2006-2008

**THESIS**
**SUBMITTED TO THAPAR UNIVERSITY, PATIALA TOWARDS PARTIAL FULFILLMENT OF THE DEGREE OF**

# MASTER OF ENGINEERING
# IN
# CIVIL ENGINEERING
## (WITH SPECIALIZATION IN STRUCTURAL ENGINEERING)


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We are pleased to recommend that the Thesis work entitled **"Estimation of Wind Load on Tall Buildings"** is a bonafide work carried out by **Achyut Khajuria**, in partial fulfillment for the award of degree of Master of Engineering (Civil), specializing in **Structural Engineering** from Thapar University, Patiala during the year 2006-2008. The project report is approved and may be accepted as it satisfies the academic requirement, as per syllabus, in respect of project work prescribed for the degree of Master of Engineering (Civil).

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We are pleased to certify that the Thesis work entitled **"Estimation of Wind Load on Tall Buildings"** submitted by **Achyut Khajuria** , as major project in the year 2006-2008 is a satisfactory account of his work based on syllabus and is approved for the award of Degree of **Master of Engineering (Civil)** specialization in **STRUCTURAL ENGINEERING**.

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# Abstract

Any Tall building can vibrate in both the directions of Along wind and Across wind caused by the flow of wind. Modern Tall buildings designed to satisfy lateral drift requirements, still may oscillate excessively during wind storm . These oscillations can cause some threats to the Tall building as buildings with more and more height becomes more vulnerable to oscillate at high speed winds. Sometimes these oscillations may even cause discomfort to the occupants even if it is not in a threatening position for the structural damage. So an accurate assessment of building motion is an essential prerequisite for serviceability . There are few approaches to find out the Response of the Tall buildings to the Wind loads. An Analytical approach given by Davenport and mentioned in the IS 875: part 3 -1987 is used which is only applicable to a regular shape building but for an irregular shaped building and but for its accurate response and behavior under the high wind speeds is provided from a prototype measurements in a wind tunnel. A simulated wind tunnel experiment on an appropriate model of the building yields results which give a deeper insight into the phenomenon and provides more precise information, overcomes the shortcomings of the analytical formulation. In the wind tunnel testing the two methods are used for determining the response of any Tall building with an irregular or regular shape under the high wind speeds, one is the Dynamic Analysis method used to determine the wind loads on the buildings in which Base Forces can be economically obtained from HFFB test while the other one is Pressure measurement studies which are used for the safe designs of Individual structural elements as roofs and walls, and Individual cladding units including glazing and their fixing.

This report deals with the wind tunnel studies carried out on a scaled down model of proposed tall building named Signature towers from Unitech company going to be build in Greater Noida, India which is 150m in height, carried out at the Boundary Layer Wind Tunnel, IIT Roorkee. The experimental results have been projected to estimate the full scale values using appropriate scaling laws. These have also been compared with the prototype responses computed analytically. The analytical values in the along wind direction have been obtained using the Davenport's `Gust Factor Approach'(1967). Pressure Studies are also done on the building for their claddings



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design and to know the pressure distribution on the building faces. The building models have been tested in a boundary layer flow corresponding to terrain category - III, as defined in IS: 875-part-3, 1987, ($\alpha$ = 0.18 as per IS: 4998) at a wind speed of 10.78 m/s at model top for stand-alone and interfering situations.

Forces and moments (mean, maximum, minimum and R.M.S.) in along-wind and across-wind directions at the base of the building and torsional moment about vertical axis and coefficients of pressures (mean, maximum, minimum and R.M.S.) have been obtained for different wind incidence angles ($0^0$ and $360^0$) for stand-alone and interference condition.

The results have been presented in the form of plots and tables and design values have been deduced. Peak accelerations at the top of the building have been obtained on the basis of the measured base moments and compared to those as computed from the formulation given in the IS:875.

Story wise lateral design forces are computed by analytical method and check of Base Forces obtained from Pressure Studies with that obtained from Dynamic Anal ysis is discussed in Conclusion. Comparison of various design parameters is also done and interesting results have been obtained. It is found that the gap between the analytical estimates and wind tunnel test results depends upon the plan aspect ratio and shape of building. In the present case, experimental test results are higher than those of analytical estimates .



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# NOTATIONS AND SYMBOLS

## USED IN CODAL AND DAVENPORT PROCEDURE

| | | |
|---|---|---|
| C | = | A constant in response-reduced velocity relationship |
| $C_D$ | = | Drag (Along wind force) co-efficient |
| $C_L$ | = | Lift (Across-wind force) co-efficient |
| D | = | Dimension of building cross-section |
| f | = | Frequency |
| $g_f$ | = | Peak factor |
| $g_y$ | = | Peak factor for Across-wind response |
| G | = | Gust factor |
| h | = | Height of the building |
| k | = | Terrain drag co-efficient |
| $K_O$ | = | Generalized stiffness of the building in first mode of vibration |
| L | = | A characteristic length scale, in Davenport's gust factor approach |
| $M_O$ | = | Generalized mass of the building in first mode of vibration |
| n | = | A constant in response-reduced velocity relationship |
| $n_o$ | = | frequency of vibration in the fundamental mode |
| $\bar{P}_h$ | = | Mean wind pressure at the top of the building |
| r | = | Roughness factor in Davenport gust factor approach |
| R | = | Resonant response component, in Davenport gust factor approach |
| RV | = | Reduced velocity |
| S | = | Size reduction factor, in Davenport gust factor approach |
| $S_f(f)$ | = | Power spectral density of force |
| $S_u(f)$ | = | Power spectral density of longitudinal velocity fluctuations |

| | | |
|---|---|---|
| $S_y(f)$ | = | Power spectral density of Across-wind displacement |
| T | = | Averaging period for mean wind |
| $\bar{V}$ | = | Mean wind speed |
| $V_h$ | = | Hourly mean wind speed at height 'h' |
| $V_{10}$ | = | Hourly mean wind speed at 10m height |
| $V_Z$ | = | Hourly mean wind speed at height 'Z' from a reference level |
| $V_b$ | = | Regional basic wind speed |
| $\bar{X}$ | = | Mean value of Along-wind displacement |
| $X_{RMS}$ | = | RMS value of Along-wind displacement |
| $X_{peak}$ | = | Peak value of Along-wind displacement |
| $X_{max}$ | = | Maximum value of Along-wind displacement |
| $\bar{Y}$ | = | Mean value of Across-wind displacement |
| $Y_{RMS}$ | = | RMS value of Across-wind displacement |
| $Y_{peak}$ | = | Peak value of Across-wind displacement |
| $Y_{max}$ | = | Maximum value of Across-wind displacement |

## USED IN DYNAMIC ANALYSIS PROCEDURE (By BALENDRA)

| | | |
|---|---|---|
| $\sigma_X^2$ | = | Variance or mean square value of displacement |
| $\sigma_F^*$ | = | RMS value of generalized force |
| $\sigma_x$ | = | RMS displacement in X direction |
| $\ddot{\sigma}_X = \sigma_D$ | = | Tip RMS Acceleration |
| $\bar{F}_Z$ | = | Mean component of the wind load |
| $g_B$ | = | Peak factor of background component |
| $g_D$ | = | Resonant peak factor |
| $\sigma_{D,F}$ | = | RMS Resonant component or Inertial Force |
| $\sigma_{B,F}$ | = | RMS Background force or Non-resonating Force component |
| $\Delta_Z$ | = | Lateral force interval |

| | | |
|---|---|---|
| $\ddot{\sigma}_D(Z)$ | = | RMS Acceleration at any Height z |
| $\bar{U}(Z)$ | = | Mean Wind velocity at height z |
| $\mathrm{P}(Z)$ | = | Mean pressure at height z |
| $\bar{M}$ | = | Mean values of measured overturning Base moments |
| $\sigma_M$ | = | RMS value of measured overturning moment |
| $\rho_a$ | = | Air density |
| $\rho_b$ | = | Building Bulk density |
| $C_{\bar{M}}$ | = | Mean overturning moment coefficient |
| $\alpha$ | = | Power Law coefficient of wind profile |
| $C_{\sigma M}$ | = | RMS Moment coefficient |
| $\mathrm{P}_\sigma(Z)$ | = | RMS pressure at height z |
| $\phi$ | = | Mode shape parameter |
| $Z$ | = | Building elevation (level) |
| $H$ | = | Height of the building |
| $m_0$ | = | Mass per unit height |
| $B$ | = | Width of building perpendicular to wind |
| $D$ | = | Depth of building, along the wind |
| $F_{MAX}(Z)$ | = | Max force at any height z |
| $V_H$ | = | Velocity at building top |
| M* | = | Generalized mass in first mode |
| K* | = | Generalized Stiffness in first mode |
| no | = | Fundamental sway frequency of building |
| $M(t)$ | = | Time varying Base overturning moment |
| $F^*(t)$ | = | Generalized force |
| $S_X$ | = | Power spectrum of the response |
| $\lvert H(n)\rvert$ | = | Mechanical admittance |

| | | |
|---|---|---|
| $S_{F^*}$ | = | Generalized wind force spectrum or power spectrum of the generalized force |
| $S_{Mx}$ | = | Measured moment spectrum or power spectrum of the measured base moment |
| $M_y$ | = | Base overturning moment generated by Fx |
| $M_x$ | = | Base overturning moment generated by $F_y$ |
| $n$ | = | Frequency |
| $\zeta$ | = | Damping Ratio |
| $T$ | = | Time period |

## Pressure Measurement Studies

| | | |
|---|---|---|
| Cp | = | pressure coefficient |
| Cpmin | = | Minimum value of pressure measurement |
| Cpmax | = | Maximum value of pressure measurement |
| Cprms | = | Root Mean Square value of pressure measurement |
| $V_z$ | = | Design wind speed at height z in m/s |
| $V_b$ | = | Basic wind speed |
| K1 | = | Risk coefficient (For the design life of structure) |
| K2 | = | Terrain, height and structural size factor |
| K3 | = | Local topography factor (For local topographic influence.) |

## Greek Symbols

| | | |
|---|---|---|
| $\alpha$ | = | Power law co-efficient of ABL profile |
| $\mu$ | = | Poisson's ratio |
| $\rho_a$ | = | Air density at ambient temperature and pressure (=1.2 kg/m$^3$) |
| $\rho_b$ | = | Building bulk density |
| $\sigma_x$ | = | RMS along-wind displacement at top of building |

| | | |
|---|---|---|
| $\sigma_y$ | = | RMS Across-wind displacement at top of building |
| $\zeta$ | = | **C**ritical damping ratio |
| $\xi_o$ | = | Reduced frequency in Davenport's gust factor approach |

## Subscripts

| | | |
|---|---|---|
| a | = | Air |
| b | = | Building |
| D | = | Drag |
| RMS | = | Root mean square |
| x | = | Along-wind direction |
| y | = | Across-wind direction |

# CHAPTER - 1

# Introduction

## 1.0 General

## 1.1 Importance of Wind Loads on the Tall Buildings

Buildings are defined as structures utilized by the people as shelter for living, working or storage. As now a days there is shortage of land for building more buil dings at a faster growth in both residential and industrial areas the vertical construction is given due importance because of which Tall Buildings are being build on a large scale.

Wind in general has two main effects on the Tall buildings:

- Firstly it exerts forces and moments on the structure and its cladding
- Secondly it distributes the air in and around the building mainly termed as *Wind Pressure*

Sometimes because of unpredictable nature of wind it takes so devastating form during some Wind Storms that it can upset the internal ventilation system when it passes into the building. For these reasons the study of air -flow is becoming integral with the planning a building and its environment.

Wind forces are studied on four main groups of building structures:

i. Tall Buildings
ii. Low Buildings
iii. Equal-Sided Block Buildings
iv. Roofs and Cladding

Almost no investigations are made in the first two categories as the structure failures are rare, even the roofing and the cladding designs are not carefully designed, and localised wind pressures and suctions are receiving more attention. But as Tall buildings are flexible and are susceptible to vibrate at high wind speeds in all the three directions(x, y, z) and even the building codes do not incorporate the expected maximum wind speed for the life of the building and does not consider the high local suctions which cause the first damage. Due to all these facts the Wind Load estimation for Tall Buildings are very much important.



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## 1.2 Codal criteria for the buildings to be examined for Dynamic Effects of Winds [BIS (1987)]

Flexible slender structures and structural elements shall be investigated to ascertain the importance of wind induced oscillations or excitations along and across the direction of wind.

In general, the following guidelines may be used for examining the problems of wind induced oscillations:

a) Buildings and closed structures with a height to minimum lateral dimension ratio of more than about 5.0, and

b) Buildings and closed structures whose natural frequency in the first mode is less than1.0 Hz.

Any building or structure which satisfy either of the above two criteria shall be examined for dynamic effects of wind.

## 1.3 Response Parameters

Wind induced response of a tall building is a function of many parameters. These include the geometric and dynamic characteristic of building as well as the turbulence characteristic of the approach flow. A few analytical approaches are available for the estimation of the wind induced response of the tall buildings in along and across wind direction.

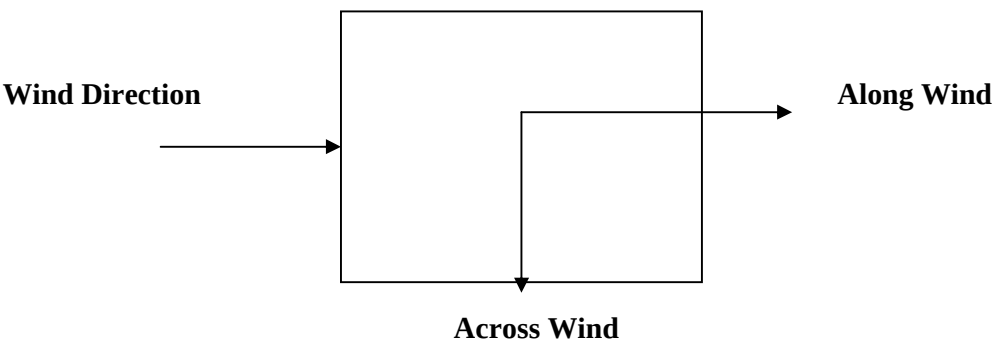


**Fig. 1.1 Along and Across Wind Response**

Under the action of wind flow, structure experience aerodynamic forces that include the drag force and lift force. Drag (along-wind) force acting in the direction of the mean wind and the lift (across-wind) force acting perpendicular to that direction as



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shown in **Fig 1.1**. The Along-wind motion primarily results from pressure fluctuations in the windward and the leeward faces, which generally follow the fluctuations in the approach flow; at least in the low frequency range. The Across-wind motion is introduced by pressure fluctuations due to vortex shedding in the separated shear layers and wake flow field.

## 1.4 Estimation of the Wind load on Tall Buildings

Wind load on a Tall building can be determined by:

1. **Analytical Method** given in the code IS 875: part 3-1987 which is given by A.G.Davenport. The analytical method is usually acceptable for a building with regular shape and size and is almost based on the geometric properties of the building and without incorporating the effects of the nearby buildings.
2. Secondly the Estimation of Wind Load through **Wind tunnel testing** with a scaled building model used. In Wind Tunnel Testing for the structural design the ***Dynamic analysis*** of the scaled model building is done with *Balendra's approach* and for the cladding design the ***Surface Pressure Measurement analysis*** with *Pressure Measurement system* is done. Also the effects of the nearby buildings have been taken into consideration as the Interference effects on the buildings in a same procedure being used for an Isolated building model.

Wind Tunnel testing of an aero elastic model of a building can be used to find out wind loads. Moreover it is very difficult to fabricate an aero elastic model (because of its mass and stiffness distribution), an alternative approach given by Balendra (1996) can be used. As per this procedure a rigid model of the building is mounted on a high sensitive stiff force balance(HFFB), High frequency force balance (HFFB) measurements are utilized to measure the varying fluctuating wind loads on buildings in form of the forces $\mathbf{F_X}$, $\mathbf{F_Y}$ their corresponding moments $\mathbf{M_X}$, $\mathbf{M_Y}$ and the generalized torque $\mathbf{M_Z}$ required for the determination of the Base Forces and Base Moments on the building in the Standalone condition and with the same method but with also incorporating the models of the nearby buildings, the Interference effects are taken in form of the values for Forces and Moments.

In the Surface Pressure Measurement analysis the Rigid Model Studies with pressure tapings or transducers have been used in the Wind Tunnel Testing. A Pressure Measurement system is used which records the values of the Pressure Coefficients on



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their respective taping locations on the building model in the Wind Tunnel. These Pressure distribution i.e., either pressure or suction on the faces of the building required for the cladding designs of the building. The pressure variations on the building models are also taken for both the conditions of Standalone and Interference effects.

## 1.5 Objectives

Following are the main objectives of the work:

1. Wind tunnel testing of the scaled building model with High Frequency Force Balance Technique to find out the Dynamic behaviour of the building in response with the wind to find out the Base Forces, and Base moments even elaborating with storey wise lateral forces on the building.
2. Wind Tunnel Testing to study the fluctuating Pressures over the building required for the designing of structural and cladding design of the building
3. Wind tunnel testing to study the Interference effects on the building of the nearby buildings of different shapes and sizes for both Dynamic Analysis with Surface Pressure Measurement Analysis.
4. To find out Analytical Response of the building as per given in IS 875: part 3-1987 by A.G.Davenport .
5. Comparison of the Results of the Wind Tunnel Testing ( ***Dynamic analysis*** of the building by *Balendra's approach*) and Analytical Method for the building are discussed.

## 1.6 Scope of the Work

The scope of the present work includes the study of the Wind load estimation on Tall buildings for the structural design purpose with the analytical approach given by actor Approach in IS 875: part 3-1987 and the Dynamic Analysis using Wind Tunnel Testing and for the cladding design by Surface Pressure Measurements.

In the Measurements, the Wind Tunnel Testing has been done in which experimental work is done for the both conditions of Standalone and Interference conditions for a scaled building model with High frequency force balance (HFFB) technique and Pressure



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Measurements System respectively. In the Standalone condition the building model is kept alone in the Flow conditions in the Wind Tunnel whereas in the Interference condition the model of the surrounding buildings which effects during the wind flow are also taken in account to know the behaviour of the wind. This testing is performed through the following steps:

1. Establishing the desired flow conditions using various augmentation devices and thus controlling the velocity variations at the model top height.
2. Experimental determination of the forces and moments over the building High frequency force balance (HFFB) technique and thereafter obtaining the required Storey wise lateral forces and Base forces with Base moments for both standalone and interference conditions.
3. Experimentally determining the Pressure Fluctuations on the building with Pressure Measurements System for both standalone and interference conditions.

### 1.6.1 The Prototype and Model Used for the Study

The building studied is named as SIGNATURE TOWERS going to be build at Greater Noida, India by UNITECH Company. The prototype is considered to be situated in an open terrain with well obstructions, defined in terrain category 3 in IS 875: part3-1987 [Indian Standard Code of Practice for Design Loads(other than Earthquake) for Buildings and Structures part 3 – Wind Loads]. In the first phase of [illegible] and Surface Pressure Measurements studies are done on the Building as Isolated i.e., Standalone condition whereas in the second phase the Interference Effect of the nearby buildings is considered for both the Analysis. In the Dynamic analysis the Story wise Lateral forces on the Wing I. Wing II, Wing III and Central Tower of the buildings are obtained with the required flow conditions in the Wind Tunnel and then in the second phase the Pressure coefficients on all the faces of the Wings and central tower are obtained. In this way the Dynamic and Pressure responses on the Signature towers are studied.

## 1.7 Organization of the thesis

The thesis has been divided into 6 Chapters. Chapter-2 presents an account of state-of-the-art in this field and a brief description of related works in the field, i.e.,



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historical paper, wind tunnel test, full scale measurements for Dynamic and Pressure measurement studies, and analytical estimate (along wind response). Chapter-3 includes details of wind tunnel testing, types, models, and instrumentation and the methodology behind the analysis. In Chapter-4 Dynamic Analysis done with [illegible] aero elastic scaled model for a particular Building named Signature Towers is discussed in which the model is mounted on a high sensitive stiff force balance (HFFB), High frequency force balance (HFFB) measurements are utilized to measure the varying fluctuating wind loads on buildings in form of the forces $\mathbf{F_X}$, $\mathbf{F_Y}$ their corresponding moments $\mathbf{M_X}$, $\mathbf{M_Y}$ and the generalized torque $\mathbf{M_Z}$ required for the determination of the Base Forces and Base Moments on the Signature Towers for both Isolated and Interference effects on building model. Chapter-5 presents the Surface Pressure Measurements done on the same Building model with their experimental results obtained form Wind tunnel testing for both Isolated and Interference effects on building model. Conclusions of the study are given in Chapter-6. A list of references, directly referred in the work, is alphabetically arranged after Chapter-6.



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# CHAPTER - 2

# Literature Review

## 2.0 GENERAL

This chapter presents a review of relevant literature to bring out the background of the study undertaken in this dissertation. 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

## 2.1 Historical Works

Between 1931 and 1936, when the Empire State Building was constructed, J.Rathbun made full-scale measurements on it **[Rathbun (1940)].** Earlier in 1933, Dryden & Hill made measurements on a five-foot scaled model of the Empire State Building.

The wind sensitivity of buildings and structures depends on several factors, the most important of which are the meteorological properties of the wind, type of exposure, and the aerodynamic and mechanical characteristics of the structure, An inventory of those various factors is presented, including indications of their relative influence on the global response **[A.G.Davenport (1998)].**

Isyumov overviews the action of wind on tall buildings and structures with emphasis on the overall wind-induced structural loads and responses. Also discussed the local wind pressures on components of the exterior envelope and the effects of buildings on winds in pedestrian areas. These may include buildings and structures of unusual shape, those located in complex settings or those with dynamic systems which amplify the time varying wind forces and whose motions may in turn alter the force field. On the other hand, there is a growing population of buildings and structures for which the action of wind is well understood and can be predicted, with aerodynamic data drawn from past



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experience and from building codes. While we may not have all the answers, the wind engineering community has reached a stage of development which permits candidates for special attention to be identified **[N. Isyumov (1999)].**

Davenport presents an overview of the progress wind engineering has made over the past four decades, since the establishment of the wind engineering conferences, the paper offers an opinion regarding gaps that need to be fulfilled. In response to the collapse of the Tacoma Narrows Bridge and the near failure of several other suspension bridges in the U.S. the plans for the Honshu-Shikoku bridges (presented at the conference by Professor Hirai of Tokyo University and the Mayor of Kobe (Dr. Hiraguchi)] as well as the plans for bridges in the U.K., there was a well articulated discussion on the dynamics of suspension bridges - mostly focusing on dynamic section models in smooth flow, There were papers on the galloping instability of prisms by Parkinson (1963) and of transmission lines by Richards, The treatment of gust pressures was discussed by Harris (1963) and Davenport and these were corroborated qualitatively by Newberry (1963) from full scale measurements of pressures on the façade of a tall building **[A.G. Davenport(1999)].**

Ahsan pays tribute to the "father of wind engineering," Jack E. Cermak, for his many valuable and pioneering contributions to the subject, followed by a reflection on the recent developments in wind effects on structures and an outlook for the future. This discussion encompasses the following topics: modeling of wind field; structural aerodynamics; computational methods; dynamics of long-period structures; model- to full-scale monitoring; codes/standards and design tools; damping and motion control devices **[Ahsan Kareem(2003)].**

Holmes discusses the progress made in understanding wind loads on structures, and related aspects of wind engineering, emerging issues in 2003, and prospects for the next forty years.

Although the name 'wind engineering' was coined in the nineteen-seventies, resulting in the International Conference on 'Wind Effects on Buildings and Structures' becoming the International Conference on 'Wind Engineering' in 1979, the foundations of modern wind engineering were firmly set in the early nineteen-sixties. Several papers in the I$^{st}$ International Conference on Wind Effects on Buildings and Structures at Teddington, U.K. in 1963 set the scene for the next forty years **[Holmes J. (2003)].**

**Ning Lina, Chris Letchforda, Yukio Tamurab, Bo Liangc and Osamu Nakamurad** studied nine models with different rectangular cross-sections and were tested in a wind tunnel to study the characteristics of wind forces on tall buildings. The data was briefly



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reported (Local wind forces acting on rectangular prisms. Proceedings of 14th N ational Symposium on Wind Engineering, 4 6 December 1996, Japan Association for Wind Engineering, Tokyo, pp. 263 268.). In the present paper, local wind forces on tall buildings are investigated in terms of mean and RMS force coefficients, power spectral density, and spanwise correlation and coherence. The effects of three parameters, elevation, aspect ratio, and side ratio, on bluff -body flow and thereby on the local wind forces are discussed. The overall loads and base moments are obtained by integration of local wind forces. Comparisons are made with results obtained from high -frequency force balances in two wind tunnels.

**Holmes and Lewis (1986, 1987 and 1989)** 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 mod els of the buildings in wind tunnels. In the present study the choice of tubing system for pressure measurements is largely based on the work of Holmes and Lewis (1987).

### 2.1.1 Dynamic Analysis of Wind Force

**Whitbread (1963)** has presented an account of various flow parameters required to be

satisfactory answers using floor roughening devices .

**Davenport & Isyumov (1967)** have discussed various available techniques to simulat e the ABL in the long test section wind tunnels. They have emphasized that for correct modelling of flow complete turbulence characteristics including velocity profile, turbulence intensity profile, length scales and energy spectrum should be made available for natural wind. Flow characteristics in the new boundary layer wind tunnel at the University of Western Ontario are presented. `Power law' variation of velocity profile is used. Counihan (1969) evaluated the use of a system of `elliptic wedge' generators and a castellated barrier to produce a simulated rough wall boundary layer. Good agreement between the boundary layer flow so produced and neutral atmospheric boundary layer is obtained.

**Fujimoto et al. (1975)** have tested a 1:400 scaled aero elastic mod el of rectangular tall building (1:1.2:3.75) in smooth flow and two boundary layer flows. Values of along wind and across wind response are presented versus reduced velocity and a relationship is established. Experimental gu st factors are compared with Davenport (1967). A four



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mass model was also tested in natural wind, and contribution of higher modes is reported to be negligible on displacements and about 10% on accelerations .

**Cermak (1977)** states that: "A common procedure is to mount the model on a set of gimbals fixed to a rigid platform placed beneath the wind tunnel floor. Two pairs of mutually perpendicular helical springs attached to a rod rigidly fixed to the structural shell and passing below the gimbals provide the desired natural frequencies. Strain-gauges attached to the spring mounts can be used to give a voltage output proportional to sway amplitude. Adjustable magnetic damping is provided conveniently by attaching to the support rod a metal plate that passes between the poles of an electromagnet. Variation of current through the magnet permits control of critical damping ratio". A very simple and useful alternative system designed by Kareem & Cermak (1975) may be constructed by clamping the building base to two leaf springs placed perpendicular to each other and fixed to a rigid frame mounted in the wind tunnel floor. Strain -gauges

by pneumatic dampers attached to a rod extending beneath the wind tunnel floor.

**Parera (1978)** studied the interaction between along wind and across wind vibrations of tall slender structures (1:1:6.3) using one degree-of-freedom and two degree-of-freedom aero elastic models. A new gimbal system to allow either one d.o.f. or two d.o.f. is also developed.

Cermak contributed significantly towards the laboratory simulation of ABL, between 1960 and 1990. His works [Cermak (1971, 1972, 1975, 1976, 1977, 1979,1981,1982,1984, 1987 and 1990)] have treated various aspects of ABL characteristics and simulation in detail. Wind tunnel design criteria have been established. Mathematical similarity criterion has been discussed and governing equations have been formulated. Uses of short test section wind tunnels with vortex generators and grids have been outlined. Closed-circuit meteorological wind tunnels have been designed with flexible ceilings and temperature control facility.

A new wind tunnel testing technique has been developed which makes use of integrated local pressures, measured by a Synchronous Pressure Acquisition Network (SPAN), to determine overall wind-induced response. The integrated pressure modal load or IPML technique has the potential of addressing all of the limitations of the conventional high-frequency force-balance technique while still maintaining the same advantages that that technique has over the aero elastic modelling. Outlines the approach and presents several experimental results including comparisons with data from matched high-frequency force-balance tests **[Steckley et al. (1992)].**



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**Katagiri et al. (1995)** have described a new type of multi degree-of-freedom aero-elastic model. Experimental results of m.d.o.f model are compared with dynamic force balance tests and two degree-of-freedom aero elastic model tests and a good agreement is seen.

**Nakayama et al. (1995) and Cooper et al. (1995)** presented their study on a super tall building with tapered cross-section. In first part the study is aimed at comparing the various wind tunnel modelling techniques. In the second part results of unsteady aerodynamic forces measured using manifold pressure taps at nine levels are presented. Also effects of edge configurations and tapering are studied.

**T. Kijewskit and A. Kareem (1998)** has given an evaluation and comparison of seven of the world's major building codes and standards is conducted in this study, with specific discussion of their estimations of the along wind, across wind and, torsional response, where applicable, for a given building. The codes and standards highlighted this study are those of the United States, Japan, Australia, the United Kingdom, Canada, China and Europe. In addition, the response predicted by using the measured power spectra of the along wind, across wind and torsional responses for several building shapes tested in a wind tunnel are presented and a comparison between the response predicted by wind tunnel data and that estimated by some of the standards is conducted. This study serves not only as a comparison of the response estimates by international codes and standards, but also introduces a new set of wind tunnel data for validation of wind tunnel-based empirical expressions.

**Holme J. et al (2003)** discusses mode shape corrections and reviews processing methodologies for the determination of the overall wind loading and response of tall buildings using the high-frequency base balance technique. It is concluded that mode shape correction factors currently used for twist modes, are conservative. The effect of cross-correlations between base moments is found to be significant when calculating the response for coupled modes.

In the present paper, **[Lina N. et al. (2005)]** local wind forces on tall buildings are investigated in terms of mean and RMS force coefficients, power spectral density, and span wise correlation and coherence. The effects of three parameters, elevation" aspect ratio, and side ratio, on bluff body flow and thereby on the local wind forces are discussed. The overall loads and base moments are obtained by integration of local wind forces. Comparisons are made with results obtained from high-frequency force balances in two wind tunnels.



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Simulation of atmospheric boundary layer inside the test section of the open type wind tunnel at the Department of Civil Engineering, Ruh r University, and Bochum, Germany is attempted. Trapezoidal spires or castellated tripping fence are used for horizontal vortex generation, while the elliptical shark fins are used for vertical vortex generation. Square grids are also used to increase the level of turbulence in flow. Velocity data are obtained in two directions using two cross-wire hot wire probes of Dantec Dynamics make at different heights from the test section floor. The mean wind speed, rms wind speed and integral length scale are obtai ned at different heights. These values are compared with corresponding field data obtainable from Engineering Science Data Unit (ESDU) assuming different geometric scales by **[Mitra D. & Kasperski M. (2006)].** The analysis shows that with a geometric scale r atio of 1:200 to 1:150, the simulated boundary layer can be considered as the simulation of open country boundary layer up to a level of 30 to 35 meters in full scale .

Unusual structural shapes arising out of daring architectural forms need wind tunnel studies to assess the wind forces on such structures. Paper presents the results of a wind tunnel model testing of a 60 m high war memorial at Jammu. The test results are particularly useful in the design of the shield and its attachments with the tower **[Gairola A. et al. (2006)].**

### 2.1.2 Pressure Measurement System

**K.M. Lam, a, , M.Y. H. Leunga and J.G. Zhaoa** studied the Interference effects on wind loading of a row of closely spaced tall buildings . Interference effects on a row of square-plan tall buildings arranged in close proximity are investigated with boundary layer wind tunnel experiments. Wind forces and moments on each building in the row are measured with the base balance under different wind incidence angles and different separation distances betw een buildings. As a result of sheltering, inner buildings inside the row are found to experience much reduced wind load components acting along direction of the row (x) at most wind angles, as compared to the isolated building situation. However, these loa d components may exhibit phenomena of upwind-acting force and even negative drag force. Increase in x-direction wind loads is observed on the upwind edge building when wind blows at an oblique angle to the row. Other interference effects on y-direction wind loads and torsion are described. Pressure measurements on building walls and numerical computation of wind flow are carried out at some flow cases to explore the interference mechanisms. At wind angle around 30° to the row, wind is visualized to flow thr ough the narrow building gaps at high



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speeds, resulting in highly negative pressure on associated building walls. This negative pressure and the single-wake behavior of flow over the row of buildings provide explanations for the observed interference effects. Interference on fluctuating wind loads is also investigated. Across-wind load fluctuations are much smaller than the isolated building case with the disappearance of vortex shedding peak in the load spectra. Buildings in a row thus do not exhibit resonant across-wind response at reduced velocities around 10 as an isolated square-plan tall building.

**Peter A. Irwin** studied the procedure for determining wind pressures on the exterior cladding of tall buildings. The methods used in a pressure model study are reviewed including measurement system frequency response, the determination of peak pressure coefficients, combining wind tunnel and meteorological data and evaluating internal pressures. In addition, an assessment is made of the uncertainties involved in wind tunnel testing as compared with using building code methods.

**H. Ueda, K. Hibi, Y. Tamura and K. Fujii** studied a multi-channel simultaneous fluctuating pressure measurement system using electronically scanning pressure transducers has been developed for wind tunnel tests in the wind engineering field. The fluctuating wind pressures acting on tall building models are measured simultaneously using the multi-channel pressure measurement system, and the fluctuating wind pressures are integrated into the resultant fluctuating wind forces. The spectra of the wind forces agree well with the results obtained by a force-balance system. The pressure measurement system has also been used to investigate the properties of the fluctuating wind forces acting on the beams supporting flat roofs, and the gust factors of the load effects on the beams are examined.

**Ning Lina, Chris Letchforda, Yukio Tamurab, Bo Liangc and Osamu Nakamurad** studied nine models with different rectangular cross-sections and were tested in a wind tunnel to study the characteristics of wind forces on tall buildings. The data was briefly reported (Local wind forces acting on rectangular prisms. Proceedings of 14th National Symposium on Wind Engineering, 4–6 December 1996, Japan Association for Wind Engineering, Tokyo, pp. 263–268.). In the present paper, local wind forces on tall buildings are investigated in terms of mean and RMS force coefficients, power spectral density, and spanwise correlation and coherence. The effects of three parameters, elevation, aspect ratio, and side ratio, on bluff-body flow and thereby on the local wind forces are discussed. The overall loads and base moments are obtained by integration of local wind forces. Comparisons are made with results obtained from high-frequency force balances in two wind tunnels.



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**Holmes and Lewis (1986, 1987 and 1989)** performed extensive experimental work on the fluctuating pressure measurements using a small diameter 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. In the present study the choice of tubing system for pressure measurements is largely based on the work of Holmes and Lewis (1987).

### 2.1.3 Full Scale Measurements

The study by **[T Kijewski & A. Kareem (1999)]** addresses one of the issues facing modern skyscrapers by examining the dynamic characteristics of a 244 meter tall building through the analysis of measured full scale data. The building under consideration was studied for a period of five years, during which time; it experienced numerous severe wind events. The acceleration and pressure data collected during this period provides an excellent opportunity to study the response of tall buildings under the action of wind in an urban environment and extract the dynamic characteristics, particularly, the inherent damping, over various levels of excitation through a host techniques including the random decrement technique and AR spectral estimations for random data.

