STUDIES IN WIND LOAD- FRAME INTERACTION

A DISSERTATION

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

of

MASTER OF TECHNOLOGY

in

CIVIL ENGINEERING

(With Specialization in Structural Engineering)

By

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CANDIDATE'S DECLARATION

I here by declare that the work which is being presented in the dissertation entitled, " **STUDIES IN WIND LOAD – GRAVITY FRAME INTERACTION**" in partial fulfillment of the requirements for the award of the degree of **Master of Technology** in **Civil Engineering,** with **specialization in Structural Engineering,** submitted in the Department of Civil Engineering, Indian Institute of Technology, Roorkee, is an authentic record of my work carried out from August 2004 to June 2005 under the guidance and supervision of **Dr. J. Prasad**, Associate Professor, Department of Civil Engineering, Indian Institute of Technology, Roorkee .

The matter embodied in this dissertation has not been submitted by me for the award of any other degree or diploma.

Date: 30th June, 2005

Place: Roorkee

A. Jeevan
(ARRA JEEVAN)

CERTIFICATE

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

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Date: 30th June, 2005 (ARRA JEEVAN)

ABSTRACT

RC floor systems are generally framed into a system of RC columns to obtain multi-storey buildings. For medium rise buildings (up to 20-story about), framed RC columns are the main structural sub-system out of the many choices available for vertical systems. These vertical systems are required to resist lateral loads (Wind, Earthquake) along with the gravity loads from the floors. Although, the gravity loads constitute the main loads for the vertical system, from design point of view, the influence of lateral loads rapidly increases with the increase in number of stories (height). Appropriate incorporation of lateral loads in the analysis process, thus, becomes essential. A comparative study of various wind loading standards provides knowledge of the sophistication in wind load provisions and leads to a better understanding of the specifications. Analysis of frames needs extensive use of computer with a lot of pre-and-post processing of data. Approximate methods of analysis have proved to be quite useful for preliminary analysis of frames subjected to lateral loads. This dissertation primarily deals with the assessment of wind loads on frames followed by a study of the applicability of approximate analysis for plane frames of various characteristics. The study has been carried out using the computer programs specially written for this purpose. The programs have been developed in the Microsoft Visual Basic programming language.

Four different frames with varying number of stories and number of bays have been used for the study of wind loads as obtained by various wind loading codes. These frames, for a particular loading have been studied for comparison of results obtained from various methods of analysis. Relevant observations and conclusions arrived at are reported.

CONTENTS

CHAPTER 1

INTRODUCTION

For the structural engineer the major difference between low and tall buildings is the influence of wind forces on the behaviour of the structural elements. Generally, it can be stated that a tall building is one in which the horizontal loads (especially wind loads) are an important factor in the structural design. Advances in materials with improved confidence in analysing structures due to high speed digital computers have resulted in lighter and slender structures, which are vulnerable to lateral loads. The lateral deflections of a tall building, sized for gravity loads only (called "gravity frame"), will exceed due to additionally applied lateral loads.

1.1 Wind Loads:

Wind loading on structures has always been a nagging worry for engineers. Efforts in codes to put it into straightforward criteria as in the case of other loads have never been satisfactory. As a matter of fact, serious investigations about a particular aspect of wind loading or the behaviour of a particular type of structure under wind loads have always followed, never preceded structural failures. In the entire spectrum of structural engineering technology perhaps the area that has remained most elusive to successful codification is wind loading on structures. Wind loads on structures are determined by the satisfaction of the requirements of the code provisions within a country. National wind load standards are developed on the basis of research and prevalent professional practice in their respective countries and are formulated independently of each other. A comparative study of the standards provides knowledge of the sophistication in wind load provisions and leads to a better understanding of the specifications.

The recent revisions in the codes have been essentially propelled by the following two concepts:

(a) Statistical approach to the evaluation of wind behaviour and occurrence.

(b) Separation of dynamic component of wind loading and its interaction with the dynamic response of the structure.

These revisions recognize not only the improving state of understanding of the characteristics of wind but also the recent trends in structural design and conceptions. The present day structures have an ever increasing tendency towards forms which are exceedingly wind-sensitive because of their slenderness, flexibility, size and lightness. Added to this are the introductions of a broader range of materials and the subjection of the material to a higher range of stresses. All these factors have demanded a more realistic and more exact description of wind loading than was hitherto available.

1.2 Analysis of Tall Buildings for Lateral Loads

The analysis of tall buildings pertains to the determination of the influence of applied loads on forces and deformations in the individual structural elements such as beams, columns and walls. Both exact and approximate methods exist for the analysis of tall buildings for lateral loads. The exact analysis of a problem is obtained if the three conditions of equilibrium, compatibility and material characteristics are satisfied. There are two approaches for analysing a statically indeterminate structure. First method is 'force' or 'flexibility' or 'consistent deformations' method, in which the forces are assumed to be the basic unknowns and the deflections at these points are found assuming the supports to be absent. Then the forces necessary to cause the known displacements at those points are determined. This leads to a system of equations in which the number of equations equals the number of unknowns. Second method is 'stiffness' or 'displacement' method, in which deformations are assumed as the basic unknowns, satisfying compatibility criterion. The forces are expressed in terms of these unknowns and the equilibrium equations are used to determine the unknown deformations. This also leads to a number of equations equal to the number of unknown displacements.

1.3 Exact Analysis:

The "exact" analysis does not, in general, present the exact values of stresses and the structural capacity in limit state, for two reasons. Firstly, strength and workmanship are random variables represented by statics of their distributions in the analysis. Secondly, simplifying assumptions are necessary to make the problem tractable. The loading is idealized into a few cases with simple (often uniform) spatial distributions on the surface of the structure, influence of cladding is neglected; the three dimensional behaviour is often simplified into plane analysis; real stress - strain behaviour is idealized; and stress analysis is simplified. Exact analysis while merely a convention, is assumed well defined for any problem at hand, serving as a basis of comparison for approximate methods.

1.4 Approximate Analysis

Even in today's high-tech computer oriented world with all its sophisticated design capabilities, there still is a need to undertake approximate analysis of structures. First it provides a basis for selecting preliminary member sizes because the design of a structure, no matter how simple or complex begins with a tentative selection of members. Member sizes in tall buildings are arrived at from a trial-and-error procedure. With the preliminary sizes, an analysis is made to determine if design criteria are met. If not, an analysis of modified structure is made to improve its agreement with the requirement, and the process is continued until a design is obtained within the limits of acceptability.

Second, because of the ever-increasing cost of labour and building materials, it is almost mandatory for the structural engineer to compare several designs before choosing the one most likely to be the best from the point of view of structural economy. How well it minimizes the premium required by the mechanical, electrical, and curtain wall systems of the myriad structural systems which present themselves as possibilities, only two or three schemes may be worthy of further refinement requiring full-blown computer solutions. Approximate methods are all that may be required to logically arrive at cost figures and.-to sort out few final contenders from among the innumerable possibilities. It is very time consuming, costly, and indeed unnecessary to undertake a complete sophisticated analysis for all the possible schemes. Preliminary designs are very useful in weeding out the weak solutions.

Sophisticated computer analyses are indispensable in reducing inaccuracies caused by approximate methods. Although such computer analysis may intimidate the structural engineer by virtue of their unbelievable amount of documents and output, the prudent engineer will always verify the reasonableness of the computer analysis using approximate hand-calculated values for shears and moments. Approximate analysis is, therefore, a powerful tool in providing the engineer with (1) a basis for preliminary sizing of members, (2) and orderly method for evaluating several schemes to select the most likely one for further study, and (3) methods for obtaining approximate values of forces and moments to check on the validity of the computer solutions.

Niels, C., made a study on the impact of approximations on safety and economy. He investigated some well known approximate methods including portal method and found that many approximate methods have modest dispersion and can be more effective than some exact methods from viewpoints of overall economy of analysis.

1.5 Present work:

Computer software has been developed in Microsoft Visual Basic language for the generation of lateral forces due to wind and to perform analysis of frames subjected to laterals forces using approximate methods. The use of computer software is limited only to the regular rectangular rigid frames, that is, the frames having vertical and horizontal members only and having a natural frequency of more than 1. For comparative study of wind load provisions, code provisions of Indian, U.S.A., British, Australian and Canadian wind load standards and for comparative study of analysis for lateral loads, approximate methods, namely, portal frame method, modified portal frame method, cantilever method and factor method have been incorporated in the software.

The functions of the computer software are:

i. Calculation and distribution of lateral forces due to wind along the height of the building, according to Indian, U.S.A., British, Australian and Canadian wind load standards.

ii. Computation of bending moment in the members of the frame for wind loads using approximate methods, namely, portal frame method, modified portal frame method, cantilever method and factor method.

The chapter-wise organization of the present study is outlined below.

Chapter 1	Gives an introduction to the study.
Chapter 2	Presents a comprehensive review of literature related wind and analysis of high-rise structures.
Chapter 3	Presents the methods for assessment of wind loads according to Indian, U.S.A., Australian, British and Canadian wind loading codes in detail. For illustration of calculation of loads, and example is also given.
Chapter 4	Presents the methods of analysis used for analysing frames. All the methods are illustrated with an example. This chapter also presents the computer program developed for the study. All elements of the program are explained in detail.
Chapter 5	Contains the analytical results of the frames studied.
Chapter 6	Gives a brief summary of this study along with overall conclusions and recommendations for further study.

CHAPTER 2

WIND AND WIND CHARACTERISTICS

2.1 GENERAL

The purpose of present study is to study the interaction between wind loads and gravity frame. Wind loads are to be generated according to Indian, U.S.A., Australian, British and Canadian wind loading code provisions and analysis is to be performed on various frames for these loads using approximate analysis methods. With this point in mind, it was decided to carry out comprehensive review of literature related to wind characteristics, wind loading code provisions, and analysis methods. Literature related to wind characteristics is presented in this chapter. Wind loading code provisions and analysis methods are presented in the subsequent chapters.

2.2 WIND CHARACTERISTICS

Fundamentally, the design of civil engineering structures under wind action requires the knowledge of the wind characteristics, and the structure form and stiffness which will affect its response to wind. Wind is moving air. Air is like a fluid and its flow is governed by the laws of fluid mechanics. Flow of air or wind is caused due to temperature differentials in the atmosphere. Wind is always turbulent, even a mild breeze will have some turbulence. Wind storms can be simple gusty winds, cyclones (or hurricanes or typhoons) and tornados. The latter two have a spiral motion and proceed forward with a linear velocity. Occurrence of storms and their intensity depends very much on the geographical location. Many countries in the south-east Asian region such as India, Bangladesh, Philippines etc., are visited by cyclones or other wind storms.

Wind is essentially a random phenomenon as shown in Fig. 2.1 and is often broken down for convenience in to a mean and a fluctuating component. The wind field in the atmosphere is affected by the presence of the earth and the nature of the surface over which it flows. Within the earth's boundary layer, both components not only vary with height as shown in Fig. 2.2, but also depend upon the approach terrain and topography, as seen from Fig. 2.3.

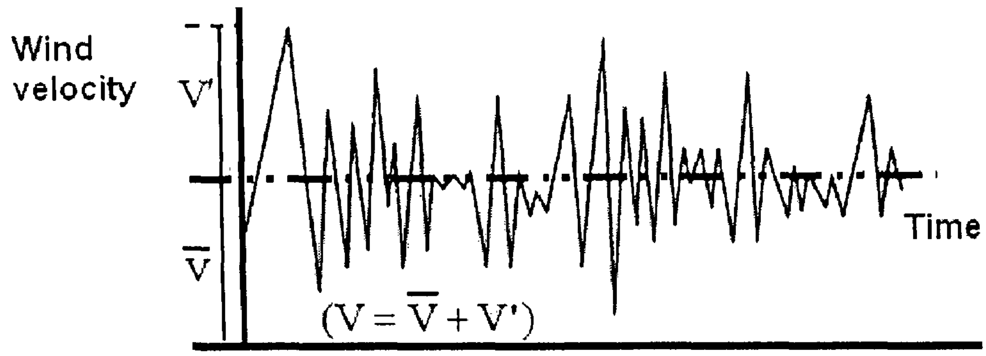


Fig. 2.1. Variation of wind velocity with time [5]

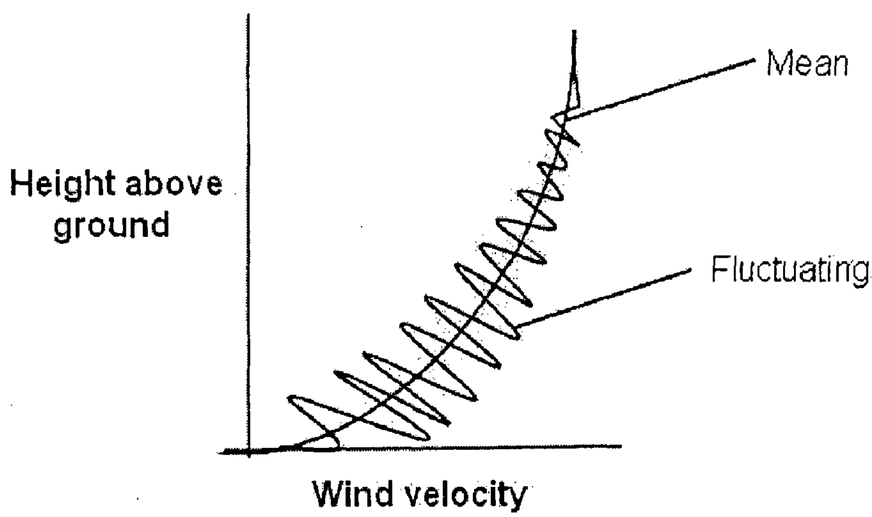


Fig. 2.2 Variation of wind velocity with height [5]

2.2.1 Boundary Layer Wind Field

When air flows over the earth's surface, a frictional force is exerted between the two. Due to viscous and eddy stresses in the air this frictional force is spread through a thin layer of air adjacent to the surface, producing a velocity gradient in that layer. This layer of the flow is called the 'boundary layer' and the viscous forces in it can be of the same order of magnitude as pressure forces. Outside this layer viscous forces are very small compared to pressure forces and can be ignored.

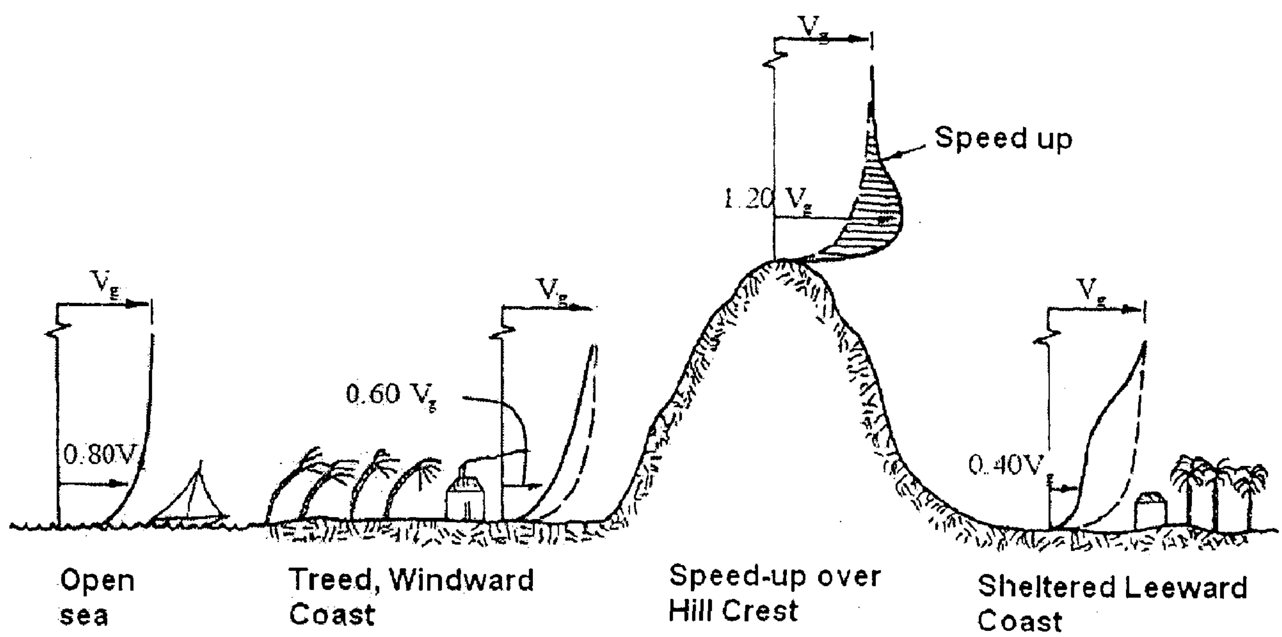


Fig.2.3. Variation of wind velocity with approach terrain and topography [5]

It is evident that the velocity increase which takes place along a vertical line must be continuous from zero on the earth's surface to maximum some distance away. The height at which velocity ceases to increase is called the gradient height and the velocity at that height is called gradient velocity. The usual convention is that the boundary layer extends to a distance δ from earth's surface such that velocity u at that distance is 99% of the local main-stream velocity u_1 which would exist there in the absence of the boundary layer.

As already mentioned, wind is created by the differential heating of the air by the sun, so that temperature gradient are part of the physical process generating wind. Two kinds of flow exist- laminar and turbulent. Whereas in case of laminar floe the air flows in lamina or layers with only a viscous force, turbulent flow allows particles of air to flow in a wild, random path with measurements of wind speeds at a point representing average values. Turbulent flow is basically caused due to the wind blowing over the rough surface of the earth, and its intensity being large near the earth's surface. Due to greater transference of momentum which takes place in turbulent flows, a turbulent boundary layer tends to be thicker than a laminar one.

The frictional force exerted by the surface of the earth on the air continually opposes motion so that the layers of air adjacent to the surface lose momentum. Viscous and eddy forces throughout the boundary layer strive to make good this loss in momentum of the surface layers from the free-stream outside the boundary layer, and in consequence the thickness of the boundary layer increases the thickness of the boundary layer increases with distance downwind.

Velocity of wind very close to earth's surface depends upon the terrain. In case of smooth surfaces, velocity is non-zero [24].

The mean wind profile over a terrain can be represented arithmetically by a log law or power law, i.e.

$$u \propto \log y$$

Or $$\left(\frac{u}{U}\right) = \left(\frac{y}{\delta}\right)^n$$

Where, u = velocity at height y from the surface

U = free stream (gradient) velocity

δ= boundary layer thickness.

The value of the exponent 'n' lies between 1/6 and 1/10, with a general value of 1/7. The velocity distribution in the major portion of the turbulent boundary layer follows a logarithmic law [24].

2.3 WIND CONDITIONS

The wind condition of concern for building design is primarily that of a wind storm, specifically high-velocity, ground-level winds. These winds are generally associated with cyclones and tornados.

2.3.1 Cyclones

Cyclones are large vortices in the atmosphere extending from 150km to 1200 km in a lateral direction with fierce winds spiraling around a central low pressure area. A cyclone covers roughly 300 to 500 km per day. The highest wind speed in cyclones which have the Indian coast in the past hundred years have been about 270 km per hour. The intensity of a cyclonic disturbance is measured by the strength of the associated wind.

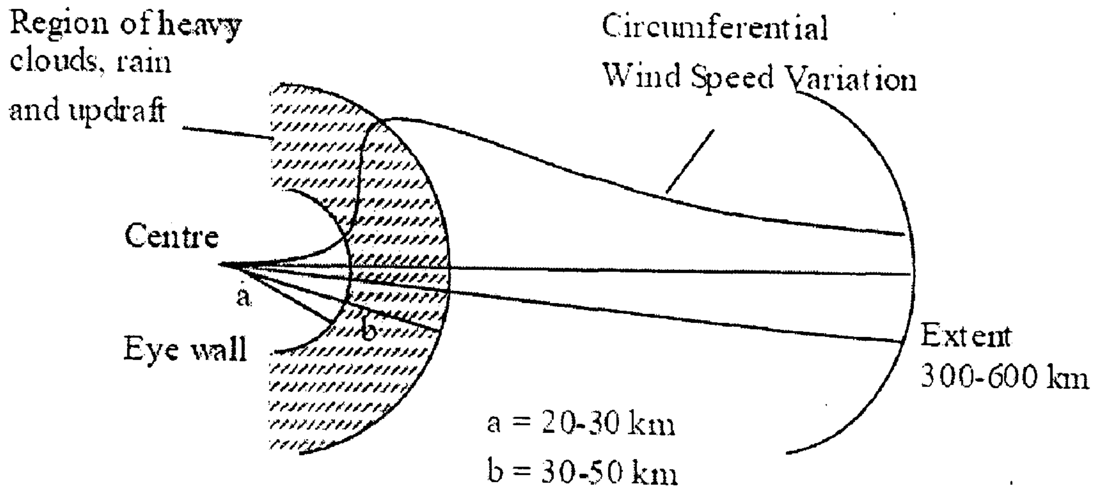
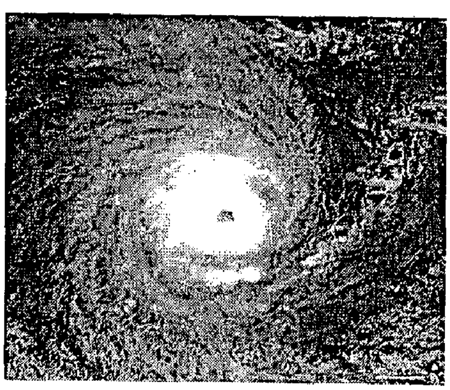


Fig. 2.4. General structure of a cyclone [5]



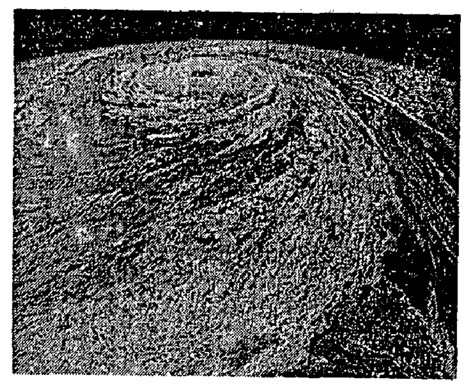


Fig. 2.5. Cyclonic storms [39]

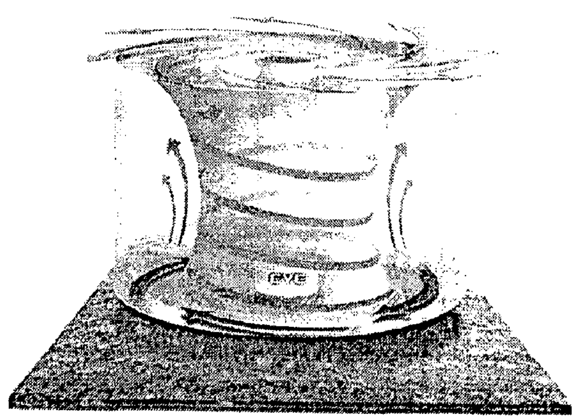


Fig. 2.6. Structure of a cyclone (schematic) [39]

A mature tropical cyclone consists of a central region of light winds as its 'eye'. The eye has an average diameter of about 20 to 30 km, but it can be 40 to 50 km in large mature storms. The pressure is lowest in this region with either clear or partly clouded skies. The eye is surrounded by a ring of very strong winds extending on an average up to 30 to 50 km beyond the centre. This area is called 'wall cloud' region. It is the most dangerous part of the cyclone, because the strongest winds and torrential rains occur in this zone. Surrounding this region, winds spiraling in a counterclockwise manner in the northern hemisphere, extend outwards to large distances, with speed decreasing as one moves further away from the centre. The rate of decrease of winds may be rapid or gradual. In the later case, speeds of 65 to 75 km per hour may be encountered even up to 600 km from the centre. There is a certain amount of symmetry in the wind distribution around the centre of a cyclone. The strongest winds are often observed to the right of the cyclone's track.

2.3.2 Tornados

Tornados are an infrequent wind phenomenon. These are high velocity spiral motions which usually have a small diameter and travel for relatively small distances. The wind field of a tornado resembles that of a Rankine combined vortex, with the tangential velocity varying along the radius. The presence of the ground, which provides a solid boundary, complicates this field by retarding the tangential motion nearer the ground surface. The excessive pressure gradient thus created gives rise to a large radial velocity and also results in vertical velocities within the tornado cover. Thus the tornado exerts vertical, radial and tangential forces. In addition, a drag force due to translational velocity is also exerted, although it is much smaller than the other force components.

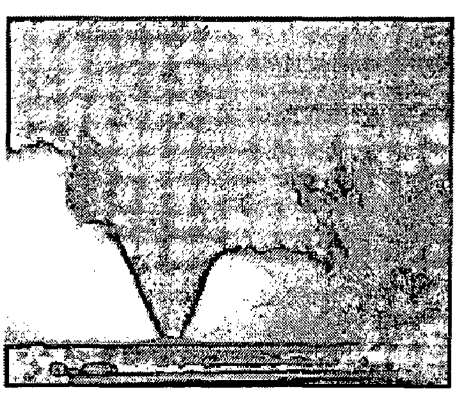


Fig.2.7. Tornados [39]

Vibrations of the structure as well as effects due to pressure drops in the tornado core also occur; the former are short lived because of the transitory nature of the tornado and the latter are important in closed buildings but such effects are negligible for latticed systems. While records of the atmospheric wind speeds and gustiness in most of the areas are available as a result of measurements over a period of times, no such records of tornado speeds are available, firstly because they are short duration unpredictable isolated occurrences and, secondly because of instrumentation difficulties. Consequently, most of the estimates of tornado velocities are made on the basis of the damage caused and assessment of failure.

2.4 DYNAMIC NATURE OF WIND

The wind speed at any height never remains constant and it has been found convenient to resolve its instantaneous magnitude into an average or mean value and fluctuating component around this average value. The average value depends on the averaging time which varies from a few seconds to several hours. The magnitude of fluctuating component of the wind speed, which is called as gust, depends on the averaging time. A structure must be designed by taking into account both the average wind speed and size of gust.

Unlike the mean flow of wind, which can be considered as static, wind load associated with gustiness or turbulence changes rapidly and even abruptly, creating effects much larger than if the same loads were applied gradually.

If the wind gust reaches its maximum value and vanishes in a time much shorter than the period of the building, its effects are dynamic. The gust can be considered static loads if the wind load increases and vanishes in time much longer than the period of the building.

The dynamic amplification of response would depend on how the gust frequency correlates with the natural frequency of the structure and also on the size of gust in relation to the building size [8].

Actual measurements on structures in wind tunnel have shown that the largest dynamic response generally occurs in lower mode or modes of vibration. The reason for this is that the greatest energy in the wind almost exists at the lowest frequencies .

Both positive and negative pressure on tall buildings does not vary in proportion to height above ground. Typically, the positive pressure contours, instead of being horizontal are usually found to follow a more concentric pattern [3].

CHAPTER 3

WIND LOAD ASSESSMENT

3.1 GENERAL

Wind loads on buildings or structures are determined by the satisfaction of the requirements of the code provisions within a country. National wind load standards are developed on the basis of research and prevalent professional practice in their respective countries and are formulated independently of each other. The wind load standards followed in India, U.S.A., Australia, Britain and Canada are tabulated below and will be referred as respective codes hereafter.

Table 3.1. Wind Loading Codes

Country	Name of the Wind Loading Code
INDIA	IS: 875 (Part3)-1987 Code of Practice for Design Loads (other than Earthquake) for Buildings and Structures, Part3: Wind Loads
U.S.A.	ASCE7-02 Minimum Design Loads for Buildings and other Structures (2002)
AUSTRALIA	AS/NZS 1170.2:2002 Structural Design Actions, Part2: Wind Actions (2002)
BRITAIN	BS 6399-2:1997 Loading for Buildings, Part2: Code of Practice for Wind Loads
CANADA	National Building Code of Canada (1995)

3.2 EFFECT OF WIND

As already mentioned, wind is moving air. The air has a particular mass (density or weight) and moves in a particular direction at a particular velocity. It thus has kinetic energy of the form expressed as

$$K = \frac{1}{2}mv^2 \qquad \cdots\cdots(3.1)$$

If an obstacle is placed in the path of the wind so that the moving air is stopped or deflected from its path, then all or part of the kinetic energy of the moving air is transformed in to the potential energy of pressure. The intensity of pressure at any point on an obstacle depends on the shape of the obstacle, the angle of incidence of the wind, the velocity and density of the air, and the relative stiffness of the engaged structure.

3.3 FACTORS DETERMINING WIND LOADS

Wind loads on buildings and structures are functions of the wind flow and of the effect of that flow on the structural system or structural or nonstructural component being considered [32].

The wind flow depends on

1. the basic wind speed
2. the mean recurrence interval of the wind speed judged to be appropriate for the design of the type of building or consideration
3. the characteristics of the terrain surrounding the building or structure
4. the characteristic height above ground for the point or system being considered
5. directional properties of the wind climate

The effect of the wind flow on the structural system or structural or nonstructural component being considered depends upon

1. the aerodynamics of the building or structure
2. the position(s) of the area(s) acted on by the wind flow
3. the magnitude(s) of the area(s) of interest
4. the porosity of the building envelope
5. the selection of the probability that the peak fluctuating wind load acting on a system or element will be exceeded during the wind storm considered in design
6. the susceptibility of the structural system under consideration to steady and time-dependent (dynamic) effects induced by the wind load

3.4 BASIC WIND SPEED

The first data required for wind load assessment on structures is the regional wind speed. The basic wind speed is the reference speed determined by routine meteorological measurements at a number of stations within a country. This important data concerning basic wind speed invariably finds a place in that country's code of practice for wind effects on structures. There is an increasing tendency in all international codes to represent the basic wind speed by means of isopleths (lines of equal wind speed) on the map of country. The wind speed fluctuates from moment to moment and can be averaged over any selected time interval at and near its peak values. It is evident that the smaller the time interval, the higher is the average wind speed. In different codes, the averaging time for basic wind speed has been selected variously between two seconds to one hour, depending upon the nature and practice adopted by the meteorologists of the particular country. Correlation between extreme wind speeds for different averaging times is also usually indicated in the codes directly or indirectly.

Since the wind speed varies with height, the basic wind speed indicated in the codes is applicable only for a particular reference height (generally about 10m). A formula relating the wind speed at various heights to the basic wind speed at the reference height is normally given in the codes to meet the designer's requirements. The basic wind speed at a meteorological station is arrived at by statistical analysis. The basic wind speed represents the extreme value which is likely to be exceeded on an average only once during a specified period (termed as "return period") normally in the range of 1 to 100 years. The codes normally give broad guidelines as to what return periods are to be taken for various types of structures. In addition, they specify how the wind velocities corresponding to the different return periods are interrelated. It is highlighted that these relationships can vary not only from country to country but sometimes also different localities in the same country.

3.4.1 Indian Code [12]

Basic wind speeds incorporated in current wind speed map of India have been arrived at by probabilistic analysis of yearly maximum 3 second peak winds by EV (Extreme Value) Type I distribution. The reference wind speed all over the country has been categorized into six ranges of basic wind speeds, varying from 33 m/sec to 55 m/sec for a 50 year return period and a probability of exceedence level of 63%.

Since the data for analysis came from DPA (Dines Pressure Tube Anemographs) which has an averaging time of about 3 seconds, analysis gives gust velocity values averaged over 3 seconds. Effects of Orography, Palghat gap, cyclonic and dust storms have been suitably incorporated in the preparation of the basic wind velocity map [Ref.]. Design wind speed up to 10m height from mean ground is taken to be constant.

3.4.2 U.S.A. Code [4]

The basic wind speed map of U.S.A. code contains map with two zones in the majority of the country, and closely specified contours for Alaska and the coastal regions adjacent to the Gulf of Mexico and the Atlantic Ocean. In the latter case, the effects of hurricanes are of particular concern. The U.S.A. code requires mountainous terrain, gorges and special regions to be examined for unusual wind conditions. Wind speeds are derived from extreme value analysis.

The basic wind speed is given as 3 second gust wind speed in miles per hour and metres per second at 33 ft (10 m) above ground in flat open country (Exposure C) for a return period of 50 years. It represents the speed from any direction. These are given in contours of 40, 45, 49, 54, 58, 63 and 67 m/s.

3.4.3 Australian Code [6]

The basic wind speed is a 3 second gust measured at 10m height in open country terrain, and values are specified for ultimate and serviceability limit states, permissible stress, for four regions of the country. The risk of exceedence for the serviceability and ultimate limits wind speeds are 5% in 1year and 50 years, respectively corresponding to return periods of 50 years and 1000 years. The values of the basic wind speeds have been given at 10 m height. The code caters to cyclones but tornadoes are not taken into account. The basic wind speed is given in the form of a map with four regions, denoted by A, B, C and D. Two of these regions (C and D) comprise a coastal strip exposed to the effects of tropical cyclones. Three separate basic wind speeds are specified for each region for design by permissible stress methods, serviceability and ultimate limit states. These correspond approximately to gust wind speeds with 20 year, 50 year, 1000 year return periods, respectively.

3.4.4 British Code [7]

The basic wind speed in this code is defined as the hourly mean wind speed with an annual risk of exceedance of 0.02, irrespective of wind direction, at 10m above flat open terrain at sea level which extends at least 100 m in all directions. The geographical variation of the basic wind speed is given by the contour map. Contours are marked at intervals of 1 m/s, ranging from 20 m/s in south midlands to 31 m/s north of Shetlands.

3.4.5 Canadian Code [20]

The basic wind speeds in this code are determined by extreme value analysis of meteorological observations of hourly mean wind speeds, taken at sites (usually airports) chosen in most cases to be representative of a height of 10 m in an open exposure. The reference wind pressure is determined from the basic wind speed and for convenience; they are tabulated for many Canadian locations. An appendix of the code contains a description of the procedures followed in obtaining the reference wind pressures for three different levels of probability of being exceeded per year (1 in 10, 1 in30, 1 in 100).

3.5 WIND LOADING

Wind loading is evaluated as the effective design pressure on the structure. This wind pressure takes account of the unsteady nature of the wind and in recent times also the dynamic characteristics of the structure due to its flexibility. Nevertheless, the approach to design remains in essence to find an equivalent static pressure which can be used as a simplified "umbrella loading". Even though each code has a set of recommendations, which differ in details from those in others, the recent international codes do have a similar approach to the issue of wind loading. Broadly speaking, the wind loading is expressed as:

$$p = qEGDS \qquad \cdots\cdots(3.2)$$

Where, p = effective design pressure on the structure

q = free-stream dynamic pressure corresponding to a reference velocity of wind at location

E = exposure factor which takes account of variations in wind loading due to height and surrounding terrain

G = gust factor which takes account of effective wind loading due to size of structure

D = dynamic response factor which takes account of vibrations set up in the structure if it is flexible

S = overall aero-dynamic shape factor of the structure, which includes modifications for differing pressure on external and internal faces while considering a structural part or element only.

The various parameters indicated above appear separately or in a combined fashion in the codes as thought convenient by those responsible for their formulation. For simpler and conventional structures or part thereof, the codes often neglect factors like D, while sometimes factors like G are tacitly accounted for in the loading data.

3.5.1 Indian Code [12]

The design wind speed is dependent on the geographical basic wind speed, return period, height above ground, structure size and local topography. The format of the equation of design wind speed adopted in the code is as follows:

$$v_d = v_b k_1 k_2 k_3 \qquad \cdots\cdots(3.3)$$

Where, v_d = design wind speed at height z in m/sec

v_b = basic wind speed of the site

k_1 = probability factor / risk co-efficient

k_2 = terrain, height and structure size factor

k_3 = local topography factor

3.5.1.1 Probability Factor / Risk Co-efficient:

The coefficient k_1 is identified in the Code for return periods of 5, 25, 50 and 100 years along with recommendations regarding "mean probable design life of structures" of various types. For general buildings and structures of permanent nature, the return periods have been recommended to be 50yrs, and a life of 50yrs for the structure. The normal value of risk level recommended is 0.63 for structures whose life expectancy is equal to the return period. The factor k_1 has been used to assess the effect of different risk levels. Although not given in the code, the risk level can be found from the formula [Ref. 1]

$$r = 1 - (1 - 1/T)^N \qquad \cdots\cdots(3.4)$$

Where, N = desired life of the structure in years

T = return period in years

r = risk level

The designer or owner must decide the amount of risk he is prepared to take. Using the values of risk level, life of structure and the quantities A and B corresponding to basic wind speed one can obtain the value of k_1 from the expression:

$$k_1 = \frac{A - B[\ln\{-1/N \ln(1-r)\}]}{A + 4B} \qquad \cdots\cdots(3.5)$$

3.5.1.2 Terrain, Height and Structure Size Factor:

The coefficient k_2 has been indicated at different heights in a convenient tabular form which identifies four terrain categories (1, 2, 3 or 4) and three classes (i.e. sizes) of structures (A, B or C). The coefficient k_2 has different values when dealing with wind speeds averaged over one hour, which are required for evaluating dynamic effects. The velocity profile for a given terrain category does not develop to full height immediately at the start of the terrain category but develops gradually. The code has added the concept of fetch length and developed height for considering the effect of change of terrain category. The relation between the developed height and the fetch for the four terrain categories is given in the code.

3.5.1.3 Local Topography Factor:

The coefficient k_3 allows for undulations in the local terrain in the form of hills, valleys, cliffs, escarpments and caters to both upwind and downwind slopes. The influence of the topographic feature is recommended to extend $1.5L_e$ upwind and $2.5L_e$ downwind of the summit of crest of the feature where L_e is the effective horizontal length of the hill depending on slope as given below:

Slope	L_e
3°<θ<17°	L
θ > 17°	Z/0.3

Where, L is the actual length of the upwind slope in the direction of wind, Z is the effective height of the feature and θ is the upwind slope in the direction of wind.

