Analysis, design and testing of tower-like structures subjected to wind loading

S.Gomathinayagam
Deputy director, Structural Engineering Research Centre, CSIR Chennai

1.0 Introduction

Wind implies power, a source of renewable energy and stated in simpler terms, it is the air in motion. Wind power is being conceived as one of the viable environment friendly alternatives to thermal or nuclear mode of electric power generation. On one hand, man wants to harness the abundantly available natural wind power, but on the other hand, he finds wind as a challenging force to reckon with, when designing structures for power generation, transmission, communication, habitat development, and installation of residential, commercial and industrial buildings, and infrastructure/life-line support systems including bridges and offshore platforms. While wind power is proportional to the cube of wind velocity, wind loading is proportional to only the square of the later. Wind loading on structures located on the earth's surface, onshore as well as offshore, is highly influenced by the turbulence generated in the atmospheric boundary layer. The man's pursuit of designing taller buildings in the built-up areas, complex shaped offshore platform top-side structures(*Gomathinayagam et. al, 2000a),* and understanding the wind flow around trussed frame works aimed at improving the understanding of dynamic behavior of towers, resulted in some practical method of tackling the design problems using wind tunnel and/or analytical simulations with insufficient measured field data in many of the cases.

Tower like structures have one of their three dimensions much bigger/longer than the other two lateral dimension, allowing a two dimensional fluid flow around the structure. The focus in this lecture has been to study dynamic wind loading features which may cause peak stress levels owing to background turbulence in wind and/or resonant response due to proximity of structural frequencies to the excitation frequencies in wind. Another aspect is the fluctuating/alternating stress levels in structures/ components subjected to prolonged exposure to wind turbulence every day, throughout their design life. Such fluctuating stress ranges cause wind induced fatigue, which is a localized cumulative damage phenomenon. The wind loading and response analysis, design and testing of rigid fixed-base towers, compliant guyed towers and static, dynamic and fatigue design of wind mill support towers are the focus in this section.

2.0 Analysis of Tower Like Structures For Wind Loading

All environmental loads are essentially dynamic, with excitation energy in specific range of frequencies and Wind is no exception. Normal wind turbulence spectra have significant energy up to 1 Hz. Measured spectra (*Shanmugasundaram et.al, 1999*) during cyclone winds have the energy spread up to 10 Hz. Wind induced loads are thus dynamic always. Today, the design offices are flooded with general/special purpose software capable of modeling using popular Finite Element Method(FEM), any complex large structural systems. Finite element method enables mathematical modeling of any tower, using an assemblage of stiffness and mass of various elements at the tower interface which are identified by a nodal co-ordinate system and element connectivity. The general dynamic equilibrium equation at any instant of time 't' , using FEM is given by

$$[M]\{\ddot{X}(t)\} + [C]\{\dot{X}(t)\} + [K]\{X(t)\} = \{F(t)\} \quad . \qquad (1)$$

where [K],[M],[C] = Stiffness , Mass and Damping Matrices of the structural system
{X(t)}, { $\dot{X}$ (t)}, { $\ddot{X}$ t)} = Displacement, Velocity, and Acceleration Vectors
{F(t)} = Instantaneous Force vector (Wind pressure system and flow induced effects on structure)

Owing to the uncertainties in the evaluation of aerodynamic transfer functions (a function or set of parameters that convert the wind velocity to wind loads on structures) in time domain to characterize wind force {F(t)} , even for the analysis of observations of controlled wind tunnel investigations, a time domain method is rarely adopted, and frequency domain is preferred.

2.1 Static Analysis

Most of the tower-like structures are designed following static analysis procedures, wherein either steady state (mean) or quasi-steady(factored mean) wind loads are used adopting the popular GRF methods. The inertial and damping effects are usually either neglected or indirectly taken into account in GRF and hence the equilibrium Eq.1 for static analysis becomes

$$[K]\{X(t=0)\} = \{F(t=0)\} \quad (2)$$

In modern engineering analysis of tall tower-like structures, after evaluating the static displacements, the coordinates are updated and the wind loads are applied on the structure with the updated coordinate system to get more accurate response. The procedure is also known as " P-Δ effect" and the second analysis is referred as second order analysis. Alternatively geometric non-linear/large displacement analysis can be used with incremental loading.

2.2 Free Vibration Analysis

We know, that based on the ratio of stiffness and mass there exists a fundamental frequency for any tower, which is an important dynamic characteristic of the tower. If this frequency is closer to any of the wind induced frequencies, there is possible structural resonance which should be avoided. Neglecting damping and by setting the external forces zero, we get an equation for free response as

$$[M]\{\ddot{X}(t)\} + [K]\{X(t)\} = 0 \quad (3)$$

Free vibration analysis can be done with or without condensation of degrees of freedom. The fundamental frequency is required to evacuate peak response using Gust response factor. Thus accurate evaluation of first global mode of frequency of the structure is a pre-requisite for contemporary design practice for wind sensitive structures.

2.3 Dynamic Response Analysis

The stochastic approach will be using wind turbulence spectrum which can be obtained using fast fourier transform techniques (FFT) converting the measured time domain information in frequency domain to use the wind loading chain given later in Fig.5. The variance(square of standard deviation when mean is zero) of response can be evaluated, which is the area under the power spectrum of response(Fig. 2). A detailed derivation is given in section 3.0.

2.4 Fatigue Analysis

Wind induced fatigue in structures has not been normally analysed, except for towers that support wind mills, across wind oscillations of steel chimneys, buildings, and vibrating process equipment. Using a spectrum of wind speed variation, a selected number of bins (groups) of stress range amplitudes in the stress spectrum for the entire design life of tower can be identified for fatigue analysis. Under the i^{th} constant amplitude cyclic loading condition, for a given stress range $\Delta\sigma_i$ a simple damage index d_i, based on Palmgren – Miner's rule is evaluated as ,

$$d_i = \frac{n_i}{N_i} \qquad (4)$$

where in n_i is the actual number of stress cycle occurring in i^{th} stress range N_i is the permissible number of stress cycles as per (Stress range vs Number of cycles to fail under constant amplitude loading) S-N curve (ECCS, 1985). SERC has a state-of the art Fatigue testing Laboratory (www.sercm.org) for developing S-N data for materials/components. The recent research (Lakshmanan et al, 2003,2004; Gomathinayagam et al, 2006) has also focused in the in-service wind induced fatigue load cycle estimation ie. 'n_i' to be used in Eq.4 using full scale testing in natural wind. For random excitation, the total cumulative damage 'D' can be defined as a sum of individual damages, for the intended design life..

$$D = \sum_{i=1}^{\text{No of bins of cyclic loading}} d_i \qquad (5)$$

According to ECCS-TC6 selection of a particular S-N curve depends on the structural detail category which is user selectable for fatigue design using tower analysis program developed at SERC. (*Gomathinayagam et al, 1995*)

3.0 Wind Loading on Tower-like Structures

The basic philosophy in evaluating the wind loading on structures stems from the fact, that the dynamic wind pressure which is proportional to the square of instantaneous velocity gets transferred as force (integrated pressures over the projected area) on the intercepting structures on

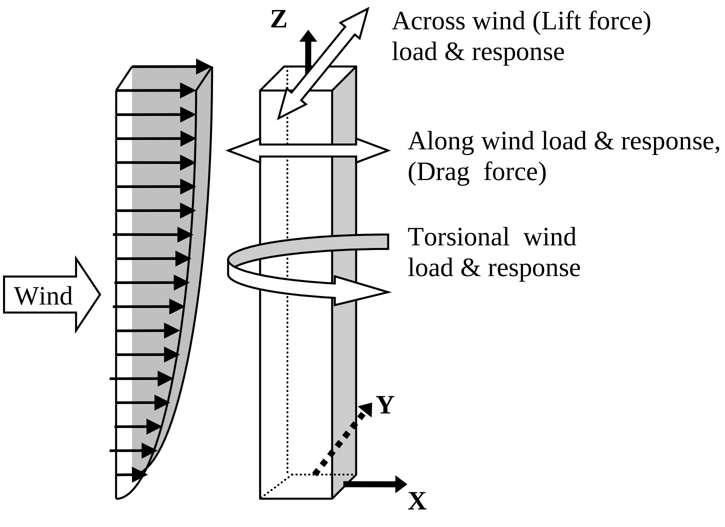


Fig.1 Primary wind load effects on tower-like structures

a plane normal to the mean wind direction. A combination of fluid dynamics and similitude laws paved the way to reduce the gap in determining the loading coefficients by conducting a series of controlled wind tunnel experiments with modeling of atmospheric turbulence. It is essential to embark on more field measurements to enhance understanding of wind and its dynamic effects, with particular reference to wind turbulence in extreme wind , under which conditions engineered structures seem to collapse (*Harikrishna 2007; Shanmugasundaram 2000*). Depending on the flow of wind around or on the structures the loading can be stated to be along wind (in the mean wind direction, Drag), across wind (perpendicular to the mean wind direction, Lift) and torsional as shown in Fig.1. In this case it is shown that the mean wind direction is along the X-axis of the structure for clarity of terminology of types of wind loading and responses. In reality mean wind direction can be skewed to the X-axis in the XY plane to any angle assuming 2-D flow around the tower-like structure. In this topic our discussions will be focused on issues related to along wind loading and response. The across wind effects (*Arunachalam, 2007*) with specific reference to chimneys and torsional wind effects specially to bridge sections (*Selvirajan, 2007*) are dealt elsewhere in this book.

