INTRODUCTION

1.1 WIND

Wind is the flow of gases on a large scale. On earth, wind consists of bulk movement of air. n human civilization, wind has inspired mythology, influenced the events of history, expanded the range of transport and warfare, and provided a power source for mechanical work, electricity, and recreation. Wind has powered the voyages of sailing ships across Earth's oceans. Hot air balloons use the wind to take short trips, and powered flight uses it to increase lift and reduce fuel consumption. Areas of wind shear caused by various weather phenomena can lead to dangerous situations for aircraft. When winds become strong, trees and man-made structures are damaged or destroyed.

1.2 CAUSE

Wind is caused by differences in pressure. When a difference in pressure exists, the air is accelerated from higher to lower pressure. Globally, the two major driving factors of large scale winds (the atmospheric circulation) are the differential heating between the equator and the poles (difference in absorption of solar energy leading to buoyancy forces) and the rotation of the planet.

1.3 MEASUREMENT

Wind direction is reported by the direction from which it originates. For example, a *northerly* wind blows from the north to the south. Wind speed is measured by anemometers, most commonly using rotating cups or propellers. When a high measurement frequency is needed (such as in research applications), wind can be measured by the propagation speed of ultrasound signals or by the effect of ventilation on the resistance of a heated wire.

Sustained wind speeds are reported globally at a 10 meters (33 ft) height and are averaged over a 10 minute time frame. The United States reports winds over a 1 minute average for tropical cyclones, and a 2 minute average within weather observations, while India typically reports winds over a 3 minute average. Knowing the wind sampling average is important, as the value of a one-minute sustained wind is typically 14 percent greater than a ten-minute sustained wind. A short burst of high speed wind is termed a wind gust, one technical definition of a wind gust is: the maxima that exceed the lowest wind speed measured during a ten minute time interval by 10 knots (19 km/h).

1.4 WIND EFFECT ON STRUCTURE

Wind pressure on a building surface depends primarily on its velocity, the shape and surface structure of the building, the protection from wind offered by surrounding natural terrain or man-made structures, and to a smaller degree, the density of air which decreases with altitude and temperature.

Wind is moving air. The air has a particular mass and moves in a particular direction at a particular direction at a particular velocity. It thus has kinetic energy of the from expressed as,

$$q_s = (1/2)\ pv^2 \tag{1.1}$$

where,

The density of air *p* is 0.0765 pcf, for conditions of standard atmosphere, temperature (59 °F), and barometric pressure (29.92 in. of mercury).

For wind velocity in mph. The equation reduces to:

$$q_s = 0.00256V^2 \tag{1.2}$$

where,

q_s = Pressure
V = Velocity of wind in miles per second

In an engineered structure, wind loads have long been a factor in the design of lateral force resisting system, with added significance as the height of the building increased. For many decades, the cladding systems of high-rise buildings, particularly around corners of buildings, have been scrutinized for the effects of wind on building enclosure. Glass and curtain wall systems are regularly developed and tested to resist cladding pressures and suctions induced by the postulated wind event.

As wind hits the structure and flows around it, several effects are possible, as illustrated in Figure 1.1. Pressure on the windward face and suction on the leeward face creates *drag forces*. Analogous to flow around an airplane wing, unsymmetrical flow around the structure can create *lift forces*. Air turbulence around the leeward corners and edges can create *vortices*, which are high-velocity air currents that create circular updrafts and suction streams adjacent to the building. Periodic shedding of vortices causes the building to oscillate in a direction transverse to the direction of the wind and may result in unacceptable accelerations at the upper floors of tall buildings.

The effects of downdrafts must also be considered: Downdrafts have been known to completely strip trees in plaza areas and to buffet pedestrians dangerously. Some tall buildings that extend into high wind velocity regions have been known to sway excessively in strong wings. High suction forces have blown off improperly anchored lightweight roofs.

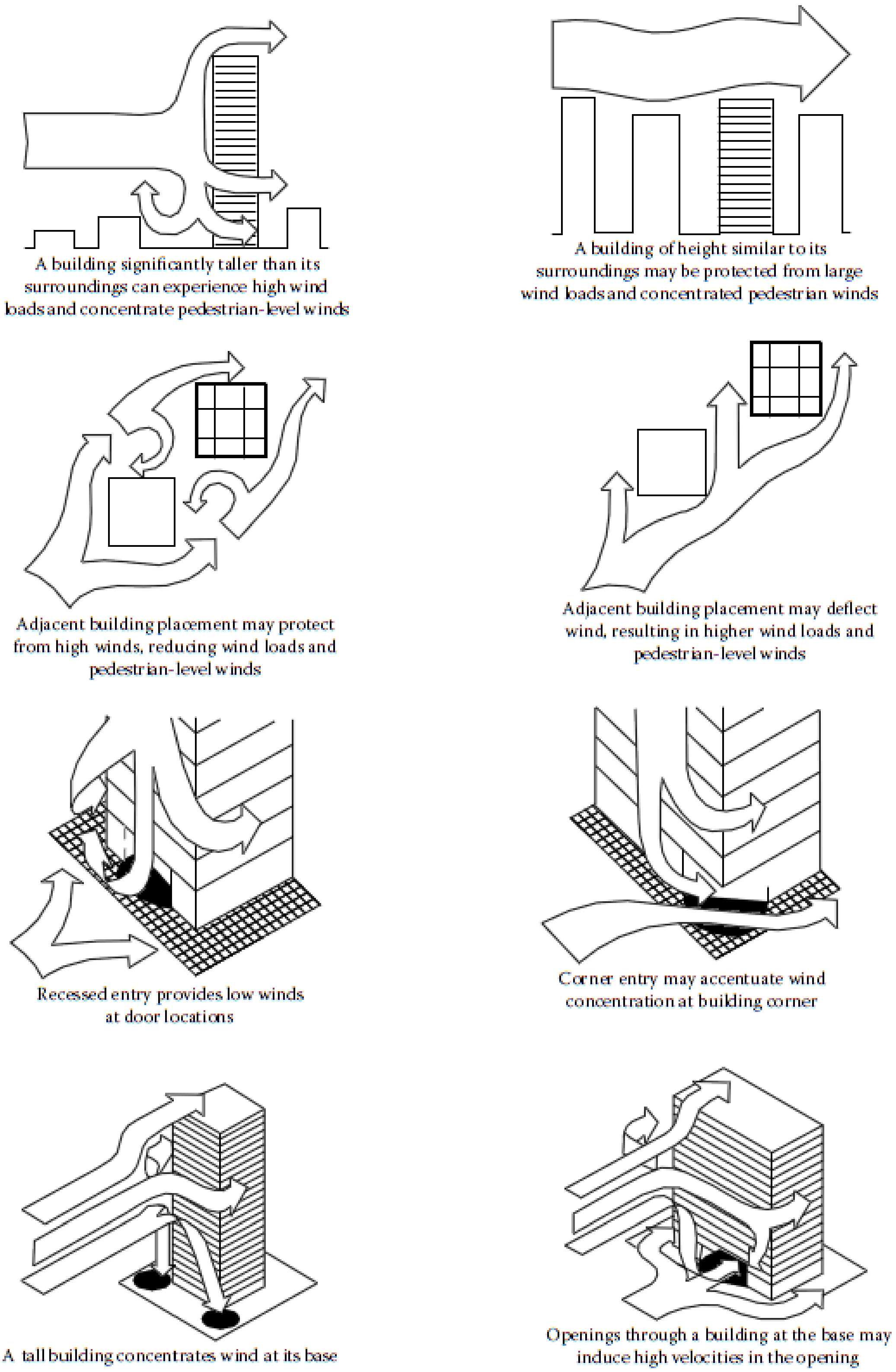

Fig 1.1 Wind flow around building (Bungale S. Taranath)

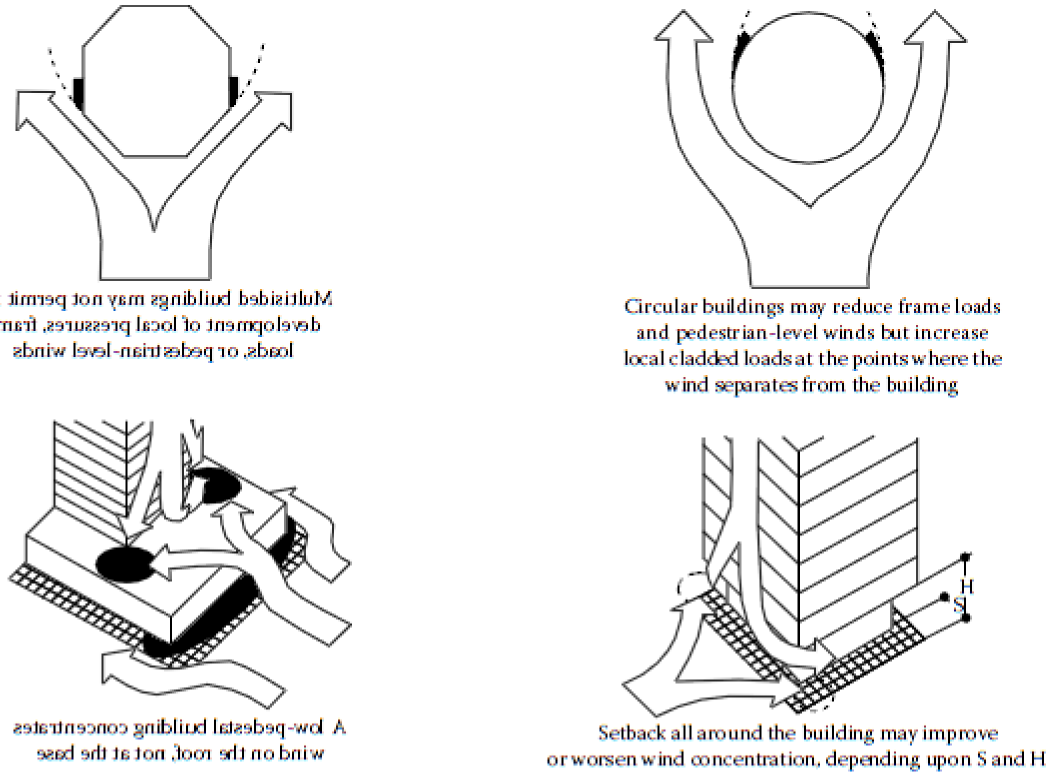

Fig 1.1 (cont.) Wind flow around building (Bungale S. Taranath)

1.5 CHARACTERISTICS OF WIND

The flow of wind is complex because many flow situations arise from the interaction of wind with structures. However, in wind engineering, simplifications are made to arrive at design wind loads by distinguishing the following characteristics:

- Variation of wind velocity with height.
- Wind turbulence.
- Statistical probability.
- Vortex shedding phenomenon.
- Dynamic nature of wind–structure interaction.

VARIATION OF WIND VELOCITY WITH HEIGHT

The roughness of the earth's surface which causes drag, converts some of the wind's energy into mechanical turbulence. Since turbulence is generated at the surface, surface wind speed is much less than wind speed at high levels. Turbulence includes vertical as well as horizontal air movement and hence the effect of surface frictional drag is propagated upward. The effect of frictional drag gradually decreases with height, and at gradient level (around 1000–2000 ft) frictional drag effect is negligible. At and above this level wind blows almost parallel to isobars (lines on a map having equal barometric pressure). For strong winds, the shape of wind speed profile depends mainly on the degree of surface roughness, caused by the overall drag effect of buildings, trees, and other projections that impede flow of wind at the earth's surface. This is illustrated in the three typical wind velocity profiles shown in Figure 1.2.

The viscosity of air reduces its velocity adjacent to the earth's surface to almost zero. The maximum retarding effect occurs in wind layers nearest to the ground. These layers in turn successively slow the higher layers. Thus the effect of slowdown reduces at each layer as the height increases, and eventually becomes negligible. The height at which the slowdown effect ceases to exist is called *gradient height*, and the corresponding velocity, *gradient velocity*. This characteristic increase of wind velocity with height is a well-understood phenomenon, as evidenced by higher design pressures specified at higher elevations in most building standards.

At heights of approximately 1200 ft (366 m) aboveground, the wind speed is virtually unaffected by surface friction. Its movement at and above this level, is solely a function of seasonal and local wind effects. The ensueing height in which the wind speed is affected by topography is called the *atmospheric boundary layer*.

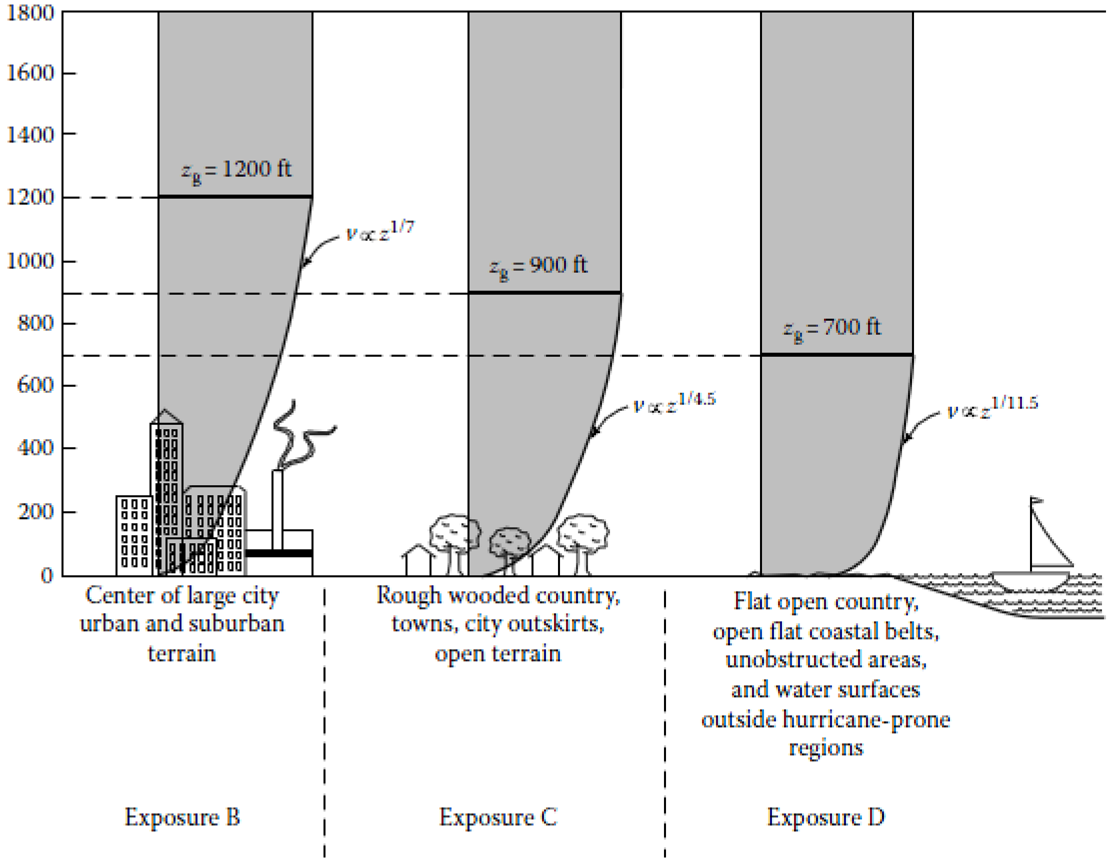

Fig 1.2 Wind velocity profiles as defined in the ASCE 7-05. Velocity profiles are determined by fitting curves to observed wind speeds. (Bungale S. Taranath)

The wind speed profile within *atmospheric boundary layer* is given by:

$$V_z = V_g\,(Z/Z_g)^{1/\alpha} \tag{1.3}$$

Where

V_z = mean wind speed at height Z aboveground
V_g = gradient wind speed assumed constant above the boundary layer
Z = height aboveground
Z_g = nominal height of boundary layer, which depends on the exposure (Values for Z_g are given in Fig. 1.2.)

α = power law coefficient

With known values of mean wind speed at gradient height and exponent α, wind speeds at height z are calculated by using Equation 1.3. The exponent 1/α and the depth of boundary layer z_g vary with terrain roughness and the averaging time used in calculating wind speed. The coefficient α signifies that wind speed reaches its maximum over a greater height in an urban terrain than in the open country.

WIND TURBULANCE

Motion of wind is turbulent. A concise mathematical definition of turbulence is difficult to give, except to state that it occurs in wind flow because air has a very low viscosity—about one-sixteenth that of water. Any movement of air at speeds greater than 2–3 mph (0.9–1.3m/s) is turbulent, causing particles of air to move randomly in all directions. This is in contrast to the laminar flow of particles of heavy fluids, which move predominantly parallel to the direction of flow.

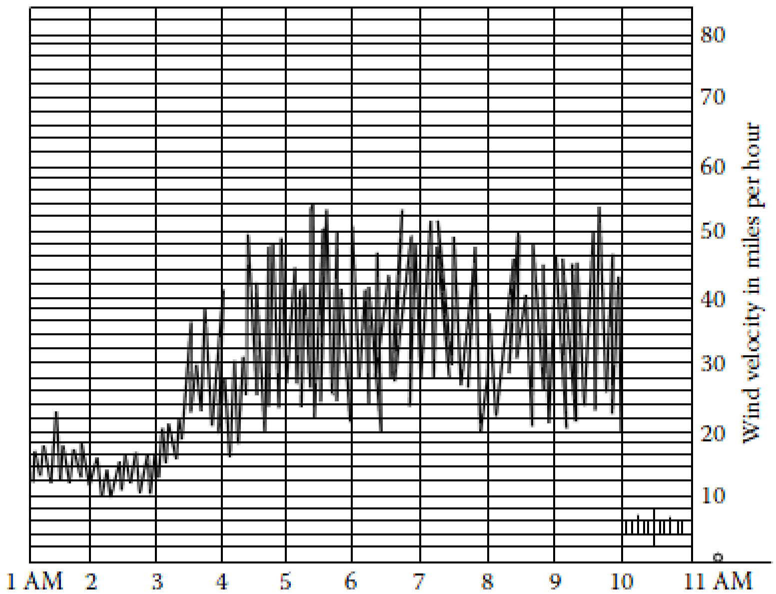

Fig 1.3 Schematic record of wind speed measured by an anemometer. (Bungale S. Taranath)

The velocity profiles, shown in Figure 1.3, describe only one aspect of wind at lower levels. Superimposed on mean speed are gusts and lulls, which are deviations above and below the mean values. These gusts and lulls have a random distribution over a wide range of frequencies and amplitudes in both time and space, as shown in Figure 1.3, which is a schematic record of the unsteady nature of wind speed measured by an anemometer. Gusts are frequently the result of the introduction of fast moving parcels of air from higher levels into slower moving air strata. This mixing produces turbulence due to surface roughness and thermal instability. When this occurs, turbulence may result with eddies separating first from one side and then forming again. Turbulence generated by obstacles may persist downwind from projections as much as 100 times their height. Large-scale topographical features are not included in the above-mentioned surface roughness. They can influence the flow, so they are given special consideration in design by using a topographic factor, K_{zt}. For instance, wind is usually much stronger over the brow of a hill or ridge. This is because, to pass the same quantity of air over the obstructing feature, a higher speed is required. Large valleys often have a strong funnelling effect that increases wind speed along the axis of the valley.