Design wind pressure is calculated from the design wind speed using the relation

$$p_d = 0.6 v_d^2 \qquad \cdots\cdots(3.6)$$

Where, p_d = design wind pressure at height z in N/m^2

v_d = design wind speed at height z in m/s

3.5.1.4 Wind Pressure and Forces on Buildings/Structures:

The code stipulates requirements for calculation of wind loading from three different points of view:

- The building/structure taken as a whole,
- Individual structural elements such as roofs and walls
- Individual cladding units such as sheeting and glazing including their fixtures.

The wind loading is given in terms of pressure coefficients (C_p) and force coefficients (C_f) and can be determined by the following equations:

$$F = C_f A_e p_d \qquad \cdots\cdots(3.7)$$

$$F = (C_{pe} - C_{pi}) A p_d \qquad \cdots\cdots(3.8)$$

Where, F = wind load

C_f = force coefficient

A_e = effective frontal area obstructing wind,

p_d = design wind pressure

C_{pe} = external pressure coefficients

A = surface area of structural elements

C_{pi} = internal pressure coefficients

For rectangular clad buildings with d/h or d/b ratio greater than 4, force due to frictional drag is also to be accounted for in addition to the above forces which are estimated from the following expressions

$$\text{If } h \le b,\ F' = C_f'(d-4h)bp_d + C_f'(d-4h)2hp_d \qquad \cdots\cdots(3.9)$$

$$\text{If } h > b,\ F' = C_f'(d-4b)bp_d + C_f'(d-4b)2hp_d \qquad \cdots\cdots(3.10)$$

Where,

C_f' has the values ranging from 0.01 to 0.04 depending upon the surface roughness.

H = height of the structure above mean ground level,

b = breadth of the structure normal to wind stream, and

d = depth of the structure parallel to wind stream.

3.5.2 U.S.A. Code [4]

The code treats main wind force resisting system and all components and cladding of the buildings separately. The design wind loads for buildings and other structures are arrived at by using one of the following three methods.

1. Method 1-simplified procedure
2. Method 2-analytical procedure
3. method 3-wind tunnel procedure

The code specifies minimum design wind loads for both main wind force resisting system and all components and cladding. The wind loads calculated by any of the above methods should not be less than those specified in the code.

Simplified procedure provides tables of design pressures for main wind force resisting system and components and cladding for simple diaphragm buildings of regular shape, with heights of 30 ft (10 m) or less, having enclosed and partially enclosed classifications. The design pressures are given for exposure B with multiplying factors to convert pressures to other exposure categories.

Wind tunnel procedure is recommended when the building does not meet requirements for using analytical method or having unusual shapes or response characteristics.

Analytical procedure is reviewed in this section.

3.5.2.1 Velocity Pressure

The velocity pressure at elevation z, q_z in N/m^2 is determined as follows

$$q_z = 0.613 k_z k_{zt} k_d V^2 I \text{ (V in m/sec)} \qquad \cdots\cdots(3.11)$$

The *basic wind speed* V is as described in section 3.4.2

The *velocity pressure exposure co-efficient* k_z reflects the dependence of the velocity pressure on terrain roughness, that is, exposure category, and height above ground. This co-efficient will be discussed in detail in the next section.

The *importance factor* I is a function of the building and structure classification. Buildings and structures in this code are classified in to four categories (I, II, III, and IV) in increasing order of the hazard to human life. The importance factor I has a value of 0.87 for category-I buildings/structures and a value of 1.0 for category-II buildings/structures and a value of 1.15 for both category-III and category-IV buildings/structures. The importance factor may be interpreted as defining the mean recurrence interval of the effective velocity $I^{1/2}V$. For I>1 (I<1), the interval is larger (smaller) than 50 years.

The *topography factor* k_{zt}, which reflects the wind speed-up effect over hills and escarpments, can be determined using the relation,

$$k_{zt} = (1 + k_1 k_2 k_3)^2 \qquad \cdots\cdots(3.12)$$

Where, k_1 = factor to account for shape of topographic feature and maximum speed-up effect,

k_2 = factor to account for reduction in speed-up with distance upwind or downwind of crest,

k_3 = factor to account for reduction in speed-up with height above local terrain.

3.5.2.2 Estimation of Pressures or Forces:

Wind pressures are specified in code for buildings other than open buildings (buildings having all walls at least 80% open). The wind force on an area acted up on by a uniform pressure is equal to the pressure times the area. This is done separately for main wind force resisting system and for components and cladding.

Pressures calculated for the design of main wind force resisting system are specified in the code for rigid buildings (height / least horizontal dimension > 4, fundamental frequency of vibration < 1 Hz), low-rise buildings (which are a class of rigid buildings), flexible buildings and arched roofs.

For rigid buildings of all height, pressures are given by the equation

$$p = qGC_p - q_h(GC_{pi}) \qquad \cdots\cdots(3.13)$$

The first and second term in the above equation represent the external and the internal pressure, respectively.

$q = q_z$ for windward walls evaluated at height z above ground;

$q = q_h$ for leeward walls, side walls, and roofs evaluated at height h (mean roof ht.);

$q_i = q_h$ for windward walls, side walls, leeward walls, and roofs of enclosed buildings and for negative internal pressure evaluation in partially enclosed buildings;

G = Gust effect factor;

C_p = external pressure co-efficient;

(GC_{pi}) = internal pressure co-efficient.

3.5.3 Australian Code [6]

The code covers structures within the following criteria:

a) Buildings less than 200 m high.
b) Structures with roof spans less than 100 m.
c) Structures other than offshore structures, bridges and transmission towers.

The wind loads on structures and elements of structures or buildings are determined as follows.

3.5.3.1 Site Wind Speed:

The site wind speeds $V_{sit,\beta}$ defined for the 8 cardinal directions β at the reference height z above ground are determined using the formula

$$V_{sit,\beta} = V_R M_d (M_{z,cat} M_s M_t) \qquad \cdots\cdots(3.14)$$

The *regional wind speeds* V_R are as explained in section 3.4.2

The *wind directional multipliers* M_d defined for 8 cardinal directions are given in a separate section of the code for all the regions. The country is divided in to 9 non-cyclonic regions (A1 to A7, W and B) and 2 cyclonic regions (C and D).

The *terrain/height multiplier* $M_{z,cat}$ takes in to account the effect of terrain and the height of structure on the wind speed. This multiplier will be discussed later in detail.

The *shielding multiplier* M_s allows for reductions in velocity when there are buildings upwind of greater or similar height. It has a value of 1.0 where the average upwind ground gradient is greater than 0.2 or where the effects of shielding are not applicable for a particular wind direction or are ignored. Only the buildings within a 45^0 sector of radius 20 times the mean height of the building and whose height is greater than or equal to mean height of the building are considered to be providing shielding. It depends on the shielding parameter s.

The *topographic multiplier* M_t takes in to account the speed-up of wind over hills or escarpments. It depends on the hill shape multiplier M_h, lee (effect) multiplier M_{lee}, height of the topographic feature and position of the structure from the crest.

3.5.3.2 Design Wind Speed:

The building orthogonal design wind speeds $V_{des,\theta}$ are taken as the maximum cardinal direction site wind speed $V_{sit,\beta}$ linearly interpolated between cardinal points within a sector ±45 degrees to the orthogonal direction being considered. For ultimate limit states design, the minimum design wind speed is specified as 30 m/s.

3.5.3.3 Design Wind Pressures:

The design wind pressure p acting normal to a surface in Pascals is given as:

$$p = (0.5\rho_{air})[V_{des,\theta}]^2 C_{fig} C_{dyn} \qquad \cdots\cdots(3.15)$$

Where ρ_{air} = density of air, taken as 1.2 kg/m^3

$V_{des,\theta}$ = design wind speed

C_{fig} = aerodynamic shape factor

C_{dyn} = dynamic response factor (the value is 1.0 except where the structure is wind sensitive)

3.5.3.4 Wind Forces:

The code provides three methods of deriving wind forces on buildings and other structures

Forces derived from wind pressures

Wind forces on surfaces or structural elements, such as a wall or a roof is the vector sum of the forces calculated from the pressures applicable to the assumed areas, represented as

$$F = \sum (p_z A_z) \qquad \cdots\cdots(3.16)$$

Where p_z = design wind pressure at height z (in Pascals)

A_z = a reference area at height z up on which the pressure at that height acts.

Forces derived from frictional drag

Forces on a building element, such as a wall or a roof, are calculated as the vector sum of the forces calculated from distributed frictional pressures applicable to the assumed areas.

$$F = \sum (f_z A_z) \qquad \cdots\cdots(3.17)$$

Where f_z = design frictional distribute pressure parallel to the surface at height z in Pascals

Forces derived from force co-efficients

Forces on whole building are calculated using force co-efficients.

$$F = (0.5\rho_{air})[V_{des,\theta}]^2 C_{fig} C_{dyn} A_{ref} \qquad \cdots\cdots(3.18)$$

Where $A_{ref} = lxb$

3.5.4 British Code [7]

The code gives methods for determining the gust peak wind loads on buildings and components that should be taken in to account in design using equivalent static procedures.

Two alternative methods are given.

a) A standard method which uses a simplified procedure to obtain a standard effective wind speed which is used with standard pressure co-efficients to determine the wind loads for orthogonal design cases.

b) A directional method in which effective wind speeds and pressure co-efficients are determined to derive the wind loads for each direction.

The methods given in this code do not apply to buildings which, by virtue of the structural properties, e.g. mass, stiffness, natural frequency or damping, are particularly susceptible to dynamic excitation. These are advised to be assessed using established dynamic methods or wind tunnel tests.

The outline of the procedure for calculating wind loads is illustrated in a flow chart. The code suggests that the wind loads should be calculated for each of the loaded areas under consideration, depending on the dimensions of the building. These may be:

a) The structure as a whole;

b) Parts of the structure, such as walls and roofs; or

c) Individual structural components, including cladding units and their fixings.

The standard method is reviewed in this section.

3.5.4.1 Site Wind Speed:

The site wind speed V_s for any particular direction is calculated from the equation

$$V_s = V_b S_a S_d S_s S_p \qquad \cdots\cdots(3.19)$$

The *basic wind speed* V_b is as discussed in section……

The *altitude factor* S_a accounts for the increase in wind speed with altitude of the site above sea level, even when the site is on flat ground. Its calculation depends on whether topography is considered to be significant, which is indicated by simple criteria. When topography is not considered significant, it depends on the site altitude above mean sea level, when topography is considered significant, it depends on the altitude of the upwind base of the significant topography above mean sea level, effective slope of the topographic feature and topographic location factor (which depends on the horizontal position over hills or escarpments).

The *direction factor* S_d is used to adjust the basic wind speed to produce wind speeds with the same risk of being exceeded in any wind direction. Values of direction factor are given for all wind directions in 30^0 intervals. If the orientation of the building is unknown or ignored, the value of the direction factor is taken as 1.0 for all directions.

The *seasonal factor* S_s is used to reduce the basic wind speed for buildings which are expected to be exposed to the wind for specific sub annual periods (1 month, 2 months, 4 months), in particular for temporary works and buildings during construction. Values of seasonal factor for buildings, which maintain the risk of being exceeded of Q=0.02 in the stated period are given in an annexure. For permanent buildings and buildings exposed to the wind for a continuous period of more than 6 months a value of 1.0 is used for S_s.

The *probability factor* S_p is used to change the risk of the basic wind speed being exceeded from the standard value of Q=0.02 annually. It is given by:

$$S = \sqrt{\frac{5 - \ln(-\ln(1-Q))}{5 - \ln(-\ln 0.98)}} \qquad \cdots\cdots(3.20)$$

Where Q is the annual probability required. This expression corresponds to a Fisher-Tippet type 1 (FT1) model for dynamic pressure that has a characteristic product (mode/dispersion ratio) value of 5, which is valid for the UK climate only. For all normal design application, where adjustments for risk are made through the partial factors, the standard value of risk, Q=0.02, is used and S_p=1.0.

3.5.4.2 Effective Wind Speed:

The effective wind speed V_e is calculated from:

$$V_e = V_s S_b \qquad \cdots\cdots(3.21)$$

Where V_s = site wind speed

S_b = terrain and building factor

The terrain and building factor S_b, which takes in to account the effective height of the building, its upwind distance from sea and terrain will be discussed later in detail.

3.5.4.3 Dynamic Pressure:

Dynamic pressure q_s (in MPa) is given by

$$q_s = 0.613V_e^2 \qquad \cdots\cdots(3.22)$$

Where V_e is the effective wind speed (in m/s)

3.5.4.4 Wind Load:

The external surface pressure p_e and internal surface pressure p_i are given by

$$p_e = q_s C_{pe} C_a \qquad \cdots\cdots(3.23)$$

$$p_i = q_s C_{pi} C_a \qquad \cdots\cdots(3.24)$$

Where q_s = dynamic pressure;

C_{pe} = external pressure co-efficient for the building surface;

C_{pi} = internal pressure co-efficient for the building;

C_a = size effect factor (different for external and internal pressures).

The size effect factor C_a accounts for the non-simultaneous action of gusts across an external surface and for the response of internal pressures. Values of size effect factor depend on the site exposure and the diagonal dimension. For external pressures the diagonal dimension is the largest diagonal of the area over which load sharing takes place. For internal pressures an effective diagonal dimension is defined dependent on the internal volume. For all individual structural components, cladding units and their fixings, the diagonal dimension should be taken as 5m, unless there is adequate load sharing capacity to justify the use of a diagonal length greater than 5m.

The net surface pressure p acting across a surface is given as:

a) For enclosed buildings

$$p = p_e - p_i \qquad \cdots\cdots(3.25)$$

b) For free-standing canopies and building elements

$$p = q_s C_p C_a \qquad \cdots\cdots(3.26)$$

Where q_s = dynamic pressure;

C_p = net pressure co-efficient for the canopy surface or element;

C_a = size effect factor for external pressures.

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

$$P = pA \qquad \cdots\cdots(3.27)$$

Where p = net pressure across the surface;

A = loaded area.

3.5.5 Canadian Code [20]

Three different approaches to the problem of determining design wind loads on buildings are mentioned in code.

1) Simple procedure
2) Detailed procedure
3) Special wind tunnel tests or other experimental methods

The simple procedure is appropriate for use with the majority of wind loading applications, including the structure and cladding of low and medium rise buildings and the cladding design of high rise buildings. These are situations where the structure is relatively rigid. Thus, dynamic actions of the wind do not require detailed knowledge of the dynamic properties of the buildings and can be dealt with by equivalent static loads.

The other two approaches are required whenever the building is likely to be susceptible to wind-induced vibration. This may be true for tall and slender structures for which wind loading plays a major role in the structural design. Wind tunnel testing is appropriate when more exact definition of dynamic response is needed and for determining exterior pressure co-efficients for cladding design on buildings whose geometry deviates markedly from more common shapes for which information is already available.

Simple procedure is reviewed in this section.

3.5.5.1 Reference Wind Pressure:

Reference wind pressure, q in kPa is determined by

$$q = C\bar{V}^2 \qquad \cdots\cdots(3.28)$$

The *reference wind speed* $\bar{V}$ is as discussed in section.....

The factor C depends on the atmospheric pressure and the air temperature. The air temperature in turn is influenced mainly by elevation above sea level, but also varies somewhat in accordance with changes in the weather. C has a value of $50x10^{-6}$ when $\bar{V}$ is in km/h and a value of $650x10^{-6}$ when $\bar{V}$ is in m/s, which is representative of Canadian conditions.

3.5.5.2 Net Specified Pressure:

The net specified pressure due to wind on part or all of a surface of a building is the algebraic difference of the external pressure or suction, p and the specified internal pressure or suction, p_i

$$p = qC_eC_gC_p \qquad \cdots\cdots(3.29)$$

$$p_i = qC_eC_gC_{pi} \qquad \cdots\cdots(3.30)$$

The *reference wind pressure* q is as given above. It is based on the level of probability of being exceeded per year (1 in 10 for cladding and for design of structural members for deflection and vibration, 1 in 30 for design of structural members for strength and 1 in 100 for the design of structural members for strength for post-disaster buildings).

The *exposure factor* C_e reflects changes in wind speed and height, and also the effects of variations in the surrounding terrain and topography. It will be discussed in detail later.

The *gust effect factor* C_g is defined as the ratio of the maximum effect of the loading to the mean effect of the loading. For small structures or structures and components having a relatively high rigidity, a simplified set of dynamic gust factors are given (2.5 for cladding elements and small structural components and 2.0 for structural systems).

C_{pe} and C_{pi} are the external and internal pressure co-efficients respectively.

3.5.5.3 Specified Wind Loading:

The net wind load for the building as a whole is the algebraic difference of the loads on the windward and leeward surfaces, and in some cases calculated as the sum of the products of the external pressures or suctions and the areas of the surfaces over which they are averaged.

3.6 VARIATION OF WIND VELOCITY WITH HEIGHT AND TERRAIN

There is an increase of wind speed with height because the ground acts as a retarding surface. The character of the surrounding terrain determines the exposure conditions of the structure and hence also plays an important role in determining the wind gustiness.

The terrain is generally classified in codes into 3 or 4 categories depending on the nature of its roughness (or height and density of the surrounding obstacles). The greater the roughness, the more "gusty" is the wind at lower layers of the atmosphere.

The variation of wind speed with height is generally correlated by a power-law formula of the following type:

$$V_z/V_o = \left(z/z_o\right)^p \qquad \cdots\cdots(3.31)$$

Where V_z is the wind velocity at height z above the ground, while V_o is the basic wind velocity at the reference height z_o. The exponent p is a variable depending on the terrain roughness and the averaging time of the basic wind. Many codes also recognise that the greater the terrain roughness, the higher the level at which the wind speed is unaffected by ground friction. This level, incidentally, at which the wind reaches a constant maximum velocity, is called the "gradient height".

3.6.1 Indian Code [12]

The terrain roughness, height and structure size coefficient k_2 has been indicated at different heights in a convenient tabular form which identifies four terrain categories (1, 2, 3 or 4) and three classes (i.e. sizes) of structures (A, B or C). The coefficient k_2 has different values when dealing with wind speeds averaged over one hour, which are required for evaluating dynamic effects. The velocity profile for a given terrain category does not develop to full height immediately at the start of the terrain category but develops gradually. The Code has added the concept of fetch length and developed height for considering the effect of change of terrain category. The relation between the developed height and the fetch for the four terrain categories is given in the code.

3.6.2 U.S.A. Code [4]

The terrain is classified in to three categories: Exposure B, Exposure C and Exposure D with decreasing roughness of the terrain. The velocity pressure exposure co-efficient k_z is given in a table of the code. The equations required for calculation of k_z are also given. The code mentions that for a site located in the transition zone between exposure categories, the category resulting in the largest wind forces shall be used, with an exception that an intermediate exposure between the categories is permitted in a transition zone provided that it is determined by a rational analysis method defined in the recognised literature.

3.6.3 Australian Code [6]

The terrain is classified in to four categories: Category 1, Category 2, Category 3, and Category 4 with increasing roughness of the terrain. The variation of the effect of terrain roughness on wind speed is taken from the values for fully developed profiles. The terrain/height multiplier $M_{z,cat}$ is given both for serviceability and ultimate limit states in a tabular form. The terrain/height multiplier $M_{z,cat}$ to be applied to the basic wind pressure can be evaluated separately for each terrain category.

3.6.4 British Code [7]

The terrain is classified in to three categories: Sea, Country and Town. The terrain and building factor S_b accounts for topography and size effects in the directional method, whereas the standard method accounts for topography in the altitude factor and excludes size effects. The terrain and building factor S_b to be applied to the basic wind pressure can be evaluated separately for each terrain category. The values of S_b in standard method are given for heights up to 100 m. For heights more than 100 m, directional method is to be used.

3.6.5 Canadian Code [20]

The question of wind speed variation with height and terrain has been approached by two alternatives: Simple Procedure and Detailed Procedure. In both the procedures, the objective is to evaluate the Exposure Factor C_e. In the Simple Procedure, the variation in surrounding terrain has been ignored and the velocity profile assumed on the basis of 'open terrain' condition. The wind velocity is kept variable interminably, without the concept of a gradient height. The velocity profile is based on the 1/10 power law. Of course, 1/5 power law is used if applied to the wind pressure instead of the wind velocity. For ready reference, the Code gives a table to correlate height with Exposure Factor, C_e which is to be applied to the basic wind pressure. In the Detailed Procedure, the terrain classification is not very different from the U.S.A. Code. The Exposure Factor C_e to be applied to the basic wind pressure can be evaluated separately for each terrain category. Interestingly, the code recognises the fetch/developed height concept as in the Australian Code, but does not go further than stating that the more favourable velocity profile corresponding to Exposure Conditions B and C should not be used unless the fetch exceeds one mile. The Detailed Procedure, which is given in a Supplement to the Code, is to be used only in conjunction with a dynamic analysis of the structure.

3.7 DYNAMIC EFFECTS

Gust loading decreases with an increase in size of structure. However, the gust loading also depends of frequency and damping of the structure and decreases with an increased value of these parameters. Again, the rougher the surrounding terrain or exposure conditions the higher the gust loading. Some recent codes make an attempt to take cognizance of the parameters discussed in the above paragraph in a comprehensive fashion and such an approach to design is called the "dynamic analysis". Flexible and slender structures or elements are invariably subjected to wind-induced excitation and oscillations. This is an aerodynamic problem caused by the fluctuating forces of the turbulent wind and the resulting dynamic response of the structure. The wind-induced excitation and oscillations can be in the direction of wind or "along-wind" (also known as "buffeting") as well as perpendicular to it, viz., "across-wind". The excitation force in the two cases can be conveniently separated, even though the structural response is likely to traverse an elliptical path. These two types of oscillations are separately dealt with in the codes. However, no clear guidelines are given in the codes for wind-induced instability phenomena like flutter and galloping, for which wind-tunnel experiments are recommended.

For "Along-wind" action, the objective is to translate the loading and structural response into one simplified factory which can be applied in the static analysis of the structure. This factor is often called the "gust factor". The approach in principle involves first the evaluation of the dynamic properties of the structure (essentially period of vibration and damping). The next issue-which is more complicated-is to define the excitation force. The wind gust spectra from random short duration meteorological micro-measurements are first identified and then these are modified by the "aerodynamic admittance function" to take account of the alteration in the excitation force of turbulent wind by its encounter with the structure. With the knowledge of the excitation force as well as the dynamic properties of the structure, calculations can be done with some degree of certainty for the case of single-degree of freedom systems.

The "Across-Wind" action of structures is less well understood as the excitation force can result from more than one source and the interaction between these are too complex to be amenable for theoretical or even semi-empirical treatment. However, for tall structures like buildings and chimneys, it is recognised that in isolated conditions the main source for across-wind motion is predominantly due to wake excitation. The other two sources, namely, turbulence of the wind and effects of movements of the structure itself are not as important in such cases. Wake excitation is induced essentially by "vortex shedding", a phenomenon in which vortices are shed alternatively from one side and then the other, thereby giving rise to a fluctuating force acting at right angles to the wind direction. Large oscillations under high acceleration develop when the shedding frequency is resonant with the natural frequency of the structure.

In both along-wind as well as across-wind actions, dynamic analysis is definitely indicated for all structures when their height is more than 4 or 5 times the least lateral dimension. Other indications are low frequency, low damping and light weight.

3.7.1 Indian Code [12]

The Code incorporates a new section relating to dynamic effects, recognizing the advent of a large number of tall, flexible and slender structures in the country's skyline. Structures which require investigation under this section of the Code are the following:

- Those with height to minimum lateral dimension of more than 5.0.
- Structures with a first mode natural frequency of less than 1 Hz.

Guidelines are given in the Code for the approximate determination of natural frequency of multi-storied buildings and the designer is cautioned regarding certain structural responses such as cross-wind motions, interferences of upwind obstructions, galloping, flutter and ovalling. The Code encourages use of wind tunnel model testing, analytical tools and reference to specialist advice when wind induced oscillations of the structure reach significant proportions.

The use of random response method for across wind response of structures of non-circular cross-sections is not given in this Code. Because of the local short period intense nature of wind in most parts of our country, the calculated gust factors as per the Indian Code are found to be generally slightly higher than in other codes which are based on the turbulence characteristics of the steady fully developed pressure system wind.

3.7.2 U.S.A. Code [4]

This standard contains procedures for the calculation of dynamic response for wind-sensitive structures, such as slender, flexible, lightly damped tall buildings. Wind-sensitive structures are classified as those with a first mode natural frequency less than 1 Hz, and a height to breadth (or depth) ratio greater than four. An analytical procedure for the determination of a 'gust effect factor', G_f for the along-wind vibrations of flexible buildings and other structures, is presented in the body of the standard. Expressions for maximum along-wind displacement and standard deviation and maximum along-wind acceleration are also given. However, no analytical procedure for cross-wind response is given.

3.7.3 Australian Code [6]

This standard also contains procedure for calculation of dynamic response for wind-sensitive structures, such as slender, flexible, lightly damped tall buildings. It classifies wind-sensitive structures as those with a first mode natural frequency less than 1 Hz, and a height to breadth (or depth) ratio greater than five. The dynamic along-wind and cross-wind responses of tall buildings and towers are dealt with in section 6 of the standard. An approach based on dynamic response factor, C_{dyn} is adopted to determine a gust (response) factor, G from which the design base overturning moment is calculated, by multiplying the mean base overturning moment by it. To determine the mean wind pressures, a different set of terrain height multipliers is provided to convert the basic gust speed to an hourly mean wind speed.

Cross-wind base overturning moment and acceleration can be determined from cross-wind force spectrum coefficients, derived from wind tunnel test data for a series of square and rectangular section buildings, with the incident wind normal to a face. Suggested values of damping for a range of steel and concrete structures under different stress levels are given. The importance of aero elastic instabilities, such as lock-in, galloping, flutter and interference are discussed separately.

3.7.4 British Code [7]

The British standard contains a 'dynamic augmentation factor', C_r, which is, in fact, not applied directly as a factor, but in the form $(1+C_r)$ to the overall horizontal loads on buildings. It is intended for application to mildly dynamic structures. If the value of C_r obtained from the graph in code exceeds 0.25, or if the height of the structure exceeds 300m, the user is referred to other codes, and other references, for further information.

3.7.5 Canadian Code [20]

As with the standards discussed previously, the Canadian code defines two separate procedures for the estimation of wind loads on structures – a simplified or detailed procedure. The detailed analysis is an equivalent procedure based on wind tunnel test results and should be used for light-weight buildings or those of extreme height, with low frequencies or suffering from low damping, and proceeds as follows. Once the reference wind pressure (q) is determined, which provides the static pressure intended to produce the same load effect as the dynamic resonant response to the actual fluctuating component of the wind, the peak alongwind acceleration may be determined.

RMS accelerations may then be found by dividing the peak acceleration by the peak factor. Having recognized that, while the primary deflection may be in the alongwind direction, the across-wind acceleration significantly affects occupant comfort and serviceability, the Canadian Code provides an expression for this acceleration at the top of the building based on a variety of wind tunnel studies.

A large part of the detailed procedure pulls required values from figures, allowing much room for human error, especially in the log-log plots, though, for the most part; exact equations are inset on each figure for a more precise analysis. However, to the credit of this code, its authors did recognize the significance of across-wind response when considering issues of occupant comfort and serviceability and has provided expressions for the across-wind acceleration to address this. The expressions for the alongwind and across-wind peak acceleration are both compact and conveniently share many of the same parameters, saving considerable computational effort. The torsional response is neglected.

3.8 SHAPE FACTORS

The wind pressure corresponding to a wind velocity can be evaluated from:

$$q = 1/2\rho V^2 = kV^2 \qquad \cdots\cdots(3.32)$$

The wind force on a structure is given by:

$$F = SAq \qquad \cdots\cdots(3.33)$$

Where $S =$ Shape factor

$A =$ Area of the structure exposed to wind

$q =$ dynamic pressure.

The shape factor S is a non-dimensional factor which is determined essentially by the geometry of the structure. The shape factor S can be thought of as being applicable to Aq, in which case it is called a "force coefficient" or to q, in which case it is called a "pressure coefficient". Both these coefficients are in use and serve different purposes. When the force on a building or structure as a whole is required for "global" effects, it is adequate to take account of the "force coefficient". When the force on a part of a building or structure is required, it becomes necessary to examine the pressure distribution at various points, and this can be conveniently evaluated by "pressure coefficient". Unless specifically indicated otherwise, "force coefficients" relate to wind drag, i.e., force in the direction of wind. Information on wind force in other directions is not readily available in codes but reference can be made to some of the well-known literature on the subject.

Wind pressure can vary considerably especially over local areas of a structure. This can be advantageously prescribed by altering the "pressure coefficients" at such locations. Examples of such cases are edges of walls and roofs. "Pressure coefficients" invariably relate to wind pressure applied perpendicular to the surface of the structural element. This also permits the usage of "internal" and "external" pressure coefficients which can be applied respectively to the interior and exterior faces of the structural element to obtain the net wind effect. These concepts are well known and have appeared in codes of practice all over the world for many years.

The present day codes, however, are recognising the need to highlight the variation of shape factor-when used to evaluate the drag force-with wind velocity and aspect ratio (height/breadth ratio). The variation of shape factor as drag coefficient C_D with Reynolds number R_e is also available for various body shapes, the latter being defined as follows:

$$R_e = \frac{\rho V D}{\mu} \qquad \cdots\cdots(3.34)$$

Where, ρ = density of air,

V = wind velocity,

D = nominal structural dimension, and

μ = viscosity of air

3.8.1 Indian Code [12]

Force coefficients applicable to the building/structure as a whole as well as to structural frameworks which are temporarily or permanently unclad are also given. For evaluating force coefficients for the clad building/structure as a whole; the code gives guidance for a variety of plan shapes and height to breadth ratios. Force coefficients on unclad buildings/structures, frameworks and their individual members, are comprehensively covered by the Code. The frameworks included are those that are single (i.e. isolated) and multiple. The effects of "shielding" in parallel multiple frames and the effect of different solidity ratios have been incorporated. Global force coefficients for square or triangular lattice towers with flat sided or rounded members are included. Force coefficients for individual members of various structural shapes and wires/cables have been given separately.

Pressure coefficients are applicable to structural elements like walls and roofs as well as to the design of cladding. The calculation process implies the algebraic addition of C_{pe} and C_{pi} to obtain the final wind loading by the use of equation (3.8). The Code indicates both these coefficients separately for a wide variety of situations generally encountered in practice. Internal pressure coefficients are largely dependent on the percentage of openings in the walls and their location with reference to wind direction. The Code indicates C_{pi} for a range of values with a possible maximum (i.e. positive pressure) and a possible minimum (i.e. negative pressure) with the provision that both the extreme values would have to be examined to evaluate critical loading on the concerned member. Three cases have been specifically indicated for arriving at C_{pi} depending upon the permeability:

- Openings up to 5% of wall area
- Openings from 5% to 20% of wall area
- Openings larger than 20% of wall area (including buildings with one side open

3.8.2 U.S.A. Code [4]

Depending on the openings present in a building, buildings are classified in to three types:

- Open buildings
- Partially enclosed buildings
- Enclosed buildings

The Code indicates internal pressure co-efficients for a range of values with a possible maximum value and a possible minimum value with the provision that both the extreme values would have to be examined to evaluate critical loading on the concerned member. The Code indicates external pressure co-efficients and force co-efficients for a wide variety of situations generally encountered in practice.

3.8.3 Australian Code [6]

The code indicates possible maximum and possible minimum values for internal pressure co-efficients depending on openings in each wall of a building. Dominant openings (sum of all openings in that surface exceeds the sum of openings in each of the other surfaces considered one at a time) are separated from potential openings and values of internal pressure co-efficients are given separately for buildings with dominant openings. The Code indicates external pressure co-efficients and force co-efficients for a wide variety of situations generally encountered in practice.

3.8.4 British Code [7]

Depending on the openings present in a building, buildings are classified in to three types:

- Enclosed buildings
- Buildings with dominant openings (area of the opening ≥ twice the sum of the openings in other faces)
- Open-sided buildings

The Code indicates internal pressure co-efficients for a range of values with a possible maximum value and a possible minimum value with the provision that both the extreme values would have to be examined to evaluate critical loading on the concerned member. The Code indicates external pressure co-efficients and force co-efficients for a wide variety of situations generally encountered in practice.

3.8.5 Canadian Code [20]

Depending on the openings present in a building, buildings are classified in to three types:

- Category 1 (buildings having small uniformly distributed openings)
- Category 2 (buildings in which significant openings, if any, can be relied on to be closed in storms but in which background leakage may not be uniformly distributed)
- Category 3 (buildings with large or significant openings)

The Code indicates internal pressure co-efficients for a range of values with a possible maximum value and a possible minimum value with the provision that both the extreme values would have to be examined to evaluate critical loading on the concerned member. The Code indicates external pressure co-efficients and force co-efficients for a wide variety of situations generally encountered in practice.

Sharma, V.R., Sheetharamulu, K., and Chaudhary K. K. [30] have compared revised IS code provisions with corresponding provisions of British, U.S.A. and Canadian codes. Comparison has been made by two ways. Firstly, the qualitative comparison has been made by comparing terms and factors used in various codes for computing wind loads. Secondly, quantitative comparison has been carried out by computing overall wind loads on typical structures. The analysis yields wide variation in design values computed as per various codes.

Ted Stathopolous and Hanqing Wu [36] have illustrated the details of changes in code provisions of the 1995 edition of National Building Code of Canada and their significance to building design. The methodology for the evaluation of wind loads on buildings is discussed. Pressure co-efficients over various building roofs are illustrated.

Mehta, K.C., Das, N.K., and Mc Donald, J.R. [17] have compared the wind load specifications contained in the four national standards (U.S.A., Australia, Britain and Canada). The comparison included the procedure to determine wind loads in each of the standards and the significant differences in the wind load provisions. Some of the specific items compared were definition, reference wind speed, terrain and height effects, levels of approaches, and special items.

Rao, P.S., Neeta Sharma, and Rachna Sehgal [26] have explained the wind load specifications of IS code. Shortfalls of the 1964 code and basic wind speed, design wind speed, wind pressures and forces on buildings, dynamic effects of 1987 code are discussed qualitatively.

Rao, G.N.V.[27] has explained the rationale behind some of the important provisions made in IS:875-1987. Certain provisions in respect of topics such as design wind speed, forces and internal/external pressures, force co-efficients for framed structures, dynamic effects, wind tunnel tests etc. are highlighted. Some examples on the application of the code are also given.

Venkateshwarlu, B.[37] has presented a background to wind load provisions in IS:875-1987. the code recommendations for the pressure coefficients used to calculate the wind force normal to the surfaces of the structural elements and cladding units, and the force coefficients used to calculate the along-wind force on different structures are discussed. The improvements required in dynamic analysis of wind-sensitive structures are also briefly dealt with.

Tandon, M.C., [35] presented an over-view to facilitate comparison of the stipulations in the codes. The codes considered were British, U.S.A., French, Australian, and Canadian. The provisions regarding wind loading, relationship between wind velocity and pressure, basic wind speeds and return periods, variation of wind velocity with height and terrain, gusts in relation to the size of structure, dynamic effects, and shape factors have been presented.

3.9 ILLUSTRATIVE EXAMPLE

For illustrating the methods of evaluating wind loads as per the wind loading codes in this work, a building with the following data is considered. A 10-storey building (storey height=3.2m), 20m x 11m in plan is located in Delhi with a number of buildings (up to 10 m) with a few isolated tall structures around. Topography of the area is flat (slope < 3 degrees). Plan and elevation of the building are shown below.

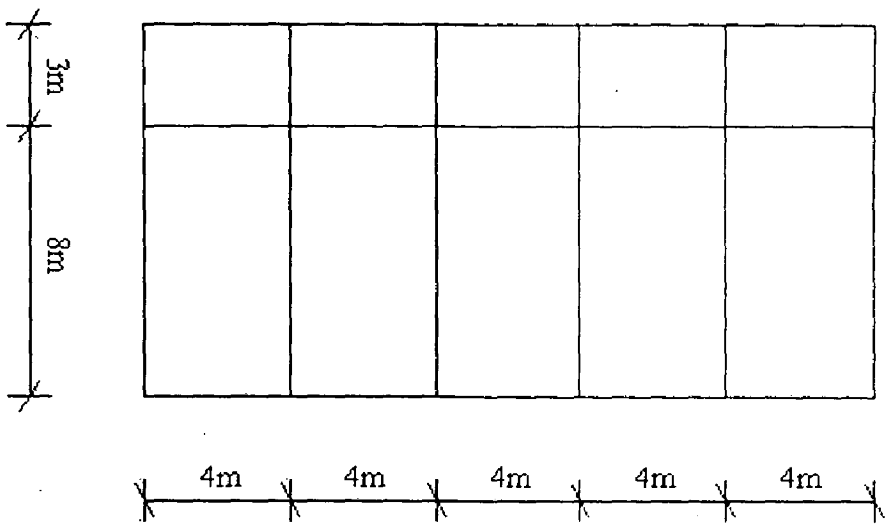


Fig: 3.1 Typical Plan of the Building

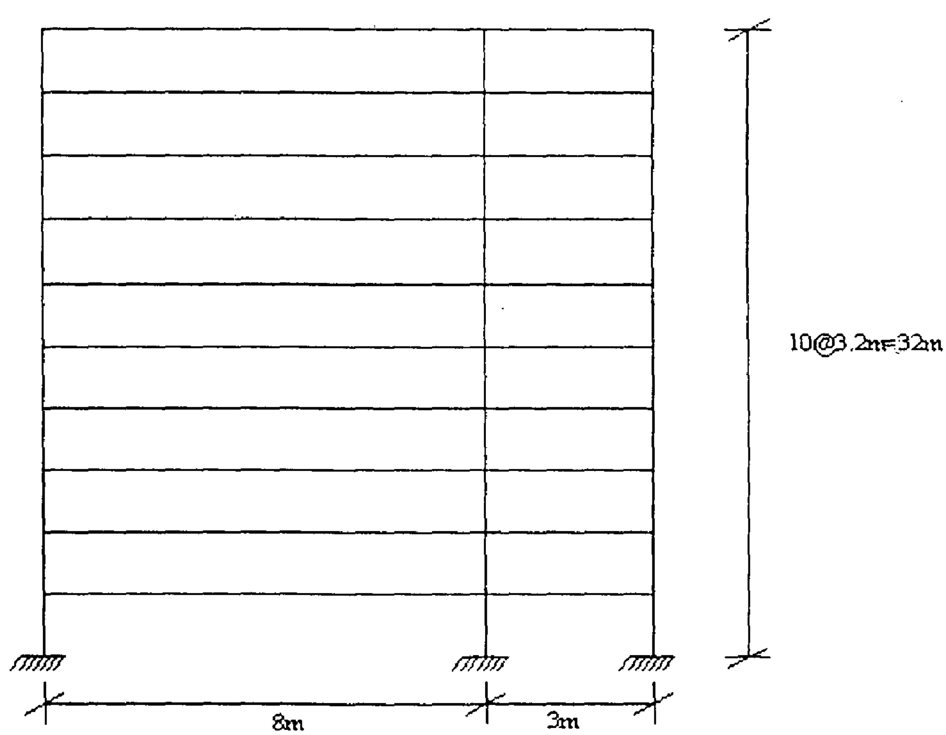


Fig: 3.2 Elevation of the Building

3.9.1 Indian Code:

$v_b = 47$ m/s

Maximum horizontal / vertical dimension is 32m (between 20m and 50m). Hence, Class B.