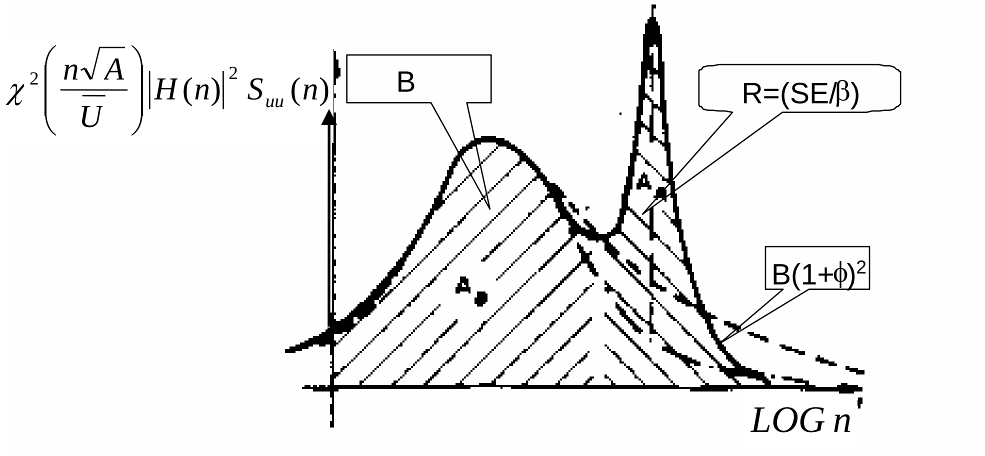
3.1 Evaluation of wind loading on structures : Basis of codal provisions

In reality mean wind direction can be skewed to the X-axis in the XY plane to any angle θ(t), assuming 2-D flow in any horizontal layer, around the tower-like structure. The instantaneous wind loading is always a combination of drag and lift (Fig.1) as shown in Eq.6

$$F_X(t) = -(F_{Drag}(t)\ \cos\theta(t) + F_{Lift}(t)\ \sin\theta(t))$$

$$F_Y(t) = (F_{Drag}(t)\ \sin\theta(t) - F_{Lift}(t)\ \cos\theta(t)) \qquad (6)$$

For static structures (which are rigid), an extreme gust wind velocity prevailing in any region (*IS875, 1989*) specified as the basic wind speed and loads are computed using a steady state loading due to the extreme wind of the region. For dynamic structures (which are relatively flexible and wind sensitive) a Gust response factor(GRF) or Gust effectiveness factor(GEF) is used eliminating the need for high computational time and complexity of dynamic computation



$$G = 1 + g\,r\,\sqrt{B(1+\phi)^2 + SE/\beta} \quad \text{with} \quad \phi = \frac{g\,r\sqrt{B}}{4}$$

Fig. 2 Components of Gust response factor (*Vickery 1989; IS875, 1989*) using Eq.1. The basis of the GEF method used mostly for along wind response can be explained using Fig.2. For most structures, the mean overall force coefficients have been obtained for typical structures only through experimental studies, mostly using wind tunnel models. For the fluctuating part, the existing analytical methods are strongly dependant on the statistical description of random wind loads and application of spectral analysis techniques (*Davenport 1967*). For the case of tall buildings and tower-like structures the overall force coefficients or external/internal/effective pressure coefficients have been prescribed based on wind tunnel model studies. While interpreting the codal provisions(*Abraham, 2007*), pressure coefficients are used for a point, overall force coefficients(Eq.6) are applied for a projected area(in XZ or YZ plane), drag and lift coefficients are also for projected areas, but defined for use only in the along or across wind directions respectively. With this introduction it is now possible to discuss the mechanism of transfer of wind load effects, analysis, design and full scale testing of tower-like structures subjected to wind loading.

4.0 Along wind response Analysis of fixed base tower-like structures

Specially in the applications to wind response computations, owing to certain characteristic features of wind turbulence spectra, a discrete frequency domain approach has been used based on the pioneering contributions of Davenport (*1967*), involving practical application of random vibration theory. A frequency domain computational model is explained for the evaluation of dynamic wind response of full scale structures using measured/site-specific wind characteristics. The power spectral density of response in terms of modal coordinates of jth mode may be written in the form ,

$$S_{\eta j}(n) = \frac{1}{K_j^2}\left|H_j(n)\right|^2 S_{f_j}(n) \qquad (7)$$

where $S_{f_j}(n)$ are to be evaluated using a standard wind turbulence spectrum (Appendix-A),

and the square of receptance (mechanical admittance) is expressed as,

$$\left|H_j(n)\right|^2 = \frac{1}{\left(1-\dfrac{n^2}{n_j^2}\right)^2 + 4\xi_j^{\,2}\left(\dfrac{n}{n_j}\right)^2} \qquad (8)$$

where $n_j = \omega_j / 2\pi$. The symbols K_j , ω_j & n_j are used to indicate the generalised stiffness, natural frequency of jth mode in radians/second and in Hz (cycles /second) respectively. In the various steps involved in the general spectral analysis of structures subjected to random wind loading spectral analysis of three dimensional tower like structures may be treated analogous to the methods of wind response analysis of line-like structures (*Simiu and Scanlan 1996* and *Holmes 2001*). A practical design method unique to specific wind sensitive slender structures discussed in this lecture have been essentially based on (*Venkateswarlu et al. 1994*) with a focus to utilize as many realistic site specific wind loading parameters (*Gomathinayagam, 2005*) as possible to compute the along wind response of tower-like structures. Computation of along-wind response spectra at various levels of the structure have been considered. The improvement over the existing gust response method(GRF in Section 3.1) is consideration of multiple realistic modes of vibration with variation of design parameters along the height. Any effects due to self-induced or excited oscillations, vortex shedding effects, galloping and buffeting/flutter have not been addressed in the methods described in this lecture. The various mathematical models schematically depicted in (Fig. 5) can be implemented with discrete frequency step integration, using field measured/available site specific design parameters. Considering response in the along wind direction the response will consist of two parts viz. the mean part (steady-state or static) and the fluctuating part (unsteady and dynamic).

Mean Wind Response (effect of $\overline{U}$)

Before examining the dynamic effects of wind on structures, it is useful to recognize that the natural wind loading and response of structures has two parts owing to the two wind components along the wind, viz. the mean wind speed $\overline{U}$ and its fluctuations $u(t)$. Natural wind consists of a steady mean flow averaged over a suitable period upon which are superimposed the fluctuations which are normally called gusts or turbulence, discussed earlier , as given below:

$$U(t) = \overline{U} + u(t) \rightarrow \textit{gives the wind force components}$$

$$F(t) = 0.5\rho_a A_e C_f \overline{U}^2 + \rho_a A_e C_f \overline{U} u(t) = F(\overline{U}) + f(t) \qquad (9)$$

neglecting higher order terms involving $u(t)^2$. Realizing that the quasi-steady response of structures can be expressed as a linear combination of their vibration modes, mean wind response would involve inclusion of all the possible modes of the structure (or structural model) while using a mode superposition approach (*Simiu and Scanlan 1996*). Alternatively a static analysis of the structure following Eq.(2) would directly yield the mean or static response of the structure to wind loads, which in fact would be the preferred computational analysis and design approach using only the first mean force term corresponding to $F(\overline{U})$. When the structure is rigid and does not warrant the detailed dynamic analysis to get spectra of response it is possible to use the present GRF method involving only first mode of vibration. More accurately fluctuating response can be computed including significant modes of vibration using a multi modal spectral approach(*Venkateswarlu et.al., 1994; Gomathinayagam, 2005*) which is explained below.

Fluctuating Wind Response (effect of $u(t)$)

Most structural components or structures are bluff bodies (*Holmes 2001*), some with partial openings, resisting the wind force. The dynamic along wind loading (Drag forces in Fig.1) on small areas very much depends on the size of energy containing eddies. The moving air mass may be visualized as a cluster of bubbles of spherical or ellipsoidal shape with widely varying sizes. The lattice plate model proposed by Cook (*1985*) defines that the drag forces on any small area (such as a finite element with projected area '*A*' normal to wind) depend only on the local wind speed. The drag forces if evaluated at two locations the effect of turbulence and their correlations (simultaneous occurrence in the same sign) have to be accounted in the dynamic wind load computations. A transfer function of converting the wind velocity fluctuations to dynamic wind loads on structures, represents the dependence on eddy size in relation to the structure size and the energy transferred as wind force. It is obvious from Fig.3, even a larger eddy having an order of the structure size (in this case the separation of node 'i' and 'k') may not have the same effect on dynamic drag, while the smaller eddy has practically no effect on node 'k'. Due to the larger eddy the node 'i' has a positive pressure while the other has a suction, which is negative pressure. Hence only eddies of very large sizes (bigger than the overall dimensions of the entire structure) can cause correlated overall drag force effects. Thus the analytical evaluation or characterization

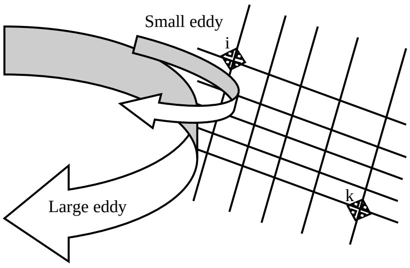


Fig. 3 Correlation and dynamics of eddies (Cook 1985)

of a time domain aerodynamic load transfer function becomes very complicated for even simple structures. Hence a frequency domain approach based on statistical theory of random vibrations is often adopted since any random excitation is a combination of several harmonics (sine waves) with varying amplitudes, phase and frequencies.