Every structure has a natural frequency of vibration. Should dynamic loading occur at or near its natural frequency, structural damage, out of all proportion to size of load, may result. It is well known, for example, bridges capable of carrying far greater loads than the weight of a company of soldiers may oscillate dangerously and may even break down under dynamic loading of soldiers marching over them in step. Similarly, certain periodic gust within the wide spectrum of gustiness in wind may find resonance with the natural vibration frequency of a building, and although the total force caused by that particular gust frequency would be much less than the static design load for the building, dangerous oscillations may be set up. This applies not only to the structure as a whole, but also to components such as curtain wall panels and sheets of glass. A second dynamic effect is caused by instability of flow around certain structures. Long narrow structures such as smoke stacks, light standards, and suspension bridges are particularly susceptible to this sort of loading; causing an alternating pattern of eddies to form in its wake. A side thrust is thus exerted on the object similar to the lift on an aerofoil, and since this thrust alternates in direction, a vibration may result. Side-to-side wobbling effect of a straight stick pulled through water is an example of this phenomenon.

In regions where less frequent storms contribute significantly to the wind climate, available wind records may not be sufficient for design purposes. Such regions would include, for example, those frequented by tornadoes or by tropical cyclones. The severest of the latter are commonly termed hurricanes. Along the U.S. Gulf Coast and Florida Peninsula in the United States, severe tropical cyclones dominate the climate of strong winds. Along the New England Coast such storms contribute to the wind climate but to a lesser extent than along the Gulf Coast. Because of the rarity of these storms and their relatively small size, a typical 20 year record is not sufficient to obtain a reliable statistical estimate. Furthermore, there is the difficulty that instruments often fail in hurricane force winds. Similar comments can be made regarding the contribution to the wind climate by tornado generating thunderstorms in the Midwestern U.S. region.

The reliability of the wind model can also be affected by severe topography in two ways. Large hills or mountains can severely distort surface wind measurements, and can essentially increase the height at which gradient conditions are first approximated. Furthermore, severe winds can originate in regions near mountain ranges due to thermal instabilities in the atmosphere. These down slope winds are referred to by several names such as Santa Ana and Chinooks and are particularly prevalent in West Coast areas and areas just east of the Rocky Mountains. Their detailed structure is not well understood, particularly in regions close to the mountains where significant vertical flows can occur, leading to severe spatial in homogeneities near the ground. Away from the close proximity of the mountains, the flow appears to take on the characteristics of “normal” storm winds, although little information exists on the boundary-layer structure away from the surface. In areas affected by such winds, conservative modelling of the approaching flows is the current state of the art.

PROBABLISTIC APPROCH

In many engineering sciences, the intensity of certain events is considered to be a function of the duration recurrence interval (return period). For example, in hydrology the intensity of rainfall expected in a region is considered in terms of a return period because the rainfall expected once in 10 years is less than the one expected once every 50 years. Similarly, in wind engineering the speed of wind is considered to vary with return periods. For example, the fastest mile wind 33 ft (10 m) aboveground in Dallas, Texas, corresponding to a 50 year return period, is 67 mph (30 m/s), compared to the value of 71 mph (31.7 m/s) for a 100 year recurrence interval.

A 50 year return-period wind of 67 mph (30 m/s) means that on the average, Dallas will experience a wind faster than 67 mph within a period of 50 years. A return period of 50 years corresponds to a probability of occurrence of 1/50 = 0.02 = 2%. Thus the chance that a wind exceeding 67 mph (30 m/s) will occur in Dallas within a given year is 2%. Suppose a building is designed for a 100 year lifetime using a design wind speed of 67 mph. What is the probability that this wind will exceed the design speed within the lifetime of the structure? The probability that this wind speed will not be exceeded in any year is 49/50. The probability that this speed will not be exceeded 100 years in a row is (49/50)100. Therefore, the probability that this wind speed will be exceeded at least once in 100 years is

$$1 - (49/50)^{100} = 0.87 = 87\% \qquad (1.4)$$

This signifies that although a wind with low annual probability of occurrence (such as a 50 year wind) is used to design structures, there still exists a high probability of the wind being exceeded within the lifetime of the structure. However, in structural engineering practice it is believed that the actual probability of overstressing a structure is much less because of the factors of safety and the generally conservative values of wind speeds used in design.

It is important to understand the notion of probability of occurrence of design wind speeds during the service life of buildings. The general expression for probability P that a design wind speed will be exceeded at least once during the exposed period of n years is given by

$$P = 1 - (1 - P_a)^n \qquad (1.5)$$

where
P_a is the annual probability of being exceeded (reciprocal of the mean recurrence interval)
n is the exposure period in years

Consider again the building in Dallas designed for a 50 year service life instead of 100 years. The probability of exceeding the design wind speed at least once during the 50 year lifetime of the building is

$$P = 1 - (1 - 0.02)^{50} = 1 - 0.36 = 0.64 = 64\% \qquad (1.6)$$

Thus the probability that wind speeds of a given magnitude will be exceeded increases with a longer exposure period of the building and the mean recurrence interval used in the design. Values of P for a given mean recurrence interval and a given exposure period are shown in Table 1.1.

Annual Probability P_a	Mean Recurrence Interval ($1/P_a$) Years	Exposure Period (Design Life), *n* (Years)					
		1	5	10	25	50	100
0.1	10	0.1	0.41	0.15	0.93	0.994	0.999
0.04	25	0.04	0.18	0.34	0.64	0.87	0.98
0.034	30	0.034	0.15	0.29	0.58	0.82	0.97
0.02	50	0.02	0.10	0.18	0.40	0.64	0.87
0.013	75	0.013	0.06	0.12	0.28	0.49	0.73
0.01	100	0.01	0.05	0.10	0.22	0.40	0.64
0.0067	150	0.0067	0.03	0.06	0.15	0.28	0.49
0.005	200	0.005	0.02	0.05	0.10	0.22	0.39

Table 1.1 Probability of Exceeding Design Wind Speed during Design Life of Building. (Bungale S. Taranath)

Wind velocities (measured with anemometers usually installed at airports across the country) are averages of the fluctuating velocities measured during an infinite interval of time. The benchmark velocity usually reported in the United States, until the publication of the American Society of Civil Engineers' ASCE 7-95 standard, was the average of the velocities recorded during the time it takes a horizontal column of air, 1 mile long, to pass a fixed point. This is commonly referred to as the *fastest mile wind*.

For example, if a 1 mile column of air is moving at an average velocity of 60 mph, it passes an anemometer in 60 s, the reported velocity being the average of the velocities recorded in 60 s. The fastest mile used in design is the highest velocity recorded in 1 day. The annual extreme mile is the largest of the daily maximums. Furthermore, since the annual extreme mile varies from year to year, wind pressures used in design are based on a wind velocity having a specific mean recurrence interval. Mean recurrence intervals of 20 and 50 years are generally used in building design, the former for determining comfort of occupants, and the latter for designing lateral resisting elements.

VORTEX SHEDDING

In general, wind buffeting against a bluff body is diverted in three mutually perpendicular directions, giving rise to these sets of forces and moments, as shown in Figure 1.4. In aeronautical engineering, all six components, as shown in Figure 1.4, are significant. However, in civil and structural engineering the force and moment corresponding to the vertical axis (lift and yawing moment) are of little significance. Therefore, aside from the effects of uplift forces on large roof areas, flow of wind is considered two-dimensional, as shown in Figure 1.5, consisting of *along wind* and *transverse wind*.

The term along wind—or simply wind—is used to refer to drag forces while transverse wind is the term used to describe crosswind. Generally, in tall building design, the crosswind motion perpendicular to the direction of wind is often more critical than along-wind motion.

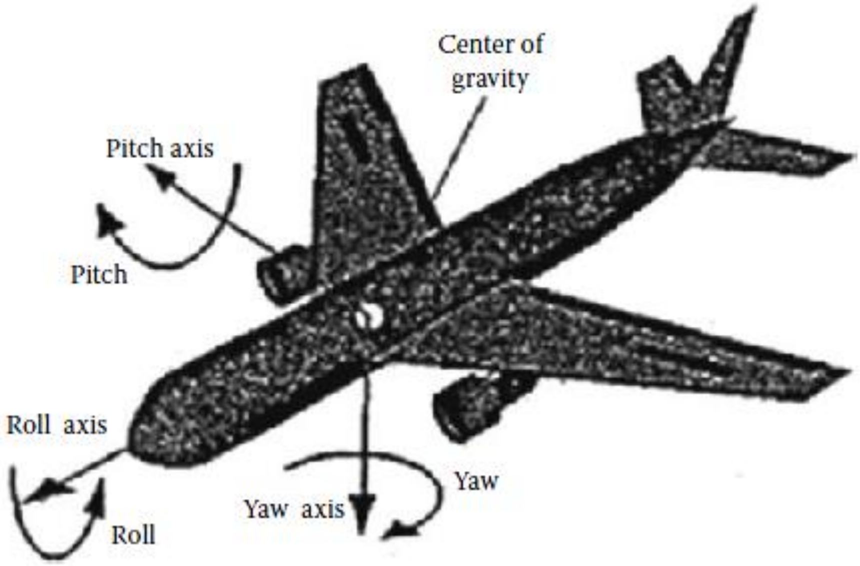

Fig 1.4 Critical components of wind in aeronautical engineering (Bungale S. Taranath)

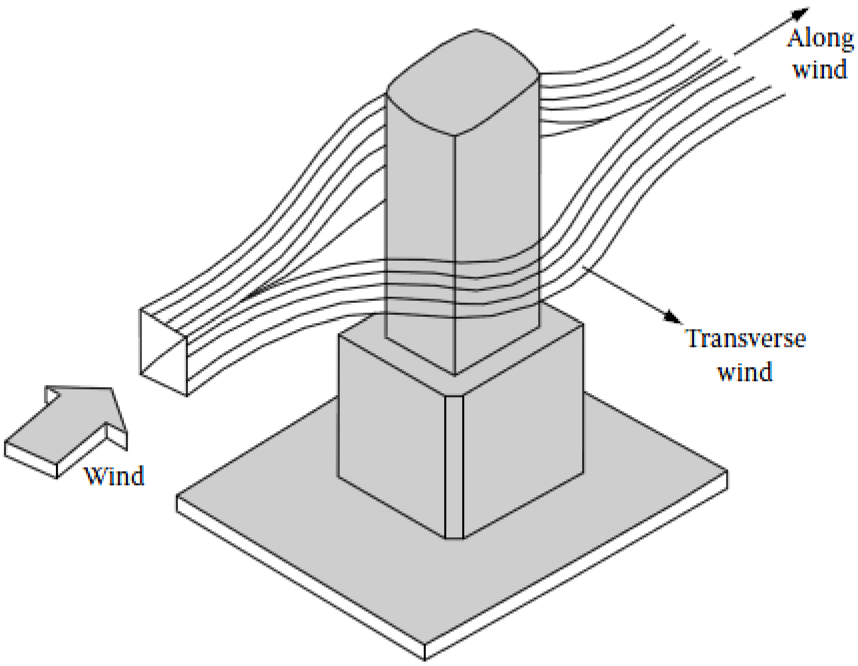

Fig 1.5 Simplified wind flow consisting of along wind and across wind.(Bungale S. Taranath)

Consider a prismatic building subjected to a smooth wind flow. The originally parallel upwind streamlines are displaced on either side of the building, as illustrated in Figure 1.6. This results in spiral vortices being shed periodically from the sides into the downstream flow of wind. At relatively low wind speeds of, say, 50–60 mph (22.3–26.8 m/s), the vortices are shed symmetrically in pairs, one from each side. When the vortices are shed, that is, break away from the surface of the building, an impulse is applied in the transverse direction.

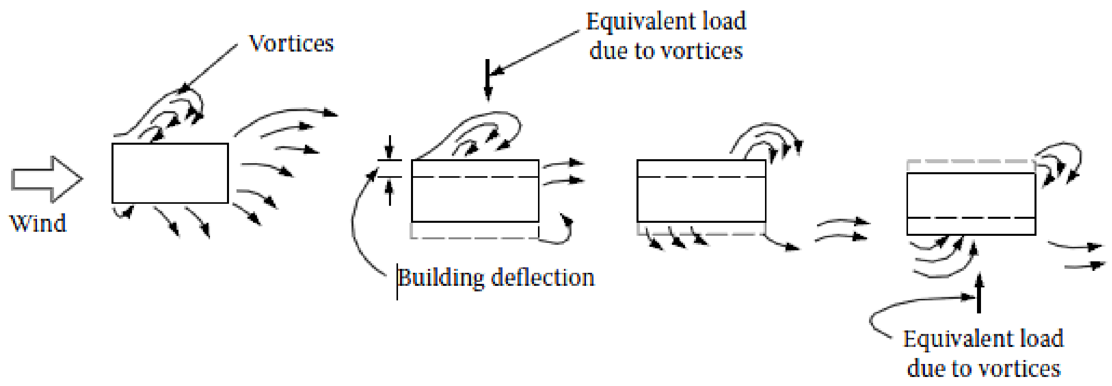

Fig 1.6 Vortex shedding; periodic shedding of vertices generates building vibrations in the transverse direction (Bungale S. Taranath)

At low wind speeds, since the shedding occurs at the same instant on either side of the building, there is no tendency for the building to vibrate in the transverse direction. Therefore the building experiences only along-wind oscillations parallel to wind direction. However, at higher speeds, vortices are shed alternately, first from one side and then from the other side. When this occurs, there is an impulse in the along-wind direction as before, but in addition, there is an impulse in the transverse direction. However, the transverse impulse occurs alternately on opposite sides of the building with a frequency that is precisely half that of the along-wind impulse. This impulse due to transverse shedding gives rise to vibrations in the transverse direction. The phenomenon is called *vortex shedding* or *Karman Vortex Street*, terms well known in the field of fluid mechanics.

There is a simple formula to calculate the frequency of the transverse pulsating forces caused by vortex shedding:

$$f = \left\{ \frac{V \times S}{D} \right\} \tag{1.7}$$

Where

f is the frequency of vortex shedding in hertz
V is the mean wind speed at the top of the building
S is a dimensionless parameter called the Strouhal number for the given shape
D is the diameter of the building

In Equation 1.7, the parameters V and D are expressed in consistent units such as ft/s and ft, respectively.

The Strouhal number is not a constant but varies irregularly with wind velocity. At low air velocities, S is low and increases with velocity up to a limit of 0.21 for a smooth cylinder. This limit is reached for a velocity of about 50 mph (22.4 m/s) and remains almost a constant at 0.20 for wind velocities between 50 and 115 mph (22.4 and 51 m/s).

Consider for illustration purposes, a circular prismatic-shaped high-rise building having a diameter equal to 110 ft (33.5 m) and a height-to-width ratio of 6 with a natural frequency of vibration equal to 0.16 Hz. Assuming a wind velocity of 60 mph (27 m/s), the vortex-shedding frequency is given by

$$f = \frac{V \times 0.2}{110} = 0.16 \text{ Hz} \tag{1.8}$$

Where V is in ft/s.

If the wind velocity increases from 0 to 60 mph (27.0 m/s), the frequency of vortex excitation will rise from 0 to a maximum of 0.16 Hz. Since this frequency happens to be very close to the natural frequency of the building, and assuming very little damping, the structure would vibrate as if its stiffness were zero at a wind speed somewhere around 60 mph (27 m/s). Note the similarity of this phenomenon to the ringing of church bells or the shaking of a tall lamppost whereby a small impulse added to the moving mass at each end of the cycle greatly increases the kinetic energy of the system. Similarly, during vortex shedding an increase in deflection occurs at the end of each swing. If the damping characteristics are small, the vortex shedding can cause building displacements far beyond those predicted on the basis of static analysis.

When the wind speed is such that the shedding frequency becomes approximately the same as the natural frequency of the building, a resonance condition is created. After the structure starts resonating, further increase in wind speed by a few percent will not change the shedding frequency, because the shedding is now controlled by the natural frequency of the structure. The vortex shedding frequency has, so to speak, locked in with the buildings natural frequency. When the wind speed increases significantly above that causing the lock-in phenomenon, the frequency of shedding is again controlled by the speed of the wind. The structure vibrates with the resonant frequency only in the lock-in range. For wind speeds either below or above this range, the vortex shedding will not be critical.

Vortex shedding occurs for many building shapes. The value of *S* for different shapes is determined in wind-tunnel tests by measuring the frequency of shedding for a range of wind velocities. One does not have to know the value of *S* very precisely because the lock-in phenomenon occurs within a range of about 10% of the exact frequency of the structure.

DYNAMIC NATURE OF WIND

Unlike the mean flow of wind, which can be considered as static, wind loads associated with gustiness or turbulence change rapidly and even abruptly, creating effects much larger than if the same loads were applied gradually. Wind loads, therefore, need to be studied as if they were dynamic in nature. The intensity of a wind load depends on how fast it varies and also on the response of the structure. Therefore, whether the pressures on a building created by a wind gust, which may first increase and then decrease, are considered as dynamic or static depends to a large extent on the dynamic response of the structure to which it is applied. Consider the lateral movement of an 800-ft tall building designed for a drift index of *H*/400, subjected to a wind gust. Under wind loads, the building bends slightly as its top moves. It first moves in the direction of wind, with a magnitude of, say, 2 ft (0.61 m), and then starts oscillating back and forth. After moving in the direction of wind, the top goes through its neutral position, then moves approximately 2 ft (0.61 m) in the opposite direction, and continues oscillating back and forth until it eventually stops. The time it takes a building to cycle through a complete oscillation is known as a *period*. The period of oscillation for a tall steel building in the height range of 700 to 1400 ft (214 to 427 m) normally is in the range of 5 to 10 seconds, whereas for a 10-story concrete or masonry building it may be in the range of 0.5 to 1 seconds. The action of a wind gust depends not only on how long it takes the gust to reach its maximum intensity and decrease again, but on the period of the building itself. 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

On the other hand, the gusts can be considered as static loads if the wind load increases and vanishes in a time much longer than the period for the building. For example, a wind gust that develops to its strongest intensity and decreases to zero in 2 seconds is a dynamic load for a tall building with a period of, say, 5 to 10 seconds, but the same 2-second gust is a static load for a low-rise building with a period of less than 2 seconds.

1.6 ALONG AND CROSS-WIND LOADING

Not only is the wind approaching a building a complex phenomenon, but the flow pattern generated around a building is equally complicated by the distortion of the mean flow, flow separation, the formation of vortices, and development of the wake (Mendis P et.al)

Large wind pressure fluctuations due to these effects can occur on the surface of a building. As a result, large aerodynamic loads are imposed on the structural system and intense localised fluctuating forces act on the facade of such structures. Under the collective influence of these fluctuating forces, a building tends to vibrate in rectilinear and torsional modes, as illustrated in Fig. 3. The amplitude of such oscillations is dependent on the nature of the aerodynamic forces and the dynamic characteristics of the building.