Design Pressure:

Desired life of structure, N=100 years

Return period, T=60 years

Hence, risk level, $r = 1-(1-1/T)^N = 1-\left(1-\frac{1}{60}\right)^{100} = 0.814$

$$k_1 = \frac{A - B[\ln\{-1/N\ln(1-r)\}]}{A+4B}$$

A=88.0, B=20.5 for $v_b = 47$ m/s

$$k_1 = \frac{88.0 - 20.5\times\left[\ln\left\{-\frac{1}{100}\ln(1-0.814)\right\}\right]}{88.0+4\times 20.5} = 1.01$$

k_3=1.0 for flat topography

$v_d = v_b k_1 k_2 k_3 = 47\times 1.01\times k_2 \times 1.00 = 47.47\times k_2$ m/s

$p_d = 0.6v_d^2 = 0.6\times(47.47\times k_2)^2$

$=1352.04\times k_2^2 \ N/m^2$

$=1.35\times k_2^2 \ kN/m^2$

Pressure coefficients:

$H/W = 32/11 = 2.91$

$L/W = 20/11 = 1.82$

Windward face:

External pressure coefficient, $C_{pe} = 0.70$

Permeability (cladding opening), $\gamma = 15\%$ of clad area (medium opening: 5%-20%)

Internal pressure coefficient, $C_{pi} = \pm 0.5$

Leeward face:

External pressure coefficient, $C_{pe} = -0.40$

Permeability (cladding opening), $\gamma = 21\%$ of clad area (medium opening: >20%)

Internal pressure coefficient, $C_{pi} = \pm 0.7$

Wind forces:

Wind force on i^{th} storey block area $= F_i = (F_i)_{WF} - (F_i)_{LF}$

WF = windward face; LF = leeward face

$(F_i)_{WF}$ = [(effective wind pressure) x (effective clad area) $]_{WF}$

$$= \left[\left\{\left(\frac{p_i + p_{i+1}}{2}\right)(C_{pe} - C_{pi})\right\} \times \{h_i \times s \times (1-\gamma)\}\right]_{WF}$$

$$= \left[\left(\frac{p_i + p_{i+1}}{2}\right) \times h_i \times s \times \{(C_{pe} - C_{pi}) \times (1-\gamma)\}\right]_{WF}$$

Where, p_i is the wind pressure on i^{th} storey

h_i is the storey height

s is the frame spacing.

Now, $F_i = (F_i)_{WF} - (F_i)_{LF}$

$$= \left(\frac{p_i + p_{i+1}}{2}\right) \times h_i \times s \times \left[\{(C_{pe} - C_{pi}) \times (1-\gamma)\}_{WF} - \{(C_{pe} - C_{pi}) \times (1-\gamma)\}_{LF}\right]$$

$$= \left(\frac{p_i + p_{i+1}}{2}\right) \times h_i \times s \times \left[(k_i)_{WF} - (k_i)_{LF}\right]$$

Where, $(k_i)_{WF} = \left[(C_{pe} - C_{pi}) \times (1-\gamma)\right]_{WF}$

$$= [\{0.70 - (-0.5)\} \times (1 - 0.15)] = 1.02$$

$$(k_i)_{LF} = \left[(C_{pe} - C_{pi}) \times (1-\gamma)\right]_{LF}$$

$$= [(-0.40 - 0.70) \times (1 - 0.21)] = -0.869$$

Now, $(k_i)_{WF} - (k_i)_{LF} = 1.02 - (-0.869) = 1.889$

Now, $F_i = \left(\frac{p_i + p_{i+1}}{2}\right) \times 3.2 \times 4.0 \times 1.889 = 12.09(p_i + p_{i+1})$

Wind force at i^{th} storey, $P_i = \dfrac{F_i + F_{i-1}}{2}$

The values of terrain factors, wind pressures, forces on storey block areas and forces at nodes are tabulated below.

Table 3.1: Wind Pressures and Forces as per Indian Code

Storey	Ht. (m)	k_2	p_i (kN/m²)	F_i (kN)	P_i (kN)
1	32.0	1.04	1.45	34.70	17.35
2	.28.8	1.02	1.42	33.73	34.21
3	25.6	1.01	1.37	32.64	33.19
4	22.4	0.99	1.33	31.55	32.10
5	19.2	0.97	1.28	30.22	30.89
6	16.0	0.95	1.22	28.41	29.32
7	12.8	0.91	1.13	26.36	27.38
8	9.6	0.88	1.05	25.39	25.87
9	6.4	0.88	1.05	25.39	25.39
10	3.2	0.88	1.05	12.69	19.04

3.9.2 U.S.A. Code:

Class of the building: Category III (Building representing substantial hazard to human life)

Basic wind speed, V = 47 m/s

Exposure category: B (urban and semi-urban areas)

Building enclosure type: Partially enclosed

Topography: Flat $\Rightarrow k_{zt} = 1.00$

Velocity Pressure:

$$q_z = 0.613 k_z k_{zt} k_d V^2 I$$

I = 1.15 for category III buildings

k_d = 0.85 for buildings

$$\therefore q_z = 0.613 \times k_z \times 1.00 \times 0.85 \times 47^2 \times 1.15$$

$$= 1323.65 \times k_z \quad N/m^2$$

$$= 1.32 \times k_z \ \ kN/m^2$$

Pressure Coefficients:

$$L/B = 20/11 = 1.82$$

Windward face:

$$C_p = 0.80$$

$C_{pi} = \pm 0.55$ for partially enclosed buildings

Leeward face:

$$C_p = -0.34$$

$C_{pi} = \pm 0.55$ for partially enclosed buildings

Nodal Wind Pressures:

$$p = qGC_p - q_h(GC_{pi})$$

$q = q_z$ for windward face

$q = q_h$ for leeward face

$q_h = q_z$ for windward face

$q_h = q_h$ for leeward face

Net pressure, $p = p_{WF} - p_{LF}$

$$= \left[qGC_p - q_h(GC_{pi})\right]_{WF} - \left[qGC_p - q_h(GC_{pi})\right]_{LF}$$

$$= \left[q_z GC_p - q_z(GC_{pi})\right] - \left[q_h GC_p - q_h(GC_{pi})\right]$$

Now, $q_h = 1.32 \times 1.00 = 1.32$

G = 0.85 for rigid buildings

Hence,

$$p = \left[q_z \times (0.85 \times 0.80 - 0.85 \times (-0.55))\right] - \left[1.32 \times 0.85 \times (-0.34) - 1.32 \times 0.85 \times 0.55\right]$$

$$= (q_z \times 1.1475) + 0.99$$

Wind forces:

Wind forces on storey block areas and at storey levels are calculated as in the case of Indian code and are tabulated below.

Table 3.2: Wind Pressures and Forces as per U.S.A. Code

Storey	Ht. (m)	k_z	p_i (kN/m^2)	F_i (kN)	P_i (kN)
1	32.0	1.00	2.14	27.20	13.60
2	.28.8	0.97	2.11	26.75	26.98
3	25.6	0.94	2.07	26.24	26.50
4	22.4	0.90	2.03	25.73	25.98
5	19.2	0.87	1.99	25.15	25.44
6	16.0	0.82	1.94	24.45	24.80
7	12.8	0.77	1.88	23.62	24.03
8	9.6	0.71	1.81	22.59	23.10
9	6.4	0.63	1.72	23.30	22.94
10	3.2	0.81	1.92	12.29	17.79

3.9.3 Australian Code:

Reference wind speed, $V_R = 47$ m/s

Terrain category: Category 3 (suburban housing)

Design Wind Speed:

Site wind speed, $V_{sit,\beta} = V_R M_d (M_{z,cat} M_s M_t)$

Number of upwind shielding buildings, $n_s = 10$

Average roof height of shielding buildings, $h_s = 35m$

Average breadth of shielding buildings, $b_s = 15m$

Now, shielding parameter, $s = \dfrac{l_s}{\sqrt{h_s b_s}}$

$$l_s = h\left(\frac{10}{n_s} + 5\right) = 32\left(\frac{10}{10} + 5\right) = 192$$

$$\Rightarrow s = \frac{192}{\sqrt{35 \times 15}} = 8.38$$

$M_s = 0.94$ for s=8.38

$M_d = 1.0$; Orientation of the building is unknown

$$\therefore V_{sit,\beta} = 47 \times 1.0 \times (M_{z,cat} \times 0.94 \times 1.0) = 44.18 \times M_{z,cat}$$

$$V_{des,\theta} = V_{sit,\beta} = 44.18 \times M_{z,cat} \text{ m/s}$$

Pressure Coefficients:

$$\frac{d}{b} = \frac{11}{20} = 0.55$$

Windward face:

$$C_{pe} = 0.80$$

$$C_{pi} = \pm 0.20$$

Leeward face:

$$C_{pe} = -0.50$$

$$C_{pi} = \pm 0.20$$

Design Wind Pressures:

$$p = (0.5\rho_{air})[V_{des,\theta}]^2 C_{fig} C_{dyn}$$

$\rho_{air} = 1.2$ kg/m^3

$C_{dyn} = 1.0$ for rigid structures

$$\therefore p = (0.5 \times 1.2)[44.18 \times M_{z,cat}]^2 \times [0.8 - (-0.2) - (-0.5 - 0.2)] \times 1.0$$

$$\Rightarrow p = 1990.9 \times M_{z,cat}^2 \text{ N/m}^2$$

$$\Rightarrow p = 1.99 \times M_{z,cat}^2 \text{ kN/m}^2$$

Wind forces:

Wind forces on storey block areas and at storey levels are calculated as in the case of Indian code and are tabulated below.

3.9.4 British Code:

Basic wind speed, $v_b = 47$ m/s

Topography: flat

Site altitude above MSL: 50m

Effective Wind Speed:

Site wind speed, $V_s = V_b S_a S_d S_s S_p$

Table 3.3: Wind Pressures and Forces as per Australian Code

Storey	Ht. (m)	$M_{z,cat}$	p_i (kN/m²)	F_i (kN)	P_i (kN)
1	32.0	1.01	2.03	25.47	12.74
2	.28.8	0.99	1.95	24.45	24.96
3	25.6	0.97	1.87	23.49	23.97
4	22.4	0.95	1.80	22.53	23.01
5	19.2	0.93	1.72	21.31	21.92
6	16.0	0.90	1.61	19.71	20.51
7	12.8	0.86	1.47	18.18	18.94
8	9.6	0.83	1.37	17.54	17.86
9	6.4	0.83	1.37	17.54	17.54
10	3.2	0.83	1.37	8.77	13.15

$$S_a = 1 + 0.001 \times 50 = 1.05$$

Annual probability required, Q=0.814 (for life of structure = 100 years, return period = 60 years)

$$\Rightarrow S_p = \sqrt{\frac{5 - \ln(-\ln(1-Q))}{5 - \ln(-\ln 0.98)}} = \sqrt{\frac{5 - \ln(-\ln(1-0.814))}{5 - \ln(-\ln 0.98)}} = 0.71$$

$S_d = 1.0$ (Orientation of the building is unknown)

$S_s = 1.0$ (For permanent buildings)

$$\therefore V_s = 47 \times 1.05 \times 1.0 \times 1.0 \times 0.71$$

$$= 35.04 \text{ m/s}$$

Effective wind speed, $V_e = V_s \times S_h$

$$= 35.04 \times S_h \text{ m/s}$$

Pressure Coefficients:

$$\frac{D}{H} = \frac{11}{32} = 0.34$$

Windward face:

$$C_{pe} = 0.85$$

$$C_{pi} = -0.30$$

Leeward face:

$$C_{pe} = -0.50$$

$$C_{pi} = 0.20$$

Dynamic Pressure:

$$q_s = 0.613V_e^2$$

$$= 0.613 \times (35.04 \times S_b)^2 = 752.64 \times S_b^2 \text{ N/m}^2$$

$$= 0.75 \times S_b^2 \text{ kN/m}^2$$

External surface pressure, $p_e = q_s C_{pe} C_a$

Diagonal dimension, $a = \sqrt{L^2 + H^2} = \sqrt{20^2 + 32^2} = 37.74$

$$\Rightarrow C_a = 0.86$$

$$\therefore p_e = 0.75 \times S_b^2 \times [0.85 - (-0.5)] \times 0.86 = 0.87 \times S_b^2$$

Internal surface pressure, $p_i = q_s C_{pi} C_a$

Diagonal dimension, $a = \sqrt[3]{L \times W \times H} = \sqrt[3]{20 \times 11 \times 32} = 8.90$

$$\Rightarrow C_a = 0.96$$

$$\therefore p_e = 0.75 \times S_b^2 \times (-0.3 - 0.2) \times 0.96 = -0.36 \times S_b^2$$

Net surface pressure, $p = p_e - p_i$

$$= [0.87 - (-0.36)] \times S_b^2$$

$$= 1.23 \times S_b^2$$

Wind forces:

Wind forces on storey block areas and at storey levels are calculated as in the case of Indian code and are tabulated below.

Table 3.4: Wind Pressures and Forces as per British Code

Storey	Ht. (m)	S_b	p, (kN/m^2)	F_i (kN)	P_i (kN)
1	32.0	1.86	2.60	54.02	27.01
2	.28.8	1.84	2.55	52.71	53.37
3	25.6	1.81	2.48	51.25	51.98
4	22.4	1.79	2.41	49.71	50.48
5	19.2	1.76	2.33	47.85	48.78
6	16.0	1.72	2.23	44.95	46.40
7	12.8	1.65	2.06	40.82	42.88
8	9.6	1.56	1.84	35.21	38.01
9	6.4	1.42	1.52	27.04	31.13
10	3.2	1.19	1.06	11.10	19.07

3.9.5 Canadian Code:

Reference wind speed, $\bar{V} = 47$ m/s

Reference wind pressure, $q = C\bar{V}^2$

$$= 650 \times 10^{-6} \times 47^2$$

$$= 1.44 \text{ kN/m}^2$$

Pressure Coefficients:

Windward face:

$C_{pe} = 0.80$

$C_{pi} = \pm 0.70$

Leeward face:

$C_{pe} = -0.50$

$C_{pi} = \pm 0.70$

Wind Pressures:

$p = qC_eC_gC_p$

$p_i = qC_eC_gC_{pi}$

$C_g = 1.0$

Net pressure on windward face = $(p - p_i)_{WF}$

$$=1.44 \times C_e \times 1.0 \times [0.8 - (-0.7)]$$

$$=2.16 \times C_e$$

Net pressure on leeward face = $(p - p_i)_{LF}$

$$=1.44 \times 1.1 \times 1.0 \times (-0.5 - 0.7)$$

$$=-1.90\,\text{kN/m}^2$$

Hence, net wind pressure, $p = 2.16 \times C_e + 1.90$

Wind forces:

Wind forces on storey block areas and at storey levels are calculated as in the case of Indian code and are tabulated below.

Table 3.5: Wind Pressures and Forces as per Canadian Code

Storey	Ht. (m)	C_e	p_i (kN/m²)	F_i (kN)	P_i (kN)
1	32.0	1.3	4.70	58.75	29.38
2	.28.8	1.2	4.48	57.34	58.05
3	25.6	1.2	4.48	57.34	57.34
4	22.4	1.2	4.48	55.94	56.64
5	19.2	1.1	4.26	54.53	55.23
6	16.0	1.1	4.26	54.53	54.53
7	12.8	1.1	4.26	53.18	53.86
8	9.6	1.0	4.05	51.84	52.51
9	6.4	1.0	4.05	50.43	51.14
10	3.2	0.9	3.83	24.81	37.47

CHAPTER 4

METHODS OF ANALYSIS

4.1 GENERAL

The purpose of structural analysis is to determine the stress resultants and displacements in the various members of a structure under any loading (static or dynamic). Structural analysis is a mathematical process by which the engineer verifies the adequacy of the structure with respect to its strength and stiffness. It is not always possible or necessary to obtain rigorous mathematical solutions for loads which may not be the primary loads on structural frames. For building structural frames, gravity loads are the primary loads whereas lateral loads due to wind or earthquake may be considered as non-primary loads from the point of view of rigorous analysis. For lateral loads, therefore, approximate or rigorous analysis may be adopted depending upon the importance of the structure itself. Also, approximate analysis may be adopted to quickly decide about the cross-sectional dimensions of the structural members which may be finally analysed rigorously, if need be. This clearly brings out the importance of approximate analysis particularly for lateral loads on structural frames. The present chapter discusses various methods which have been used for analysis of rigid frames.

In analysing three dimensional building frames for lateral loads, it is assumed to be made of a series of two dimensional frames, with assumed independent actions. This results in the frame with degree of indeterminacy many times less than that of the original frame. Further, to make the structure determinate two more assumptions are generally made. The first one is about the location of points of contraflexure in beams and columns. The second assumption is about the relative magnitude of shears in beams and columns. Based on above two assumptions the two well known methods called as Portal and Cantilever methods are presented hereunder.

4.2 DEFINITION OF THE APPROXIMATE METHODS

4.2.1 Basic requirements

The approximate methods are valid for simple frame structures (a structure composed of either perfectly vertical columns or perfectly horizontal beams) loaded with horizontal concentrated loads applied at the joints. The following figure shows two typical arrangements of structure and loading for which these methods can be used.

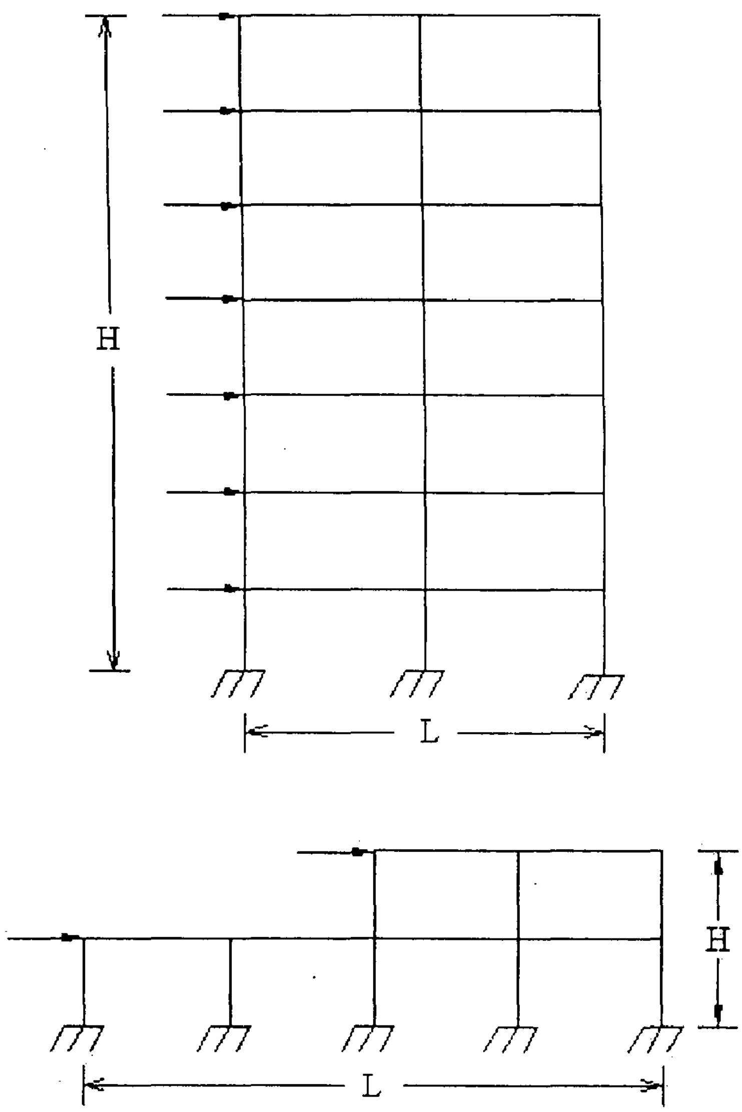


Fig. 4.1: Typical arrangements of structure and loading for which approximate methods can be used

The structure on the left is a high frame. The structure on the right is a low frame.

4.2.2 Basic Assumptions for Portal Method and Cantilever Method

The methods considered here are based on the following two assumptions regarding the location of the inflection points in the members of a given frame:

1. The point of contraflexure in a given beam is at mid-span

2. The point of contraflexure in a given column is at mid-height

Because there is no intermediate load on any of the members, the bending moment diagram in a given member will be a straight line passing through the inflection point. Typical bending moment diagrams conforming to these two assumptions are shown in the following figure:

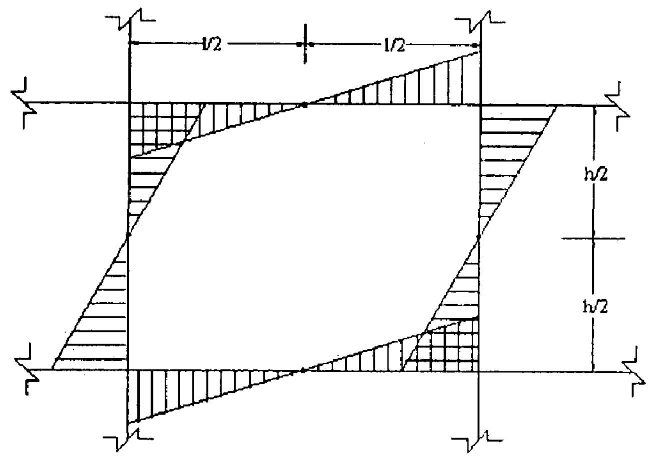


Fig. 4.2.: Typical bending moment diagrams

4.2.3 Low frames versus high frames

The two basic assumptions given above are not sufficient to allow the response of a given frame to be calculated. In addition, we need an assumption of the distribution of forces in the columns of the frames. Depending on the type of frame, two different assumptions are possible:

For low frames (a simple frame structure in which the total length L is greater than the total height H), a distribution of shear forces in the columns is assumed. The method defined by this assumption, plus assumptions (1) and (2) above, is called the "Portal method".

For high frames (a simple frame structure in which the total height H is greater than the total length L), a distribution of axial forces in the columns is assumed. The method defined by this assumption, plus assumptions (1) and (2) above, is called the "Cantilever method".

4.3 PORTAL METHOD

Portal method was proposed by Albert Smith in his paper "Wind stresses in frames of office buildings" in the journal western society of engineers (April, 1915). In this method, a wind bent is treated as a series of consecutive single-bay portal frames in the determination of axial stresses in the columns due to overturning effect. Interior columns are considered as part of two such portals, and the direct compression arising from the effect on leeward columns of one portal is offset by the direct tension arising from the overturning effect on the windward columns of the adjacent column.

Apart from assumptions (1) and (2) above, it is assumed that the total horizontal shear on each storey is divided between the columns of that storey so that each interior column carries twice as much shear as each exterior column. Treating a wind bent as a series of single-bay portal frames, it can be understood that exterior column corresponds to a single portal leg while an interior column corresponds to two portal legs. Hence it becomes reasonable to assume interior columns to carry twice the shear of exterior columns. The steps in the analysis of multi-storey rigid frame by portal method are outlined below:

1. Pass an imaginary section between any two floors through the columns at their mid-height. Since the section passes through the points of inflection of all columns, only shear and axial load act on the cut. The total shear distributed to all columns equals the sum of all lateral loads above the cut. Assume that the shear on interior columns is twice as large as the shear on exterior columns unless properties of the columns indicate that some other distribution of forces is more appropriate.

2. Compute the moments at the ends of the columns. The column end moments equal the product of the column shear and the half-storey height.

3. Compute the moment at the end of the girders by considering equilibrium of the joints. Start with an exterior joint and proceed systematically across the floor, considering free bodies of the girders and joints. Since all girders are assumed to have a point of inflection at mid-span, the moments at each end of a girder are

equal and act in the same sense (clockwise or counterclockwise). At each joint the moments in the girders balance those in the columns.

4. Compute the shear in each girder by dividing the sum of the girder end moments by the span length.
5. Apply the girder shears to the adjacent joints and compute the axial force in the columns.
6. To analyse an entire frame, start at the top and work down.

The procedure is illustrated in an example given at the end of the section.

4.4 MODIFIED PORTAL METHOD

In this method shear in columns are not assumed as in the case of portal frame method (twice for inner columns). Instead, they are assumed in the ratio in which the columns are spaced. With this modification, it is named as 'Modified Portal Frame Method'. The ratio for external columns equals half the end beam span divided by the total width of the frame. For interior columns, the ratio equals half the beam span on either side of the column divided by the total width of the frame. This assumption is justifiable because not every time the columns are spaced equally. If the columns are spaced equally, this assumption leads to that of portal frame method. The steps in the analysis of multi-storey rigid frame by modified portal frame method remains same as that of portal frame method except in determining the shear forces in columns. The procedure is illustrated in example given below.

4.5 CANTILEVER METHOD

The cantilever method was proposed by A. C. Wilson in his paper "Wind Bracing with Knee Braces or Gusset Plates" Engineering records (1908). Cantilever method, as the name implies, is based on the assumption that a building frame behaves as a cantilever beam. In this method, it is assumed that the cross section of the imaginary beam is composed of the cross-sectional areas of the columns. On any horizontal section through the frame, it is assumed that the longitudinal stresses in the columns-like those in a beam-vary linearly from the centroid of the cross section. The forces in the columns created by these stresses make up the internal couple that balances the overturning moment produced by the lateral loads.

To analyze a frame by the cantilever method, we carry out the following steps:

1. Cut free bodies of each story together with the upper and lower halves of the attached columns. The free bodies are cut by passing sections through the middle of the columns (midway between floors). Since the sections pass through the points of inflection, only axial and shear forces act on each column at that point.
2. Evaluate the axial force in each column at the points of inflection in a given story by equating the internal moments produced by the column forces to the moment produced by all lateral loads above the section.
3. Evaluate the shears in the girders by considering vertical equilibrium of the joints. The shear in the girders equals the difference in axial forces in the columns. Start at an exterior joint and proceed laterally across the frame.
4. Compute the moments in the girders. Since the shear is constant, the girder moment equals $M_G = V\left(\frac{L}{2}\right)$
5. Evaluate the column moments by considering equilibrium of joints. Start with the exterior joints of the top floor and proceed downward.
6. Establish the shears in the columns by dividing the sum of the column moments by the length of the column.
7. Apply the column shears to the joints and compute the axial forces in the girders by considering equilibrium of forces in the x direction.

The details of the method are illustrated in an example given at the end of the section.

4.6 FACTOR METHOD

Whereas the portal and cantilever methods depend on certain stress assumptions that make possible a stress analysis based on the equations of statics, the factor method depends on certain assumptions regarding the elastic action of structure which make possible an approximate slope-deflection analysis of the bent. Although it is based upon the slope-deflection method of analysis, it is possible to formulate a relatively simple set of rules by which the method can be applied without the knowledge of the elastic principles upon which it is based.

We need to compute K=I/L values for each girder and each column. It is not necessary to use absolute values of K, since the stresses depend upon the relative stiffness of the members of the bent. It is, however, necessary for the K values of the various members to be in correct ratio to each other.

The factor method is applied by carrying out the following six steps:

1. For each joint compute the girder factor 'g' by the relation:

$$g = \frac{\sum K_c}{\sum K} \cdots\cdots(4.1)$$

   Where $\sum K_c$ denotes the sum of K values for columns meeting at that joint and $\sum K$ denotes the sum of K values for all members of that joint. Write each value of g thus obtained at the near end of each girder meeting at the joint where it is computed.

2. for each joint compute the column factor c by the relation:

$$c = 1 - g \cdots\cdots(4.2)$$

   Where, g is the girder factor for that joint as computed in step 1. Write each value of c thus obtained at near end of each column meeting at that joint where it is computed. For the fixed column base of the first storey, take c=l.

3. From steps 1 and 2, there is a number at each end of each member of the bent. To each of these numbers, add half of the number at the other end of the member.

4. Multiply each sum obtained from step 3 by the K value for the member in which it occurs. For columns, call this product the column moment factor C; for girders, call this product the girder moment factor G.

5. The column moment factors C from steps 4 are actually the approximate relative values for column end moments for the story in which they occur. The sum of column end moments in a story can be shown by statics to be equal to the total horizontal shear on that story multiplied by the story height. Hence the column moment factors C can be converted into column end moments, by direct proportion, for each story.

6. The girder moment factors G from step 4 are actually approximate relative values

for girder end moments for each joint. The sum of girder end moments at each joint is equal, by statics, to sum of column end moments at that joint, which can be obtained from step 5. Hence, the girder moment factors G can be converted into girder end moments, by direct proportion for each joint.

The details of the method are illustrated in example given below.

4.7 ILLUSTRATIVE EXAMPLE

For the purpose of illustrating the methods of analysis explained above, a 10-storey building is considered. Plan and elevation of the building are shown in Fig. 4.3 and Fig. 4.4. The loads indicated on the frame are due to wind for the conditions given below. Loads are arrived at using the program developed for the present study. The details of beams, columns and slab are given below.

Type of structure	Multi storey R. C frame
Location	Delhi
Basic Wind Speed	47 m/s
Terrain Category	Category 3
Class of Structure	Class B
Mean Probable Life	100 Years
Return Period	60 Years
Topography	Flat (Slope < 3^{o})
Number of stories	Ten (G+9) as shown in fig: 4.4
Storey height	3.2m
Slab thickness	120mm for slab panels whose shorter dimension is less than equal to 4m
Beam webs	250x250mm for span of 4.0m
	450x250mm for span of 8.0m
Size of columns	400x600, 400x600 and 300x300 (up to 5 stories)
	400x500, 400x500 and 300x300 (for stories 6-10)

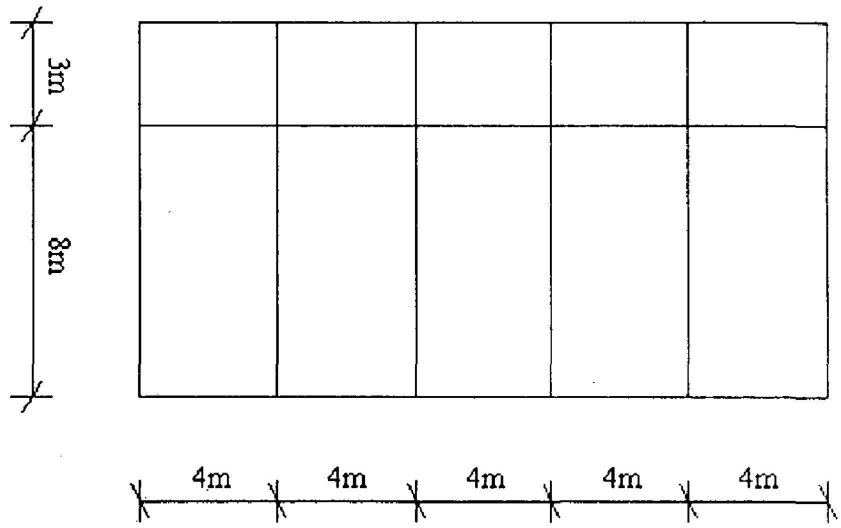


Fig: 4.3 Typical Plan of a Building

4.7.1 Portal Frame Method:

Step 1:- Computing shear force in columns

Pass horizontally a section A-A through the middle of the row of columns supporting the roof, and consider the upper free body shown in fig 4.5 below.

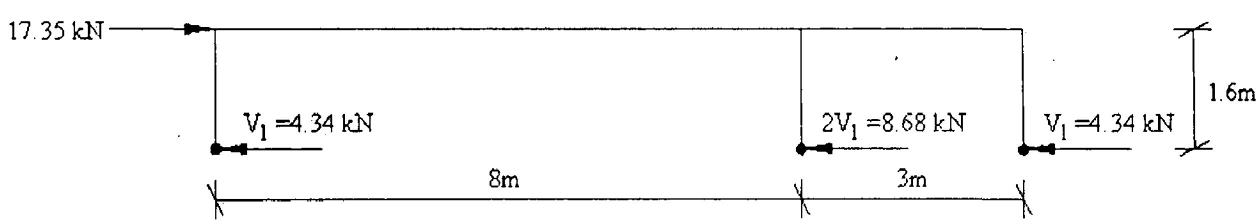


Fig 4.5: Free Body of Roof and Columns Cut by Section A-A Which Passes Through the Point of Inflection of Columns.

Establish the shear in each column by equating the lateral load above the cut (17.35KN) at joint 1) to the sum of column shears. Let V_1 represent the shear in the exterior columns and $2V_1$ equal the shear in the interior columns.

$$\sum F_x = 0$$

$$\Rightarrow 17.35 - (V_1 + 2V_1 + V_1) = 0$$

$$\Rightarrow V_1 = 4.34KN$$

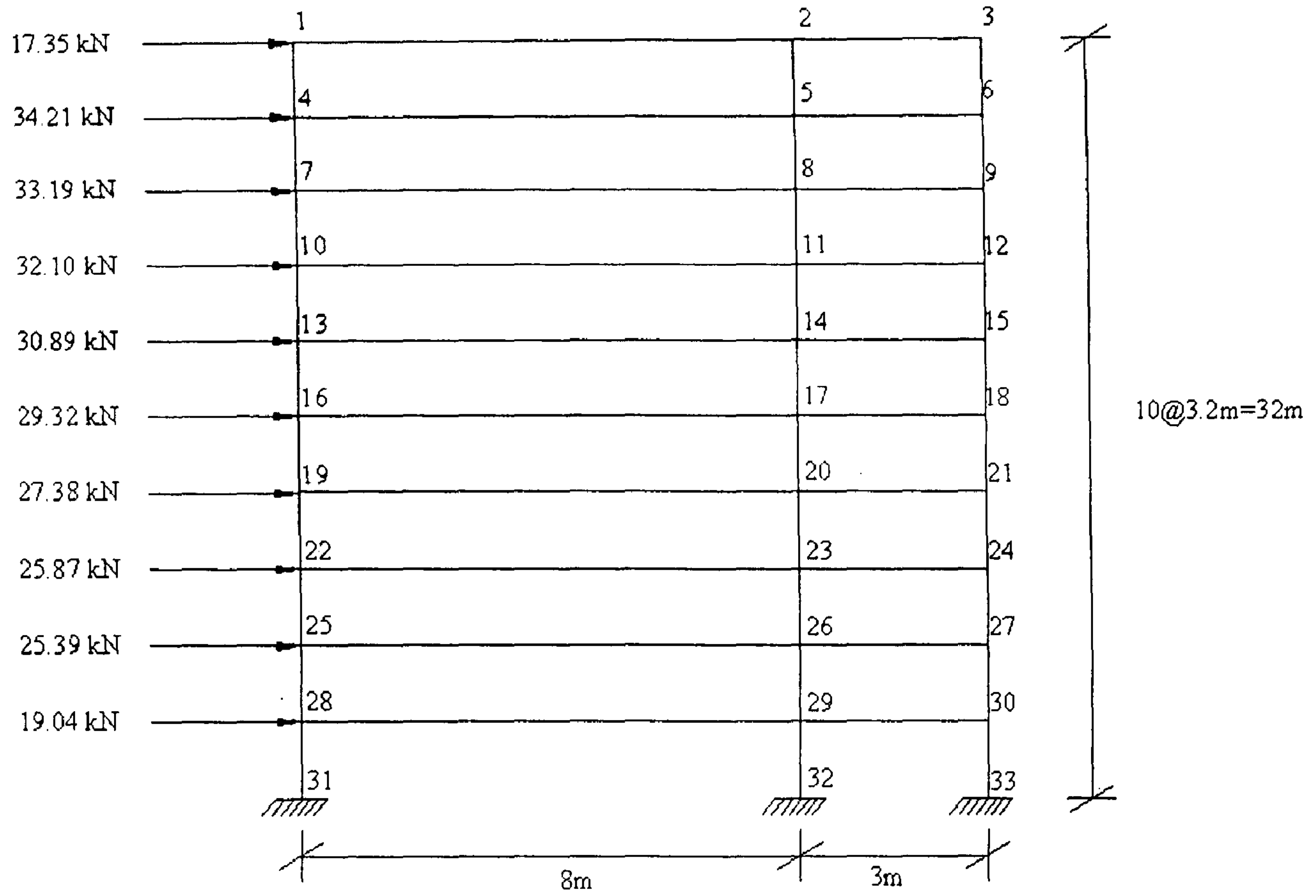


Fig: 4.4 Elevation

Step 2:- Computing moments in columns

Compute moments at the top of the columns by multiplying the shear forces at the points of infection by 1.6m, the half-storey height.