The design of tall building is often influenced by wind-induced vibration such as accelerations in the matter of occupants comport. Consequently, vibration periods and damping becomes important parameter in the determination of such motions. This paper is concerned with the natural periods and damping ratios of steel buildings. It describes the vibration measurement methods employed for testing buildings and presents reliable methods of assessing natural period and damping from ambient vibration tests. This paper describes the findings from full-scale measurement of micro-tremor vibration of 21 typical high-rise buildings in Korea. Regression formulas of natural periods and damping ratios for steel-framed tall buildings are suggested. Finally, obtained natural periods arc compared with empirical expressions of structural standards and eigenvalue analysis **[Yoon S.W (2003)].**

The design of high rise building is often influenced by wind- induced motions such as

dynamic (vibration periods and damping) properties become important parameters in the determination of such motions. The approximate method and empirical expressions used to quantify these parameters at the design phase tend to yield values significantly



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different from each other. In view of this it is a need to examine how actual buildings in the field respond to dynamic wind loading in order to ascertain a more realistic model for the dynamic behavior of buildings. This paper describes the findings from full scale measurements of the wind induced response of typical high rise buildings in Singapore, and recommends an empirical forecast model for periods of vibration of typical building in Singapore, an appropriate computer model for determining the periods of vibration, and appropriate expression which relate the wind speed to accelerations in buildings based on wind tunnel force balance model test and field result **[Balendra T. et al. (2003)]**.

## 2.2 Analytical Work

### 2.2.1 Analytical Response

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 (1961a)** 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 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.

**Harris (1963)** gives the limitations of existing methods of assessing wind loadings are discussed, and the results of communication theory needed for the new statistical method of structural design and then introduced, together with a discussion of the



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methodological results upon which the application of the new method depends. The pressure / velocity relationship which is used at present is treated by an exact method, and the results show that approximations previously used are adequate in most cases. The need for an improved pressure / velocity relationship is briefly discussed, and some experiments at present being under taken. The application of statistical methods to multi-degree of freedom system is then introduced, and some experiments to find out the basic nature of wind structure are described. Finally, the need for adequate methods for the solution of non-linear problems is pointed out.

**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 (1961a&b)]. 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 building 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 and



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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 o f 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.

**Chiu (1970)** state that wind velocities fluctuate randomly during a storm a nd therefore are not amenable to simple mathematical formulation of time varying Wind forces for use in dynamic analysis. The turbulent wind shows variations of velocities both vertically and laterally. It also shows random direction fluctuations. Because of these complexities, equivalent static wind forces usually have been assumed by engineers for structural analysis. This assumption may suffice for most structures but may not be adequate when analyzing tall slender structures which could be dynamically s ensitive to fluctuating wind forces. For such structures, dynamic stresses may be much more critical than static stresses.

On the basis of his analysis and experiments, Vickery develop ed 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 correlat ion 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 experi - mental results available in literature. Some of these assumptions are analyzed with a view to further improving the accuracy of the gust factor by correctly modeling 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



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dynamic part of response and the gust response factor are reduced considerably. Later he showed **[Simiu (1974b)]** 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 underes timates the resonant amplification effects. Simiu (1973a, 1974a&b, 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 [Simiu and Lozier (1975)] which was further mo dified 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 presen ted, 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



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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.

**Takeno et al. (1975)** studied the effect of wind velocity fluctuations on a simple elastic structure consisting of a concentrated mass. The wind induced response of a continuous structure is due mainly to drag and lift forces oriented in the direction parallel and normal to the wind flow, respectively. Due to the spatial variation of mean and fluctuating wind velocities, these forces are function of time and space. The lift force is produced by alternating oscillation of vortices, while the main contribution to the drag force comes from the wake formed on the leeward side of the structure. At critical wind velocities there is also a possibility of self-excited oscillation known as galloping.

**Yang & Lin (1981)** have used a transfer matrix approach for analyzing the wind induced vibrations of a multi-storey building.

Contributions of **Yang et al. (1981), Kareem (1986), Islam et al. (1990) and Kareem (1992)** towards the estimation of dynamic response of tall rectangular buildings using random vibration theory/transfer matrix formulation, and pressure measurements on faces and evaluating the covariance integration have provided alternate analytical solutions of the problem.

**Isyumov (1982)** has discussed the use of `direct' aero-elastic simulations for the study of dynamic behavior of prototypes. A review of the aero-elastic model requirements is presented. Isyumov has described a `stick' type two degree-of-freedom model and also multi degree-of-freedom aero elastic models.

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 behavior 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.

**Reinhold (1983)** describes use of aero elastic and elastic models for the study of wind effects. A new technique using numerous pressure transducers to directly measure the fluctuating wind loads is presented. He has suggested the use of pressure transducers with aero elastic models



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**Solari G. (1985)** presents a mathematical model according to which wind is described as a stochastic stationary Gaussian process made up of a mean speed profile and an equivalent turbulent fluctuation perfectly cross-correlated in space. The high precision of the result is obtainable when estimating the dynamic along wind response of structures by means of this method, and the wide range of applications that this procedure allows; render it particularly suitable for practical engineering calculations and standards applications.

**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 modeling 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 modeled 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 te[illegible] general. The other topics in the response of [illegible] in this paper any structure not included in the list of other topics will be considered as an honorary building.

**Blackmore (1985)** compared the response obtained through various experimental methods. He has used synthesized response (obtained via loading spectra on a rigid model, measured by both force balance and pneumatic averaging technique) and direct modal response using both linear mode and multi degree-of-freedom dynamic models. He recommended use of a rigid model on a force balance and obtaining the force spectra by using a suitable mechanical admittance function. For higher reduced



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velocities motion of the structure itself affects the response and for this an aero elastic model is needed.

**Solari G. (1987)** formulates a theoretical consistent definition of "wind response spectrum" according to which the structural behavior to wind gusts can be evaluated, with a high level of precision and simplicity, by both an approximate dynamic analysis and an equivalent static approach. The method herein presented is based upon the "equivalent wind spectrum technique", 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.

**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 method 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.

**Kwok (1988) and Kwok et al. (1988)** conducted model tests on an aero elastic model of CAARC standard tall building and studied the effects of aerodynamic modifications to the building cross-section. Horizontal slots, slotted corners and chamfered corners used in the study, are found to reduce the along wind and across wind responses.

**Balendra et al. (1988)** investigate the along-wind response of a slender vertical structure in a turbulent atmospheric boundary layer by using a liberalized time domain technique. The methodology employs the classical flexural beam theory. The atmospheric wind turbulence is modeled by using co sinusoidal functions with closely spaced frequencies and with random phase angles uniformly distributed between 0 and 2n, the amplitudes of co sinusoidal functions are determined by using the power spectrum of the fluctuating wind velocity. The proposed model predicts the peak responses by using the predetermined quasi-static drag coefficients of a given geometric shape. The predicted peak responses are found to compare reasonably well with the



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published experimental results for a square building and a rectangular one. By referring to the proposed model, the influence of damping and shape of the building on along wind response is discussed.

**Solari G. (1989)** formulates a theoretically consistent definition of the wind response spectrum based upon the equivalent wind spectrum technique, a calcu lation procedure by means of which wind is schem atized 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 behavior 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 form al 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.

The along wind response of a slender vertical structure in a turbulent atmospheric boundary layer is investigated by **Balendra et al. (1989)**, using a linearised time domain technique. The methodology employs the classical flexural beam theory. The atmospheric wind turbulence is modelled by using co sinusoidal function with closely spaced frequencies and with random phase angles uniformly dist ributed between 0 and 2ã. The amplitudes of co sinusoidal functions are determined by using the power spectrum of the fluctuating wind velocity. The proposed model predicts the peak responses by using the predetermined quasi-static drag coefficient of a gi ven geometric shape.

Effects of orientation of principal axis of stiffness on the dynamic response of slender square building model have been reported by **Isyumov et al. (1990)**. 1:1:5 & 1:1:10 proportioned `stick' type aero elastic models have been used. Or ienting the square building's principal stiffness axis along the diagonals helps in reducing the response. Effect of frequency separation in two directions is also discussed.

**Cheong H.F. et al. (1992)** presented an experimental technique to determine the distribution of wind loads along the height of a slender and tall building using an aero elastic model which simulates the correct mode shape of the prototype. The dynamic pressures acting on the model have been measured simultaneously from two pressure



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tappings at a time to compute the auto and cross power spectral densities, from which the modal force and, hence, the acceleration at any height is computed to determine the distribution of dynamic shear and moment. The technique could also be imple mented using a rigid model when the motion of the building is not expected to modify the pressure distribution significantly. Since simultaneous measurements of pressures from all the tapings are not required, the proposed technique can be easily implemented in any wind tunnel laboratory.

This study discusses the application of a numerical simulation technique to estimates wind-induced vibrations of tall buildings. We simulated fluctuating wind forces acting on a tall building in along-wind and across-wind directions using the technique which was developed by **Tamura et al. (1988)**. The power spectrum of the fluctuating wind force in along wind direction is expressed as the product of the power spectrum of the fluctuating wind speed and the aerodynamic admittance. While in the case of the across-wind direction the power spectrum of the lift force is approximated by a mathematical expression based on wind tunnel data. These techniques are proceeded to use for analy - ses of the along-wind and across wind vibrations of tall buildings. Estimating displacement and acceleration. Further analyses of the response of a building installed with a Tuned Mass Damper (TMD) are also carried out to examine its efficiency response analyses on along-wind and across-wind vibrations of tall buildings in time domain.

**Davenport (1993b)** has described some powerful tools which enable the wind loading to be defined in general terms and where necessary simplified. Three kinds of shape functions can be of paramount importance: the influence functions relating the responses to the load distribution: the mode shapes describing the distribution of inertia loading and finally: the description of the loading pattern.

**D.Y.N. Yip, R.G.J. Flay (1995)** state the theory currently used to predict the wind-induced response of buildings from force balance measurements is briefly reviewed. For buildings with coupled 3-D mode shapes, sources of uncertainty in the technique's response predictions are not just from errors in the mode shape corrections, but also from limitations in not being able to allow for coupled terms and higher mode effects. A new force balance data analysis technique which is designed to overcome these limitations is described. The new method eliminates the need to guess mode shape correction factors for buildings with non-linear sway and non-uniform torsional mode shapes. The reliability and accuracy of this method has been examined and validated analytically and experimentally to date. Some results of the analytical studies are



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presented here which demonstrate the power of the new method. It is believed that the new method will become the standard force balance data analysis method of tomorrow.

**J.Xie, P.A. Irwin & M.Accardo (1999)** state, for a building design there are usually three wind load components to consider: two orthogonal horizontal loads; and one torsional load. As each load component generally does not reach its maximum value at the same instant as the other components, nor even for the same wind direction, it is important to consider how these predicted peak load components should be combined for structural design.

Based on the consideration of the cross-correlations of the various load components and the practical range of structural influence factors, an approach to determining an optimized set of linear load combination factors is given. The proposed set of load combinations possesses the property that, for any given structural member, there will be at least one of the combinations which causes the 50 year load effect (e.g., stress) to be reached or slightly exceeded. With this set of load combinations, the predicted wind loads from wind tunnel tests and the follow-up analysis can be presented in the form of equivalent static wind loads for a particular return period, typically 50 years, somewhat similar to the format of a building code. An example of use of the proposed load combination approach is given.

**Yin Z. et al. (2002)** says most international codes and standards provide guidelines and procedures for assessing the along-wind effects on tall structures. Despite their common [illegible] approach, sizeable scatter exists among the wind effects predicted by the various codes and standards under similar flow conditions. This paper presents a comprehensive assessment of the source of this scatter through a comparison of the along-wind loads and their effects on tall buildings recommended by major international codes and standards. ASCE 7-98 (United States), AS1170.2-89 (Australia), NBC-1995 (Canada), RLB-AIJ-1993 (Japan), and Eurocode-1993 (Europe), are examined in this study. The comparisons consider the definition of wind characteristics, mean wind loads, GLF, equivalent static wind loads, and attendant wind load effects. It is noted that the scatter in the predicted wind loads and their effects arises primarily from the variations in the definition of wind field characteristics in the respective codes and standards. A detailed example is presented to illustrate the overall comparison and to highlight the main findings of this paper.

**Mundhada (2002)** analyzed a three bay symmetric Portal for vertical loading alone and then for wind load also. The building was assumed to be on stilts. Total numbers of 9 cases were considered, starting from G+2 and ending with G+10. K [illegible]



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contribution method was used for vertical loading whereas the modified moment distribution method was used for lateral loading. A basic wind pressure of 1 KN/m² was assumed. To being with, member stiffness was maintained constant for the fir st case. Based on the results obtained and the design experience of the author, member stiffness was altered suitable.

**Prasad V. et al. (2002)** gives introduction of wind loads on multistory building. Generally multistoried buildings are affected by wind lo ads due to induction of stresses in the structural members. The analysis of a reinforced concrete framed building for wind load can be done by considering the effect of internal pressures acting on it. The external effect is due to wind pressure acting upo n the entire height including any projection if exists in the building. The effect of internal pressure is also considered in case of multistoried buildings with no openings. The analysis of structural rigid frame is done by considering the direct effect o f wind pressure and also due to transmission of moments from other members. The present paper highlights the case when wind pressure is directly striking the wall and then transmits the load of the frame. The author has tried to analyze the seven storied r esidential buildings for wind loads by approximate method.

**Kalehsar, H.E. and Kumar, K. (2002)** studied the effect of wind turbulence parameters on the model of a tall rectangular building with proportion of 12:2:1 and height 300 m. The study has shown that the RMS Across-wind respond of the building is affected more than its RMS Along-wind response for long after body as well as short after body orientation. An aero elastic instability has been observed and predictable only through wind tunnel testing.

**Eimani, H. et al. (2004)** presents the relations for along- wind and Across-wind motions of tall rectangular buildings have been suggested. The study has been made analytically and experimentally and modifications have been made in the analytical procedure for Along-wind response. The effects of height and after body of the building on its response are investigated. For this purpose, two boundary layer shear flows with power law exponent □=0.18 and □=0.3 have been generated. In the present study, two tall rectangular building of height × plan dimensions as 300m× 50m× 25m (proportions 12:2:1 and 10:2:1) were chosen. The study has been made in large size boundary layer wind tunnel.

**Liang S. et al. (2005)** states that tall buildings under wind action usually oscillat e simultaneously in the along-wind and across-wind directions as well as in torsional modes. While several procedures have been developed for predicting wind-induced



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loads and responses in along-wind direction, accurate analytical methods for estimating across-wind and torsional response have not been possible yet. Simplified empirical formulas for estimation of the across-wind dynamic responses of rectangular tall buildings are presented in this paper. Unlike established empirical formulas in codifications, the formulas proposed in this paper are developed based on simultaneous pressure measurements from a series of tall building models with various side and aspect ratios in a boundary layer wind tunnel. Comparisons of the across-wind responses determined by the proposed formulas and the results obtained from the wind tunnel tests as well as those estimated by two well-known wind loading codes are made to examine the applicability and accuracy of the proposed simplified formulas. It is shown through the comparisons that the proposed simplified formulas can be served as an alternative and useful tool for the design and analysis of wind effects on rectangular tall buildings.

**Chen X. et al. (2005)** shows that High frequency force balance (HFFB) measurements have recently been utilized to identify the distribution of spatiotemporally varying fluctuating wind loads on buildings. These developments, predicated on their ability to compute any response component of interest, based on actual building characteristics, attempt to offer a framework that eliminates the need for mode shape corrections generally necessary in the traditional HFFB technique. To examine the effective ness of these schemes with significant practical implications to wind tunnel modeling technology, this technical note utilizes a recent approach to identify the along wind loading on buildings. The predictions are compared to a widely utilized analytical loading model. It is noted that, akin to the traditional HFFB technique, the accuracy of these identification schemes clearly depends on the assumed wind loading model.

**Kumar A. et al. (2006)** deals with the development of a numerical code to study flow over prismatic buildings in tandem arrangement by means of Large Eddy Simulations (LES). Flow over two buildings in succession with different spacing and heights have been considered in this study. Two dimensional unsteady Navier-Stokes equations have been solved using LES turbulence model. Streamline plots, isovorticity lines, surface pressure distribution (Cp) and velocity profiles have been obtained. Two and three dimensional experimental surface pressure distribution have been generated by conducting experiment in the 60cm×60cm test section wind tunnel facility available in the department and compared with predicted Cp values. Some significant differences have been observed between 2D and 3D surface pressure distributions.



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**Bodhisatta H. 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.



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# CHAPTER - 3
# Methodology

## 3.0 General

For the simulation of the natural wind flow for Tall buildings, we have used ***Rigid Model Studies*.** In this chapter a test program has been discussed for the building under **Standalone** and **Interference** conditions. Various instruments are used along with the data acquisition system for the Base force measurements and the Surface pressure measurements over the building model in the wind tunnel described in this chapter. The salient features of Boundary layer Wind Tunnel used for the experiment are also mentioned in brief.

## 3.1 The Wind Tunnel

The experiment has been conducted in the Boundary Layer Tunnel at Wind Engineering Centre, Department of Civil Engineering (University of Roorkee), Roorkee, India as shown in **Fig 3.1**. The Tunnel is an open circuit tunnel with continuous flow of wind. It is a suction type tunnel in which suction flow is made with a blower fan (125HP) which has a test section of 2.1m x 2.0m size. The length of the test section is 15m. At the entrance of the tunnel has an elliptical effuser profile with contraction ratio 9.5:1 along with a squared-holed Honeycomb (6m x 6m), which helps to develop a smooth flow in the test section. A manually controlled turntable is installed at 12m downstream of the test section entrance. Models under study are installed at the centre of the turn table. Continuously variable wind speed in the range of 2m/sec to 20m/sec can be achieved by remotely operated 'dynodrive' using eddy current controls. The other instruments of which wind tunnel is equipped of are discussed below.

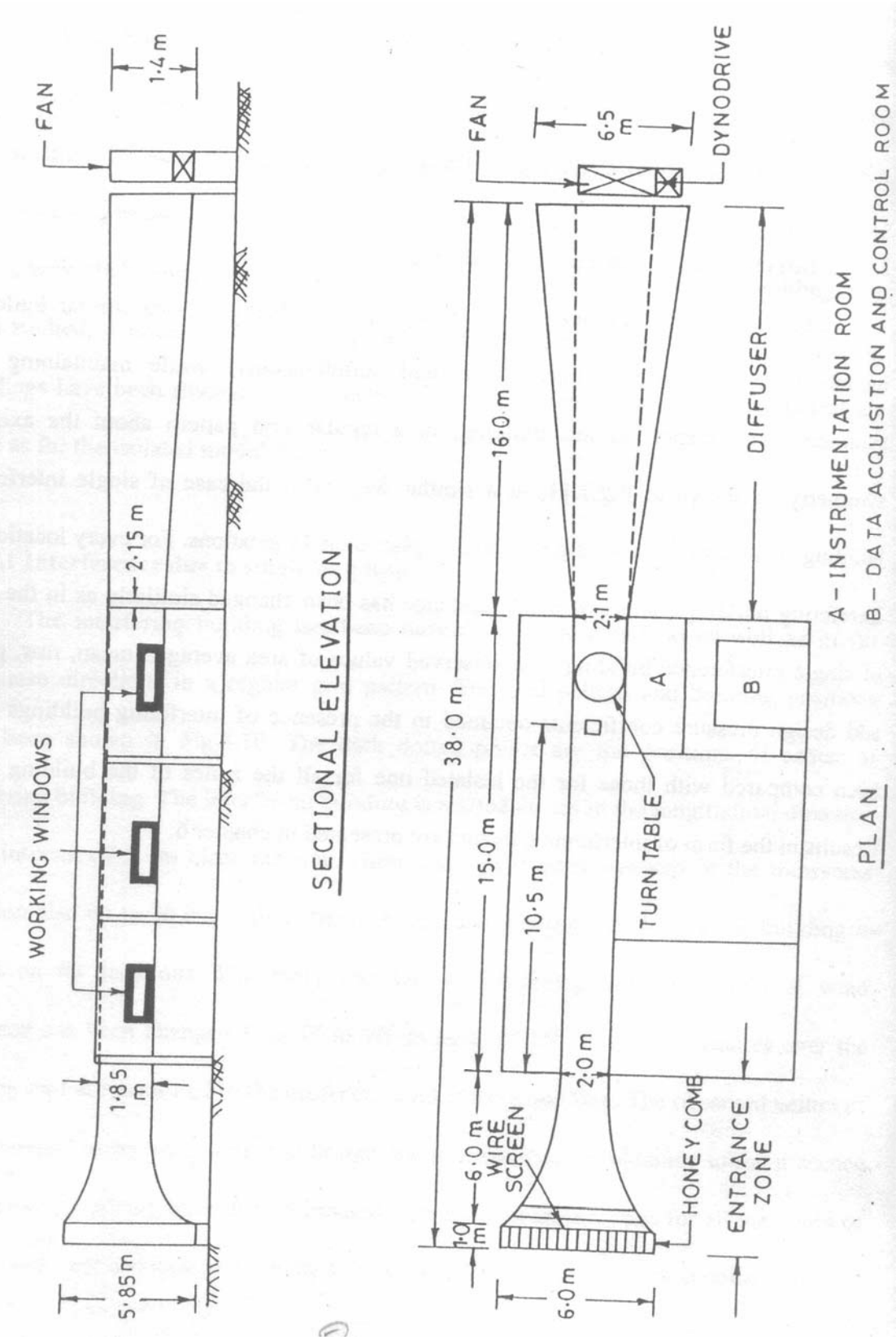



**Fig 3.1 Boundary Layer Tunnel at Wind Engineering Centre, Department of Civil Engineering (University of Roorkee), Roorkee, India**

## 3.2 Wind Tunnel Instrumentation

Because most of the instruments used in wind tunnel tests are standard equipment discussed in many fluid mechanics texts, only a brief discussion of certain salient features of each important instrument will be given in this chapter.

### 3.2.1 Pitot tube

Pitot tube is the basic instrument used for measuring wind speed in a wind tunnel. It is based on the principle of conversion of kinetic energy to pressure at a stagnation point-the tip of the Pitot tube. The pressure differential sensed by the tube is proportional to the square of the velocity. This instrument is accurate, reliable, convenient, and economical. Furthermore, it does not require calibration. However, the Pitot tube is inaccurate at low speeds (about less than 5 m/s) and unsuitable for measuring turbulence.

### 3.2.2 Hot-Wire Anemometer

The sensing element of a hot-wire anemometer is a fine wire made of tungsten, platinum, or a special alloy. The wire is finer than human hair, and its length is only about 1 mm. The two ends of the wire are welded to two pointed electrodes (support needles) connected to a source of electricity. The turbulence in the wind causes changes of heat transfer from the wire, which in turn causes the resistance of the wire to fluctuate. The electronic circuit automatically adjusts the current going through the wire to keep the wire at constant temperature. Consequently, the velocity fluctuations (turbulence) can be determined from the fluctuations of the current through the wire. A variant of the hot wire anemometer is the hot-film anemometer. The sensing element of a hot film is a coated metal film laid over a tiny glass wire. The rest are the same as for hot wires. The device is more robust than the hot wires and hence can be used not only in air but also in water and contaminated environments.

Hot-wire or hot-film anemometers can be used to measure both mean velocity and turbulence. They can measure rapid changes of velocities with frequency response higher than I kHz. Due to the small size of its sensing element, the velocity measured by a hot wire is often considered as the point velocity. Calibrations of hot wires are done by using a Pitot tube placed alongside a hot wire in a wind tunnel having approximately a uniform flow.

### 3.2.3 Manometers

Manometers are the standard equipment for measuring mean (time-averaged) pressure and for calibrating pressure transducers. Like Pitot tubes, manometers are accurate, reliable, and economical, and do not require calibration.

### 3.2.4 Pressure Transducers

Pressure transducers can measure both mean and fluctuating pressures and are discussed in section 3.4 for Surface Pressure Measurements.

### 3.2.5 Other Sensors

Many other transducers (sensors) may be needed in a wind tunnel study. These include strain gauges for measuring strain, accelerometers for measuring the accelerometers of models, and so on. They are standard sensors familiar to most structural engineers and hence not explained here.

### 3.2.6 Data Acquisition Systems

Modern data acquisition systems for wind tunnel tests consist of on-line processing of data by digital computers. Many mini and micro computers equipped with an analog-to-digital converter can perform such duties. The computer records the signals from various transducers, analyzes the signals, and prints or plots the results in desired forms. Such systems have brought great convenience to wind tunnel testing.

## 3.3 Flow Simulation

An issue for prime importance for experimental work with building models in a wind tunnel is the modeling of the characteristics of the atmospheric boundary layer (ABL) is to be modeled in the wind tunnel in order that structure models respond as closely as possible to their prototype behavior. The general criteria for ABL simulation include the duplication of vertical distribution of mean wind speed, longitudinal turbulence intensity as per the distribution in the field, and the integral scale of longitudinal turbulence, as closely as possible at the same geometric scale used for the building model. For tall buildings models, scale ratio of the order of 1:250 to 1:500 is appropriate.

### 3.3.1 Velocity Measurement in the Wind Tunnel

Measurements of the fluctuating velocity of the incoming flow in the wind tunnel have been made using a single hot wire probe. The hot wire probe and the associated instrumentation have been calibrated to give a voltage velocity relationship, to enable

the conversion of the acquired voltages to the wind velocities. The calibration has been carried out in smooth flow, with turbulence level not exceeding 0.5% at 1m height in the test section. Static velocity head has been measured using a standard pitot-tube, connected to KBS Baratron Transducer and its digital display unit. Corresponding head has been converted to velocities and simultaneous values of voltage output at mean value unit of hot-wire system were recorded for a range of wind speed between 2m/s and 20m/s

### 3.3.2 Establishing Flow Conditions

For flow measurements with the hot-wire system, a sampling frequency beyond 1 KHz and a 4-second length of record was adopted as minimum requirement (Gupta 1996). In the present study, instantaneous velocity fluctuations have been recorded using hot-wire probe at a sampling frequency of 4 KHz for a duration of approximately 4 seconds viz. a total of 16384 samples are recorded at each point for flow characteristic measurement.

The mean velocity and longitudinal turbulence intensity variation obtained in the wind tunnel are presented in **Fig 3.2**. Theoretical velocity profile variation (solid line), corresponding to $\alpha = 0.18$, is in good agreement with measured values. The theoretical curve, which is called power law, is given as

$$\mathbf{( V_Z / V_O ) = ( Z / Z_O )^{\alpha}}$$

Where $\boldsymbol{\alpha = 0.18}$

$V_O$ is the velocity at $Z_O$ = 1m height from the tunnel floor. For different values of ( $Z / Z_O$ ), ( $V_Z / V_O$ ) values are calculated. The values of mean velocity and longitudinal turbulence intensity at the topmost height of the building model have been found to be 10.78 m/sec.

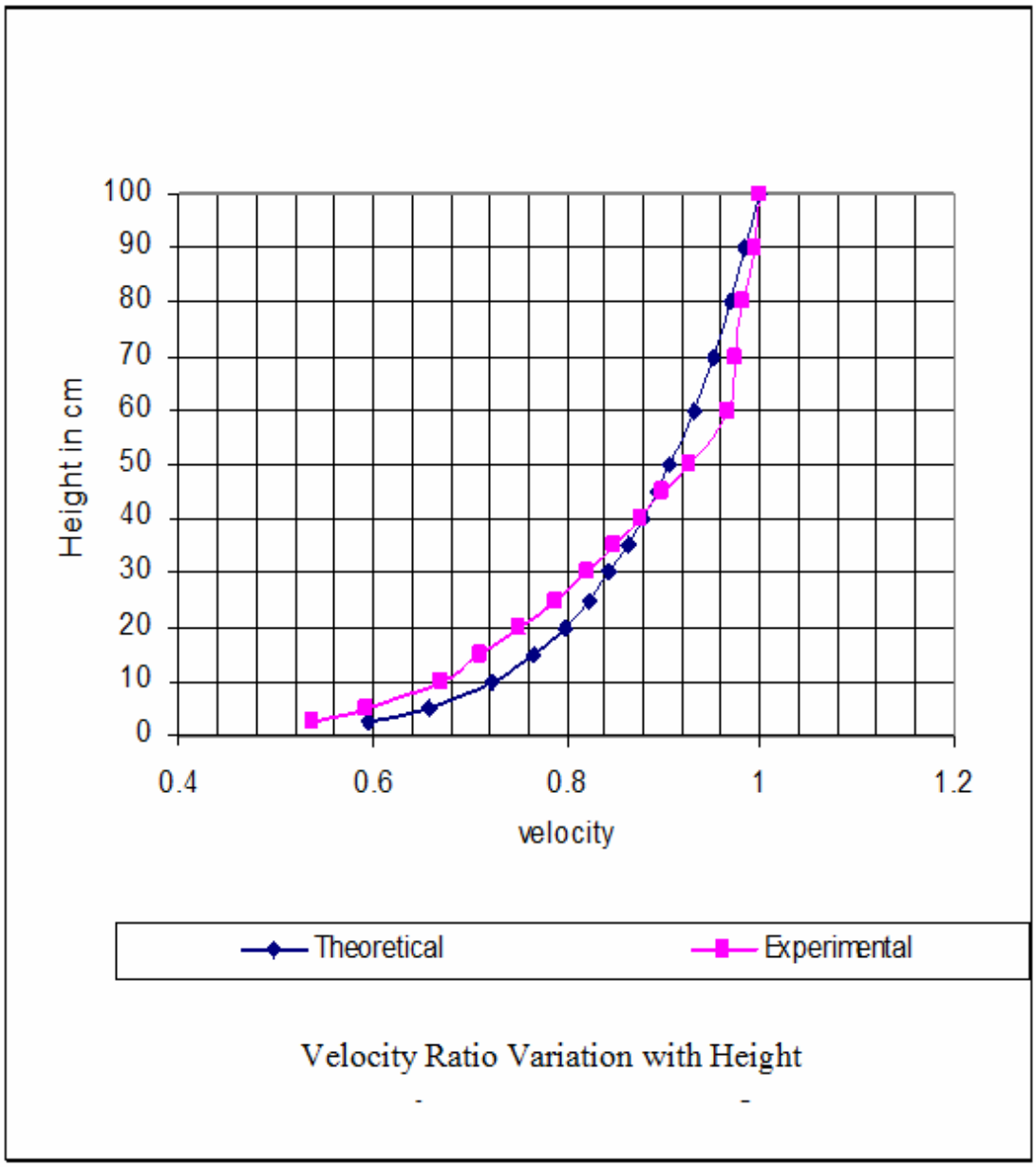



**FIG 3.2 Variation in Velocity with Height during Flow Conditions**

## 3.4 Surface pressure measurements

Holmes and Lewis (1986, 1987 and 1989) performed extensive experimental work on the fluctuating pressure measurements using a small diameter 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. In the present study the choice of tubing system for pressure measurements is largely based on the work of Holmes and Lewis (1987).

Surface pressure on the faces of the building models have been measured by providing steel taps of 1-mm internal diameter are flushed to model surface, which in turn are connected to small diameter tubing. Two stage restricted tubing has been used to measure pressure on each tap. Pressure measuring system consists of 500mm Vinyl tube with 40mm restrictor at 400mm from pressure point and a ZOC22 Scanivalve pressure scanner. Internal diameter of Vinyl tube is 1.5mm and internal diameter of the restrictor is 0.4 mm. This system is close to system suggested by Holmes and Lewis (1987) who obtained a linear response up to 200 Hz. The system used has therefore been considered suitable for study made and reported in this thesis. Reference static pressure has been measured in the tunnel floor at 1.5 m from the centre of test building.

A 32 channel ZOC22 pressure scanner from Scanivalve Corporation Ltd. is used to measure pressure. The output signal in the form of voltage from pressure scanner has been recorded using PCL206 ADC Card. A computer program is developed to acquire the voltage signal from Scanivalve through PCL206 ADC Card Data has been recorded at a sampling of 1000 samples/sec/channel. 8192 samples of pressure data from each channel have been recorded, thus giving a record of approximately 20 seconds. All the readings have been repeated once to ensure repeatability.

Wind pressures measured on the faces of the building models are expressed in the form of a non dimensional pressure coefficients defined as follow:

$$Cp(i) = \frac{P(i) - P(o)}{1/2\rho V^2} \qquad \text{.....3.1}$$

Where Po = Static (ambient. atmospheric) reference pressure

□= Air Density

V = Mean velocity measured at topmost height of the model

Since the pressure at any point on the roof of the building is fluctuating with time, the pressure coefficient can also be treated as time varying quantity. Following statistical quantities of pressure coefficient were obtained from sampled time history Cp(i):

Mean value = Cpmean = $\frac{1}{N}\sum_{i=1}^{N} Cp(i)$ , where N is the total number of Samples

rms value = Cprms = $\frac{1}{N-1}\sum_{i=1}^{N}\sqrt{(Cp(i) - Cpmean)^2}$

peak value = Cpmin = $Min^m$ of Cp(i)

Cpmax = $Max^m$ of Cp(i)

In wind tunnel testing ***Rigid Model Studies with pressure tapings or transducers*** have been used. In which Rigid models are used to determine the fluctuating local pressures on the exterior surfaces of the building. **Fig 3.3** shows the model of the Signature being tested in Wind Tunnel.

**Fig 3.3 Model of the Signature being tested in Wind Tunnel for Pressure Measurement**

It is common to use Perspex as the construction material. The exterior features of the building that are considered to be important with regard to the wind flow are simulated to the correct length scale; using architectural drawings. The model is instrumented with a large number of pressure taps (500 to 800) around the model surface to obtain a good distribution of pressures. More tapings are required in regions of high-pressure gradients, such as corners, Slits opening etc. The pressure tapings are connected by plastic tubing to miniature electronic pressure transducers which can

measure the fluctuating pressures. The length of plastic tubing is kept as short as possible to minimize the damping of fluctuating pressures in the tubing. As it is uneconomical to use a single transducer for each pressure tapping, the transducer is mounted onto a pressure-scanning device, such as a Scanivalve, which automatically switches the pressure transducer between 40 to 50 pressure taps, one at a time. Pressure data is acquired by an on-line computer system capable of sampling data at a high speed. The setup is shown in the **Fig 3.4**

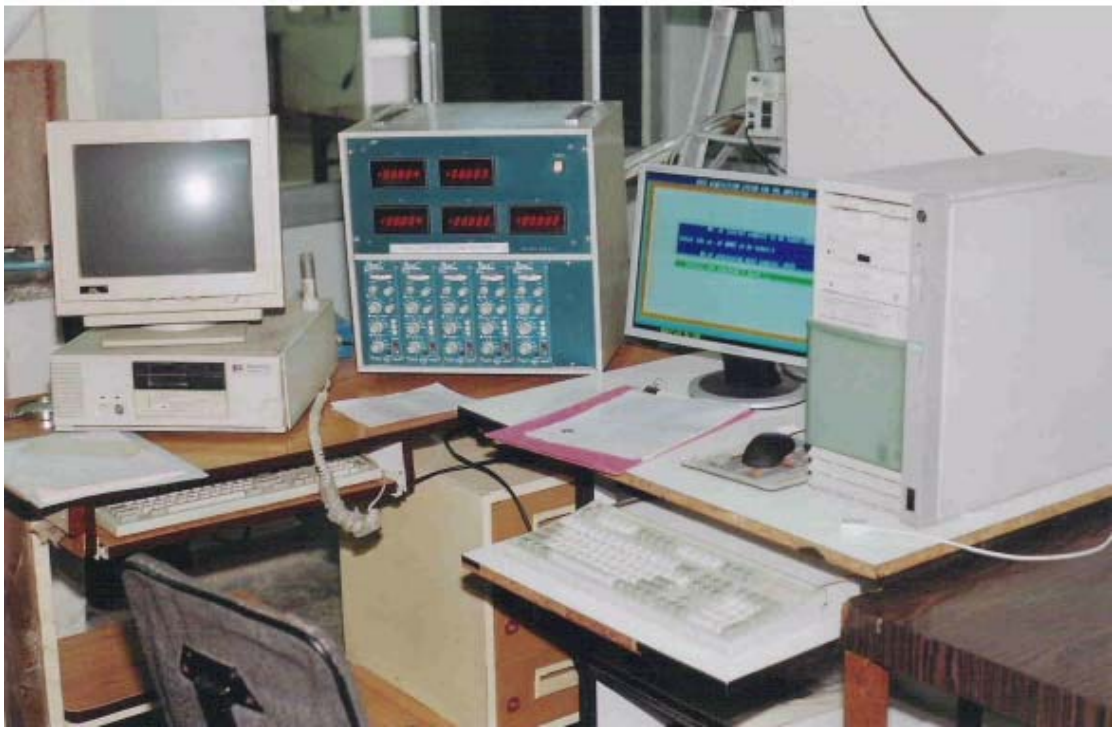


**Fig 3.4 Setup on which Pressure data is acquired by an on-line computer system**

Usually a rate of 1000 samples/sec is used. As the transducer measures only the pressure differentials, the static pressure upstream of the tunnel is used as the reference pressure. The model is mounted on a turntable in the boundary-layer wind tunnel with surrounding buildings within a radius of about 500m. Because of the ease of construction, near field features are simulated by polystyrene foam or wooden blocks. The turntable provides the facility to test the model at different wind directions by simply rotating the turntable to the desired angle. The pressure measurements are taken for wind directions spaced 10°to 20° apart. For each wind direction, the data are collected for a duration equivalent to 1 hour in the prototype, to

obtain stationary values for mean and root mean square pressures. The data record is divided into segments corresponding to 5 to 10s duration in full scale, and the maximum and minimum values of pressure are calculated for each segment. These individual maximum and minimum values are used in an extreme-value analysis to determine the most probable maximum and minimum values applicable for the whole sample period. The maximum and minimum pressures are expressed as pressure coefficients. The calculated data are presented in the form of pressure contours or isobars.