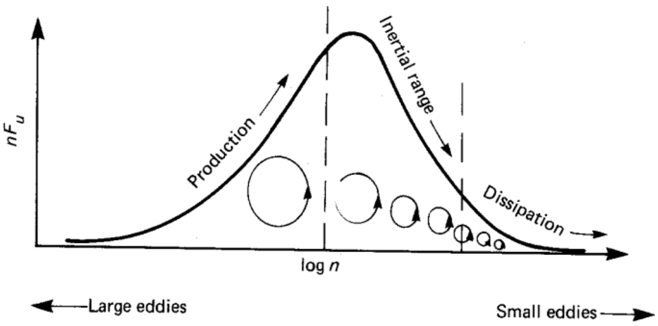


Fig. 4 Concept of energy cascading (Cook, 1985)

In Fig.4 , the wind force energy level and eddy frequencies are shown in the phases of production, inertial motion and dissipation of eddies in wind. As the larger eddies which are at low frequencies get broken into smaller eddies having higher frequencies in presence of structure,

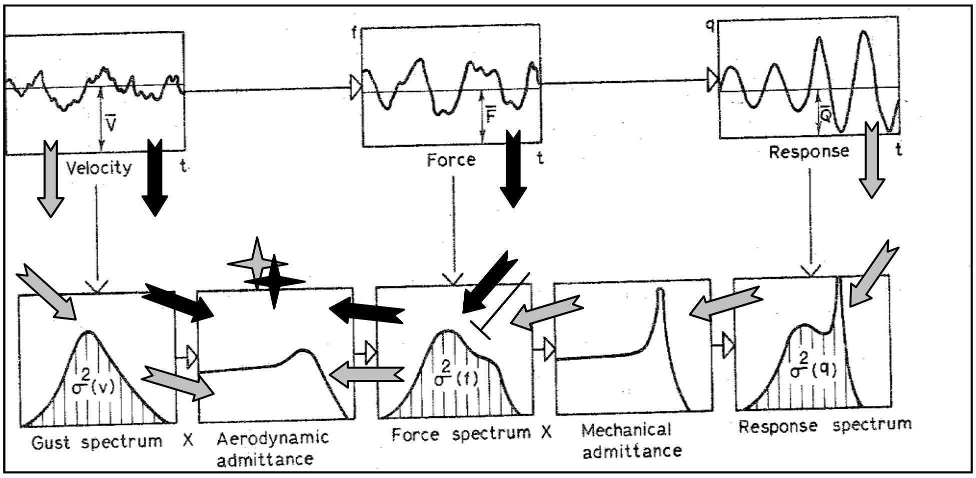


Fig. 5 Wind loading and response chain in the Davenport (1967) model

the energy is either transferred or dissipated as friction/heat/pressure. This phenomena is known as energy cascading as given in Fig.4, and is the basis of the form of aerodynamic admittance function shown in Fig.5. Eddies in the flow field have uncertainties regarding their stability and spatial correlations on the size of the structure. Structure size exposed to wind may have to be treated as varying or random, since the mean direction shifts continuously in the flow field. It is the aerodynamic admittance (Fig.5) which accounts for the partial correlations of wind pressures (Fig.3) at different points on the structure and the energy cascading (Fig.4) of smaller eddies. Davenport (*1967 & 1977*) suggested both mode dependent and mode independent aerodynamic admittance functions, which are frequency dependent.

Aerodynamic Admittance Function (Mode Independent)

The total transfer of energy of wind velocity fluctuations to a spectrum of force is valid only for small areas (structures) that are fully engulfed in an eddy of larger size than the structure. In nature, the ideal condition is never feasible due to the phenomenon of energy cascading, which implies that large sized eddies have higher excitable energy at low frequency; and as the eddy size becomes smaller at higher frequencies, they have lower energy levels. The larger eddies will result in higher correlated force than the partially correlated force by smaller eddies. For larger structures, for the non-coherent eddies (not acting simultaneously on the whole of the structure having a projected area "A"), an adjustment had to be done for the relative sizes of the eddy and the structure. Hence a form of aerodynamic admittance function, $\chi^2(n)$ has been suggested (Davenport *1963 & 1967*) as a reducing function which varies from unity at low frequency to zero at high frequency and modifies velocity spectrum to closely approximate the dynamic wind load spectrum in the entire range of frequencies as given in Eq.(10)

$$S_{ff}(n) \; = \; \left(\rho A C_D \overline{U}\right)^2 S_{uu}(n)\, \chi^2(n) \qquad (10)$$

Power form of Aerodynamic admittance (*Davenport 1977***)**

An empirically fitted relationship for $\chi^2(n)$, as a function of the reduced frequency is given by

$$\chi^2(n) = \frac{1}{\left[1+\left(\frac{2.n\sqrt{A}}{\overline{U}}\right)^{4/3}\right]^2} \qquad (11)$$

where, the reduced frequency $\left(\frac{n\sqrt{A}}{\overline{U}}\right)$ is the ratio of characteristic linear structural dimension $\sqrt{A}$ to the characteristic eddy size $\left(\overline{U}/n\right)$, with '*A*' representing the projected area normal to the wind direction, going by the definition of Davenport (1963). The aerodynamic admittance has been in many cases an experimentally measured function similar to drag/lift/moment coefficients. The statistical treatment of evaluating the peak responses and the role of this power form of aerodynamic admittance has been well illustrated in the popular conceptual model (Fig. 5) of Davenport (1967). There is also another model which takes into account coherence decay in x, y, and z directions which is of exponential form and this is important for three-dimensional towers which deviate from line like structures with respect to spatial correlations.

Exponential Form of $\chi^2(n)$ (*Vellozzi and Cohen 1968*)

Vellozzi and Cohen (1968) recommended the following expression for aerodynamic admittance,

$$\chi^2(n) = N(x)\,N(y)N(z) \tag{12}$$

where

$$N(x) = \frac{1}{\zeta} - \frac{1-e^{-2\zeta}}{2\zeta^2} \;\;,\;\; N(y) = \frac{1}{\mu} - \frac{1-e^{-2\mu}}{2\mu^2} \;,\; N(z) = \frac{1}{\gamma_z} - \frac{1-e^{-2\gamma}}{2\gamma_z^2} \tag{13}$$

ζ = 3.85 (n Δx/Um) ; μ = 11.5 (n Δy/Um) ; γ_z = 3.85 (n Δz/Um) (14)

Δz = H, height of the structure

Δy = b, width of the structure in the across-wind direction

Δx = (4H) or (4b) which ever is less

Um = average mean wind speed over the height of the structure

A realistic dynamic wind loading model requires an empirical/experimental characterization of the relationship between fluctuations of velocity and force either in the time domain or frequency domain. The later holds the promise of enabling practical evaluation of aerodynamic admittance from either forces or responses

Joint Acceptance Function (Mode dependent aerodynamic admittance)

Joint acceptance functions (JAF) introduced by Davenport (1977) for various structures describes the interaction of structural vibration modes with the wind loading correlations on the structure (*Nigam and Narayanan 1994*). The 'JAF' has been explained as the one incorporating the sensitivity of interaction between the turbulence characteristics and structural vibration modes. Thus the JAF depends on the relative size of the span (distance between two points on a structure) to the scale of turbulence in the span wise direction. The complexity increases further when aero-elasticity is to be considered in the wind load analysis, when the mode of vibration has positive and negative signs at different points on the structure, instead of monotonic mode shape. This aspect of mode dependency of the admittance has been clarified (*Dyrbye and Hansen 1996*) by the use of influence functions. JAF has applications to long span bridge aerodynamics.

Computation Dynamic Wind Response on Components and Structures

The wind fluctuations $u(t)$, being random cause fluctuating turbulent wind force, in a band of excitation frequencies. However, the excitation energy in the wind is negligible beyond about 1Hz. Thinking in frequency domain, $u(t)$ is represented by its power spectral density (PSD), $S_{uu}(n)$ indicating energy in the wind in various frequencies '*n*'. Most wind sensitive structures vibrate dynamically owing to wind turbulence, only in a few fundamental modes. Hence for the computation of fluctuating wind response using mode superposition, considering limited number of well separated modes is deemed acceptable. Then, considering the dynamic response of the structure due to wind turbulence, it can be shown that, the jth modal mechanical admittance function $H_j^2(n)$, given in Eq.(8) is sufficient to evaluate the modal responses of the tower (Fig.6). At this stage it has been assumed that the required eigen-frequencies and mode shapes have already been evaluated using either a direct 3-D FEM model or a simplified (*Gomathinayagam et al., 1995*) and yet sufficiently accurate model with suitable lumping of masses, generally at the various floor/panel levels. The dynamic model derived using the later approach (via the flexibility coefficient technique) is often referred to as a generalized pole model, schematically shown in (Fig. 6). It may be noted that the number of experimental measurements or analytically simulated wind data ('L' levels) will generally be fewer than the number of levels ('N') used in the structural modeling using lumped parameter.

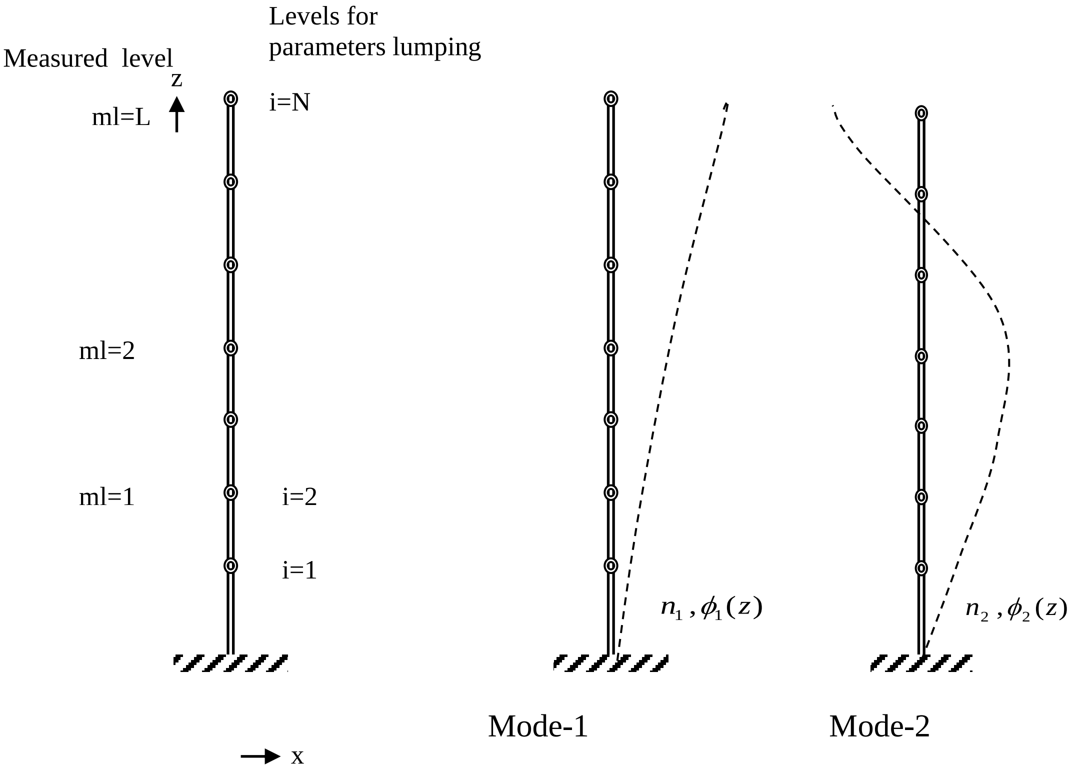


Fig.6. Idealised pole model of a tower 'N' levels with lumped masses

In the j[th] mode, the generalized mass may be computed using the summation over all the 'N' levels as

$$M_j^* = \sum_{i=1}^{N} \int_{z_i - \frac{h_i}{2}}^{z_i + \frac{h_i}{2}} \phi_j^2(z)\, m(z)\, dz \qquad (15)$$

where m(z) represents mass per unit length of segment of height 'h_i' at a level 'z' above ground.