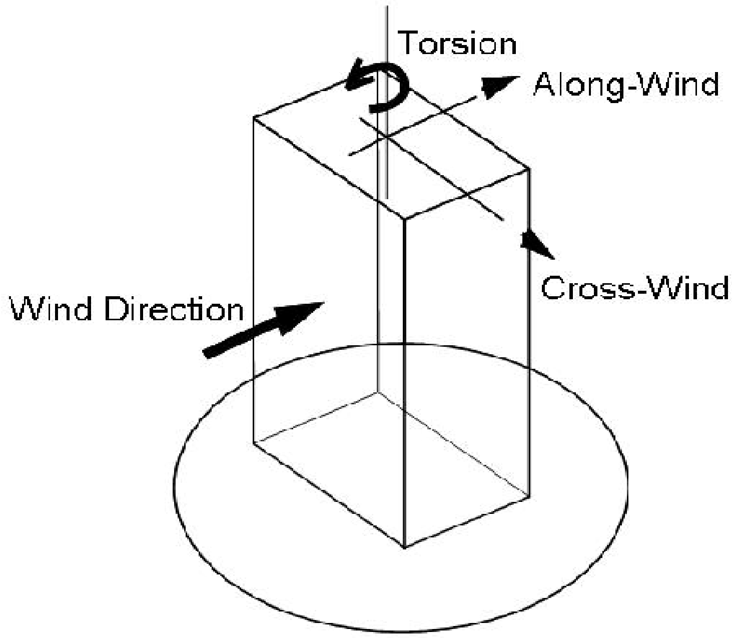

Fig 1.7 Wind response direction (Mendis P et.al)

ALONG WIND LOADING

The along-wind loading or response of a building due to buffeting by wind can be assumed to consist of a mean component due to the action of the mean wind speed (eg, the mean-hourly wind speed) and a fluctuating component due to wind speed variations from the mean. The fluctuating wind is a random mixture of gusts or eddies of various sizes with the larger eddies occurring less often (i.e. with a lower average frequency) than for the smaller eddies. The natural frequency of vibration of most structures is sufficiently higher than the component of the fluctuating load effect imposed by the larger eddies. i.e. the average frequency with which large gusts occur is usually much less than any of the structure's natural frequencies of vibration and so they do not force the structure to respond dynamically. The loading due to those larger gusts (which are sometimes referred to as "background turbulence") can therefore be treated in a similar way as that due to the mean wind. The smaller eddies, however, because they occur more often, may induce the structure to vibrate at or near one (or more) of the structure's natural frequencies of vibration. This in turn induces a magnified dynamic load effect in the structure which can be significant.

The separation of wind loading into mean and fluctuating components is the basis of the so-called "gust-factor" approach, which is treated in many design codes. The mean load component is evaluated from the mean wind speed using pressure and load coefficients. The fluctuating loads are determined separately by a method which makes an allowance for the intensity of turbulence at the site, size reduction effects, and dynamic amplification (Davenport, 1967).

The dynamic response of buildings in the alongwind direction can be predicted with reasonable accuracy by the gust factor approach, provided the wind flow is not significantly affected by the presence of neighbouring tall buildings or surrounding terrain.

CROSS WIND LOADING

There are many examples of slender structures that are susceptible to dynamic motion perpendicular to the direction of the wind. Tall chimneys, street lighting standards, towers and cables frequently exhibit this form of oscillation which can be very significant especially if the structural damping is small.

Crosswind excitation of modern tall buildings and structures can be divided into three mechanisms (AS/NZ1170.2, 2002):

- *Vortex shedding*

The most common source of crosswind excitation is that associated with ‘vortex shedding’. Tall buildings are bluff (as opposed to streamlined) bodies that cause the flow to separate from the surface of the structure, rather than follow*)* the body contour (Fig. 1.8). For a particular structure, the shed vortices have a dominant periodicity that is defined by the Strouhal number. Hence, the structure is subjected to a periodic cross pressure loading, which results in an alternating crosswind force. If the natural frequency of the structure coincides with the shedding frequency of the vortices, large amplitude displacement response may occur and this is often referred to as *the critical velocity effect*. The asymmetric pressure distribution, created by the vortices around the cross section, results in an alternating transverse force as these vortices are shed. If the structure is flexible, oscillation will occur transverse to the wind and the conditions for resonance would exist if the vortex shedding frequency coincides with the natural frequency of the structure. This situation can give rise to very large oscillations and possibly failure.

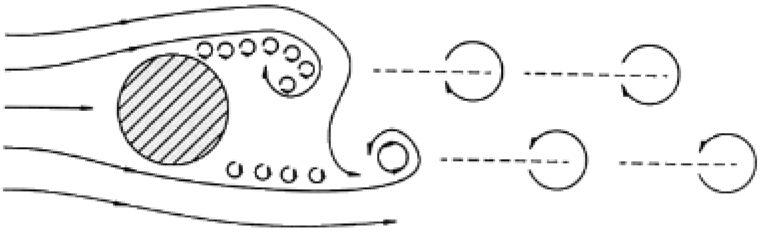

Fig 1.8 Vortex formation in the wake of a bluff object (Bungale S. Taranath)

- *Turbulence mechanism*

The ‘incident turbulence’ mechanism refers to the situation where the turbulence properties of the natural wind give rise to changing wind speeds and directions that directly induce varying lift and drag forces and pitching moments on a structure over a wide band of frequencies. The ability of incident turbulence to produce significant contributions to crosswind response depends very much on the ability to generate a crosswind (lift) force on the structure as a function of longitudinal wind speed and angle of attack. In general, this means sections with a high lift curve slope or pitching moment curve slope, such as a streamline bridge deck section or flat deck roof, are possible candidates for this effect.

- *Higher derivatives of crosswind displacement*

There are three commonly recognized displacement dependent excitations, i.e., ‘galloping’, ‘flutter’ and ‘lock-in’, all of which are also dependent on the effects of turbulence in as much as turbulence affects the wake development and, hence, the aerodynamic derivatives. Many formulae are available to calculate these effects (Holmes, 2001).

Recently computational fluid dynamics techniques (Tamura, 1999) have also been used to evaluate these effects.

LITERATURE REVIEW

2.1 GENERAL

This chapter presents a review of relevant literature to bring out the background of the study undertaken. The research contributions which have a direct relevance are treated in greater detail. Some of the historical works which have contributed greatly to the understanding of the wind loading on structures are also described. First, a brief review of the historical background is presented. The concepts of structural aerodynamics, aerodynamics of bluff bodies, wind loading, and dynamic response of structures, related to work carried out in this thesis, are then discussed. The amount of the literature on the subject has increased rapidly in recent years; particularly to wind such as tall, slender buildings and lightweight Structures. Several of this is available in the proceedings of the conferences which are very helpful to understand the recent developments in wind engineering.

2.2 HISTORIC WORK

Baker (2007) published a paper on past, present and future of wind engineering. This paper firstly considers the history of wind engineering in five rather arbitrary time periods— the ‘‘traditional’’ period (up to 1750), the ‘‘empirical’’ period (1750–1900), the ‘‘establishment’’ period (1900–1960), the period of growth (1960–1980), and the modern period (1980 onwards). In particular it considers the development of the discipline in terms of the socio-economic and intellectual contexts of the time.

In the first period, the traditional period (up to 1750) covers a vast range of different social and intellectual contexts. In wind engineering terms, in most parts of the world, the style of structures to withstand prevailing wind conditions evolved by experience and the development of tradition (**Aynsley et al., 1977**). The people of this time and region had a highly developed ‘‘ritual’’ system that made extensive use of solar and lunar observations, and based much of their daily and seasonal routine around such systems. In this, circular geometries were of major importance—for example in burial mounds and the earlier Neolithic and bronze age henge monuments, such as Stonehenge or Avebury. However, over the centuries they evolved into polygonal and eventually conical structures.

During the second period, the empirical period the social phenomenon that became known as the Industrial revolution began in earnest. This structure was constructed in a traditional style that would have been used for timber bridges. Intellectually this period saw the development of classical hydrodynamics building on the work of Euler, Newton and Bernoulli and later through Navier’s formulation of the fundamental equations of fluid flow in 1845. The use of the techniques of potential flow was extensively studied mathematically. Scientific experimentation was also gaining in respectability and Hadley and Smeaton carried out the first fluid mechanics model experiments in 1759. The first long span bridges that were built to improve communication links inevitably suffered from adverse effects of the wind leading to some spectacular downfalls—such as the 1836 collapse of the Brighton Chain pier due to aero elastic oscillations, and, most famously, the collapse of the Tay bridge in 1879.

In the establishment period (1900-1960) the industrial revolution was coming of age and beginning to influence every aspect of society. Nowhere was this more true than in the military sphere and it is an unfortunate fact that many of the technological advances that were made in the 20th century were largely driven by military considerations, as the European empires vied for supremacy both within Europe and around the world. Between 1931 and 1936, when the Empire State Building was constructed, J.Rathbun made full-scale measurements on it **[Rathbun (1940)].** In this context the work of Baker was taken forward at the National Physical Laboratory in Teddington under the direction of Thomas Stanton, who carried out full scale measurements with tower arrays to attempt to find the ‘‘size’’ of wind gusts (**Davenport, 1999**).

This era also saw the birth of what the author regards as three of the main wind engineering tools.

Firstly there was the development of the wind tunnel driven by the nascent aeronautical industry, although the first wind tunnels predated the advent of aeroplanes, with the pioneering work of Wenham in 1871 (**Surrey, 1999**). In 1893 Irminger measured pressure distributions on a variety of shapes using the flow through a chimney **(Irminger, 1895).** Eiffel made his first wind tunnel measurements in 1909. In the 1930s Irmiger made measurements on building models in low turbulence wind tunnels. **(Irminger and Nokkentved, 1936)**.

Secondly there was the development of codes of practice with the realisation of the need to provide engineers with practical guidance on design to enable environmental loads such as wind to be properly defined.

Thirdly this period saw the beginnings of full-scale measurements of wind loads on structures. It is in the interaction between wind tunnel and full-scale tests that most progress was made in the field of wind engineering during this period. A further wind engineering ‘‘milestone’’ of this era was the collapse of the Tacoma Narrows bridge, about which a very great deal has been written.

The period of 1960 to 1980 is known as the period of growth. During this short 20-year period there were major changes in the nature of society in Europe and across the world, and these were paralleled with significant advances within the discipline of wind engineering. During this period we saw the important contribution of Dr Alan Davenport. In 1961 he elucidated the concept of the wind loading chain, which gave a conceptual framework to the study of wind effects on structures **(Davenport, 1961).** In 1961 Pasquil developed his classification of atmospheric stability that was to remain in use for many decades. During this period, the process of codification of wind effects began in earnest, and a significant number of codes were developed by National Standards Organisations.

The period after 1980 is also known as the modern period. At that time, major societal changes were beginning. There have been significant advances in wind tunnel testing techniques, particularly in terms of instrumentation, with the use of large number of simultaneously monitored pressure transducers and the increasingly frequent use of LDA and PIV techniques for velocity measurements. The last 20 years have also been extremely busy in terms of code development and revision across the world.

Holmes and Lewis performed extensive experimental work on the fluctuating pressure measurements using a small dia meter connecting tube to transmit the pressure from the connecting point, or tap, to the pressure transducer. Their authentic work has provided sufficient guidelines to develop a range near optimum systems for the measurement of fluctuating pressure on models of the buildings in wind tunnels. **(Holmes, 2003)**

This period has also seen a seeming increase in the frequency of major wind disasters both in temperate and in tropical regions.

2.3 ANALYTICAL WORK

The along wind response of isolated tall structures can be estimated using basic principles of random vibration theory in conjunction with information on the characteristics of the oncoming flow, and the aerodynamic loads it induces on the structure. The effect of atmospheric turbulence on the response of an elastic structure immersed in turbulent flow was first published by Liepmann in 1952.

Davenport (1961) gives the statistical concepts of the stationery time series are used to determine the response of a simple structure to a turbulent, gusty wind. This enables the peak stresses, accelerations, deflections, etc., to be expressed in terms of the mean wind velocity, the spectrum of gustiness, and the mechanical and aerodynamic properties of the structure. In this connection it is pointed out that the resistance in fluctuating flow may be significantly greater than that in steady flow, such as that prevailing in most wind tunnel tests. An expression for the spectrum of gustiness near the ground is given, which takes in to account its variation with mean wind velocity, roughness of the terrain, and the height about ground level. The statistical distribution of peak values over a large number of years is related to the statistical distribution of mean values by means of a so called “gust factor”. A map the signing the climate of extreme hourly wind speeds over the British Isles is provided. In association with the gust factor, this enables predictions of extreme peak wind loads with any given return period to be made.

Davenport (1963b) attempts to trace the involution of a satisfactory to the loading of structures by gusts. It is suggested that a statistical approach based on the concepts of the stationary random series appears to offer a promising solution. Some experiments to determine the aerodynamic response of structures to fluctuating turbulent flow are described. Example are given of the application statistical approach to estimate the wind loading on a variety of structures, in noting including long span cables, suspension bridge, towers and skyscrapers.

The along wind response of isolated tall structures can be estimated using basic principles of random vibration theory in conjunction with information on the characteristics of the oncoming flow, and the aerodynamic loads it induces on the structure. The effect of atmospheric turbulence on the response of an elastic structure immersed in turbulent flow was first published by Liepmann in 1952. Using this concept Davenport developed models representing the turbulent wind flow near the ground **Davenport (1961).** These included a height independent expression for the spectrum of longitudinal velocity fluctuations. He further developed the "Gust Factor Approach" for analytical prediction of along wind response of tall buildings; **Davenport (1967)**. Davenport emphasized that the fluctuating component of the building motion can be conveniently divided into one part responding to wind frequency components significantly lower than the buil ding natural frequency; and the other part exhibiting a resonant response. The ratio of this 'background' response to 'resonant' response depends on the relation between the geometric and dynamic properties of the building to those of the turbulent natural wind. So in different situations either of these dynamic phenomenons may dominate. Davenport showed how spectral analysis could be used to determine building response spectral density (stresses or displacements). He shows how the various statistical processes transformed into their spectral components. This phenomenon can be represented and analyzed in the following manner. By starting with the `gusts', represented as velocity spectrum multiplying this on a frequency basis by the aerodynamic admittance (the transfer function squared), the force (or pressure) spectrum can be determined. From this, by multiplying by the mechanical admittance, the response spectrum is determined.

Vellozzi and Cohen (1968) published a procedure for the along wind response of tall buildings in which a reduction factor was introduced for the fluctuating pressures on the leeward face of a building as it is understood that there is no perfect correlation between fluctuating pressures on windward and leeward faces of a building. However, it was shown by Simiu (1973a) that owing to the manner in which this factor is applied, the procedure of Vellozzi & Cohen underestimates the resonant amplification effects.

On the basis of his analysis and experiments, Vickery developed a further refinement of the **Gust Factor Method'; Vickery (1971),** As Vickery notes, his method tended to give conservative results for aspect ratio over four. Vickery concluded that his refined method could predict a building gust factor to a typical accuracy of 5 -10% for well defined basic data, compared with other methods. Vellozzi and Cohen (1968) published a procedure for the along wind response of tall buildings in which a reduction factor was introduced for the fluctuating pressures on the leeward face of a building as it is understood that there is no perfect correlation between fluctuating pressures on windward and leeward faces of a building.

Analysis of three dimensional structures subjected to random loading yields an expression of the dynamic response which reflects unequivocally the effect of the along wind cross correlation of the loads. This effect and the error involved in ignoring or overestimating it, are then evaluated using generally accepted assumptions and experimental results available in literature. Some of these assumptions are analyzed with a view to further improving the accuracy of the gust factor by correctly modelling in its expression the physical features of the actual flow. Simiu (1973a) has shown that by incorporating along wind cross-correlation between windward & leeward sides, the dynamic part of response and the gust response factor are reduced considerably. Later he showed **[Simiu (1974)]** that by considering variation of spectra with height, the responses further reduce. He also showed [Simiu (1976)] that the dynamic response and the gust factors estimated using either Davenport (1967) or Vickery (1971) may be as high as few hundred percent, while those using Vellozzi & Cohen (1968) are on the lower side. For a typical building [Simiu & Lozier(1975)], he calculated the gust factor as 1.96 while the same using Davenport(1967) approach was 2.83, using Vickery(1971) was 3.38 and using Vellozzi & Cohen(1968) was 1.53.

It was shown by **Simiu (1973a)** that owing to the manner in which this factor is applied, the procedure of Vellozzi & Cohen underestimates the resonant amplification effects.

Simiu (1973a, 1974, 1976, 1980) has developed a procedure for determination of along wind response incorporating meteorological parameters. He showed that dynamic response of three dimensional tall structures may be represented as a sum of contributions due to the pressures on the windward side, the pressures on the leeward side, and the along wind cross-correlation of these pressures. Later, he presented improved forms of longitudinal wind spectra in which the variation of spectra with height is taken into account. A program for the computation of the along wind deflection and accelerations was developed incorporating these meteorological and aero dynamical changes which was further modified by Simiu in 1980. Graphs and charts have been developed for the simplified hand calculations; **Simiu (1976) & (1980).**

In current methods for determining Along-wind structural response, it is assume that wind profiles are described by empirical power laws and that turbulence spectra are independent of height. In this paper, the adequacy of these assumptions is assessed in the light of recently established results of boundary layer meteorology. An improved method for determining wind profiles is presented, and expression for the dynamic Along-wind response, including def lections and accelerations, are proposed. In addition to the variation of wind spectra with height, these expressions take in to account the pressure correlations in the Along-wind direction, determined in accordance with basic theory and known experimental results.

Peyrot et al. (1974) presented a method in which Wind forces at discrete points on a tall building are simulated on the digital computer as a multi dimensional stochastic process. The cross-correlation structure of the wind is treated in a simplified manner. Building responses to wind samples are obtained in the time domain by the finite element method. Mathematical models of both and building are designed to minimize computer time and yet retain the essential characteristics of the response. The random response of tall buildings to wind loading can be studied either in the frequency domain or in the time domain.

The problem of dynamic along wind response of structures to forces induced by atmospheric turbulence is treated in this paper **Solari G. (1982)**. Starting from the classical formulation, the study analyzes the behaviour of two structural standard models, called point-like and three dimensional, respectively. The treatment of the problem presented in the paper leads to a closed form expression of the along wind response. The remarkable simplicity and the very high precision of the proposed method is pointed out in general terms and illustrated by two examples. In conclusion some prospects for possible future applications referred to this solution are outlined and briefly discussed.