## 3.5 Dynamic Analysis of the Wind Forces on the Building

### 3.5.1 High-frequency-Force Balance Model

An aero elastic model provides comprehensive information on the dynamics loads and motion of the prototype. However, construction of an aero elastic model is complex and costly, and cannot be carried out until the essential structural features, such as the distribution of stiffness and mass of the prototype are finalized. The high-frequency force balance technique provides an alternative method which is more economical and time efficient. In this method, generalized wind-induced forces in a building with a linear mode-shape are determined by measuring the dynamic base moment acting in a rigid model simulating the geometry of the building. The model is mounted on a high sensitive stiff force balance, which measures the base overturning moment. The frequencies of the model and the balance are chosen to be sufficiently high, so that there are no distortions in the dynamic wind loads due to resonance in the frequency range of interest. The power spectrum of the measured base moment is the same as the power spectrum of the generalized force corresponding to a linear mode. From the power spectrum of the generalized force the root mean square dynamic displacement, acceleration, shear and moment are determined analytically.

### 3.5.2 High-Frequency Base Balance Technique [ASCE Manual (1987)]

The generalized wind-induced forces on a building or a structure with a mode shape which varies linearly with height can be evaluated by measuring the dynamic base moments. In this procedure, a stiff geometrical replica of the structure is mounted on a highly sensitive and stiff force balance which measures the base moments and base torque. The frequency of the model and balance system must be sufficiently high to

avoid distortions of the dynamic wind loads in the frequency range which affects the resonant response of the full-scale structure. The frequency of the dynamic loading in the wind tunnel, which corresponds to the full-scale excitation at the natural frequency of the structure $f_{op}$ and a full-scale wind speed of $V_p$' is

$$f_m = f_{op} \frac{L_p}{L_m} \frac{V_m}{V_p} \qquad \text{…….. 3.2}$$

in which $V_m$ = the corresponding model wind speed, L = characteristic dimension of the structure, and m and p respectively denote model and full-scale values. The technique just described is commonly used to evaluate the wind-induced dynamic loads on tall buildings. The base moments provide direct measures of the generalized forces associated with the fundamental sway modes of vibration. The measured base torque requires adjustment in order to provide an estimate of the generalized torque, which is the integral of the torque per unit height weighted by the mode shape taken over the height of the building. Corrections to the generalized sway forces can be made in situations where the fundamental sway mode shapes do not vary linearly with height. The loads in the two sway directions and the torque can be combined for dynamic systems which have three-dimensional modes of vibration. Analytical methods are used to evaluate the dynamic structural response once the generalized forces $\mathbf{F_X}$, $\mathbf{F_Y}$ their corresponding moments $\mathbf{M_X}$, $\mathbf{M_Y}$ and the generalized torque $\mathbf{M_Z}$ are determined.

Although this technique has become widely accepted in studies of tall buildings, there are still situations where aero elastic simulations are desirable.

High frequency force balance (HFFB) measurements are utilized to varying fluctuating wind loads on buildings. These developments, predicated on their ability to compute any response component of interest, based on actual building characteristics, attempt to offer a framework that eliminates the need for mode shape corrections generally necessary in the traditional HFFB technique. The predictions are compared to a widely utilized analytical loading model. It is noted that, akin to the traditional HFFB technique, the accuracy of these identification schemes clearly depends on the assumed wind loading model.

## 3.6 Test Program

### 3.6.1 Isolated Model Study (*Standalone condition*)

All buildings models have been tested in the wind tunnel under the simulated flow condition as described earlier for the angle of wind incidence of $0^o$ to $360^o$ with increments of $15^o$. All the values for pressure coefficients have been recorded for mean, maximum, minimum and rms values in case of Standalone condition and values for $F_X$, $F_Y$, $M_X$, $M_Y$, $M_Z$ in case of Dynamic analysis of wind forces of Standalone condition with corresponding terrain category 3 for both analysis are presented in chapter 4. **Fig 3.5** shows the model of the building being tested in wind tunnel in standalone condition.

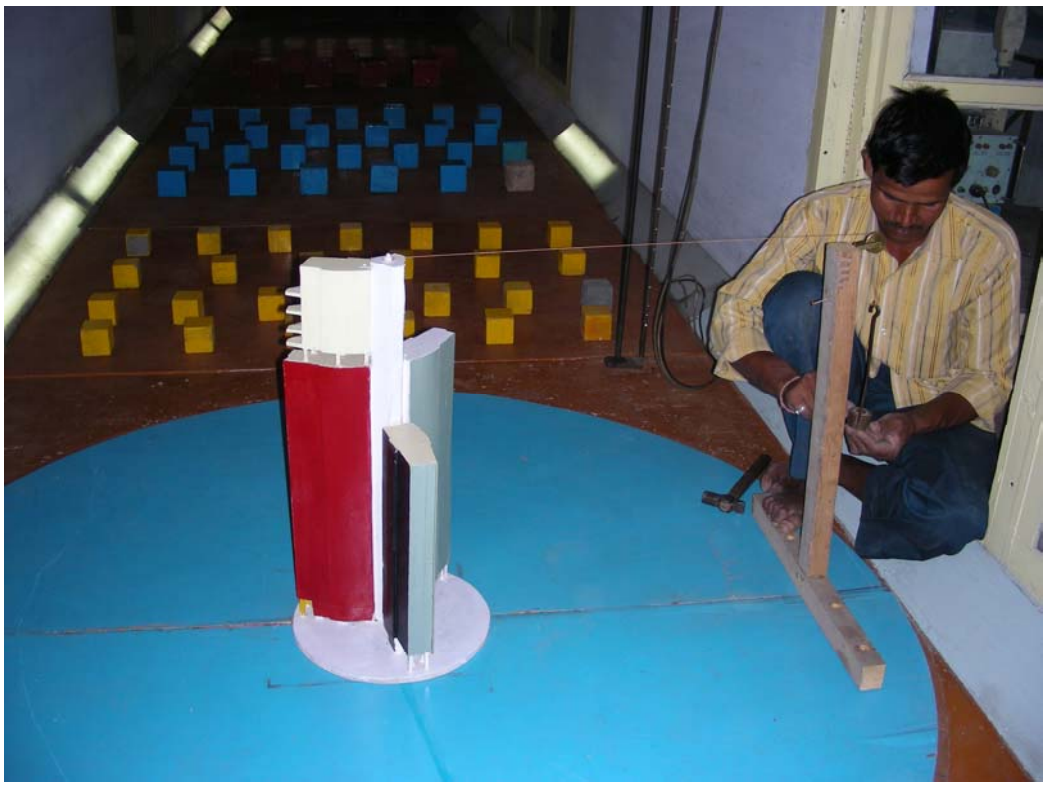


**Fig 3.5 shows the Signature building model being tested in wind tunnel in standalone condition.**

### 3.6.2. Study of Interference Effects (*Interference Condition*)

All the buildings and surroundings which interfere in simulated flow and were provided in the building surroundings map are incorporated in the wind tunnel in case of Interference Condition. And again all the values for pressure coefficients and forces and moments in case of Dynamic analysis of wind force are obtained and presented in chapter 4 with the same procedure followed in case of Standalone

condition with little difference of interfering buildings included. **Fig 3.6** shows the model of the building being tested in wind tunnel in interference condition.

**Fig 3.6 shows the Signature building model being tested in wind tunnel in interference condition.**

# CHAPTER -4

# Dynamic Analysis of Wind Forces on Tall Buildings

## 4.0 General

This thesis is an attempt to study behavior of the tall buildings under simulated atmospheric boundary layer and to evaluate various experimental and analytical techniques to compute dynamic response and present a detailed comparison.

Researchers have laid down several analytical procedures during last few decades. Even though there are several grey areas which need to be addressed to achieve a better prediction of the response, i.e., a designer is interested in storey wise horizontal forces for dynamic analysis and design of structural frames. Hence, emphasize is given to compute the story wise lateral forces on building by analytical procedure and through base forces obtained by Wind tunnel testing on scaled model of building and surrounding terrain.

As all the proposed method for computation of forces is given for regular shape (square or rectangular) of building i.e., plan ratio 1:2 or 1:3. But as the plan ratio changes (1:3 to1:7or more) it can produce significant changes in the responses of a tall building. So at time of design we have to consider it.

The present study has been undertaken to bridge a few of the many existing gaps mentioned above, in the field of wind induced response of tall buildings.

Wind tunnel test on a typical rigid or aero-elastic model of tall building yields dynamic base moment/ forces or tip acceleration & displacement in two principle direction, i.e., along wind and across wind direction as well as torsional forces.

## 4.1 Theoretical Background

### 4.1.1 Analytical Estimation of the Dynamic Wind Response

#### 4.1.1.1 Dynamic Wind response by using 'Davenport Gust Factor Approach'

Since early 1960's, when Davenport's (1961) explained statistical concepts of the stationary time series for the determination of the response of simple structures to a turbulent gusty wind, efforts have been made to express peak stresses, accelerations, defalcations, etc., in terms of the mean wind velocity, the spectrum of the gustiness

and the mechanical and aerodynamic properties of the structure. Still today Davenport's (1967) 'gust loading factor approach' forms the most acceptable approach for prediction of mean and fluctuating response of slender structures [Lee (1988)]. In Davenport's work (1961) it is mentioned that "the practice of calculating pressure and force co-efficient based on the highest instantaneous recorded velocity, in 1930's and 1940's, was though simple but not ideal. Sherlock in 1947 advocated the use of an average instead of an instantaneous velocity, together with certain 'gust factor' which would allow for the additional effects of gusts. He recommended use of '5-min' averaging period. Later he was able to determine the ratio of the 'most probable' 2-sec. mean velocity to simultaneous 5-min. velocity. This was the so called '2-sec. gust factor' Davenport(1961b & Onwards) applied the concept of the stationary time series Along with applications of the tools of probability and statistics to define a rational design with velocity and the structural response[Davenport(1960),(1961a)].

Based on the large number of data collected at several stations, Davenport (1960) proposed an expression of gustiness of the wind. He later used the theory of probability distribution of peaks occurring with a certain averaging period and defined a peak factor to estimate the gust factors for evaluating the fluctuating response of structures. Davenport defined three different types of surface roughness and velocity profiles (power law, α) and terrain drag co-efficient (k) associated with them. A detailed description of development of the 'gust factor approach', the wind characteristics and application of statistical concepts to 'Gust Loading' of structures can be found in various works of Davenport (1960, 1961a, 1961b, 1962, 1963a, 1963b & 1964). The following below section outline the theory of this approach.

#### 4.1.1.2 Davenport's Gust Factor Approach

Let us consider a tall slender building of height 'h', being exposed to mean wind speed $V_h$ at top. Then the mean wind pressure near the top of building is given by:

$$P_H = \frac{1}{2}\rho_a C_D V_H^2$$

Where,

$\rho_a$ =The air density is affected by altitude and depends on the temperature and pressure to be expected in the region during wind storms. Unless otherwise specified, the value of $\rho_a$ shall be 1.208 kg/m3.

$C_D$= is Drag or force co-efficient of structure, depending up on the shape and size (Aspect ratio) of building.

The mean wind velocity variation with height is assumed to follow a Power law, as given by equation:

$$Vz / V_G = (Z / Z_G)^{\alpha}$$

Where,

$V_z$ is the means wind velocity at height Z,

$V_G$ is the mean velocity at the gradient height,

$\alpha$ is power law coefficient

Here coefficient $\alpha$ depends up on the roughness of the upstream terrain to which the building is exposed.

It follows from the nature of the wind that a structure exposed to wind will experience a steady wind load associated with the mean wind velocity and a fluctuating component associated with the gust and turbulence.

An empirical expression obtained from measurements suggested by Davenport (1961a) has the non dimensional form:

$$\frac{fS_u(f)}{\bar{V}_{10}} = \frac{4.0kX^2}{\left[1+X^2\right]^{4/3}}$$

where,

$$X = \left(\frac{n_o L}{\bar{V}_h}\right)$$

In which $S_u(f)$ is the power spectral density of wind fluctuations at frequency $n_o$, $V_{10}$ is a reference wind velocity taken at 10m height, L is a length scale which is approx. 1200m, and 'k' is the Drag co-efficient of terrain defined in Davenport(1961a&b)

Davenport showed that the average largest response during a period 'T' is given by:

$$X_{\max} = \bar{X} + g_f \sigma_x$$

where,

$\bar{X}$ is the response to mean wind load, and

$g_f$ is the peak factor

$$g_f = \sqrt{2\log_e}\nu T + \frac{0.577}{\sqrt{2\log_e}\nu T}$$

Where ν is the number of times the mean value is crossed per unit time. For a lightly damped system $\nu = n_o$, the natural frequency of the system. Davenport has suggested 600secs to 3600secs as the appropriate averaging period 'T', considering the spectral gap in the wind spectrum. Now the important parameters to be evaluated are the mean response ($\bar{X}$), RMS response ($\sigma_x$), and the peak factor ($g_f$). From the above equations

$$\frac{X_{\max}}{\bar{X}} = 1 + g_f \frac{\sigma_x}{\bar{X}}$$

This gives,

$$G = 1 + g_f \frac{\sigma_x}{\bar{X}}$$

Where G, called 'Gust Factor', is the ratio of expected maximum response to the mean response.

#### 4.1.1.3 Mean response

1. For a mean wind velocity $V_h$ at top of the building, following a power law variation with height, the mean tip displacement is given by:

$$\bar{X} = \frac{Mean\ \ Generalised\ \ force\ in\ \ first\ \mathrm{mod}\, e}{Generalised\ stiffness\ in\ \ first\ \mathrm{mod}\, e}$$

A generalized mean wind load F can be defined by the equation:

$$F = \int_0^H \bar{P}(Z)\, \bar{\mu}(Z)\, dZ\, B$$

Where,

$\bar{\mu}(Z)$ is the linear mode shape. Assuming

$$\bar{\mu}(z) = \sqrt{3}\left(\frac{Z}{H}\right)$$

$$F = \sqrt{3}\left(\frac{B}{H^{(1+2\alpha)}}\right)\bar{P}_H \int_0^H Z^{(1+2\alpha)}\, dZ$$

Or

$$F = \frac{\sqrt{3}}{2}\left[\frac{A}{(1+\alpha)}\right]\bar{P}_H$$

Where A, is the projected area of the building normal to the mean wind flow and B is the width of the building. The mean response of the first mode at the building height can be obtained as:

$$\bar{X} = \frac{F}{K_o}$$

or

$$\bar{X} = \frac{C_d\, P_h\, b\, h}{2(1+\alpha)\, K_o}$$

Where, $\frac{1}{2(1+\alpha)}$ takes into consideration the pressure variation along the height, and

$M_o = 1/3\ \rho_b\ b\ D\ h$, is the generalized mass in first mode,

$K_o = 4\pi^2\ M_o\ n_o^2$, is the generalized stiffness in first mode.

$\rho_b$ is the building bulk density of building.

2. knowing the mean response $\bar{X}$ , the expected maximum response calculated by knowing 'peak factor' and 'gust factor'. Fluctuating part of RMS response may then be obtained.

$$\frac{X_{\max}}{\bar{X}} = 1 + g_f \frac{\sigma_x}{\bar{X}}$$

Where $g_f$ is the peak factor, and $\sigma_x$ is the RMS response.

$$\frac{X_{\max}}{\bar{X}} = 1 + g_f \frac{\sigma_x}{\bar{X}} = G$$

Where 'G' is called the gust factor

#### 4.1.1.4 Response to Turbulence

It was shows that the fluctuating motion of a building is composed of two components, one is that part caused by wind frequency components significantly lower then the building natural frequency, called 'background response' and second is that part exhibiting a resonant response with building natural frequency, called as 'resonant response'.

Davenport presented an expression for gust factor which is composed of two parts, corresponding to each of the above, as follows:

$$G = 1 + g_f r \sqrt{(B + R)}$$

$g_f$ is peak factor

r = roughness factor

$$r = 4.0\sqrt{k}\left(\frac{10}{h}\right)^{\alpha}$$

B = background turbulence excitation

$$B = 2\left[1 - \frac{1}{\left\{1 + \left(\frac{457}{h}\right)^2\right\}^{\frac{1}{3}}}\right]$$

R= resonant response excitation,

$\varsigma$ =Critical Damping Ratio

$$R = \frac{sF}{\varsigma}$$

S = size reduction factor

$$S = \frac{\pi}{3}\frac{1}{\left(1+\frac{8\xi_0}{3}\right)}\frac{1}{\left(1+\frac{10\xi_0 b}{h}\right)}$$

$$\xi_0 = \frac{hn_o}{\bar{V}}$$

$\xi_0$ = reduced frequency

$$E = \frac{(X)^2}{\left\{1+(X)^2\right\}^{\frac{4}{3}}}$$

E=Gust energy ratio

$$\text{Where } X = \left(\frac{n_o L_h}{\bar{V}_h}\right)$$

Considering the above analytical procedure given by Davenport in IS 875: part 3-1987, the response of the given Signature Towers is calculated analytically with the sufficient data provided.

### 4.1.2 Based upon the above Analytical Analysis the Analytical Response of Signature Building Davenport's Gust factor method and Codal procedure is given below

## Given:-

| | | | |
|---|---|---|---|
| $\alpha$ | = | 0.18 | |
| z (ref) | = | 10 | m |
| Plan length | = | 87 | m |
| Plan width | = | 71 | m |
| Height of building | = | 150 | m |
| Bulk density | = | 110 | $Kg/m^3$ |
| Face width | = | 87 | m |
| Face depth | = | 71 | m |
| Interval | = | 10 | m |
| Natural period | = | 5.10 | sec |
| Critical damping | = | 0.035 | |

## Calculation:-

| | | | |
|---|---|---|---|
| From code $V_b$ | = | 47 | m/sec |
| $K_1$ | = | 1 | |
| $K_2$ | = | 0.50 | (At 10m from Table-33) |
| $K_3$ | = | 1 | |
| $V_{ref}$ (at 10m) | = | $V_b * K_1 * K_2 * K_3$ | |
| | = | 47*1*0.50*1 | |
| | = | 23.5 | m/sec |
| At top of building | | | |
| $K_1$ | = | 1 | |
| $K_2$ | = | 0.84 | (At 150m from Table-33) |
| $K_3$ | = | 1 | |
| $V_z$ | = | 47*1*0.84*1 | |
| $V_z$ | = | 39.48 | m/sec |
| $V_z / V_G$ | = | $( Z / Z_G )^{\alpha}$ | |
| Vz | = | $V_G (Z / Z_G)^{\alpha}$ | |
| | = | $23.5(150/10)^{0.18}$ | |
| | = | 38.26 | m/sec |
| Pz | = | $0.6(V_z)^2$ | |
| | = | $0.6*(38.26)^2$ | |
| | = | 878.375 | $N/m^2$ |

**Calculation for Gust Factor, (G)**

| | | | |
|---|---|---|---|
| T | = | 3600 | sec |

Building frequency $n_0$ = 1/Natural period

= 1/5.10

= 0.196 Hz

$m_0$ = building mass/m height

= $\rho_b$*B*D

= 110*87*71

= 679470 Kg/m

$M_0$ = Generalized mass in 1st mode

= 1/3 $m_0$ H

= 1/3*679470*150

= $3.4*10^7$ Kg

$g_f$ = is the peak factor, given by

$g_f$ = $\sqrt{2\log_e} vT + \frac{0.577}{\sqrt{2\log_e} vT}$

gf = (2*logn $n_0$T)0.5+0.577/(2*logn $n_0$T)0.5

gf = 3.78

$C_f$ from graph = 1.2

$K_0$ = Generalized stiffness in 1st mode

$K_o$ = $4\pi^2$ Mo $n_o^2$

= $4*3.14^2*3.4*10^7*0.196^2$

= 51513283 N/m

$L_h$ = From graph (IS-Code)

= 1800

$X$ = $\left( \frac{n_o L_h}{\bar{V}_h} \right)$

X = 0.196*1800/38.26

X = 9.225

Gust energy ratio = $E = \frac{(X)^2}{\left\{1+(X)^2\right\}^{\frac{4}{3}}}$

E = $9.225^2/ (1+9.225^2)^{4/3}$

E = 0.224

$\xi_0$ = reduced frequency,

$\xi_0$ = $\frac{hn_o}{\bar{V}}$

$\xi_0$ = 150*0.196/38.26

= 0.769

S = is size reduction factor,

S = $\frac{\pi}{3}\frac{1}{\left(1+\frac{8\xi_0}{3}\right)}\frac{1}{\left(1+\frac{10\xi_0 b}{h}\right)}$

S = (3.14/3)*(1/ (1+8*0.769/3))*(1/ (1+10*0.769*87/150))

S = 0.063

R = is resonant response excitation,

R = $\frac{SE}{\varsigma}$

R = 0.063*0.224/0.035

R = 0.402

B = is background turbulence excitation,

B = $2\left[1-\frac{1}{\left\{1+\left(\frac{457}{h}\right)^2\right\}^{\frac{1}{3}}}\right]$

B = $2*(1-1/ (1+ (457/150)^2)^{1/3})$

B = 1.08

k = 0.011 (by Simiu Scalan)

r = is roughness factor,

r = $4.0\sqrt{k}\left(\frac{10}{h}\right)^{\alpha}$

r = $4*(0.011)^{0.5}(10/150)^{0.18}$

r = 0.258

F = $\frac{C_d P_h b h}{2(1+\alpha)}$

F = (1.2*878.375*87*150)/ (2(1+0.18))

F = 5827975.2 N

$\bar{X}$ = The mean response of the first mode at the building height.

$\bar{X}$ = $\frac{Mean\ Generalised\ force\ in\ first\ mode}{Generalised\ stiffness\ in\ first\ mode}$

$\bar{X}$ = $\frac{F}{K_o}$

$\bar{X}$ = 5827975.2/51513283

$\bar{X}$ = 0.113

G = $1+g_f r\sqrt{(B+R)}$

G = $1+3.78*0.258*(1.08+0.402)^{1/2}$

G = 2.186

**Calculation for Force, (F)**

Ae = effective frontal area at ht 'z'

= width*interval

= 87*10

= 870 $m^2$

Ae = effective frontal area at ht. z for top level

= width*interval/2

= 87*5

= 435 $m^2$

$F_h$ = $G A_e C_f P_z$

$F_h$ = 2.186*435*1.2*878.375/1000

$F_h$ = 1002.4 KN

**Table 4.1 Analytical Response of the Signature building and showing the storey wise Lateral Forces**

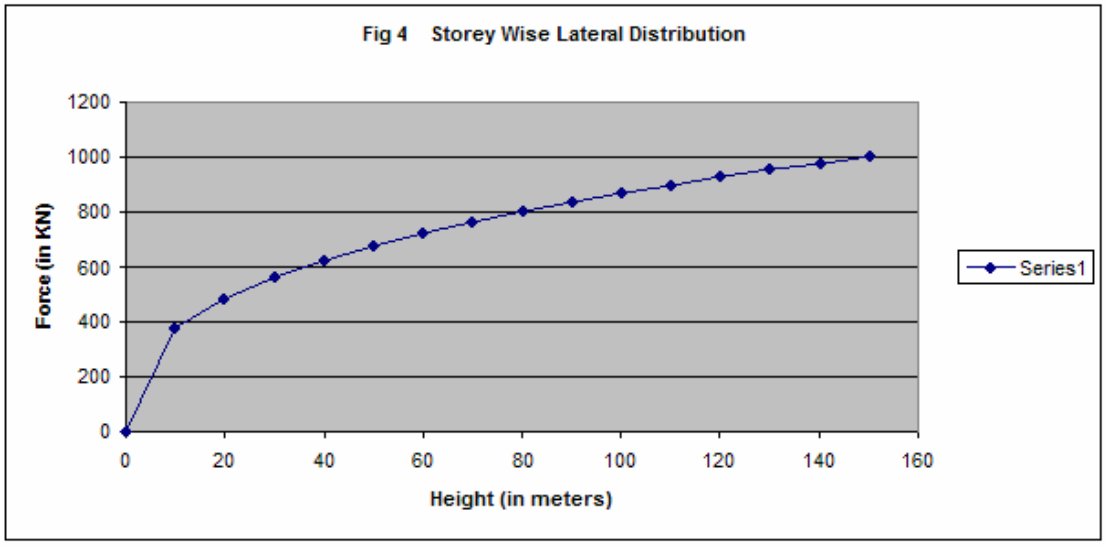
**ANALYSIS OF 150M TALL SIGNATURE BUILDING**

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| α | 0.18 | | **Building Parameters** | | | | | **G, Gust Factor Calculations** | | |
| Zref | 10 | m | Ae=effective frontal area at ht 'z' | | | 870 | m2 | T sec. | 3600 | sec |
| Plan Length | 87 | m | Ae=effective frontal area at ht. z f( | | | 435 | m2 | Bldg Nat frequency n1 | 0.196 | Hz |
| Plan breadth | 71 | m | Cf= from graph | | | 1.2 | | gf= | 3.781 | |
| Story height | 3 | m | G=gust factor | | | 2.19 | | Cd or Cf | 1.2 | |
| k1 | 1 | | ρa= | | | 1.2 | Kg/m3 | Ph | 878.29 | N/m2 |
| k3 | 1 | | Natural Period used | | | 5.10 | Sec | Ko= | 51513283 | N/m |
| Vb (fig.1) | 47 | m/s | Frequency used, Hz | | | 0.196 | Hz | F= | 5827975.2 | N |
| Density= | 110 | Kg/m3 | Critical damping Coeff, ξ = | | | 0.035 | | X mean= | 0.113 | m |
| Height of Building, h | 150 | m | bldg mass/m height, mo= | | | 679470 | Kg/m | k=surface drag coeffic | 0.011 | |
| face width | 87 | m | Generalised mass M1*=M0 | | | 33973500 | Kg | r= Roughness factor | 0.258 | |
| depth | 71 | m | Natural period of building | | | Secs | Frequency Hz | B= | 1.080 | |
| Interval | 10 | m | Emperical @ weaker axis | | | 2.83 | 0.353 | Red freq n' = | 0.769 | |
| Vz at top= | 38.26 | m/s | Emperical @ stronger axis | | | 2.21 | 0.453 | S= | 0.063 | |
| Pz at top= | 878.29 | N/m2 | IS875 pt3 1987: RC frame only | | | 5.00 | 0.200 | Lh from IS875 pt3 Fig.8 | 1800 | |
| k2 at 10m Table 33 | 0.5 | | IS875pt3:SMRF or Wall (LAB) | | | 1.45 | 0.691 | XE, for GER | 9.225 | |
| Vref at 10m | 23.5 | m/s | IS875pt3:SMRF or Wall (SAB) | | | 1.60 | 0.624 | E=gust energy ratio | 0.224 | |
| | | | Software modelling(LAB) | | | 4.40 | 0.227 | R= | 0.402 | |
| | | | Software modelling(SAB) | | | 5.10 | 0.196 | G=Gust factor | 2.186 | |
| height | K2 from table 33 | Vz(k2) | Vz(α) | Z/Zref | V/Vref | Pz(N/m2) | Fz(kN) | **Acceleration at top in alongwind** | | |
| 0 | 0 | 0.0 | 0 | 0.000 | 0.000 | 0 | 0 | 0.053903558 | m/s2 | |
| 10 | 0.5 | 23.5 | 23.5 | 0.077 | 0.630 | 331.35 | 378.1 | 0.005494756 | g | |
| 20 | 0.59 | 27.7 | 26.6 | 0.154 | 0.714 | 425.26 | 485.3 | 0.549475619 | %g | |
| 30 | 0.64 | 30.1 | 28.6 | 0.231 | 0.768 | 492.10 | 561.6 | **BASE BENDING MOMENT** | | |
| 40 | 0.67 | 31.5 | 30.2 | 0.308 | 0.809 | 545.79 | 622.9 | Base Moment = | 972068 | kN-m |
| 50 | 0.7 | 32.9 | 31.4 | 0.385 | 0.842 | 591.45 | 675.0 | | 972.068 | MN-m |
| 60 | 0.718 | 33.7 | 32.4 | 0.462 | 0.870 | 631.57 | 720.8 | | | |
| 70 | 0.736 | 34.6 | 33.4 | 0.538 | 0.895 | 667.61 | 761.9 | **IMPORTANT** | | |
| 80 | 0.754 | 35.4 | 34.2 | 0.615 | 0.916 | 700.49 | 799.4 | | | |
| 90 | 0.772 | 36.3 | 34.9 | 0.692 | 0.936 | 730.83 | 834.0 | Enter data here only | | |
| 100 | 0.79 | 37.1 | 35.6 | 0.769 | 0.954 | 759.08 | 866.3 | | | |
| 110 | 0.8 | 37.6 | 36.2 | 0.846 | 0.970 | 785.58 | 896.5 | Read Results here | | |
| 120 | 0.81 | 38.1 | 36.8 | 0.923 | 0.986 | 810.57 | 925.0 | | | |
| 130 | 0.82 | 38.5 | 37.3 | 1.000 | 1.000 | 834.27 | 952.1 | Do not alter these | | |
| 140 | 0.83 | 39.01 | 37.78947938 | 1.1 | 1.0134 | 856.8269 | 977.8 | | | |
| 150 | 0.84 | 39.5 | 38.26170235 | 1.2 | 1.0261 | 878.3747 | 1002.4 | | | |
| | | | | | | Sum | 11459 | kN | | |

### 4.1.3 Result Discussion for Analytical Analysis of Signature Towers

The analytical response of the Signature Towers is determined. In the analytical response by using the Davenport Gust Factor Approach given in the code IS 875: part 3-1987 and with the data provided the Base Forces and Base Moments are determined by examining the Storey wise Lateral forces. The calculation's for the topmost height (150m) is being shown and on the similar grounds and by using the same methods and formulas the forces and moments are determined for every height with a 10m interval height.

It is seen from the **Table 4.1** that with the increase in the height of the building the Storey wise Lateral Forces are also increasing as it can be seen in the graph shown in **Fig 4** in which from 10m height there is a linear increment in the Force value for Storey wise Lateral Forces.



## 4.2 Dynamic Analysis of Tall Building by *Balendra,T.*

### 4.2.1 General

1) Natural frequency in X & Y direction is obtained from resonant peaks in power spectra. More than one frequency can be obtained. If a particular frequency is common in both X & Y direction, it will be torsional mode.

**2) Estimate natural frequency from Empirical formulae**

Proto type Field values

| | | | |
|---|---|---|---|
| UBC (1997) | T | = | $0.0731H^{0.75}$ |
| Ellis (1998) | T | = | H/46 |
| Lagomarsino (1998) | T | = | H/55 |
| Ono et al. (1998) | T | = | 0.0149H |
| Hong & Hwang (2000) | T | = | H/77 |

**Suggested Values**

| | | | |
|---|---|---|---|
| (For weaker axis) | T | = | H/53 |
| (For stronger axis) | T | = | H/68 |

**Emprical forecast model for fundamental torsional period**

| | | | |
|---|---|---|---|
| Ellis (1998) | T | = | H/72 |
| Lagomarsino (1998) | T | = | H/78 |

3) **Estimate structural damping**

| | | |
|---|---|---|
| Translational modes | 8%-18% | $1^{st}$ & $2^{nd}$ mode |
| Torsional mode | 10 %– 22% | $1^{st}$torsional |

### 4.2.2 Principles of High Frequency force balance

The fundamental principle of the force balance technique is that the modal forces due to wind loads can be estimated from the measured base moments experienced by a rigid model. This principle stands as tall buildings have a fundamental mode shape which is approximately a straight line. The rigid model is mounted on a highly sensitive and stiff force balance sensor that measures the base overturning and torsional moments.

### 4.2.3 The governing equation of motion of the structure is expressed as

$$M(z)\ddot{x}(z,t) + C(z)\dot{x}(z,t) + K(z)x(z,t) = F(z,t)$$

Where M (z), C (z) and K (z) are the mass, damping coefficient, stiffness per unit height of the system respectively. If the displacement, x(t) can be expressed in model coordinate q(t) as x(z,t)=ϕ(z)q(t), where ϕ(z) is the mode shape. Then above eq. can be written as

$$M_r^* \ddot{q}_r + C_r^* \dot{q}_r + k_r^* q_r = F_r^*(t)$$

r=1, 2, 3, 4…….n

Since the mode of vibration is assumed to be liner, ϕ1=z/H, for constant mass per unit height, $m_o$ while for the first fundamental mode shape:

$$M_1^* = \int_0^H m_0 \phi_1^2 dz$$

$$M_1^* = \int_0^H m_0 (\frac{z}{H})^2 dz$$

$$M_1^* = \frac{1}{3} m_0 H$$

$$K_1^* = w_1^2 M_1^*$$

$$K_1^* = (2\pi n_1)^2 M_1^*$$

In which $m_o$ is mass per unit height and $n_1$ is fundamental sway frequency of the building. $F_1^*(t)$ Can be obtained from the measured base overturning moments, M (t) using the following relationship:

$$F_1^*(t) = \int_0^H F(t)\phi_1(z)dz$$

$$F_1^*(t) = \frac{1}{H}\int_0^H F(t)\phi_1(z)dz$$

$$F_1^*(t) = \frac{M(t)}{H}$$

Where H is the total height of the building.