Generalised modal force spectral density

Using fluctuating wind force considering complete aerodynamic admittance of $\chi^2(n) = 1.0$, in Eq.(10), the power spectral density (PSD) of fluctuating generalized force in the j[th] mode is given (Simiu and Scanlan 1996 and Venkateswarlu et al. 1994) as,

$$S_{f_{ji}^* f_{jk}^*}(n) = \sum_{i=1}^{N}\sum_{k=1}^{N}(\rho C_{f_i}\, p_{ji}^* A_{e_i})(\rho C_{f_k}\, p_{jk}^* A_{e_k}) S_{u_i u_k}(n) \qquad (16)$$

where

C_{f_i} - Solidity ratio based force coefficient in the case of towers or effective pressure coefficient in the case of buildings (IS875 (1989))

$S_{u_i u_k}(n)$ - Cross spectral density of velocity fluctuations at levels i and k

$p_{ji}^* A_{e_i}$ - Generalised lumped parameter at the i^{th} level (for projected area of A_{e_i}) in the j^{th} mode with p_{ji}^*, given as

$$p_{ji}^* = \left\{ \int_{z_i - \frac{h_i}{2}}^{z_i + \frac{h_i}{2}} \phi_j(z)\overline{U}(z)dz \right\} \quad \text{(using a mean velocity profile and mode shape)} \qquad (17)$$

In any terrain, (*Venkateswarlu et al, 1989*), the mean wind speed, $\overline{U}(z)$ at required levels have been obtained using the Power-law/Log-law fitted to measured values at site. The cross spectral density $S_{u_i u_k}(n)$ denotes the correlation of two continuous fluctuating velocity records at z_i and z_k levels of the segments i and k. In homogeneous turbulence, the quadrature spectrum (imaginary part of the cross spectrum) vanishes (*Simiu and Scanlan 1996*) and the co-spectrum (real part) defines clearly the cross spectrum adequately (*Vickery 1970*). Representing the cross spectral density using,

$$S_{u_i u_k}(n) = S_{u_i u_i}^{1/2}(z_i, n)\, S_{u_k u_k}^{1/2}(z_k, n)\, coh(z, z'; n) \qquad (18)$$

where the coherence function $coh(z, z'; n)$ is given as

$$coh(z, z'; n) = \exp\left[- \frac{2nC_z h_{ik}}{\left(\overline{U}_i + \overline{U}_k\right)} \right] \qquad (19)$$

where

C_z - Exponential decay coefficient (*IS875,1989*) for vertical direction (derived from measurements)

h_{ki} - Distance between points 'k' and 'i' along the height

To account for reduction in fluctuating wind forces due to lack of correlation which depends upon the characteristic eddy size in relation to the structure size, at any level and the energy cascading (Fig. 4.) effect, the aerodynamic admittance, $\chi^2(n, z_i)$ at the i^{th} level is used in Eq.(18). This has been experimentally obtained from wind tunnel or full scale measurements and some suggested standard forms are given in Eq.11 and Eq.12. When the mean wind speeds vary at different levels in a discrete idealization using aerodynamic admittance at the two levels, the PSD of generalized force in Eq.(18) may be written as,

$$S_{f_{ji}^* f_{jk}^*}(n) = \sum_{i=1}^{N}\sum_{k=1}^{N}(\chi(n,z_i)\rho C_{f_i} p_{ji}^* A_{e_i})(\chi(n,z_k)\rho C_{f_k} p_{jk}^* A_{e_k}) S_{u_i u_k}(n) \qquad (20)$$

where

$\chi(n, z_i)$ - square root of aerodynamic admittance at the i^{th} level (deduced from measurements or adopted standard forms)

Now, considering the vertical coherence of wind velocities between the levels, using Eq.(18) and Eq.(19), Eq.(20) becomes,

$$S_{f_{ji}^* f_{jk}^*}(n) = \sum_{i=1}^{N}\sum_{k=1}^{N}(\chi(n,z_i)\rho C_{f_i} p_{ji}^* A_{e_i}) S_{u_i u_i}^{\frac{1}{2}}(n)$$

$$(\chi(n,z_k)\rho C_{f_k} p_{jk}^* A_{e_k}) S_{u_k u_k}^{\frac{1}{2}}(n) \exp^{\left\{-C_z h_{ki} n/\left(\left(\overline{U}_k + \overline{U}_i\right)/2\right)\right\}} \qquad (21)$$

where

$S_{u_i u_i}^{\frac{1}{2}}(n) = S_{uu}^{\frac{1}{2}}(n, z_i)$ denotes the square root of PSD of velocity fluctuations at the i^{th} level

The j^{th} modal response is evaluated as

$$S_{\eta_j}(n) = \left[\frac{\left|H_j(n)\right|^2}{M_j^{*2}\omega_j^4}\right] S_{f_{ji}^* f_{jk}^*}(n) \qquad (22)$$

where

$\left|H_j(n)\right|^2$ - mechanical admittance function (Eq.8) at 'n_j' with $\xi_j = \xi_{T_j}$

ξ_{T_j} - denotes the total damping, which has two parts (i) the structural damping, ξ_{st_j} (ii) aerodynamic damping, ξ_{a_j}. The structural damping ξ_{st_j} is of the order 1 to 5% of critical damping for this class of structures. The values of aerodynamic damping for most lattice towers are highly varying. Cracked RCC structure can have higher damping. There has been no clear guidelines on the choice of suitable value for the aerodynamic damping for lattice towers and for tall buildings. In general, the aerodynamic damping is less than the structural damping (ξ_{st_j} in the range of 0.01-0.05 is assumed) in the measured low wind conditions structures except for suspension bridges. However, the slight increase in aerodynamic damping with mean wind speeds, is observed in field measurements (*Shanmugasundaram, 1999*). Assuming Rayleigh proportionate damping, considering the relative velocity $(\overline{U} - \dot{x})$ of the structure, ignoring wind turbulence effects, the aerodynamic damping in the j^{th} mode may be given as (*Holmes 1996 and Holmes 2001*)

$$\xi_{a_j} = \frac{C_j^*}{2M_j^*\omega_j} = \frac{C_j^*}{4\pi M_j^* n_j} \qquad (23)$$

where the generalized viscous damping constant in the j[th] mode may be computed by using

$$C_j^* = \sum_{i=1}^{N} C_{f_i} \left\{ \int_{z_i - \frac{h_i}{2}}^{z_i + \frac{h_i}{2}} \rho \phi_j^2(z) \overline{U}(z) dz \right\} a_{e_i} \qquad (24)$$

where a_{e_i} = Area per unit height (or effective width) = $\left(\frac{A_{e_i}}{h_i} \right)$

at i[th] level, the j[th] modal generalized mean viscous coefficient (given by the flower-bracketed term in Eq.(24)) acting over the unit-height-area, a_{e_i}, scaled by the force coefficient C_{f_i} applicable for the i[th] level. Now the total damping may be computed for evaluating Eq.13, by adding the user specified material/structural damping ratio with the aerodynamic damping: i.e.

$$\xi_{T_j} = \xi_{a_j} + \xi_{st_j} \qquad (25)$$

In view of Eq.(21) to Eq.(25), the modal response may be written as,

$$S_{\eta_j}(n) = \left[\frac{\rho^2 \left| H_j(n) \right|^2}{M_j^{*2} \omega_j^4} \right] \left\{ \begin{array}{l} \sum_{i=1}^{N} \sum_{k=1}^{N} \chi(n, z_i) C_{f_i} p_{ji}^* A_{e_i} S_{uu}^{\frac{1}{2}}(n, z_i) \\ \qquad \chi(n, z_k) C_{f_k} p_{jk}^* A_{e_k} S_{uu}^{\frac{1}{2}}(n, z_k) \exp^{\left\{ -C_z h_{ki} n / \left(\left(\overline{U}_k + \overline{U}_i \right) / 2 \right) \right\}} \end{array} \right\} \qquad (26)$$

$S_{uu}^{\frac{1}{2}}(n, z_i)$ - square root of wind turbulence spectrum at the i[th] level (derived from measurements)

Once, the modal responses are evaluated (Eq.(26)), the response of the structure is given by,

$$S_x(n,z) = \sum_{j=1}^{'m'\,modes} \phi_j^2(z) S_{\eta i}(n) \qquad (27)$$

where $S_x(n,z)$ – spectral density of response at level 'z'.

It may be noted the above implies well separated modes. A computer program in frequency domain (*Gomathinayagam, 2005*) has been developed for computation of power spectral density of response $S_x(n,z)$ at any desired level 'z' on the structure, with measured/varying input wind turbulence characteristics, along the height of the structure. The structural idealization and the lumped parameter model details along with pre-computed mode shapes $\phi_j(z)$ (Fig.6) and frequencies 'n_1' are read by the program from an input-file and the program illustrates the applications of the formulations to analyse full scale tower-like structures.The area under the spectrum $S_x(n,z)$ gives the variance or mean square value, $\sigma_x^2(z)$ of the dynamic response. The peak response can be obtained as

$$\hat{x}(z) = \overline{x}(z) + g \sigma_x(z) \qquad (28)$$

where

$\hat{x}(z)$ = peak response

$\bar{x}(z)$ = mean response

$\sigma_x(z)$ = r.m.s of the dynamic response which is square root of area under $S_x(n,z)$

g = statistical peak factor (defined for stationary random process) Simiu and Scanlan (1996)

The wind response spectrum which typically resembles Fig. 2 comprises of two parts, (i) the quasi-static or stedy state or broad banded response spectrum area and (ii) the resonant or narrow banded response spectrum are. Accordingly, the r.m.s. dynamic response has been divided into background response (in the low-frequency region) and resonant response (around the natural frequency) and the corresponding peak response is given as

$$\hat{x}(z) = \bar{x}(z) + \sqrt{[g_B \sigma_{xB}(z)]^2 + [g_R \sigma_{xR}(z)]^2} \qquad (29)$$

4.1 Analysis and Design of guyed towers

Guyed towers, are compliant and light weight, and have lateral supports at several intermediate guy points. These towers are likely to get excited in multiple modes. Hence the use of multiple modes is important in response computation. Since the structure of a guyed mast is relatively light and supports only modest equipment loads, the members are designed mainly for lateral wind loads. Further the actual behaviour of guyed towers is extremely complicated. The behaviour of the mast is non-linear due to its slenderness and also due to the large displacement it experiences under sustained wind loading. The guys also exhibit, in general, geometric and material non-linear behaviour, especially at low values of pretension. During orissa super cyclone, a few tall slender guyed masts have collapsed due to extreme wind forces during cyclones. Hence, an accurate assessment of wind load effects on guyed mast is essential for safe and efficient design of such guyed masts.

