam Cross-sections	
	with Stiff. Degr.	w/o stiff. Degr.	with stiff. Degr.	w/o stif. Degr.
Level	Area, cm ²	Area, cm ²	Area, cm ²	Area, cm ²
1 to 10	2025	1600	1875	1875
11 to 20	2025	1600	1875	1875
21 to 25	1600	1600	1875	1875
Shear-wall frame bldg-1				
level				
1 to 10	3025	2500	5500	3500
11 to 20	3025	2500	5500	3500
21 to 25	2500	2500	4500	3000
Shear wall thickness=30cm				
Tubed Frame-2				
Level				
1 to 10	5625	4225	5200	4800
11 to 20	4900	4225	5200	4800
21 to 30	4900	4225	5200	4800
Shear-wall frame bldg-2				
Level				
1 to 10	2025	1600	1950	1650
11 to 20	2025	1600	1950	1650
21 to 30	2025	1600	1625	1375
Shear wall thickness=30cm				
Tubed Frame-3				
Level				
1 to 15	3600	3025	3500	3300
16 to 30	3025	2500	3500	3300
31 to 35	3025	2025	3300	3300
Shear-wall frame bldg-3				
1 to 10	2025	1600	1750	1500
11 to 25	2025	1600	1750	1500
26 to 35	1600	1600	1750	1500
Shear wall thickness=30cm				

Table 6.13 Cross-Sectional Properties of Sample Buildings (continued)

Building Type	Column Cross-sections		Beam Cross-sections	
	with Stiff. Degr.	w/o stiff. Degr.	with stiff. Degr.	w/o stif. Degr.
Level	Area, cm ²	Area, cm ²	Area, cm ²	Area, cm ²
Tubed Frame-4				
1 to 15	6400	5625	7750	7250
16 to 29	6400	5625	7500	7000
30 to 40	6400	4900	7500	7000
Shear-wall frame bldg-4				
level				
1 to 15	3600	3025	3500	3150
16 to 30	3600	2500	3500	3150
31 to 40	3600	2025	2700	2400
Shear wall thickness=30cm				
Tubed Frame-5				
Level				
1 to 15	4225	3600	5600	4800
16 to 30	4225	3025	4200	4800
31 to 43	3600	2500	4200	3600
Shear-wall frame bldg-5				
Level				
1 to 15	4000	2600	4000	3200
16 to 30	4000	2600	4000	3200
31 to 43	3750	2400	4000	3200
Shear wall thickness=30cm				
Tubed Frame-6				
Level				
1 to 20	5625	4225	6000	5400
21 to 39	4550	4225	6000	5400
40 to 50	4225	3600	6000	5400
Shear-wall frame bldg-6				
1 to 20	9900	8400	5700	4800
21 to 40	9600	8400	5225	4400
41 to 50	9600	8400	4750	4000
Shear wall thickness=30cm				
Tube in Tube building -7				
level				
1 to 20	3600	3025	4500	4500
21 to 40	3600	3025	4500	4500
41 to 50	2500	1600	4500	4500

6.5 Comparison of Framed Tube and Frame-Shear Wall Buildings Based on Concrete Requirement

After the analysis is completed, the concrete requirement of each building to resist lateral load is calculated. The computed concrete requirements are based on dynamic analysis results including stiffness degradation and p-delta effect considerations. The results are presented in table 6.14

Table 6.14 Concrete requirement in m³

Building No.	Conc. requirement of sample buildings in m ³	
	Framed-tube	Frame-Shear wall
1	1284	2087
2	4709	5748
3	3727	5225
4	19572	26316
5	8108	11323
6	13019	20641

7. CONCLUSIONS AND RECOMMENDATIONS

In order to investigate the advantage of using framed tube and tube in tube structural systems for tall buildings, a comprehensive literature survey and analysis of sample buildings are performed.

In the literatures it is indicated that the economic height limit for concrete buildings, which were based on the regular beam-column frame or shear-wall frame type construction have progressively raised, due to the emergency of newer structural systems, such as tubed frame and tube in tube structural systems. By using tubed frame and tube in tube structural system, wide column free space can also be achieved.

From the analyses results of the sample buildings the following conclusions are made.

- The effect of shear lag on the tube action which was stated in the literatures, as it causes non-linear pressure distribution along the column envelope, is observed also in the sample framed tube buildings.
- P-delta effects and stiffness degradation should be taken into account, as they have much influence on the strength and stability of the structure.
- Framed tube buildings require less amount of concrete than shear-wall frame buildings and this is pronounced, as the building gets taller. This result conforms also to what is stated in the literatures
- A dynamic and equivalent static analysis methods were conducted on the sample buildings. From the results of the dynamic analysis, it was observed that the buildings have large fundamental periods, which means lower acceleration. While applying the equivalent static analysis method on the other hand, the fundamental period of the buildings are smaller and the accelerations are higher. This implies that using an equivalent static analysis method for tall buildings makes the structure unnecessarily stiffer and uneconomical. As it is also stated in [12], a dynamic analysis method should be applied for buildings having larger fundamental periods.
- It is recommended that architects have to familiarise themselves to these tubular structural systems as they have influence on the architectural expressions of the building.

REFERENCES

1. Smith, B. S. and Coull, A., „Tall Building Structures: Analysis and Design“, John Wiley & Sons, Inc., 1991
2. Prof. Dr. –Ing. G. König, Dr. –Ing. S.Liphardt, „Hochhäuser aus Stahlbeton“ , Ernst & Sohn, Sonderdruck aus dem Beton-Kalender 1985
- 3 Schueller, Wolfgang, „High-rise building Structures“, John Wiley & Sons, 1977
- 4 Fintel Mark, F., „Handbook of Concrete Engineering“, CBS Publishers & Distributers, 1986
- 5 A. Coull & E.stafford Smith, „Tall Buildings, with particular reference to shear wall structures“, Pergamon Press, 1967
- 6 Mir M. Ali, „Performance characteristics of tall framed tube buildings in seismic zones“, Elsevier Science Ltd,1996
7. Journal of Structural Engineering. Vol. 120, No. 9-11, 1994, P. 3105, „Negative shear lag in Framed tube structures“,
- 8 Journal of Structural Engineering vol. 120 Nos. 1-4, 1994, P 1221- „ Simple Method for Approximate Analysis of framed tube structures“
- 9 Prof. DDr.-Ing. Konard Bergmeister & Prof. Dr.-Ing. Johann-Dietrich Woerner, "Beton Kalender - Hochhaeser und Geschossbauten", Ernst & Sohn, 2003
- 10 Site URL
- 11 W. F. Chen, "The Civil Engineering Hand Book,", 1995
- 12 Ethiopian Building Code Standard-Design of Structures for Earthquake Resistance Ministry of Works & Urban Development e (EBCS-8 1995)
- 13 S.M.A. Kazini, R. Chandra, “Analysis of Shear-walled Buildings “

14. Macgregor, James, “ Reinforced Concrete Mechanics and Design”, Prentice Hall, Inc., 1997
15. ETABS Manual