Morteza A. M. et al. (1985) investigates the dynamic responses of tall buildings subject to wind loading. One of the objectives of this research is to study the importance of the torsional dynamic response, coupled with translational responses. Finite element modelling is used to assemble the stiffness matrix of the structure. Torsional degrees of freedom are considered in the stiffness formulation of elements and systems. Aerodynamic forces on a tall building are calculated assuming a deterministic, pseudo turbulent approach. These aerodynamic forces are distributed over the height of the building. The equivalent concentrated aerodynamic loads , acting at each floor level are calculated using the principle of virtual displacements. The governing differential equations are nonlinear. An iterative method of solution is used to calculate the responses. In order to simplify the solution procedure, a method of linearization is applied to the aerodynamic forces and the final result is a set of second order differential equations with constant coefficients. A 15 -story building is modelled as an application. One comparative study has been made between the finite element model and an equivalent continuous cantilever beam model. A second comparative study is between nonlinear and linear models. The results are presented as response spectra for different gust frequencies.

According to **Lawson (1985)**, the term “Building” is difficult to interpret because it is general. The other topics in the response of structures series are “chimneys” ‘ towers”, “Bridges”, industrial roofs” and “Cooling towers”. And they are more specific, so that in this paper any structure not included in the list of other topics will be considered as an honorary building.

Solari G. (1988), state the equivalent wind spectrum technique is a mathematical model according to which wind is schematized as a stochastic stationary Gaussian process made up of a mean-speed profile on which an equivalent turbulent fluctuation, perfectly coherent in space, is super imposed. The equivalent criterion is formulated by defining a fictitious velocity fluctuation, random function of time only, giving rise to power spectra of fluctuating modal force that approximate, optimally, the corresponding modal spectra related to the actual turbulence configuration. This paper presents the basic assumptions and the theoretical steps leading to the characterization of the equivalent velocity fluctuation through a power spectrum assigned in closed form. The met had proposed herein allows one to estimate the dynamic along-wind response of structures, both in frequency and in time domain, with a high level of precision and simplicity; furthermore it makes it possible to treat wind effects, as well as those of earthquakes, through the well-known response spectrum technique .

Solari G. (1989) formulates a theoretically consistent definition of the wind response spectrum based upon the equivalent wind spectrum technique, a calculation procedure by means of which wind is schematized as a stochastic stationary Gaussian process characterized by a mean velocity profile on which an equivalent turbulent fluctuation, perfectly coherent in space, is superimposed. The method presented herein allows the evaluation of the dynamic along-wind response of structures, as well as of the structural behaviour to the seismic ground motion, by the well - known response spectrum technique. This procedure, parallelly applied to wind and earthquake actions, reveals significant conceptual and formal analogies, leading to results characterized by the same order of approximation.
On the basis of the turbulence theory, by the analogous method, a new longitudinal wind velocity spectrum of fluctuations is established by Yuxin & Yiran (1989). New expressions for Mean along wind displacement, spectral density and rms response are formulated and a computer program is developed.

Hajra B and P. N. Godbole (2006) presented that most international codes and standards have kept pace with the changing scenario in wind engineering and have updated their codes and standards. The IS -875 (part-3)-1987 still makes use of hourly mean wind speed and cumbersome charts to arrive at the Gust Factor for calculating Along Wind response on a tall building. A document "Review of Indian Wind Code-IS- 875 (part-3) 1987",prepared by the Indian Institute of Technology, Kanpur suggests revision in the present IS-code to make it consistent and bring it close to the available international standards. This paper discusses the present IS-code, the revisions suggested by IIT Kanpur together with other international codes for computing Along Wind response on a tall building with the help of three examples of tall buildings.

Yin Z. et al. (2002) in their paper did a comparative study of major International codes and standards for along wind load effects on tall structure. ASCE 7 (United states), AS1170.2-89 (Australia), NBC-1995 (Canada), RLB-AIJ-1993 (Japan), and Eurocode-1993 (Europe) are examined in this study.

Most international codes and standards utilize the ‘‘gust loading Factor” (GLF) approach for assessing the dynamic along-wind loads and their effects on tall structures. Several modifications based on the first GLF model by Davenport followed, which include Vellozzi and Cohen (1968), Vickery (1970) Simiu and Scanlan (1996), and Solari (1993a, b. Variations of these models have been adopted by major international codes and standards.

In their study they took an example of a tall building to compare the estimates of wind load effects based on the codes and standards considered. The building particulars are H = 200 m, B = D 33 m; f_1 = 0.2 Hz, and linear mode shape in two translation directions; ξ = 0.01; C_d = 1.3; and building density = 180 kg/m3. The building is located at the edge of a central business district with exposure A on one side and exposure C on the other; and the basic 3 s gust wind velocity = 40 m/s. For simplicity, the effects of the wind direction, topography, shielding, importance, and return period are ignored in the following discussion.

Following formulas were used to find the Gust loading factor:

	ASCE 7	AS1170.2	NBC	RLB-AIJ	Eurocode
G^{a}	$0.925\left(\frac{1+r\sqrt{g_Q^2\mathrm{B}+g_R^2\mathrm{R}}}{1+g_v\cdot r}\right)^{b,c}$	$1+r\sqrt{g_v^2\mathrm{B}(1+w)^2+g_f^2\mathrm{R}}^{d}$	$1+g_f r\sqrt{\mathrm{B}+\mathrm{R}}$	$1+g_f r\sqrt{\mathrm{B}+\mathrm{R}}$	$\frac{1+g_f r\sqrt{\mathrm{B}+\mathrm{R}}}{1+3.5r}^{c}$
T	3,600 s	3,600 s	3,600 s	600 s	600 s
$\overline{z}$	$0.6H$	H	H	H	$0.6H$
r	$r=1.7I_{\overline{z}}$ [e]	$r=2\cdot I_{\overline{z}}$	$r=\sqrt{2K/C_{eH}}$ [f]	$r=(3+3\alpha)/(2+\alpha)\cdot I_{\overline{z}}$ [g]	$r=2I_{\overline{z}}$
g	$g_Q=g_v=3.4$ $g_R=g_R(T,f_1)$ [h]	$g_v=3.7$; $g_f=\sqrt{2\ln(T\cdot f_1)}$	$g_f=g_R(T,\upsilon)$; [g] $\upsilon=f_1\sqrt{SE/(SE+\zeta\mathrm{B})}$	$g_f=\sqrt{2\ln(T\cdot\upsilon)+1.2}$ $\upsilon=f_1\sqrt{\mathrm{R}/(\mathrm{B}+\mathrm{R})}$	$g_f=g_R(T,\upsilon)$ [h] $\upsilon=\sqrt{(\upsilon_0^2\mathrm{B}+f_1^2\mathrm{R})/(\mathrm{B}+\mathrm{R})}$
B	$\frac{1}{1+0.63\left(\frac{B+H}{L_{\overline{z}}}\right)^{0.63}}$	$\frac{1}{1+\frac{\sqrt{36H^2+64B^2}}{L_H}}$	$\frac{2}{3}\int_0^{914/H}\frac{1}{1+\frac{xH}{457}}\frac{1}{1+\frac{xB}{122}}\frac{x}{(1+x^2)^{4/3}}dx$	$1-\frac{1}{\left\{1+5.1\left(\frac{L_H}{\sqrt{HB}}\right)^{1.3}\left(\frac{B}{H}\right)^{k}\right\}^{1/3}}$	$\frac{1}{1+0.9\left(\frac{B+H}{L_{\overline{z}}}\right)^{0.63}}$
E^{i}	$9.5N_1/(1+10.3N_1)^{5/3}$ [e]	$0.6N_1/(2+N_1^2)^{5/6}$	$2N_1^2/3(1+N_1^2)^{4/3}$	$4N_1/(1+71N_1^2)^{5/6}$	$6.8N_1/(1+10.2N_1)^{5/3}$
S	$R_H R_B(0.53+0.47R_D)$ [j]	$\frac{1}{\left[1+3.5\frac{f_1H}{V_H}\right]\left[1+4\frac{f_1B}{V_H}\right]}$	$\frac{1}{\left[1+\frac{8f_1H}{3V_H}\right]\left[1+\frac{10f_1B}{V_H}\right]}$	$\frac{0.84}{\left[1+\frac{2.1f_1H}{V_H}\right]\cdot\left[1+\frac{2.1f_1B}{V_H}\right]}$ [k]	$R_H R_B$ [j]

[a]Expressions for GLF in this table are not necessarily reproduced from the original codes and standards, but are rewritten in the standard form [refer to Eq. (2.1) or Eq. (2.2)].
[b]0.925 is an adjustment factor used to make the wind load in the updated code consistent with the former version.
[c]Numerator is the displacement GLF and the denominator is the GF for the wind velocity pressure.
[d]w is an approximate consideration of the quadratic wind velocity term (Vickery 1995).
[e]A 3 s low-pass filter has been included (Solari and Kareem 1998).
[f]K is provided for different terrains in NBC.
[g]A 0.75 factor is used to account for nonuniform load distribution (RLB-AIJ 1994).
[h]$g_R(T,f_1)$; see Eq. (2.3) by substituting relevant parameters.
[i]$E=f_1S_v(f_1)/\sigma_v^2$ and $N_1=f_1L_{\overline{z}}/V_{\overline{z}}$.
[j]$R_l=1/\eta-1/2\eta^2(1-e^{-2\eta})$ for $\eta>0$; and $R_l=1$ for $\eta=0$. R_H, $\eta=4.6f_1H/V_{\overline{z}}$; R_B, $\eta=4.6f_1B/V_{\overline{z}}$; and R_D, $\eta=15.4f_1D/V_{\overline{z}}$.
[k]Aerodynamic admittance function is equal to 0.84 at zero frequency.

TABLE 2.1 Calculation of Gust loading factor in codes and standards

$$G=1+g\cdot r\cdot\sqrt{\mathrm{B}+\mathrm{R}} \tag{2.1}$$

$$G=1+r\cdot\sqrt{g_B^2\cdot\mathrm{B}+g_R^2\cdot\mathrm{R}} \tag{2.2}$$

$$g=\sqrt{2\ln(\upsilon T)}+0.5772/\sqrt{2\ln(\upsilon T)} \tag{2.3}$$

Mean wind velocity in codes and standard are given in table 2.2

	ASCE 7				AS1170.2 (fitted)				NBC		RLB-AIJ		Eurocode (fitted)	
	3 s		1 h		3 s		1 h		1 h		10 min		10 min	
	b	α	*b*	α	*b*	α	*b*	α	*b*	α	*b*	α	*b*	α
A	0.66	0.20	0.30	0.33	0.76	0.14	0.29	0.28	0.43	0.36	0.39	0.35	0.55	0.29
B	0.85	0.14	0.45	0.25	0.91	0.10	0.45	0.20	0.67	0.25	0.58	0.27	0.77	0.21
C	**1.00**	0.11	0.65	0.15	**1.04**	0.07	0.58	0.16	**1.00**	0.14	0.79	0.20	**1.00**	0.16
D	1.09	0.09	0.80	0.11	1.18	0.04	0.69	0.13			**1.00**	0.15	1.17	0.12
E											1.23	0.10		

Note: Basic wind velocity refers to the condition where the coefficient *b* is equal to unity, which is shown in bold in this table.

Table 2.2 Mean wind velocity profiles in codes and standards (Eq. 2.4)

The wind profile provided in the codes and standards can be expressed in term of following general power law:

$$V(z) = V_0.b.(z/10)^{\alpha} \tag{2.4}$$

Turbulence intensity profile can be expressed in terms of a power law:

$$I(z) = c.(z/10)^{-d} \tag{2.5}$$

	ASCE 7		AS1170.2 (fitted)		NBC (derived)[a]		RLB-AIJ		Eurocode (fitted)	
Terrain	*c*	*d*	*c*	*d*	*c*	*d*	*c*	*d*	*c*	*d*
A	0.450	0.167	0.453	0.300	0.621	0.360	0.402	0.400	0.434	0.290
B	0.300	0.167	0.323	0.300	0.335	0.250	0.361	0.320	0.285	0.210
C	0.200	0.167	0.259	0.300	0.200	0.140	0.259	0.250	0.189	0.160
D	0.150	0.167	0.194	0.300			0.204	0.200	0.145	0.120
E							0.162	0.150		

[a]Turbulence intensity profile is not explicitly available in NBC. The data herein are derived by rewriting the code procedure in the standard form.

Table 2.3 Turbulence Intensity profiles in codes and standards (Eq. 2.5)

In the results GLF is expressed in terms of base bending moment response which is calculated using formula:

$$\hat{M} = G_Y \cdot \overline{M} \tag{2.6}$$

Where,

$\overline{M}$ = base bending moment under the mean wind speed
$\hat{M}$ = peak base bending moment response

Results are given in table 2.4

		ASCE 7		AS1170.2[a]		NBC		RLB-AIJ		Eurocode	
		A	*C*	*A*	*C*	*A*	*C*	*A*	*C*	*A*	*C*
V_0 (m/s)			40 (3 s)		40 (3 s)		26 (1 h)[b]		27 (10 min)[b]		27 (10 min)[b]
$\bar{z}$ (m)			120		200		200		200		120
$V_{\bar{z}}$ (m/s)		27.5	38.1	26.7	37.3	32.6	39.5	30.4	42.3	30.7	39.3
r		0.506	0.225	0.368	0.210	0.423	0.303	0.276	0.180	0.422	0.254
$L_{\bar{z}}$ (m)		190	250	2115	2115	1220	1220	258	258	197	236
B		0.583	0.624	0.633	0.633	0.300	0.300	0.582	0.582	0.500	0.529
E		0.140	0.144	0.094	0.117	0.170	0.191	0.080	0.100	0.106	0.109
S		0.048	0.079	0.080	0.123	0.077	0.101	0.154	0.212	0.087	0.121
R		0.525	0.889	0.596	1.138	1.031	1.524	0.967	1.655	0.726	1.039
g_f		$g_R=3.79$; $g_v=3.40$		$g_R=3.63$; $g_v=3.70$		3.759	3.768	3.209	3.235	3.208	3.225
G_B	3 s	0.447	0.316							0.386	0.315
	10 min							0.676	0.443	0.958[c]	0.596[c]
	1 h	1.214[c]	0.559[c]	1.083	0.618	0.870	0.626				
G_R	3 s	0.472	0.421							0.466	0.442
	10 min							0.872	0.747	1.154[c]	0.835[c]
	1 h	1.283[c]	0.742[c]	1.030	0.813	1.614	1.411				
G	3 s	0.990	1.051							1.009	1.073
	10 min							2.103 (78.2%)[g]	1.868 (100.8%)[g]	2.500[c] (92.9%)[g]	2.026[c] (109.2%)[g]
	1 h	2.691[c]	1.854[c]	2.495 (92.8%)[g]	2.021 (109.2%)[g]	2.833 (105.3%)[g]	2.544 (137.3%)[g]				
$\bar{M}$ (kN·m)	3 s	1,035,400	1,465,400								
	10 min							367,810 (86.3%)[g]	833,050 (105%)[g]	528,250 (124%)[g]	837,510 (106%)[g]
	1 h	425,980[d]	790,360[d]	297,600 (69.9%)[g]	644,490 (81.5%)[g]	417,880 (98.1%)[g]	735,690 (93.1%)[g]				
$\hat{M}$ (kN·m)	10 min							773,410 (75.5%)[g]	1,556,400 (101.1%)[g]	1,320,400 (128.6%)[g]	1,696,700 (110.3%)[g]
	1 h	1,024,808 1,146,260[e]	1,539,848 1,465,015[e]	742,420 (72.4%)[g]	1,302,400 (84.5%)[g]	1,183,900 (115.7%)[g]	1,871,300 (121.1%)[g]				

[a]Neglect the correction for the quadratic term.
[b]Wind velocity GFs of 0.65 and 0.676 are used to transfer the 3 s gust wind velocity to those in 1 h and 10 min mean, respectively.
[c]Displacement GLF, computed by neglecting the dominator in the GLF expression.
[d]Computed based on the 1 h mean wind profile in ASCE 7.
[e]Product of the GLF and the mean base bending moment both based on the 1 h averaging time.
[f]Correspond to the base bending moment in e.
[g]Comparing with the result by ASCE 7. For simplicity, the effect of a shorter averaging time (10 min) in RLB-AIJ and Eurocode is not considered.

Table 2.4 Wind load calculation as per various international codes and standards

In the conclusion it is observed that all major international codes and standards are based on the GLF approach for estimating the maximum wind load effects in the along-wind direction; however, each employs unique definitions of wind field characteristics, including the mean wind-velocity profile, turbulence intensity profile, wind spectrum and turbulence length scale. These nuances in the wind field characteristics have resulted in discrepancies not only in the GLF estimates, but also in the mean wind loads, which correspondingly lead to significant variations in the estimates of the ESWL and associated wind induced load effect.

METHODOLOGY

3.1 INTRODUCTION

There are two major methods of calculating wind load on a structure which are followed in India. In this report we are going to study the latter in detail.

- Wind tunnel experimental method for static and dynamic effect.
- Detailed analytical method as per IS 875 Part 3 for static and dynamic method.

3.2 WIND TUNNEL

A ***wind tunnel*** is a research tool used in aerodynamic research. It is used to study the effects of air moving past solid objects.

Wind tunnels were first proposed as a means of studying vehicles (primarily airplanes) in free flight. The wind tunnel was envisioned as a means of reversing the usual paradigm: instead of the air's standing still and the aircraft moving at speed through it, the same effect would be obtained if the aircraft stood still and the air moved at speed past it. In that way a stationary observer could study the aircraft in action, and could measure the aerodynamic forces being imposed on the aircraft.

Later, wind tunnel study came into its own: the effects of wind on manmade structures or objects needed to be studied, when buildings became tall enough to present large surfaces to the wind, and the resulting forces had to be resisted by the building's internal structure. Determining such forces was required before building codes could specify the required strength of such buildings.