The static response of guyed masts to mean wind loads can be reliably estimated using existing analytical methods with non-linear stiffness characteristics of the guys and large deformation effects of the mast. The complex interaction between the mast and guys in response to gusty winds typically results in a large number of active vibration modes (as many as 10 modes or more), some of which are closely spaced. The dynamic analysis is further complicated by the random nature of wind loads, which vary in both time and space. Hence for the analysis of guyed masts, a non-linear transient dynamic analysis is most appropriate which will be computationally intensive and time consuming for the designers. Hence, two equivalent static analysis methods using wind patch load conditions suggested by *IASS (1981)* and Davenport (*1992*) along with the usual 3-sec gust loading approach given in *IS:875(Part 3)-1987* are recommended, for a practical design.

(i) 3-Sec Gust Method

In this method, wind loads are calculated along the height using the 3-sec gust wind speed profile (k_2 factor) given in *IS:875 (Part 3)* for the given terrain category. These loads have to be applied as static loads on the guyed mast.

(ii) IASS Patch Load Method

Since the turbulent eddies within the winds are generally small compared to the mast height, wind pressure fluctuations tend to be poorly correlated along the height of the mast and have localised peaks. Hence, *IASS* International Association for Shell and Space Structures" has recommended that the guyed mast should investigated for patch wind loading in addition to the full wind loading as per 3-sec gust method.

The patch wind loading is done for two cases :

(i) By considering 3-sec gust loading over the entire guyed mast and then by reducing the gust loading over any one span between adjacent guy levels to the hourly mean wind loading. (Mean patch over gust)

(ii) By considering hourly mean wind loading over the entire guyed mast and then by increasing the mean wind loading over any one spans between adjacent guy levels to the gust loading. (Gust patch over mean)

The hourly mean wind loads are calculated using the hourly mean wind speed profile ($\bar{k}_2$ factor) given in IS: 875 (Part 3) for the given terrain category.

(iii) Dynamic Patch Load Method (Davenport 1992)

In this method, the peak dynamic response was approximated by,

$$\hat{x} = \bar{x} + \hat{x}_{PL} \qquad (30)$$

where $\bar{x}$ = mean response of the structure

$\hat{x}_{PL}$ = peak fluctuating response of the structure

Here the mean response, $\bar{x}$, of the structure can be determined by applying the hourly mean wind load on the guyed mast. The peak fluctuating response of the structure is evaluated using a series of static load patterns to recreate the dynamic effects of gusting wind. The specified load patterns consist of wind load patterns applied to each span between adjacent guy levels and also from mid point to mid point of adjacent spans. The patch loads should be applied to the tower in its static equilibrium position, obtained from the hourly mean wind loading condition. For each wind load pattern, an equivalent static wind pressure is obtained using the IS code provisions as given below :

$$p_{PLi}(z) = r_o\, \bar{p}_o\, \bar{k}_2(z) \qquad (31)$$

where $\bar{p}_o$ = reference mean wind pressure at 10 m above ground level

$\bar{k}_2(z)$ = hourly mean wind speed factor at a height of z (given in Table 33 of IS code)

r_0 = roughness factor at the reference height of 10 m above ground level

= $g_f\, r / 3.7$ (value of $g_f\, r$ is given in Fig. 8 of IS Code)

The responses of the structure due to these wind load patterns are combined to obtain the r.m.s. of fluctuating response given by

$$\tilde{x}_{PL} = \sqrt{\sum_{i=1}^{n} x_{PLi}^{\,2}} \qquad (32)$$

where x_{PLi} is the response from the i[th] load pattern and 'n' is the total number of load patterns.The peak fluctuating response, $\hat{x}_{PL}$, is obtained as

$$\hat{x}_{PL} = \tilde{x}_{PL}\, \lambda_B\, \lambda_R\, \lambda_{TL}\, g \qquad (33)$$

where λ_B is the background scaling factor, λ_R is the resonant magnification factor, λ_{TL} is the turbulence length scale factor, and g is the statistical peak factor. For simplicity, Davenport assigned conservative numerical values to obtain the peak fluctuations response as

$$\hat{x}_{PL} = 3.78\, \tilde{x}_{PL} \qquad (34)$$

All the three methods have been used by SERC in the design of several guyed towers ranging from 45m to 120m tall and member sizes have been arrived by choosing the maximum of forces arising out of the three methods. One of the guyed tower 50m tall, designed and erected by SERC was also tested in full scale at site to compare the measured responses (*Harikrishna et al, 2003*).

5.0 Design and testing of tower-like structures

For the design of sections peak stress responses are needed and the same are computed using the peak displacement responses at all levels using standard retrieval methods for element stress evaluation in commercially available finite element software systems. For static analysis(Eq.2) for the 3-second gust or GRF factored mean loads, or any other equivalent static loads as in guyed tower, could be used to arrive at the global displacement response either using linear or nonlinear analysis. Using the transformed local displacement vectors at the member degrees of freedom one will be able to evaluate the member end forces/moments and extreme stresses, along the wind direction or compute the principal stresses at any point in shell structures by numerical integration to assess the factor of safety against permissible stresses of the material. In the case of guyed towers after computing the wind loads in all the three methods in various combinations of load cases nonlinear-large displacement analysis should be adopted in the commercial software packages with small increments of computed wind loading. The critical load case should be identified for bending and shear and the member designs should be checked.

For dynamically sensitive structures, the free vibration analysis using (Eq.3) is done using many of the available software or by Rayleigh's coefficient approach for lumped parameter models of tower-like structures. In section 4.0 we discussed an equivalent pole model of tower like structure having one lateral displacement degree of freedom in the lateral direction along the direction of wind at each level. However, three dimensional structures are multi degree of freedom systems which require condensation and retrieval techniques to use linear modal superposition using the spectral approach. Several text books address this practical issue and brief representative steps are given in Appendix-A. For design of structures member level extreme stresses computed using an array/matrix(3D-models) of peak displacements (mean and fluctuating using Eq.28) should be within the limits of material strength. As a precaution, it may be stated that in the case of guyed towers evaluation of natural modes and frequencies are applicable, but linear modal superposition is not applicable since it is geometrically nonlinear. In the case of very slender guyed towers it is preferable to apply the mean wind load and update physically the geometry of the tower with the resulting deformations at all nodes and go for solving the eigen value problem to obtain the natural frequencies and mode shapes.

Testing of tower like structures either in wind tunnel or in full scale has certain pros and cons, but helps in disaster preparedness (*Lakshmanan et al, 2002*). The results of response for a survival wind speed(extreme wind) may require wind tunnel studies prior to design, since simulation of such high wind conditions are not instantly possible as and when needed in full scale. The essentials of wind tunnel testing is covered in another lecture (*Ramesh Babu, 2007*). Some typical application of along wind response computation, together with the results of full scale experiments conducted by SERC are discussed in what follows.

6.0 Applications :

(i) Along wind Response of a 101m Tall Microwave Tower

A 101m tall microwave tower situated at Chennai(Madras), on the east coast of India, was instrumented at 5 levels with anemometers and accelerometers and the data were acquired simultaneously. The tower has a square base of 14 x 14 m at the bottom and tapers to 1.8 m x 1.8 m at 90.5 m level. The tower supported eleven dish antennae mounted at various levels and it also had other appurtenances such as platforms along height, ladder, cables and wave guide (Fig. 7). The details of experimental investigation and wind and terrain characteristics have been given elsewhere (*Shanmugasundaram et al, 1995, 1996*). The natural frequency and modes were

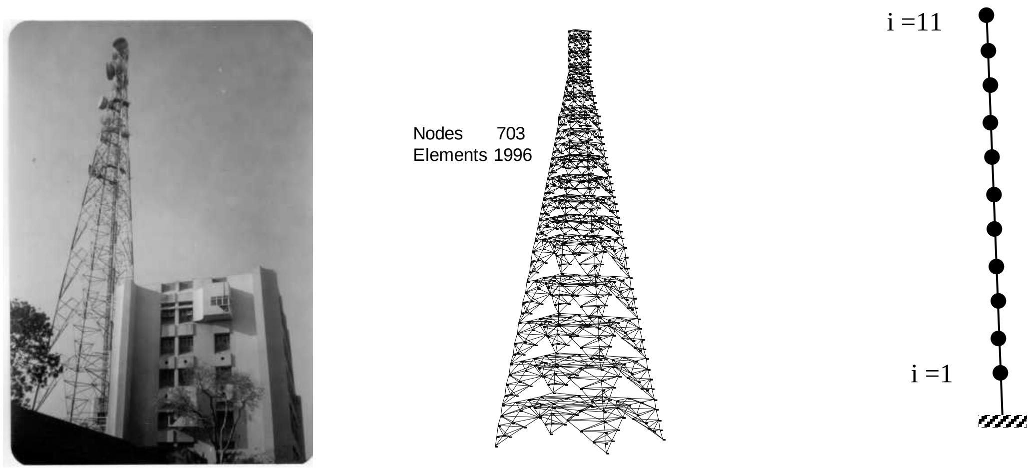


Fig. 7 View of the microwave tower near Madras harbour and 3D-FEM/pole models.