Using random vibration theory, the power spectrum of the response can be expressed as

$$S_x = \phi_1^2(z).\frac{1}{K_1^{*2}}\left|H_1(n)\right|^2.S_{F_1}(n)$$

Where $S_{F_1^*}$ is generalized wind force spectrum which is equivalent to measured moment spectrum divided by the square of the building's height or in other words, the power spectrum of the measured base moment $S_M$ is proportional to the power spectrum of the generalized force $S_{F_1^*}$ for a linear mode. Since the base overturning moment generated by $F_x$ is denoted by $M_y$, and that generated by $F_y$ is denoted by $M_x$, thus:

$$S_{F_1^* x} = \frac{S_{My}}{H^2} and S_{F_1^* y} = \frac{S_{Mx}}{H^2}$$

and $\left|H_1(n)\right|$ is the mechanical admittance function expressed as

$$\left|H_1(n)\right| = \frac{1}{\left[\left\{1-(\frac{n}{n_1})^2\right\}^2 + 4\varsigma_1^2(\frac{n}{n_1})^2\right]^{1/2}}$$

The Variance or mean square of displacement is obtained from

$$\sigma_x^2 = \int_0^\infty S_x(n)dn$$

$$\sigma_x^2 = (\frac{z}{H})^2\frac{1}{K_1^{*2}}\int_0^\infty \left|H_1(n)\right|^2 S_{F_1^*}(n)dn$$

Since the linear mode shape is assumed, only the tip displacement is required to be computed from the power spectral density of modal force. Therefore the variance of tip displacement can be taken as

$$\sigma_x^2 = \frac{1}{K_1^{*2}} \int_0^{\infty} \left|H_1(n)\right|^2 S_{F_1^*}(n) dn$$

$$\frac{1}{K_1^{*2}} \int_0^{n_1 - \Delta n} S_{F_1^*}(n) dn + \frac{\Pi n_1 S_{F_1^*}(n)}{4\varsigma_1 K_1^{*2}}$$

$$\sigma_x^2 = \frac{\sigma_{F_1^*}^2}{K_1^{*2}} + \frac{\Pi n_1 S_{F_1^*}(n)}{4\varsigma_1 K_1^{*2}}$$

While RMS acceleration $\ddot{\sigma}_X$ as

$$\ddot{\sigma}_x = (2\Pi n_1)^2 \sigma_x$$

Once the tip RMS acceleration $\ddot{\sigma}_D$ is found, the wind- induced forces can be calculated from the base overturning moment power spectra measured from the force balance test. The peak values of the wind- induced loads in X and Y directions are obtained from

$$F_{x,\max}(z) = \bar{F}_x(z) + \left[\left(g_B \sigma_{B,Fx}(z)\right)^2 + \left(g_{Dx} \sigma_{D,Fx}(z)\right)^2\right]^{\frac{1}{2}}$$

$$F_{y,\max}(z) = \bar{F}_y(z) + \left[\left(g_B \sigma_{B,Fy}(z)\right)^2 + \left(g_{Dy} \sigma_{D,Fy}(z)\right)^2\right]^{\frac{1}{2}}$$

Where,

$\overline{F_x}(z)$ is the mean component of the wind load;

$g_B$ is the peak factor of background component, taken as 3.5

$\sigma_{B,F}$ is the non-resonating component or background force

$g_D = \sqrt{2\log_e} vT + \frac{0.577}{\sqrt{2\log_e} vT}$, the resonant peak factor, $n_1$ is the fundamental frequency

$\sigma_{D,F}$ is the resonating component or inertia force

The total force is sum of mean Background and resonant force as given below.

**$F_{max}(z)$ = mean+{(Background peak factor × Background force)+ (Resonant peak factor× resonant force)}**

### 4.2.4 For Resonant force Response

Since the mode of vibration is linear, the acceleration along x-axis and y-axis at any height z will be

$$\ddot{\sigma}_{Dx}(z) = \phi_x \ddot{\sigma}_{Dx}$$

$$\ddot{\sigma}_{Dy}(z) = \phi_x \ddot{\sigma}_{Dy}$$

The Inertial force $\sigma_{D,F}$ is calculated by multiplying the lumped mass at a particular floor and the corresponding acceleration as follows:

$$\sigma_{D.F_x}(z) = m_0 \Delta_z \ddot{\sigma}_{Dx}(z)$$

$$\sigma_{D.F_y}(z) = m_0 \Delta_z \ddot{\sigma}_{Dy}(z)$$

Where $\Delta_z$ is inter storey height,$m_0$ is mass per unit height

By multiplying the computed inertial force $\sigma_{D,F}(z)$ with the corresponding peak factor $g_D$, the distribution of peak inertial forces along the height of the building can be determined.

### 4.2.5 For Mean force

Since mean load is proportional to the square of mean wind velocity $\bar{U}(z)$, the mean force distribution can be determined by employing a generally accepted power law distribution as follows:

$$\overline{F_x}(z) = p_x(z)B_x\Delta z$$

$$\overline{F_y}(z) = p_y(z)B_y\Delta z$$

Where $B_x$, $B_y$ are the breadth of the building in X, Y direction, Δz is inter- story height, $p_x$ (z), $p_y$(z) are the mean pressure at height z in X and Y direction respectively.

The mean overturning moment coefficients are defined as the mean values of measured overturning moments $\bar{M}$ normalized by $\frac{1}{2}\rho_{air}\bar{U}_H^2 BH^2$. Where $\rho_{air}$ is the air density which is 1.2 Kg/m$^3$, $\bar{U}_H$ is the mean wind speed at the tip of the building, B and H are the breadth and height of the building respectively. Thus the mean overturning moment coefficients about X and Y axes are defined as:

$$C_{\bar{M}_x} = \frac{\overline{M}_x}{\frac{1}{2}\rho_{air}\overline{U}_H^2 BH^2}$$

$$C_{\bar{M}_y} = \frac{\overline{M}_y}{\frac{1}{2}\rho_{air}\overline{U}_H^2 BH^2}$$

So $\overline{M}_x$ $\overline{M}_y$ can be related to the mean pressure distribution as

$$\bar{M}_x = \frac{1}{2}\rho_{air}C_{\bar{M}_x}\bar{U}_H^2 BH^2$$

$$= \int_0^H BP_y(z)zdz$$

$$\bar{M}_y = \frac{1}{2}\rho_{air}C_{\bar{M}_y}\bar{U}_H^2 BH^2$$

$$= \int_0^H BP_x(z)zdz$$

Where $P_x$ (z) and $P_y$ (z) can be related to the mean pressure at the top of building, $P_{0x}$ and $P_{0y}$ as:

$$\frac{P_y(z)}{P_{0y}} = \left(\frac{z}{H}\right)^{2\alpha}$$

$$\frac{P_x(z)}{P_{0x}} = \left(\frac{z}{H}\right)^{2\alpha}$$

Where α is the power law exponential of urban terrain profile, taken as 0.14. Hence, the mean base overturning moment about the x and y-axes can be expressed as follows:

$$\bar{M}_x = P_{0y} B \int_0^H \left(\frac{z}{H}\right)^{2\alpha} z dz$$

$$\bar{M}_y = P_{0x} B \int_0^H \left(\frac{z}{H}\right)^{2\alpha} z dz$$

$$P_{0x} = (\alpha + 1)\rho_{air} C_{\bar{M}_y} \bar{U}_H^2$$

$$P_{0y} = (\alpha + 1)\rho_{air} C_{\bar{M}_x} \bar{U}_H^2$$

Hence the pressure at height, z of the building, $P_x$ (z) and $P_y$(z) can be easily determined as

$$P_x(z) = (1+\alpha)\rho_{air} . C\bar{m}_y . \bar{U}_H^2 (\frac{z}{H})^{2\alpha}$$

$$P_y(z) = (1+\alpha)\rho_{air} . C\bar{m}_x . \bar{U}_H^2 (\frac{z}{H})^{2\alpha}$$

### 4.2.6 For Non resonant Response

The same definitions apply for the non-resonant or background component of wind forces with the assumption that the background wind forces follow the mean force distribution, i.e. power law distribution. This assumption is based on the fact that fluctuating background wind forces are formed of long period waves which can be approximated as static forces. To determine the background wind pressure, the RMS

moment coefficients, $C_{\sigma Mx}$ and $C_{\sigma My}$ are employed to replace mean moment coefficients, $C_{\bar{M}x}$ and $C_{\bar{M}y}$

Thus, the non-resonant force can be calculated as follows:

$$\sigma_B . F_x(z) = p\sigma_x(z) B_x \Delta_z$$

$$\sigma_B . F_y(z) = p\sigma_y(z) B_y \Delta_z$$

Where $p\sigma_x(z) = (1+\alpha)\rho_{air} C_{\sigma Mx} \bar{U}_H^2 (\frac{z}{H})^{2\alpha}$

$$p\sigma_y(z) = (1+\alpha)\rho_{air} C_{\sigma My} \bar{U}_H^2 (\frac{z}{H})^{2\alpha}$$

$$C_{\sigma M_x} = \frac{\sigma_{Mx}}{\frac{1}{2}\rho_{air} \bar{U}_H^2 BH^2}$$

$$C_{\sigma M_y} = \frac{\sigma_{My}}{\frac{1}{2}\rho_{air} \bar{U}_H^2 BH^2}$$

$\sigma_{Mx}$ and $\sigma_{My}$ , are the RMS values of the measured overturning moments $M_x$ and $M_y$ respectively.

## 4.3 Dynamic analysis of SIGNATURE Building studied

### 4.3.1 Dynamic Behavior of *'Signature Building'* from Unitech Company

The experimental study on a 1:250 scale model of a 150m tall building, proposed for construction by UNITECH Company in Noida (Uttar Pradesh, INDIA), has been made in the Boundary Layer Wind Tunnel at the Indian Institute of Technology Roorkee.

**Fig 4.1: SIGNATURE TOWERS at Noida (Uttar Pradesh, INDIA)**

### 4.3.2 Following are salient data/ parameters for the building:

| | | | |
|---|---|---|---|
| Height of Building | = | 150 | m |
| Plan Length | = | 87 | m |
| Plan breadth | = | 71 | m |
| Plan aspect ratio | = | 1: 5.5 | |
| Story height | = | 3 | m |
| $V_b$ (From fig.1 IS875 pt3, Noida (India)) | = | 47 | m/sec |
| Bldg bulk Density | = | 110 | Kg/m3 |
| Natural Period used | = | 5.10 | Sec |
| Structural Damping | = | 3.5 | % |
| Terrain category | = | 3 | (*corresponding α = 0.18*) |

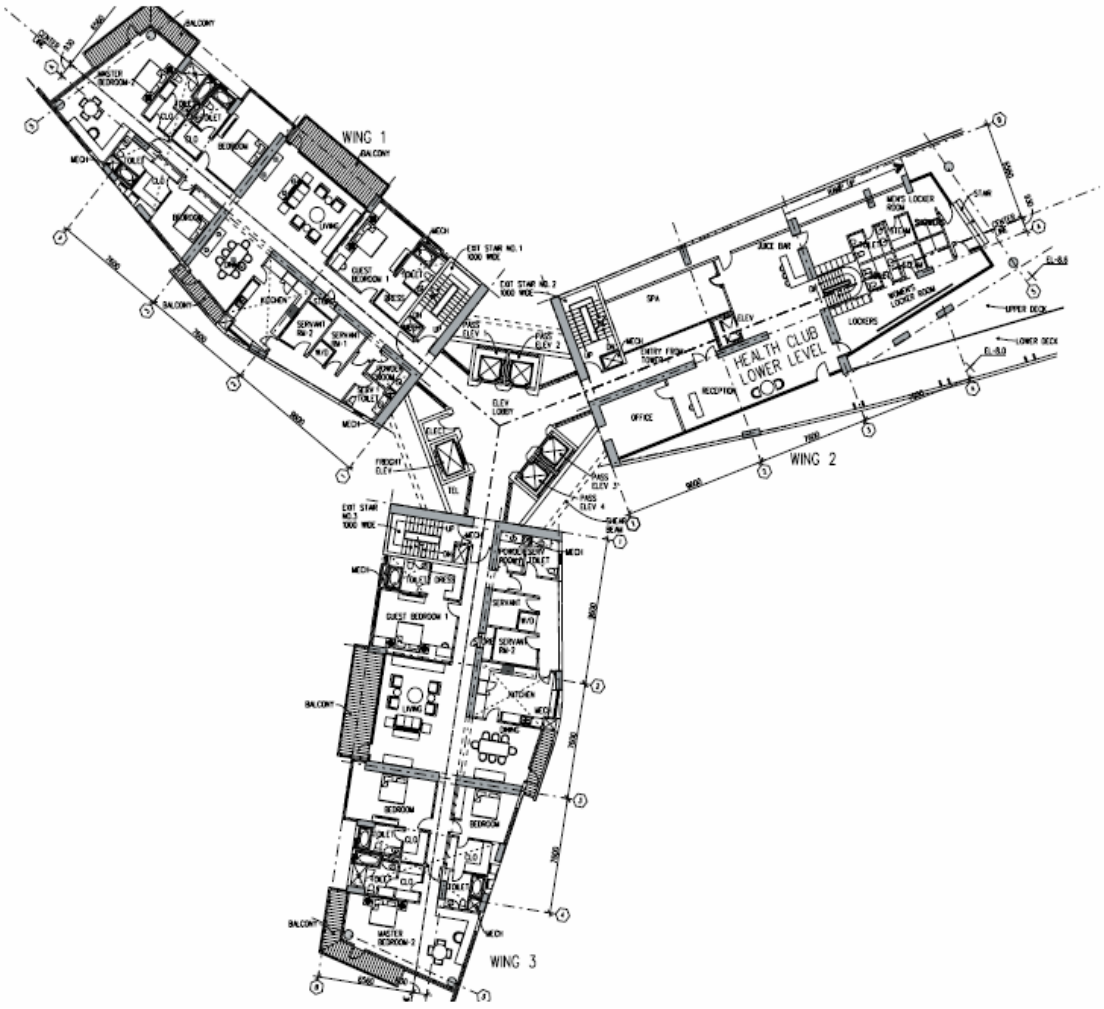



**Fig 4.2a FLOOR PLAN OF SIGNATURE TOWERS**

### 4.3.3 Test and Analysis Sequence

#### 4.3.3.1 Analytical Estimate of Dynamic Wind Force on Building

1 Theoretical calculations of Design wind velocity, pressures and forces along the height of building, as per IS-875 (Part -3).

2 Gust factor calculations as per IS-875 (Part -3) as per Davenport formulation.

3 Theoretical estimate of Max. Base shear and base moment in along-wind direction using gust factors.

4 Computation of RMS acceleration at top using code and analytical formulation.

5 The step by step procedure of calculating the Max. Story wise Lateral forces and Max. Base Moment is discussed.

#### 4.3.3.2 Dynamic Analysis of Building

##### 4.3.3.2.1 Flow and Structural Modeling

1. Establishing the Boundary Layer flow as per terrain category 3 ($\alpha = 0.18$) in Wind Tunnel using Vortex generators, Barrier wall and roughness blocks, as per IS: 875 (Part3) 1987.
2. Fabrication of Rigid Model of the Building as per periphery and significant opening and projections, for HFFB test at a geometric scale of 1:250. The model was sufficiently rigid to ensure that model resonance frequencies (adjusted to full scale) are sufficiently high.

##### 4.3.3.2.2 Wind Tunnel Measurements (Base Forces)

3. Conducting wind tunnel test for Acquiring time history of Base forces ($F_x$, $F_y$, $M_x$, $M_y$, and $M_z$), on HFFB ($0^0$ and $360^0$), in stand alone condition, at a wind speed of 10.78 m/s at model top.
4. 1000 Samples per second on each channel and a total of 8192 sample for each parameter ($F_x$, $F_y$, $M_x$, $M_y$, and $M_z$), were collected.

##### 4.3.3.2.3 Analysis of Acquired Data

5. Computing multiplying factors as per Velocity ratio ($V_r^2=3.536^2$) and length scale ratio ($L_r^{\,2}= 250^2$).

It is multiplied with the square of the Length scale and Velocity scale ratio's because

$$\text{Force} = \text{mass} * \text{acceleration}$$

$$\text{Force} = (\text{Volume} * \text{density}) * \text{acceleration}$$

$$F = L^3 * (L/T^2)$$

Or $$F = L^2 * (L^2/T^2)$$

$$\mathbf{F = L^2 * V^2}$$

6. Converting the Raw data obtained through HFFB model test for velocity and length scale ratio to Full scale values.
7. Computing statistical parameters of full scale base Forces and moments, obtained through HFFB test.
8. Ascertaining design base forces and moments as per HFFB test.
9. Obtain the spectra of along-wind moments in different orientations using FFT & tabulate the required quantities for dynamic analysis.
10. Compute the acceleration at top and dynamic lateral forces using Balendra's procedure, along the height.

#### 4.3.3.2.4 Effects of Angle of Wind Incidence

Variations base forces values with the changing of angle of wind incidence from $0^o$ to $360^o$ with an increment of $15^o$ in both Standalone and Interference conditions in the wind tunnel on the building model have been studied in this chapter and various positions of the building at different angles of wind incidence are shown below in the **Fig 4.2b**

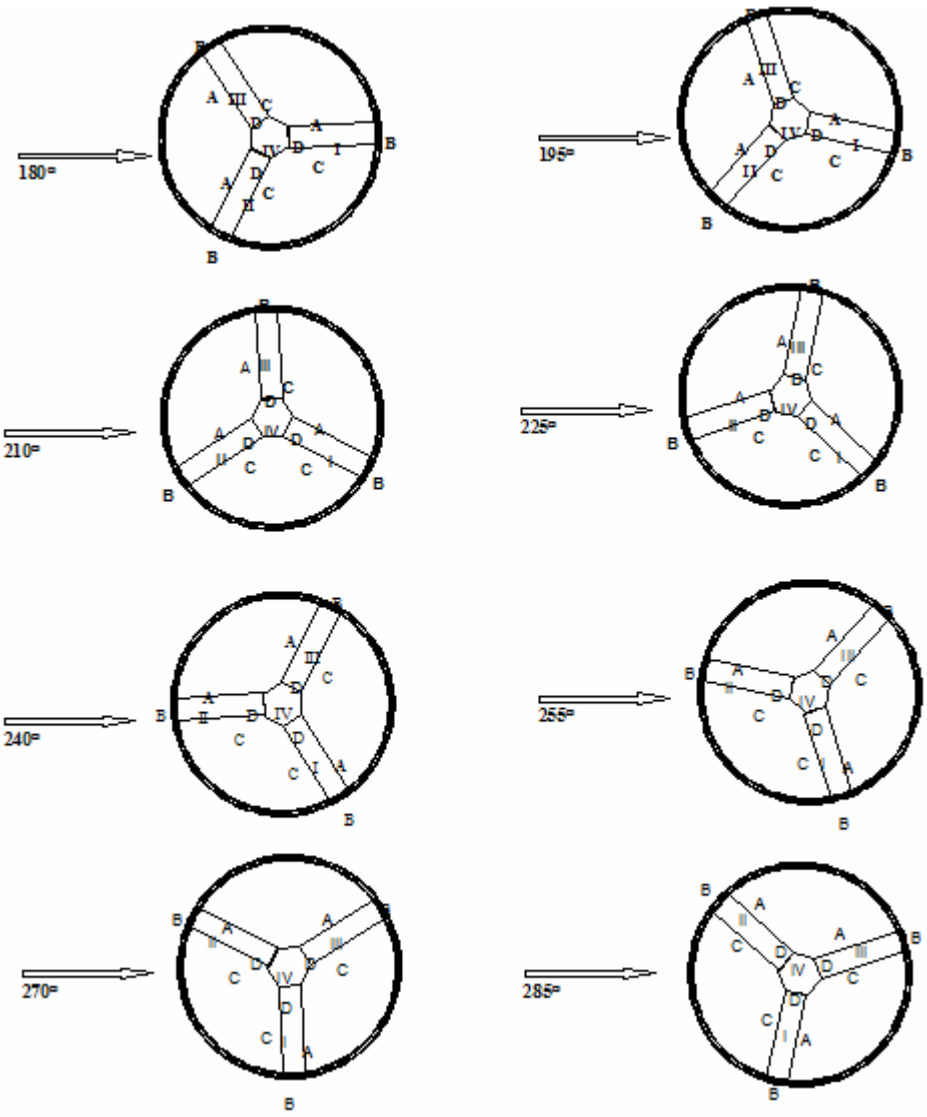



**Fig 4.2b Location of building at various angles on wind incidence**

### 4.3.4 Experimental Results

**Table 4.2** shows Typical Sample raw data for all five components i.e., $F_x$, $F_y$, $M_x$, $M_y$, and $M_z$ for various angles of Wind Incidence $0^o$ for both standalone and interference condition for the model of the building measured from the load cell when placed under the flow simulated conditions in wind tunnel.

**Table 4.3** shows the maximum values taken from the Raw data for all the five components for all the Angles of Wind Incidence from $0^o$ to $360^o$ and plotting those maximum values for all angles for each component and finding out the critical angles in both standalone and interference conditions (**as shown in the fig 4.3 to fig 4.6**) to find out **The Critical Angles**.

After these plots the critical angles worked out to be converted to full scale measurements are

**Standalone— $0^o$, $45^o$, $60^o$, $75^o$, $90^o$, $105^o$, $180^o$, $195^o$, $285^o$, $300^o$, $345^o$**

**Interference--- $0^o$, $45^o$, $60^o$, $75^o$, $90^o$, $105^o$, $180^o$, $195^o$, $285^o$, $300^o$, $345^o$**

**Table 4.4** shows Typical Processed data for all five components for building in full scale measurements, which is obtained by converting raw data to full scale model by using velocity and length scale correction for both standalone and interference. It is done by multiplying the raw data values of $F_{X,}$ $F_Y$ with the square of the length scale ratio and the square of the velocity ratio ($L_R^2 = 250^2$ and $V_R^2 = 3.536^2$)

**Table 4.5** shows the Typical Mean, Maximum, Minimum, Positive Peak Factor and Negative Peak Factor values obtained from the Full Scale Processed data sheets for the above mentioned critical angles for the Standalone condition. Similarly for interference condition is also obtained.

As per design concerns from these values for critical angles we again plotted the maximum values for all the five components to find out the **Major Critical Angles** almost common in both the conditions of standalone and interference to get the Major peaks for design as shown in **Table 4.6** and graphs which are the plots of those maximum values Full Scaled Processed Data of all five components for the above mentioned critical angles in figures from **Fig 4.7 to Fig 4.16**

Referring those graphs **(figures from Fig 4.7 to Fig 4.16)** the Major Critical Angles worked out for design are

**Standalone— $45^o$, $90^o$, $195^o$, $345^o$**

**Interference--- $45^o$, $90^o$, $195^o$, $345^o$**

**Table 4.7** shows parameters of measured response for dynamic analysis i.e., ($S_M$, Mean and RMS) for $M_x$ and $M_y$ for the above Major Critical Angles which are obtained by a program developed by IIT Roorkee to know the frequencies with their respective spectral moments.

**Table 4.8** shows the Dynamic Analysis for tip acceleration and lateral forces from wind tunnel test results for Signature building for the maximum Spectral Moment and corresponding Overturning moment of all the Major Critical Angles using the Balendra's approach.

After **Table 4.8** a reference calculation is shown for the topmost height of the building to determine the Dynamic Analysis for tip acceleration and Storey Wise lateral forces of the Signature Towers at a10m interval height

After **Table 4.9** shows the Storey Wise Lateral forces obtained by Balendra's Approach for the Dynamic Analysis in the wind tunnel testing for Signature building for maximum spectral moment of all the Major Critical Angles are summed up in.

**Table 4.2 Typical Sample Raw data for Signature Building At Angle of Wind Incidence = $0^o$**

| Sample no. | Fx (Kg) | Fy (Kg) | Mx (Kg-m) | My (Kg-m) | Mz (Kg-m) |
|---|---|---|---|---|---|
| | | | | | |
| 1 | -0.7818971 | 0.0882841 | -0.0270021 | -0.3171091 | 0.021824 |
| 2 | -0.7892212 | 0.0492212 | -0.0343262 | -0.3488482 | 0.007175 |
| 3 | -0.9064093 | 0.1004913 | -0.0660643 | -0.3610553 | -0.00259 |
| 4 | -0.8551394 | 0.0809594 | -0.0538574 | -0.3464064 | 0.002292 |
| 5 | -0.9723275 | 0.1200225 | -0.0147955 | -0.3049025 | -0.009915 |
| 6 | -0.7574836 | -0.0533186 | 0.0193856 | -0.2804886 | -0.00259 |
| 7 | -0.8136357 | -0.0411117 | 0.0682137 | -0.2536337 | -0.00259 |
| 8 | -0.7086558 | -0.1802718 | 0.0657718 | -0.2682818 | -0.007473 |
| 9 | -0.7403939 | -0.0972639 | 0.0218269 | -0.2878139 | 0.004734 |
| 10 | -0.7403931 | -0.0508761 | -0.0343261 | -0.3146681 | 0.0145 |
| 11 | -0.79898711 | 0.12978811 | -0.10268611 | -0.31222711 | 0.021824 |
| 12 | -0.78678012 | 0.20547112 | -0.15883812 | -0.26339812 | 0.031589 |
| 13 | -0.65738513 | 0.37881113 | -0.20522513 | -0.21212913 | 0.034031 |
| 14 | -0.62320614 | 0.38613514 | -0.20766614 | -0.18771514 | 0.036472 |
| 15 | -0.53043215 | 0.42275615 | -0.20522515 | -0.18039115 | 0.048679 |
| 16 | -0.56461216 | 0.38125216 | -0.18325216 | -0.19748016 | 0.038914 |
| 17 | -0.50113517 | 0.41299117 | -0.19545917 | -0.23898417 | 0.038914 |
| 18 | -0.63297118 | 0.38857718 | -0.21010718 | -0.27316418 | 0.043796 |
| 19 | -0.58902619 | 0.46181919 | -0.19790019 | -0.30002019 | 0.031589 |
| 20 | -0.7916632 | 0.3836942 | -0.1563962 | -0.2902542 | 0.019382 |
| 21 | -0.66470921 | 0.29336221 | -0.09780321 | -0.26339821 | 0.012058 |
| 22 | -0.74283422 | 0.12978822 | -0.03432622 | -0.22921922 | -0.00259 |
| 23 | -0.59635023 | 0.05166323 | 0.00717823 | -0.21701223 | -0.00259 |
| 24 | -0.66226824 | 0.03457324 | 0.00473624 | -0.22677724 | -0.014797 |
| 25 | -0.55484625 | 0.00771725 | -0.00258825 | -0.23898425 | -0.005032 |
| 8171 | -0.755041817 | 0.220120817 | -0.141748817 | -0.243867817 | 0.009617 |
| 8172 | -0.733069817 | 0.217678817 | -0.153955817 | -0.253633817 | 0.019382 |
| 8173 | -0.772131817 | 0.276272817 | -0.073389817 | -0.260957817 | 0.009617 |
| 8174 | -0.737952817 | -0.102146817 | 0.068213817 | -0.285371817 | -0.014797 |
| 8175 | -0.813635818 | -0.224216818 | 0.160986818 | -0.307344818 | -0.012356 |
| 8176 | -0.803870818 | -0.378025818 | 0.139014818 | -0.314668818 | -0.000149 |
| 8177 | -0.830725818 | -0.265720818 | 0.038916818 | -0.317109818 | -0.012356 |
| 8178 | -0.886877818 | 0.029690818 | -0.075830818 | -0.336641818 | -0.017239 |
| 8179 | -0.891760818 | 0.173733818 | -0.114893818 | -0.329316818 | 0.002292 |
| 8180 | -0.899084818 | 0.178616818 | -0.056299818 | -0.331758818 | -0.00259 |
| 8181 | -0.891760818 | -0.038669818 | 0.053564818 | -0.319551818 | -0.014797 |
| 8182 | -0.911291818 | -0.316990818 | 0.146338818 | -0.351289818 | 0.007175 |
| 8183 | -0.916174818 | -0.295017818 | 0.119482818 | -0.361055818 | 0.002292 |
| 8184 | -0.991858818 | -0.194919818 | -0.012354818 | -0.356172818 | 0.0145 |
| 8185 | -0.947913819 | 0.237209819 | -0.161279819 | -0.326875819 | 0.024265 |
| 8186 | -0.933264819 | 0.371487819 | -0.224756819 | -0.300020819 | 0.0145 |
| 8187 | -0.847815819 | 0.427639819 | -0.149072819 | -0.251191819 | 0.016941 |
| 8188 | -0.791663819 | 0.012600819 | 0.012061819 | -0.243867819 | -0.017239 |
| 8189 | -0.620764819 | -0.246189819 | 0.180518819 | -0.246309819 | -0.014797 |

| | | | | | |
|---|---|---|---|---|---|
| 8190 | -0.764807819 | -0.597751819 | 0.251318819 | -0.304902819 | -0.000149 |
| 8191 | -0.701331819 | -0.434177819 | 0.151221819 | -0.351289819 | 0.007175 |
| 8192 | -0.986975819 | -0.158298819 | -0.029443819 | -0.361055819 | -0.014797 |

## The Critical Angles

**Table 4.3 Maximum Values of all Five Components for all Angles of Wind Incidence**

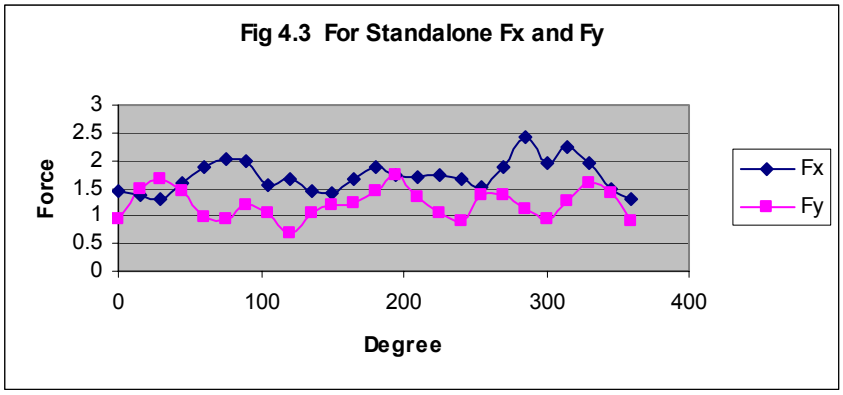
**Building in STAND ALONE Condition**

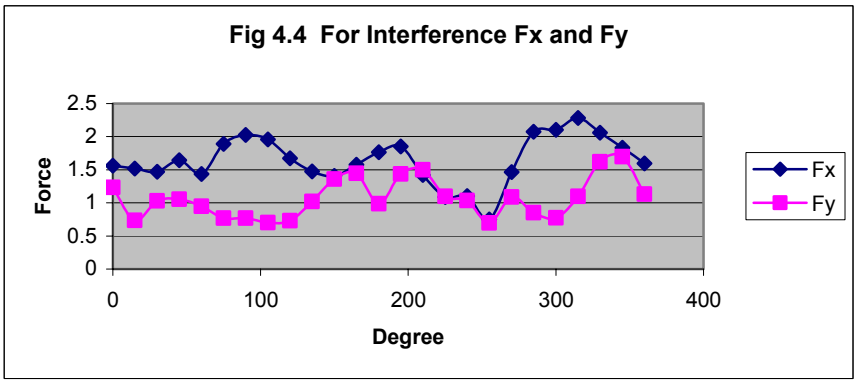
| degree | Fx (Kg) | Fy (Kg) | Mx (Kg-m) | My (Kg-m) | Mz (Kg-m) |
|---|---|---|---|---|---|
| 0 | 1.4361 | 0.942 | 0.424 | 0.536 | 0.082 |
| 15 | 1.367 | 1.493 | 0.619 | 0.522 | 0.092 |
| 30 | 1.311 | 1.669 | 0.695 | 0.544 | 0.112 |
| 45 | 1.601 | 1.457 | 0.629 | 0.659 | 0.105 |
| 60 | 1.862 | 0.988 | 0.451 | 0.791 | 0.097 |
| 75 | 2.016 | 0.931 | 0.276 | 0.778 | 0.088 |
| 90 | 2.001 | 1.202 | 0.388 | 0.766 | 0.057 |
| 105 | 1.567 | 1.047 | 0.419 | 0.656 | 0.053 |
| 120 | 1.652 | 0.686 | 0.278 | 0.671 | 0.056 |
| 135 | 1.455 | 1.059 | 0.39 | 0.583 | 0.073 |
| 150 | 1.396 | 1.198 | 0.449 | 0.563 | 0.051 |
| 165 | 1.645 | 1.223 | 0.473 | 0.617 | 0.07 |
| 180 | 1.877 | 1.452 | 0.529 | 0.659 | 0.07 |
| 195 | 1.726 | 1.726 | 0.612 | 0.632 | 0.078 |
| 210 | 1.713 | 1.352 | 0.517 | 0.634 | 0.087 |
| 225 | 1.723 | 1.047 | 0.366 | 0.622 | 0.095 |
| 240 | 1.679 | 0.905 | 0.358 | 0.62 | 0.058 |
| 255 | 1.508 | 1.367 | 0.559 | 0.671 | 0.07 |
| 270 | 1.879 | 1.381 | 0.546 | 0.788 | 0.073 |
| 285 | 2.404 | 1.118 | 0.38 | 0.9 | 0.058 |
| 300 | 1.953 | 0.947 | 0.302 | 0.754 | 0.08 |
| 315 | 2.248 | 1.281 | 0.429 | 0.837 | 0.107 |
| 330 | 1.962 | 1.591 | 0.595 | 0.727 | 0.117 |
| 345 | 1.464 | 1.425 | 0.639 | 0.6 | 0.119 |
| 360 | 1.296 | 0.898 | 0.407 | 0.495 | 0.075 |

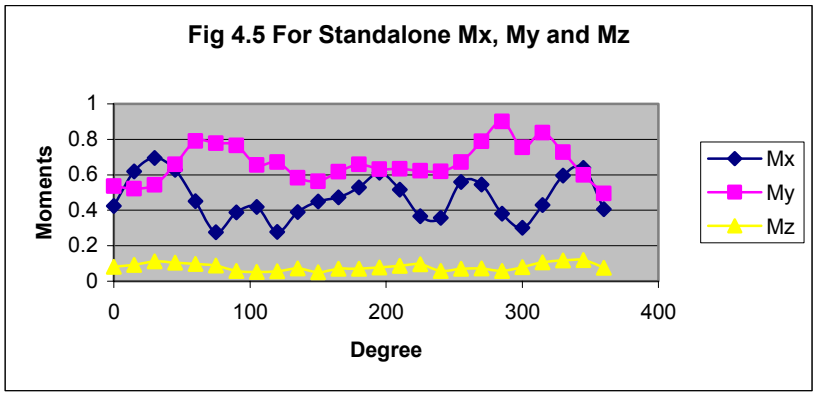
**Building in INTERFERENCE Condition**

| degree | Fx (Kg) | Fy (Kg) | Mx (Kg-m) | My (Kg-m) | Mz (Kg-m) |
|---|---|---|---|---|---|
| 0 | 1.56 | 1.232 | 0.519 | 0.603 | 0.093 |
| 15 | 1.52 | 0.737 | 0.322 | 0.556 | 0.102 |
| 30 | 1.469 | 1.028 | 0.52 | 0.602 | 0.122 |
| 45 | 1.643 | 1.054 | 0.532 | 0.649 | 0.141 |
| 60 | 1.435 | 0.947 | 0.451 | 0.61 | 0.136 |
| 75 | 1.889 | 0.771 | 0.258 | 0.725 | 0.078 |
| 90 | 2.026 | 0.771 | 0.266 | 0.766 | 0.046 |
| 105 | 1.958 | 0.703 | 0.268 | 0.764 | 0.065 |
| 120 | 1.674 | 0.732 | 0.268 | 0.701 | 0.054 |
| 135 | 1.474 | 1.022 | 0.383 | 0.617 | 0.073 |
| 150 | 1.408 | 1.357 | 0.498 | 0.598 | 0.065 |

| | | | | | |
|---|---|---|---|---|---|
| 165 | 1.577 | 1.447 | 0.546 | 0.634 | 0.058 |
| 180 | 1.767 | 0.986 | 0.434 | 0.61 | 0.068 |
| 195 | 1.853 | 1.437 | 0.544 | 0.632 | 0.073 |
| 210 | 1.423 | 1.501 | 0.588 | 0.556 | 0.063 |
| 225 | 1.089 | 1.099 | 0.419 | 0.448 | 0.091 |
| 240 | 1.103 | 1.037 | 0.402 | 0.468 | 0.117 |
| 255 | 0.749 | 0.695 | 0.261 | 0.292 | 0.063 |
| 270 | 1.464 | 1.086 | 0.4 | 0.639 | 0.078 |
| 285 | 2.075 | 0.849 | 0.339 | 0.734 | 0.085 |
| 300 | 2.104 | 0.773 | 0.27 | 0.773 | 0.065 |
| 315 | 2.282 | 1.096 | 0.397 | 0.834 | 0.092 |
| 330 | 2.06 | 1.618 | 0.598 | 0.732 | 0.1 |
| 345 | 1.833 | 1.696 | 0.703 | 0.72 | 0.117 |
| 360 | 1.596 | 1.13 | 0.478 | 0.588 | 0.08 |