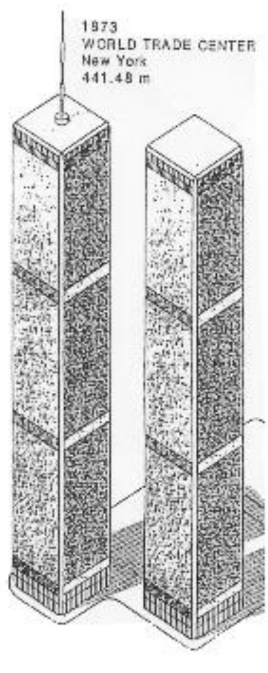

Fig 3.1 World trade centre (1973-2001) (Bungale S. Taranath)

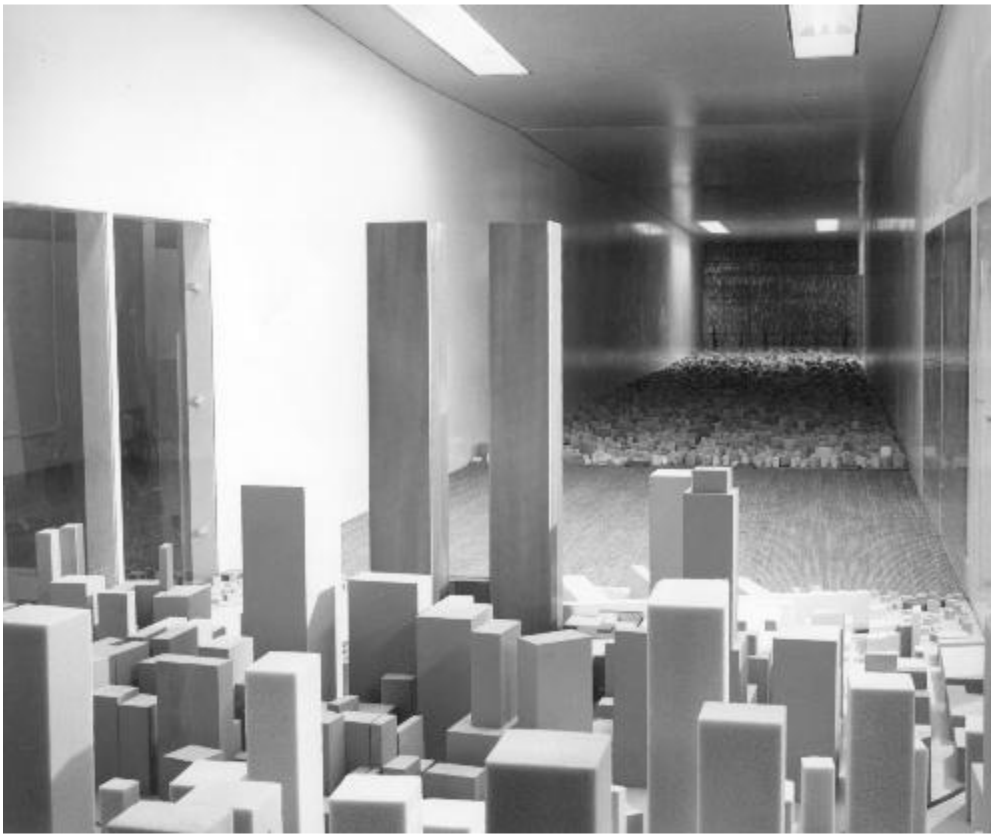

Fig 3.2 Wind tunnel test on WTC (Bungale S. Taranath)

The objectives of wind tunnel test are:

- Dynamic response
- Drag, Vortex shedding and wind separation from building surface.
- To decide building configurations (shape).

HOW IT WORKS

Air is blown or sucked through a duct equipped with a viewing port and instrumentation where models or geometrical shapes are mounted for study. Typically the air is moved through the tunnel using a series of fans. For very large wind tunnels several meters in diameter, a single large fan is not practical, and so instead an array of multiple fans are used in parallel to provide sufficient airflow. Due to the sheer volume and speed of air movement required, the fans may be powered by stationary turbofan engines rather than electric motors.

The airflow created by the fans that is entering the tunnel is itself highly turbulent due to the fan blade motion (when the fan is **blowing** air into the test section - when it is **sucking** air out of the test section downstream, the fan-blade turbulence is not a factor), and so is not directly useful for accurate measurements. The air moving through the tunnel needs to be relatively turbulence-free and laminar. To correct this problem, closely-spaced vertical and horizontal air vanes are used to smooth out the turbulent airflow before reaching the subject of the testing.

Due to the effects of viscosity, the cross-section of a wind tunnel is typically circular rather than square, because there will be greater flow constriction in the corners of a square tunnel that can make the flow turbulent. A circular tunnel provides a smoother flow.

The inside facing of the tunnel is typically as smooth as possible, to reduce surface drag and turbulence that could impact the accuracy of the testing. Even smooth walls induce some drag into the airflow, and so the object being tested is usually kept near the centre of the tunnel, with an empty buffer zone between the object and the tunnel walls. There are correction factors to relate wind tunnel test results to open-air results.

Lighting is usually recessed into the circular walls of the tunnel and shines in through windows. If the light were mounted on the inside surface of the tunnel in a conventional manner, the light bulb would generate turbulence as the air blows around it. Similarly, observation is usually done through transparent portholes into the tunnel. Rather than simply being flat discs, these lighting and observation windows may be curved to match the cross-section of the tunnel and further reduce turbulence around the window.

Various techniques are used to study the actual airflow around the geometry and compare it with theoretical results, which must also take into account the Reynolds number and Mach number for the regime of operation.

3.3 WIND ANALYSIS BY IS 875 Part 3

IS 875 Part 3 has given us two ways to analyse a building for wind load.

- Static Analysis by regular and draft code
- Dynamic analysis (Gust factor method) by draft code.

STATIC ANALYSIS

IS 875 part 3 gives guidelines to determine wind forces on different components of buildings. It consists of following steps:

a) Determine basic wind speed.
b) Obtain design wind speed.
c) Calculate design wind pressure.
d) Calculate wind pressure on building.

These steps are explained below:

Basic Wind Speed

For finding basic wind pressure in any place in India. IS 873(part 3) divides the country in six zones. It is based on peak gust velocity averaged over a short time interval of about 3 seconds over a period of 50 years. The values correspond to the speed at 10 m height above ground level and in open terrain. It may be observed that highest basic wind speed is 55m/s and the lowest is 33 m/s

Design Wind Speed

The design wind speed for any site may be obtained as:

$$V_z = k_1\, k_2\, k_3\, V_b \tag{3.1}$$

Where,

k_1 = risk coefficient
k_2 = terrain, height and structure size factor
k_3 = topography factor

Wind Pressure

The design wind pressure at any height above ground level shall be calculated using the following expression

$$P_z = 0.6\, V_z^2 \text{ N/m}^2 \tag{3.2}$$

Where,

P_z = design wind pressure in N/m^2 at height z
V_z = design wind velocity in m/s at height z

Wind Pressure on Building

For calculating the wind load on individual structural elements it is essential to take into account of pressure difference between opposite faces of such elements. If internal as well as external pressures are found then wind load acting in a direction normal to the individual structural element or cladding unit is:

$$F = (C_{pe} - C_{pi})\, A\, P_z \tag{3.3}$$

Where,

C_{pe} = external pressure coefficient
C_{pi} = internal pressure coefficient
A = surface area of structural element or cladding unit in m^2
P_z = design wind pressure in N/m^2

Positive wind coefficient indicates the force in towards the structural element and negative coefficient indicated it is away from the structure element

DYNAMIC ANALYSIS

IS 875 (Part 3) gives following requirement for use of dynamic analysis of a structure.

- Buildings and closed structures with a height to minimum lateral dimension ratio of more than 5
- Buildings and closed structures whose natural frequency in the first mode is less than 1.0 Hz

Any building or structure which does not satisfy either of the above two criteria shall be examined for dynamic effects of wind.

Time period

The fundamental time period (I) may either be established by experimental observations on similar buildings or calculated by any rational method of analysis. In the absence of such data, T may be determined as follows for multi-storeyed buildings:

a) For moment resisting frames without bracing or shear walls for resisting the lateral loads.

$$T = 0.9\ n \qquad (3.4)$$

Where,

n = number of stories

b) For all other

$$T = 0.09H/(d)^{1/2} \qquad (3.5)$$

Where,

H = Total height of the main structure of the building in meters
d= maximum base dimension of building in meters in a direction parallel to the applied wind force.

Motion due to Vortex Shedding

Slender structure- For a structure, the shedding frequency, η shall be determined by the following formula:

$$H = (SV_d)/b \qquad (3.6)$$

Where,

S = Strouhal number

a. For circular structures
   S = 0.2 for bV_z not greater than 7
   S = 0.25 for bV_z greater than 7

b. For rectangular structures
   $S = 0.15$ for all values of bV_z

V_d = Design wind velocity
b = breadth of a structure or structural members in the horizontal plane normal to the wind direction.

<u>Gust Factor (GF) or Gust effectiveness factor (GEF) method</u>

Only the method of calculating load along wind or drag load by using gust factor method is given in the code since methods for calculating load across-wind or other components are not fully matured for all types of structures.

Variation of hourly mean speed with height

$$V_z = V_b\, k_1\, k_2\, k_3\, V_b \qquad (3.7)$$

Where,

V_z = hourly mean wind speed in m/sec at height z
V_b = regional basic wind speed in m/sec
k_1 = Probability factor
k_2 = terrain, height and structure size factor
k_3 = topography factor

Along Wind load

Along wind load on a structure on a strip area (A_e) at any height (z) is given by:

$$F_z = C_f\, A_e\, p_z\, G \qquad (3.8)$$

Where,

F_z = along wind load on the structure at any height z corresponding to strip area A_e
C_t = force coefficient for the building,
A_e = effective frontal area considered for the structure at height c,
p_z = design pressure at height z due to hourly mean wind obtained as $0.6\, V_z^2$ (N/m^2),
G = gust factor (peak load/mean load), and is given by:

$$G = 1 + g_t r\, [B\,(1+\varphi)^2 + SE/\beta\,]^{1/2} \qquad (3.9)$$

Where,

g_t = peak factor defined as the ratio of the expected peak value to the root mean value of a fluctuating load
r = roughness factor which is dependent on the size of the structure in relation to the ground roughness.

*The value of $g_t r$ is given in Fig 8 of IS 875 of Fig 3.3

B = Background factor given in Fig 9 of IS 875 or Fig 3.4

SE/β = measure of the resonant component of the fluctuating wind
S = size reduction factor given in Fig 10 of IS 875 or fig 3.5
E = measure of available energy in the wind stream at the natural frequency of the structure given in Fig 11 of IS 875 or fig 3.6
β = damping coefficient (as a fraction of critical damping) of the structure as given in Table 34 of IS 875 or table 3.1
$\varphi = [g_f r(B)^{1/2}]/4$ and is to be accountable for building less than 75m high in terrain category 4 and for buildings less than 25m high in terrain category 3, and is taken as zero in all other cases.

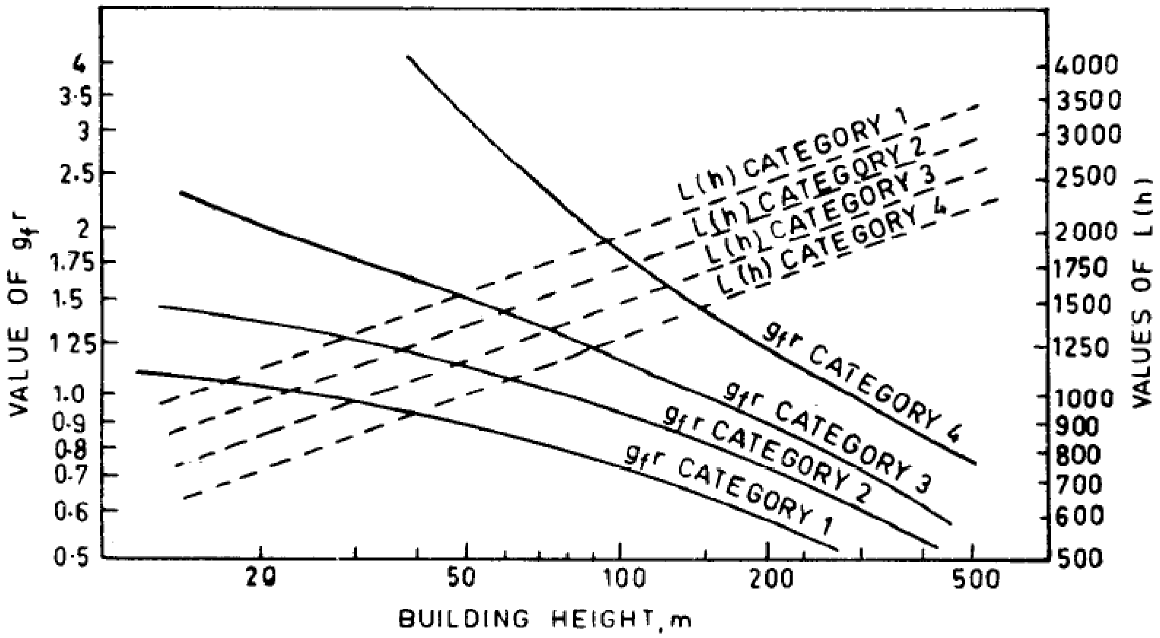

Fig 3.3 Value of $g_f r$ and L (h) (IS 875)

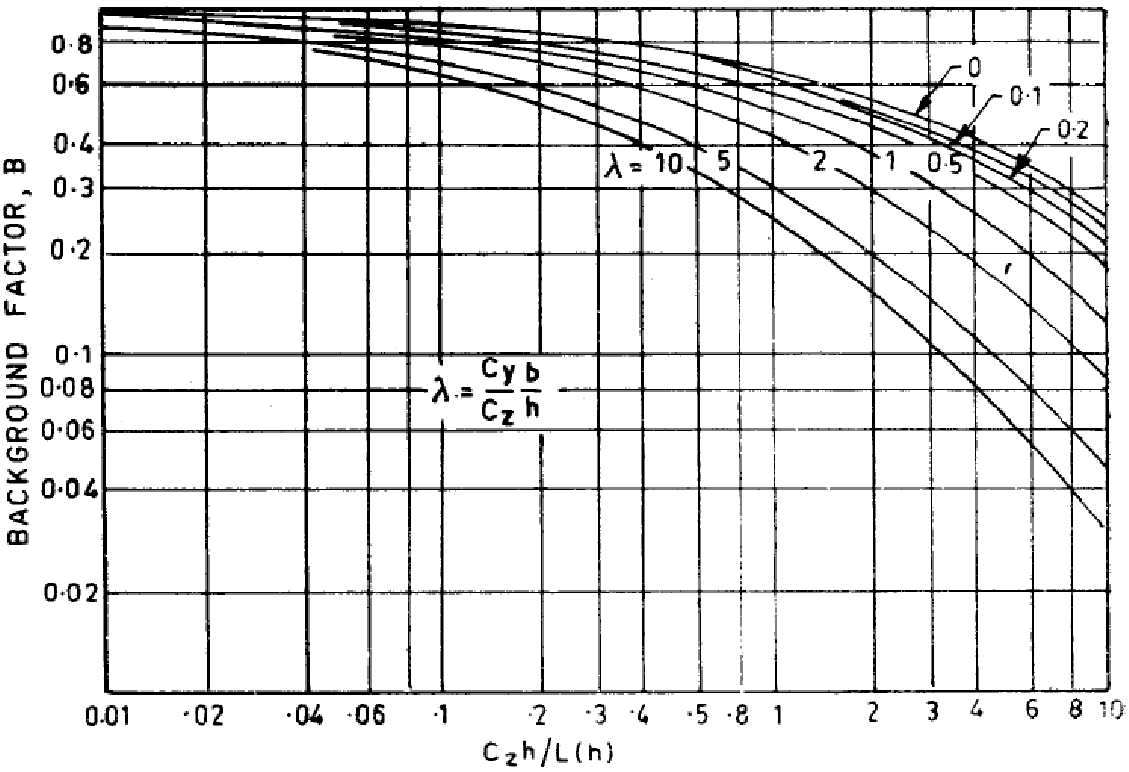

Fig 3.4 Background factor (IS 875)

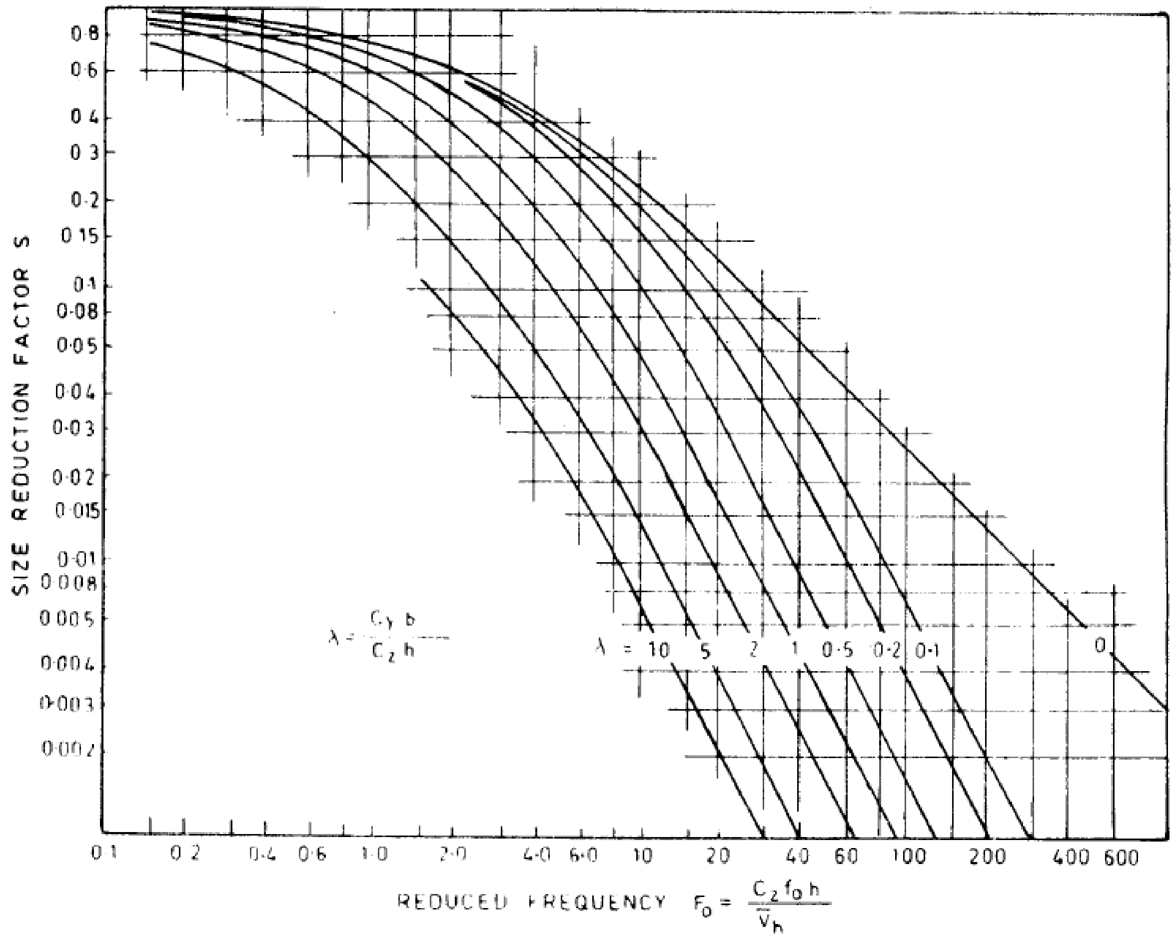

Fig 3.5 Size reduction factor S (IS 875)