Moment in column (1-4) i.e. $M_{1\text{-}4} = 4.34*1.6 = 6.94$ KN-m

Moment in column (2-5) i.e. $M_{2\text{-}5} = 8.68*1.6 = 13.88$ KN-m

Moment in column (3-6) i.e. $M_{3\text{-}6} = 14.82*1.6 = 6.94$ KN-m

Step 3:- Computing moments in beams

Isolate joint 1 as shown in the fig: 4.6 given below.

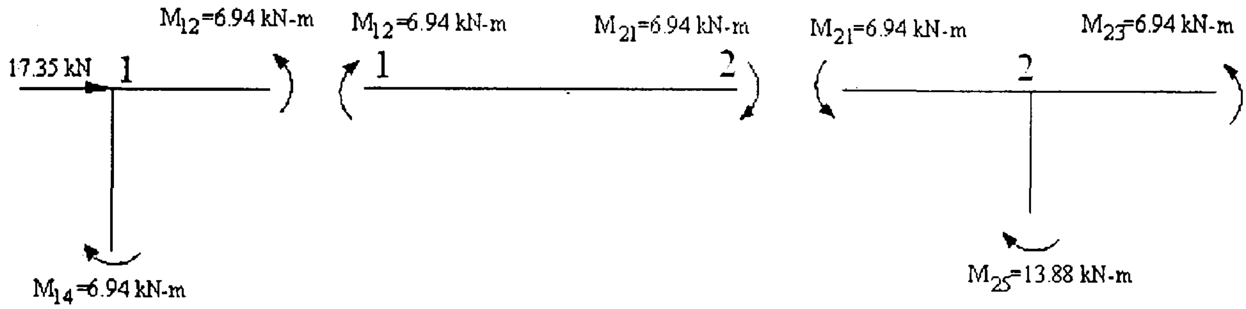


Fig 4.6: Free Body of Joint 1, Free Body of Girder (1-2), Free Body of Joint 2.

Since the beam moment must be equal and opposite of the moment in the column for equilibrium, $M_{1\text{-}2}$ = 6.94 KN-m.

Since the shear is assumed to be located at the mid span, the moment $M_{2\text{-}1}$ at the right end of the beam equals 6.94 KN-m.

Proceed to joint 2 as shown in fig: 4.7 above and use the equilibrium equations to evaluate unknown moment at the joint.

Step 4:-

Isolate the next row of beams and columns between sections A-A and B-B as shown in the fig: 4.8 given below.

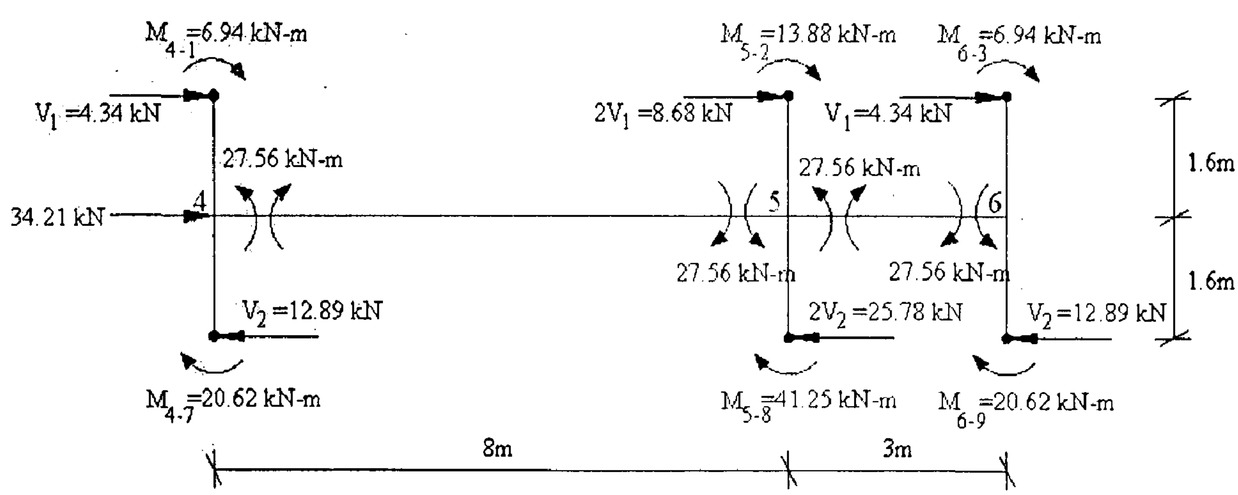


Fig 4.8: Free Body of Floor and Columns Located Between Sections A-A and B-B

Evaluate shears at points of inflection in the columns along section B-B

$$\sum F_x = 0$$

$$\Rightarrow 17.35 + 34.21 - (V_2 + 2V_2 + V_2) = 0$$

$$\Rightarrow V_2 = 12.89kN$$

Evaluate moments applied to joints 4, 5 and 6 by multiplying the shear by half-column length. Starting an exterior joint (4 for example), compute the moments in the girders following the procedure previously used to analyze the top floor.

Final values of moments are shown in the table below.

Table 4.1.: Final End Moments for Beams and Columns Using Portal Method

Sl.No.	Column No.	Moment(kN-m)	Beam No.	Moment(kN-m)
1	1-4	6.94	1-2	6.94
2	2-5	13.88	2-3	23.71
3	3-6	6.94	4-5	27.56
4	4-7	20.62	5-6	27.56
5	5-8	41.25	7-8	54.52
6	6-9	20.62	8-9	54.52
7	7-10	33.90	10-11	80.64
8	8-11	67.80	11-12	80.64
9	9-12	33.90	13-14	105.83
10	10-13	46.74	14-15	105.83
11	11-14	93.48	16-17	129.91
12	12-15	46.74	17-18	129.91
13	13-16	59.09	19-20	152.59
14	14-17	118.19	20-21	152.59
15	15-18	59.09	22-23	173.90
16	16-19	70.82	23-24	173.90
17	17-20	141.64	25-26	194.40
18	18-21	70.82	26-27	194.40
19	19-22	81.77	28-29	212.17
20	20-23	163.55	29-30	212.17
21	21-24	81.77		
22	22-25	92.12		
23	23-26	184.25		
24	24-27	92.12		
25	25-28	102.28		
26	26-29	204.56		
27	27-30	102.28		
28	28-31	109.89		
29	29-32	219.79		
30	30-33	109.89		

4.7.2 Modified Portal Method:

<u>Step 1</u>:- Computing shear force in columns

Pass horizontally a section A-A through the middle of the row of columns supporting the roof, and consider the upper free body shown in fig 4.5 below.

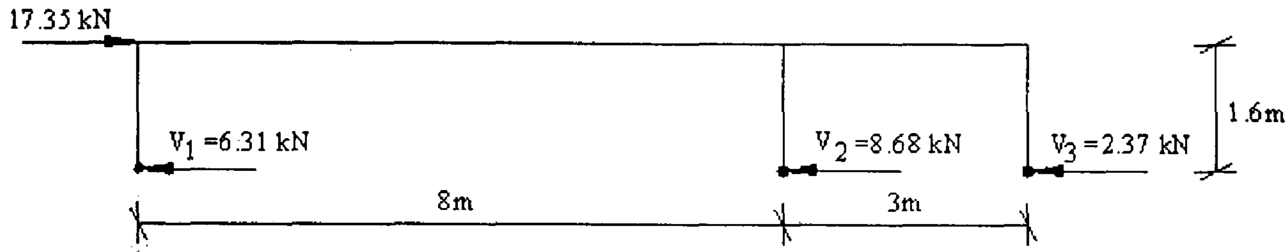


Fig 4.9: Free body of roof and columns cut by section A-A which passes through the point of inflection of columns.

Establish the shear in each column by equating the lateral load above the cut (17.35 kN) at joint 1) to the sum of column shears.

Shear in a column = (sum of lateral loads up to that storey level)*(contributory span of beam to that column/total frame width)

Shear in column 1(from left) i.e. $V_1 = 17.35 * \left(\frac{4}{11}\right) = 6.31kN$

Shear in column 2(from left) i.e. $V_2 = 17.35 * \left(\frac{5.5}{11}\right) = 8.68kN$

Shear in column 3(from left) i.e. $V_3 = 17.35 * \left(\frac{1.5}{11}\right) = 2.37kN$

Here from the steps to be followed for calculation of moments are same as the steps followed in portal frame method above.. Final values of moments are shown in table 4.2 below.

4.7.3 Cantilever Method:

<u>Step 1</u>:- Computing axial force in columns

Pass section A-A through cut mid height of the upper floor columns. The free body above section A-A is shown in fig 4.10 below.

Table 4.2.: Final End Moments for Beams and Columns Using Modified Portal Method

Sl.No.	Column No.	Moment(KN-m)	Beam No.	Moment(KN-m)
1	1-4	10.09	1-2	10.09
2	2-5	13.88	2-3	3.79
3	3-6	3.79	4-5	40.09
4	4-7	30.00	5-6	15.04
5	5-8	41.25	7-8	79.31
6	6-9	11.25	8-9	29.74
7	7-10	49.31	10-11	117.29
8	8-11	67.80	11-12	43.98
9	9-12	18.49	13-14	153.94
10	10-13	67.98	14-15	57.73
11	11-14	93.48	16-17	188.97
12	12-15	25.49	17-18	70.86
13	13-16	85.95	19-20	221.96
14	14-17	118.19	20-21	83.23
15	15-18	32.23	22-23	252.94
16	16-19	103.01	23-24	94.85
17	17-20	141.64	25-26	282.76
18	18-21	38.63	26-27	106.04
19	19-22	118.94	28-29	308.61
20	20-23	163.55	29-30	115.73
21	21-24	44.60		
22	22-25	134.00		
23	23-26	184.25		
24	24-27	50.25		
25	25-28	148.77		
26	26-29	204.56		
27	27-30	55.79		
28	28-31	159.85		
29	29-32	219.79		
30	30-33	59.94		

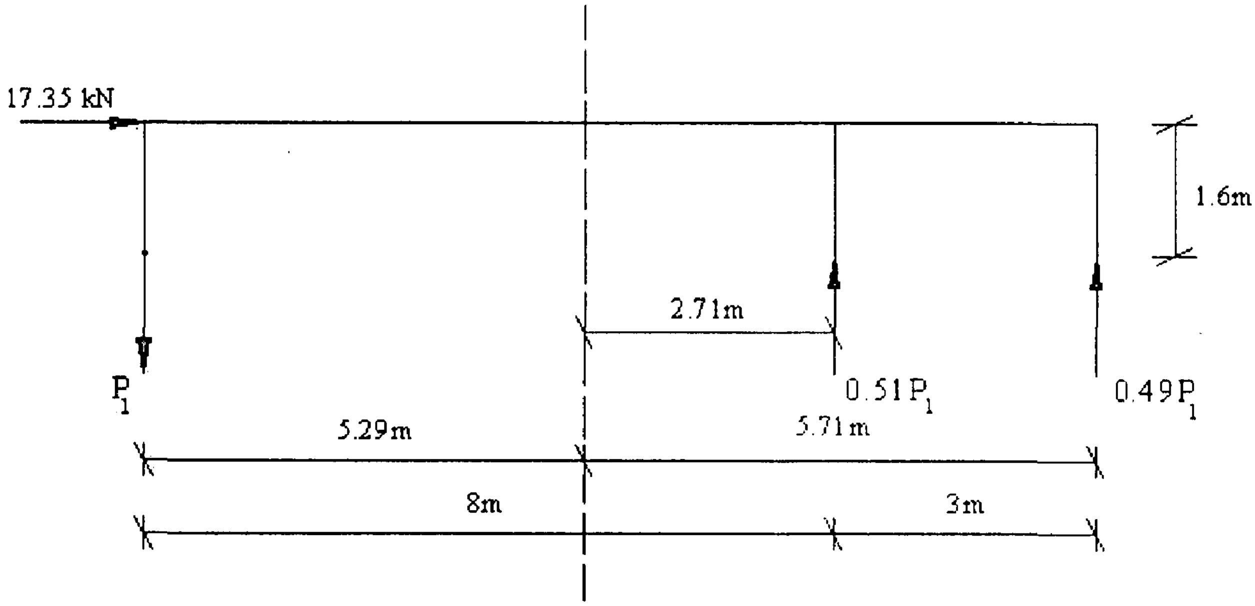


Fig 4.10: Free body of roof and columns cut by section A-A which passes through the point of inflection of columns.

Area of column (1-4) i.e. $A_{1-4} = 0.2m^2$

Area of column (2-5) i.e. $A_{2-5} = 0.2m^2$

Area of column (3-6) i.e. $A_{3-6} = 0.09m^2$

Total area of columns = $0.49m^2$

Let the distance of centre of gravity of the columns from the axis of the column (1-4) be $\overline{X}$.

Taking moments about the column (1-4)

$$0.49 * \overline{X} = 0.2*0+0.2*8+0.09*11$$

$$\Rightarrow \overline{X} = 5.29\text{m}$$

The distances of various columns from the centre of gravity of the columns are shown in the fig: 4.10 above.

Let the axial force in column (1-4) = P_1 (tensile)

Stress in column (1-4) = $\dfrac{P_1}{A_{1-4}}$ (tensile)

Stress in column (2-5) = $\left(\dfrac{2.71}{5.29}\right)\dfrac{P_1}{A_{1-4}}$ (compressive)

Stress in column (3-6) = $\left(\dfrac{5.71}{5.29}\right)\dfrac{P_1}{A_{1-4}}$ (compressive)

Now we can determine the axial forces in the columns

Axial force in column (1-4) = $\left(\frac{P_1}{A_{1-4}}\right)A_{1-4} = P_1$ (tensile)

Axial force in column (2-5) = $\left(\frac{2.71}{5.29}\right)\left(\frac{P_1}{A_{1-4}}\right)A_{2-5} = 0.51 * P_1$ (compressive)

Axial force in column (3-6) = $\left(\frac{5.71}{5.29}\right)\left(\frac{P_1}{A_{1-4}}\right)A_{3-6} = 0.49 * P_1$ (compressive)

Taking moments about the point of inflection of column (1-4)

$17.35*1.6 = (0.51P_1*8) + (0.49P_1*11)$

$\Rightarrow P_1 = 10.21KN$

Axial force in column (1-4) i.e. $F_{1-4} = 2.93$ kN

Axial force in column (2-5) i.e. $F_{2-5} = 1.49$ kN

Axial force in column (3-6) i.e. $F_{3-6} = 1.44$ kN

Step 2:- Computing shear force in beams

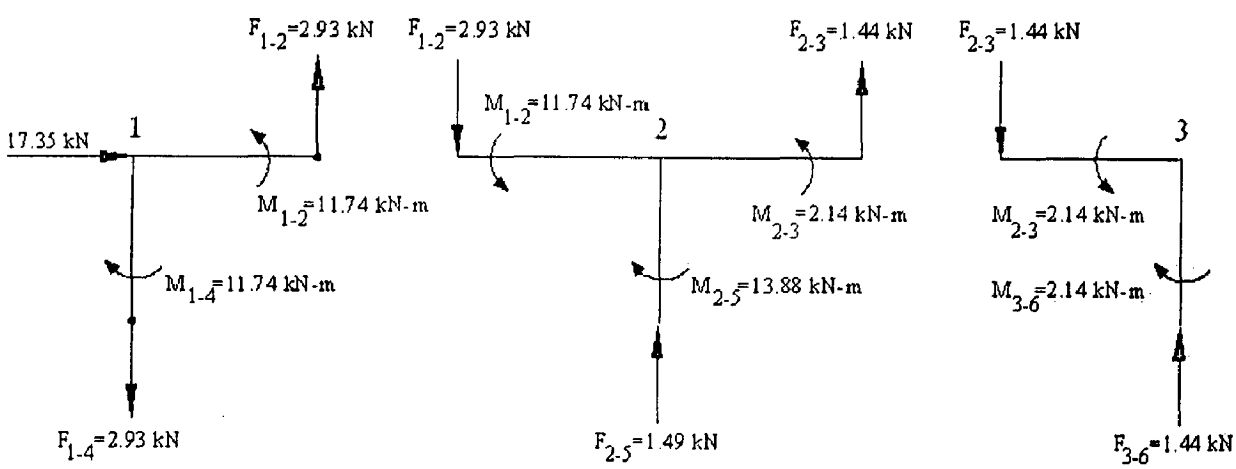


Fig 4.11: Free body of joint 1, free body of joint 2, free body of joint 3.

Considering joint (1) in fig: 4.11, for equilibrium, shear force in beam 1-2 should be equal to axial force in column 1-4. Likewise, considering joint (2), shear force in beam (2-3) can be established.

Step 3:- Computing moments in beams

Knowing shear forces in beams, moments in beams can be established, considering the equilibrium of each joint.

Moment in beam (1-2) i.e. $M_{1-2} = 2.93*4.0 = 11.74$ kN-m

Moment in beam (2-3) i.e. $M_{2-3} = 1.44*1.5 = 2.14$ kN-m

Table 4.3: Final End Moments for Beams and Columns using cantilever method

Sl.No.	Column No.	Moment(KN-m)	Beam No.	Moment(KN-m)
1	1-4	11.74	1-2	11.74
2	2-5	13.88	2-3	2.14
3	3-6	2.14	4-5	46.62
4	4-7	34.89	5-6	8.51
5	5-8	41.25	7-8	92.22
6	6-9	6.36	8-9	16.82
7	7-10	57.34	10-11	136.39
8	8-11	67.80	11-12	24.88
9	9-12	10.46	13-14	179.01
10	10-13	79.05	14-15	32.66
11	11-14	93.48	16-17	231.58
12	12-15	14.42	17-18	28.25
13	13-16	99.95	19-20	262.56
14	14-17	118.19	20-21	42.63
15	15-18	18.23	22-23	299.21
16	16-19	131.62	23-24	48.58
17	17-20	141.64	25-26	334.49
18	18-21	10.02	26-27	54.31
19	19-22	130.93	28-29	365.07
20	20-23	163.55	29-30	59.28
21	21-24	32.61		
22	22-25	168.28		
23	23-26	184.25		
24	24-27	15.97		
25	25-28	166.21		
26	26-29	204.56		
27	27-30	38.34		
28	28-31	198.85		
29	29-32	219.79		
30	30-33	20.93		

Step 4:- Computing moments in columns

At joint 1, the column moment M_{1-4} must be equal and opposite of the moment in beam, M_{1-2}

$\therefore$ Moment in column (1-4) i.e. M_{1-4} = 11.74 kN-m

Considering equilibrium of joints 2 and 3, moments in columns can be established.

Isolate next row of beams and columns between sections A-A and B-B. Following the procedure previously used to analyze the top floor, final moments in beams and columns can be calculated. Final values of moments are shown in table 4.3 above.

4.7.4 Factor Method:-

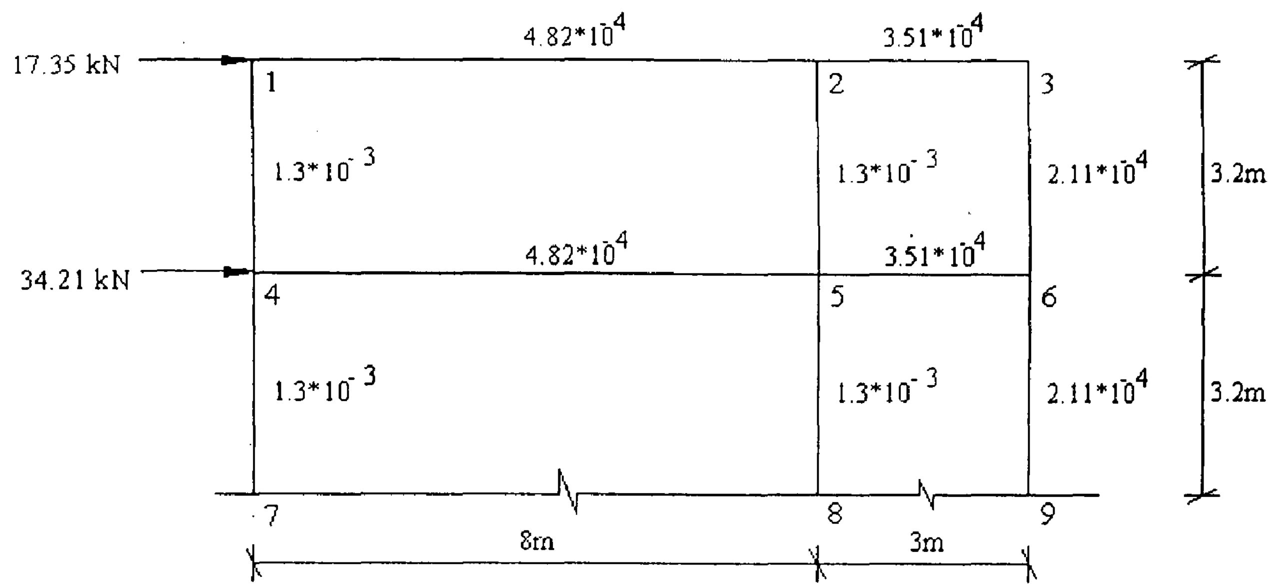


Fig 4.12: Stiffness (m^4) of beams and columns shown for top two stories of frame

<u>**Step 1**</u>:- Computation of girder factors

For joint 1: $$g_1 = \frac{1.3*10^{-3}}{4.82*10^{-4} + 1.3*10^{-3}} = 0.73$$

For joint 4: $$g_4 = \frac{1.3*10^{-3} + 1.3*10^{-3}}{4.8*10^{-4} + 1.3*10^{-3} + 1.3*10^{-3}} = 0.84$$

For joint 5: $$g_5 = \frac{1.3*10^{-3} + 1.3*10^{-3}}{4.8*10^{-4} + 3.5*10^{-4} + 1.3*10^{-3} + 1.3*10^{-3}} = 0.76$$

These factors are written at the near end of each member of the frame

<u>**Step 2**</u>:- Computation of column factors

For joint 4: $C_4 = 1 - g_4 = 1 - 0.84 = 0.16$

For joint 2: $C_2 = 1 - g_2 = 1 - 0.61 = 0.39$

These factors are written at the near end of each column at the joint.

Step 3:- Increasing the number at each end of each member by half of the number at the other end of that member.

For joint 5: member (4-5): $0.84 + (0.5*0.76) = 1.22$

For joint 5: member (5-2): $0.24 + (0.5*0.39) = 0.44$

For joint 2: member (2-3): $0.61 + (0.5*0.37) = 0.79$

Step 4:- Computation of column moment factors and girder factors

For joint 1: member (1-4): $C_{1-4} = 0.35 * 1.3 * 10^{-3} = 4.53 * 10^{-4}$

For joint 5: member (5-2): $C_{5-8} = 0.36 * 1.3 * 10^{-3} = 4.73 * 10^{-4}$

For joint 5: member (5-6): $C_{5-6} = 1.03 * 3.5 * 10^{-4} = 3.62 * 10^{-4}$

Step 5:- Determination of column moments

Storey factor for top storey, A_I = (Horizontal load at top level*storey height) / Σ (column moment factors for that storey)

$$\Rightarrow A_I = \frac{17.35 * 3.2}{4.53 * 10^{-4} + 3.79 * 10^{-4} + 6.66 * 10^{-4} + 5.70 * 10^{-4} + 1.79 * 10^{-4} + 1.61 * 10^{-4}}$$

=7195.292

Moment if column (1-4): $M_{1-4} = (4.53 * 10^{-4}) * 7195.292 = 3.26$ kN-m

Moment if column (5-8): $M_{5-8} = (4.73 * 10^{-4}) * 7195.292 = 13.24$ kN-m

Moment if column (3-6): $M_{3-6} = (1.79 * 10^{-4}) * 7195.292 = 1.29$ kN-m

Moments in other column ends for the top storey are similarly obtained by using $A_I = 7195.292$. Moments in the column ends of the third storey are obtained from A_2, which is computed by applying above equation to the third storey, leading to

$A_2 = 27936.79$

Step 6:- Determination of girder moments

Girder joint factor at any joint N,

B_N = (sum of column moments at joint N/sum of girder moment factors at joint N)

For joint 5 of the frame,

$$B_5 = \frac{(4.01 + 13.24)}{5.68 * 10^{-4} + 3.62 * 10^{-4}} = 18548.39$$

Moment in beam (5-6) = $(3.62 * 10^{-4}) * 18548.39 = 6.75kN - m$

Moment in beam (5-4) = $(5.68 * 10^{-4}) * 18548.39 = 10.59kN - m$

The shears and axial forces in columns and girder factors computed by the equations of statics, once the end moments are known.

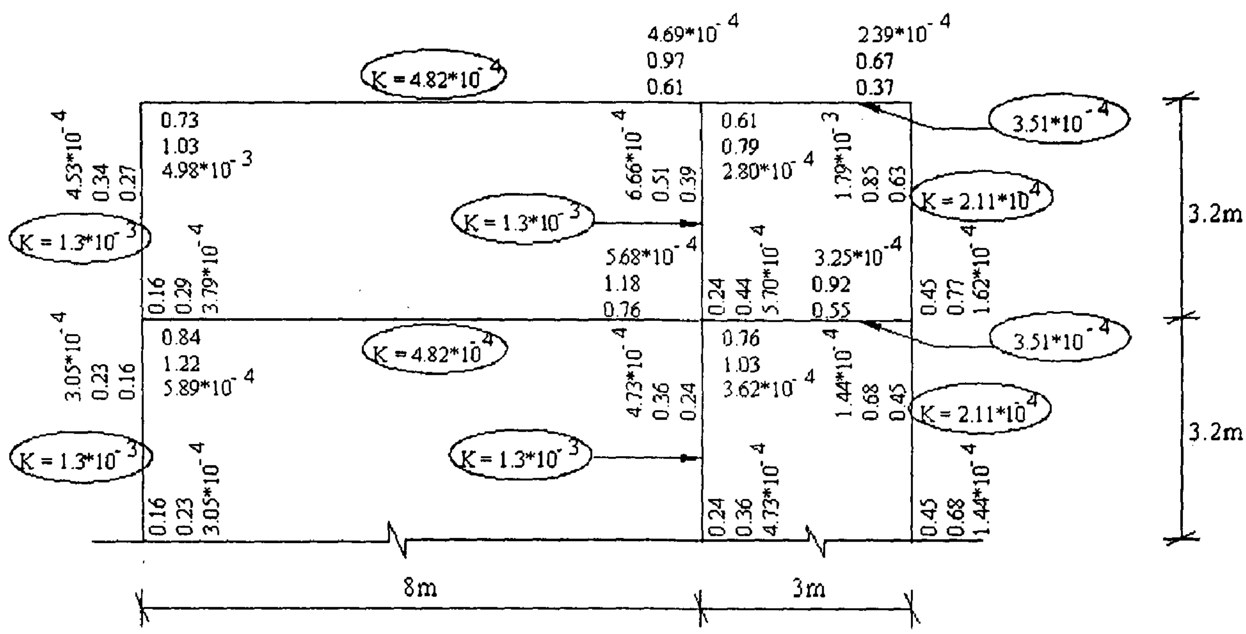


Fig 4.13: Solution for end moments by Factor method

Final values of moments are shown in the table given below.

Table 4.4: Final End Moments for Beams and Columns using Factor method

Sl.No.	Column No.	Moment(KN-m)	Beam No.	Moment(KN-m)
1	1-4	3.26	1-2	3.26
2	4-1	2.73	2-1	3.00
3	2-5	4.79	2-3	1.79
4	5-2	4.10	3-2	1.29
5	3-6	1.29	4-5	11.26
6	6-3	1.16	5-4	10.59
7	4-7	8.53	5-6	6.75
8	7-4	8.53	6-5	5.18
9	5-8	13.24	7-8	22.54
10	8-5	13.24	8-7	21.37
11	6-9	4.02	8-9	13.62
12	9-6	4.02	9-8	1.062
13	7-10	14.01	10-11	33.33

14	10-7	14.01	11-10	31.61
15	8-11	21.75	11-12	20.14
16	11-8	21.75	12-11	15.71
17	9-12	6.61	13-14	44.18
18	12-9	6.61	14-13	42.06
19	10-13	19.32	14-15	26.79
20	13-10	19.32	15-14	21.82
21	11-14	29.99	16-17	53.33
22	14-11	29.99	17-16	51.92
23	12-15	9.11	17-18	32.79
24	15-12	9.11	18-17	24.36
25	13-16	24.86	19-20	61.97
26	16-13	22.75	20-19	61.30
27	14-17	38.86	20-21	38.53
28	17-14	35.84	21-20	26.38
29	15-18	12.71	22-23	71.14
30	18-15	12.71	23-22	70.56
31	16-19	30.59	23-24	44.35
32	19-16	28.51	24-23	31.33
33	17-20	48.87	25-26	79.52
34	20-17	45.79	26-25	78.88
35	18-21	11.64	26-27	49.58
36	21-18	11.64	27-26	35.02
37	19-22	33.45	28-29	87.58
38	22-19	33.45	29-28	72.39
39	20-23	54.04	29-30	45.50
40	23-20	54.04	30-29	25.28
41	21-24	14.73		
42	24-21	14.73		
43	22-25	37.68		
44	25-22	37.68		
45	23-26	60.87		
46	26-23	60.87		
47	24-27	16.60		
48	27-24	16.60		
49	25-28	41.84		

50	28-25	41.84		
51	26-29	67.58		
52	29-26	67.58		
53	27-30	18.42		
54	30-27	18.42		
55	28-31	45.74		
56	31-28	80.36		
57	29-32	50.31		
58	32-29	82.64		
59	30-33	6.86		
60	33-30	8.82		

4.8 COMUTER PROGRAM

A computer program is written for the generation of wind loads on a typical frame of a building incorporating the Indian, U.S.A., Australian, British and Canadian codes of practice. The loads thus generated are used to analyse the typical frame using approximate methods: portal frame method, modified portal frame method, cantilever method, and factor method. This is also done by the program. The program is written in Microsoft visual basic programming language. The version used is visual basic 6.0. A brief overview of programming language is presented in this section.

4.8.1 Visual Basic

Visual Basic is a much-enhanced version of the BASIC programming language and the BASIC Integrated Development Environment (IDE). VB can create Windows programs whereas BASIC could only create DOS programs.

The "Visual" part refers to the method used to create the graphical user interface (GUI). Rather than writing numerous lines of code to describe the appearance and location of interface elements, objects can be added into place on screen. This is accomplished within the VB Integrated Development Environment (IDE), in which mouse is used to "draw" an application and keyboard is used to type in the code that is to be executed.

The "Basic" part refers to the BASIC (Beginners All-Purpose Symbolic Instruction Code) language, a language used by more programmers than any other language in the history of computing. Visual Basic has evolved from the original BASIC language and now contains several hundred statements, functions, and keywords, many of which relate directly to the Windows GUI.

VB introduced the concept of an event-driven programming model. In the old BASIC code had to be written to watch for the occurrence of user events (pressing a key, using the mouse ...). VB performs that function the only time code will execute in VB is in response to such an event. The other major concept that VB has incorporated is the concept of objects. Objects provide a way to link together both code and data into a "package" in such a way as to make handling and saving the code/data more intuitively.

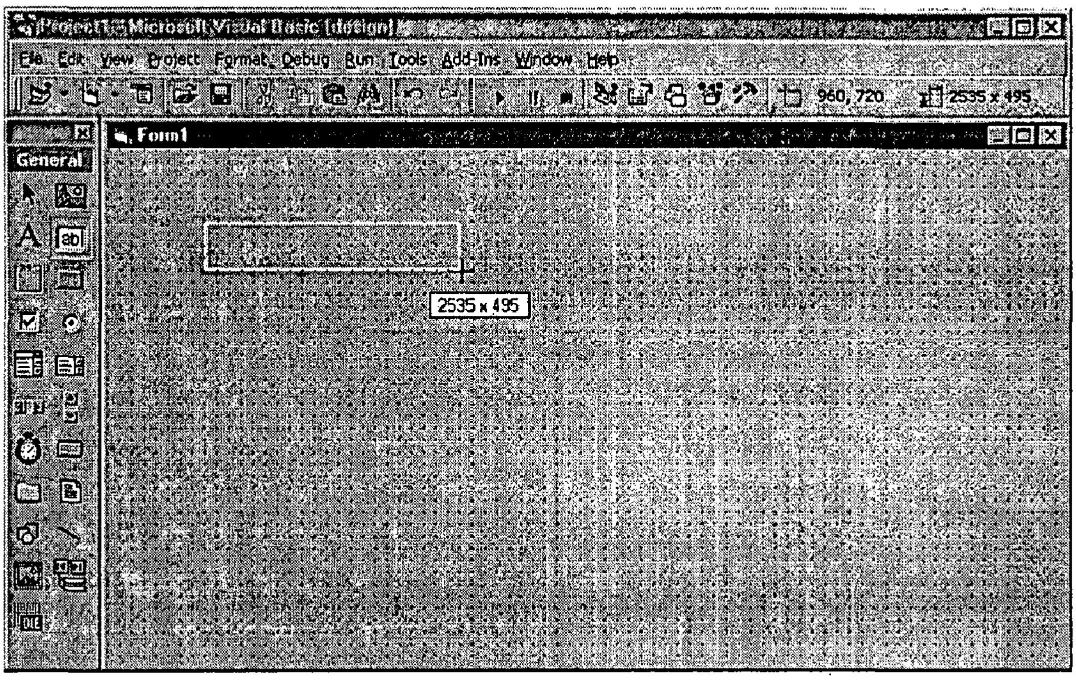


Fig.4.14. Form Designers

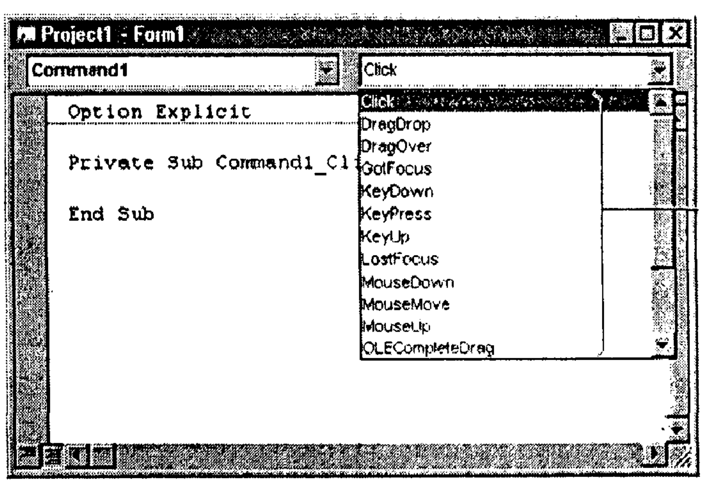


Fig.4.15. the Code Editor Window

Standard visual basic projects usually contain three types of items: global items, forms and modules. Forms are the templates on which windows are based. Modules are collections of code and data. Global items are accessible to all modules and forms in a project. The visual basic integrated development environment (IDE) is where programming is done. The visual basic IDE has three distinct states: design, run and debug. The IDE is composed of many parts, of which, the form designers and code windows are of significance. Form designers are windows in which a particular form appears. Controls can be placed in to a form by drawing them after clicking the corresponding control's tool in the toolbox. Code windows are the places where the code to be attached to an object are placed.

With this brief overview, we now proceed to the actual program.

4.8.2 Actual Program

The basic structure consists of:

1. Getting the geometry of the structure from the user.
2. Displaying the plan and elevation of the structure.
3. Allowing the user to select a wind loading code of practice (Indian, U.S.A., Australian, British or Canadian).
4. Getting the basic information needed for the calculation of wind loads (like basic wind seed, terrain category etc.), as required by the respective codes of practice.
5. Graphically displaying the values of terrain factors, nodal wind pressures, wind forces on storey block areas and nodal wind loads.
6. Allowing the user to select an approximate method used for the analysis (Portal Frame Method, Modified Portal Frame Method, Cantilever Method and Factor Method).
7. Displaying the end moments for beams and columns in a grid and also allowing the user to save those results in notepad.

The form used for getting the geometry is shown below (Fig.4.16.). The user has to enter the height, length and width of the building along with the number of stories, number of bays along length and number of bays along width. The number of stories and number of bays along length and width can be selected from the combo box or can be entered in the box.

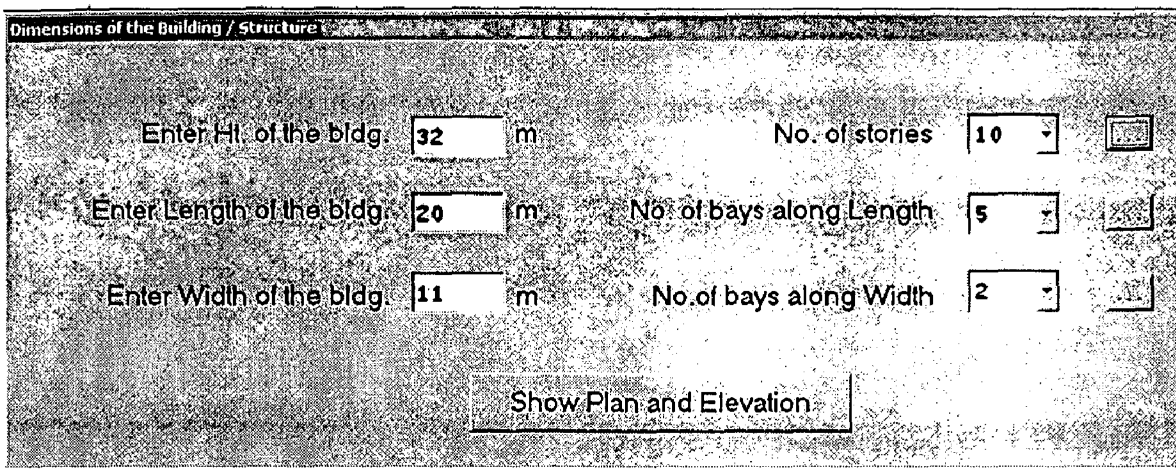


Fig.4.16. Form for entering Geometry of the Building

By clicking the ellipsis (...) command buttons, the details of the storey-heights, bay-widths are shown. The values shown in the grid (Fig.4.17.) are default values. If there are any variations in number of stories or number of bays, the user can click the reset button and enter the details accordingly.