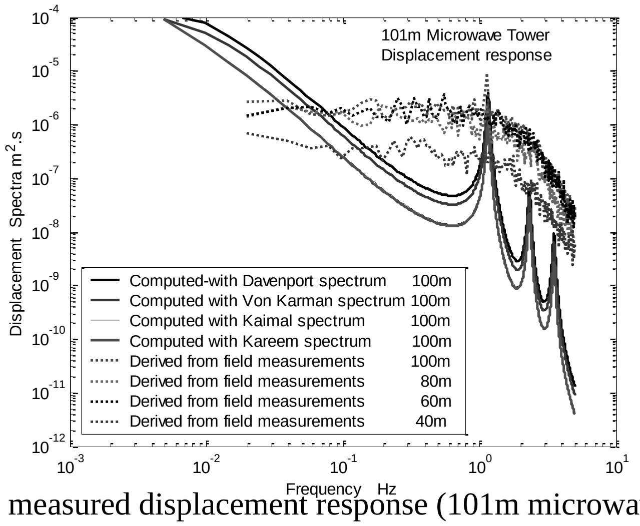


Fig.8 Computed and measured displacement response (101m microwave tower)

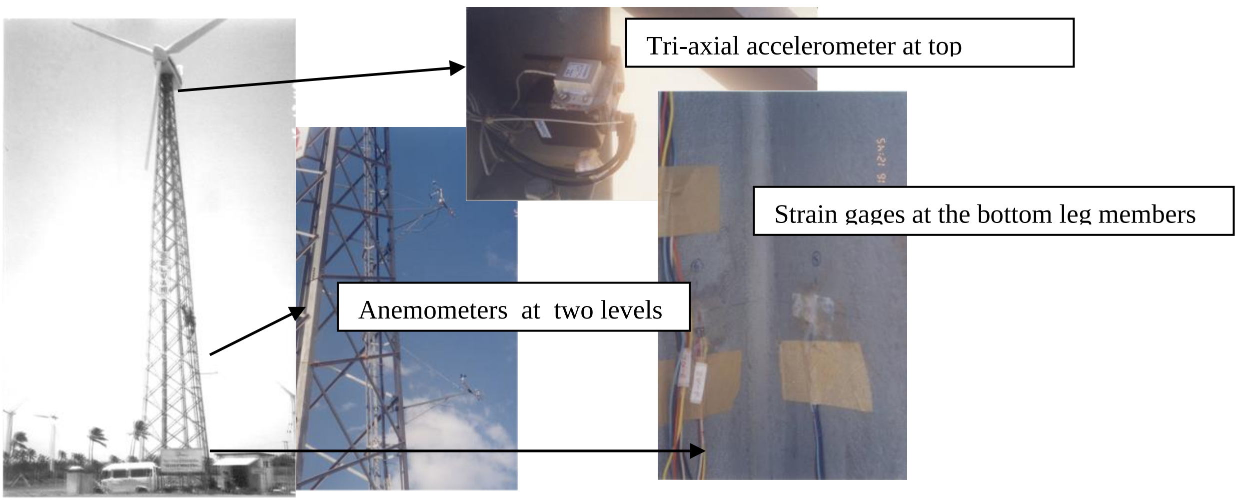
computed by adopting a 3D-truss model with masses lumped at the various joints. A finite element idealization of the tower used for free vibration analysis and an idealized pole model used for dynamic response computation are shown in Fig.7. A 3-D FEM (Finite Element Method) model has 703 nodes and 1996 truss elements and the antennae were modeled as lumped masses and the pole model has 11 levels of parameters. The first three natural frequencies are found to be 1.16, 2.32 and 3.52 Hz. of which the fundamental frequency of 1.16 Hz is seen aligning with the peak in the measured acceleration spectra at various levels. The displacement response shown in Fig.8 has been evaluated by double integrating the measured acceleration time series. The assumed overall damping(structural and aerodynamic) as a percentage of critical damping has been found to influence the dips found in the computed response spectra between the modal peaks. For practical estimation of antenna wind loads and dynamic effects additional reading is suggested (Gomathinyagam et al, 2000b).

(ii) Response of a Wind Mill Support Tower

SERC has been involved (Gomathinayagam et al, 1996 to 1999) in the analysis, design and field testing of these wind mill support towers since 1995. In the year 2002, an existing wind mill tower of 30 m hub height was extended up to 42 m hub height at its installed site, to harness more power from wind using the existing wind turbine. SERC had instrumented (*Gomathinayagam,2002,2004*) the extended wind mill tower and measured at site the wind and response characteristics to assess the structural safety. The details of the testing and results of the measured in-service fatigue loads are presented.

Details of Field Measurements

The 42m tall four legged lattice tower supporting 225 kW Wind Turbine situated about 15 km North of Kanyakumari was instrumented for wind and response measurements. The terrain has scattered palm trees, and wind mills on all the directions. The self weight of the tower is 15 tonnes. On the tower top, a weight of 11 tonnes consisting of 3 tonnes of rotor-blade system and 8 tonnes of gear-train and generator system is housed. Fig. 9 shows the view of the test tower from its west face along with details of instrumentation. The collected data were processed and analysed later at Field Experiments Laboratory, SERC and some of the results are given here to illustrate the use of analysis and full scale testing.



Used with the permission from M/s Victory Wind farms Chennai

Fig.9 View of the instrumented wind mill support tower(42m)

Measured Data Analysis

Using special purpose wind and response analysis package called "WINRES", (*Harikrishna et al, 1999*) developed at Field Experiments Laboratory, SERC, all the measured data were analysed to evaluate various statistical and spectral characteristics along with stationarity checking of wind speed data. Special purpose program was also developed for fatigue cycles evaluation using rainflow counting technique. All the measured data were grouped in bins of mean wind speed measured at 16 m level from 4 m/s to 16 m/s. The various wind conditions and number of records measured in the field experimental program (*Gomathinayagam et al, ,2002*) are given in Table 1.

Wind and Tower response characteristics

During the period of measurement most of the records indicated a predominant mean wind direction close to 270^o (West) with marginal swing in the South-West and North-West directions.

Table 1 Part results of calculated Mean Wind Speeds at Hub Heights

Wind Speed Bins (No.of Records)	Measured Average Mean Wind Speed at 16 m (m/s)	Calculated Mean Wind Speed (m/s)		Expected increase in wind power at 43.5 m
		At 31.5 m	At 43.5 m	
4 -- 5 (2)	4.55	5.27	5.65	23.2 %
5 -- 6 (10)	5.45	6.31	6.76	23.2 %
..	..	..	..	..
7 -- 8 (20)	7.45	8.61	9.23	23.2 %
10 -- 11 (32)	10.52	12.16	13.04	23.2 %
13 -- 14 (8)	13.60	15.73	16.86	23.2 %
15 -- 16 (1)	15.24	17.63	18.90	23.2 %

The measurements were observed to have highly fluctuating wind speeds at both 10 m and 16 m levels. The average mean wind speeds in various bins ranged from 4.55 m/s to 15.24 m/s. The turbulence intensities varied from 19.9% to 36.2%. The maximum recorded wind speeds were 11.28 m/s to 27.49 m/s.The average power law coefficient, α, for the terrain was evaluated as 0.215. Estimated mean wind speeds using the power law, at 31.5 m and 43.5 m levels in various bins along with the expected power increase of 23% at the extended height are given in Table 2.The measured accelerations at the top of the tower below the turbine housing were double integrated to evaluate the dynamic displacements of the tip of the tower under various operating conditions. The standard deviations of displacements are in the range of 0.0017 to 0.0050m and the maximum dynamic displacement was within 0.04m during one of the normal operating condition with average mean wind speed of 10.52 m/s. The strain time histories measured at the base of the tower were converted to dynamic stress ranges which had a varying magnitude of 25-40 MPa.

Fatigue Cycle Counting from Measured Stress Ranges

Using the measured stress ranges in various bins of wind speeds, fluctuating cycles of the stress ranges in the operating conditions were evaluated, using rain-flow counting technique (Table 2). Even though the tail ends of the wind speed bins had lesser number measured data, the nominal rated operating range had sufficient number of records to get an on the spot assessment of the possible fatigue damage, which is often cumulative for the entire designed life of turbine and tower. If the number of cycles "Ns" permitted in a specific stress range $\boldsymbol{\sigma_{si}}$, then the value of measured number of cycles "n_{si} " can be taken from the respective wind speed bin for the leg. Then the following equation can be used to estimate the annual occurrence of fatigue damage, which is cumulative sum of all the operating range of wind speed. Damage in one year D_{annual}, is given as

$$D_{annual} = \sum_{i=1}^{Ws} D_i = \sum_{i=1}^{Ws} \sum_{s=1}^{Ts} (n_{si}/N_s) * (Hi)$$

Where, D_i is fatigue damage in ith wind speed bin

n_{si} is number of cycles for the stress range $\boldsymbol{\sigma_{si}}$

Ws is the number of operating wind speed bins

Ts is the number of active stress ranges in ith bin

Hi is the number of hours of operation in the ith bin

N_s is the number of permissible cycles for stress range $\boldsymbol{\sigma_{si}}$

It may stated that the permissible number of cycles "N_s" is purely a material fracture property obtained from available fatigue S-N data for the specific detailing from literature.