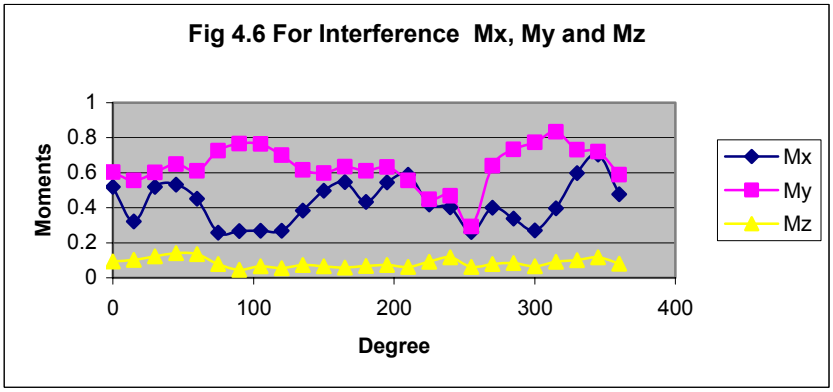




**Table 4.4 Full Scale Processed data--Signature Building**
**Angle of Wind Incidence = $0^o$**
***(Velocity ratio (Vr)= 3.536 and Length Scale ratio (Lr) = 250)***

| Time | FX (Kg) | FY (Kg) | MX (Kg-m) | MY (Kg-m) | MZ (Kg-m) |
|---|---|---|---|---|---|
| 0.005 | -611018.1802 | 68990.13965 | -6430823.10 | -51717147.69 | 4263623.936 |
| 0.01 | -616741.6421 | 38464.20207 | -7350379.12 | -57821957.24 | 1401736.7 |
| 0.015 | -708318.9859 | 78529.52933 | -14221955.52 | -58672864.61 | -505992.76 |
| 0.02 | -668253.815 | 63266.20889 | -11581506.09 | -56482088.53 | 447774.288 |
| 0.025 | -759831.1588 | 93792.30276 | -4461529.13 | -46839800.1 | -1937034.06 |
| 0.03 | -591940.1041 | -41666.13988 | 4485156.20 | -44882378.11 | -505992.76 |
| 0.035 | -635820.4996 | -32126.98464 | 13864628.28 | -38900900.8 | -505992.76 |
| 0.04 | -553783.3268 | -140874.4797 | 15209089.47 | -43136734.85 | -1459955.172 |
| 0.045 | -578585.2555 | -76007.45824 | 5537315.42 | -46537171.73 | 924853.176 |
| 0.05 | -578584.6304 | -39757.4336 | -6040147.19 | -51783526.13 | 2832778 |
| 0.055 | -624373.271 | 101423.6973 | -21760016.12 | -50539684.83 | 4263623.936 |
| 0.06 | -614834.0455 | 160566.6396 | -33720741.69 | -41160040.05 | 6171353.396 |
| 0.065 | -513717.5541 | 296024.2304 | -45052008.16 | -32837626.32 | 6648432.284 |
| 0.07 | -487008.1773 | 301747.622 | -45624760.44 | -28515393.64 | 7125315.808 |
| 0.075 | -414509.3862 | 330365.33 | -45627225.48 | -28298904.41 | 9510124.156 |
| 0.08 | -441219.5601 | 297931.7879 | -40791232.43 | -31190086.35 | 7602394.696 |
| 0.085 | -391615.0854 | 322734.4277 | -43591486.95 | -40129350.71 | 7602394.696 |
| 0.09 | -494639.1264 | 303655.9688 | -46133616.59 | -45081241.49 | 8556161.744 |
| 0.095 | -460298.0503 | 360891.3769 | -44707503.28 | -50903152.06 | 6171353.396 |

| | | | | | |
|---|---|---|---|---|---|
| 0.1 | -618649.9576 | 299840.1348 | -35576509.47 | -46342834.74 | 3786545.048 |
| 0.105 | -519441.0004 | 229249.6592 | -22947158.11 | -42757891.14 | 2355699.112 |
| 0.11 | -580492.2582 | 101423.7832 | -8404956.01 | -35057938.37 | -505992.76 |
| 0.115 | -466021.4653 | 40372.54106 | 726127.66 | -34590517.76 | -505992.76 |
| 0.12 | -517533.4898 | 27017.46584 | 472748.24 | -35635422.76 | -2890801.108 |
| 0.125 | -433587.9311 | 6030.691316 | -606664.95 | -39426321.17 | -983071.648 |
| 0.13 | -548059.5211 | -32126.64079 | 32468.41 | -43232503.1 | -29109.236 |
| 0.135 | -469837.3462 | -16864.03147 | 2638810.48 | -46927534.83 | -983071.648 |
| 0.14 | -616741.7046 | -96993.75607 | 11612293.89 | -43512926.37 | -1937034.06 |
| 0.145 | -553782.9283 | -184754.3985 | 28345099.10 | -41228912.43 | -4321842.408 |
| 0.15 | -616741.7202 | -301133.5226 | 35541146.97 | -37789546.27 | -3367879.996 |
| 0.155 | -534704.4771 | -280147.5295 | 33758592.24 | -39163672.05 | -5752688.344 |
| 0.16 | -584308.186 | -287778.4552 | 19100678.58 | -38809695.4 | -3367879.996 |
| 40.805 | -422535.5248 | -226727.5929 | 31909944.76 | -32677969.22 | -1937034.06 |
| 40.81 | -332865.2367 | -228635.9085 | 13817404.70 | -32271999.33 | -3844763.52 |
| 40.815 | -393683.1853 | 46096.38297 | -16540688.42 | -35542124.08 | -983071.648 |
| 40.82 | -389746.0835 | 183461.5002 | -37443137.42 | -38944810.34 | 4263623.936 |
| 40.825 | -464030.4257 | 273130.4504 | -37037167.53 | -39129350.22 | 4263623.936 |
| 40.83 | -397221.7295 | 143397.0327 | -9585203.38 | -44540558.52 | -29109.236 |
| 40.835 | -521127.0469 | -55021.39226 | 16632940.75 | -43417157.52 | 447774.288 |
| 40.84 | -481077.9177 | -262976.9929 | 32994201.24 | -42180507.91 | -983071.648 |
| 40.845 | -523098.0909 | -251530.2255 | 24217196.92 | -39569538.99 | 1878815.588 |
| 40.85 | -444914.1412 | -123703.5603 | -1772497.77 | -39925798.15 | 1401736.7 |
| 40.855 | -488837.3535 | 172014.7332 | -30573862.69 | -37759956.92 | 1878815.588 |
| 40.86 | -471647.8325 | 170106.4178 | -32926706.77 | -39955481.8 | 3786545.048 |
| 40.865 | -502198.3474 | 215895.0507 | -17953970.37 | -40875030.08 | 1878815.588 |
| 40.87 | -475477.6978 | -79823.24334 | 14663563.55 | -46092028.98 | -2890801.108 |
| 40.875 | -534664.072 | -175215.5773 | 34385889.53 | -49394118.02 | -2413917.584 |
| 40.88 | -527035.4031 | -295410.5433 | 32106617.43 | -50952781.85 | -29109.236 |
| 40.885 | -548082.2849 | -207649.1273 | 11081068.06 | -51078149.88 | -2413917.584 |
| 40.89 | -591992.389 | 23202.06771 | -15203246.52 | -54159004.23 | -3367879.996 |
| 40.895 | -595852.8691 | 135765.3344 | -24720185.19 | -52664047.46 | 447774.288 |
| 40.9 | -601569.2818 | 139581.1841 | -13336942.48 | -53045259.69 | -505992.76 |
| 40.905 | -595879.2188 | -30218.76137 | 10970801.38 | -50756318.04 | -2890801.108 |
| 40.91 | -611137.4198 | -247714.3768 | 32738552.69 | -56701131.85 | 1401736.7 |
| 40.915 | -614968.5901 | -230543.4442 | 27204244.01 | -58545141.21 | 447774.288 |
| 40.92 | -674095.743 | -152321.2616 | 137694.39 | -56600521.58 | 2832778 |
| 40.925 | -639717.8083 | 185369.0359 | -34613201.81 | -51452155.64 | 4740507.46 |
| 40.93 | -628234.8447 | 290301.3848 | -48771939.30 | -46397401.89 | 2832778 |
| 40.935 | -561455.2254 | 334181.7022 | -34721005.66 | -37976448.27 | 3309661.524 |
| 40.94 | -517584.439 | 9846.985456 | 2191508.16 | -37280597.66 | -3367879.996 |
| 40.945 | -384119.7083 | -192386.5111 | 38489352.60 | -39994639.89 | -2890801.108 |
| 40.95 | -496595.1316 | -467116.7455 | 56922855.24 | -49556168.04 | -29109.236 |
| 40.955 | -447142.965 | -339290.8618 | 35226421.40 | -59449379.92 | 1401736.7 |
| 40.96 | -670352.3312 | -123703.5621 | -3680227.63 | -57618399.62 | -2890801.108 |

**Table 4.5 Processed Full Scale Data for Critical Angles in case of Standalone**

| 0 degree | FX | FY | MX | My | Mz |
|---|---|---|---|---|---|
| Mean | -551733.3249 | 86569.81967 | -13171459.29 | -48049536.06 | 2404513.235 |
| Max | 100226.1225 | 736738.1636 | 68342661.4 | -7004932.639 | 16187665.68 |
| Minm | -1122322.622 | -522444.1191 | -93698792.79 | -88540222.88 | -10045226.15 |
| RMS | 112841.5538 | 168802.7731 | 20994423.74 | 9522978.599 | 3324831.592 |
| Ppf | 5.778 | 3.852 | 3.883 | 4.310 | 4.146 |
| Npf | -5.057 | -3.608 | -3.836 | -4.252 | -3.744 |
| 45 degree | | | | | |
| Mean | -590497.3971 | -461953.8301 | 56620633.92 | -58978226.81 | -7679551.051 |
| Max | -148631.8792 | 160259.7519 | 142134613.7 | -19426178.59 | 2861691.872 |
| Minm | -1179052.662 | -1138988.148 | -12059141.04 | -107816647.2 | -20509312.72 |
| RMS | 137827.2123 | 178117.752 | 20537983.75 | 11812317.57 | 2865825.542 |
| Ppf | 3.206 | 3.493 | 4.164 | 3.348 | 3.678 |
| Npf | -4.270 | -3.801 | -3.344 | -4.135 | -4.477 |
| 60 degree | | | | | |
| Mean | -648522.4252 | -161165.2532 | 28803769.33 | -68900605.52 | -5683315.165 |
| Max | -27512.72228 | 461700.3122 | 99518887.43 | -19104347.03 | 5246500.22 |
| Minm | -1455691.493 | -772680.5242 | -40167149.64 | -132709924.8 | -19078466.78 |
| RMS | 155613.623 | 178000.4218 | 19590378.93 | 13467617.31 | 3217559.132 |
| Ppf | 3.991 | 3.499 | 3.610 | 3.697 | 3.397 |
| Npf | -5.187 | -3.435 | -3.521 | -4.738 | -4.163 |
| 75 degree | | | | | |
| Mean | -811665.3545 | 206474.0326 | -10241284.7 | -72730644.13 | -5430281.033 |
| Max | -282134.8049 | 727925.769 | 43116426.69 | -25946855.05 | 3160403.428 |
| Minm | -1400370.779 | -275604.8916 | -65264979.93 | -129248526 | -17349104.66 |
| RMS | 168275.3401 | 138806.3129 | 14381941.61 | 13792060.16 | 2507843.969 |
| Ppf | 3.147 | 3.757 | 3.710 | 3.392 | 3.426 |
| Npf | -3.498 | -3.473 | -3.826 | -4.098 | -4.753 |
| 90 degree | | | | | |
| Mean | -827206.2382 | 439142.0745 | -37306859.87 | -72452104.93 | -1527932.448 |
| Max | -224111.9907 | 939697.3436 | 6639737.179 | -21248363.27 | 7930020.124 |
| Minm | -1429518.317 | 14389.38687 | -91072103.25 | -126288562.6 | -11148446.66 |
| RMS | 144155.6788 | 117497.2363 | 12683413.85 | 12003335.64 | 2539224.456 |
| Ppf | 4.184 | 4.260 | 3.465 | 4.266 | 3.725 |
| Npf | -4.178 | -3.615 | -4.239 | -4.485 | -3.789 |
| 105 degree | | | | | |
| Mean | -697992.8956 | 263972.7637 | -26282738.11 | -71379168.94 | 2176369.77 |
| Max | -308882.5886 | 818468.4183 | 47786560.49 | -29501822.3 | 10493195.8 |
| Minm | -1146618.933 | -345322.0762 | -94756422.88 | -109544575.5 | -7154425.044 |
| RMS | 121412.4821 | 128754.5968 | 15140951.22 | 11354211.4 | 2149726.719 |
| Ppf | 3.205 | 4.307 | 4.892 | 3.688 | 3.869 |
| Npf | -3.695 | -4.732 | -4.522 | -3.361 | -4.340 |
| 180 degree | | | | | |
| Mean | -729358.0624 | 37077.38691 | -2983800.754 | -60577434.78 | 837302.7979 |
| Max | -155934.2343 | 1003530.653 | 122515103.5 | -17712570.95 | 13831966.56 |
| Minm | -1453783.52 | -1135172.115 | -105932601.4 | -107177580 | -13354887.68 |
| RMS | 178683.0135 | 257382.1611 | 27786446.54 | 13081874.22 | 3590751.132 |
| Ppf | 3.209 | 3.755 | 4.517 | 3.277 | 3.619 |
| Npf | -4.054 | -4.555 | -3.705 | -3.562 | -3.952 |
| 195 degree | | | | | |

| | | | | | |
|---|---|---|---|---|---|
| Mean | -640596.1021 | 383406.308 | -35230412.72 | -54227243.35 | 2747595.933 |
| Max | -164512.277 | 1348851.105 | 62077812.55 | -20665552.3 | 15262812.5 |
| Minm | -1348851.525 | -488410.4502 | -142311003.1 | -102250327.7 | -9062349.868 |
| RMS | 154629.035 | 208779.6105 | 23875958.81 | 11144205.85 | 2948470.698 |
| Ppf | 3.079 | 4.624 | 4.076 | 3.012 | 4.245 |
| Npf | -4.580 | -4.176 | -4.485 | -4.309 | -4.005 |

285 degree

| | | | | | |
|---|---|---|---|---|---|
| Mean | -851706.999 | -222218.4418 | 22588123.04 | -78318883.45 | 698324.1822 |
| Max | -223034.4564 | 337690.1084 | 87859933.98 | -29526464.64 | 11447158.22 |
| Minm | -1712802.74 | -873796.3981 | -29809861.87 | -145928349.9 | -10970079.33 |
| RMS | 183087.3273 | 156334.249 | 15352276.53 | 14742221.54 | 2731328.852 |
| Ppf | 3.434 | 3.581 | 4.252 | 3.310 | 3.935 |
| Npf | -4.703 | -4.168 | -3.413 | -4.586 | -4.272 |

300 degree

| | | | | | |
|---|---|---|---|---|---|
| Mean | -870001.2028 | 35093.16641 | -4579775.562 | -76357165.42 | 3797413.053 |
| Max | -339212.8417 | 719260.0733 | 69753007.15 | -31682875.21 | 15739696.02 |
| Minm | -1512926.466 | -740247.141 | -69433494.03 | -124532793.3 | -8108387.456 |
| RMS | 152140.5899 | 166153.1362 | 16032342.43 | 12572882.41 | 3060367.013 |
| Ppf | 3.489 | 4.118 | 4.636 | 3.553 | 3.902 |
| Npf | -4.226 | -4.666 | -4.045 | -3.832 | -3.890 |

315 degree

| | | | | | |
|---|---|---|---|---|---|
| Mean | -878913.1711 | 277033.7808 | -30413863.05 | -76897964.19 | 6245979.344 |
| Max | -281699.7338 | 1001622.398 | 48428087.96 | -34126877.02 | 20986391.61 |
| Minm | -1584131.85 | -391109.6873 | -99604433.66 | -136371383.9 | -4292733.172 |
| RMS | 184761.9664 | 183694.4242 | 18149739.15 | 14421303.07 | 2876158.257 |
| Ppf | 3.232 | 3.945 | 4.344 | 2.966 | 5.125 |
| Npf | -3.817 | -3.637 | -3.812 | -4.124 | -3.664 |

330 degree

| | | | | | |
|---|---|---|---|---|---|
| Mean | -826989.1259 | 549462.3133 | -60110058.68 | -71530283.2 | 8319364.192 |
| Max | -285310.3731 | 1243919.955 | 5340235.102 | -28702831.42 | 22894316.43 |
| Minm | -1520558.149 | -82037.83385 | -136991026.8 | -118774783.1 | -1907924.824 |
| RMS | 158135.8935 | 156542.754 | 17581143.37 | 12981502.23 | 2901334.981 |
| Ppf | 3.425 | 4.436 | 3.723 | 3.299 | 5.024 |
| Npf | -4.386 | -4.034 | -4.373 | -3.639 | -3.525 |

345 degree

| | | | | | |
|---|---|---|---|---|---|
| Mean | -583299.8107 | 452152.7948 | -55691764.56 | -57968999.32 | 10955590.77 |
| Max | -44577.06146 | 1114185.204 | 28317314.6 | -10555828.21 | 23371199.96 |
| Minm | -1123724.972 | -232758.384 | -143371166.9 | -101418645.7 | -2384808.348 |
| RMS | 136483.4461 | 181496.1313 | 22221831.57 | 11676484.94 | 3319469.026 |
| Ppf | 3.947 | 3.648 | 3.780 | 4.061 | 3.740 |
| Npf | -3.960 | -3.774 | -3.946 | -3.721 | -4.019 |

## Major Critical Angles for design

**Table 4.6 Maximum and Minimum Values For Fx, Fy, Mx, My and Mz in case of both Standalone and Interference conditions for all angles of wind incidence**

### Standalone (Maximum Values)

| Degree | Fx max | Fy max | Mx max | My max | Mz max |
|---|---|---|---|---|---|
| 0 | 100226.1225 | 736738.1636 | 68342661.4 | -7004932.64 | 16187665.68 |
| 45 | -148631.8792 | 160259.7519 | 142134613.7 | -19426178.6 | 2861691.872 |
| 60 | -27512.72228 | 461700.3122 | 99518887.43 | -19104347 | 5246500.22 |
| 75 | -282134.8049 | 727925.769 | 43116426.69 | -25946855.1 | 3160403.428 |
| 90 | -224111.9907 | 939697.3436 | 6639737.179 | -21248363.3 | 7930020.124 |
| 105 | -308882.5886 | 818468.4183 | 47786560.49 | -29501822.3 | 10493195.8 |
| 180 | -155934.2343 | 1003530.653 | 122515103.5 | -17712570.9 | 13831966.56 |
| 195 | -164512.277 | 1348851.105 | 62077812.55 | -20665552.3 | 15262812.5 |
| 285 | -223034.4564 | 337690.1084 | 87859933.98 | -29526464.6 | 11447158.22 |
| 300 | -339212.8417 | 719260.0733 | 69753007.15 | -31682875.2 | 15739696.02 |
| 315 | -281699.7338 | 1001622.398 | 48428087.96 | -34126877 | 20986391.61 |
| 330 | -285310.3731 | 1243919.955 | 5340235.102 | -28702831.4 | 22894316.43 |
| 345 | -44577.06146 | 1114185.204 | 28317314.6 | -10555828.2 | 23371199.96 |

### Standalone (Minimum Values)

| Degree | Fx min | Fy min | Mx min | My min | Mz min |
|---|---|---|---|---|---|
| 0 | -1122322.622 | -522444.1191 | -93698792.8 | -88540222.9 | -10045226.15 |
| 45 | -1179052.662 | -1138988.148 | -12059141 | -107816647 | -20509312.72 |
| 60 | -1455691.493 | -772680.5242 | -40167149.6 | -132709925 | -19078466.78 |
| 75 | -1400370.779 | -275604.8916 | -65264979.9 | -129248526 | -17349104.66 |
| 90 | -1429518.317 | 14389.38687 | -91072103.2 | -126288563 | -11148446.66 |
| 105 | -1146618.933 | -345322.0762 | -94756422.9 | -109544576 | -7154425.044 |
| 180 | -1453783.52 | -1135172.115 | -105932601 | -107177580 | -13354887.68 |
| 195 | -1348851.525 | -488410.4502 | -142311003 | -102250328 | -9062349.868 |
| 285 | -1712802.74 | -873796.3981 | -29809861.9 | -145928350 | -10970079.33 |
| 300 | -1512926.466 | -740247.141 | -69433494 | -124532793 | -8108387.456 |
| 315 | -1584131.85 | -391109.6873 | -99604433.7 | -136371384 | -4292733.172 |
| 330 | -1520558.149 | -82037.83385 | -136991027 | -118774783 | -1907924.824 |
| 345 | -1123724.972 | -232758.384 | -143371167 | -101418646 | -2384808.348 |

### Interference (Maximum Values)

| Degree | Fx max | Fy max | Mx max | My max | Mz max |
|---|---|---|---|---|---|
| 45 | -199167.0457 | 122102.7964 | 116377152.9 | -17068649.35 | -6200462.632 |
| 60 | -95625.93018 | 217495.0601 | 100030142.8 | -9709646.872 | 0 |
| 75 | -176751.0823 | 602881.3398 | 38157066.77 | -27186972.44 | 6200462.632 |
| 90 | -336593.88 | 602881.2467 | 34166278.35 | -30603530.92 | 9062349.868 |
| 105 | -123226.2132 | 524659.7914 | 72098871.55 | -9282302.313 | 4292733.172 |
| 180 | -65499.19721 | 734522.7262 | 97809857.71 | -13545313.97 | 13354887.68 |
| 195 | -131952.3167 | 1123724.971 | 77306244.67 | -10457490.28 | 14308850.09 |
| 285 | -118989.0658 | 486502.6316 | 77258944.33 | -21541223.37 | 16693658.44 |

| | | | | | |
|---|---|---|---|---|---|
| 300 | -380940.304 | 604789.4243 | 49293171.18 | -32798931.46 | 12878004.15 |
| 315 | -389613.0799 | 856625.4436 | 22342840.13 | -32631854 | 18124504.37 |
| 330 | -335286.9872 | 1264905.914 | 2864208.253 | -25612466.02 | 19555545.67 |
| 345 | 26423.05295 | 1325957.142 | 35447066.87 | -231903.846 | 22894316.43 |

**Interference (Minimum Values)**

| Degree | Fx min | Fy min | Mx min | My min | Mz min |
|---|---|---|---|---|---|
| 45 | -1163789.859 | -824192.3596 | -7985379.172 | -107319534.1 | -27663933.13 |
| 60 | -1074121.036 | -740247.0141 | -23257617.84 | -101600883.2 | -26709970.72 |
| 75 | -1413718.109 | -259468.15 | -50558209.47 | -141658180.1 | -15262812.5 |
| 90 | -1549176.207 | -213680.1034 | -61448143.85 | -124680691.8 | -7631308.568 |
| 105 | -1063716.639 | -576171.6193 | -62208090.91 | -90692319.4 | -20032429.2 |
| 180 | -1236288.263 | -770772.2873 | -81713859.41 | -97126927.69 | -10970079.33 |
| 195 | -1317793.27 | -602881.3776 | -124322526.1 | -101099926.5 | -10970079.33 |
| 285 | -1543452.657 | -663932.6282 | -46842144.78 | -119758212.3 | -10016312.28 |
| 300 | -1558714.928 | -513212.1473 | -63073162.41 | -127293840.6 | -7154425.044 |
| 315 | -1642660.566 | -251836.9656 | -90016318.89 | -136245998.4 | -4769616.696 |
| 330 | -1459600.049 | -28617.90046 | -136445501.4 | -119441164.3 | -1907924.824 |
| 345 | -1432796.791 | -293809.9812 | -159575238.2 | -119165469.9 | -3815654.284 |

**Fig 4.7 to Fig 4.16 Plots of Maximum and Minimum Values For Fx, Fy, Mx, My and Mz in case of both Standalone and Interference conditions to find out Major Critical Angles**

**STANDALONE condition**

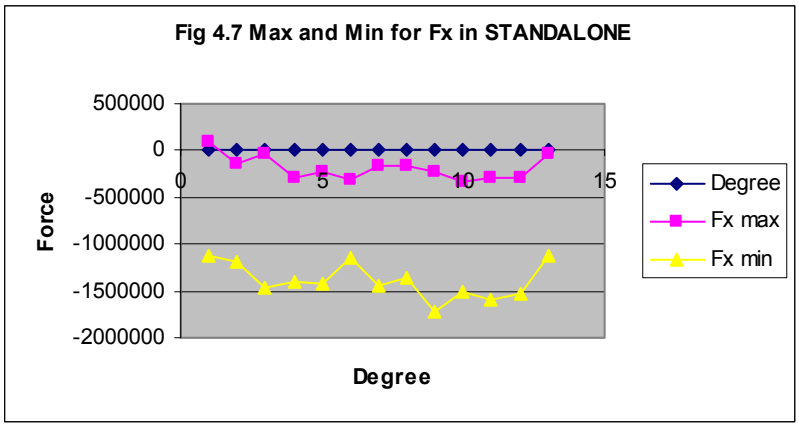


Fig 4.8 Max. and Min. for Fy in STANDALONE

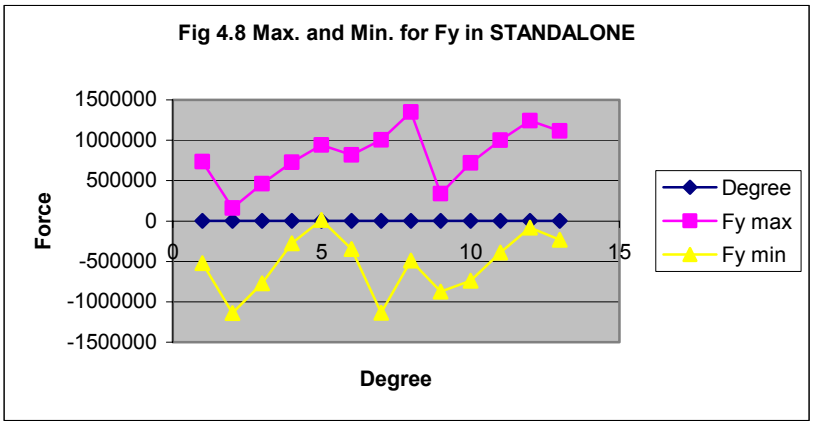



Fig 4.9 Max. and Min. for Mx in STANDALONE

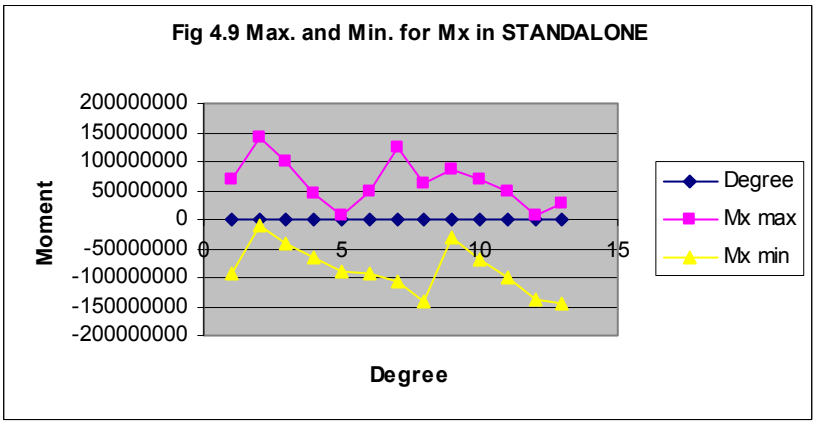



Fig 4.10 Max. and Min. for My in STANDALONE

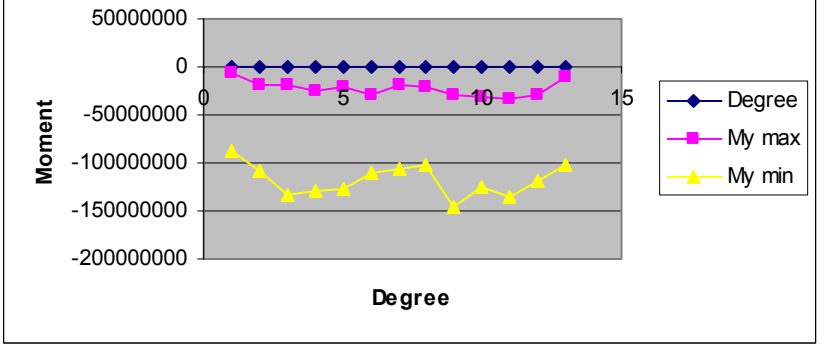
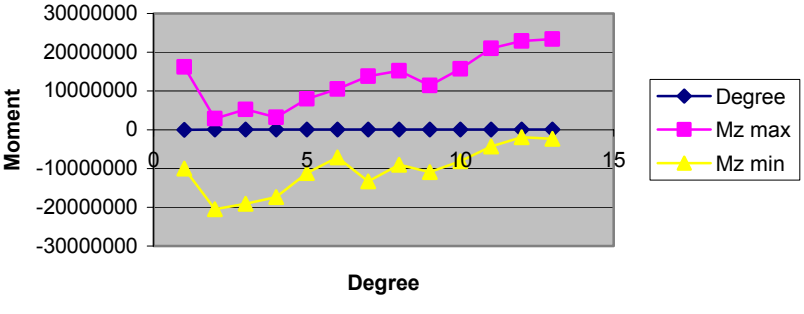



Fig 4.11 Max. and Min. for Mz in STANDALONE



## INTERFERENCE condition

Fig 4.12 Max. and Min. for Fx in INTERFERENCE

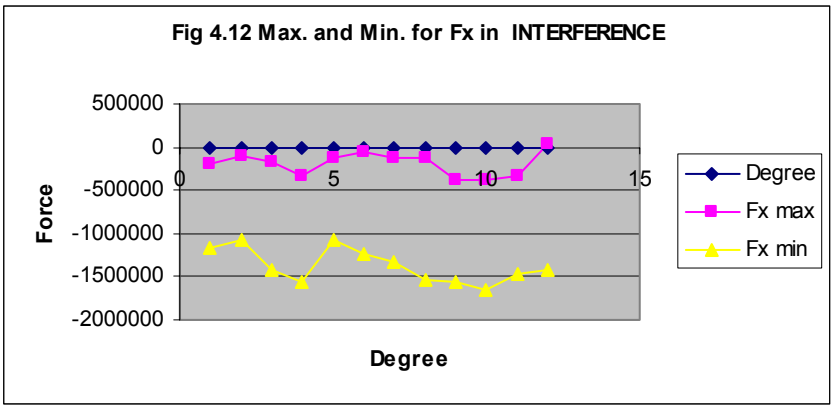



Fig 4.13 Max. and Min. for Fy in INTERFERENCE

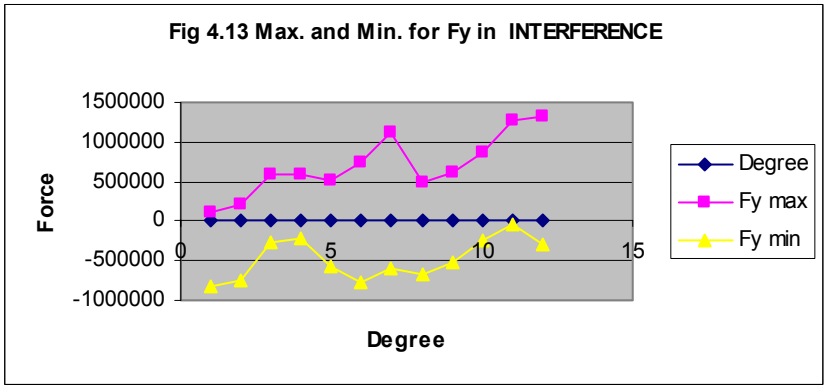



Fig 4.14 Max. and Min. for Mx in INTERFERENCE

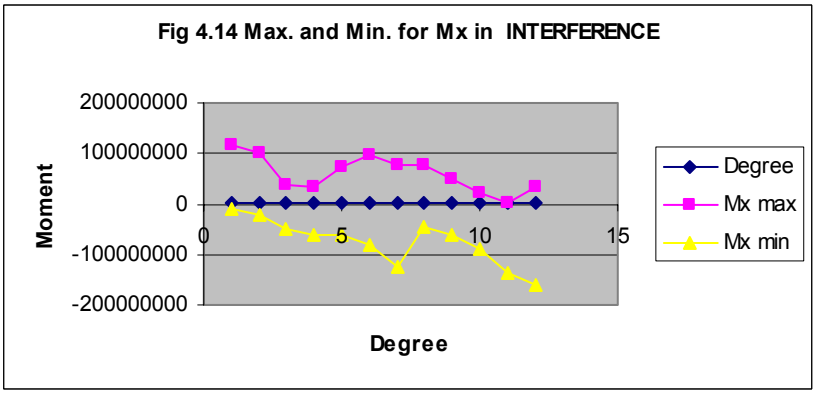
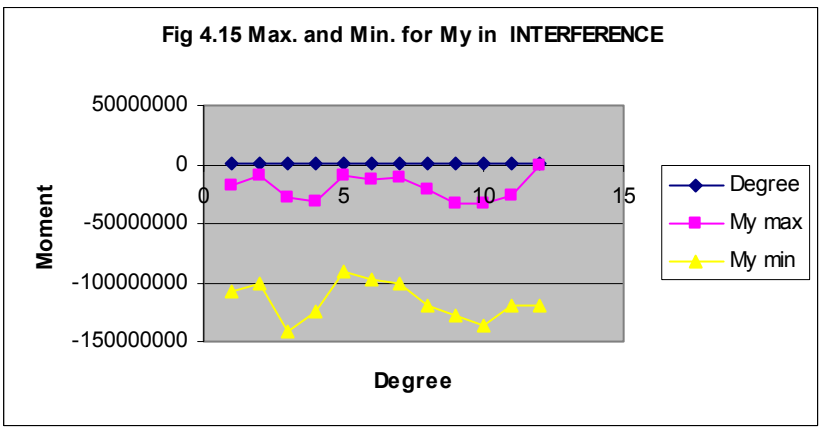