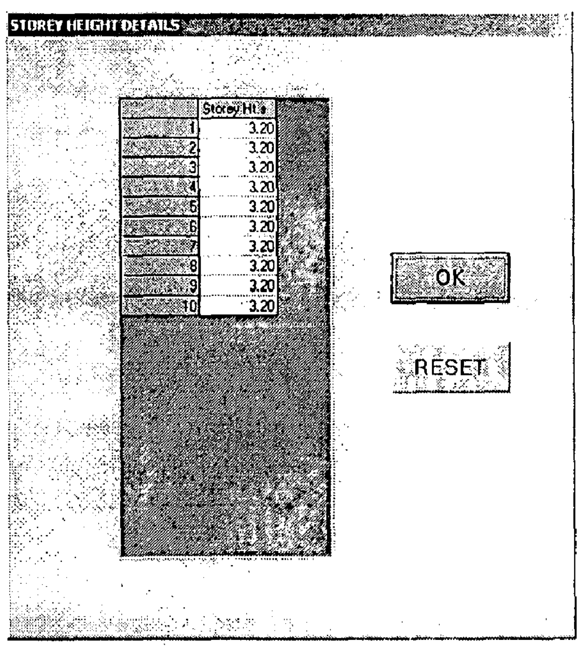


Fig.4.17. Storey Height / Bay Width Details Form

By clicking the 'show plan and elevation' button in the input form (Fig.4.16.), the plan and elevation of the building are displayed in a separate form (Fig.4.18.). Storey-heights and bay-widths are indicated in both plan and elevation. The user is asked to select a frame (by entering the corresponding number) from the plan of the building.

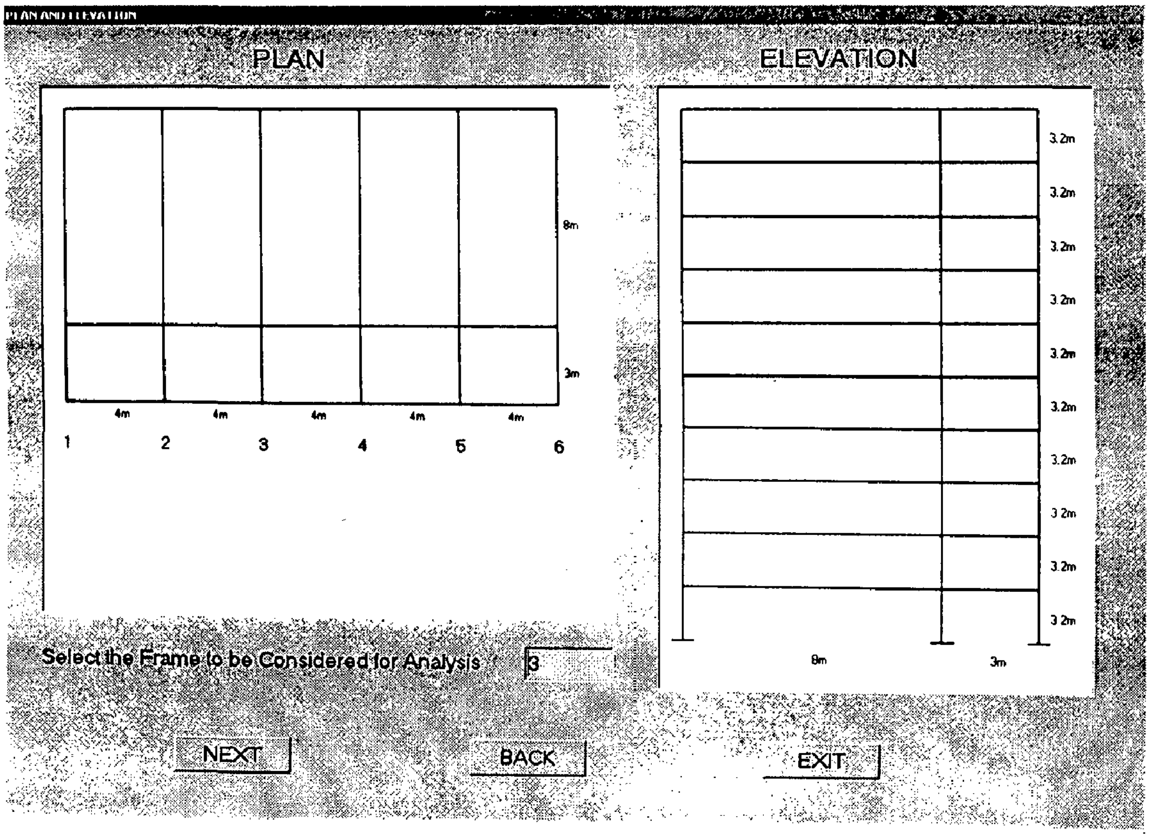


Fig.4.18. Form Showing Plan and Elevation of the Building

By clicking the 'Next' button in the form, the user is directed to a form (Fig.4.19.) where the code selection is done. The user has to select the code of practice to be used for generation of wind loads. If the user wishes to change the geometry of the building, 'Back' button is to be used. 'Exit' button is used to end the program.

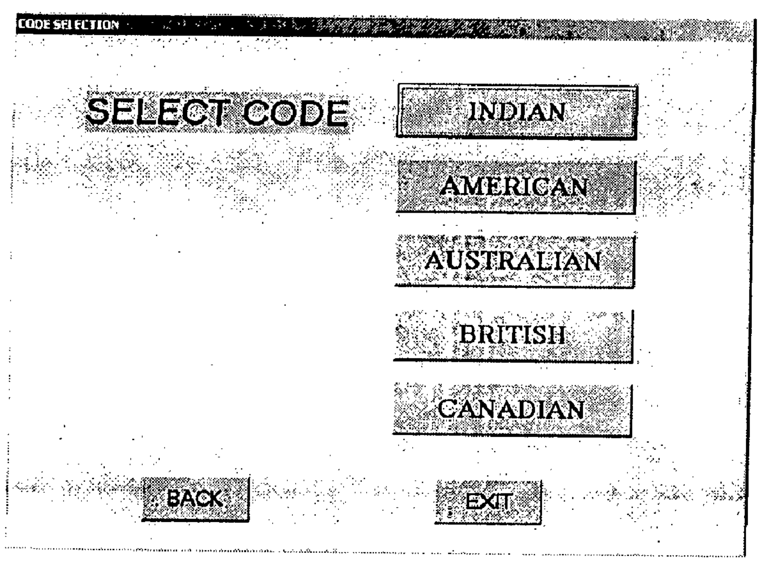


Fig.4.19. Code Selection Form

Here the user is asked to select the code of practice to be used for generating wind loads. 'Next', 'Back' and 'Exit' buttons are placed in every form of the program, by which the user can jump to any form, if need be. By clicking the buttons with names of countries, the user is directed to the respective input forms. The input form for Indian code is shown in Fig.4.20.

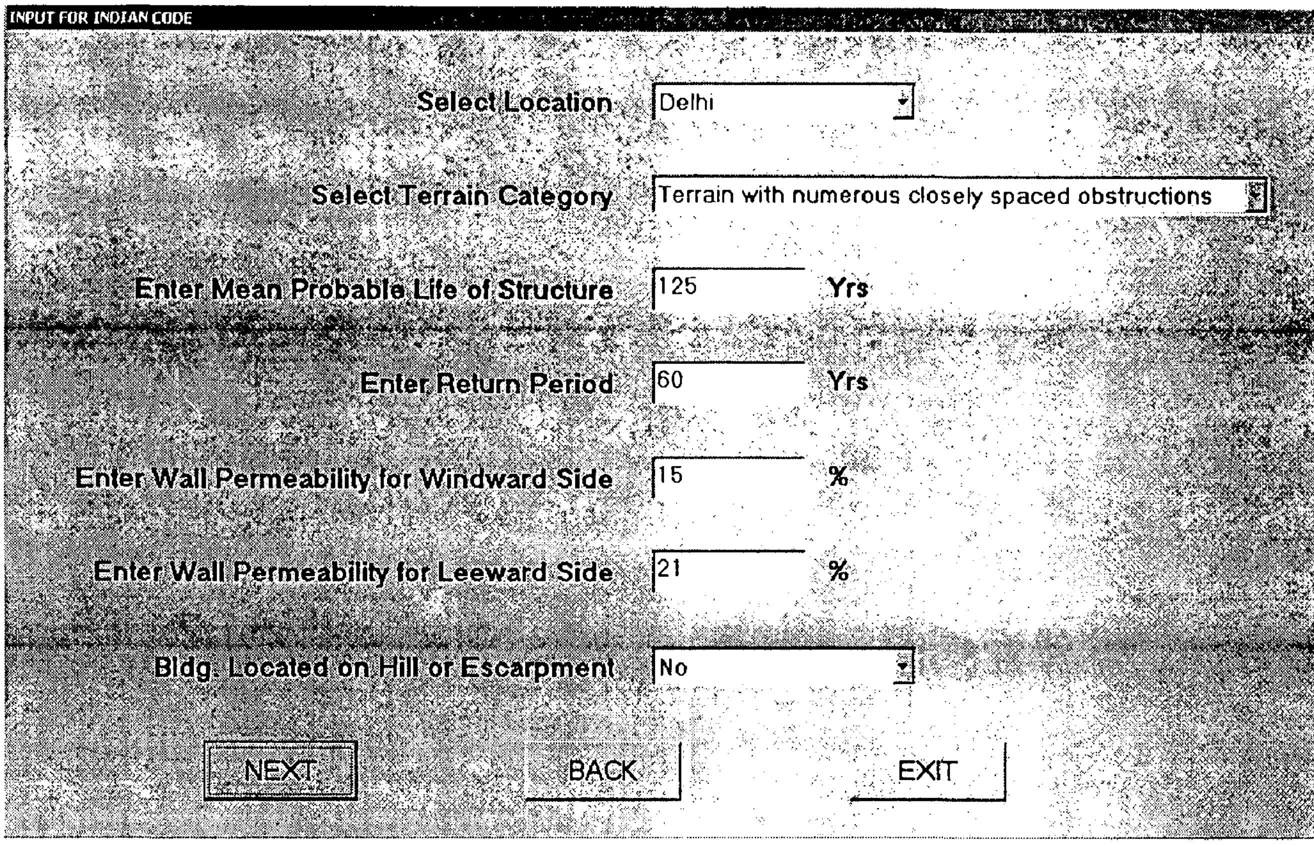


Fig.4.20. Input Details Form for Indian Code

The user has to enter the details in corresponding text boxes or combo boxes. These are the minimum details needed for the calculation of wind loads using Indian code. The list of cities/towns given in the Indian code for which basic wind speeds are given is included in the combo box corresponding to 'Select Location' label. Terrain categories are listed in the combo box corresponding to 'Select Terrain Category' label. The user has to select 'Yes' or 'No' listed in the combo box corresponding to 'Bldg. Located on Hill/ Escarpment' label. If building is located on hill/escarpment, the user is directed to a form (Fig.4.21.) containing the input details required for calculating the effect of topography, if not, the user is directed to the form (Fig.4.22.) containing output commands.

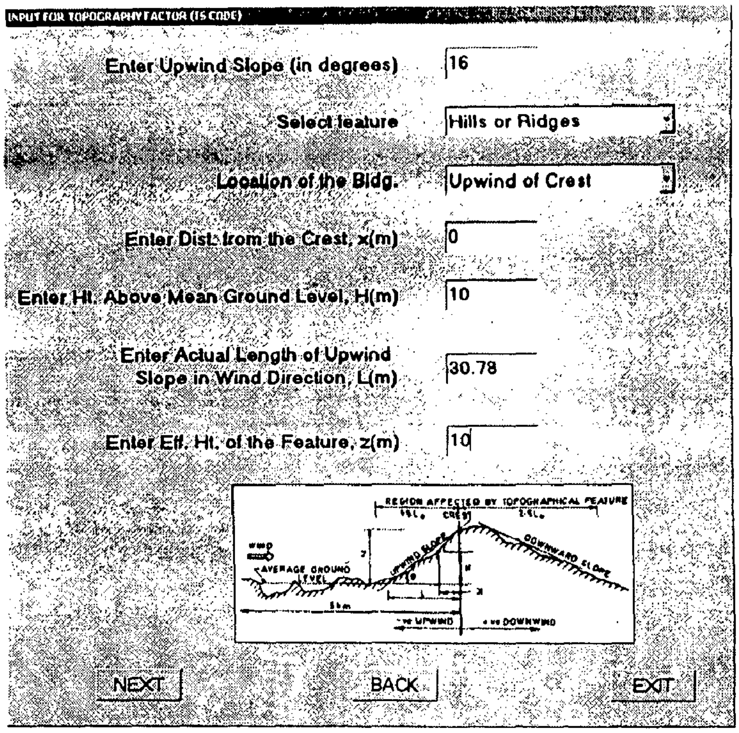


Fig.4.21. Input Form for Calculating Topography Factor as per Indian Code

Topographic features, 'Hills or Ridges' and 'Cliffs or Escarpments' are listed in the combo box corresponding to 'Select feature' label. A picture containing the notations used is shown in the form. By clicking the 'Next' button, the user is directed to the form (Fig.4.22.) containing output commands.

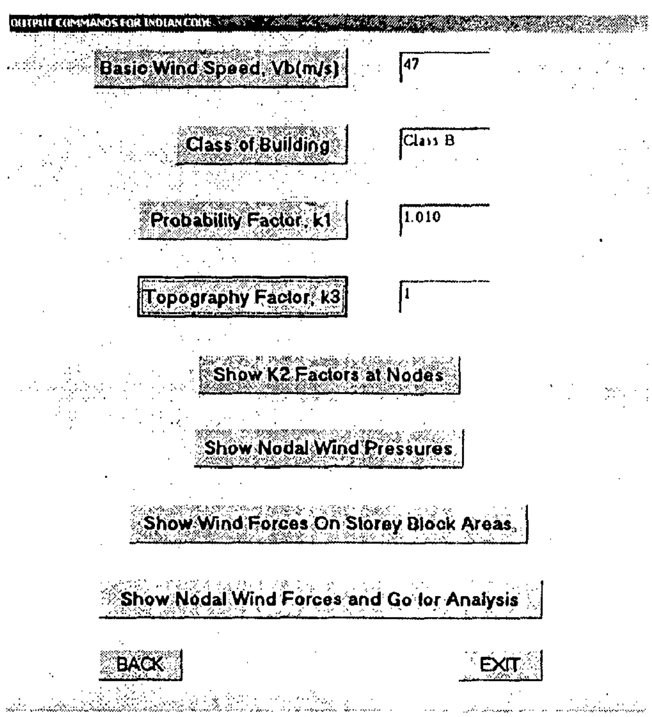


Fig.4.22. Output Form for Indian Code

The basic wind speed, class of the building, probability factor and topography factor are now one click away. The terrain factors, wind pressures, forces on storey block areas and wind loads are shown graphically (Fig.4.35. to Fig.4.38.) when the user clicks the corresponding buttons.

The input form for U.S.A. code is shown in Fig.4.23. The user has to enter the details in corresponding text boxes or combo boxes. A text box is provided for entering basic wind speed so that the basic wind speed considered for Indian code can be used entered. The program is basically intended for the comparison of wind loads as per various codes. Hence the basic wind speed used in Indian can be entered for all the remaining codes. The basic information to be input by the user for calculation of wind loads as per U.S.A. code is provided in the form. By clicking the 'Next' button, the user is directed to the form(Fig.4.24.) containing the input details needed for the calculation of topographic factor if the building is situated on a hill or escarpment else to the form(Fig.4.25.) containing output commands. The results are displayed as in the case of Indian code.

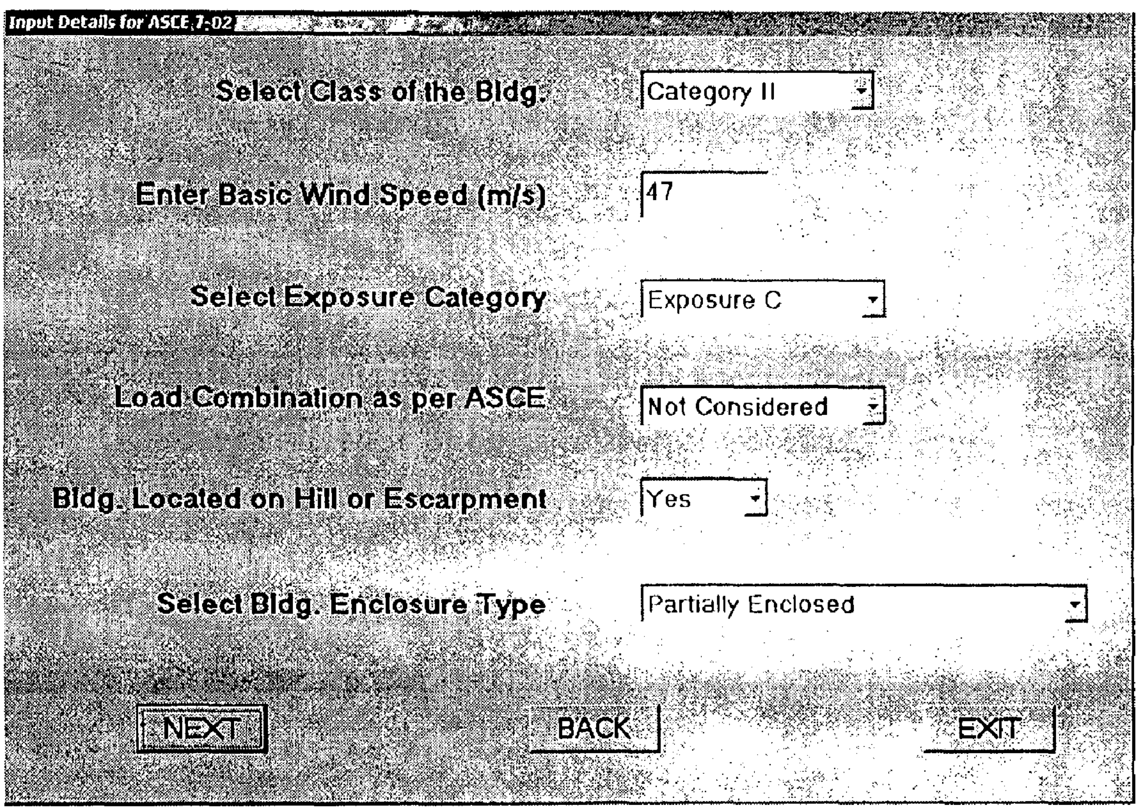


Fig.4.23. Input Details Form for U.S.A. Code

The user has to enter the type of topographic feature, location of the building on the feature and the distances as indicated in the figure shown in the form.

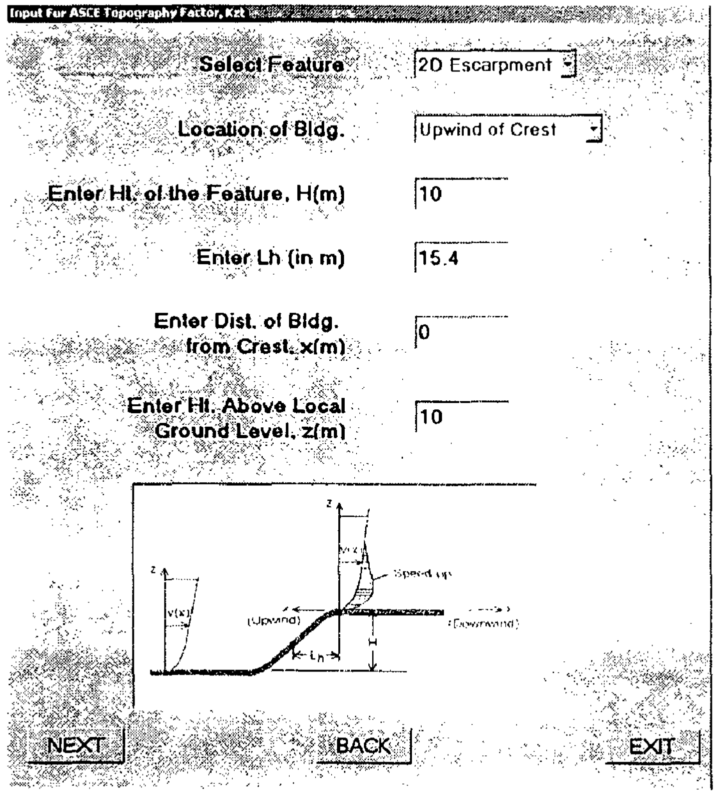


Fig.4.24. Input Form for Calculating Topography Factor as per U.S.A. Code

The output results for U.S.A. code consisting of wind directionality factor, importance factor, and topographic factor are displayed in the output form by clicking the corresponding buttons, terrain factors, wind pressures and loads are displayed in separate forms (Fig.4.35. to Fig.4.38.) graphically, as in the case of Indian code.

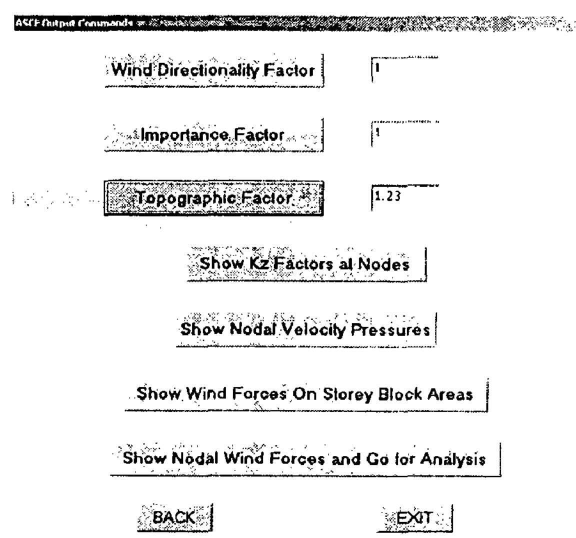


Fig.4.25. Output Form for U.S.A. Code

The input form for Australian code is shown in Fig.4.26. In this form option buttons are used for keeping the information required concise. Detailed description is provided in the form of tool tip texts on the form. Text is displayed when the cursor is moved on to the corresponding option. As per Australian code, the effect of shielding is included. The user has to choose whether the shielding effect is applicable or not.

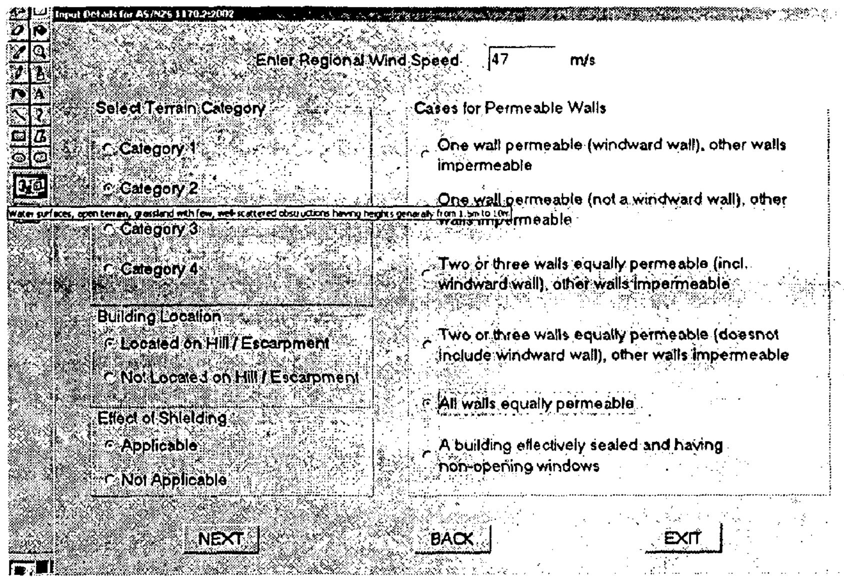


Fig.4.26. Input Details Form for Australian Code

If the effect of shielding is applicable, the user is directed to a form (Fig.4.27.) containing the details required for calculating the shielding factor when the user clicks 'Next' button. The user is required to enter the details in corresponding text boxes.

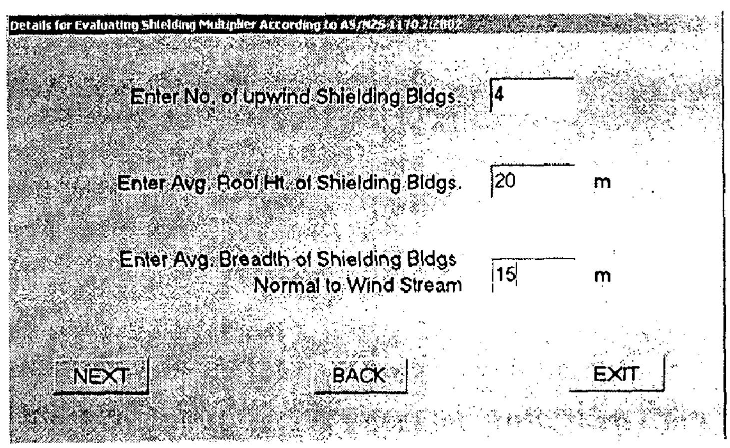


Fig.4.27. Input Form for Calculating Shielding Multiplier as per Australian Code

If the building is located on hill/escarpment, the details for calculating the topographic factor are to be entered in a separate form (Fig.4.28.). A picture is provided in the form in order to assist the user in entering the details of topographic feature and the location of the building on the feature.

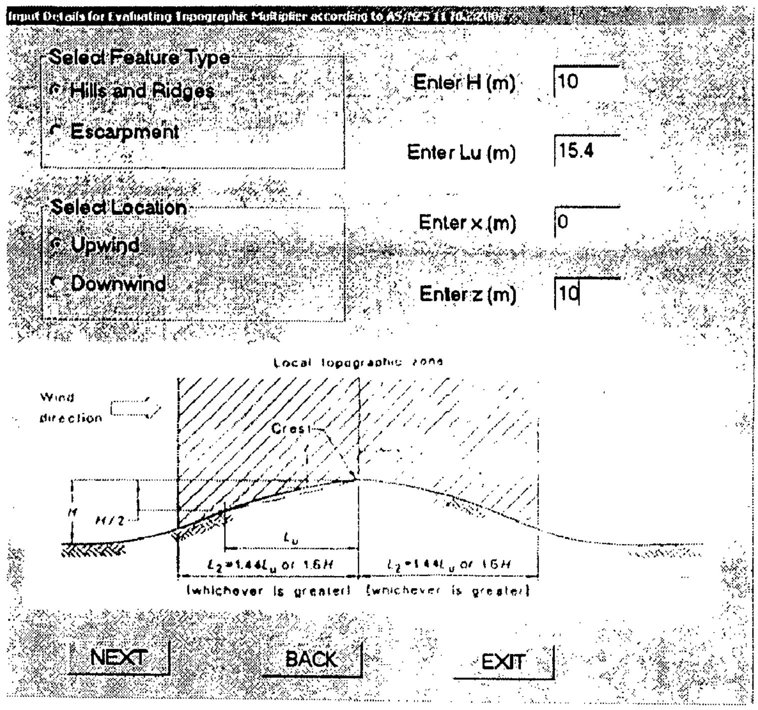


Fig.4.28. Input Form for Calculating Topography Factor as per Australian Code

The shielding factor and topographic multiplier are displayed by clicking the corresponding buttons. The terrain factors, wind pressures and loads are displayed graphically (Fig.4.35. to Fig.4.38.) by clicking the corresponding buttons as in the case of Indian code.

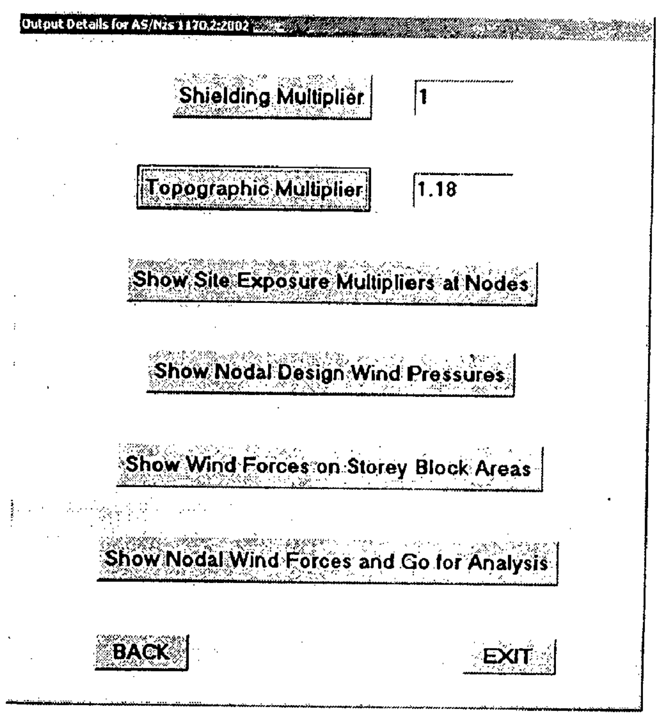


Fig.4.29. Output Form for Australian Code

The input form for British code is shown in Fig.4.30. The user has to enter the details in respective list boxes or combo boxes. In this form the details required for the calculation of topographic factor are included as much information is not needed for the calculation.

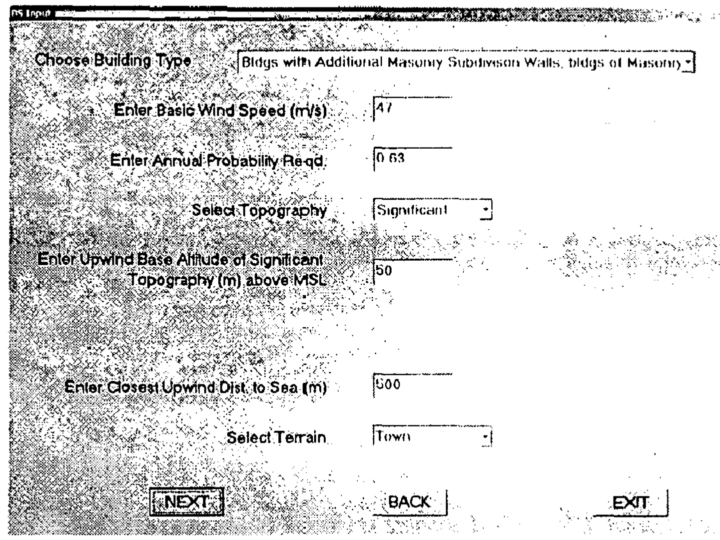


Fig.4.30. Input Details Form for British Code

By clicking the 'Next' button the user is directed to the output form (Fig.4.31.). The output results consists of dynamic augmentation factor, probability factor, altitude factor, site wind speed, the terrain and building factor at nodes, effective wind speed at nodes, design wind pressures and loads. The results are displayed graphically as in the case of Indian code (Fig.4.35. to Fig.4.38.)

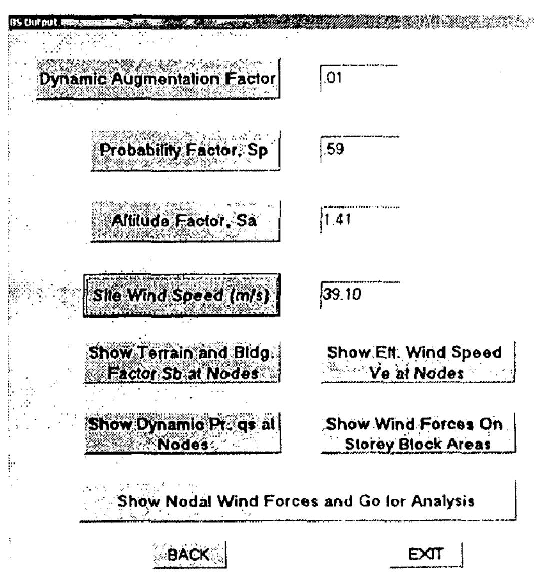


Fig.4.31. Output Form for British Code

The input form for Canadian code is as shown Fig.4.32. In this form also option buttons are used. If the building is located on hill/escarpment, the user is directed to a form (Fig.4.33.) containing the details required for the calculation of topographic factor.

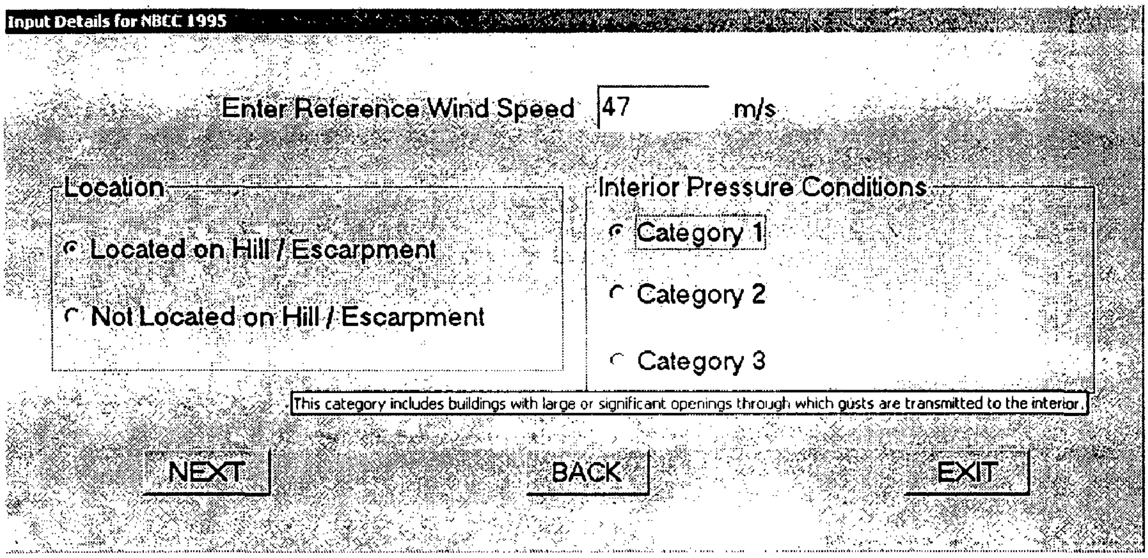


Fig.4.32. Input Details Form for Canadian Code

The type of topographic feature and the location of the building on that feature are to be selected from the corresponding option buttons. The distances to be entered are as indicated in the picture shown in the form.

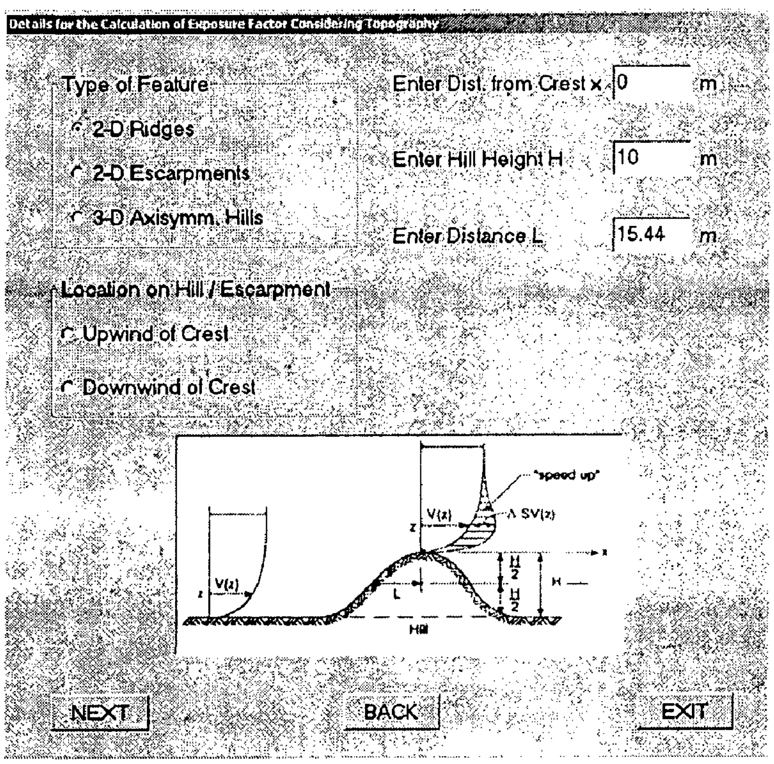


Fig.4.33. Input Form for Calculating Topography Factor as per Canadian Code

By clicking the 'Next' button, the user is directed to the output form (Fig.4.34.) by clicking the buttons in the form, results are displayed graphically in separate forms (Fig.4.35. to Fig.4.38.)