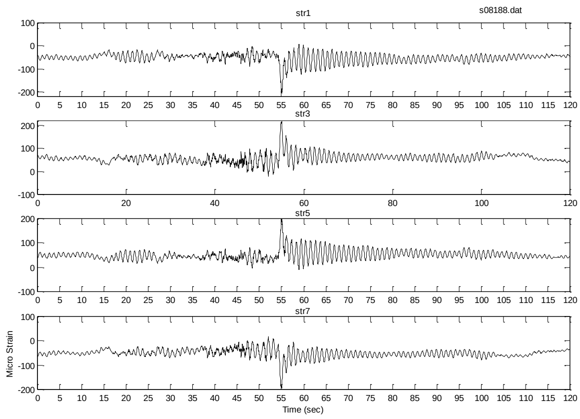


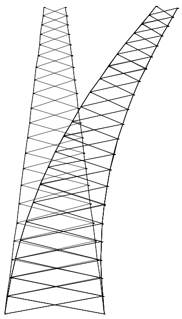
Fig. 10 Time Histories of Leg Member Strains during Over Speed Brake Event

Table 2 Measured Cycles of Dynamic Stress Ranges per Hour for Leg 3

	Mean wind speed Bins (m/s)								
Stress Range MPa	7-8	8-9	9-10	10-11	11-12	12-13	13-14	14-15	15-16
3.2	4623	4796	4652	4330	3948	3910	3436	3180	3204
9.5	56	126	226	387	508	592	745	796	820
12.6	20	42	72	138	211	275	374	416	428
15.8	8	18	27	53	83	127	165	162	216
22.1	3	3	4	8	12	23	28	20	36
28.4	1	1	1	1	2	2	4	6	4
31.5	0	1	1	1	1	2	3	2	0
34.7	0	0	0	1	0	0	1	2	4
37.8	0	0	1	0	0	0	0	0	0
41.0	0	0	0	0	0	0	1	0	0

Measurements under Special Turbine Operating Conditions

In addition to measurements under normal operating conditions, measurements were also made under the following special operating conditions (i) turbine starting from stalled condition, (ii) turbine brake while operating at high and low wind speeds, (iii) free wheeling of rotor with generator off, (iv) manual yawing of Turbine while operating at high wind speeds, and (v) grid failure. Typical strain response during a sudden braking event is depicted in Fig.10. The evaluated displacement spectra of two components at the top of tower, on two diagonally opposite legs of the tower are given in Fig.11 along with the computed displacement spectra. However, the resonant response is quite well predicted by the computational model with the use of input wind parameters derived from field measurements. In the computation, two modes have been considered. The spectra (Fig.12) computed using measured strain traces were more accurate representation of wind response, which clearly indicate the background response as well as resonant response.



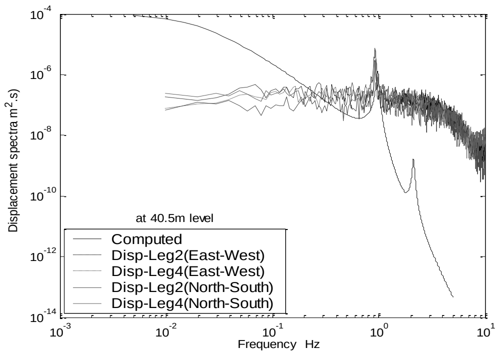


Fig 11 Computed and measured responses of the turbine support tower (0.95 Hz)

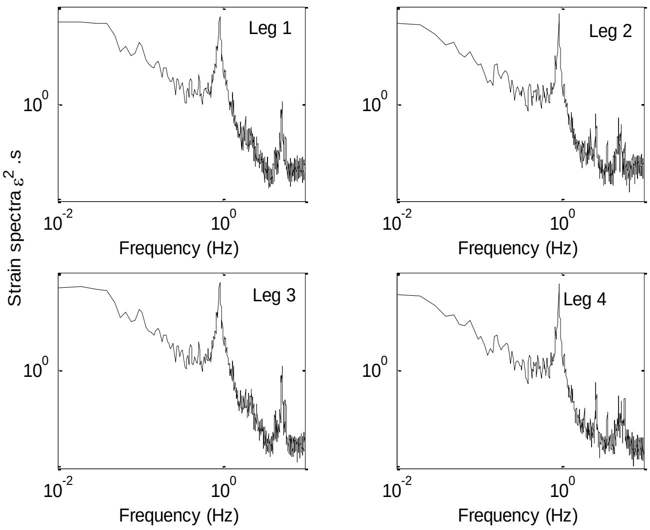


Fig. 12 Spectra of measured strain at the bottom of four legs

In the computational model, nominal wind loading based on projected areas of rotor blades in stationary but stalled (least resistance to wind) position was also included.

(iii) Wind response of a latticed derrick drilling tower on deck of an offshore platform

Due to difficulties in simulation using wind and wave tunnels, only a limited number of sections, having a base width of 8.2m has been used as shown in Fig.13 to evaluate the dynamic wind response. 3-D beam elements were used to model the latticed derrick structure by scaling up to proto type dimensions Free vibration analysis has been done using a lumped parameter model and the results are given in Fig. 13. Results of the study on the effects of variation of mean wind speed and standard deviation of wind speed along height are given in Fig.14. The effect of mean wind speed is to slightly increase the aerodynamic damping. The widening of the resonant regime, as seen in Fig.14(a), towards the low frequency regime of wind turbulence spectrum is the characteristic of increased aerodynamic damping and the decay of the response spectrum to the right side of the resonant peak becomes steeper as the wind speed increases.

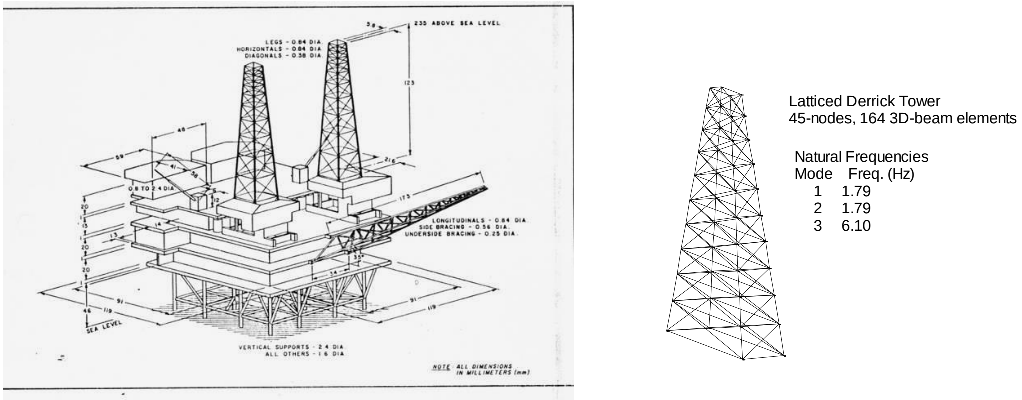
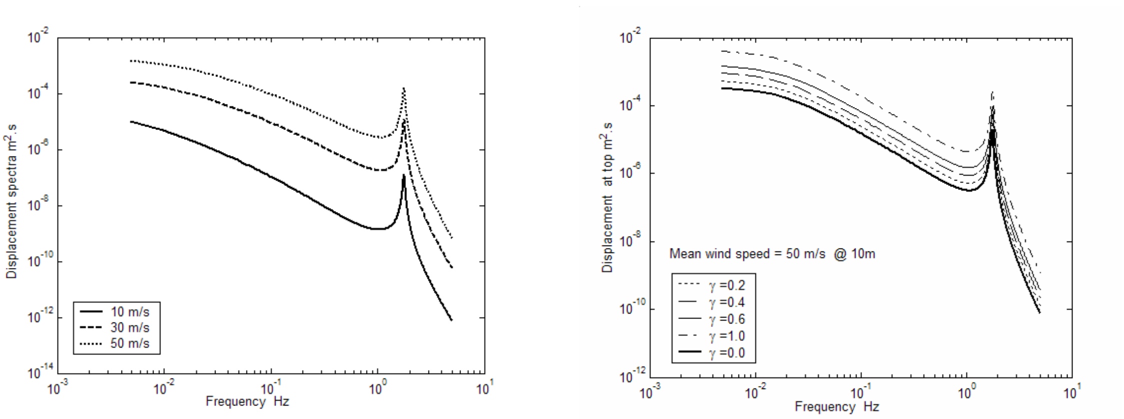


Fig. 13 View of offshore deck tower (*Vickery and Pike 1985*) and FEM model



(a) Variation Mean wind speed (b) Std. deviation of wind speed along height

Fig. 14 Displacement response spectra on top of a drilling tower

There is a school of thought that the velocity fluctuations remain constant along the height, while the mean wind velocity increases resulting in decrease of turbulence intensity over the height above the surface (ground or ocean). However over a decade of multi-level wind measurements at several sites in India, it has been observed that the variance of velocity fluctuations varies along the height, which means the wind turbulence spectrum is height dependant (*Kaimal et al., 1972*). For individual sites, this variation may be fitted to a power form with an even powered function represented by the index (2γ) with respect to a value at reference height. The variation of γ thus provides an insight into the variation of the standard deviation along the height and hence the height dependency of wind turbulence spectrum may have to be used in the wind response computation at least in some cases. When γ =0.0 the standard deviation of wind velocity remains constant along the height of structure.

Some tips for tower-like structures

Analysis

- Finite Element Modeling (FEM) of tower like structures should avoid local modes which may involve a few plan-braces, or a weak beam/column of a tall building frame.
- The frequency used in GRF must be for the first global bending mode and should be used in "Hz" units (cycles/second) in the given formula. It is essential to verify the mode shape before the frequency is used.
- While using GRF method we must bear in mind that only first (that too a linear mode) mode is used for the derivation of the codal expressions.

Design

- Design of steel sections should be as per working stress design while using IS875 loading for microwave communication towers, guyed towers and other tower like structures.
- Transmission towers however make use of full strength of material without load factors applied on to the wind load calculations, to minimize weight as per industry-acceptance.
- For important towers in cyclone prone areas additional factor of safety may be exercised.
- Wind mill tower designs should aim at tuning the fundamental frequency (by choice of sections and adjusting the mass and stiffness) for avoiding resonance due to the proximity of tower frequency not only to rotor RPM but also to blade passing frequency.
- Fatigue of wind mill towers under operational loads must be checked for the design life, for Indian turbulent wind conditions.

Testing

- As the analysis and design involve several assumptions on various inputs specially the foundation fixity or soil conditions, which may need verification by testing for future improvement in designs.
- Wind tunnel testing can get basic wind loading parameters in a scaled physical model including responses and interference effects .
- Full scale testing can give the confidence of the design in terms of integrity of stiffness, mass, and damping through ambient vibration measurements.
- In the case of wind mill support towers operational fatigue load spectrum for Indian turbulent wind conditions can be developed using measurements, for design of towers of taller hub height for the same wind turbine to harness more seasonal wind power.

The author earnestly hopes that the bibliography given would be useful to learn from SERC's experience in the analysis, design and testing of tower-like structures.

Appendix –A

Spectral response of tower-like multi-degree-of-freedom (MDOF) structures

For a multi-degree of freedom system (MDOF) the implementation of structural response prediction after evaluation of the eigen values (Eq.9) and eigen vectors (mode shapes) in a condensed coordinate system, in tower-like structures the lumped parameter model applied for spectral approach. But, if a stress spectrum is sought for design purposes (like the one shown in measured strain in Fig. 12), the displacement spectra of total system, (ie. at all the degrees of freedom) has to be retrieved from the condensed co-ordinate system. The operations involved in condensation and retrieval is possible in few dynamic analysis software and the essential steps as given in (*Keshava Rao 1991 ; Mario Paz 1997*) are given for completeness.