Fig 4.15 Max. and Min. for My in INTERFERENCE



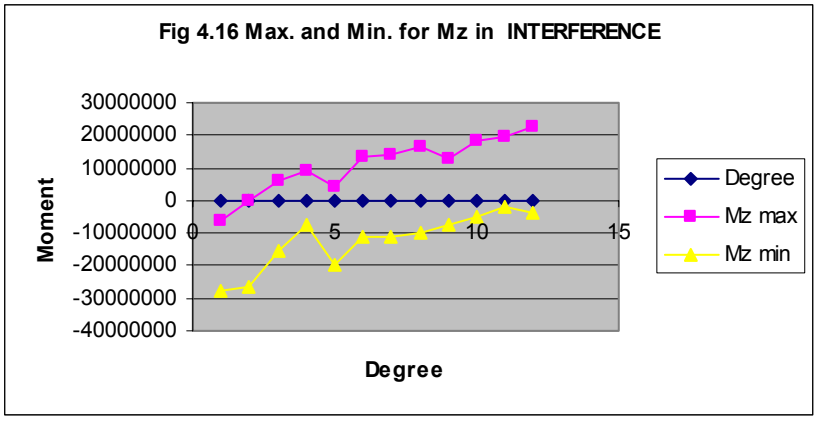



**Table 4.7 Parameters of Measured Response for Dynamic Analysis**

| Parameters of Measured Response for Dynamic Analysis | | | | | | |
|---|---|---|---|---|---|---|
| | SM | | Mean | | RMS | |
| | Mx | My | Mx | My | Mx | My |
| 45 degree | 2060958.3 | 1573669.7 | 56620633.92 | 58978226.81 | 20537983.75 | 11812317.57 |
| | | | | | | |
| | | | | | | |
| 90 degree | 1050101.2 | 927848.5 | -37306859.9 | 72452104.93 | 12683413.85 | 12003335.64 |
| | | | | | | |
| | | | | | | |
| 195 degree | 1779545.7 | 1269312 | -35230412.7 | 54227243.35 | 23875958.81 | 11144205.85 |
| | | | | | | |
| | | | | | | |
| 345 degree | 1627188.3 | 1984474.9 | -55691764.6 | 57968999.32 | 22221831.57 | 11676484.94 |
| | | | | | | |

**Table 4.8 Dynamic Analysis for Tip Acceleration and Design Lateral Forces from Wind Tunnel Test Results (for 345$^{o}$ in X-dir$^{n}$ in Standalone condition)**

**Dynamic Analysis for Tip Acceleration and Design Lateral Forces from Wind Tunnel Test Results: 345/SA/Fx**

| Building Parameters | | | RMS & Peak Acceleration Computation | | |
|---|---|---|---|---|---|
| α= | 0.18 | | Effective frontal area at ht 'z', Ae | 870 | m2 |
| Zref= | 10 | m | Ae top M | 435 | m2 |
| Plan Width | 87 | m | Building mass per unit height, mo= | 679470 | Kg/m |
| Plan depth | 71 | m | Generalised Mass in I mode, M1* | 33973500 | kg |
| Story height= | 3.5 | m | Generalised Stiffness in I mode, K1* | 5251099 | Kg/m |
| Vref at 10m height | 23.5 | m/s | Spectral Moment from FFT, Sm | 1984474.9 | Kg2-m2 |
| Vb (fig.1) = | 47 | m/s | Generalised Wind Force Spectrum, SF1*(n) | 88 | Kg2 |
| * Bldg bulk Density | 110 | Kg/m3 | RMS Base Moment from HFFB, σMy | 11676484.94 | Kg-m |
| Height of Building, h | 150 | m | RMS generalised force, σF* | 77843 | Kg |
| * Face width | 87 | m | Variance of top displacement, σx^2 | 0.0002 | m2 |
| * Depth | 71 | m | RMS displacement at top, σx | 0.0148 | m |
| Interval | 10 | m | RMS acceleration at top, σ"x | 0.0225 | m/s2 |
| Vh | 38.3 | m/s | Mean OT Base Moment from HFFB, Mmean | 57968999.32 | Kg-m |
| ρ= | 1.2 | Kg/m3 | 1/2ρ(Vmean)^2*B*H^2 | 1752720 19.8 | Kg |
| Natural Period used | 5.1 | Sec | Cmy mean | 0.3307 | |
| Frequency used, Hz | 0.196 | Hz | Cσmy | 0.0666 | |
| ξ (Damping coefficient) | 0.035 | | Background peak factor, gB | 3.5 | |
| n/n1= | 1 | At resonance | Resonant Peak factor, gD | 3.78 | |
| H1(n) | 14.285714 | | RMS Acceln at Top, % g | 0.23 | |
| Cf (Fig-4-IS875pt3) | 1.2 | | Peak Acceln at Top, % g | 0.87 | |

Computation of Mean, Resonant and Background lateral force component along the height using Random Vibration Theory

| height | ϕ=z/H | Sx | σDx"(z) | σDFx(z) | px(z) | Fxmean(z) | pσx(z) | σBFx(z) | Fxmax(z) | Lateral force |
|---|---|---|---|---|---|---|---|---|---|---|
| 0 | 0.000 | 0 | 0 | 0 | 0 | 0 | 0 | 0.00 | 0 | 0 |
| 10 | 0.067 | 2.90124E-12 | 0.00149851 | 1037.9098 | 258.63196 | 22936.77926 | 52.095296 | 4620.07 | 39576.4901 | 32114 |
| 20 | 0.133 | 1.1605E-11 | 0.00299701 | 2075.8196 | 331.93495 | 29437.65651 | 66.86045203 | 5929.52 | 51625.7609 | 41892 |
| 30 | 0.200 | 2.61112E-11 | 0.00449552 | 3113.7294 | 384.10136 | 34064.03472 | 77.36814098 | 6861.39 | 60809.8898 | 49344 |
| 40 | 0.267 | 4.64199E-11 | 0.00599402 | 4151.6392 | 426.01392 | 37781.05073 | 85.81043565 | 7610.10 | 68698.4964 | 55746 |
| 50 | 0.333 | 7.25311E-11 | 0.00749253 | 5189.549 | 461.64846 | 40941.30076 | 92.98817228 | 8246.66 | 75843.4925 | 61543 |
| 60 | 0.400 | 1.04445E-10 | 0.00899103 | 6227.4588 | 492.96563 | 43718.66434 | 99.29627579 | 8806.09 | 82506.0169 | 66950 |
| 70 | 0.467 | 1.42161E-10 | 0.01048954 | 7265.3686 | 521.09569 | 46213.37957 | 104.9624126 | 9308.59 | 88830.3259 | 72082 |
| 80 | 0.533 | 1.8568E-10 | 0.01198804 | 8303.2784 | 546.75729 | 48489.17895 | 110.1313354 | 9767.00 | 94904.3306 | 77010 |
| 90 | 0.600 | 2.35001E-10 | 0.01348655 | 9341.1883 | 570.43937 | 50589.42445 | 114.9015304 | 10190.04 | 100785.432 | 81783 |
| 100 | 0.667 | 2.90124E-10 | 0.01498505 | 10379.098 | 592.49159 | 52545.12567 | 119.3434285 | 10583.97 | 106513.102 | 86430 |
| 110 | 0.733 | 3.5105E-10 | 0.01648356 | 11417.008 | 613.17375 | 54379.32366 | 123.5093615 | 10953.43 | 112115.607 | 90977 |
| 120 | 0.800 | 4.17779E-10 | 0.01798207 | 12454.918 | 632.68485 | 56109.66602 | 127.4394119 | 11301.97 | 117613.864 | 95438 |
| 130 | 0.867 | 4.9031E-10 | 0.01948057 | 13492.827 | 651.18111 | 57750.00709 | 131.1650463 | 11632.37 | 123023.784 | 99828 |
| 140 | 0.933 | 5.68644E-10 | 0.02097908 | 14530.737 | 668.78771 | 59311.44815 | 134.7114785 | 11946.89 | 128357.745 | 104156 |
| 150 | 1.000 | 6.5278E-10 | 0.02247758 | 15568.647 | 685.60668 | 60803.03928 | 138.0992638 | 12247.34 | 133625.574 | 108431 |
| | | | Total mean force at base in Kg = | | | 695070.0792 | Total force at base in Kg = | | 1384829.91 | 1123725 |
| | | | computed base Moment in Kg-m = | | | 59114195.23 | O.T. base Moment in Kg-m = | | 121804764 | 98838892.57 |

### 4.3.5 Model analysis of Signature Building by Balendra's Procedure

## Given:

| | | | |
|---|---|---|---|
| Building Plan | = | B*D | |
| Width=B | = | 87 | m |
| Depth=D | = | 71 | m |
| Height of Building=H | = | 150 | m |
| $\rho_a$ | = | 1.2 | $Kg/m^3$ |
| $\rho_b$ | = | 110 | $Kg/m^3$ |
| Time period=T | = | 5.10 | sec |
| $S_M$ | = | 1984474.9 | Kg2-m2 |
| $\zeta$ | = | 0.035 | |
| $\sigma_M$ | = | 11676484.94 | Kg-m |
| $\bar{M}$ | = | 57968999.32 | Kg-m |
| $V_H$ | = | 38.3 | m/s |
| α | = | 0.18 | |
| Δz | = | 10 | m |
| $g_B$ | = | 3.5 | |

**Calculation:**

| | | | |
|---|---|---|---|
| Natural frequency ($n_0$) | = | 1/T | |
| | = | 1/5.10 | |
| | = | 0.196 | Hz |
| $m_0$ | = | mass per unit height | |
| | = | $\rho_b$*B*D | |
| | = | 110*87*71 | |
| | = | 679470 | Kg/m |

$M_1^*$ = generalized mass in first mode

$M_1^*$ = $\frac{1}{3} m_0 H$

= 1/3*679470*150

= 33973500 Kg

$K_1^*$ = Generalized stiffness in first mode

$K_1^*$ = $(2\pi n_1)^2 M_1^*$

= (2*3.14*0.196)$^2$ *33973500

= 51472080.29 N/m

= 51472080.29/9.81

= 5251099 Kg/m

$S_{F_1^*}$ = generalized wind force spectrum

$S_{F_1^*}$ = $\frac{S_M}{H^2}$

= 1984474.9/ (130)$^2$

= 88 Kg$^2$

$|H_1(n)|$ = the mechanical admittance function

$|H_1(n)|$ = $\frac{1}{\left[\left\{1-(\frac{n}{n_1})^2\right\}^2 + 4\varsigma_1^2(\frac{n}{n_1})^2\right]^{1/2}}$

= 14.28571

$S_x$ = Power spectral of the responce

$S_x$ = $\phi_1^2(z).\frac{1}{K_1^{*2}}|H_1(n)|^2.S_{F_1}(n)$

= 6.5278*10$^{-10}$ m$^2$ (at top of building)

$\sigma_F^*$ = $\sigma_M$/H

= 11676484.94/150

= 77843 Kg

$\sigma_x^2$ = Variance or mean square value

$\sigma_x^2$ = $\frac{\sigma_{F_1^*}^2}{K_1^{*2}} + \frac{\Pi n_1 S_{F_1^*}(n)}{4\varsigma_1 K_1^{*2}}$

= $2*10^{-4}$ $m^2$

$\sigma_x$ = $\sqrt{\sigma_x^2}$

= 0.0148 m (at top of building)

$\ddot{\sigma}_x$ = RMS acceleration at Top

$\ddot{\sigma}_D = \ddot{\sigma}_x$ = $(2\Pi n_1)^2 \sigma_x$

= 0.0225 $m/sec^2$

**For Resonant force**

$\ddot{\sigma}_D(z)$ = Acceleration at any Height Z

$\ddot{\sigma}_D(z)$ = $\phi \ddot{\sigma}_D$

= 0.0225 $m/sec^2$

$\sigma_{D.F}(z)$ = Resonant / Inertial force

$\sigma_{D.F}(z)$ = $m_0 \Delta_z \ddot{\sigma}_D(z)$

= 152728.5 N

= 152728.5/9.81

= 15568.65 Kg

**For Mean force**

$\bar{M}$ = Mean overturning moments

$\bar{M}$ = 57968999.32 Kg-m

C = $\frac{1}{2}\rho_{air} U_H^2 \bar{B} \bar{H}^2$

= 1719418514/9.81 Kg-m

= 175272019.8 Kg-m

$C_{\bar{M}}$ = Mean overturning moment cofficient

$C_{\bar{M}}$ = $\dfrac{\overline{M}}{\frac{1}{2}\rho_{air}\,\overline{U_H^2}\,BH^2}$

= 0.3307

$P(z)$ = Mean pressure at height z

$P(z)$ = $(1+\alpha)\rho_{air}.C\bar{M}.\bar{U}_H^2(\frac{z}{H})^{2\alpha}$

= 685.6067 N/m$^2$

$\overline{F}(z)$ = Mean component of wind load

$\overline{F}(z)$ = $p(z)B\Delta z$

= 60803.40 Kg

**For Non resonant**

$C_{\sigma M}$ = RMS Moment Coefficient

$C_{\sigma M}$ = $\dfrac{\sigma_M}{\frac{1}{2}\rho_{air}\,\overline{U}_H^{\,2}\,BH^2}$

= 11676484.94/175272019.8

= 0.0666

$p\sigma(z)$ = RMS Pressure at height z

$p\sigma(z)$ = $(1+\alpha)\rho_{air}C_{\sigma M}\,\overline{U}_H^{\,2}(\frac{z}{H})^{2\alpha}$

= 138.099264 N/m$^2$

$\sigma_{BF}(z)$ = Background or Non-resonant Force

$\sigma_{BF}(z)$ = $p\sigma(z)B\Delta_z/2$

= 12247.34 Kg

$g_D$ = the resonant peak factor

$g_D$ = $\sqrt{2\ln(600n_1)}$

= 3.78

$g_B$ = the peak factor of background component,

= 3.5

$F_{\max}(z)$ = The peak values of the wind- induced load

$F_{\max}(z)$ = $\bar{F}(z) + \left[\left(g_B \sigma_{B,F}(z)\right)^2 + \left(g_D \sigma_{D,F}(z)\right)^2\right]^{\frac{1}{2}}$

= 133625.57 Kg

**Table 4.9 Storey Wise Lateral Forces**
**By Balendra's Procedure for Signature Tower**

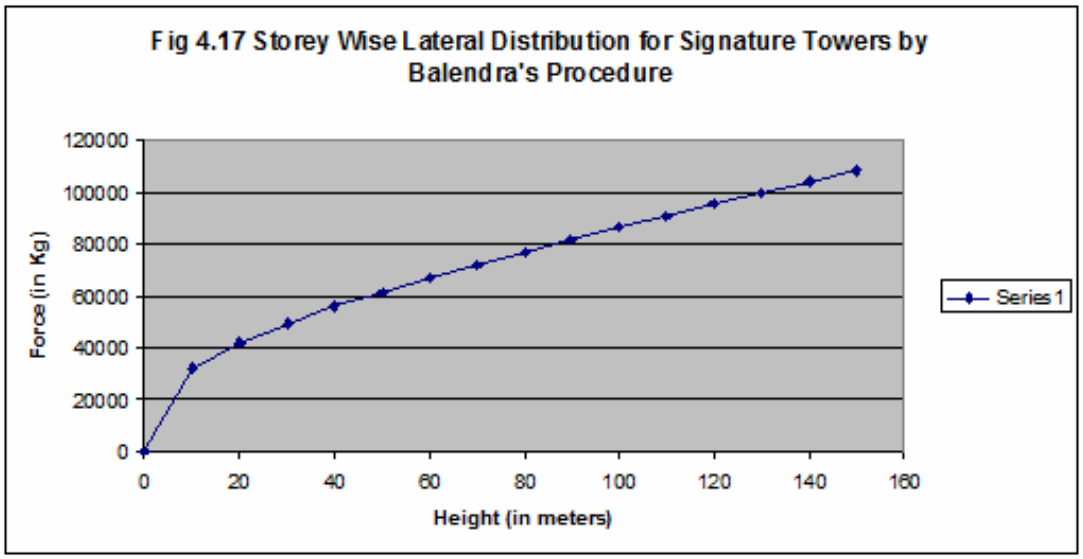
Storey Wise Lateral Forces

| height | ϕ=z/H | Sx | σDx"(z) | σDFx(z) | px(z) | Fxmean(z) | pσx(z) | σBFx(z) | Fxmax(z) | Lateral force |
|---|---|---|---|---|---|---|---|---|---|---|
| 0 | 0.000 | 0 | 0 | 0 | 0 | 0 | 0 | 0.00 | 0 | 0 |
| 10 | 0.067 | 2.9E-12 | 0.001499 | 1037.91 | 258.632 | 22936.78 | 52.0953 | 4620.07 | 39576.49 | 32114 |
| 20 | 0.133 | 1.16E-11 | 0.002997 | 2075.82 | 331.935 | 29437.66 | 66.86045 | 5929.52 | 51625.76 | 41892 |
| 30 | 0.200 | 2.61E-11 | 0.004496 | 3113.729 | 384.1014 | 34064.03 | 77.36814 | 6861.39 | 60809.89 | 49344 |
| 40 | 0.267 | 4.64E-11 | 0.005994 | 4151.639 | 426.0139 | 37781.05 | 85.81044 | 7610.10 | 68698.5 | 55746 |
| 50 | 0.333 | 7.25E-11 | 0.007493 | 5189.549 | 461.6485 | 40941.3 | 92.98817 | 8246.66 | 75843.49 | 61543 |
| 60 | 0.400 | 1.04E-10 | 0.008991 | 6227.459 | 492.9656 | 43718.66 | 99.29628 | 8806.09 | 82506.02 | 66950 |
| 70 | 0.467 | 1.42E-10 | 0.01049 | 7265.369 | 521.0957 | 46213.38 | 104.9624 | 9308.59 | 88830.33 | 72082 |
| 80 | 0.533 | 1.86E-10 | 0.011988 | 8303.278 | 546.7573 | 48489.18 | 110.1313 | 9767.00 | 94904.33 | 77010 |
| 90 | 0.600 | 2.35E-10 | 0.013487 | 9341.188 | 570.4394 | 50589.42 | 114.9015 | 10190.04 | 100785.4 | 81783 |
| 100 | 0.667 | 2.9E-10 | 0.014985 | 10379.1 | 592.4916 | 52545.13 | 119.3434 | 10583.97 | 106513.1 | 86430 |
| 110 | 0.733 | 3.51E-10 | 0.016484 | 11417.01 | 613.1738 | 54379.32 | 123.5094 | 10953.43 | 112115.6 | 90977 |
| 120 | 0.800 | 4.18E-10 | 0.017982 | 12454.92 | 632.6849 | 56109.67 | 127.4394 | 11301.97 | 117613.9 | 95438 |
| 130 | 0.867 | 4.9E-10 | 0.019481 | 13492.83 | 651.1811 | 57750.01 | 131.165 | 11632.37 | 123023.8 | 99828 |
| 140 | 0.933 | 5.69E-10 | 0.020979 | 14530.74 | 668.7877 | 59311.45 | 134.7115 | 11946.89 | 128357.7 | 104156 |
| 150 | 1.000 | 6.53E-10 | 0.022478 | 15568.65 | 685.6067 | 60803.04 | 138.0993 | 12247.34 | 133625.6 | 108431 |

### 4.3.6 Result Discussion for Dynamic Analysis by Balendra's approach for Signature Towers

In the Dynamic Analysis by Balendra's approach firstly the Wind Tunnel testing is used to find out the Mean Overturning Moments and the Spectral Moments to Analysis the Response of the Signature Towers by Balendra's approach. In the Wind Tunnel Testing the building is tested for An isolated condition and Interference condition of the nearby buildings and the values of Forces and their Respective Moments are being determined. After the Forces and Moments are known their Spectral Moments and Mean Overturning moments are determined with those values in a program developed in IIT Roorkee. Those Values are then used in the Balendra's approach to find out the Base Forces and Base Moments with their Storey Wise lateral Distribution on the building A reference calculations for Balendra's procedure at the topmost height for the Maximum Spectral Moment is shown in **Table 4.8** and its Storey Wise lateral Distribution is shown in **Table 4.9** respectively**.**

It is seen from the **Table 4.9** that with the increase in the height of the building the Storey wise Lateral Forces are also increasing as it can be seen in the graph shown in **Fig 4.17** in which from 10m height there is a linear increment in the Force value for Storey wise Lateral Forces.



## 4.4 Comparison of the Result's for Signature Towers for Analytical Response and Dynamic Analysis by Balendra's Approach.

The comparison of the Storey Wise Lateral Distribution obtained by both the Analysis is shown in the graph of **Fig 4.19** whereas the Differences in the Base Forces and Base Moments with the two different analysis for the Signature Towers is shown in **Table 4.10.** In both the comparison of Storey Wise Lateral Forces and Base Forces and Base moments are much higher in Wind Tunnel Testing than those for the Analytical Analysis.

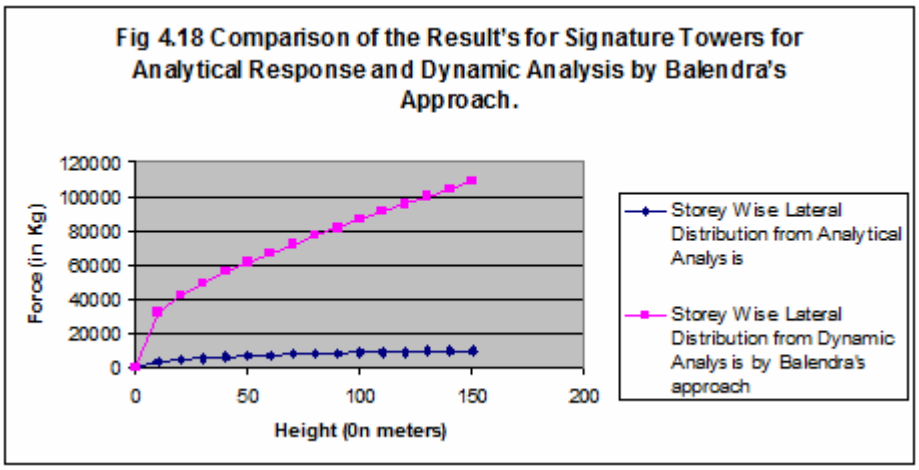



**Table 4.10 Comparison between the Base Forces And Base Moments by Analytical And Wind Tunnel Testing**

| Sr. No. | SIGNATURE TOWERS | WT | IS-CODE (Davenport approach |
|---|---|---|---|
| 1 | BASE FORCE (Kg) | 1123725 | 112412.79 |
| 2 | BASE MOMENT (Kg-m) | 98838892.6 | 9535987.08 |

# CHAPTER - 5

# Pressure Measurement on Tall Buildings

## 5.0 General

As discussed earlier wind loads on faces of the building depends upon the flow pattern around the building. Surface pressures, both mean and fluctuating components on the building face are not only strongly influenced by building geometry, face shape and wind incidence angle but also depend on the surroundings and wind flow characteristics. Wind tunnel tests on rigid models of building faces in a simulated flow conditions provide detailed information on the effects of various parameters that influence the wind pressure on these buildings. Wind pressure on Tall building is highly fluctuating and random in nature. Design pressure coefficients for any level on the face of the building can be obtained from this wind tunnel testing experimentally and can be used to calculate the force at particular level to know the wind base force and base moments thereby showing the effect of wind load on that building as discussed in chapter 6.

## 5.1 Pressure Studies on Signature Building

### 5.1.1 Parameters Studied

In the present study, rigid model of ***Signature building*** have been subjected to excessive wind tunnel testing. Details of building dimensions and model scale used have been described in the Chapter 4. Model of building has been tested under simulated flow conditions as mentioned in Chapter 3. Surface pressures for Signature building model have been measured for angle of wind incidence from $0^{o}$ to $360^{o}$ with increments of $15^{o}$. Wind pressures measured on the faces of the Wings of the Signature Tower have been expressed in the form of pressure coefficients as discussed in Chapter 3. Mean velocity at the top of the model height having being used as the reference velocity. The flow was maintained such as to get the required velocity of 10.78 m/s. The mean pressure coefficients (Cpmean), root mean square pressure coefficient (Cprms), minimum and maximum pressure coefficient (Cpmin, Cpmax) have been calculated from each pressure history record. All the values which we have obtained from the analysis are multiplied with a factor of 0.794 so as to convert the



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pressure coefficient values from mean hourly wind speed to 3 sec gust wind speed.

#### 5.1.1.1 Velocity Factor

We have taken velocity at top of the building model height for on-line computer system in which parameters are feed and it gives the value of all the pressure coefficients for 8192 samples at every angle of incidence and for both Standalone and Interference conditions. It is shown below that how we have obtained the velocity factor as 10.78 m/s and for the reference velocity at every calculation we have calculated the velocity at 1m height of the wind tunnel which also is shown below.

**1)** *At building height* =5.85

Now h = 3.4 (pressure head obtained from barometer in wind tunnel lab)

Therefore *at building height* velocity = 5.85

= **10.78 m/sec**

**2)** *Reference height at 1m top* =5.85

Where h = 3.7

Therefore *Reference height at 1m top* = 5.85

**= 11.176410 m/sec**

#### 5.1.1.2 Conversion factor for mean hourly approach to 3 gust factor approach at Reference height(10m)

***Conversion factor for 3 gust appraoch***

The

are with effect of hourly mean wind as stated in the methodology of wind tunnel but needs to be converted into 3 sec gust results and all the pressure coefficients are multiplied by the factor calculated for 3 sec gust approach as calculated below.

According to IS: 875(part 3) the wind load, **F**, acting in a direction normal to the individual structural element or cladding unit is:

$$F = Cp*A*Pz \qquad (5.1)$$

Where

Cp = pressure coefficient

A = surface area of structural element or cladding unit, and

Pz = design wind pressure



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But

$$\mathbf{Pz = 0.5*\dot{\rho}*Vz^2} \qquad \square\ \square\ \square\square 2$$

$\dot{\rho}$ = air density (taken as ***1.2 Kg/m$^3$*** )

V = design velocity and

The design wind speed Vz. can be calculated from equation:

$$\mathbf{Vz = V_b \times K_1 \times K_2 \times K_3} \qquad \square\ \square\ \square\square 3$$

Where,

$V_z$ = Design wind speed at height z in m/s

$V_b$ = Basic wind speed applicable at 10m height above mean ground over a short time interval of about 3 sec. gust for a return period of 50 years.

Now, for mean hourly wind

$Vz_H$ (*at 62m i.e., at top of the model height*) = $V_b$ x 1 x 0.721 x 1

($k_1$ = 1, $k_2$ = 0.721 from table33 and $k_3$ = 1) taken from IS: 875(part 3)

lll$^y$

for terrain categories in wind load condition wind speed is given as

$Vz_T$ (*at 62m i.e., at top of the model height*) = $V_b$ x 1 x 1.039 x 1

Therefore converting mean hourly wind speed into 3 gust wind speed we have

$$\mathbf{Vz = Vz_T \ /\ Vz_H}$$

$$\mathbf{Vz} = 1.039/0.721$$

$$= 1.44$$

Therefore $\mathbf{Vz^2} = 2.07$

***Conversion factor for Reference height (10m)***

Design wind speed:

At 62m height $Vz_{62}$ = $V_b$ x 1 x 1.039 x 1 (from table2 IS: 875(part 3))

At 10m height $Vz_{10}$ = $V_b$ x 1 x 0.820 x 1 (from table2 IS: 875(part 3))

Therefore $\mathbf{Vz_{10}\ /\ Vz_{62}}$ = 0.820/1.039

Making $Vz_{10}$ = 0.78 $Vz_{62}$

$Vz_{10}^2 = 0.78^2\ Vz_{62}^2$

**But the conversion factor for 3 sec. gust factor approach applied to the pressure coefficients is:**



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With reference to equation 5.1, 5.2 and 5.3

$Cp_{10} = Cp_{62} / (2.07 \times 0.78^2)$

**$Cp_{10} = \underline{0.794} \times Cp_{62}$**

So the factor to be multiplied to the pressure coefficients obtained for mean, max., min. and rms values from the wind tunnel testin g is 0.794.

### 5.1.2 Effects of Angle of Wind Incidence

Variations of averaged mean, peak (minimum and maximum), rms values with the changing of angle of wind incidence from $0^o$ to $360^o$ with an increment of $15^o$ in both Standalone and Interference conditions in the wind tunnel on the building model have been studied in this chapter and various positions of the building at different angles of wind incidence are shown in Fig 5.1

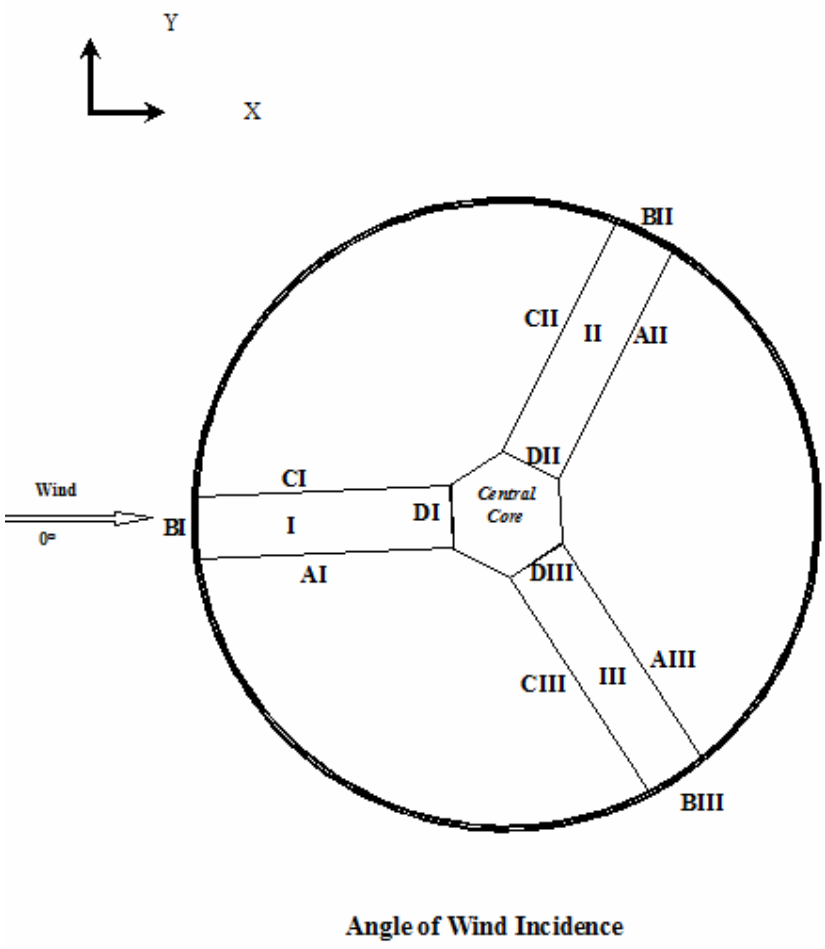


**Fig 5.1 Location of building at various angles on wind incidence**



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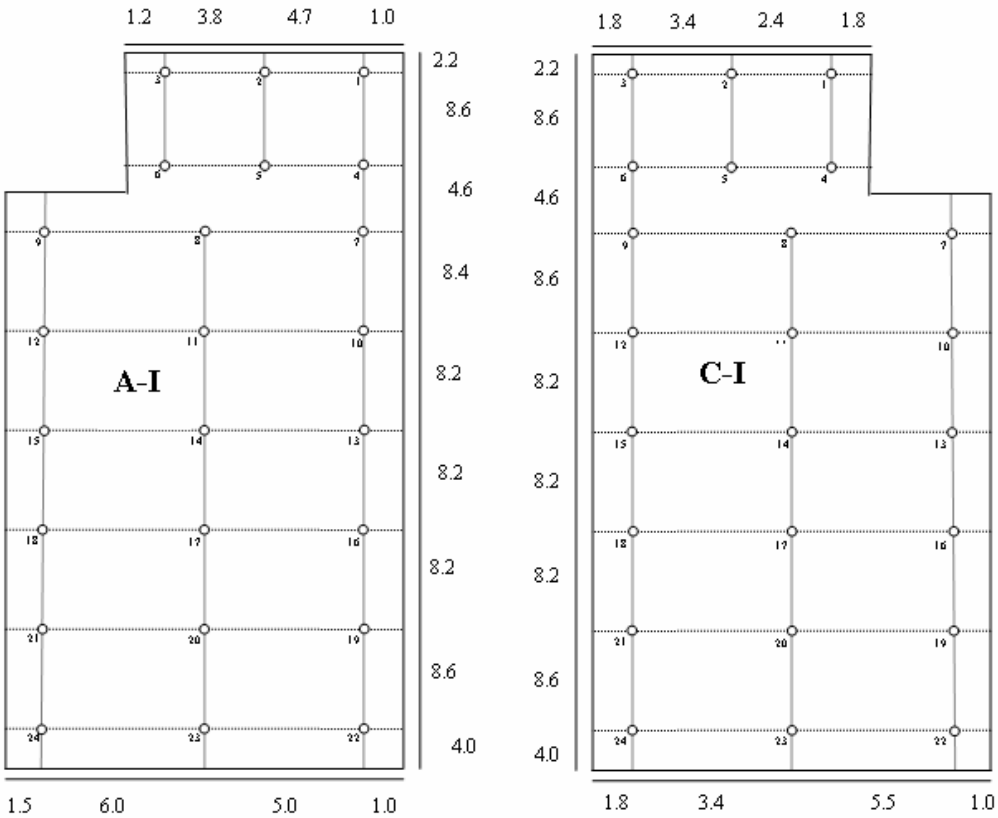
### 5.1.3 Pressure Fluctuations on the Building

The Pressure variations for mean, peak (minimum and maximum), rms values on Signature building has been studied and are presented on form of tables in which coefficients of pressures for mean, peak (minimum and maximum), rms values are obtained from the Wind tunnel experimentally on the building model. Again the flow conditions are maintained in the wind tunnel as in case of Dynamic analysis for the same building model but the velocity taken for pressure measurement is at the topmost height of the building model which is 10.78m/sec as calculated above and then the coefficients of pressures on the fixed taping locations on various wings and their respective faces of Signature Building are obtained. **Fig 5.2a, 5.2b, 5.2c, 5.2d, 5.2e, 5.2f** shows the Taping locations on the various faces of the respective wings of Signature building are shown.




All dimensions are in cm.

**Fig 5.2a Locations of tapings on Faces A and Face C of Wing I of Signature building**



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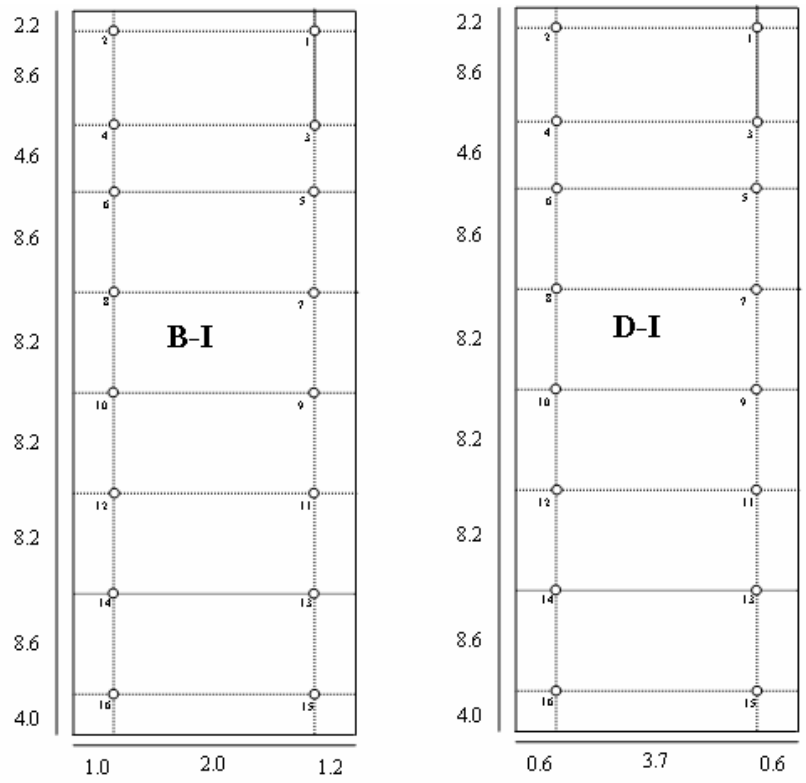
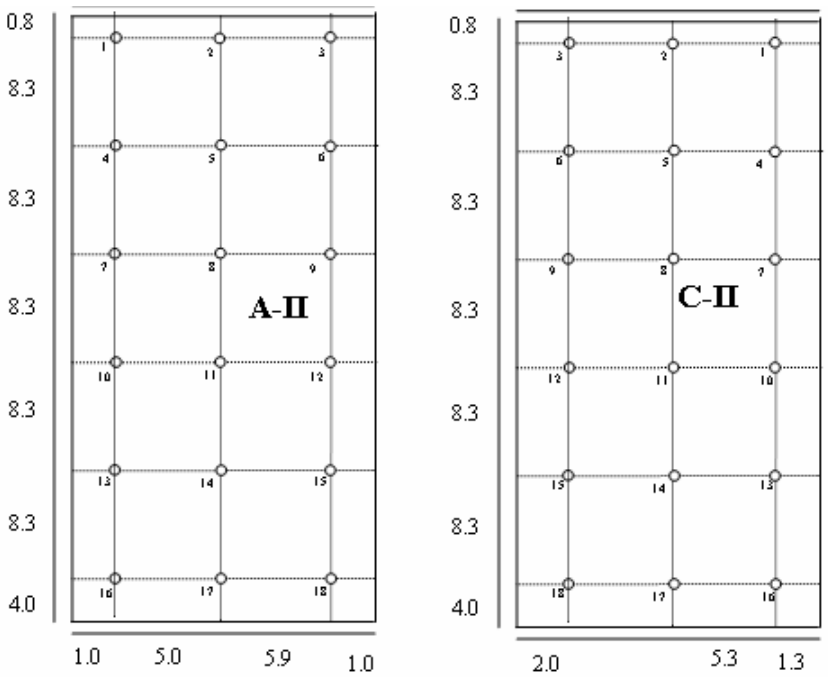



All dimensions are in cm.