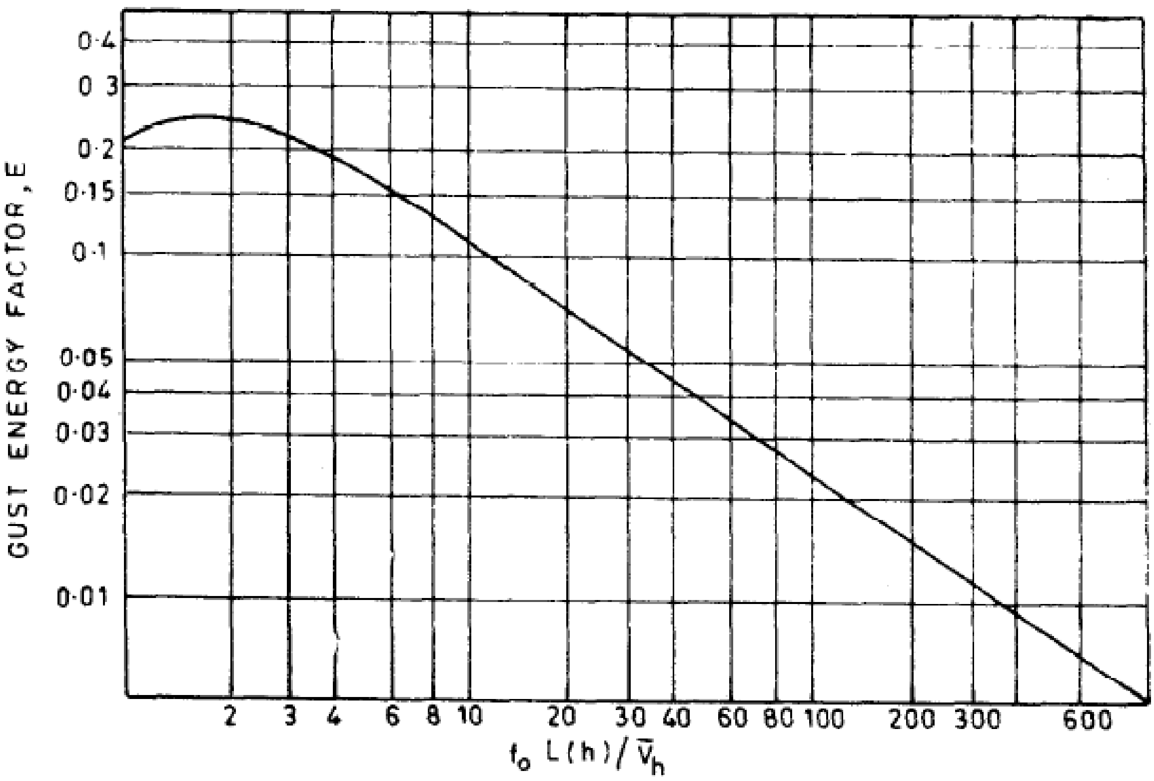

Fig 3.6 Gust energy factor (IS 875)

SUGGESTED VALUES OF DAMPING COEFFICIENT	
NATURE OF STRUCUTRE	DAMPING COEFFICIENT (β)
Welded steel structures	0.01
Bolted steel structures	0.02
Reinforced concrete structures	0.016

Table 3.1 Damping coefficient (IS 875)

In Fig 3.3, 3.4. 3.5, 3.6,

$$\lambda = \frac{C_y\, b}{C_z\, h} \text{ and } F_o = \frac{C_z f_o\, h}{\bar{V}_h} \tag{3.10}$$

Where,

C_y = lateral correlation constant which may be taken as 10 in the absence of more precise data,
C_z = longitudinal correlation constant which may be taken as 12 in absence of more precise data
b = breadth of a structure normal to the wind stream
h = height of structure
$V_h = V_z$ = hourly mean wind speed at height z,
f_0 = natural frequency of the structure,
L(h) = a measure of turbulence length scale from Fig 3.3

Peak acceleration

Peak acceleration along the wind direction at the top of the structure is given by the following formula:

$$a = (2\pi f_o)^2\, x\, g_f r \sqrt{\frac{SE}{\beta}} \tag{3.11}$$

IMPLEMENTATION

INTRODUCTION

In this study we are going to learn about the wind effect on structures having different height. We will check the base shear of the different building. Building 5 specification will be same as the building used in paper by **Yin Z. et al. (2002).**

For this we have taken 5 different types of structure with varying height and dimension. The specifications of the building are given below:

- *Building 1*

No. of storey = 5
Total height = 17.5 m
Width = 16 m
Beadth = 20 m

- *Building 2*

No. of storey = 10
Total height = 35 m
Width = 20 m
Beadth = 35 m

- *Building 3*

No. of storey = 50
Total height = 175 m
Width = 24 m
Beadth = 40 m

- *Building 4*

No. of storey = 100
Total height = 350 m
Width = 40 m
Beadth = 80 m

- *Building 5*

Total height = 200 m
Width = 33 m
Beadth = 33 m

BUILDING 1

Breadth, b = 20 m
Width, a = 16 m
Height, h = 17.5 m

As per IS 875 Part 3

- Height to minimum lateral dimension ratio = 1.093 < 5
- Natural frequency of building

  T = 0.1 n = 0.5

  Natural frequency = 2 Hz ≥ 1 Hz

Dynamic analysis is not required in this building.

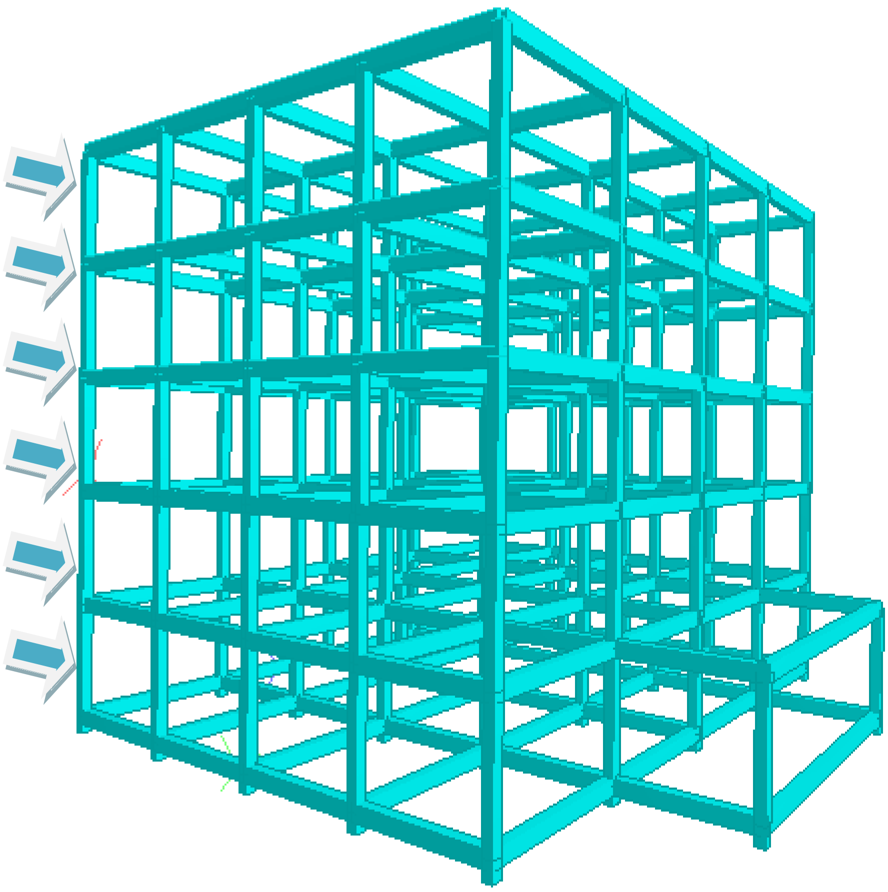

Fig 4.1 3-D model of 5 story structure.

WIND LOAD CALCULATION

(5 STORY)

GIVEN			
DIMENSION OF STRUCTURE			
Breadth, b	20	m	
Width, a	16	m	
Height, h	17.5	m	
DESIGN LIFE OF STRUCTURE	50	yrs	
PROBABITY FACTOR (k1)	1		table 1 of IS 875
TERRAIN CATERGORY	3		clause 5.3.2.1
TOPOGRAPHY FACTOR (k3)	1		
BASIC WIND SPEED	47	m/sec	APPENDIX A
MEAN WIND SPEED FACTOR (10m)	0.88		table 2
MEAN WIND VELOCITY AT 10 m height, V_{10}	47.00	m/sec	
LOAD CALCULATION			
EXTERNAL PRESSURE COEFFICIENTS (C_p) for walls of rectangular clad buildings (Table 4)			
BUILDING HEIGHT RATIO (h/b)	0.88		
BREADTH WIDHT RATIO (b/a)	1.25		
AT 90^o			
A	0.7		
B	-0.25		
C	-0.6		
D	-0.6		
AT 0^o			
A	-0.6		
B	-0.6		
C	0.7		
D	-0.25		
NOTATIONS FOR TABLE			
For table 1, 3			
h(m) = Height of building			
dh(m) = Diffrence in height of building			
$k_{2(avg)}$ = Average of k_2 factor obtained from table 2 as per heigh			
$p_d = 0.6 \times (V_{10} \times k_1 \times k_2 \times k_3)^2 C_p$			
A (m^2) = Area of wall (dh x b)			
Force (F, kN) = Area x p_d			
For table 2, 4			
z(m) = Average of height			
F_{TOTAL} = Total Force (Absolute $F_{WALL\ A}$ + Absolute $F_{WALL\ B}$)			
Bending Moment = Total Force x Height			

Table 4.1 Wind load calculation of 5 story structure

TABLE 1 : FORCE CASE 1 AT 90^{O}

S.No	h(m)	dh(m)	$k_{2(avg)}$	p_d		A (m^2)	F (kN)	
				WALL A	WALL B		WALL A	WALL B
1	5	5	0.88	0.72	-0.26	100	71.8	-25.7
2	10	5	0.88	0.72	-0.26	100	71.8	-25.7
3	15	5	0.91	0.77	-0.27	100	76.8	-27.4
4	17.5	2.5	0.95	0.83	-0.30	50	41.7	-14.9

TABLE 2 : B.M CASE 1 AT 90^{o}

S.No	h(m)	z (m)	F (Kn)$_{WALL\ A}$	F (Kn)$_{WALL\ B}$	F $_{TOTAL}$	B.M
1	5	2.5	71.8	-25.7	97.51	243.77
2	10	7.5	71.8	-25.7	97.51	731.30
3	15	12.5	76.8	-27.4	104.27	1303.36
4	17.5	16.25	41.7	-14.9	56.58	919.41
TOTAL					355.86	3197.84

BASE SHEAR FORCE	355.86	Kn
BASE BENDING MOMENT	3197.84	kNm

TABLE 3 : FORCE CASE 1 AT 0^{O}

S.No	h(m)	dh(m)	$k_{2(avg)}$	p_d		A (m^2)	F (kN)	
				WALL A	WALL B		WALL A	WALL B
1	5	5	0.88	0.72	-0.26	80	57.5	-20.5
2	10	5	0.88	0.72	-0.26	80	57.5	-20.5
3	15	5	0.91	0.77	-0.27	80	61.5	-22.0
4	17.5	2.5	0.95	0.83	-0.30	40	33.4	-11.9

TABLE 4 : B.M CASE 1 AT 0^{o}

S.No	h(m)	z (m)	F (Kn)$_{WALL\ A}$	F (Kn)$_{WALL\ B}$	F $_{TOTAL}$	B.M
1	5	2.5	57.5	-20.5	78.01	195.01
2	10	7.5	57.5	-20.5	78.01	585.04
3	15	12.5	61.5	-22.0	83.41	1042.69
4	17.5	16.25	33.4	-11.9	45.26	735.53
TOTAL					284.69	2558.27

BASE SHEAR FORCE	284.69	Kn
BASE BENDING MOMENT	2558.27	kNm

Table 4.1(cont.) Wind load calculation of 5 story structure

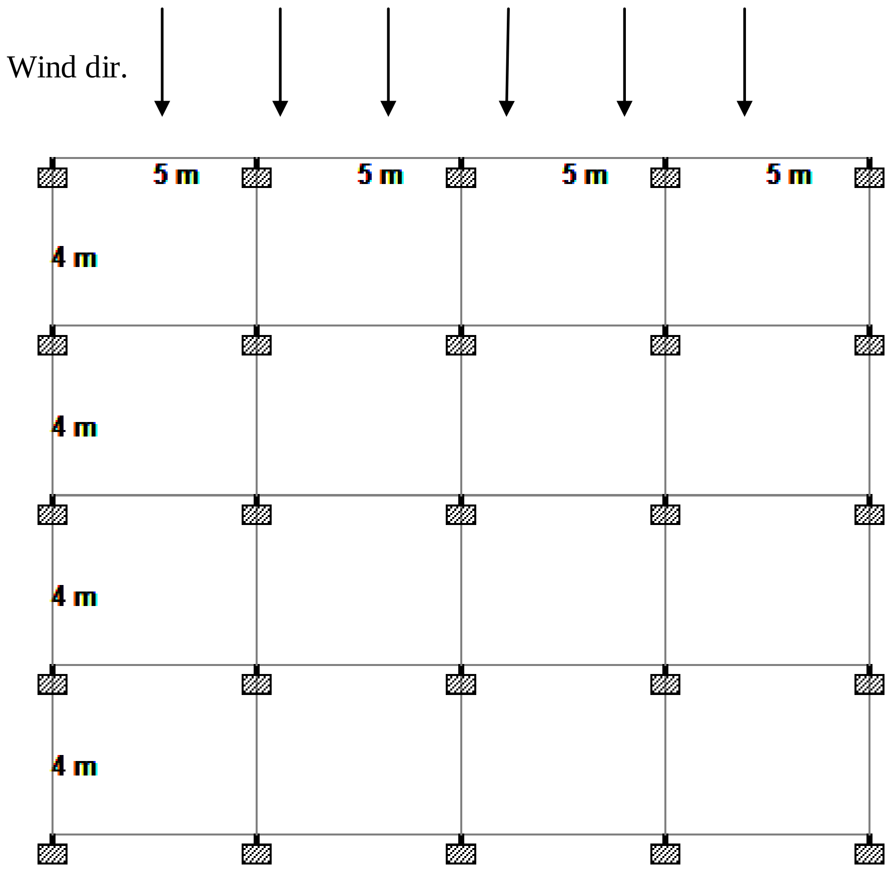

Fig 4.2 Plan of 5 story structure

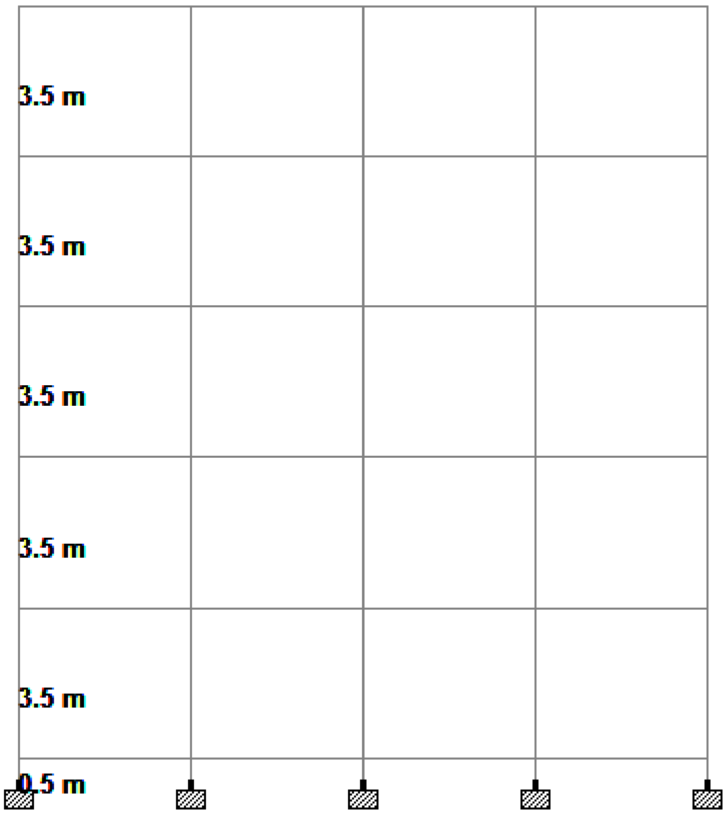

Fig 4.3 Elevation of 5 story structure

BUILDING 2

Breadth, b = 35 m
Width, a = 20 m
Height, h = 35 m

As per IS 875 Part 3

- Height to minimum lateral dimension ratio = 1.75 < 5
- Natural frequency of building

  T = 0.1 n = 1

  Natural frequency = 1 Hz ≥ 1 Hz

Dynamic analysis is not required in this building.

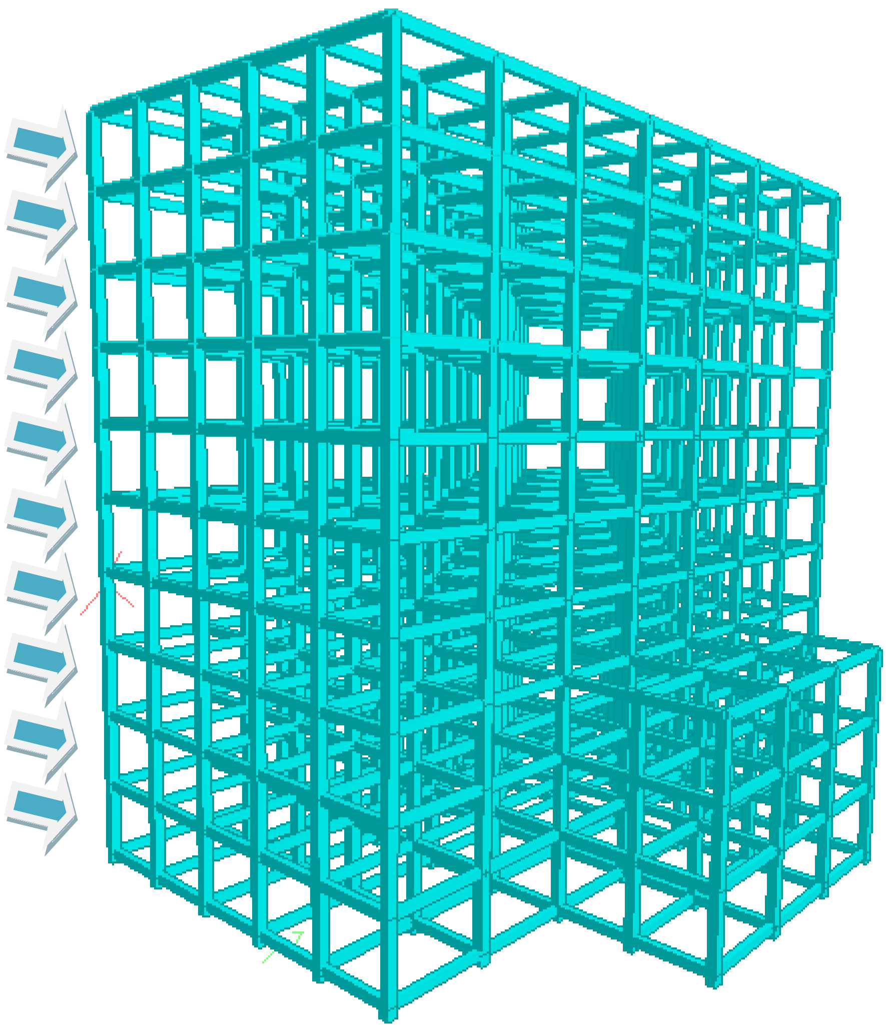

Fig 4.4 3-D model of 10 story structure

WIND LOAD CALCULATION

(10 STORY)