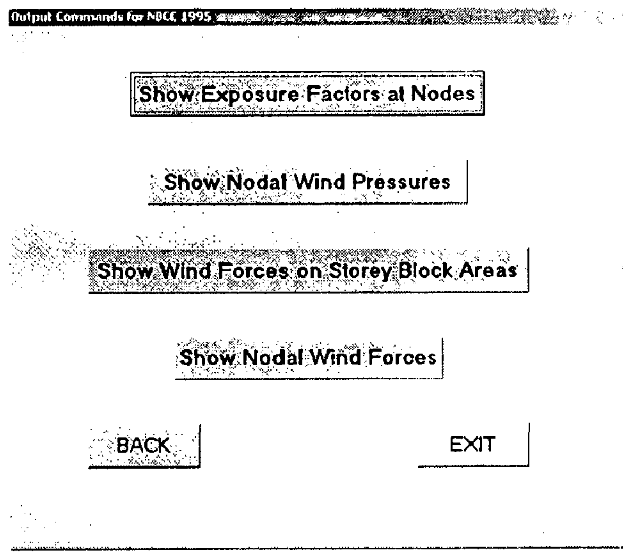


Fig.4.34. Output Form for Canadian Code

The terrain factors, wind pressures at nodes, wind forces on storey block areas and wind loads at nodes are displayed graphically. The frame displayed includes the storey-heights and bay-widths. The arrows displayed to indicate the values at nodes are drawn to a scale, so that the variation can be seen.

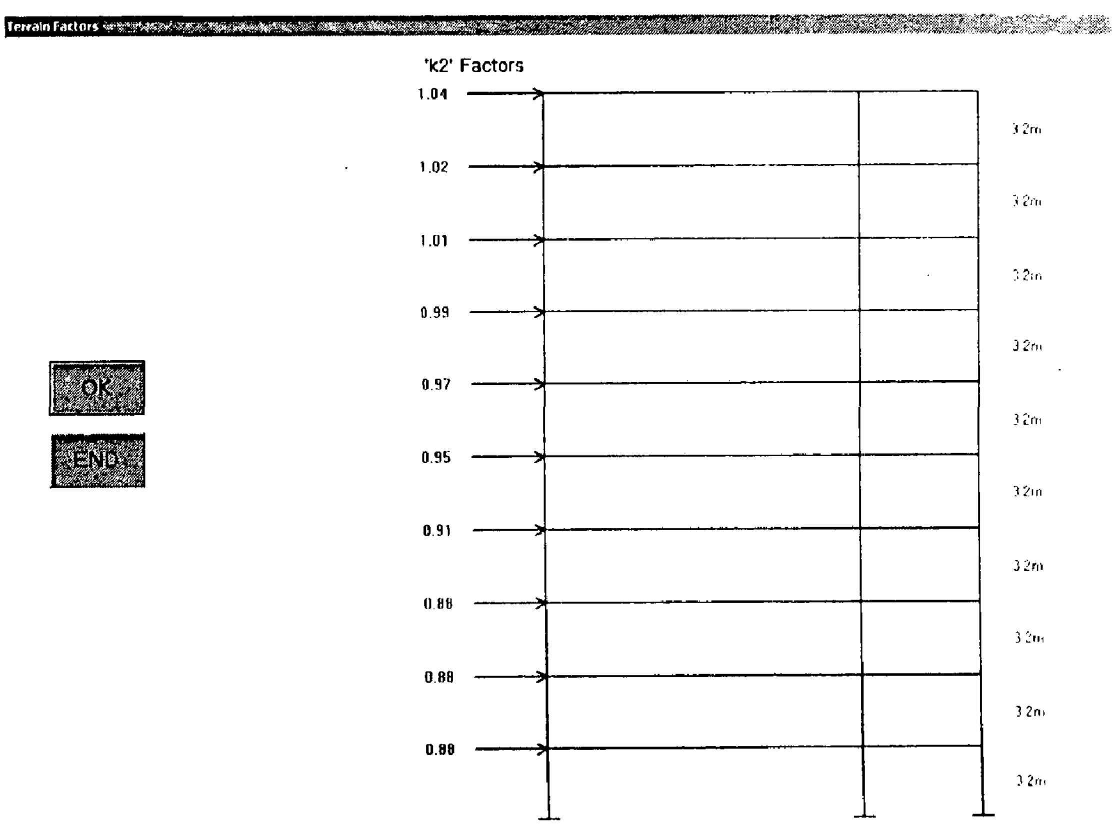


Fig.4.35. Form Displaying the Terrain Factors

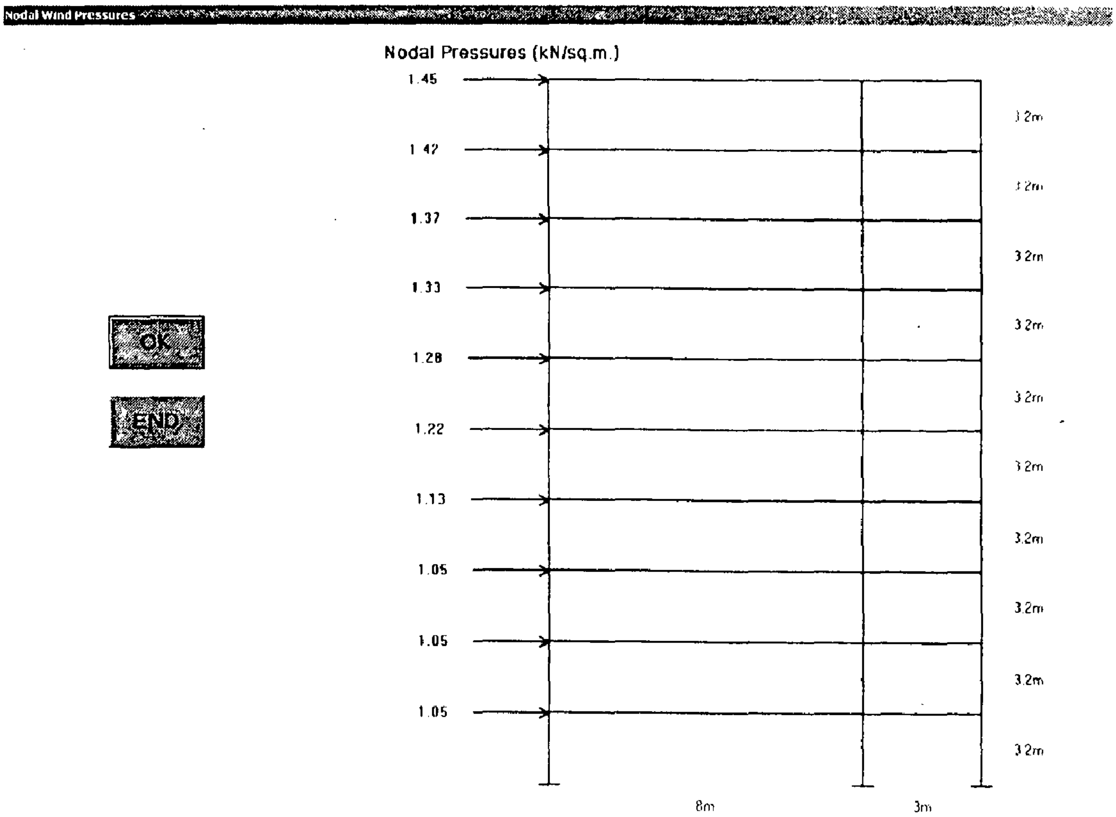


Fig.4.36. Form Displaying the Nodal Wind Pressures

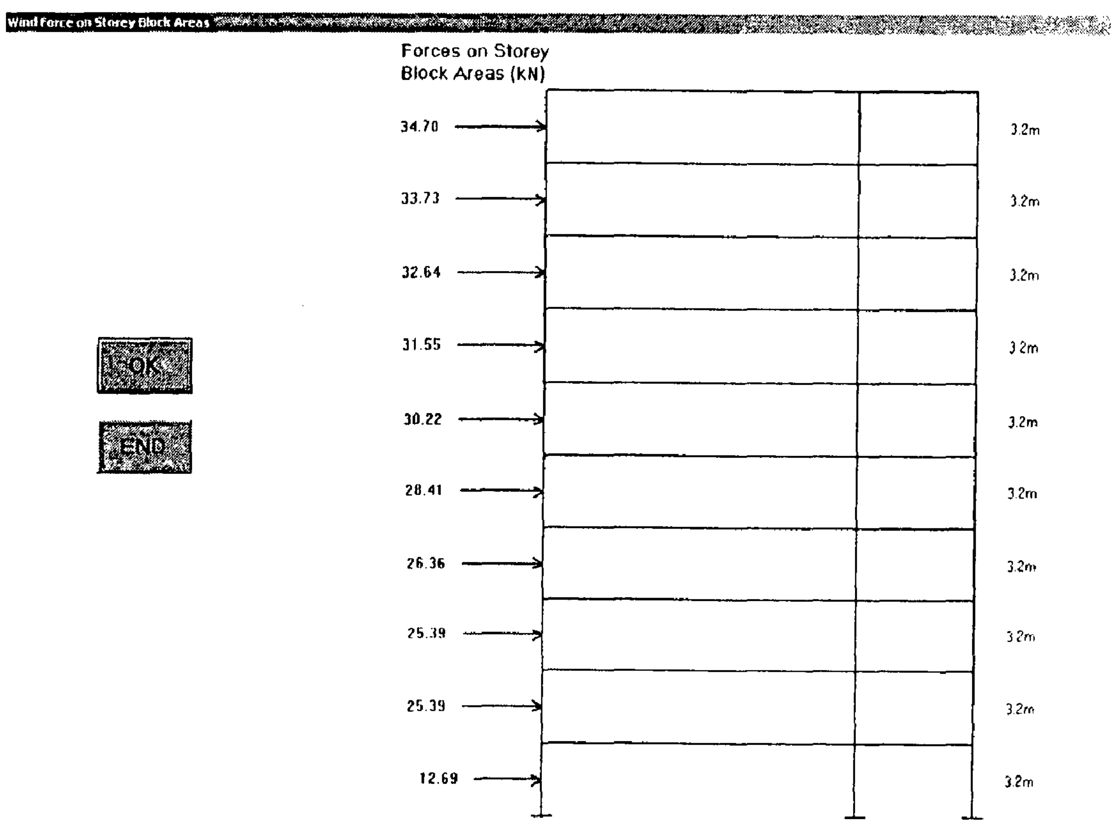


Fig.4.37. Form Displaying the Wind Forces on Storey Block Areas

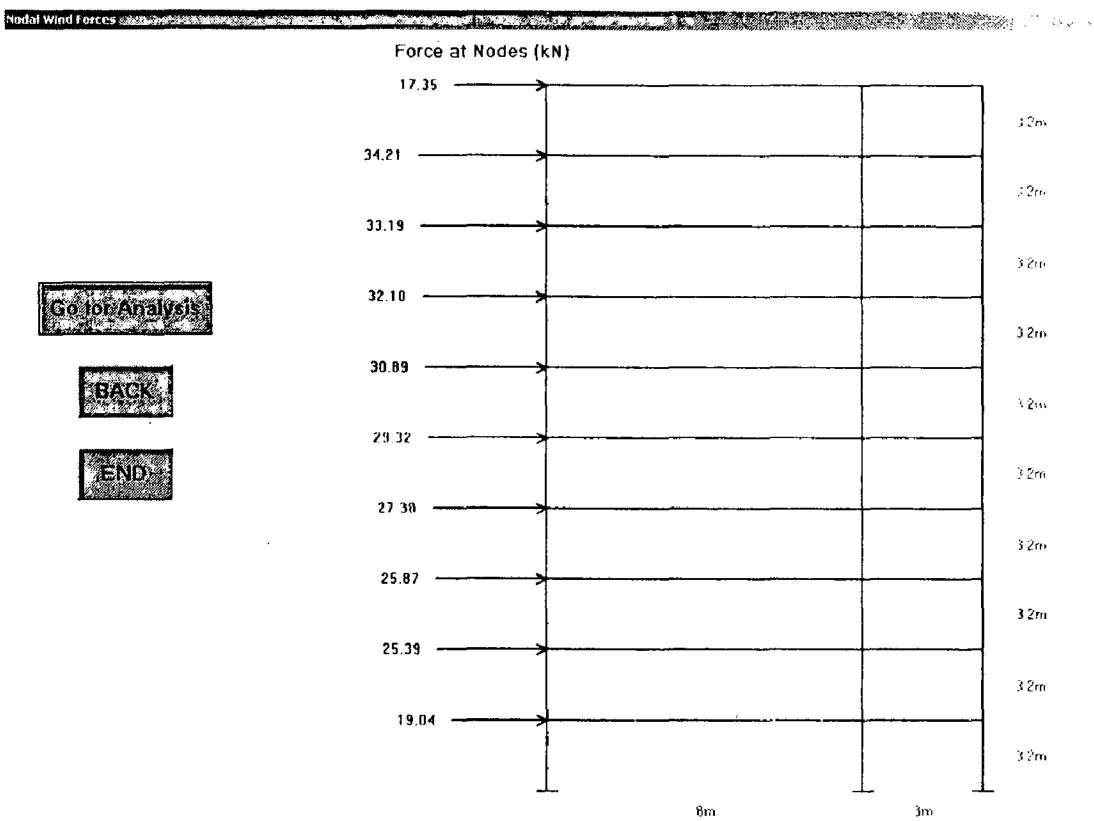


Fig.4.38. Form Displaying the Wind Loads at Nodes

In the form containing the wind loads at nodes (Fig.4.38.) there is a button named 'Go for Analysis'. By clicking this button the user is directed to a form (Fig.4.39.) containing the methods of analysis. The user has to choose a method for analysis from the four methods displayed on command buttons.

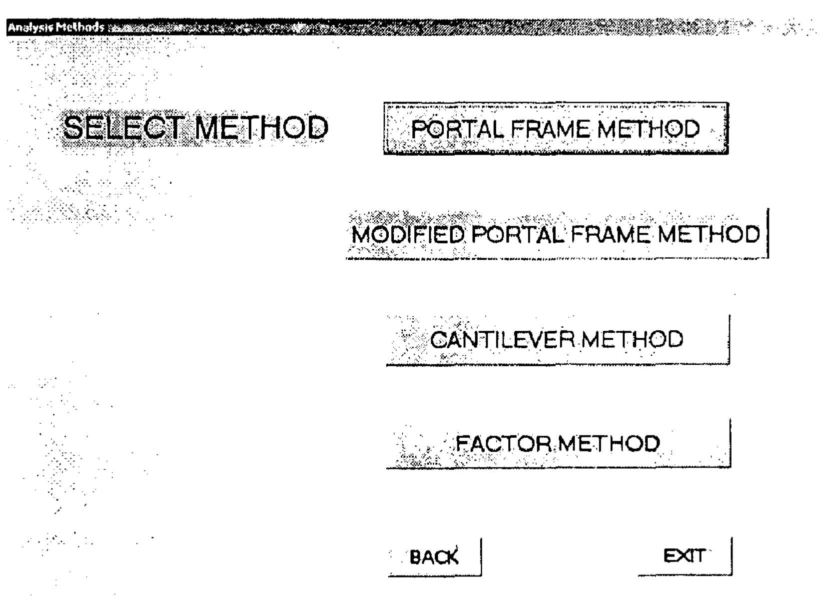


Fig.4.39. Form Displaying the Methods of Analysis

By clicking the buttons named 'Portal Frame Method' and 'Modified portal Frame Method', the end moments of all the beams and columns are displayed in a grid with numbering as indicated in a figure beside the grid (Fig.4.40).

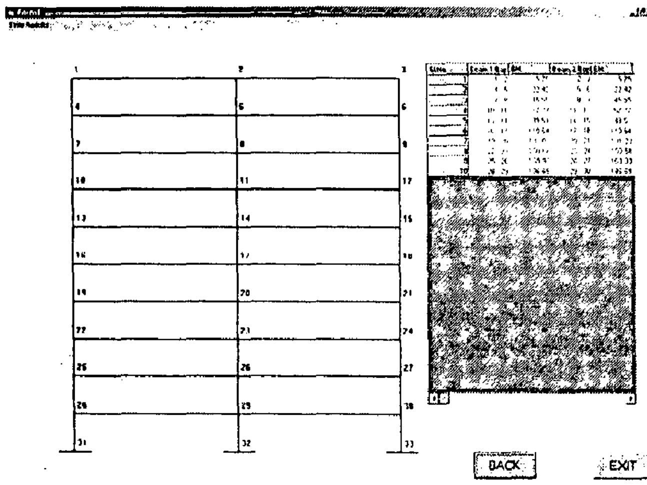


Fig.4.40. Form Displaying the End Moments in Columns and Beams in a Grid

End moments are displayed in the form of a grid under the headings of beams in each bay and the columns in each row. These results can also be saved in to a notepad (Fig.4.40.) by clicking the 'Save Results' button at the top left of the form.

If 'Cantilever Method' is chosen for the analysis, the user is directed to a form (Fig.4.41.) where the dimensions of columns need to be entered for all the stories. After entering the dimensions of columns, bending moment results are displayed in a grid (Fig.4.40.) and can be saved in to a notepad.

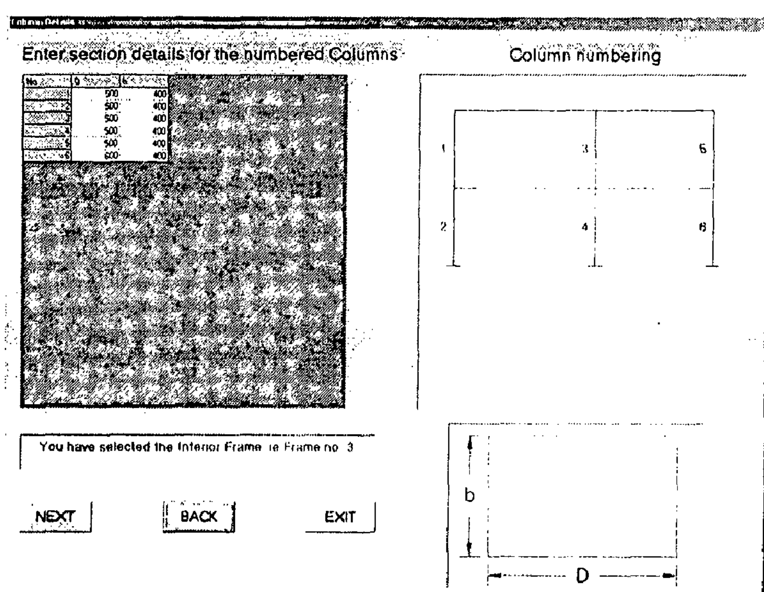


Fig.4.41. Form containing the grid for filling the column dimensions

If 'Factor Method' is chosen for the analysis, the user is directed to a form (Fig.4.42.)

where the dimensions of beams need to be entered for floor and roof. Then the form containing the grid in which column dimensions are to be entered for all stories (Fig.4.41.). Bending moment results are then displayed by clicking the next button and then can be saved in to a notepad.

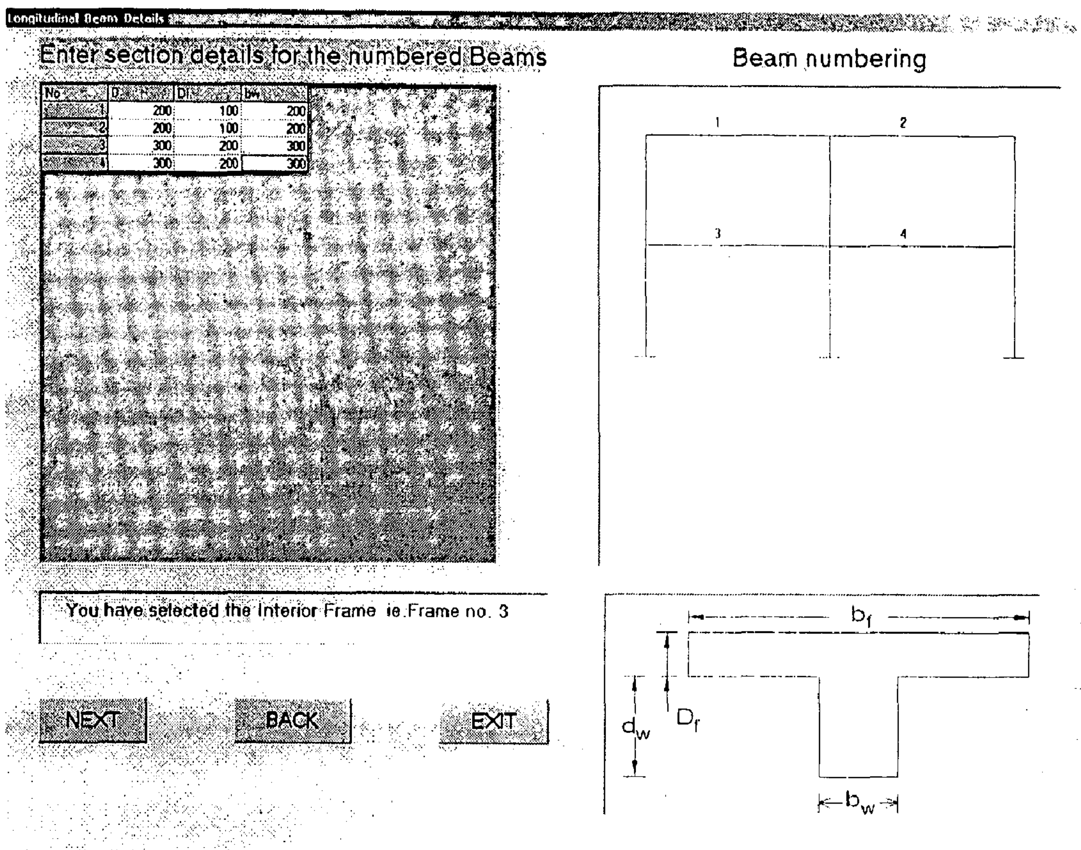


Fig.4.42. Form containing the grid for filling the Beam dimensions

This completes the program in all aspects. The program is made very user-friendly and included all the aspects required for the calculation of wind loads according to various wind loading codes and end moments in beams and columns according to various methods. Although this program is written basically for studying the effect of wind loads on gravity frames, this program can be used for many purposes. Wind loads can be generated without seeing the code and avoiding many calculations, which are time consuming. Analysis is done just by one click for any number of stories and any number of bays.

CHAPTER 5

CASE STUDIES

5.1 GENERAL

Theoretical treatments of methods of assessment of wind loads and methods of frame analysis have already been presented in chapter 3 and chapter 4 respectively, followed by their computer programs developed and discussed in chapter 4. In this chapter, it is intended to adopt suitable RC frames of structurally distinct characteristics for which the wind loads can be assessed using the code provisions of all the countries and analysis can be done using all the methods presented earlier. The results obtained by various codes and methods could then be studied with a view to inferring about the accuracy and applicability of all the code provisions and analysis methods.

Three wind speeds and four types of frames have been adopted for the study. The basic wind speeds considered were 33, 44 and 55 m/s. The number of stories of the frames considered was 15 and 17, and the number of bays considered was 2 and 3. The storey height for 2-bay frames was 3m and that of 3-bay frames was 3.2m.

RC floor beams are primarily designed for moments due to gravity loads whereas the columns are required to carry primarily axial loads with small moments. Thus, the axial loads in columns increase rapidly with the increasing numbers of stories whereas the beam moment remains more or less unchanged. Keeping this fact in mind, the column cross-sections have been changed after a few stories while keeping the beams sections unchanged.

5.2 LATERAL LOADS

Lateral loads were generated using the program developed for this study. All the wind load parameters, like location of the building, terrain characteristics, permeability of walls etc. were kept common and the wind speed is being changed. The loads thus developed for Indian code on all the frames, for a particular basic wind speed (44 m/s) were used for analysing the frames with the methods considered in this study. The loads lumped at storey levels according to all the codes were shown as base shear. The base shears as per all the other codes are compared with that of Indian code. The base shear as per Indian code is kept unit and the values from remaining codes are written as the multiples. Wind loads, as obtained by all the codes are plotted on graphs to denote the variation in values of loads. These are plotted for each frame and basic wind speed combination.

All the frames considered were sized for gravity loads. The materials used for the estimation of sizes of beams and columns were M25 grade concrete and Fe415 grade steel. Each frame is analysed by all the approximate methods for the wind loads obtained by the Indian code for a basic wind speed of 44 m/s. The moments thus obtained are tabulated, separately for beams and columns.

Identification of beams and columns is done by numbering them from top to bottom. The moments obtained for beams by all the methods considered are tabulated separately and those of columns are tabulated separately. As 'Factor Method' is treated as the best method over the other approximate methods due to its consideration of the sizes of beams and columns, it is selected as the base method for comparing the values of moments obtained from the remaining analysis methods.

FRAME 1: 15 Stories – 2 Bays

Storey height = 3m

Bay widths – 8m, 3m

Frame spacing – 4m

Terrain category – Category 4 (Terrain with numerous large high closely spaced obstructions)

Desired life of structure – 100 years

Return period – 100 years

Permeability of cladding – medium (16% on windward face and 18% on leeward face)

Sizes of columns – 600x400, 600x400 and 300x300 for stories 1 to 5
500x400, 500x400 and 300x300 for stories 6 to 10 and
500x300, 500x300 and 300x300 for stories 11 to 15

Sizes of beam webs - 250x250mm for span of 4.0m
450x250mm for span of 8.0m

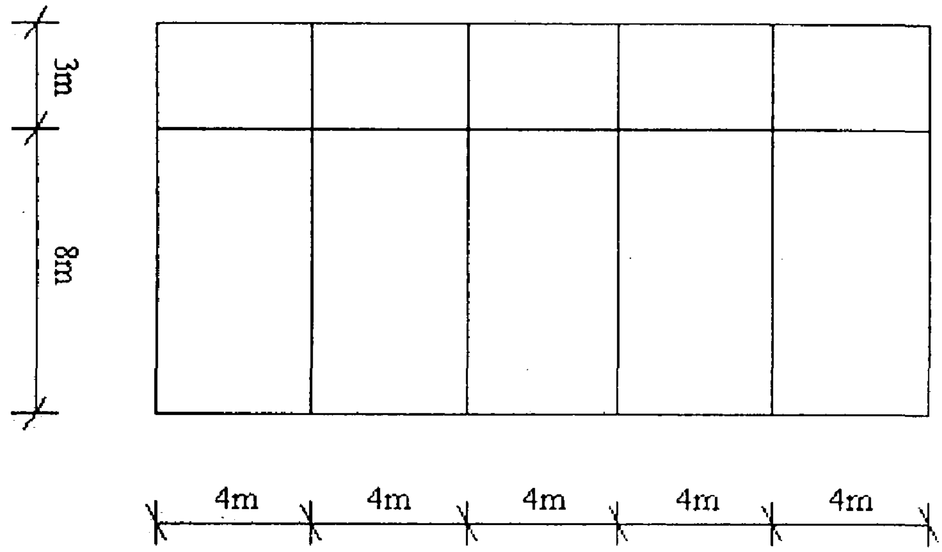


Fig.5.1. Plan of 15s – 2b Frame

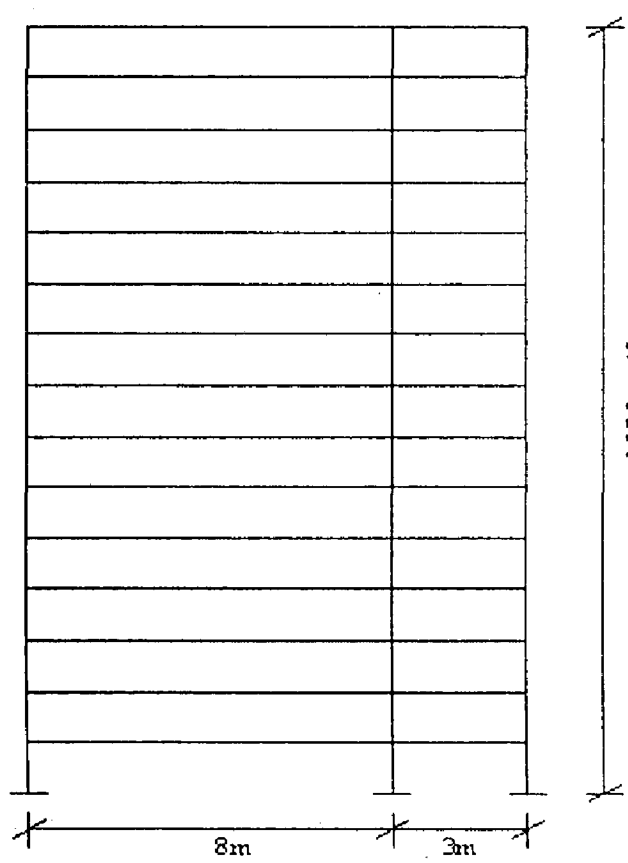


Fig.5.2. Elevation of 15s – 2b Frame

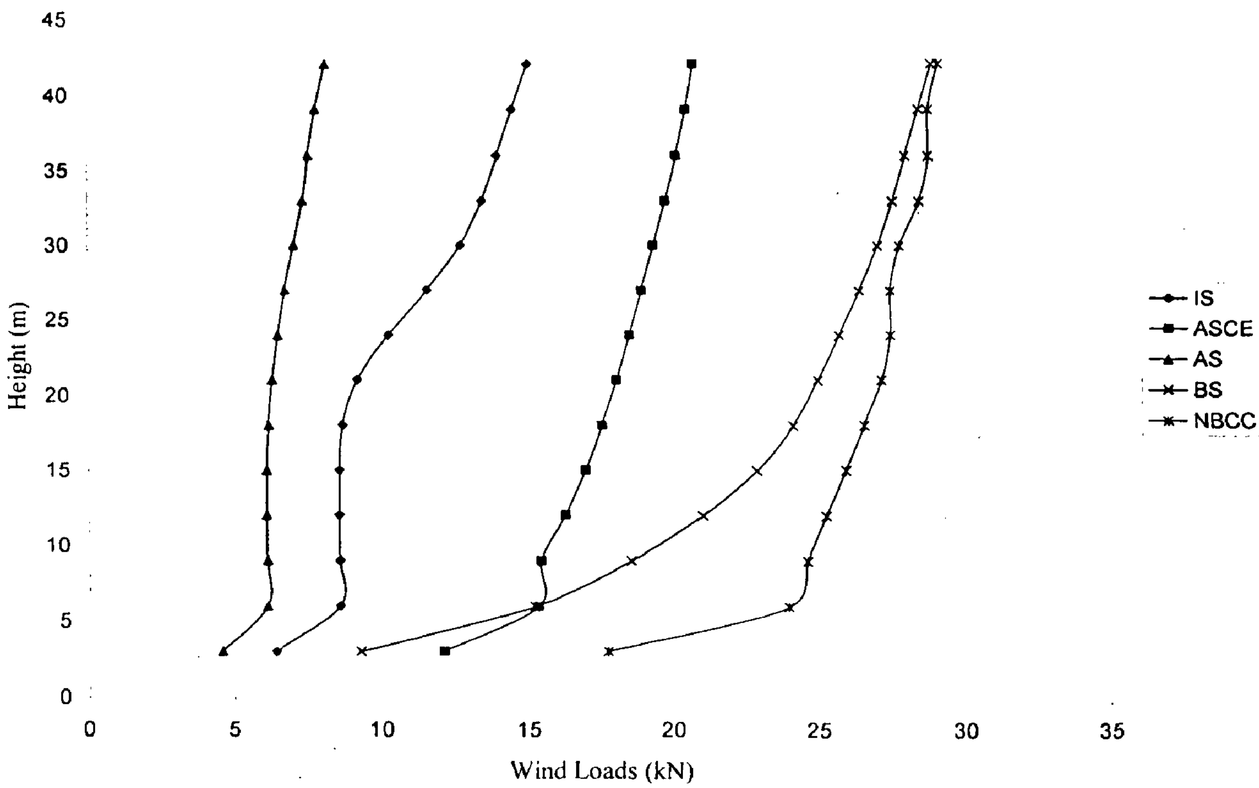


Fig.5.3 Wind Load distribution on 15s – 2b frame as per various codes (basic wind speed = 33m/s)

Table 5.1: Base Shear for 15s-2b frame as per various codes (Basic wind speed = 33 m/s)

Code / Term	IS	ASCE	AS	BS	NBCC
Base Shear (kN)	157.9	260.16	97.05	342.84	384.03
Relative Base Shear with IS Value reduced to unity	1.0	1.65	0.61	2.17	2.43

- The base shear given by NBCC is the largest followed by the values obtained from BS, ASCE, IS and AS in descending order.
- The base shear given by AS, only is lesser than that given by IS.

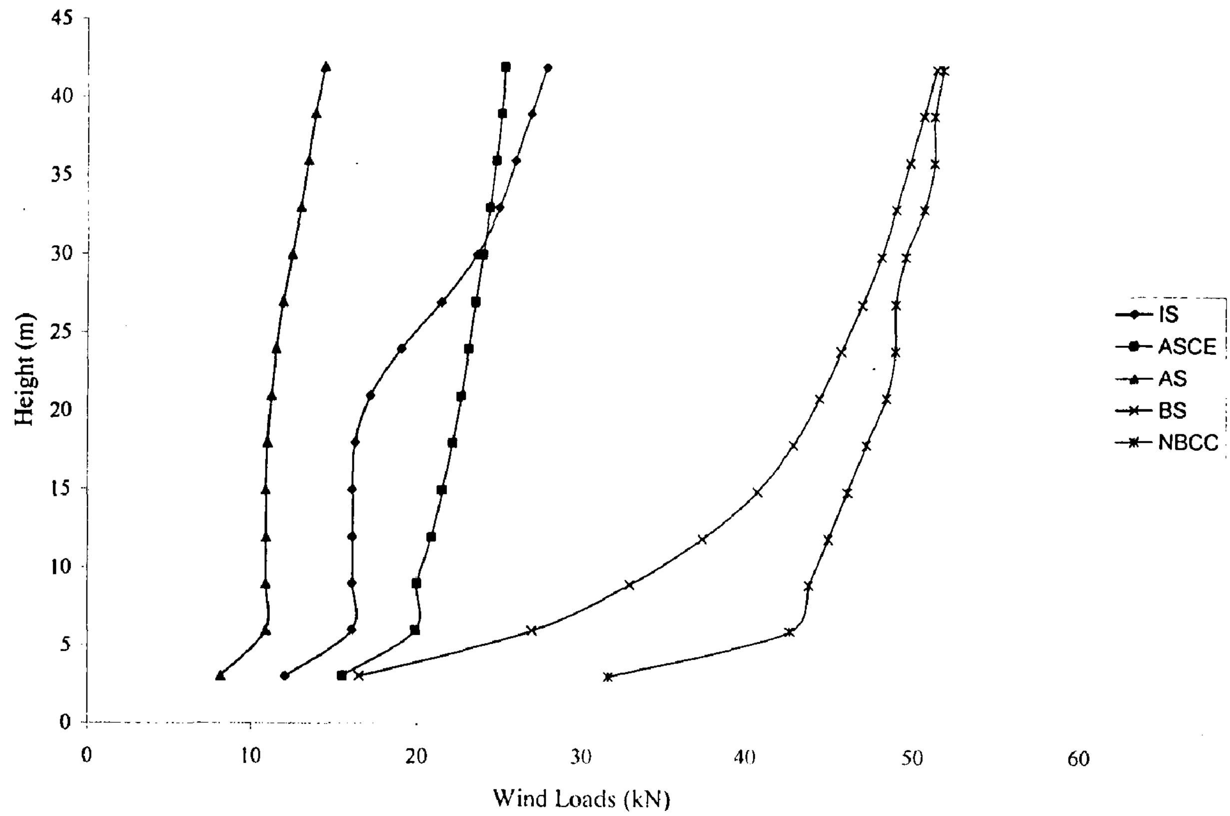


Fig.5.4 Wind Load distribution on 15s – 2b frame as per various codes (basic wind speed = 44m/s)

Table 5.2: Base Shear for 15s-2b frame as per various codes (Basic wind speed = 44 m/s)

Code / Term	IS	ASCE	AS	BS	NBCC
Base Shear (kN)	294.11	325.74	172.23	609.37	683.37
Relative Base Shear with IS Value reduced to unity	1.0	1.11	0.59	2.07	2.32

- The base shear given by NBCC is the largest followed by the values obtained from BS, ASCE, IS and AS in descending order.
- The base shear given by AS, only is lesser than that given by IS.

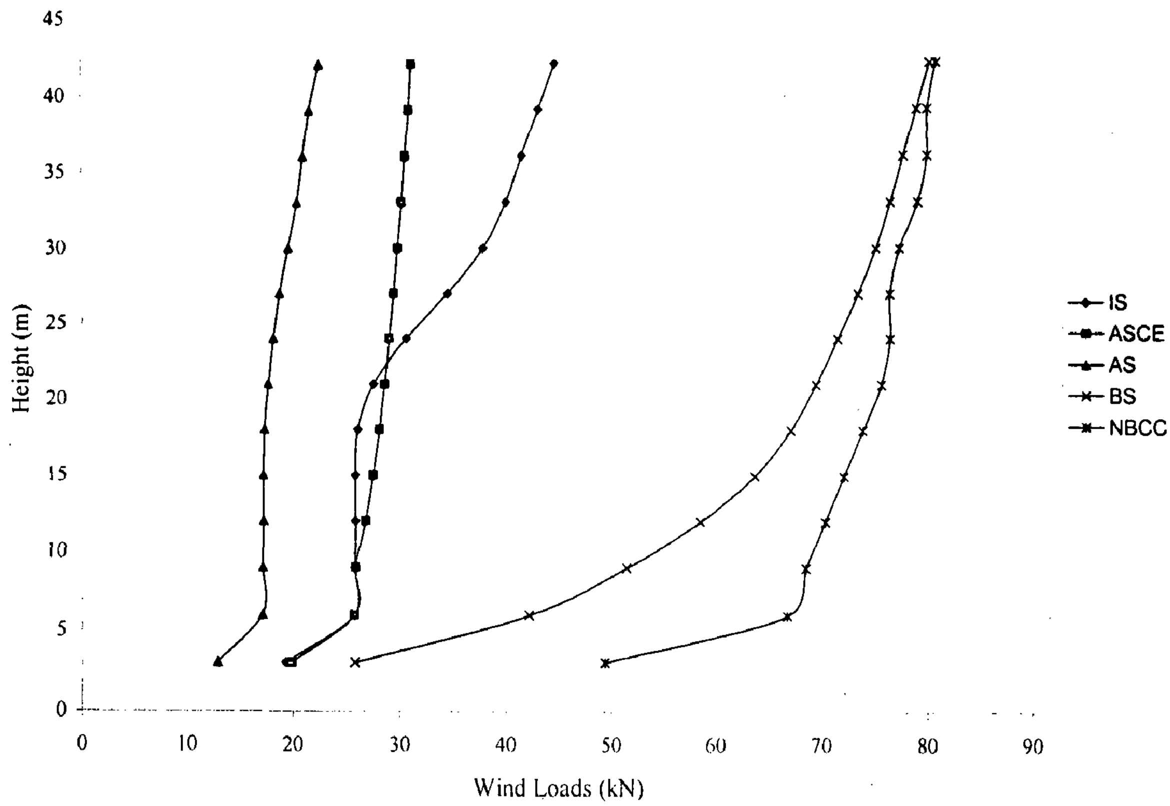


Fig.5.5 Wind Load distribution on 15s – 2b frame as per various codes (basic wind speed = 55 m/s)

Table 5.3: Base Shear for 15s-2b frame as per various codes (Basic wind speed = 44 m/s)

Code / Term	IS	ASCE	AS	BS	NBCC
Base Shear (kN)	470.95	409.29	269.97	952.03	1067.61
Relative Base Shear with IS Value reduced to unity	1.0	0.87	0.57	2.02	2.27

- The base shear given by NBCC is the largest followed by the values obtained from BS, ASCE, IS and AS in descending order.
- The base shear given by AS, only is lesser than that given by IS.