**Fig 5.2b Locations of tapings on Faces B and Face D of Wing I of Signature building**




All dimensions are in cm.

**Fig 5.2c Locations of tapings on Faces A and Face C of Wing II of Signature building**



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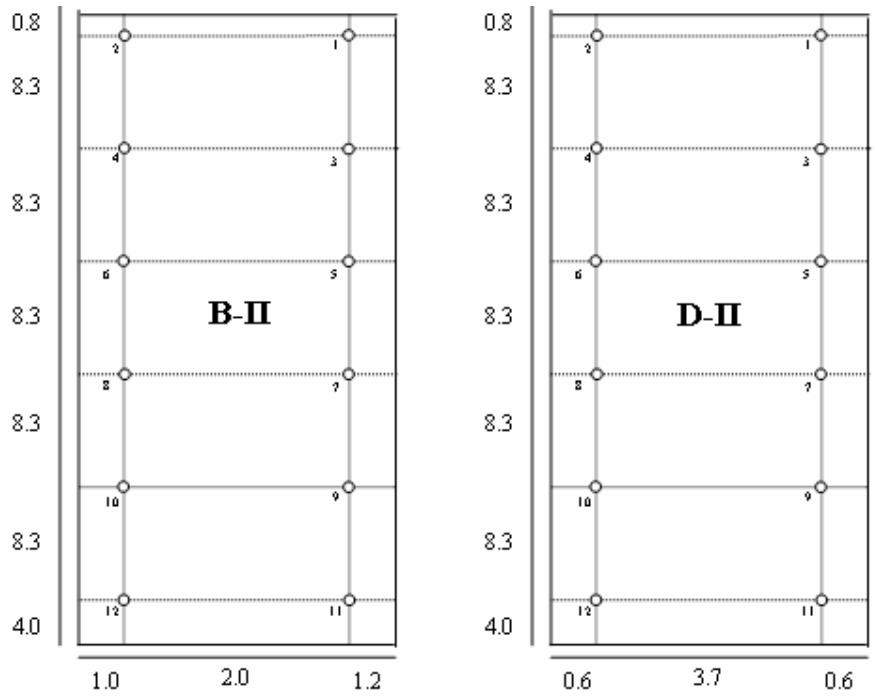
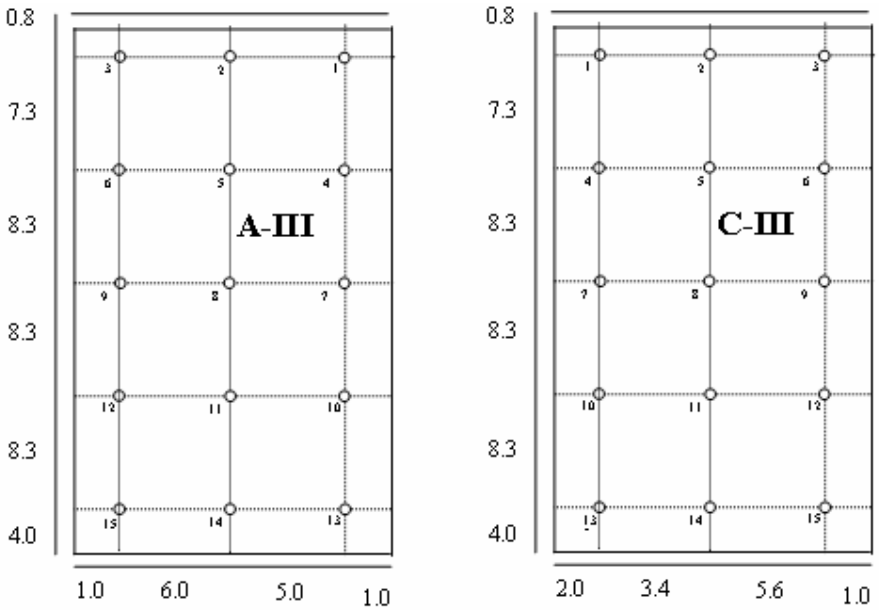



All dimensions are in cm.

**Fig 5.2d Locations of tapings on Faces B and Face D of Wing II of Signature building**




All dimensions are in cm.

**Fig 5.2e Locations of tapings on Faces A and Face C of Wing III of Signature building**



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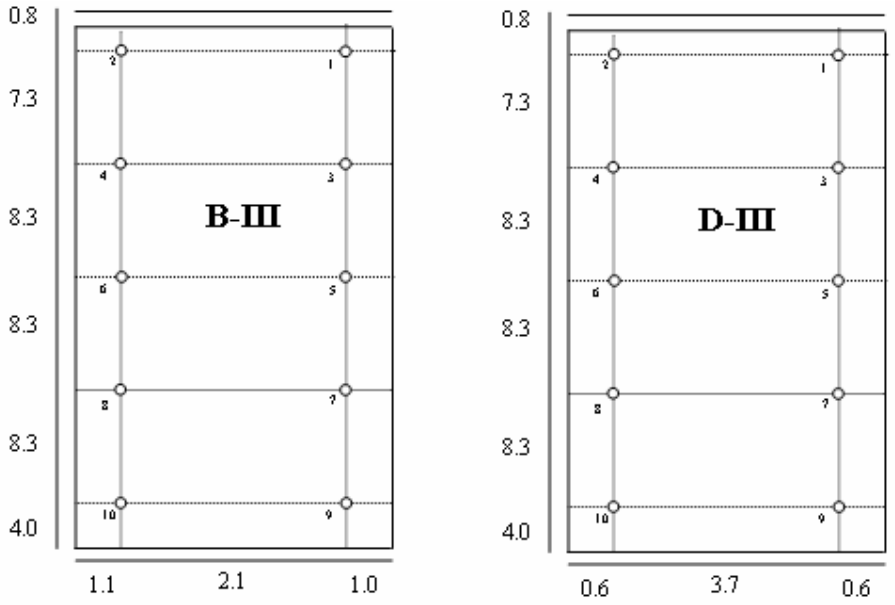


All dimensions are in cm.

**Fig 5.2f Locations of tapings on Faces B and Face D of Wing III of Signature building**

After all the locations were known and fixed and the Flow Simulation being maintained in the wind tunnel the experiment results for pressure variations are then obtained and are discussed below.

## 5.2 Experimental Results

**Table 5.1** shows the Typical Raw Data obtained from the wind tunnel testing for the mean, peak (minimum and maximum), rms values of the pressure coefficients from the on-line computer system recording all the values in wind tunnel on the building model with a velocity of 10.78 m/sec at the model height for all the faces of wing 1 of Signature building in ***Standalone condition*** when the angle of Wind Incidence is $\mathbf{0^o}$

**Table 5.2** shows the Typical Raw Data obtained from the wind tunnel testing for the mean, peak (minimum and maximum), rms values of the pressure coefficients from the on-line computer system recording all the values in wind tunnel on the building model with a velocity of 10.78 m/sec at the model height fo r all the faces of wing 1 of Signature building in ***Interference condition*** when the angle of Wind Incidence is $\mathbf{0^o}$



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Similarly in the same way the raw data is obtained for the major critical angles as obtained in the Dynamic analysis which are **45°, 90°, 195°, 345°** which are important for the design of claddings of the building.

**Table 5.3a, Table 5.3b, Table 5.3c, Table 5.3d** shows the Typical Process Data converted into 3 Gust from Mean Hourly and variation in the values of mean, peak (minimum and maximum), rms at the Reference height (10m) with a conversion factor of **0.794** and assembling the Maximum and the Minimum values for particular building of Signature Towers named Wing I when the Angle of Incidence are **45°, 90o, 195o, 345o** (the major critical angles) and its respective faces for both Standalone and Interference Condition.

In the tables **Table 5.3a, Table 5.3b, Table 5.3c, Table 5.3d** it has been observed that at some faces of the Wing I for a particular Angle of Wind Incidence the suction is there showing the maximum negative values of the pressure coefficients and at some faces pressure is severe showing the maximum positive values. Even the difference in the coefficients of pressure in both the cases of suction and pressure for Standalone and Interference is observed for the same faces of the same building for different Major Critical Angles of Wind Incidence is seen.

On the similar basis the values for the pressure coefficients for maximum values of suction and pressure are determined for Wing II, Wing III and Central Tower of the Signature Towers.

For the Structural design maximum of all the values of Pressure coefficients irrespective of the sign (either pressure or suction) from all Major Critical Angles in both Standalone and Interference Conditions for the Wing I, Wing II, Wing III and Central Tower of Signature Building has been tabulated in **Table 5.4a, Table 5.4b, Table 5.4c, Table 5.4d respectively as the All Azimax Values by** considering particular taping locations at certain level, thereby dividing the buildings into certain levels with taking the Maximum Pressure Coefficient Value of all the taping locations coming under consideration at that particular level for both Standalone and Interference Conditions.

For the reference of designer these pressure coefficients obtained can be plotted in form of contours as shown in **Fig 5.3** to know the pressure distribution at a particular face of the building for the ease of design. **Fig 5.3** typical pressure distributions for Faces A, B, C and D for Wing I of Signature Towers with reference from the Values of pressure coefficients given in **Table5.3a.**



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**Table 5.1 Typical Raw Data for the mean, peak (minimum and maximum), rms values of the pressure coefficients on the building model in Standalone condition (Wing-I, Angle of Wind Incidence = $0^{o}$)**

| | | | WING 1 (Standalone) | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | Angle of Wind Incidence = 0 | | | | | | |
| | FACE-A | | | | | FACE-B | | | |
| TAPPING NO. | Cpmean | Cpmax | Cpmin | CPrms | TAPPING NO | Cpmean | Cpmax | Cpmin | CPrms |
| A1 | -0.3702 | 0.173702 | -0.8644 | 0.132267 | B1 | 0.095006 | 0.360826 | -0.26687 | 0.084475 |
| A2 | -0.36753 | -0.04799 | -0.7964 | 0.114325 | B2 | 0.065919 | 0.321281 | -0.28227 | 0.085953 |
| A3 | -0.30325 | 0.029767 | -0.69449 | 0.10517 | B3 | -0.10352 | 0.189756 | -0.55865 | 0.101915 |
| A4 | -0.34435 | 0.235505 | -0.85088 | 0.136194 | B4 | -0.07378 | 0.215033 | -0.55751 | 0.09704 |
| A5 | -0.31618 | 0.055961 | -0.86143 | 0.13129 | B5 | -0.08002 | 0.235022 | -0.56166 | 0.096606 |
| A6 | -0.29366 | 0.201513 | -0.86073 | 0.127326 | B6 | -0.04354 | 0.259188 | -0.46507 | 0.096402 |
| A7 | -0.30309 | 0.119672 | -0.77358 | 0.121822 | B7 | -0.0309 | 0.207886 | -0.46809 | 0.089064 |
| A8 | -0.30611 | 0.104293 | -0.8131 | 0.123369 | B8 | -0.03293 | 0.235578 | -0.41626 | 0.088764 |
| A9 | -0.44065 | 0.056347 | -1.24732 | 0.149657 | B9 | -0.18386 | 0.166266 | -0.65456 | 0.094842 |
| A10 | -0.22792 | 0.084449 | -0.78466 | 0.120808 | B10 | -0.05382 | 0.177589 | -0.32939 | 0.079874 |
| A11 | -0.26125 | 0.088432 | -0.85311 | 0.129028 | B11 | -0.0719 | 0.200475 | -0.42722 | 0.085639 |
| A12 | -0.22197 | 0.104849 | -1.00568 | 0.12861 | B12 | -0.04183 | 0.187511 | -0.34361 | 0.080382 |
| A13 | -0.23562 | 0.082638 | -0.8589 | 0.126407 | B13 | -0.05269 | 0.215371 | -0.43646 | 0.086999 |
| A14 | -0.20167 | 0.141013 | -0.82467 | 0.125788 | B14 | -0.0225 | 0.270825 | -0.33273 | 0.082443 |
| A15 | -0.2564 | 0.087128 | -1.14411 | 0.141611 | B15 | -0.04387 | 0.231087 | -0.39661 | 0.083094 |
| A16 | -0.20848 | 0.07781 | -0.74302 | 0.112489 | B16 | -0.04195 | 0.215057 | -0.38849 | 0.076078 |
| A17 | -0.28868 | 0.039472 | -1.04692 | 0.143129 | | | | | |
| A18 | -0.29285 | 0.094033 | -1.28206 | 0.15861 | | | | | |
| A19 | -0.24789 | 0.058472 | -0.93135 | 0.127664 | | | | | |
| A20 | -0.2389 | 0.078099 | -0.88758 | 0.131956 | | | | | |
| A21 | -0.28049 | 0.065835 | -1.18955 | 0.158775 | | | | | |
| A22 | -0.25203 | 0.08201 | -0.9561 | 0.140702 | | | | | |
| A23 | -0.26803 | 0.061441 | -1.04909 | 0.14921 | | | | | |
| A24 | -0.28149 | 0.065642 | -1.21388 | 0.157211 | | | | | |
| | FACE C | | | | | FACE D | | | |
| TAPPING NO. | Cpmean | Cpmax | Cpmin | CPrms | TAPPING NO | Cpmean | Cpmax | Cpmin | CPrms |
| C1 | -0.04974 | 0.204579 | -0.42311 | 0.088766 | D1 | 0.344683 | 0.649854 | 0.167014 | 0.056535 |
| C2 | -0.18163 | 0.214864 | -0.58182 | 0.110846 | D2 | -0.1623 | 0.126866 | -0.47668 | 0.082176 |
| C3 | -0.24093 | 0.161558 | -0.68341 | 0.102508 | D3 | -0.14591 | 0.219547 | -0.553 | 0.092152 |
| C4 | -0.15985 | 0.172012 | -0.57639 | 0.116406 | D4 | -0.1268 | 0.173267 | -0.86484 | 0.078564 |
| C5 | -0.15893 | 0.248349 | -0.64491 | 0.121893 | D5 | -0.10929 | 0.173291 | -0.4544 | 0.082649 |
| C6 | -0.16255 | 0.219958 | -0.64915 | 0.1259 | D6 | -0.11353 | 0.177589 | -0.49839 | 0.084585 |
| C7 | -0.15732 | 0.161631 | -0.61091 | 0.106087 | D7 | -0.1122 | 0.170177 | -0.43337 | 0.078774 |
| C8 | -0.1594 | 0.169863 | -0.5544 | 0.105909 | D8 | -0.13442 | 0.164431 | -0.43912 | 0.080241 |
| C9 | -0.38465 | -0.00051 | -0.86962 | 0.125152 | D9 | -0.09482 | 0.158613 | -0.39665 | 0.079324 |
| C10 | -0.15906 | 0.14758 | -0.74567 | 0.119767 | D10 | -0.06964 | 0.197336 | -0.35793 | 0.073413 |
| C11 | -0.14992 | 0.185652 | -0.58689 | 0.110922 | D11 | -0.06809 | 0.192267 | -0.38714 | 0.079316 |
| C12 | -0.11571 | 0.19823 | -0.50189 | 0.103241 | D12 | -0.0734 | 0.187293 | -0.36797 | 0.074375 |
| C13 | -0.15344 | 0.156271 | -0.6887 | 0.12826 | D13 | -0.08287 | 0.161655 | -0.36947 | 0.079705 |
| C14 | -0.12115 | 0.18599 | -0.51413 | 0.110112 | D14 | -0.05708 | 0.227224 | -0.37633 | 0.075806 |
| C15 | -0.15308 | 0.157864 | -0.5664 | 0.111811 | D15 | -0.06425 | 0.200885 | -0.35438 | 0.079069 |
| C16 | -0.15011 | 0.120951 | -0.62745 | 0.115899 | D16 | -0.05476 | 0.229445 | -0.39825 | 0.074046 |
| C17 | -0.22079 | 0.115399 | -0.65715 | 0.118415 | | | | | |
| C18 | -0.18423 | 0.146156 | -0.60225 | 0.118048 | | | | | |
| C19 | -0.19009 | 0.103231 | -0.79002 | 0.127339 | | | | | |
| C20 | -0.15885 | 0.13872 | -0.60968 | 0.108498 | | | | | |
| C21 | -0.17373 | 0.136137 | -0.66055 | 0.118048 | | | | | |
| C22 | -0.16287 | 0.159071 | -0.80661 | 0.127662 | | | | | |
| C23 | -0.14051 | 0.139975 | -0.51186 | 0.094032 | | | | | |
| C24 | -0.19733 | 0.138044 | -0.65864 | 0.123335 | | | | | |



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**Table 5.2 Typical Raw Data for the mean, peak (minimum and maximum), rms values of the pressure coefficients on the building model in Interference condition (Wing-I , Angle of Wind Incidence = 0 $^{o}$)**

| | | | WING 1 (Interference) | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | Angle of Wind Incidence = 0 | | | | | | |
| | FACE-A | | | | | FACE-B | | | |
| APPING NO | Cpmean | Cpmax | Cpmin | CPrms | TAPPING NO | Cpmean | Cpmax | Cpmin | CPrms |
| A1 | -0.37802 | 0.098234 | -0.79502 | 0.123461 | B1 | 0.021904 | 0.328403 | -0.44414 | 0.081438 |
| A2 | -0.34147 | 0.025301 | -0.77139 | 0.106283 | B2 | 0.033009 | 0.338084 | -0.41032 | 0.082353 |
| A3 | -0.27231 | 0.069288 | -0.65497 | 0.100282 | B3 | -0.15673 | 0.191036 | -0.77464 | 0.099597 |
| A4 | -0.31401 | 0.18937 | -0.82459 | 0.130044 | B4 | -0.13167 | 0.169477 | -0.72378 | 0.098056 |
| A5 | -0.27576 | 0.105018 | -0.78824 | 0.124686 | B5 | -0.1274 | 0.158637 | -0.75876 | 0.093996 |
| A6 | -0.27186 | 0.097051 | -0.74792 | 0.119308 | B6 | -0.11147 | 0.180751 | -0.85735 | 0.094142 |
| A7 | -0.2746 | 0.128073 | -0.74104 | 0.114647 | B7 | -0.09421 | 0.178868 | -0.52125 | 0.087497 |
| A8 | -0.28022 | 0.07076 | -0.75007 | 0.113603 | B8 | -0.10367 | 0.177371 | -0.54689 | 0.088597 |
| A9 | -0.51149 | -0.0773 | -1.26026 | 0.1463 | B9 | -0.31836 | 0.18768 | -1.0677 | 0.114461 |
| A10 | -0.21718 | 0.083072 | -0.66533 | 0.107182 | B10 | -0.10887 | 0.165035 | -0.58337 | 0.086608 |
| A11 | -0.21424 | 0.091426 | -0.70526 | 0.113016 | B11 | -0.12456 | 0.170442 | -0.55382 | 0.093659 |
| A12 | -0.20662 | 0.100527 | -0.7203 | 0.115291 | B12 | -0.08306 | 0.172905 | -0.5755 | 0.088489 |
| A13 | -0.19983 | 0.125562 | -0.64698 | 0.111389 | B13 | -0.09756 | 0.203227 | -0.52103 | 0.094529 |
| A14 | -0.20003 | 0.090243 | -0.63402 | 0.110759 | B14 | -0.07902 | 0.191567 | -0.50855 | 0.089393 |
| A15 | -0.25171 | 0.076796 | -0.8406 | 0.127427 | B15 | -0.10576 | 0.245814 | -0.55087 | 0.097386 |
| A16 | -0.2017 | 0.062769 | -0.63735 | 0.099817 | B16 | -0.08874 | 0.219813 | -0.48031 | 0.083414 |
| A17 | -0.30175 | 0.010043 | -0.85907 | 0.125712 | | | | | |
| A18 | -0.27734 | 0.093864 | -1.08909 | 0.142028 | | | | | |
| A19 | -0.23527 | 0.061731 | -0.7591 | 0.111743 | | | | | |
| A20 | -0.22395 | 0.105887 | -0.71494 | 0.114956 | | | | | |
| A21 | -0.26555 | 0.096375 | -1.0383 | 0.143231 | | | | | |
| A22 | -0.22781 | 0.139975 | -0.8257 | 0.123059 | | | | | |
| A23 | -0.24558 | 0.109918 | -0.95233 | 0.13025 | | | | | |
| A24 | -0.27701 | 0.068056 | -0.99419 | 0.14203 | | | | | |
| | FACE C | | | | | FACE D | | | |
| APPING NO | Cpmean | Cpmax | Cpmin | CPrms | TAPPING NO | Cpmean | Cpmax | Cpmin | CPrms |
| C1 | -0.01279 | 0.247745 | -0.33166 | 0.087602 | D1 | 0.306756 | 0.514441 | -0.4271 | 0.055541 |
| C2 | -0.18188 | 0.208273 | -0.61256 | 0.112145 | D2 | -0.24368 | 0.022331 | -0.55708 | 0.084285 |
| C3 | -0.25892 | 0.196443 | -0.67267 | 0.10414 | D3 | -0.21936 | 0.113443 | -0.73153 | 0.100805 |
| C4 | -0.15766 | 0.185555 | -0.58699 | 0.111274 | D4 | -0.19576 | 0.135292 | -0.51654 | 0.084312 |
| C5 | -0.15726 | 0.265948 | -0.62731 | 0.119326 | D5 | -0.17265 | 0.090025 | -0.4411 | 0.080926 |
| C6 | -0.13895 | 0.267686 | -0.57728 | 0.121623 | D6 | -0.18438 | 0.112719 | -0.56326 | 0.091817 |
| C7 | -0.13061 | 0.178264 | -0.546 | 0.09875 | D7 | -0.17216 | 0.112091 | -0.44317 | 0.077502 |
| C8 | -0.12261 | 0.170273 | -0.45742 | 0.09937 | D8 | -0.21372 | 0.04925 | -0.57844 | 0.090332 |
| C9 | -0.27903 | 0.056203 | -0.74048 | 0.112904 | D9 | -0.15154 | 0.094685 | -0.4123 | 0.076416 |
| C10 | -0.13079 | 0.13225 | -0.761 | 0.108569 | D10 | -0.1313 | 0.137126 | -0.46642 | 0.083074 |
| C11 | -0.1234 | 0.160592 | -0.58781 | 0.103766 | D11 | -0.11758 | 0.134157 | -0.37283 | 0.07561 |
| C12 | -0.09629 | 0.170563 | -0.50541 | 0.098396 | D12 | -0.15471 | 0.146107 | -0.50573 | 0.086713 |
| C13 | -0.14674 | 0.184759 | -0.73264 | 0.116726 | D13 | -0.13875 | 0.102314 | -0.42881 | 0.077077 |
| C14 | -0.0762 | 0.167738 | -0.4841 | 0.103646 | D14 | -0.12644 | 0.236519 | -0.4636 | 0.08661 |
| C15 | -0.1261 | 0.164407 | -0.55985 | 0.106494 | D15 | -0.11851 | 0.139299 | -0.39182 | 0.075782 |
| C16 | -0.11492 | 0.144417 | -0.70055 | 0.108473 | D16 | -0.11438 | 0.14379 | -0.45976 | 0.085598 |
| C17 | -0.1066 | 0.184831 | -0.61185 | 0.108499 | | | | | |
| C18 | -0.11346 | 0.186762 | -0.58578 | 0.109785 | | | | | |
| C19 | -0.09934 | 0.174402 | -0.71885 | 0.115498 | | | | | |
| C20 | -0.0754 | 0.172567 | -0.55169 | 0.10169 | | | | | |
| C21 | -0.11925 | 0.142848 | -0.60555 | 0.11077 | | | | | |
| C22 | -0.10673 | 0.168608 | -0.72465 | 0.114326 | | | | | |
| C23 | -0.10319 | 0.127325 | -0.50037 | 0.093273 | | | | | |
| C24 | -0.16226 | 0.135509 | -0.70946 | 0.114311 | | | | | |



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**Table 5.3a Process Data in 3 Gust from Mean Hourly at the Reference height (10m) for Wing I when the Angle of Incidence is 45 $^{o}$ for both Standalone and Interference Condition.**

| | | | WING 1 OF INCIDENCE--45 | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | FACE A | | | | | | FACE B | | |
| | Standalone | | Interference | | | Standalone | | Interference | |
| APPING N | Cpmax | Cpmin | Cpmax | Cpmin | TAPPING N | Cpmax | Cpmin | Cpmax | Cpmin |
| A1 | 0.597969 | -0.0346 | 0.538047 | -0.03701 | B1 | 0.459015 | -0.67194 | 0.605789 | -0.46766 |
| A2 | 0.632798 | -0.03811 | 0.652657 | -0.01825 | B2 | 0.446785 | -0.70334 | 0.613476 | -0.45997 |
| A3 | 0.752047 | -0.03387 | 0.745089 | 0.112521 | B3 | 0.438197 | -0.99946 | 0.6309 | -0.80675 |
| A4 | 0.647041 | 0.014472 | 0.667915 | 0.054516 | B4 | 0.362423 | -0.84521 | 0.49877 | -0.55551 |
| A5 | 0.715339 | 0.025264 | 0.715952 | 0.083384 | B5 | 0.569982 | -1.27022 | 0.643514 | -0.83248 |
| A6 | 0.79257 | 0.064158 | 0.747964 | 0.096227 | B6 | 0.416652 | -0.7143 | 0.54606 | -0.56573 |
| A7 | 0.650912 | -0.00082 | 0.619246 | 0.063352 | B7 | 0.395086 | -1.04257 | 0.544565 | -0.68223 |
| A8 | 0.696209 | 0.006134 | 0.650415 | 0.037015 | B8 | 0.397904 | -1.28894 | 0.524208 | -0.72176 |
| A9 | 0.830658 | -0.10861 | 0.72454 | -0.06138 | B9 | 0.151184 | -1.42065 | 0.398212 | -1.23113 |
| A10 | 0.737958 | 0.10539 | 0.698528 | 0.161803 | B10 | 0.236849 | -1.02829 | 0.246049 | -0.76989 |
| A11 | 0.836964 | 0.127721 | 0.724329 | 0.149267 | B11 | 0.216683 | -0.95261 | 0.269512 | -0.89978 |
| A12 | 0.773324 | 0.121588 | 0.693218 | 0.156493 | B12 | 0.225559 | -0.98207 | 0.271468 | -0.78281 |
| A13 | 0.813195 | 0.180627 | 0.789771 | 0.214709 | B13 | 0.24768 | -0.92161 | 0.333881 | -0.7204 |
| A14 | 0.802039 | 0.150302 | 0.685052 | 0.129159 | B14 | 0.234204 | -0.9926 | 0.286285 | -0.59548 |
| A15 | 0.855098 | 0.107517 | 0.731881 | 0.137651 | B15 | 0.202652 | -1.27334 | 0.29102 | -1.03162 |
| A16 | 0.751855 | 0.157625 | 0.586564 | 0.088176 | B16 | 0.209093 | -0.69184 | 0.212869 | -0.57305 |
| A17 | 0.702247 | -0.007 | 0.640543 | 0.06548 | | | | | |
| A18 | 0.764736 | 0.055493 | 0.707097 | 0.055359 | | | | | |
| A19 | 0.659826 | 0.065596 | 0.604908 | 0.087352 | | | | | |
| A20 | 0.637073 | 0.062011 | 0.75498 | 0.064906 | | | | | |
| A21 | 0.684842 | -0.0244 | 0.670752 | 0.038184 | | | | | |
| A22 | 0.736022 | 0.065116 | 0.705371 | 0.130309 | | | | | |
| A23 | 0.700522 | 0.048784 | 0.681506 | 0.106444 | | | | | |
| A24 | 0.569676 | -0.06289 | 0.533255 | 0.015699 | | | | | |
| | FACE C | | | | | | FACE D | | |
| | Standalone | | Interference | | | Standalone | | Interference | |
| APPING N | Cpmax | Cpmin | Cpmax | Cpmin | TAPPING N | Cpmax | Cpmin | Cpmax | Cpmin |
| C1 | 0.124098 | -0.25928 | 0.19671 | -0.24417 | D1 | 0.611828 | 0.228453 | 0.580985 | 0.235948 |
| C2 | 0.05559 | -0.46197 | 0.069525 | -0.44803 | D2 | 0.541613 | 0.024057 | 0.554456 | 0.056068 |
| C3 | 0.051602 | -0.46595 | 0.079301 | -0.43825 | D3 | 0.231827 | -0.17072 | 0.262592 | -0.12078 |
| C4 | 0.079071 | -0.45765 | 0.128162 | -0.37022 | D4 | 0.463443 | 0.04173 | 0.490796 | 0.069084 |
| C5 | 0.082177 | -0.43538 | 0.076981 | -0.40224 | D5 | 0.233437 | -0.14994 | 0.263167 | -0.12021 |
| C6 | 0.078803 | -0.45792 | 0.09753 | -0.40086 | D6 | 0.562717 | 0.045162 | 0.511211 | 0.089499 |
| C7 | 0.070829 | -0.42756 | 0.103205 | -0.39518 | D7 | 0.250133 | -0.13324 | 0.261519 | -0.12186 |
| C8 | 0.058196 | -0.45936 | 0.096859 | -0.36319 | D8 | 0.609777 | 0.034714 | 0.614166 | 0.058273 |
| C9 | -0.11542 | -0.70965 | -0.03205 | -0.58794 | D9 | 0.240951 | -0.23827 | 0.266867 | -0.15485 |
| C10 | 0.078841 | -0.64957 | 0.124175 | -0.41255 | D10 | 0.578397 | 0.080011 | 0.664772 | 0.128047 |
| C11 | 0.070733 | -0.50433 | 0.12751 | -0.40921 | D11 | 0.286841 | -0.17321 | 0.336546 | -0.1235 |
| C12 | 0.08072 | -0.45601 | 0.135427 | -0.34379 | D12 | 0.570423 | 0.091205 | 0.595228 | 0.116009 |
| C13 | 0.124079 | -0.60433 | 0.146699 | -0.42836 | D13 | 0.224198 | -0.21668 | 0.292094 | -0.20629 |
| C14 | 0.109339 | -0.52323 | 0.152353 | -0.32687 | D14 | 0.563791 | 0.065404 | 0.571171 | 0.111121 |
| C15 | 0.087007 | -0.52639 | 0.130539 | -0.36785 | D15 | 0.312853 | -0.24304 | 0.302291 | -0.54113 |
| C16 | 0.096035 | -0.57487 | 0.114667 | -0.36455 | D16 | 0.565554 | 0.086336 | 0.612556 | 0.114169 |
| C17 | 0.110796 | -0.65595 | 0.08925 | -0.42831 | | | | | |
| C18 | 0.135217 | -0.63153 | 0.12912 | -0.4076 | | | | | |
| C19 | 0.139472 | -0.56977 | 0.157644 | -0.43659 | | | | | |
| C20 | 0.186819 | -0.56076 | 0.137018 | -0.36137 | | | | | |
| C21 | 0.127262 | -0.62032 | 0.113421 | -0.40413 | | | | | |
| C22 | 0.183809 | -0.64045 | 0.153044 | -0.53703 | | | | | |
| C23 | 0.245321 | -0.34891 | 0.235277 | -0.18644 | | | | | |
| C24 | 0.128776 | -0.65714 | 0.088425 | -0.46747 | | | | | |