GIVEN			
DIMENSION OF STRUCTURE			
Breadth, b	35	m	
Width, a	20	m	
Height, h	35	m	
DESIGN LIFE OF STRUCTURE	50	yrs	
PROBABITY FACTOR (k1)	1		table 1 of IS 875
TERRAIN CATERGORY	3		clause 5.3.2.1
TOPOGRAPHY FACTOR (k3)	1		
BASIC WIND SPEED	47	m/sec	APPENDIX A
MEAN WIND SPEED FACTOR (10m)	0.88		table 2
MEAN WIND VELOCITY AT 10 m height, V_{10}	47.00	m/sec	
LOAD CALCULATION			
EXTERNAL PRESSURE COEFFICIENTS (C_p) for walls of rectangular clad buildings (Table 4)			
BUILDING HEIGHT RATIO (h/b)	1		
BREADTH WIDHT RATIO (b/a)	1.75		
AT 90^o			
A	0.7		
B	-0.3		
C	-0.5		
D	-0.5		
AT 0^o			
A	-0.5		
B	-0.5		
C	0.7		
D	-0.1		
NOTATIONS FOR TABLE			
For table 1, 3			
h(m) = Height of building			
dh(m) = Diffrence in height of building			
$k_{2(avg)}$ = Average of k_2 factor obtained from table 2 as per heigh			
$p_d = 0.6 \times (V_{10} \times k_1 \times k_2 \times k_3)^2 C_p$			
A (m^2) = Area of wall (dh x b)			
Force (F, kN) = Area x p_d			
z(m) = Average of height			
F_{TOTAL} = Total Force (Absolute $F_{WALL\ A}$ + Absolute $F_{WALL\ B}$)			
Bending Moment = Total Force x Height			

Table 4.2 Wind load calculation of 10 story structure

TABLE 1 : FORCE CASE 1 AT 90^{O}

S.No	h(m)	dh(m)	$k_{2(avg)}$	p_d		A (m^2)	F (kN)	
				WALL A	WALL B		WALL A	WALL B
1	10	10	0.88	0.72	-0.31	350	143.7	-61.6
2	15	5	0.91	0.77	-0.33	175	76.8	-32.9
3	20	5	0.96	0.86	-0.37	175	85.5	-36.6
4	25	5	0.99	0.91	-0.39	175	91.4	-39.2
5	30	5	1.02	0.96	-0.41	175	96.1	-41.2
6	35	5	1.04	1.00	-0.43	175	99.9	-42.8

TABLE 2 : B.M CASE 1 AT 90^{o}

S.No	h(m)	z (m)	F (Kn)$_{WALL\ A}$	F (Kn)$_{WALL\ B}$	F $_{TOTAL}$	B.M
1	10	5	143.7	-61.6	205.28	1026.39
2	15	12.5	76.8	-32.9	109.76	1371.95
3	20	17.5	85.5	-36.6	122.15	2137.61
4	25	22.5	91.4	-39.2	130.56	2937.59
5	30	27.5	96.1	-41.2	137.22	3773.54
6	35	32.5	99.9	-42.8	142.67	4636.67
TOTAL					847.63	15883.74

BASE SHEAR FORCE	847.63	Kn
BASE BENDING MOMENT	15883.74	kNm

TABLE 3 : FORCE CASE 1 AT 0^{O}

S.No	h(m)	dh(m)	$k_{2(avg)}$	p_d		A (m^2)	F (kN)	
				WALL A	WALL B		WALL A	WALL B
1	10	10	0.88	0.72	-0.10	200	143.7	-20.5
2	15	5	0.91	0.77	-0.11	100	76.8	-11.0
3	20	5	0.96	0.86	-0.12	100	85.5	-12.2
4	25	5	0.99	0.91	-0.13	100	91.4	-13.1
5	30	5	1.02	0.96	-0.14	100	96.1	-13.7
6	35	5	1.04	1.00	-0.71	100	99.9	-71.3

TABLE 4 : B.M CASE 1 AT 0^{o}

S.No	h(m)	z (m)	F (Kn)$_{WALL\ A}$	F (Kn)$_{WALL\ B}$	F $_{TOTAL}$	B.M
1	10	5	143.7	-20.5	164.22	821.11
2	15	12.5	76.8	-11.0	87.81	1097.56
3	20	17.5	85.5	-12.2	97.72	1710.08
4	25	22.5	91.4	-13.1	104.45	2350.07
5	30	27.5	96.1	-13.7	109.78	3018.83
6	35	32.5	99.9	-71.3	171.20	5564.01
TOTAL					735.17	14561.67

BASE SHEAR FORCE	735.17	Kn
BASE BENDING MOMENT	14561.67	kNm

Table 4.2(cont.) Wind load calculation of 10 story structure

Wind dir.

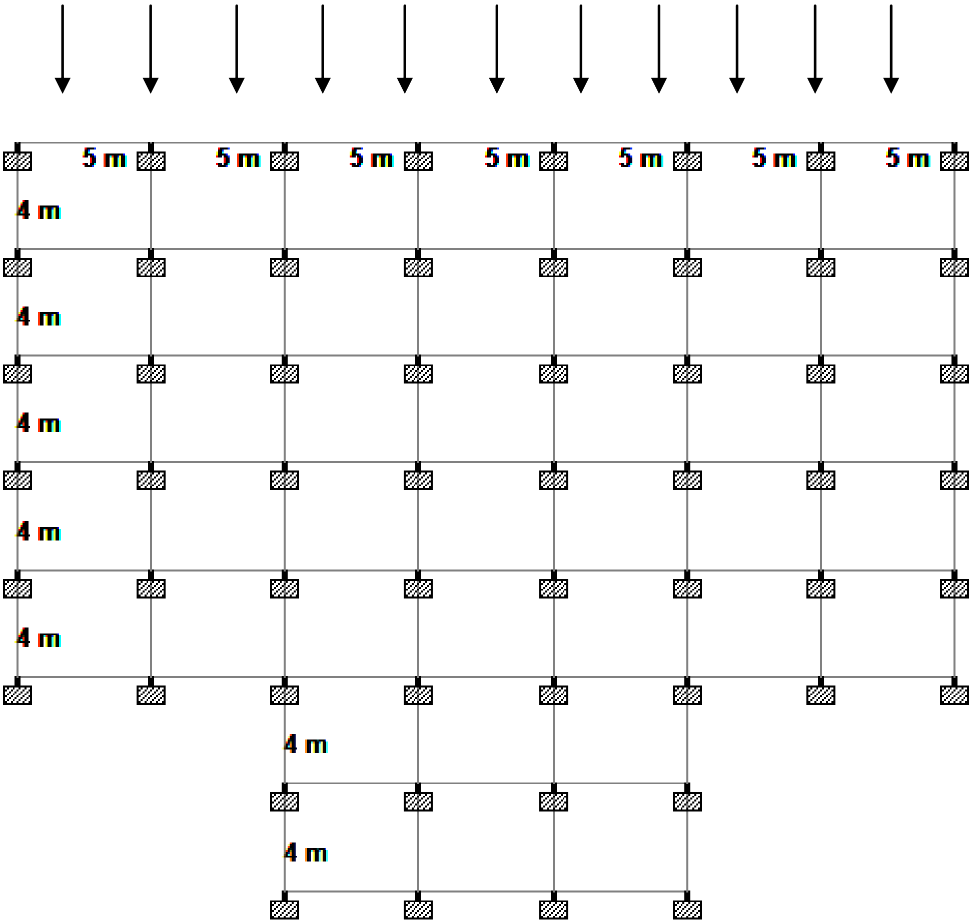

Fig 4.5 Plan of 10 story structure

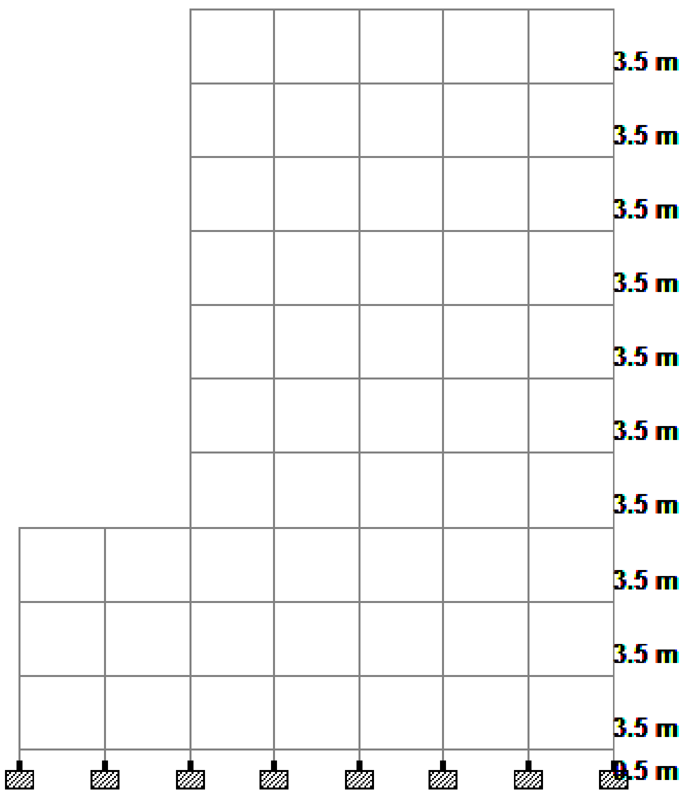

Fig 4.6 Elevation

BUILDING 3

Breadth, b = 40 m
Width, a = 24 m
Height, h = 175 m

As per IS 875 Part 3

- Height to minimum lateral dimension ratio = 7.29 > 5
- Natural frequency of building

  $T = (0.09H)/d^{1/2} = 2.49$

  Natural frequency = 0.4 Hz < 1 Hz

Dynamic analysis is required in this building.

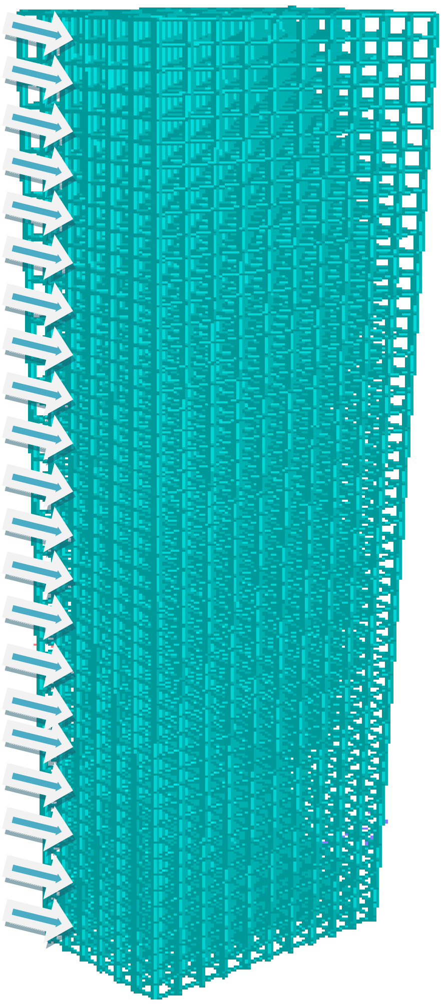

Fig 4.7 3-D model of 50 story structure

WIND LOAD CALCULATION			
(50 STORY)			
GIVEN			
DIMENSION OF STRUCTURE			
Breadth, b	40	m	
Width, a	24	m	
Height, h	175	m	
DESIGN LIFE OF STRUCTURE	50	yrs	
TYPE OF STRUCTURE (S)	0.15		clause 7.2
TERRAIN CATERGORY	3		clause 5.3.2.1
WIND SPEED FACTOR (k2)	0.86		table 33
TOPOGRAPHY FACTOR (k3)	1		
STRUCTURE DAMPING COFFICIENT (β)	0.016		Table 34
TIME PERIOD (T)	2.49		clause 7.1 note 1 (a)
SWAY FREQUENCY, (f_o)	0.40	Hz	1/T
MODE SHAPE (k)	1		
AVERAGE BUILDING DENSITY	275	kg/m^2	
BASIC WIND SPEED	47	m/sec	APPENDIX A
HOURLY MEAN WIND SPEED FACTOR (10m)	0.5		table 33
HOURLY MEAN WIND VELOCITY AT 10 m height, V_{10}	23.50	m/sec	
HOURLY MEAN WIND VELOCITY AT ROOF LEVEL, V_n	40.42	m/sec	
LOAD CALCULATION			
SHEDDING FREQUENCY (η) = SxV_d/b	0.15	< .2	
Factor g/r	0.78		Fig 8
BACKGROUND FACTORS			
Longitudnal correlation constant C_Z	12		Pg 52 of IS 875
Lateral correlation constant Cy	10		Pg 52 of IS 875
L(h)	1800		Fig 8
C_z x h/L(h)	1.17		
$\lambda = (C_y \times b)/(C_z \times h)$	0.19		required for cal. B
Background factor, B	0.6		Fig 9
Reduced frequency, $F_o = (C_z \times f_o \times h)/V_n$	20.86		
Size reduction factor, S	0.075		Fig 10
$f_oL(h)/V_n$	17.88		
Gust energy factor, E	0.09		Fig 11
SE/β	0.42		
Gust factor, $G=1+g_fr(B+(SE/\beta))^{1/2}$	1.79		

Table 4.3 Wind load calculation of 50 story structure

h(m) = Height of building									
d(h)m =Diffrence between height									
z (m) = Avg of height									
k_2 = Table 33 of IS 875									
A_c = Area of wall									
Force (along wind) at height z on strip area Ae = $C_fGA_eF_z$									
Pressure at height z due to $V_z = (V_{10} \times (k_2/0.50))$									
$F_z = C_fGA_ep_z$									
Bending Moment = Force x Area									
S.No	h(m)	dh(m)	z (m)	k_2	V_z (m/sec)	p_z (N/m^2)	A_c (m^2)	Fz (kN)	B.M
1	10	10	5	0.5	23.50	331.35	400	331.86	1659.32
2	15	5	12.5	0.59	27.73	461.37	200	231.04	2888.05
3	20	5	17.5	0.59	27.73	461.37	200	231.04	4043.27
4	30	10	25	0.64	30.08	542.88	400	543.73	13593.16
5	50	20	40	0.7	32.90	649.45	800	1300.91	52036.30
6	125	75	87.5	0.815	38.31	880.36	3000	6612.97	578635.20
7	150	25	137.5	0.84	39.48	935.20	1000	2341.63	321974.59
8	175	25	162.5	0.86	40.42	980.27	1000	2454.47	398850.92
TOTAL							7000	14047.66	1373680.80
BASE BENDING MOMENT							1373681	kNm	

Table 4.3(cont.) Wind load calculation of 50 story structure

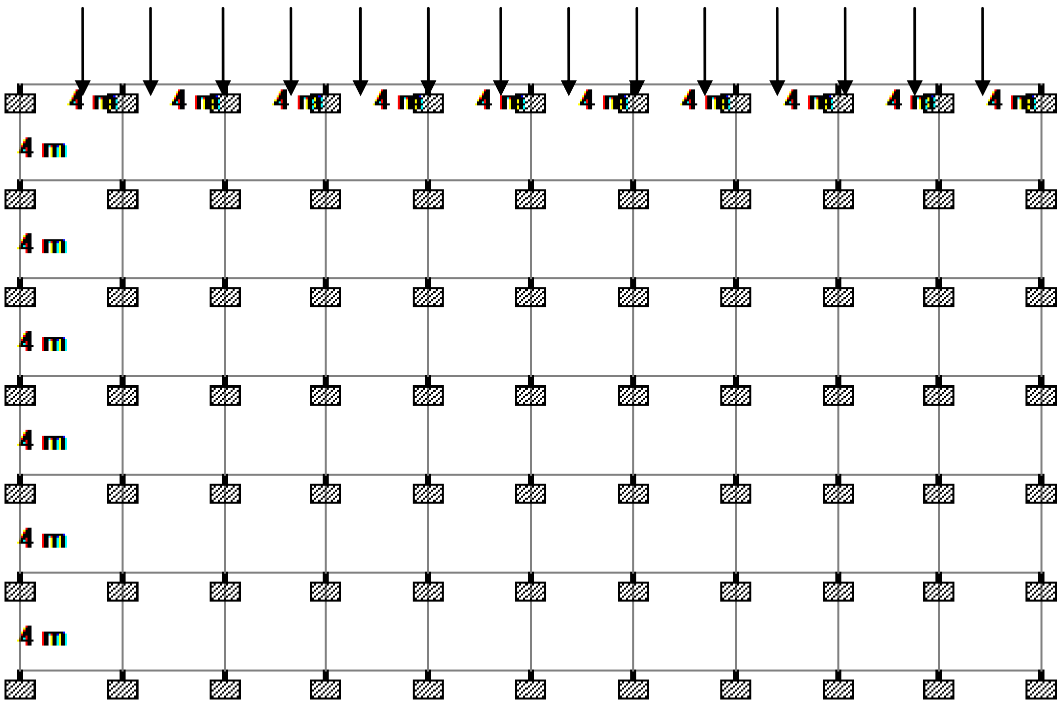

Fig 4.8 Plan

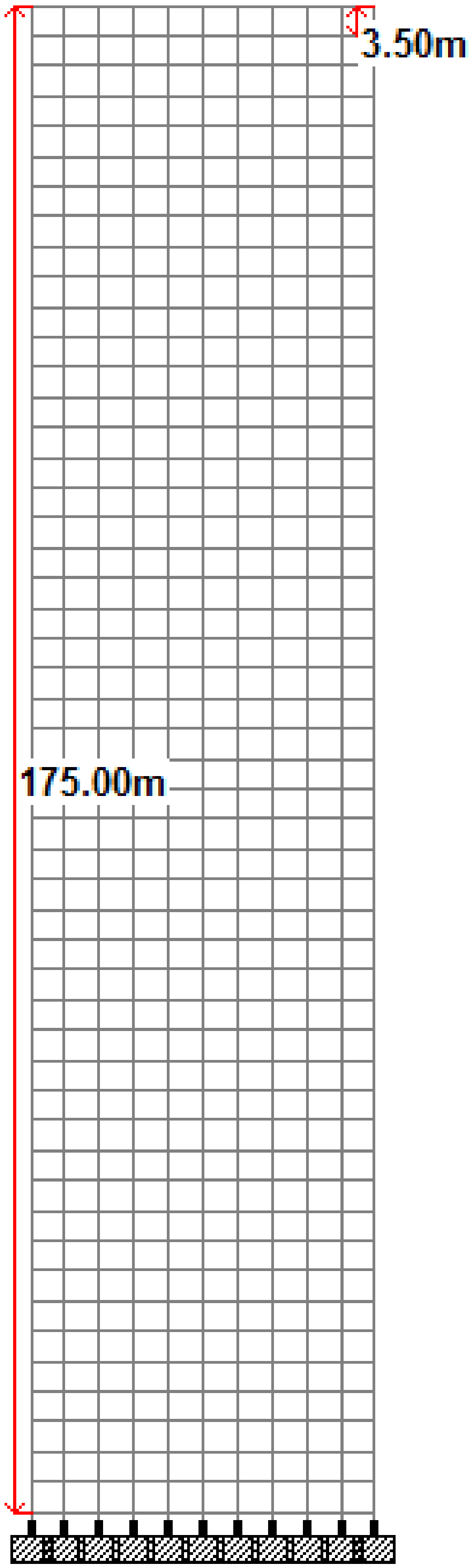

Fig 4.9 Elevation

BUILDING 4

Breadth, b = 80 m
Width, a = 40 m
Height, h = 350 m

As per IS 875 Part 3

- Height to minimum lateral dimension ratio = 8.75 > 5
- Natural frequency of building

  $T = (0.09H)/d^{1/2} = 3.52$

  Natural frequency = 0.28 Hz < 1 Hz

Dynamic analysis is required in this building.