FRAME 2: 17 Stories – 2 Bays

Storey height = 3m

Bay widths – 8m, 3m

Frame spacing – 4m

Terrain category – Category 4 (Terrain with numerous large high closely spaced obstructions)

Desired life of structure – 100 years

Return period – 100 years

Permeability of cladding – medium (16% on windward face and 18% on leeward face)

Sizes of columns – 600x400, 600x400 and 300x300 for stories 1 to 5
500x400, 500x400 and 300x300 for stories 6 to 11 and
500x300, 500x300 and 300x300 for stories 12 to 15

Sizes of beam webs - 250x250mm for span of 4.0m
450x250mm for span of 8.0m

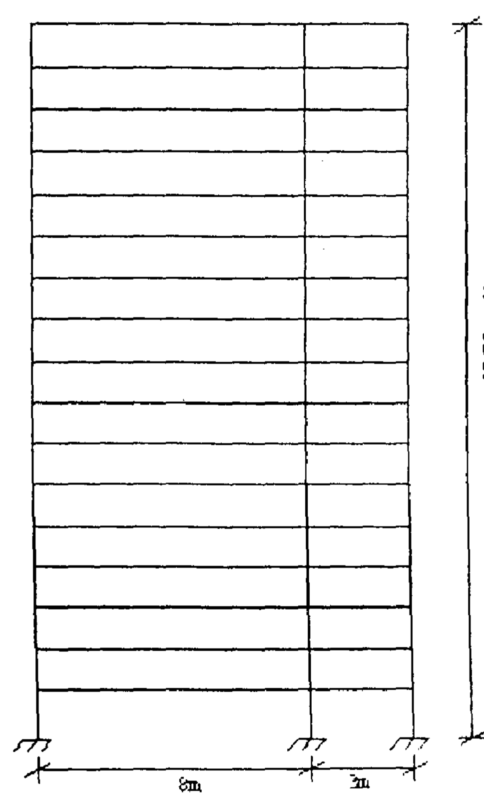


Fig.5.6. Elevation of 17s – 2b Frame

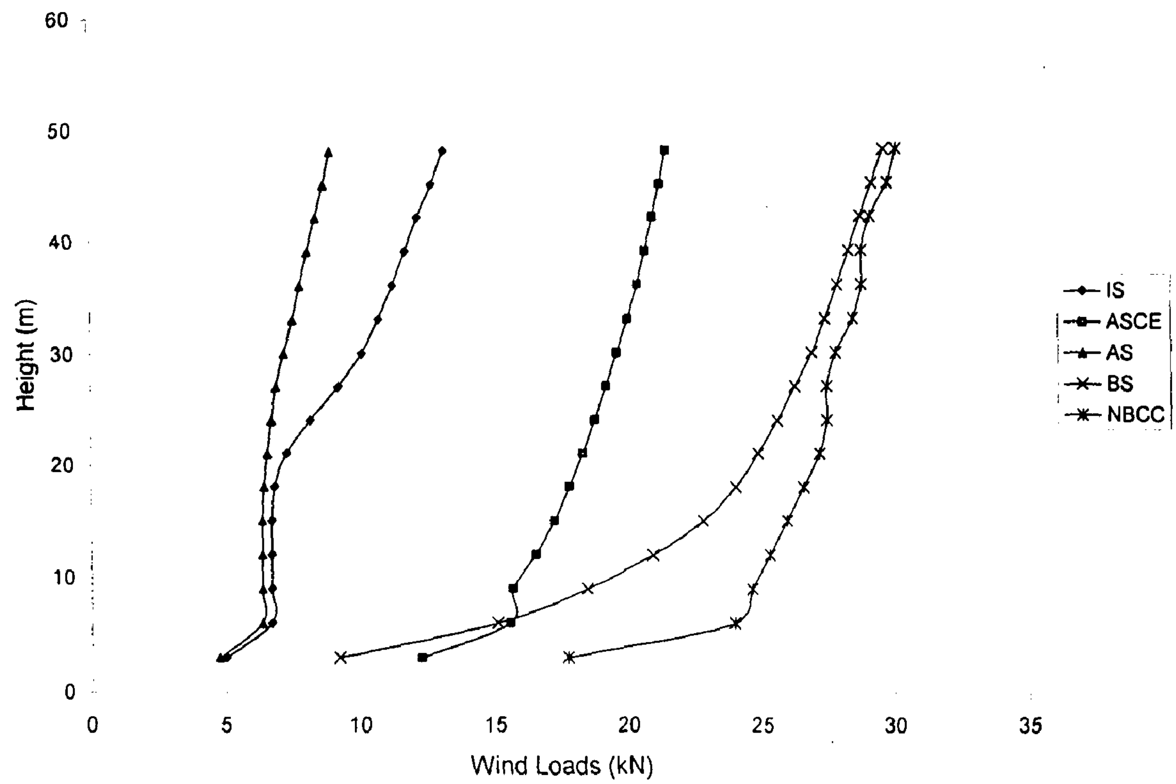


Fig.5.7 Wind Load distribution on 17s – 2b frame as per various codes (basic wind speed = 33m/s)

Table 5.4: Base Shear for 17s – 2b frame as per various codes (Basic wind speed = 33 m/s)

Code / Term	IS	ASCE	AS	BS	NBCC
Base Shear (kN)	151.98	306.42	118.05	400.28	444.27
Relative Base Shear with IS Value reduced to unity	1.0	2.02	0.78	2.63	2.92

- The base shear given by NBCC is the largest followed by the values obtained from BS, ASCE, IS and AS in descending order.
- The base shear given by AS, only is lesser than that given by IS.

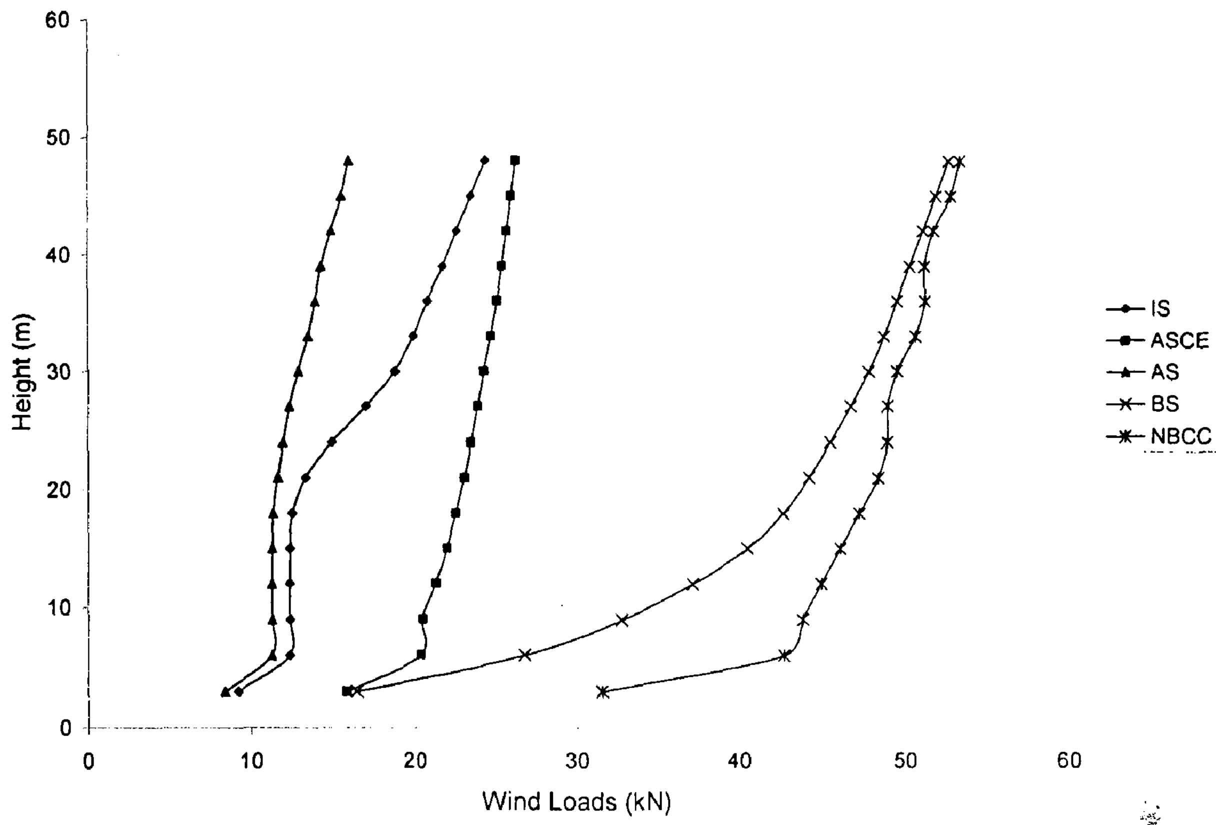


Fig.5.8 Wind Load distribution on 17s – 2b frame as per various codes (basic wind speed = 44m/s)

Table 5.5: Base Shear for 17s – 2b frame as per various codes (Basic wind speed = 44 m/s)

Code / Term	IS	ASCE	AS	BS	NBCC
Base Shear (kN)	280.58	383.85	209.82	711.47	790.17
Relative Base Shear with IS Value reduced to unity	1.0	1.37	0.75	2.54	2.82

- The base shear given by NBCC is the largest followed by the values obtained from BS, ASCE, IS and AS in descending order.
- The base shear given by AS, only is lesser than that given by IS.

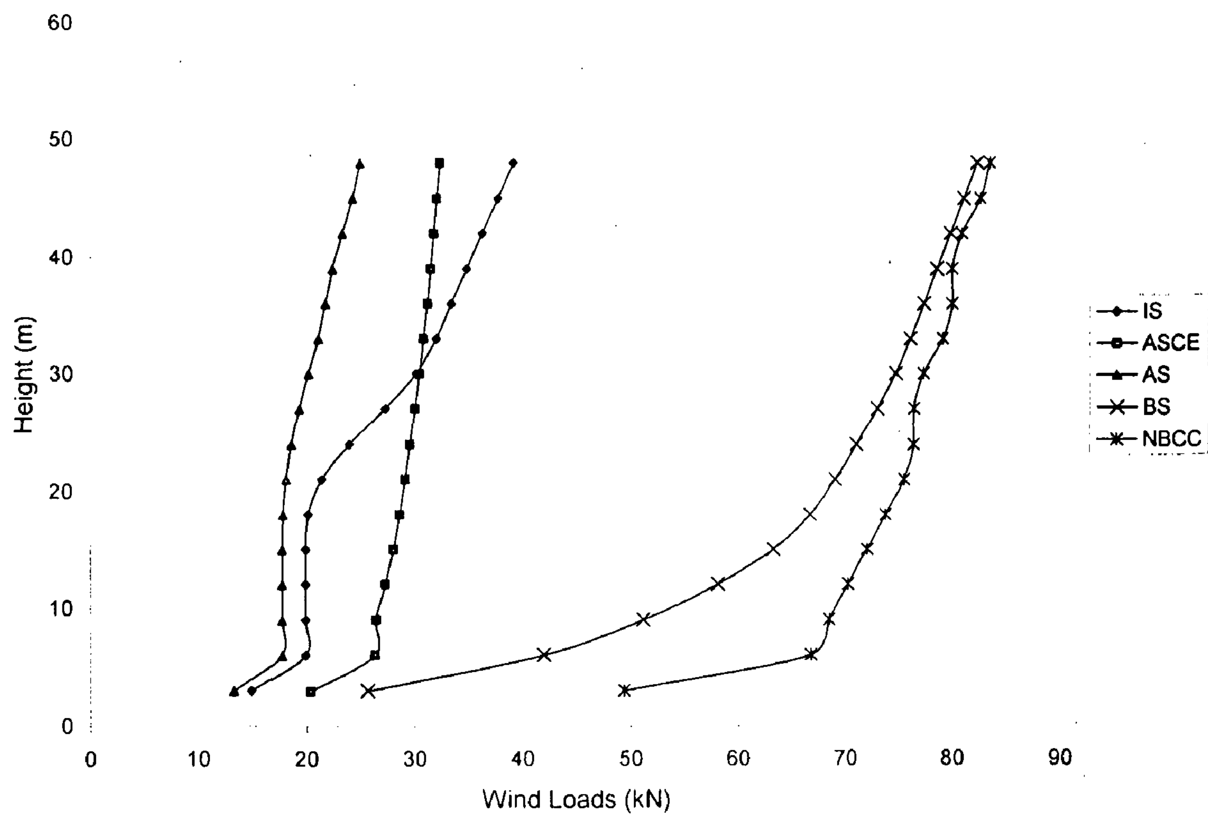


Fig.5.9 Wind Load distribution on 17s – 2b frame as per various codes (basic wind speed = 55m/s)

Table 5.6: Base Shear for 17s – 2b frame as per various codes (Basic wind speed = 55 m/s)

Term \ Code	IS	ASCE	AS	BS	NBCC
Base Shear (kN)	450.61	482.58	328.56	1111.54	1234.65
Relative Base Shear with IS Value reduced to unity	1.0	1.07	0.73	2.47	2.74

- The base shear given by NBCC is the largest followed by the values obtained from BS, ASCE, IS and AS in descending order.
- The base shear given by AS, only is lesser than that given by IS.
- The base shear given by ASCE is very near to that given by IS

FRAME 3: 15 Stories – 3 Bays

Storey height – 3m

Bay widths – 9m, 3m, and 6m

Frame spacing – 4m (9 @ 4m each)

Terrain category – Category 4 (Terrain with numerous large high closely spaced obstructions)

Desired life of structure – 100 years

Return period – 100 years

Permeability of cladding – medium (16% on windward face and 18% on leeward face)

Sizes of columns – 600x500, 600x500, 600x400 and 600x400 for stories 1 to 5
600x400, 600x400, 500x400 and 500x400 for stories 6 to 10
600x300, 600x300, 500x300, 500x300 for stories 11 to 15

Sizes of beam webs - 500x250mm for span of 9.0m
450x250mm for span of 6.0m
250x250mm for span of 3.0m

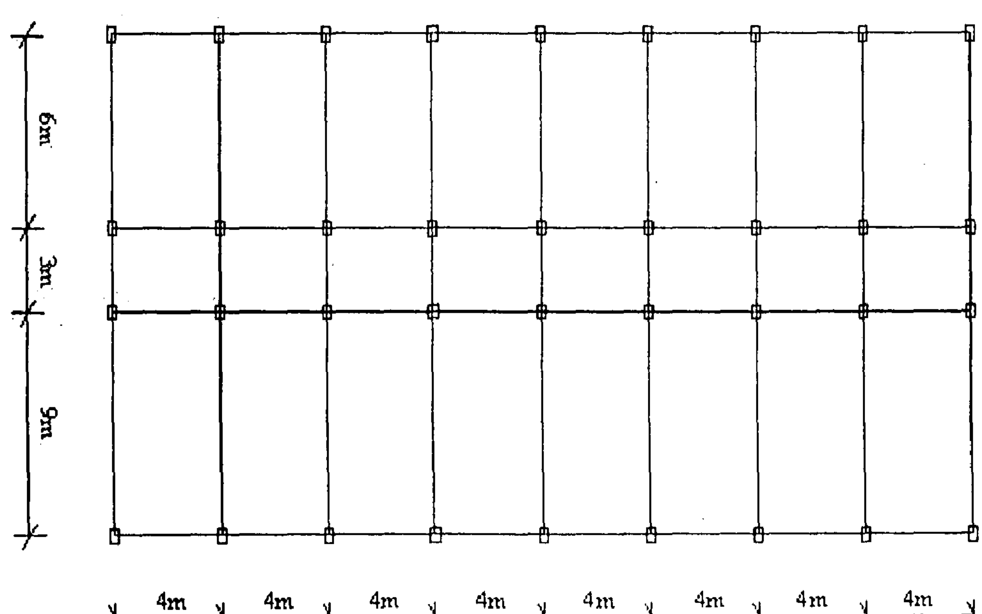


Fig.5.10. Plan of 15s – 3b Frame

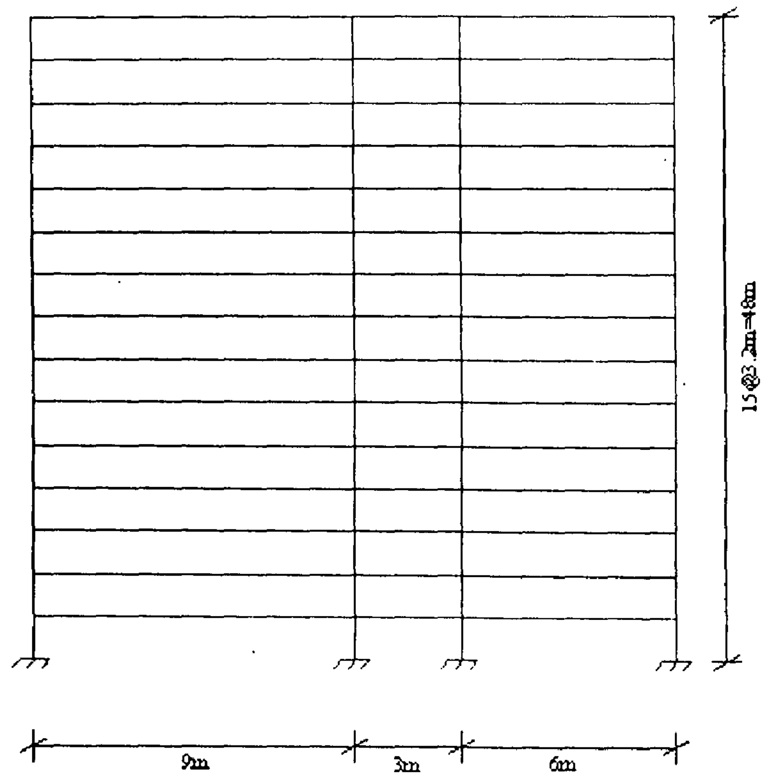


Fig.5.11. Elevation of 15s – 3b Frame

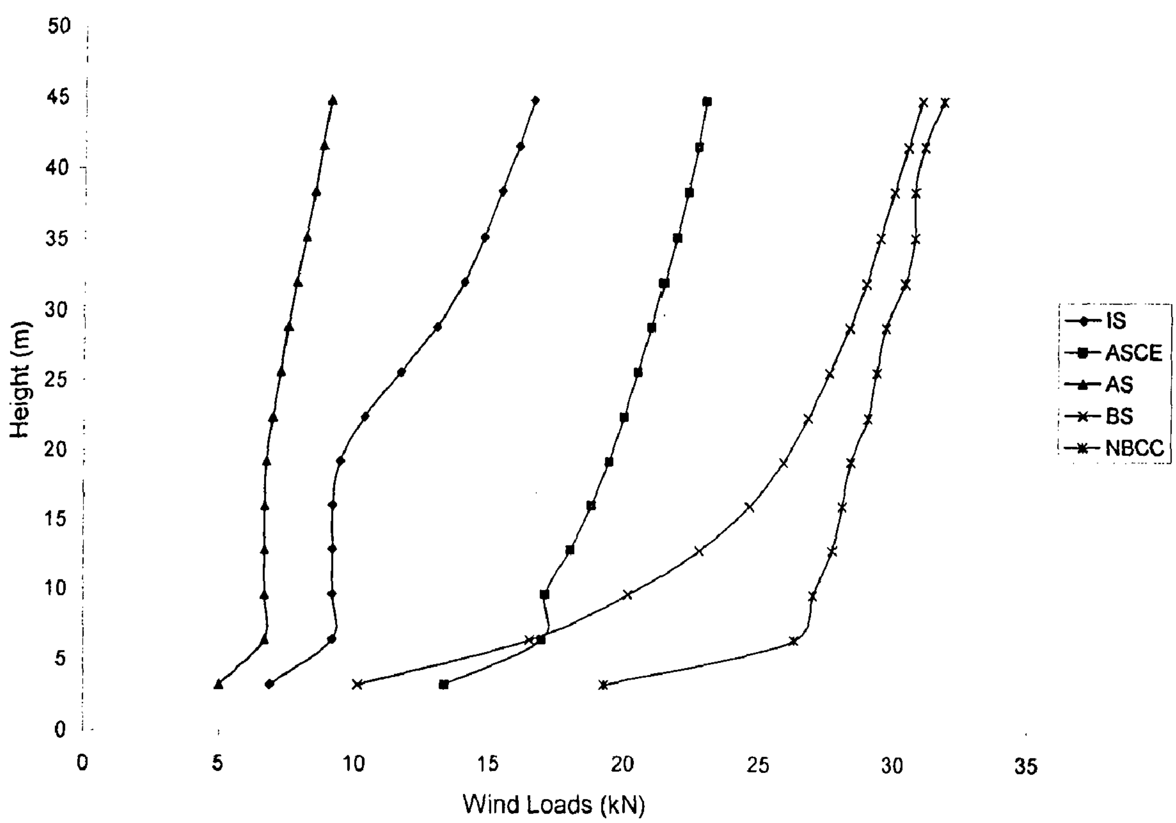


Fig.5.12 Wind Load distribution on 15s – 3b frame as per various codes (basic wind speed = 33m/s)

Table 5.7: Base Shear for 15s – 3b frame as per various codes (Basic wind speed = 33 m/s)

Code / Term	IS	ASCE	AS	BS	NBCC
Base Shear (kN)	173.25	287.62	106.9	367.91	415.26
Relative Base Shear with IS Value reduced to unity	1.0	1.66	0.62	2.12	2.4

- The base shear given by NBCC is the largest followed by the values obtained from BS, ASCE, IS and AS in descending order.
- The base shear given by AS, only is lesser than that given by IS.

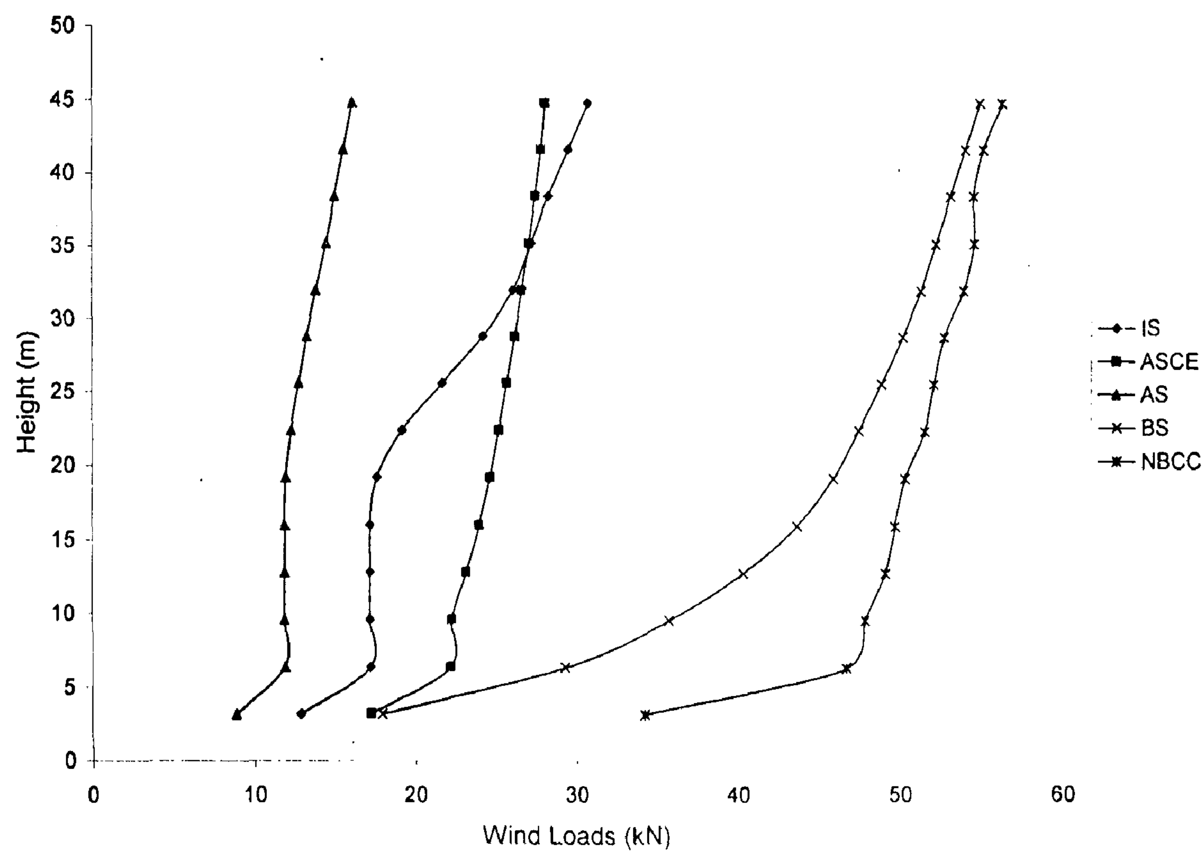


Fig.5.13 Wind Load distribution on 15s – 3b frame as per various codes (basic wind speed = 44m/s)

Table 5.8: Base Shear for 15s – 3b frame as per various codes (Basic wind speed = 44 m/s)

Term \ Code	IS	ASCE	AS	BS	NBCC
Base Shear (kN)	322.56	362.91	190.11	653.94	738.53
Relative Base Shear with IS Value reduced to unity	1.0	1.13	0.59	2.03	2.29

- The base shear given by NBCC is the largest followed by the values obtained from BS, ASCE, IS and AS in descending order.
- The base shear given by AS, only is lesser than that given by IS.

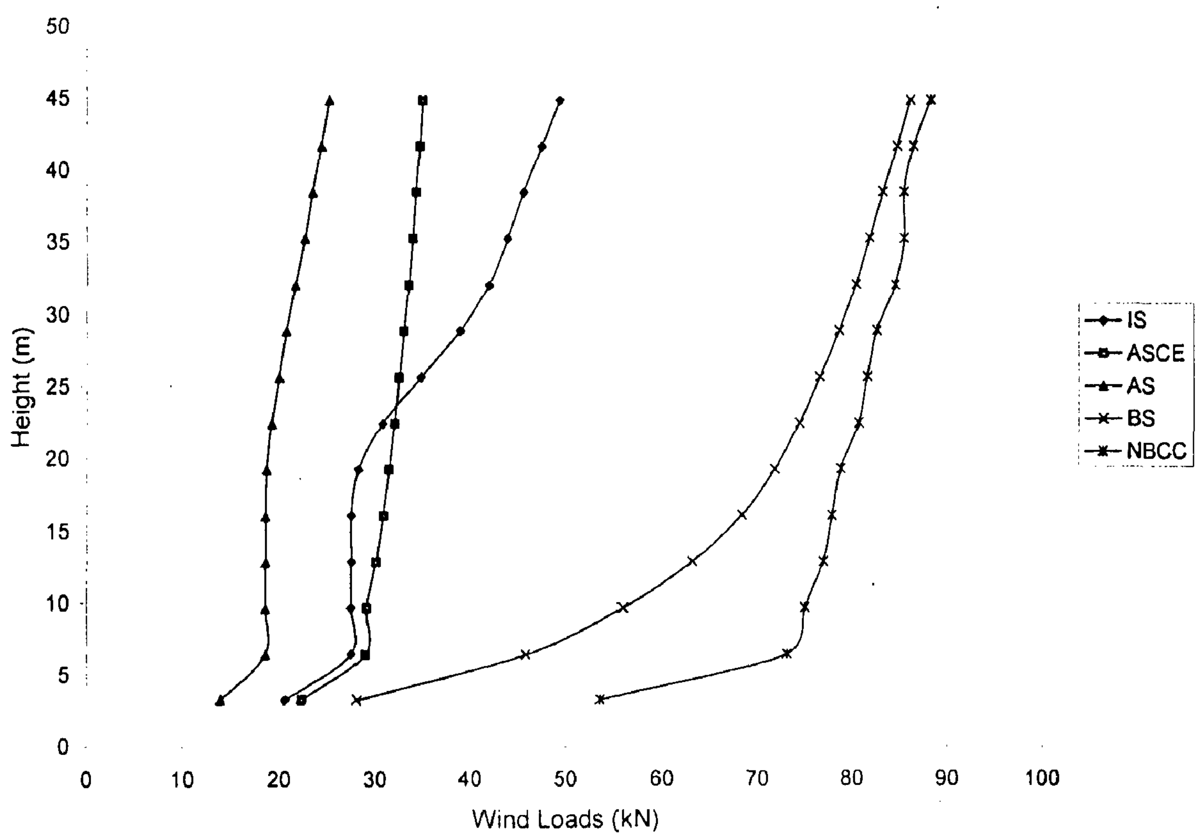


Fig.5.14 Wind Load distribution on 15s – 3b frame as per various codes (basic wind speed = 55m/s)

Table 5.9: Base Shear for 15s – 3b Frame as per various codes (Basic Wind Speed = 55 m/s)

Term \ Code	IS	ASCE	AS	BS	NBCC
Base Shear (kN)	516.43	458.85	296.74	1021.66	1153.89
Relative Base Shear with IS Value reduced to unity	1.0	0.89	0.57	1.98	2.23

- The base shear given by NBCC is the largest followed by the values obtained from BS, IS, ASCE and AS in descending order.
- The base shear given by AS and ASCE is lesser than that given by IS.

FRAME 4: 17 Stories – 3 Bays

Storey height = 3m

Bay widths – 9m, 3m, and 6m

Frame spacing – 4m (9 @ 4m)

Terrain category – Category 4 (Terrain with numerous large high closely spaced obstructions)

Desired life of structure – 100 years

Return period – 100 years

Permeability of cladding – medium (16% on windward face and 18% on leeward face)

Sizes of columns – 600x500, 600x500, 600x400 and 600x400 for stories 1 to 5
600x400, 600x400, 500x400 and 500x400 for stories 6 to 11
600x300, 600x300, 500x300, 500x300 for stories 12 to 17

Sizes of beam webs - 500x250mm for span of 9.0m
450x250mm for span of 6.0m
250x250mm for span of 3.0m

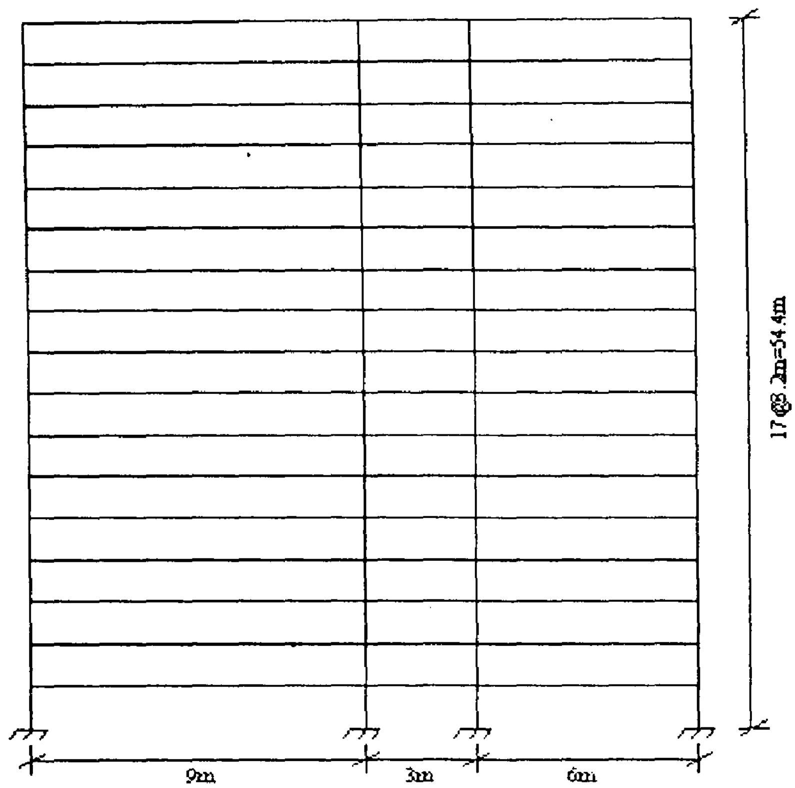


Fig.5.15. Elevation of 17s – 3b Frame

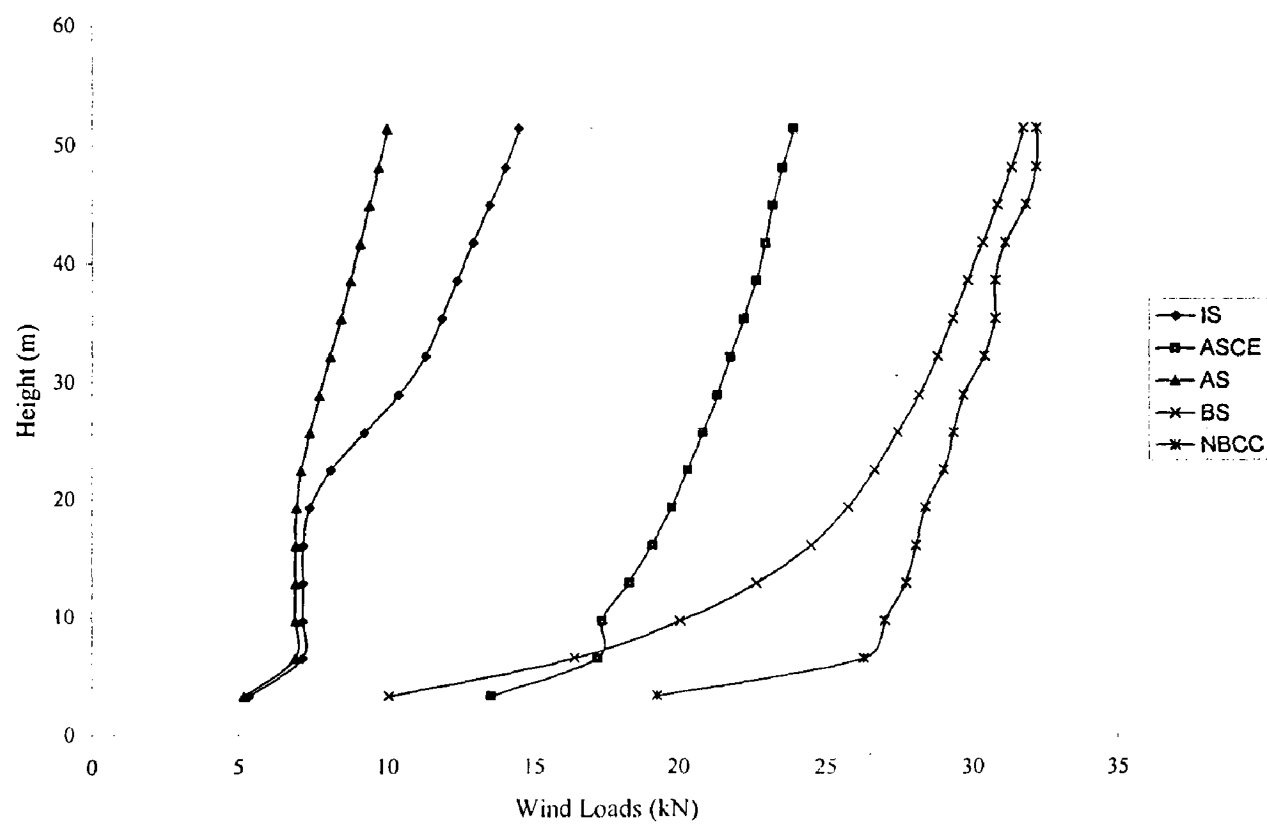


Fig.5.16 Wind Load distribution on 17s – 3b frame as per various codes (basic wind speed = 33m/s)

Table 5.10: Base Shear for 17s – 3b frame as per various codes (Basic wind speed = 33 m/s)

Term \ Code	IS	ASCE	AS	BS	NBCC
Base Shear (kN)	166.72	339.52	130.36	429.36	479.52
Relative Base Shear with IS Value reduced to unity	1.0	2.04	0.78	2.58	2.88

- The base shear given by NBCC is the largest followed by the values obtained from BS, ASCE, IS and AS in descending order.
- The base shear given by AS, only is lesser than that given by IS.