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**Table 5.3b Process Data in 3 Gust from Mean Hourly at the Reference height (10m) for Wing I when the Angle of Incidence is 90$^{o}$ for both Standalone and Interference Condition.**

| | | | WING 1 | ANGLE OF INCIDENCE--90 | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | FACE A | | | | | FACE B | | | |
| | Standalone | | Interference | | | Standalone | | Interference | |
| APPING N | Cpmax | Cpmin | Cpmax | Cpmin | TAPPING N | Cpmax | Cpmin | Cpmax | Cpmin |
| A1 | 0.885385 | 0.099467 | 0.768072 | -0.11369 | B1 | 0.190652 | -0.19272 | 0.164908 | -0.23763 |
| A2 | 0.923722 | 0.118636 | 0.844344 | -0.07575 | B2 | 0.178423 | -0.18578 | 0.172595 | -0.26829 |
| A3 | 0.858395 | 0.110815 | 0.802595 | -0.04083 | B3 | 0.054823 | -0.53941 | 0.017501 | -0.50005 |
| A4 | 0.924393 | 0.138475 | 0.89794 | 0.016179 | B4 | 0.074892 | -0.40433 | 0.038721 | -0.4405 |
| A5 | 0.930297 | 0.201885 | 0.869302 | 0.006709 | B5 | 0.090764 | -0.31178 | 0.068452 | -0.41077 |
| A6 | 0.911244 | 0.144494 | 0.786301 | -0.05712 | B6 | 0.109952 | -0.31176 | 0.066841 | -0.39321 |
| A7 | 0.883468 | 0.174225 | 0.810933 | -0.01332 | B7 | 0.107555 | -0.27582 | 0.084515 | -0.35637 |
| A8 | 0.91571 | 0.187297 | 0.746258 | -0.078 | B8 | 0.091205 | -0.34968 | 0.064158 | -0.35755 |
| A9 | 0.687947 | -0.09797 | 0.532853 | -0.34891 | B9 | -0.0405 | -0.51972 | -0.06184 | -0.6369 |
| A10 | 0.799087 | -0.3127 | 0.736865 | 0.008453 | B10 | 0.083499 | -0.41489 | 0.054363 | -0.40569 |
| A11 | 0.776161 | 0.162762 | 0.70516 | -0.02325 | B11 | 0.082502 | -0.45422 | 0.058656 | -0.40139 |
| A12 | 0.731038 | 0.117638 | 0.559037 | -0.07353 | B12 | 0.091377 | -0.38784 | 0.07978 | -0.38027 |
| A13 | 0.787049 | 0.192818 | 0.751434 | 0.04219 | B13 | 0.113498 | -0.55741 | 0.084687 | -0.43287 |
| A14 | 0.747676 | 0.172615 | 0.608377 | -0.06253 | B14 | 0.13836 | -0.49421 | 0.132935 | -0.44213 |
| A15 | 0.68835 | 0.07495 | 0.559363 | -0.11154 | B15 | 0.087639 | -0.75578 | 0.080164 | -0.43739 |
| A16 | 0.671462 | 0.134737 | 0.605732 | -0.06517 | B16 | 0.074912 | -0.61516 | 0.117025 | -0.38136 |
| A17 | 0.606403 | 0.069678 | 0.52553 | -0.0687 | | | | | |
| A18 | 0.544239 | 0.045852 | 0.49624 | -0.05965 | | | | | |
| A19 | 0.572206 | 0.05465 | 0.489896 | -0.04683 | | | | | |
| A20 | 0.569561 | 0.071173 | 0.44828 | -0.05011 | | | | | |
| A21 | 0.597662 | 0.041768 | 0.459896 | -0.13433 | | | | | |
| A22 | 0.650817 | 0.094923 | 0.57119 | 0.015296 | | | | | |
| A23 | 0.643456 | 0.087562 | 0.508987 | -0.02774 | | | | | |
| A24 | 0.50272 | -0.16819 | 0.073205 | -0.52103 | | | | | |
| | FACE C | | | | | FACE D | | | |
| | Standalone | | Interference | | | Standalone | | Interference | |
| APPING N | Cpmax | Cpmin | Cpmax | Cpmin | TAPPING N | Cpmax | Cpmin | Cpmax | Cpmin |
| C1 | 0.104929 | -0.24011 | 0.235047 | -0.26334 | D1 | 0.630997 | 0.26679 | 0.600154 | 0.293454 |
| C2 | 0.05559 | -0.40446 | -0.06466 | -0.44803 | D2 | 0.656625 | 0.081563 | 0.631131 | 0.132743 |
| C3 | 0.051602 | -0.40845 | -0.05488 | -0.45742 | D3 | 0.308502 | -0.11321 | 0.243424 | -0.17829 |
| C4 | 0.040734 | -0.53433 | -0.04436 | -0.50441 | D4 | 0.597624 | 0.080068 | 0.567471 | 0.126591 |
| C5 | 0.043839 | -0.41621 | -0.03803 | -0.47891 | D5 | 0.290943 | -0.1116 | 0.282336 | -0.0627 |
| C6 | 0.040465 | -0.40042 | -0.01748 | -0.43919 | D6 | 0.639392 | 0.025993 | 0.664561 | 0.147005 |
| C7 | 0.089998 | -0.33171 | -0.01181 | -0.39518 | D7 | 0.326808 | -0.07574 | 0.319025 | -0.04518 |
| C8 | 0.077365 | -0.32518 | 0.001016 | -0.36319 | D8 | 0.628946 | 0.073052 | 0.671672 | 0.134948 |
| C9 | -0.05791 | -0.49879 | -0.1854 | -0.62628 | D9 | 0.279289 | -0.12326 | 0.209361 | -0.13568 |
| C10 | 0.078841 | -0.30453 | 0.028332 | -0.35504 | D10 | 0.616735 | 0.099179 | 0.549759 | 0.108878 |
| C11 | 0.070733 | -0.31264 | 0.070004 | -0.33254 | D11 | 0.325179 | -0.0582 | 0.279039 | -0.08517 |
| C12 | 0.08072 | -0.30266 | 0.058752 | -0.28628 | D12 | 0.608761 | 0.091205 | 0.576059 | 0.077671 |
| C13 | 0.124079 | -0.3168 | 0.031686 | -0.35169 | D13 | 0.243367 | -0.12084 | 0.234587 | -0.14879 |
| C14 | 0.109339 | -0.31237 | 0.075679 | -0.28853 | D14 | 0.602128 | 0.065404 | 0.552002 | 0.053615 |
| C15 | 0.067839 | -0.29637 | 0.053864 | -0.32951 | D15 | 0.274515 | -0.07052 | 0.263953 | -0.10025 |
| C16 | 0.076866 | -0.28734 | 0.037992 | -0.28788 | D16 | 0.623061 | 0.086336 | 0.55505 | 0.095001 |
| C17 | 0.072458 | -0.31092 | -0.00659 | -0.3708 | | | | | |
| C18 | 0.07771 | -0.32483 | 0.052445 | -0.36927 | | | | | |
| C19 | 0.081965 | -0.28224 | 0.0618 | -0.30241 | | | | | |
| C20 | 0.090975 | -0.31157 | 0.060343 | -0.28469 | | | | | |
| C21 | 0.069755 | -0.35196 | 0.055915 | -0.34663 | | | | | |
| C22 | 0.087965 | -0.33375 | 0.095537 | -0.32618 | | | | | |
| C23 | 0.187815 | -0.19556 | 0.139434 | -0.2056 | | | | | |
| C24 | 0.090438 | -0.36961 | 0.01175 | -0.39079 | | | | | |



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**Table 5.3c Process Data in 3 Gust from Mean Hourly at the Reference height (10m) for Wing I when the Angle of Incidence is 195° for both Standalone and Interference Condition.**

| | | | WING 1 | ANGLE OF INCIDENCE--195 | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | FACE A | | | | | | FACE B | | |
| | Standalone | | Interference | | | Standalone | | Interference | |
| APPING NO | Cpmax | Cpmin | Cpmax | Cpmin | TAPPING NO | Cpmax | Cpmin | Cpmax | Cpmin |
| A1 | 0.061244 | -0.62883 | 0.173842 | -0.74626 | B1 | 0.267327 | -0.21189 | 0.203246 | -0.23763 |
| A2 | 0.019399 | -0.57483 | 0.020089 | -0.5358 | B2 | 0.255097 | -0.20495 | 0.268439 | -0.24912 |
| A3 | 0.042804 | -0.5706 | 0.035846 | -0.50088 | B3 | 0.131498 | -0.40523 | 0.094176 | -0.50005 |
| A4 | 0.110316 | -0.65643 | 0.112022 | -0.61639 | B4 | 0.151567 | -0.36599 | 0.115396 | -0.51717 |
| A5 | 0.082771 | -0.56897 | 0.083384 | -0.54918 | B5 | 0.205776 | -0.46513 | 0.125958 | -0.46827 |
| A6 | 0.083326 | -0.58758 | 0.05789 | -0.65135 | B6 | 0.186627 | -0.36927 | 0.143516 | -0.45071 |
| A7 | 0.075851 | -0.55672 | 0.082521 | -0.60755 | B7 | 0.18423 | -0.33333 | 0.142021 | -0.47138 |
| A8 | 0.06364 | -0.5881 | 0.037015 | -0.59555 | B8 | 0.16788 | -0.31134 | 0.140833 | -0.33839 |
| A9 | -0.07027 | -1.06705 | -0.09972 | -1.03898 | B9 | 0.074509 | -0.76892 | 0.072343 | -0.69441 |
| A10 | 0.067053 | -0.50801 | 0.04679 | -0.48993 | B10 | 0.179342 | -0.45323 | 0.150207 | -0.46319 |
| A11 | 0.051046 | -0.54318 | 0.053423 | -0.50247 | B11 | 0.12084 | -0.49256 | 0.135331 | -0.43973 |
| A12 | 0.08325 | -0.54932 | 0.06065 | -0.59109 | B12 | 0.168052 | -0.44535 | 0.156455 | -0.34193 |
| A13 | 0.084783 | -0.52862 | 0.080527 | -0.5137 | B13 | 0.151836 | -0.53824 | 0.161362 | -0.37536 |
| A14 | 0.111964 | -0.50143 | 0.052484 | -0.50341 | B14 | 0.157529 | -0.51338 | 0.190442 | -0.32711 |
| A15 | 0.06918 | -0.6784 | 0.041807 | -0.78245 | B15 | 0.221821 | -0.6216 | 0.13767 | -0.43739 |
| A16 | 0.061781 | -0.47494 | 0.049839 | -0.48689 | B16 | 0.170755 | -0.55766 | 0.1937 | -0.34302 |
| A17 | 0.031341 | -0.58206 | 0.046312 | -0.75877 | | | | | |
| A18 | 0.055493 | -0.84544 | 0.074528 | -1.17144 | | | | | |
| A19 | 0.046427 | -0.52864 | 0.068183 | -0.66023 | | | | | |
| A20 | 0.062011 | -0.51305 | 0.084074 | -0.64434 | | | | | |
| A21 | 0.071442 | -0.81032 | 0.057353 | -0.99693 | | | | | |
| A22 | 0.065116 | -0.56745 | 0.091971 | -0.69395 | | | | | |
| A23 | 0.048784 | -0.62212 | 0.087275 | -0.75615 | | | | | |
| A24 | 0.090457 | -0.65712 | 0.073205 | -0.86606 | | | | | |
| | FACE C | | | | | | FACE D | | |
| | Standalone | | Interference | | | Standalone | | Interference | |
| APPING NO | Cpmax | Cpmin | Cpmax | Cpmin | TAPPING NO | Cpmax | Cpmin | Cpmax | Cpmin |
| C1 | 0.200773 | -0.31678 | 0.235047 | -0.28251 | D1 | 0.458478 | 0.132609 | 0.427635 | 0.044261 |
| C2 | 0.20894 | -0.4428 | 0.222875 | -0.44803 | D2 | 0.062394 | -0.39766 | 0.094406 | -0.53816 |
| C3 | 0.166615 | -0.54263 | 0.117638 | -0.51493 | D3 | 0.212658 | -0.55409 | 0.166749 | -0.50416 |
| C4 | 0.174915 | -0.5535 | 0.204837 | -0.46607 | D4 | 0.118405 | -0.41832 | 0.126591 | -0.4293 |
| C5 | 0.197189 | -0.51205 | 0.307006 | -0.47891 | D5 | 0.099256 | -0.37996 | 0.148155 | -0.3694 |
| C6 | 0.174647 | -0.49626 | 0.212543 | -0.47753 | D6 | 0.121837 | -0.45323 | 0.492043 | -0.46639 |
| C7 | 0.166673 | -0.40839 | 0.179879 | -0.45269 | D7 | 0.096783 | -0.36327 | 0.108169 | -0.37105 |
| C8 | 0.134871 | -0.5552 | 0.211872 | -0.40153 | D8 | 0.111389 | -0.42534 | 0.077442 | -0.47845 |
| C9 | -0.0004 | -0.88216 | 0.025456 | -0.58794 | D9 | 0.10677 | -0.35328 | 0.151855 | -0.38487 |
| C10 | 0.117179 | -0.53456 | 0.105007 | -0.41255 | D10 | 0.156685 | -0.36087 | 0.166384 | -0.29366 |
| C11 | 0.10907 | -0.56184 | 0.108341 | -0.37088 | D11 | 0.133491 | -0.34573 | 0.164027 | -0.31519 |
| C12 | 0.099888 | -0.49434 | 0.116258 | -0.34379 | D12 | 0.148711 | -0.38801 | 0.154346 | -0.34404 |
| C13 | 0.10491 | -0.56599 | 0.108361 | -0.4667 | D13 | 0.262535 | -0.3892 | 0.177081 | -0.32131 |
| C14 | 0.128507 | -0.44656 | 0.133184 | -0.32687 | D14 | 0.161247 | -0.39465 | 0.187796 | -0.34893 |
| C15 | 0.144513 | -0.48806 | 0.130539 | -0.38702 | D15 | 0.19784 | -0.39639 | 0.16811 | -0.38778 |
| C16 | 0.076866 | -0.51736 | 0.095498 | -0.38372 | D16 | 0.143842 | -0.35454 | 0.210013 | -0.28837 |
| C17 | 0.072458 | -0.65595 | 0.08925 | -0.3708 | | | | | |
| C18 | 0.135217 | -0.61236 | 0.090783 | -0.3501 | | | | | |
| C19 | 0.081965 | -0.53143 | 0.119306 | -0.37908 | | | | | |
| C20 | 0.110144 | -0.50326 | 0.117849 | -0.3422 | | | | | |
| C21 | 0.088924 | -0.67783 | 0.113421 | -0.34663 | | | | | |
| C22 | 0.107134 | -0.5446 | 0.095537 | -0.42202 | | | | | |
| C23 | 0.130309 | -0.44475 | 0.120265 | -0.30145 | | | | | |
| C24 | 0.090438 | -0.65714 | 0.088425 | -0.40996 | | | | | |



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**Table 5.3d Process Data in 3 Gust from Mean Hourly at the Reference height (10m) for Wing I when the Angle of Incidence is $345^{o}$ for both Standalone and Interference Condition.**

| | | | WING 1 | ANGLE OF INCIDENCE--345 | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | FACE A | | | | | | FACE B | | |
| | Standalone | | Interference | | | Standalone | | Interference | |
| APPING NO | Cpmax | Cpmin | Cpmax | Cpmin | TAPPING NO | Cpmax | Cpmin | Cpmax | Cpmin |
| A1 | 0.003738 | -0.72467 | -0.07535 | -0.86127 | B1 | 0.899896 | 0.286496 | 0.912489 | 0.203246 |
| A2 | 0.038568 | -0.74735 | 0.039258 | -1.18754 | B2 | 0.906835 | 0.274266 | 0.920176 | 0.24927 |
| A3 | -0.07221 | -0.68561 | -0.04083 | -0.86509 | B3 | 0.84074 | 0.112329 | 0.880094 | 0.055839 |
| A4 | 0.014472 | -0.57976 | -0.02216 | -0.76974 | B4 | 0.937485 | 0.189905 | 0.95882 | 0.172902 |
| A5 | 0.006096 | -0.62647 | -0.03163 | -0.81755 | B5 | 0.876682 | 0.109932 | 0.911876 | 0.106789 |
| A6 | 0.044989 | -0.70259 | -0.05712 | -0.90055 | B6 | 0.934208 | 0.148289 | 0.98694 | 0.143516 |
| A7 | 0.018345 | -0.55672 | -0.05166 | -0.72257 | B7 | 0.931811 | 0.18423 | 0.927939 | 0.16119 |
| A8 | 0.025302 | -0.6456 | -0.05883 | -0.82558 | B8 | 0.934629 | 0.148711 | 1.003425 | 0.121664 |
| A9 | 0.025571 | -1.16289 | -0.06138 | -1.30735 | B9 | 0.783752 | -0.05967 | 0.858261 | -0.04267 |
| A10 | 0.067053 | -0.56552 | 0.008453 | -0.79663 | B10 | 0.67773 | 0.045162 | 0.667762 | 0.054363 |
| A11 | 0.108553 | -0.60069 | 0.015086 | -0.77083 | B11 | 0.657565 | 0.063333 | 0.652887 | -0.03719 |
| A12 | 0.102419 | -0.49181 | 0.022312 | -0.7061 | B12 | 0.666439 | 0.072209 | 0.654843 | -0.0544 |
| A13 | 0.14229 | -0.52862 | 0.023021 | -0.59038 | B13 | 0.669391 | 0.036823 | 0.602243 | -0.03033 |
| A14 | 0.169471 | -0.57811 | 0.014147 | -0.81011 | B14 | 0.655915 | 0.023348 | 0.765503 | -0.11626 |
| A15 | 0.145855 | -0.65923 | 0.022639 | -0.66744 | B15 | 0.605196 | -0.14239 | 0.521044 | -0.30321 |
| A16 | 0.138456 | -0.45578 | 0.011501 | -0.58273 | B16 | 0.554129 | -0.0401 | 0.5004 | -0.13217 |
| A17 | 0.108016 | -0.77375 | 0.027143 | -0.70127 | | | | | |
| A18 | 0.132168 | -0.86461 | 0.055359 | -0.57721 | | | | | |
| A19 | 0.123102 | -0.62448 | 0.049014 | -0.56438 | | | | | |
| A20 | 0.119517 | -0.60889 | 0.026568 | -0.58683 | | | | | |
| A21 | 0.109779 | -0.67614 | 0.038184 | -0.7094 | | | | | |
| A22 | 0.141791 | -0.54828 | 0.053634 | -0.55977 | | | | | |
| A23 | 0.144628 | -0.58378 | 0.068106 | -0.56446 | | | | | |
| A24 | 0.20547 | -0.86798 | 0.073205 | -0.88523 | | | | | |
| | FACE C | | | | | | FACE D | | |
| | Standalone | | Interference | | | Standalone | | Interference | |
| APPING NO | Cpmax | Cpmin | Cpmax | Cpmin | TAPPING NO | Cpmax | Cpmin | Cpmax | Cpmin |
| C1 | 0.603317 | 0.066592 | 0.599253 | 0.100866 | D1 | 0.57349 | 0.26679 | 0.542647 | 0.255117 |
| C2 | 0.553976 | -0.15527 | 0.510406 | -0.17967 | D2 | 0.27325 | -0.16763 | 0.266925 | -0.21229 |
| C3 | 0.473315 | -0.06341 | 0.443507 | -0.07405 | D3 | 0.346839 | -0.01737 | 0.320099 | -0.02494 |
| C4 | 0.69247 | -0.26597 | 0.626549 | -0.19771 | D4 | 0.271755 | -0.14996 | 0.241602 | -0.16094 |
| C5 | 0.695576 | -0.09034 | 0.652044 | -0.01886 | D5 | 0.444293 | 0.041749 | 0.435686 | -0.0052 |
| C6 | 0.615527 | -0.01704 | 0.634255 | 0.059193 | D6 | 0.351861 | -0.14653 | 0.242849 | -0.17886 |
| C7 | 0.684228 | -0.12086 | 0.62076 | -0.10765 | D7 | 0.518495 | 0.077614 | 0.472375 | 0.012325 |
| C8 | 0.633259 | -0.03765 | 0.614415 | 0.058522 | D8 | 0.360583 | -0.23365 | 0.307466 | -0.24843 |
| C9 | 0.70884 | -0.03874 | 0.734699 | 0.025456 | D9 | 0.56682 | 0.10677 | 0.592735 | 0.113517 |
| C10 | 0.86476 | -0.13202 | 0.795081 | 0.009163 | D10 | 0.501722 | -0.11168 | 0.511422 | -0.10198 |
| C11 | 0.818313 | 0.070733 | 0.817585 | 0.146679 | D11 | 0.536034 | 0.114323 | 0.566571 | 0.106521 |
| C12 | 0.809131 | 0.08072 | 0.787164 | 0.116258 | D12 | 0.436242 | -0.11965 | 0.480215 | -0.15235 |
| C13 | 0.852491 | -0.02927 | 0.817604 | 0.012517 | D13 | 0.511728 | 0.090017 | 0.522118 | 0.081237 |
| C14 | 0.799413 | 0.071001 | 0.80409 | 0.152353 | D14 | 0.467947 | -0.14545 | 0.494496 | -0.15724 |
| C15 | 0.815419 | 0.106175 | 0.801445 | 0.168877 | D15 | 0.50454 | 0.101996 | 0.532316 | 0.129772 |
| C16 | 0.747773 | -0.07648 | 0.68973 | 0.037992 | D16 | 0.373867 | -0.08618 | 0.344193 | -0.15419 |
| C17 | 0.705026 | 0.110796 | 0.683481 | 0.165925 | | | | | |
| C18 | 0.767785 | 0.116048 | 0.704183 | 0.186627 | | | | | |
| C19 | 0.695365 | 0.043628 | 0.675199 | 0.157644 | | | | | |
| C20 | 0.646868 | 0.110144 | 0.635406 | 0.194524 | | | | | |
| C21 | 0.740661 | 0.127262 | 0.707653 | 0.209265 | | | | | |
| C22 | 0.720534 | 0.068797 | 0.689768 | 0.133875 | | | | | |
| C23 | 0.628696 | 0.091971 | 0.580314 | 0.158602 | | | | | |
| C24 | 0.723006 | 0.147944 | 0.682656 | 0.1651 | | | | | |



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**Table 5.4a All Azimax for Wing I of Signature Towers**

WING 1

FACE A

| LEVEL | STANDALONE | INTERFERENCE |
|---|---|---|
| | | |
| 4 | 0.924393 | 1.837208 |
| 3 | 1.565434 | 1.594149 |
| 2 | 1.017956 | 1.000627 |
| 1 | 0.963823 | 1.544176 |

FACE B

| LEVEL | STANDALONE | INTERFERENCE |
|---|---|---|
| | | |
| 4 | 0.979733 | 1.21478 |
| 3 | 1.323045 | 1.386819 |
| 2 | 0.935626 | 1.214282 |
| 1 | 0.798167 | 0.784327 |

FACE C

| LEVEL | STANDALONE | INTERFERENCE |
|---|---|---|
| | | |
| 4 | 1.075232 | 0.989164 |
| 3 | 1.288944 | 1.003425 |
| 2 | 1.420653 | 1.231131 |
| 1 | 1.273341 | 1.06996 |

FACE D

| LEVEL | STANDALONE | INTERFERENCE |
|---|---|---|
| | | |
| 4 | 0.809975 | 0.734335 |
| 3 | 0.705621 | 0.690841 |
| 2 | 0.74689 | 0.949945 |
| 1 | 0.623061 | 0.656299 |

**Table 5.4b All Azimax for Wing II of Signature Towers**

WING 2

FACE A

| LEVEL | STANDALONE | INTERFERENCE |
|---|---|---|
| | | |
| 3 | 1.131588 | 0.978641 |
| 2 | 0.862363 | 2.950336 |
| 1 | 0.804167 | 0.821974 |

FACE B

| LEVEL | STANDALONE | INTERFERENCE |
|---|---|---|
| | | |
| 3 | 0.98556 | 0.963536 |
| 2 | 1.267628 | 1.26575 |
| 1 | 1.066664 | 1.037642 |

FACE C

| LEVEL | STANDALONE | INTERFERENCE |
|---|---|---|
| | | |
| 3 | 1.046881 | 0.845801 |
| 2 | 0.923914 | 0.820269 |
| 1 | 0.817336 | 0.673723 |

FACE D

| LEVEL | STANDALONE | INTERFERENCE |
|---|---|---|
| | | |
| 3 | 0.666593 | 0.544641 |
| 2 | 0.667244 | 0.62513 |
| 1 | 0.65074 | 1.037642 |



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## Table 5.4c All Azimax for Wing III of Signature Towers

WING 3

FACE A

| LEVEL | STANDALONE | INTERFERENCE |
|---|---|---|
| | | |
| 3 | 2.547908 | 2.299214 |
| 2 | 1.821758 | 1.83508 |
| 1 | 0.929473 | 1.050082 |

FACE B

| LEVEL | STANDALONE | INTERFERENCE |
|---|---|---|
| | | |
| 3 | 1.900159 | 1.905506 |
| 2 | 1.224709 | 1.321781 |
| 1 | 1.220186 | 1.232224 |

FACE C

| LEVEL | STANDALONE | INTERFERENCE |
|---|---|---|
| | | |
| 3 | 1.652882 | 1.027808 |
| 2 | 1.777038 | 1.091544 |
| 1 | 1.009579 | 0.868881 |

FACE D

| LEVEL | STANDALONE | INTERFERENCE |
|---|---|---|
| | | |
| 3 | 1.51665 | 0.810934 |
| 2 | 1.415055 | 0.63136 |
| 1 | 0.97452 | 0.712329 |

## Table 5.4d All Azimax for Central Tower of Signature Towers

CENTRAL TOWER

FACE A

| LEVEL | STANDALONE | INTERFERENCE |
|---|---|---|
| | | |
| 3 | 1.88281 | 0.763452 |
| 2 | 1.609254 | 0.704126 |
| 1 | 0.97862 | 0.604888 |

FACE B

| LEVEL | STANDALONE | INTERFERENCE |
|---|---|---|
| | | |
| 3 | 0.72157 | 0.83026 |
| 2 | 1.838378 | 0.92738 |
| 1 | 1.15275 | 0.525013 |

FACE C

| LEVEL | STANDALONE | INTERFERENCE |
|---|---|---|
| | | |
| 3 | 0.779247 | 0.943658 |
| 2 | 0.805988 | 0.73493 |
| 1 | 1.547588 | 0.583381 |



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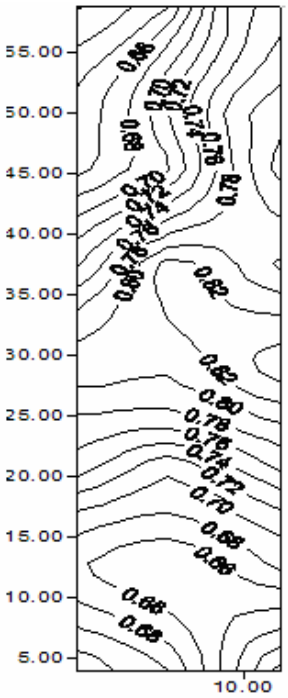
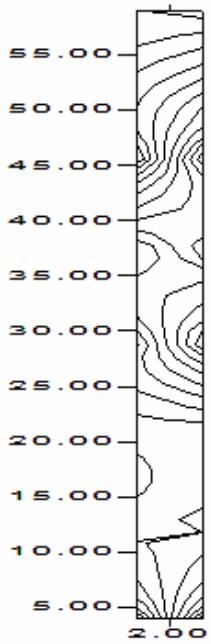
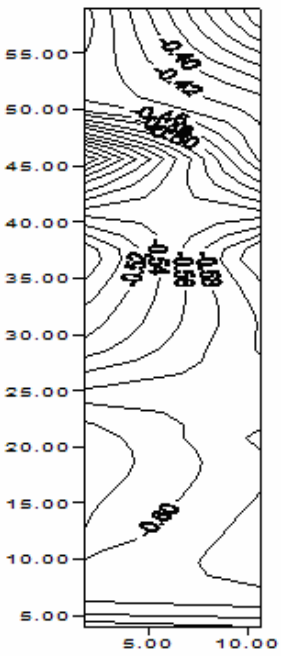
**Fig 5.3 Typical pressure distribution for Faces A, B, C and D in Standalone condition for Wing I of Signature Towers**



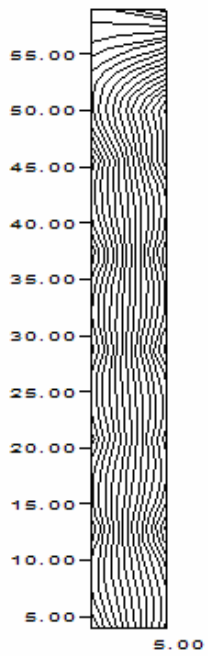

Maximum Pressure values in Standalone on face C of Wing I




Maximum Suction values in Standalone on face B of Wing I




Maximum Suction values in Standalone on face C of Wing I




Maximum Pressure values in Standalone on face C of Wing I



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## 5.3 Result Discussion for Pressure Measurement studies

In the Surface Pressure Measurement studies done with the Wind Tunnel Testing the Pressure Coefficients for Mean, Maximum, Minimum and Rms values for the Critical angles are obtained for the Isolated and Interference effects on the Signature towers. As the Signature Tower building is of an irregular shape and very tall in height the coefficients laid in the IS 875: parts 3-1987 are not sufficient as they are only applicable for regular shaped buildings for the Structural and Cladding designs.

The values of the pressure coefficients obtained from Wind Tunnel Testing a re based on Mean Hourly Approach which are then Changed to 3 sec Gust approach by the factor calculated as 0.794 and then the Pressure Coefficients values for the Structural designs are taken for the Major Critical Design angles **($45^o$, $90^o$, $195^o$, $345^o$)** as shown in **Table 5.3a, 5.3b, 5.3c, and 5.3d**, and it is observed that values of the Pressure and suction coefficients are much higher almost greater than 2, whereas the maximum values by code for the pressure coefficients is 0.8 to 0.9. Even for the cladding design the maximum of all the values irrespective of the nature i.e., either pressure or suction are very much high which are shown in **Table 5.4a, 5.4b, 5.4c .**

These values of pressure coefficients are then presented in form of contours to see the Local Pressure Distribution as shown in **Fig 5.3.**



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# CHAPTER - 6
# Conclusions

## 6.0 General

This present study has been conducted to examine the variation of wind loads on the given Signature Towers comprising of three wings viz. Wing-I, Wing-II, Wing-III and the Central tower to which they are attached.

As a first step of the study, Analytical method Laid by **Davenport** and **IS 875-part 3-1987** has been used to find out the Base forces and Base Moments on the Signature Towers. After that two methods have been adopted to examine the wind load variation firstly the Dynamic Analysis with the help of Boundary Layer Wind Tunnel and Pressure Fluctuations again with the help of Boundary Layer Wind Tunnel experimentally. The building has been examined for the Terrain Category 3 and providing with the flow conditions in the Wind Tunnel of that Terrain Category 3 as laid by **IS 875-part 3-1987**.

In the Dynamic Analysis the Signature Towers have been studied for the Standalone (Isolated) and Interference (Nearby Buildings) effects in the Wind Tunnel with provided flow conditions with the various Angles of Wind Incidence from $0^o$ to $360^o$ with an increment of $15^o$ and accordingly the Base Forces and Base Moments are calculated with the help of the **Balendra's approach.**

In the Pressure Fluctuations Analysis the Signature Towers have been studied by examining the **Coefficients of Pressure for mean, peak (minimum and maximum), rms** values on the faces of Wing-I, Wing-II, Wing-III and the Central tower of the Signature Towers with same flow conditions in the Wind Tunnel with the various Angles of Wind Incidence from $0^o$ to $360^o$ with an increment of $15^o$. From all the values obtained of the pressure coefficients the maximum values irrespective of their signs i.e., either pressure or suction values have been taken for the Major critical Angles as mentioned above for the design of Claddings and from all those maximum values the Maximum of the Values at particular Levels of building have been taken named as **All Azimax** values for the design of Structural elements.

## 6.1 Main Conclusions

- In the Dynamic Analysis of the Signature Towers the comparison has been made in the Base Forces and Base Moments being obtained from the Analytical and Wind Tunnel Analysis which concludes that the Values of Base Forces and Base Moments are much higher for those obtained from Wind Tunnel Analysis than those from the Analytical analysis as listed in **Table 4.10.** Even the Storey Wise Lateral Forces obtained on the 10m interval heights are more in Case of Wind Tunnel Testing Dynamic Analysis by Balendra's Approach than in the Analytical Analysis by Davenport Approach of IS 875: part 3-1987 code and can be seen in the Fig 4.18 where Comparison for both the Analysis is shown.
- In the Pressure Studies of the Signature towers it has been observed that there are values of Pressure Coefficients higher than those given in IS 875- part 3 thereby showing the true behavior of Pressure Distribution which makes the design of Claddings and Structural elements more safer.

## 6.2 Overview

From the study made on the Signature Towers for Both the Dynamic Behavior and Pressure variations in the along wind direction shows that the Wind Loads are much higher than those obtained from the Analytical Analysis from Codal provisions. A detailed study can be carry out to in the across wind direction for other buildings to know the differences in the Analytical and Dynamic behavior.

## ❖ List of Tall Buildings

Until recent years tall buildings in our country were mostly below 100m height. A latest collection of information shows that in Mumbai alone about 20 tall buildings ranging between 150m to 320m are either under construc tion or proposed to be built in near future. Dynamic response of such buildings under wind as well as earthquake forces should be precisely evaluated.

### Tall buildings in India

| 20 Tall buildings in India (Height omitted/or buildings in planning proposal ph ase) | | | | | |
|---|---|---|---|---|---|
| S.No. | Building | City | Height (m) | Floors | Rank |
| 1. | India International Trade Centre (Proposed) | Mumbai | 320 | 72 | 1 |
| 2. | The Imperial (U/e) | Mumbai | 252 | 60 | 2 |
| 3. | Ashok Towers 1 (U/e) | Mumbai | - | 53 | 3 |
| 4. | Planet Godrej (U/e) | Mumbai | 221 | 51 | 4 |
| 5. | Lodha Belismo | Mumbai | 205 | 66 | 5 |
| 6. | Shreepati Arcade | Mumbai | 161 | 45 | 6 |
| 7. | RNA Mirage (U/C) | Mumbai | 149 | 41 | 7 |
| 8. | Belvedere Court | Mumbai | 149 | 40 | 8 |
| 9. | Oberoi Spas (U/e) | Mumbai | - | 40 | 9 |
| 10. | Kalpataru Heights | Mumbai | 144 | 39 | 10 |
| 11. | Orchid Enclave (U/C) | Mumbai | - | 47 | 11 |
| 12. | The Legend (U/C) | Mumbai | - | 40 | 12 |
| 13. | Suraj Towers | Mumbai | - | 40 | 13 |
| 14. | Rushabh | Mumbai | - | 40 | 14 |
| 15. | Heritage (U/C) | Mumbai | 138 | 34 | 15 |



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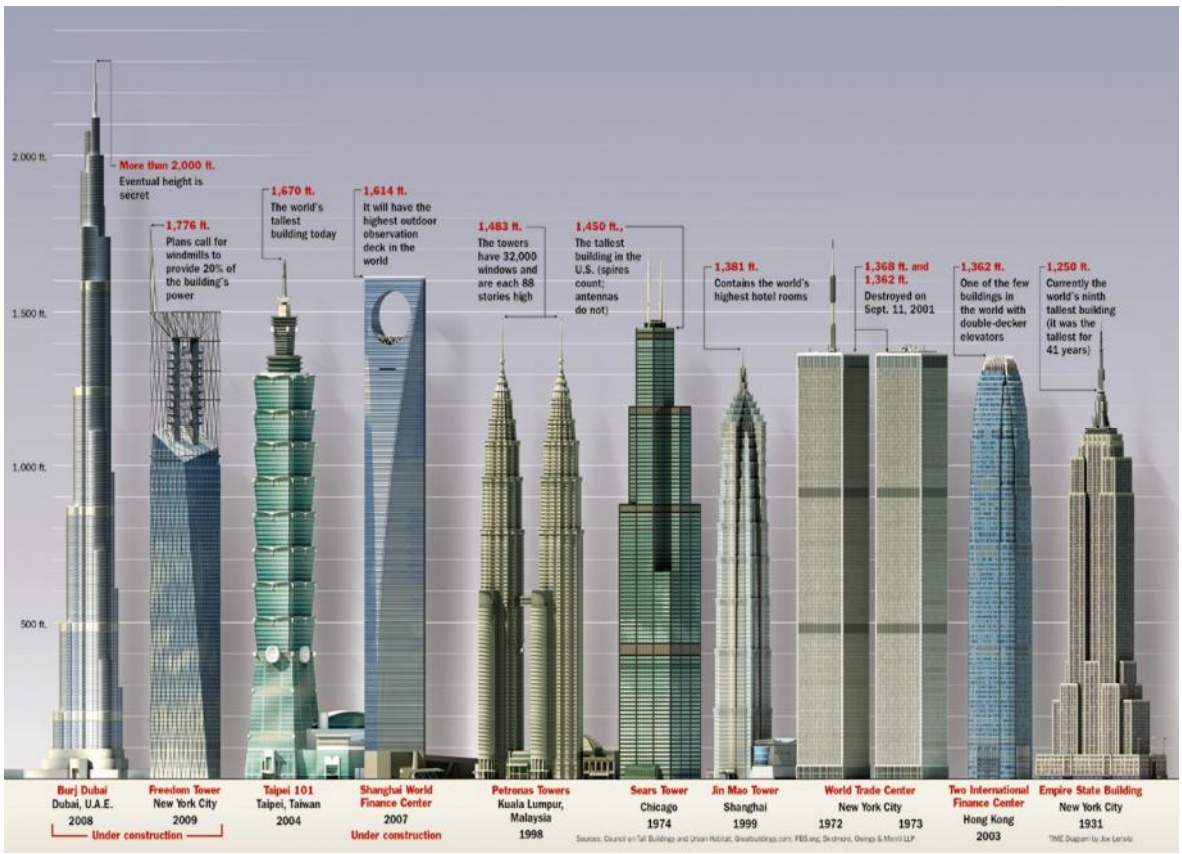
| | | | | | |
|---|---|---|---|---|---|
| 16. | KSRTC Tower (proposed) | Mumbai | - | 45 | 16 |
| 17. | Oberoi Woods Towers (U/e) | Mumbai | - | 40 | 17 |
| 18. | Uniworld city (U/C) | Kolkata | 130 | 35 | 18 |
| 19. | Oberoi Sky heights (U/C) | Mumbai | - | 35 | 19 |
| 20. | DSK Durgamata (U/C) | Mumbai | - | 32 | 20 |

## ❖ Some Tall Buildings of World



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