Wind dir.

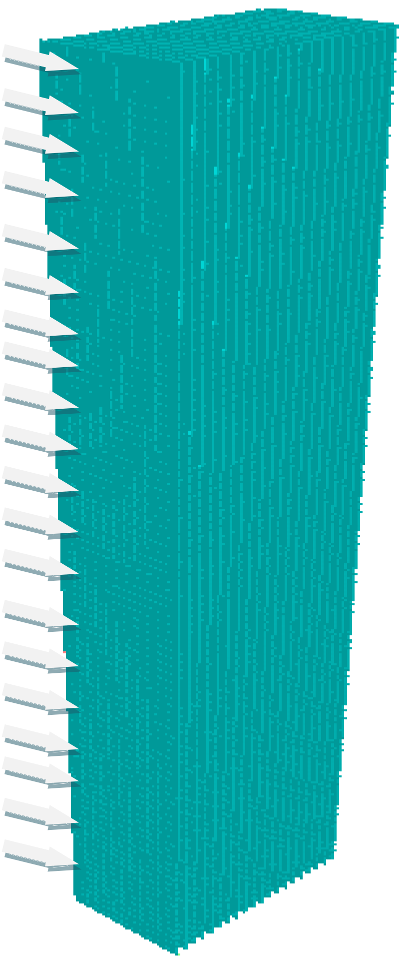

Fig 4.10 3-D model of 100 story building

WIND LOAD CALCULATION			
(100 STORY)			
GIVEN			
DIMENSION OF STRUCTURE			
Breadth, b	80	m	
Width, a	40	m	
Height, h	350	m	
DESIGN LIFE OF STRUCTURE	50	yrs	
TYPE OF STRUCTURE (S)	0.15		clause 7.2
TERRAIN CATERGORY	3		clause 5.3.2.1
WIND SPEED FACTOR (k2)	0.95		table 33
TOPOGRAPHY FACTOR (k3)	1		
STRUCTURE DAMPING COFFICIENT (β)	0.016		
TIME PERIOD (T)	3.52		clause 7.1 NOTE 1 (a)
SWAY FREQUENCY, (f_o)	0.28	Hz	1/T
MODE SHAPE (k)	1		
AVERAGE BUILDING DENSITY	275	kg/m^2	
BASIC WIND SPEED	47	m/sec	APPENDIX A
HOURLY MEAN WIND VELOCITY AT 10 m height, V_{10}	23.50	m/sec	
HOURLY MEAN WIND VELOCITY AT ROOF LEVEL, V_n	44.65	m/sec	
LOAD CALCULATION			
SHEDDING FREQUENCY (η) = SxV_d/b	0.08	< .2	
Factor g/r	0.6		Fig 8
BACKGROUND FACTORS			
Longitudnal correlation constant C_Z	12		Pg 52 of IS 875
Lateral correlation constant Cy	10		Pg 52 of IS 875
L(h)	2200		Fig 8
$C_z xh/L(h)$	1.91		
$\lambda = (C_y xb)/(C_z \text{ x } h)$	0.19		
Background factor, B	0.5		Fig 9
Reduced frequency, $F_o = (C_z x f_o x h)/V_n$	26.71		
Size reduction factor, S	0.048		Fig 10
$f_o L(h)/V_n$	13.99		
Gust energy factor, E	0.1		Fig 11
SE/β	0.30		
Gust factor, $G=1+g_f r(B+(SE/\beta))^{1/2}$	1.54		

Table 4.4 Wind load calculation of 100 story structure

h(m) = Height of building									
d(h)m =Diffrence between height									
z (m) = Avg of height									
k_2 = Table 33 of IS 875									
A_c = Area of wall									
Force (along wind) at height z on strip area Ae = $C_f G A_e p_z$									
Pressure at height z due to $V_z = (V_{10} \times (k_2/0.50))$									
$F_z = C_f G A_e p_z$									
Bending Moment = Force x Area									
S.No	h(m)	dh(m)	z (m)	k_2	V_z (m/sec)	p_z (N/m^2)	A_c (m^2)	Fz (kN)	B.M
1	10	10	5	0.5	23.50	331.35	800	590.64	2953.19
2	15	5	12.5	0.59	27.73	461.37	400	411.20	5140.03
3	20	5	17.5	0.59	27.73	461.37	400	411.20	7196.04
4	30	10	25	0.64	30.08	542.88	800	967.70	24192.55
5	50	20	40	0.7	32.90	649.45	1600	2315.30	92612.11
6	100	50	75	0.79	37.13	827.18	4000	7372.35	552926.18
7	150	50	125	0.84	39.48	935.20	4000	8335.09	1041886.21
8	200	50	175	0.88	41.36	1026.39	4000	9147.81	1600866.43
9	250	50	225	0.91	42.77	1097.56	4000	9782.15	2200984.62
10	300	50	275	0.93	43.71	1146.34	4000	10216.86	2809637.53
11	350	50	325	0.95	44.65	1196.17	4000	10661.02	3464832.75
TOTAL							16000	60211.34	11803227.65
BASE BENDING MOMENT							11803228	kNm	

Table 4.4(cont.) Wind load calculation of 100 story structure

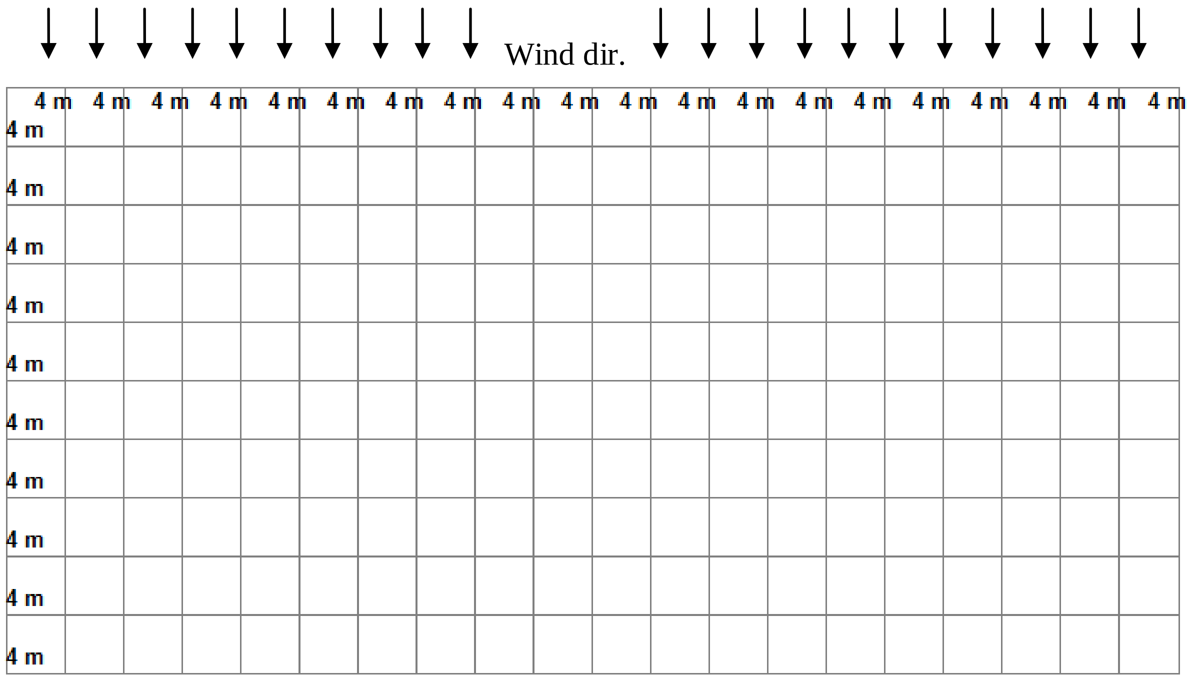

Fig 4.11 Plan

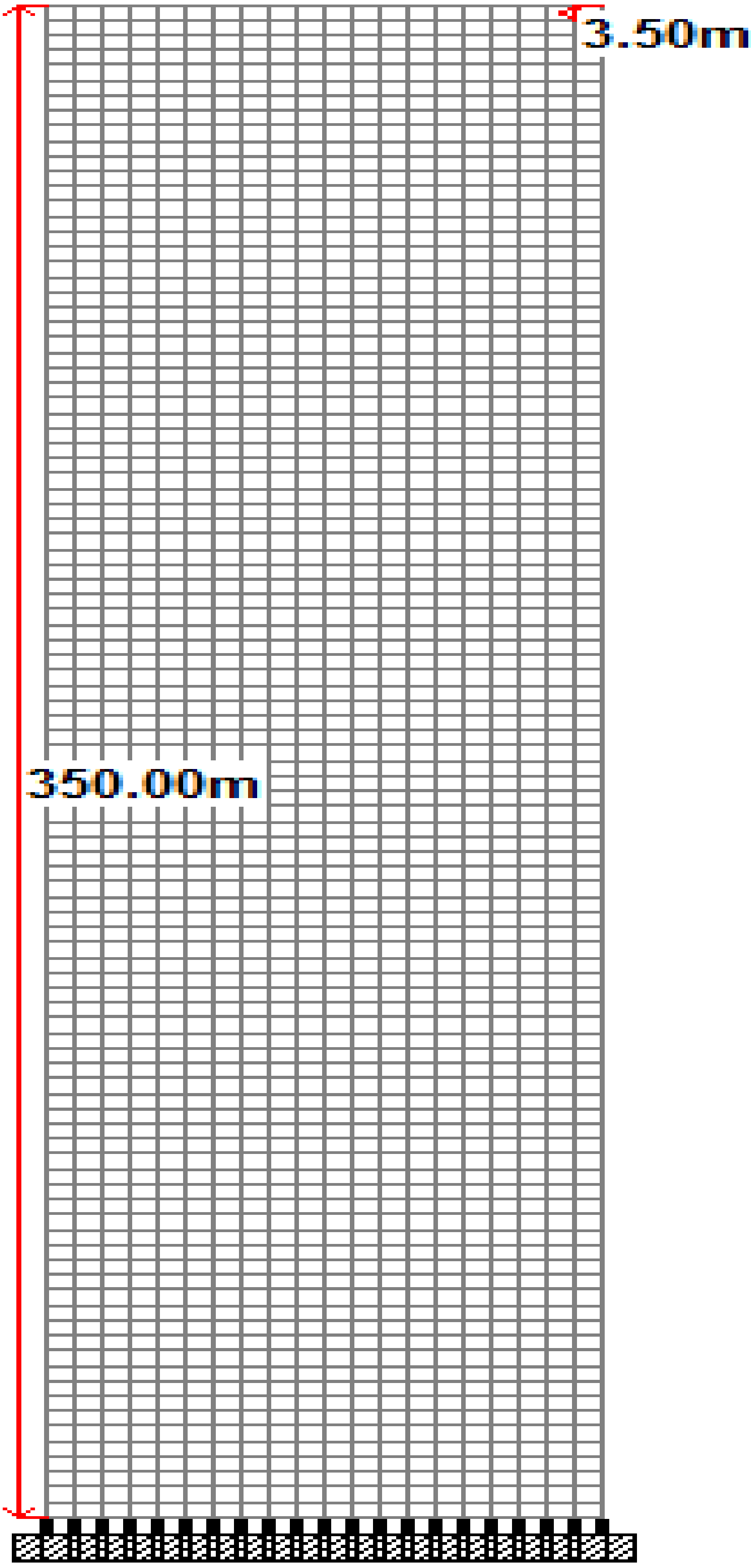

Fig 4.12 Elevation

BUILDING 5

Breadth, b = 33 m
Width, a = 33 m
Height, h = 200 m
Wind speed = 40 m/sec
Building density = 180 kg/m^3

The Gust loading factor (GLF) approach for assessing the dynamic along-wind load as per IS 875 Part 3 is done and the result will be compared with the paper of **Yin Z. et al. (2002).**

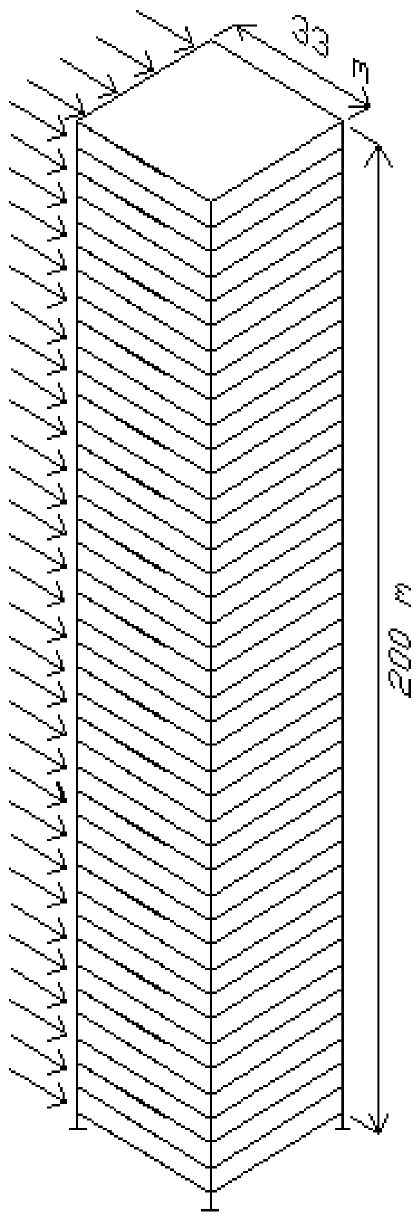

Fig 4.13 3D view of 200 m high building

WIND LOAD CALCULATION

(200 meter high building)

GIVEN

DIMENSION OF STRUCTURE			
Breadth, b	33	m	
Width, a	33	m	
Height, h	200	m	
DESIGN LIFE OF STRUCTURE	50	yrs	
TYPE OF STRUCTURE (S)	0.15		clause 7.2
TERRAIN CATERGORY	3		clause 5.3.2.1
WIND SPEED FACTOR (k2)	0.88		table 33
TOPOGRAPHY FACTOR (k3)	1		
STRUCTURE DAMPING COFFICIENT (β)	0.016		Table 34
TIME PERIOD (T)	3.13		clause 7.1 note 1 (a)
SWAY FREQUENCY, (f_o)	0.32	Hz	1/T
MODE SHAPE (k)	1		
AVERAGE BUILDING DENSITY	180	kg/m^2	
BASIC WIND SPEED	40	m/sec	As per paper
HOURLY MEAN WIND SPEED FACTOR (10m)	0.5		table 33
HOURLY MEAN WIND VELOCITY AT 10 m height, V_{10}	20.00	m/sec	
HOURLY MEAN WIND VELOCITY AT ROOF LEVEL, V_n	35.2	m/sec	

LOAD CALCULATION

SHEDDING FREQUENCY (η) = SxV_d/b	0.16	< .2	
Factor g/r	0.9		Fig 8
BACKGROUND FACTORS			
Longitudnal correlation constant C_Z	12		Pg 52 of IS 875
Lateral correlation constant Cy	10		Pg 52 of IS 875
L(h)	2150		Fig 8
C_z x h/L(h)	1.12		
$\lambda = (C_y \text{ x } b)/(C_z \text{ x } h)$	0.14		
Background factor, B	0.6		Fig 9
Reduced frequency, $F_o = (C_z \text{ x } f_o \text{ x } h)/V_n$	21.76		
Size reduction factor, S	0.08		Fig 10
$f_oL(h)/V_n$	19.49		
Gust energy factor, E	0.09		Fig 11
SE/β	0.08		
Gust factor, $G=1+g_fr(B+(SE/\beta))^{1/2}$	1.74		

Table 4.5 Wind load calculation of 200 m high structure

FORCE COEFFICIENT, C_f			
a/b	1.00		
h/b	6.06	>1	
C_f	1.45		Fig 4A

h(m) = Height of building

d(h)m =Diffrence between height

z (m) = Avg of height

k_2 = Table 33 of IS 875

A_c = Area of wall

Force (along wind) at height z on strip area Ae = $C_f G A_e p_z$

Pressure at height z due to $V_z = (V_{10} \times (k_2/0.50))$

$F_z = C_f G A_e p_z$

Bending Moment = Force x Area

S.No	h(m)	dh(m)	z (m)	k_2	V_z (m/sec)	p_z (N/m^2)	A_c (m^2)	Fz (kN)	B.M
1	10	10	5	0.5	20.00	240.00	330	199.76	998.78
2	15	5	12.5	0.59	23.60	334.18	165	139.07	1738.37
3	20	5	17.5	0.59	23.60	334.18	165	139.07	2433.72
4	30	10	25	0.64	25.60	393.22	330	327.28	8181.99
5	50	20	40	0.7	28.00	470.40	660	783.04	31321.68
6	125	75	87.5	0.815	32.60	637.66	2475	3980.48	348292.02
7	150	25	137.5	0.84	33.60	677.38	825	1409.48	193802.90
8	200	50	175	0.88	35.20	743.42	1650	3093.82	541417.63
TOTAL							6600	10071.99	1128187.10

Table 4.5 (Cont) Wind load calculation of 200 m high structure

CONCLUSION

In this report the idea of calculation of along wind force is presented using IS 875 Part 3 code method.

The results are as follows:

S.No.	TYPEE OF BUILDING	BASE SHEAR (kN)	MOMENT (kNm)
1	5 story	355.86	3197.84
2	10 story	847.63	15883.74
3	50 story	14047.66	1373680.8
4	100 story	60211.34	11803227.65

Table 5.1 Base shear and moment

From the result we can conclude that wind force on a building increases tremendously with height and as the height increases the building has to analyse by dynamic analysis.

The result of 200 m high building is also shown below in the table for different codes for terrain category C.

	ASCE 7	AS1170.2	NBC	RLB-AIJ	Eurocode	IS 875
G.L.F	1.854	2.021	2.544	1.868	2.026	1.74
Moment (kNm)	1539848	1302400	1871300	1556400	1696700	1128187

Table 5.2 Comparison of result of wind load calculation

Here we can see that G.L.F and moment are different for all codes. This is because of difference in wind characterisation (k_2 factor) in different codes.

In future we can further analyse the building using wind tunnel and we can compare it with the above result.

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