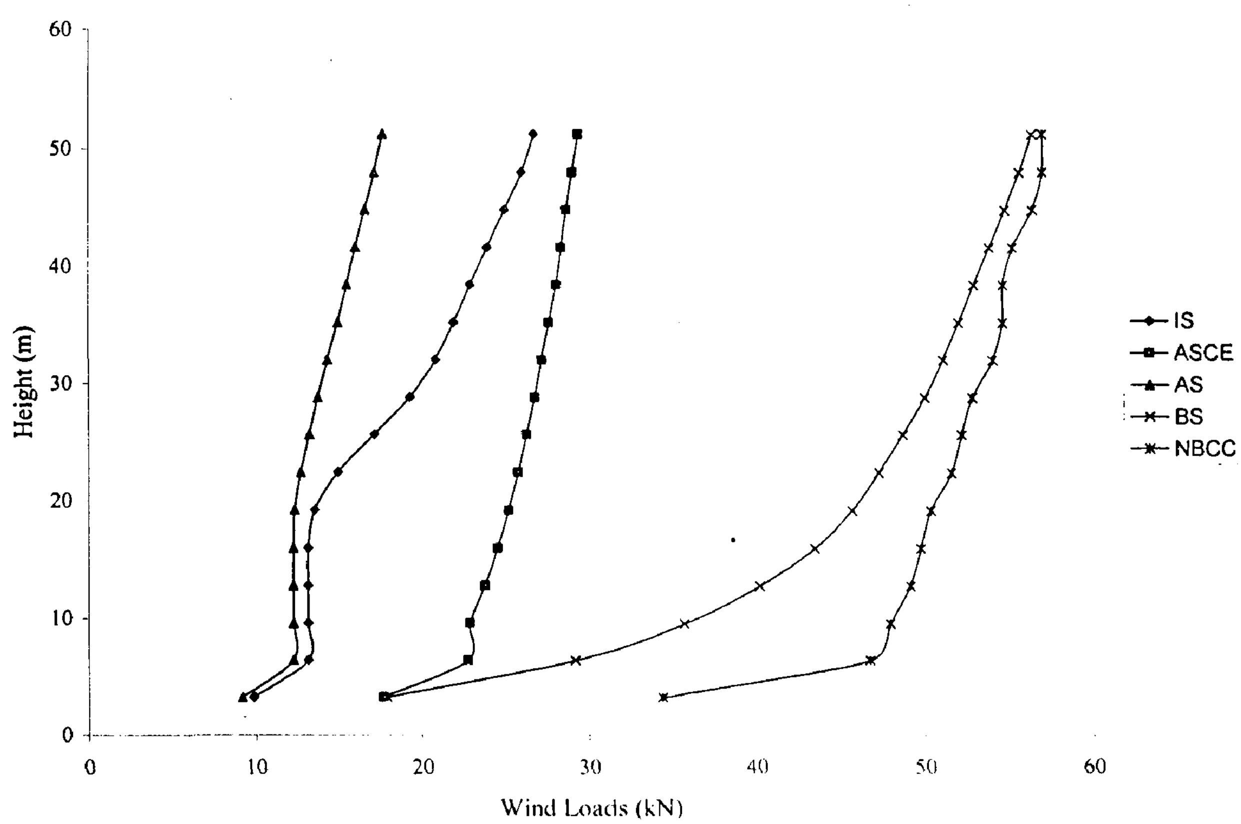


Fig.5.17 Wind Load distribution on 17s – 3b frame as per various codes (basic wind speed = 44m/s)

Table 5.11: Base Shear for 17s – 3b frame as per various codes (Basic wind speed = 44 m/s)

Code / Term	IS	ASCE	AS	BS	NBCC
Base Shear (kN)	308.71	429.22	231.88	763.15	852.45
Relative Base Shear with IS Value reduced to unity	1.0	1.39	0.75	2.47	2.76

- The base shear given by NBCC is the largest followed by the values obtained from BS, ASCE, IS and AS in descending order.
- The base shear given by AS, only is lesser than that given by IS.

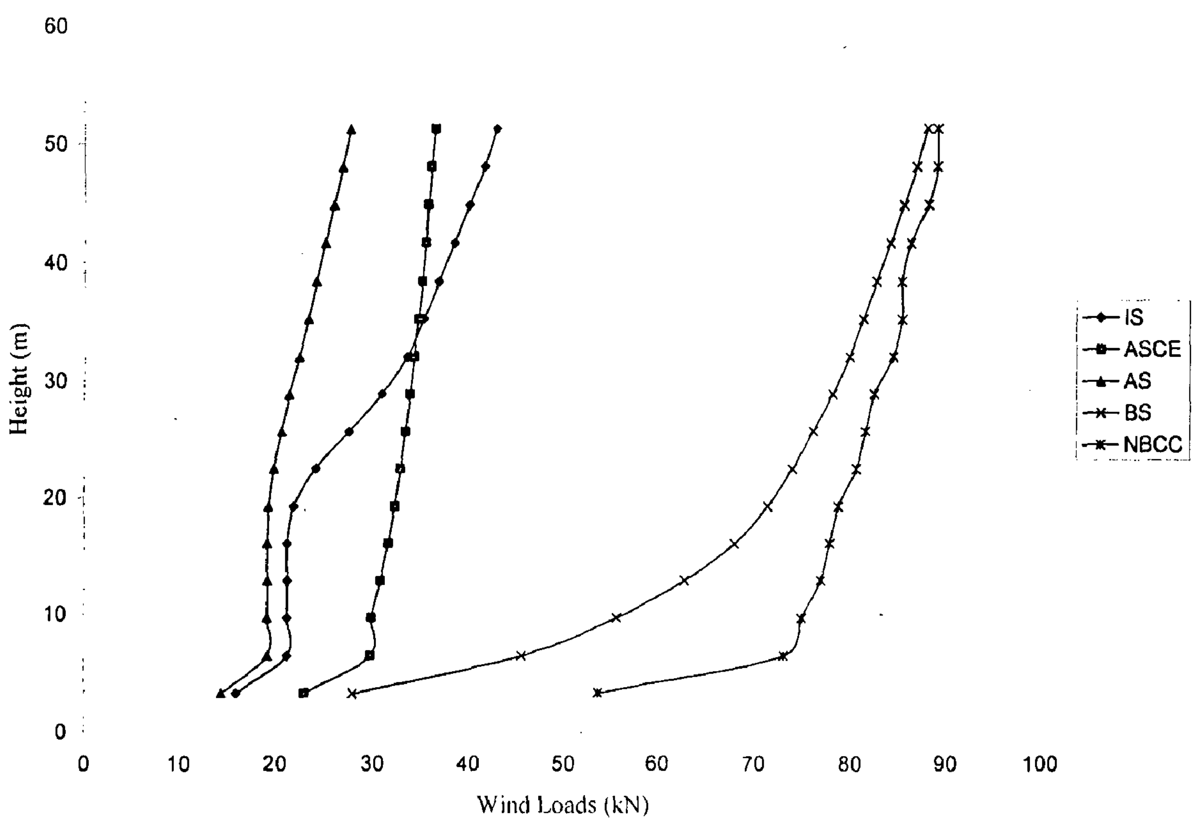


Fig.5.18 Wind Load distribution on 17s – 3b frame as per various codes (basic wind speed = 33m/s)

Table 5.12: Base Shear for 17s – 3b frame as per various codes
(Basic wind speed = 55 m/s)

Term \ Code	IS	ASCE	AS	BS	NBCC
Base Shear (kN)	495.85	543.49	362.1	1192.29	1332.06
Relative Base Shear with IS Value reduced to unity	1.0	1.1	0.73	2.4	2.69

- The base shear given by NBCC is the largest followed by the values obtained from BS, ASCE, IS and AS in descending order.
- The base shear given by AS, only is lesser than that given by IS.

5.3 MOMENTS IN FRAMES

Moments in beams and columns of the above considered frames are tabulated separately and are presented here. As 'Factor Method' is treated as the best method over the other approximate methods due to its consideration of the sizes of beams and columns, it is selected as the base method. The loads on frames were generated as per the Indian code provisions for a basic wind speed of 44 m/s. The remaining parameters are same as those considered for evaluating wind loads.

For comparing the moments in columns, average values are computed for top 5 or 7 stories, middle five stories, bottom five stories and the bottom two stories. The bottom two stories are considered, as much variation of values is seen in those stories. The values obtained by portal method, modified portal method, and cantilever method are listed as relative values by reducing the values obtained by factor method to unity. Likewise, the moments in beams are listed for different bay widths. Average relative values are computed for a single bay, for all the stories.

FRAME 1: 15 Stories – 2 Bays

Storey height = 3m

Bay widths – 8m, 3m

Frame spacing – 4m

Basic wind speed – 44 m/s

Terrain category – Category 4 (Terrain with numerous large high closely spaced obstructions)

Desired life of structure – 100 years

Return period – 100 years

Permeability of cladding – medium (16% on windward face and 18% on leeward face)

Sizes of columns – 600x400, 600x400 and 300x300 for stories 1 to 5

500x400, 500x400 and 300x300 for stories 6 to 10 and

500x300, 500x300 and 300x300 for stories 11 to 15

Sizes of beam webs - 250x250mm for span of 4.0m

450x250mm for span of 8.0m

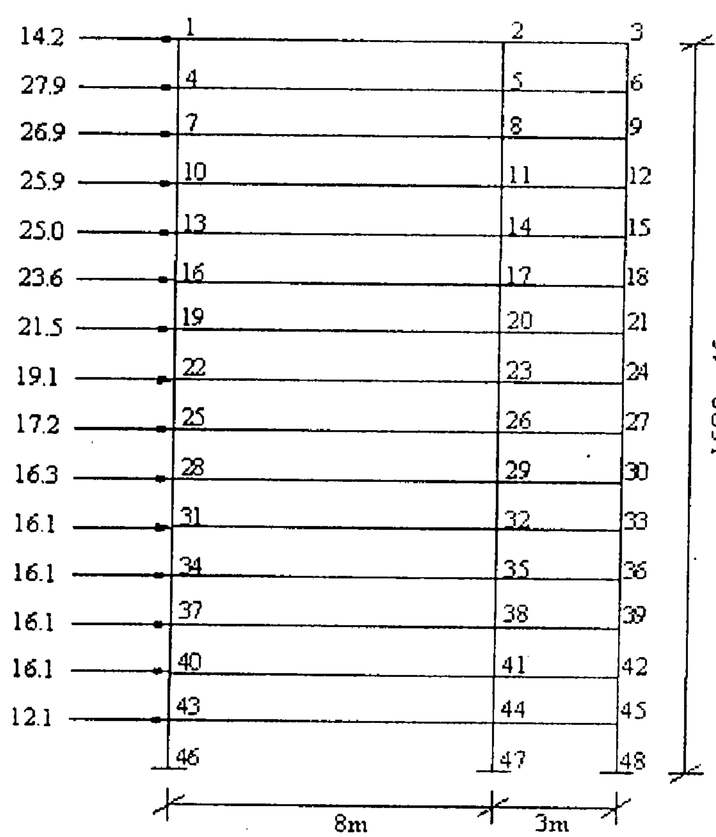


Fig.5.19. Wind loads on 15s – 2b frame

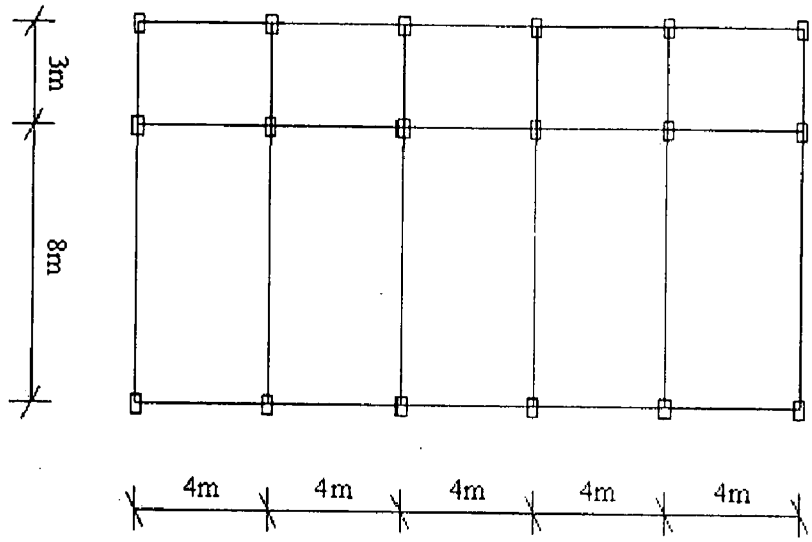


Fig.5.20. Plan of 15s – 2b Frame

Table 5.13: Relative average moments in columns of 15s-2b frame

Sl.No.	Term \ Method	Portal Method	Modified Portal Method	Cantilever Method	Factor Method
1	Relative Avg. Moments in columns of stories 11 to 15 with values of Factor Method reduced to unity	3.2	2.9	2.7	1.0
2	Relative Avg. Moments in columns of stories 6 to 10 with values of Factor Method reduced to unity	3.3	3.0	2.7	1.0
3	Relative Avg. Moments in columns of stories 1 to 5 with values of Factor Method reduced to unity	4.2	3.4	2.9	1.0
4	Relative Avg. Moments in columns of bottom two stories with values of Factor Method reduced to unity	6.7	4.0	2.0	1.0

- The values of moments given by portal method is the largest followed by the values given by modified portal and cantilever methods in descending order, at all levels.
- For the bottom two stories, the values of moments given by portal method are much higher.

Table 5.14: Relative average moments in beams of 15s - 2b frame

Sl.No.	Method / Term	Portal Method	Modified Portal Method	Cantilever Method	Factor Method
1	Relative Avg. Moments in beams of span 8m with values of Factor Method reduced to unity	2.3	3.3	3.9	1.0
2	Relative Avg. Moments in beams of span 3m with values of Factor Method reduced to unity	3.8	2.1	1.1	1.0

- The values of moments given by portal method is the largest followed by the values given by modified portal and cantilever methods in descending order, at all levels.
- For the span of 3m, the values of moments given by cantilever method are much closer to the values given by factor method.

FRAME 2: 17 Stories – 2 Bays

Storey height = 3m

Bay widths – 8m, 3m

Frame spacing – 4m

Terrain category – Category 4 (Terrain with numerous large high closely spaced obstructions)

Desired life of structure – 100 years

Return period – 100 years

Permeability of cladding – medium (16% on windward face and 18% on leeward face)

Sizes of columns – 600x400, 600x400 and 300x300 for stories 1 to 5
500x400, 500x400 and 300x300 for stories 6 to 11 and
500x300, 500x300 and 300x300 for stories 12 to 15

Sizes of beam webs - 250x250mm for span of 4.0m
450x250mm for span of 8.0m

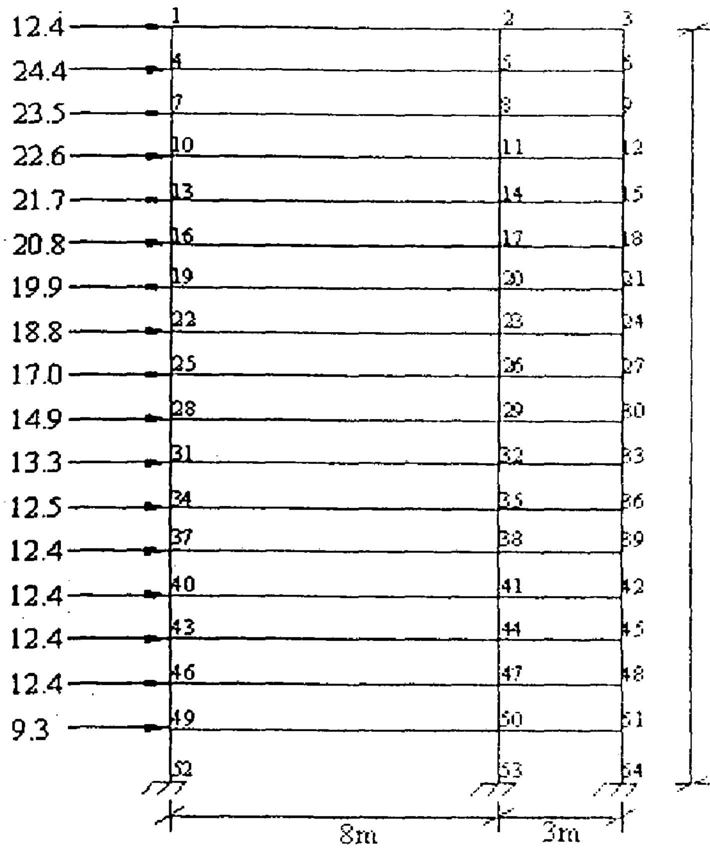


Fig.5.21. Wind loads on 17s – 2b frame

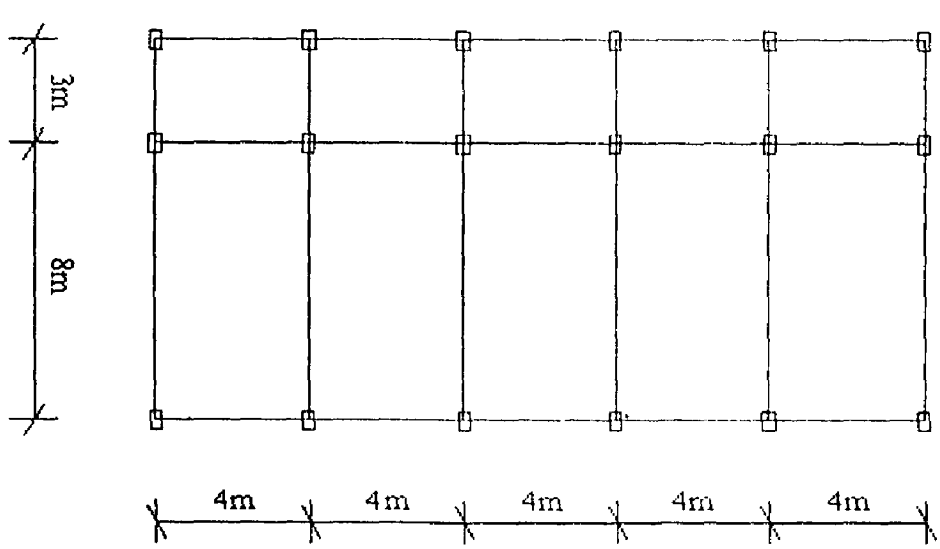


Fig.5.22. Plan of 17s – 2b frame

Table 5.15: Relative average moments in columns of 17s-2b frame

Sl.No.	Method / Term	Portal Method	Modified Portal Method	Cantilever Method	Factor Method
1	Relative Avg. Moments in columns of stories 11 to 17 with values of Factor Method reduced to unity	3.2	2.9	2.7	1.0
2	Relative Avg. Moments in columns of stories 6 to 10 with values of Factor Method reduced to unity	3.3	2.9	2.8	1.0
3	Relative Avg. Moments in columns of stories 1 to 5 with values of Factor Method reduced to unity	4.1	3.4	3.3	1.0
4	Relative Avg. Moments in columns of bottom two stories with values of Factor Method reduced to unity	6.7	4.0	2.9	1.0

- The values of moments given by portal method is the largest followed by the values given by modified portal and cantilever methods in descending order, at all levels.
- For the bottom two stories, the values of moments given by portal method are much higher.

Table 5.16: Relative average moments in beams of 17s-2b frame

Sl.No.	Method / Term	Portal Method	Modified Portal Method	Cantilever Method	Factor Method
1	Relative Avg. Moments in beams of span 8m with values of Factor Method reduced to unity	2.3	3.3	3.9	1.0
2	Relative Avg. Moments in beams of span 3m with values of Factor Method reduced to unity	3.8	2.1	1.1	1.0

- The values of moments given by modified portal method is the largest followed by the values given by cantilever and portal methods in descending order for beams of 8m span.
- For the span of 3m, the values of moments given by cantilever method are much closer to the values given by factor method.

FRAME 3: 15 Stories – 3 Bays

Storey height – 3m

Bay widths – 9m, 3m, and 6m

Frame spacing – 4m (9 @ 4m each)

Terrain category – Category 4 (Terrain with numerous large high closely spaced obstructions)

Desired life of structure – 100 years

Return period – 100 years

Permeability of cladding – medium (16% on windward face and 18% on leeward face)

Sizes of columns – 600x500, 600x500, 600x400 and 600x400 for stories 1 to 5

600x400, 600x400, 500x400 and 500x400 for stories 6 to 10

600x300, 600x300, 500x300, 500x300 for stories 11 to 15

Sizes of beam webs - 500x250mm for span of 9.0m

450x250mm for span of 6.0m

250x250mm for span of 3.0m

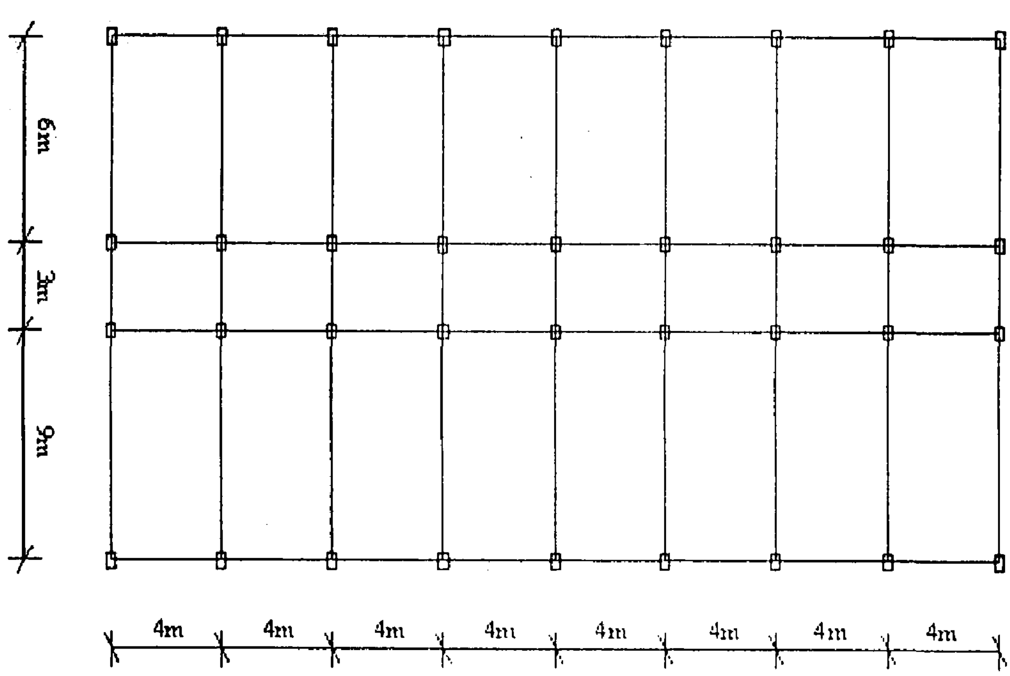


Fig.5.23. Plan of 15s – 3b Frame

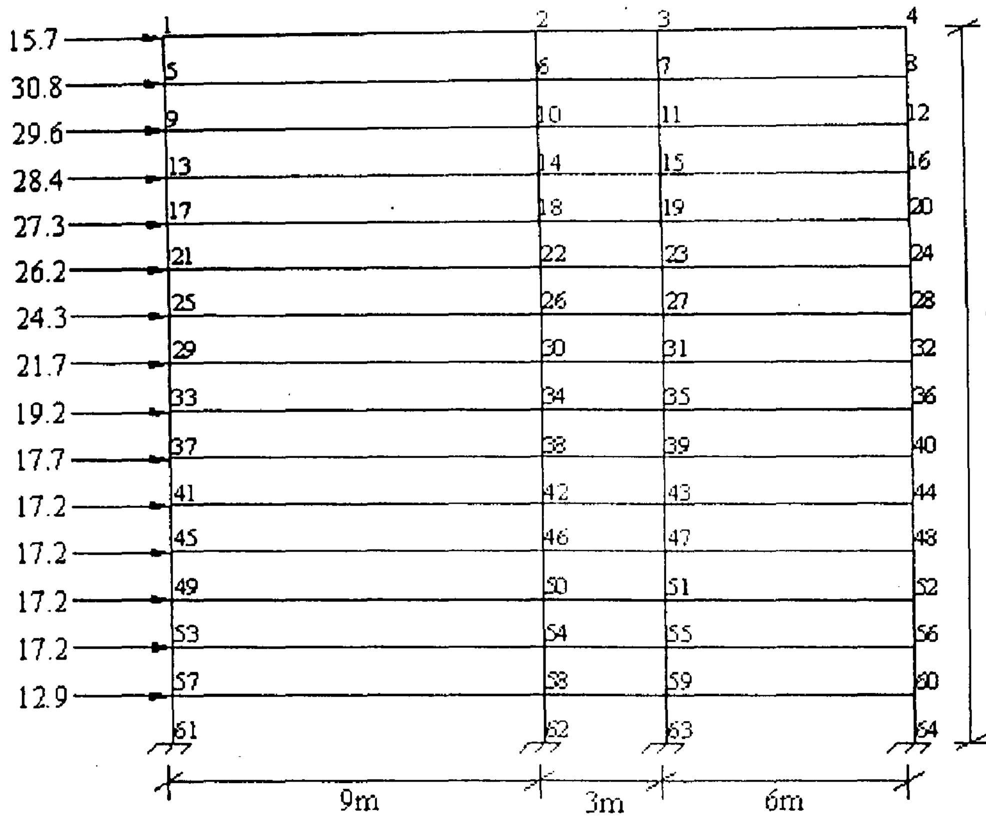


Fig.5.24. Wind loads on 15s – 3b frame

Table 5.17: Relative average moments in columns of 15s – 3b frame

Sl.No.	Term \ Method	Portal Method	Modified Portal Method	Cantilever Method	Factor Method
1	Relative Avg. Moments in columns of stories 11 to 15 with values of Factor Method reduced to unity	3.5	3.2	3.3	1.0
2	Relative Avg. Moments in columns of stories 6 to 10 with values of Factor Method reduced to unity	3.8	3.7	3.5	1.0
3	Relative Avg. Moments in columns of stories 1 to 5 with values of Factor Method reduced to unity	3.9	3.9	3.9	1.0
4	Relative Avg. Moments in columns of bottom two stories with values of Factor Method reduced to unity	5.4	4.9	4.3	1.0

- The values of moments given by all the methods are same for stories 1 to 5.
- For the bottom two stories, the values of moments given by portal method are much higher.

Table 5.18: Relative average moments in beams of 15s-3b frame

Sl.No.	Method / Term	Portal Method	Modified Portal Method	Cantilever Method	Factor Method
1	Relative Avg. Moments in beams of span 9m with values of Factor Method reduced to unity	2.1	3.1	3.4	1.0
2	Relative Avg. Moments in beams of span 3m with values of Factor Method reduced to unity	3.5	1.7	1.7	1.0
3	Relative Avg. Moments in beams of span 6m with values of Factor Method reduced to unity	6.3	6.3	5.3	1.0

- The values of moments given by modified portal method and cantilever method are same for beams of 3m span.
- The values of moments given by modified portal method and cantilever method are same for beams of 6m span, and are much higher.

FRAME 4: 17 Stories – 3 Bays

Storey height - 3m

Bay widths – 9m, 3m, and 6m

Frame spacing – 4m (9 @ 4m each)

Terrain category – Category 4 (Terrain with numerous large high closely spaced obstructions)

Desired life of structure -- 100 years

Return period – 100 years

Permeability of cladding – medium (16% on windward face and 18% on leeward face)

Sizes of columns – 600x500, 600x500, 600x400 and 600x400 for stories 1 to 5
600x400, 600x400, 500x400 and 500x400 for stories 6 to 11
600x300, 600x300, 500x300, 500x300 for stories 12 to 17

Sizes of beam webs - 500x250mm for span of 9.0m
450x250mm for span of 6.0m
250x250mm for span of 3.0m

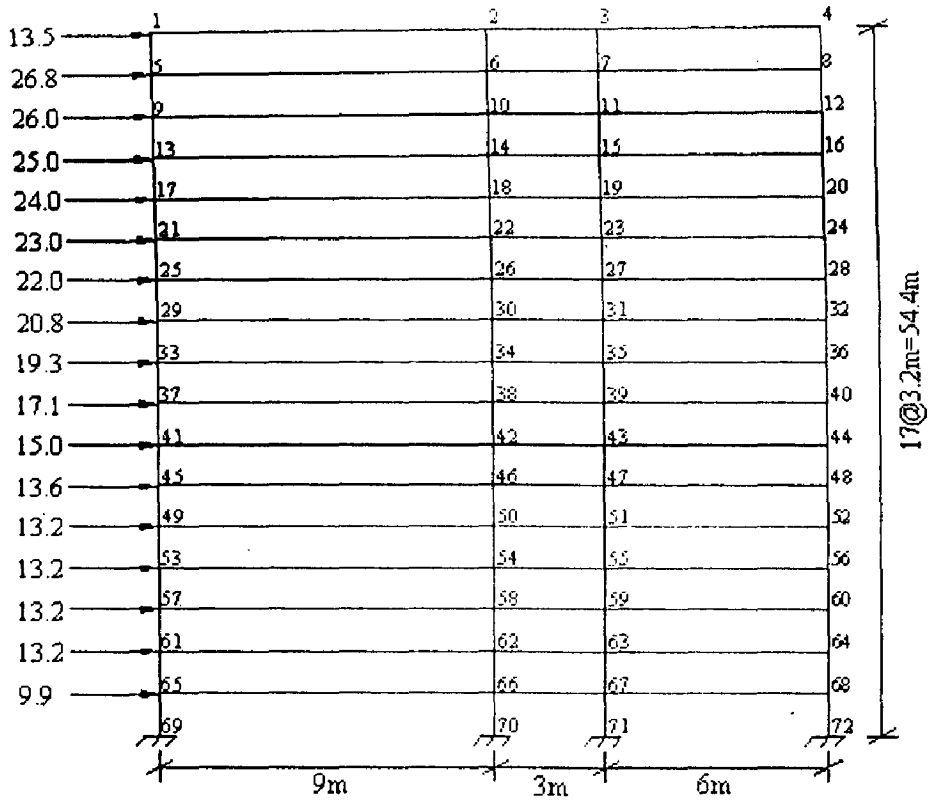


Fig.5.25. Wind loads on 17s – 3b frame

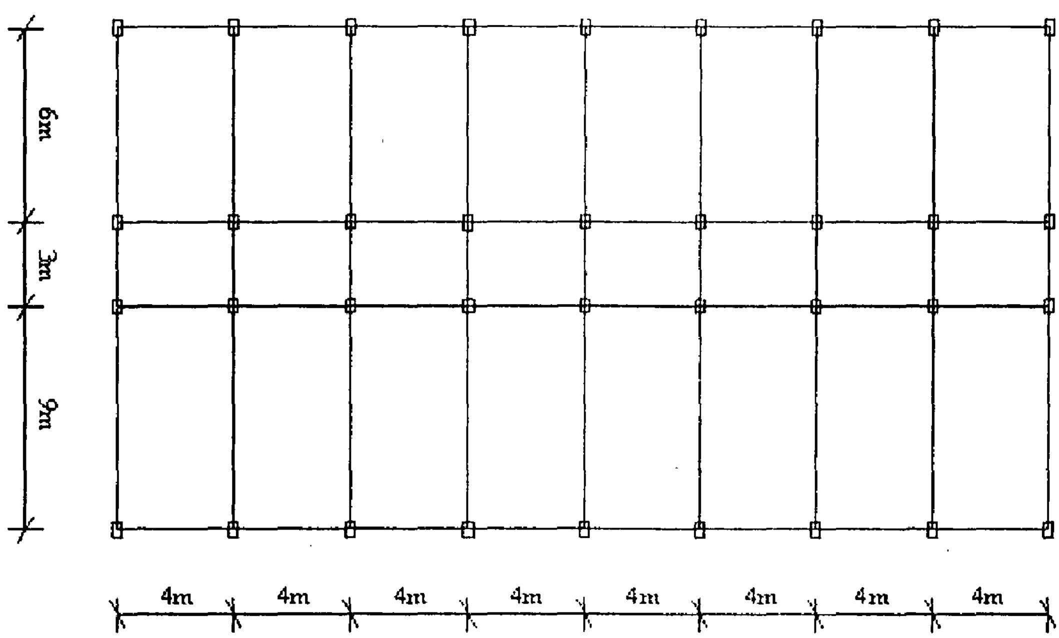


Fig.5.26. Plan of 17s – 2b frame

Table 5.19: Relative average moments in columns of 17s-3b frame

Sl.No.	Term \ Method	Portal Method	Modified Portal Method	Cantilever Method	Factor Method
1	Relative Avg. Moments in columns of stories 11 to 17 with values of Factor Method reduced to unity	3.1	3.1	3.1	1.0
2	Relative Avg. Moments in columns of stories 6 to 10 with values of Factor Method reduced to unity	2.4	3.2	3.1	1.0
3	Relative Avg. Moments in columns of stories 1 to 5 with values of Factor Method reduced to unity	3.4	3.4	3.4	1.0
4	Relative Avg. Moments in columns of bottom two stories with values of Factor Method reduced to unity	3.6	3.1	2.7	1.0

- The values of moments given by all the methods are same for stories 11 to 17 and for stories 1 to 5.

Table 5.20: Relative average moments in beams of 17s-3b frame

Sl.No.	Term \ Method	Portal Method	Modified Portal Method	Cantilever Method	Factor Method
1	Relative Avg. Moments in beams of span 9m with values of Factor Method reduced to unity	2.5	3.5	4.0	1.0
2	Relative Avg. Moments in beams of span 3m with values of Factor Method reduced to unity	4.4	2.4	2.3	1.0
3	Relative Avg. Moments in beams of span 6m with values of Factor Method reduced to unity	3.3	3.3	2.8	1.0

- The values of moments given by portal method and modified portal method are same for beams of 6m span.
- Cantilever method gives higher values for beams of 9m span, while portal method gives higher values for beams of 3m span.

CHAPTER 6

DISCUSSIONS AND CONCLUSION

This dissertation deals with the evaluation of wind load according to various wind loading codes, along the floor levels, and the various approximate methods of analysis applicable to frames subjected to lateral loads. The process of evaluation of wind load as per the wind loading codes of India, U.S.A., Australia, Britain and Canada has been fully computerised in the sense that the basic inputs relate to identifying the place and topography of the area where the building is to be located. All the essential data with regard to computation of factors such as multiplying factors and pressure coefficients have been stored as data bank and made an integral part of the computer program. This, with frame data being available, wind load at various floor levels gets computed automatically. The program has been written in Microsoft Visual Basic language of programming.

Assessment of wind loads according to various wind loading codes has been presented both qualitatively and with an illustrative problem in chapter 3. Various parameters considered by all the codes of practice considered are explained clearly in separate sections. The values of parameters in all the codes are presented in the chapter. An illustrative example is included to better understand the provisions of wind loading codes. It is found that while all of the standards reference their wind speed at 10 m above ground in a flat, open exposure, each uses gusts of different duration. The British and Canadian standards use the mean hourly wind speed in design, while the American Standard references a 3 second gust, as does the Australian and Indian Standards. It is also observed that only the British, U.S.A., and Australian codes incorporate wind direction for computing wind loads. Australian code is unique in having a shielding multiplier. British code is unique in having seasonal and altitude factors.

The various approximate methods of analysis have been dealt with theoretically in chapter 4 highlighting the assumptions and approximations involved. The factor method for frame analysis has been used to give results which are considered to be accurate among the approximate methods considered. The results obtained by the other approximate methods are compared by the ones obtained by factor method.

In chapter 5 is presented the essential information regarding all the computer programs developed. Development of these programs constitutes a major effort of the study presented in this dissertation. These programs can calculate the wind loads as per the code provisions of Indian, U.S.A. Australian. British and Canadian wind loading codes and also are able to analyse the frames for these loads using approximate methods such as Portal, Modified Portal, Cantilever and Factor methods. Loads are graphically displayed and the frames are displayed with numbering wherever required.

Four types of RC frames have been used for studying the wind loads as obtained by the five codes: Indian, U.S.A., Australian, British and Canadian, and have been analysed for different values of lateral loads by approximate methods such as Portal, Modified Portal, Cantilever and Factor.

Loads have been studied for wind speeds of 33, 44 and 55 m/s. Variation of loads obtained by all the codes has been shown in graphs. Base shear is calculated for all the loads, and is compared with the Indian code. The base shear obtained by Canadian code was on an average 2.6 times that of Indian code. The base shear obtained by British code was on an average 2.3 times that of Indian code. This higher variation for Canadian and British codes can be attributed to the use of hourly mean wind speed. The base shear obtained by U.S.A. code was on an average 1.3 times that of Indian code. For the wind speed of 55 m/s, the base shear obtained by U.S.A. code is almost equal to that of Indian code. The base shear obtained by Australian code was on an average 0.7 times that of Indian code. This can be attributed to the shielding effect considered in the Australian code.

The results of analysis methods have been compared with the results obtained by factor method with a view to studying the accuracy and applicability of one method over the others. The study clearly brings out that Portal method gave more values as compared to other methods, especially for the lower stories. This method is the simplest of all the methods in the sense that it does not need data with regard to member cross-sections whether for beams or columns. If higher accuracy is a consideration, Factor method is the preferred choice. In this method, member cross-sections are considered for evaluating moments. Cantilever method is found to give higher values for beams of 8m span and 9m span. Modified Portal method gave values which are in between the values obtained by Portal and Cantilever methods.

Scope for Future Study

It is recommended that some more in-depth study should be carried out along the following suggested lines:

1) Dynamic effects of wind as per various wind loading codes.
2) Study of very tall and slender frames by various approximate methods.
3) Effect of basement floors on tall frames subjected to lateral loads.

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