A transformation matrix is derived (*Mario Paz 1997*) by partitioning the stiffness matrix [K] as dependant or secondary degrees of freedom, X_s , and express them in terms of the remaining independent or primary degrees of freedom X_p. Mathematically this has been given as

$$\left[\begin{array}{c|c} K_{ss} & K_{sp} \\ \hline K_{ps} & K_{pp} \end{array}\right] \left\{\begin{array}{c} X_s \\ \hline X_p \end{array}\right\} = \left\{\begin{array}{c} O \\ \hline F_p \end{array}\right\} \qquad \text{(A.1)}$$

Thus the transformation to the condensed system has been achieved by

$$X_s = \overline{T}\, X_p \qquad \text{(A.2)}$$

where

$$\overline{T} = -K_{ss}^{-1} K_{sp} \qquad \text{(A.3)}$$

when a solution is sought in a primary (reduced) co-ordinate system, the condensed stiffness matrix has been obtained as,

$$\overline{K} = K_{pp} - K_{ps} K_{ss}^{-1} K_{sp} \qquad \text{(A.4)}$$

From Eq.(A.2) to Eq (A.3) it follows,

$$X = \left\{\begin{array}{c} X_s \\ \hline X_p \end{array}\right\} = [\mathrm{T}]\ X_p$$

$$\text{where}[\mathrm{T}]=\left[\begin{array}{c} \overline{T} \\ \hline I \end{array}\right] \qquad \text{(A.5)}$$

For practical use of static condensation, numerical implementation can be efficiently made in the elimination process, so that at this stage, Eq.(A.1) is reduced to

$$\left[\begin{array}{c|c} I & \overline{T} \\ \hline O & \overline{K} \end{array}\right] \left\{\begin{array}{c} X_s \\ \hline X_p \end{array}\right\} = \left\{\begin{array}{c} O \\ \hline F_p \end{array}\right\} \qquad \text{(A.6)}$$

where

$$\overline{K} = T^T\, K\, T \qquad \text{(A.7)}$$

The transformation 'T' if applied to the mass 'M' and damping 'C' replacing 'K' in Eq(A.7), the reduced mass matrix $\overline{M}$, and damping can be obtained which have been given as

$$\overline{M} = T^T\, M\, T \quad \text{and} \quad \overline{C} = T^T\, C\, T \qquad \text{(A.8)}$$

Solving dynamic equilibrium equations in the condensed coordinate system and using the frequencies and mode shapes, the mode superposition analysis of the condensed system has been performed. The modal generalized force spectral density of the condensed system has been given as

$$\bar{S}_{ff}(\omega) = \Phi_c^T \tilde{S}_{ff}(\omega)\Phi_c \qquad \text{(A.9)}$$

where is $\tilde{S}_{ff}(\omega)$ the measured/evaluated force spectrum at the condensed coordinate or the computed force spectra using the wind turbulence spectra and the aerodynamic admittance. Φ_c represents the mode shape vectors arranged in each row and $\bar{S}_{f_j f_k}(\omega)$ denotes the generalized element of the cross spectral modal force matrix involving the j^{th} and k^{th} modes of the condensed co-ordinate system. The power spectral density of modal response has been obtained using the matrix triple product,

$$\bar{S}_{\eta_j \eta_k} = H_j^*(\omega)\, \bar{S}_{f_j f_k}\, H_k(\omega) \qquad \text{(A.10)}$$

where $H(\omega)$ has been a diagonal matrix of modal receptances with its j^{th} diagonal expressed in the form of complex frequency response function, as

$$h_{jj} = \frac{1}{\omega_j^2 (A_j + iB_j)} \qquad \text{(A.11)}$$

where

$$A_j = 1 - \beta_j^2$$

$$B_j = 2\xi_j \beta_j$$

$$\beta_j = \frac{\omega}{\omega_j}$$

ω denotes the forcing frequency variable (functional indicator in radians/s). The complex frequency response function $H^*(\omega)$ has in its diagonal, complex conjugate corresponding to $H(\omega)$. Typically, the element h_{jj}^* has to be,

$$h_{jj}^* = \frac{1}{\omega_j^2 (A_j - iB_j)} \qquad \text{(A.12)}$$

Since $H^*(\omega)$ and $H(\omega)$ are diagonal, the elements of modal response matrix can be defined in the same form as given in Eqn (A.9), omitting the functional indicator ω, for convenience and can be expressed as ,

$$S_{\eta_j \eta_k}^* = h_{jj}^* \bar{S}_{f_j f_k}\, h_{kk} \qquad \text{(A.13)}$$

from Eqn (A.10) and Eqn. (A.12)

$$h_{jj}^*\, h_{kk} = \frac{1}{(R_{jk} + iI_{jk})} \qquad \text{(A.14)}$$

where

$$R_{jk} = \omega_j^2\, \omega_k^2\, (A_j A_k + B_j B_k)$$

$$I_{jk} = \omega_j^2\, \omega_k^2\, (A_j A_k - B_j B_k) \qquad \text{(A.15)}$$

It may be pointed out that, the modal matrices $H(\omega)$ and $\bar{S}_{f_j f_k}$ have been complex and hence the response $S^*_{\eta_j \eta_k}$. Separating the real and imaginary parts, for the sake of numerical implementation, and representing the real part of the force spectrum $\bar{S}_{f_j f_k}$ by $F^R(\omega)$ and the imaginary part by $F^I(\omega)$, we have

$$\bar{S}_{f_j f_k} = F^R(\omega) + \mathrm{i}\, F^I(\omega) \qquad \text{(A.16)}$$

Similarly rearranging Eqn.(4.53) the real and imaginary parts of the complex receptance whose magnitude is known as mechanical admittance while applying to the wind response analysis, it has been denoted as

$$h^*_{jj}\; h_{kk} = \frac{R_{jk}}{(R^2_{jk} + I^2_{jk})} - i\, \frac{I_{jk}}{(R^2_{jk} + I^2_{jk})} \qquad \text{(A.17)}$$

The cross modal response spectrum has been now given by

$$S_{\eta_j \eta_k} = (F^R_{jk}(\omega) + i\, F^I_{jk}(\omega)) \left[\frac{R_{jk}}{(R^2_{jk} + I^2_{jk})} - i\, \frac{I_{jk}}{(R^2_{jk} + I^2_{jk})} \right] \qquad \text{(A.18)}$$

Following the simplicity of the notation as noted in Eqn.4.55 the modal response may now be denoted as

$$S_{\eta_j \eta_k} = \eta^R_{jk} + i\, \eta^I_{jk} \qquad \text{(A.19)}$$

where

$$\eta^R_{jk} = \frac{(F^R_{jk} R_{jk} + F^I_{jk} I_{jk})}{(R^2_{jk} + I^2_{jk})} \qquad \text{(A.20)}$$

$$\eta^I_{jk} = \frac{(F^R_{jk} R_{jk} - F^I_{jk} I_{jk})}{(R^2_{jk} + I^2_{jk})} \qquad \text{(A.21)}$$

the response spectral densities which have been in modal coordinates 'η', have to be transformed to the condensed system coordinates using the modal operation,

$$S^c_{xx}(\omega) = \Phi_c\, S_{\eta\eta}(\omega)\, \Phi^T_c \qquad \text{(A.22)}$$

Using the spectra in the condensed coordinate system the variance and statistics can be computed and these represent the response of the condensed/reduced system. For the three dimensional towers, the spectral analysis programs have retrieval facility also, for computing the response statistics and variance at all the secondary degrees of freedom using those obtained in the primary degrees of freedom (Eq.A.23). The reduced system spectral analysis of three dimensional tower like structures is analogous to the popular methods of wind response analysis of line-like structures (*Simiu and Scanlan 1996; Holmes 2001*) and tower-like structures using one of the following wind turbulence spectra as the primary input for wind response.

Davenport spectrum

$$\frac{n\,S_u(n)}{u_*^2}=\frac{4.x_f^2}{\left(1+x_f^2\right)^{4/3}} \qquad (A.23)$$

where $x_f=\frac{n\,L_u^x}{\overline{U}_z}$ denotes the reduced frequency, and L_u^x, the length scale of turbulence. Measured variance of fluctuating velocity is related to the shear friction velocity u_* by the relation $\sigma_{uz}^2=\beta_t\,u_*^2$. Taking an average value of terrain dependant derived parameter β_t = 6.0 for open terrain, Davenport and Vickery (1989) have suggested a convenient form instead of Eq.(A.23):

$$\frac{n\,S_u(n)}{\sigma_{uz}^2}=\frac{2.x_f^2}{3\left(1+x_f^2\right)^{4/3}} \qquad (A.24)$$

Von Karman spectrum

$$\frac{n\,S_u(n)}{\sigma_{uz}^2}=\frac{4.x_f}{\left(1+70.8x_f^2\right)^{5/6}} \qquad (A.25)$$

The Kaimal spectrum

$$\frac{n\,S_u(n)}{\sigma_{uz}^2}=\frac{200\,x_f}{\beta_t\left(1+50\,x_f\right)^{5/3}} \quad \text{with} \quad x_f=\frac{n\,z}{\overline{U}(10)} \qquad (A.26)$$

where the reduced frequency $x_f=\frac{n\,z}{\overline{U}(10)}$, is computed with the mean wind velocity at z = 10m and the length scale L_u^x replaced by elevation z, above the surface.

Retrieval of response statistics for design

Once the peak responses are available in condensed co-ordinate system the required extreme value statistics can be computed. The mean and standard deviation of stresses in various elements are of ultimate interest in any design. To achieve this stress statistics, the statistics of all global degrees of freedom of uncondensed co-ordinate system must be available. The mean and standard deviation of global displacements may be retrieved from the condensed system displacements treating them as prescribed (imposed) displacements along the condensed coordinates in many analysis software.

Bibliography

Abraham, A., (**2007**) ‘ Wind loading standards for buildings and other industrial structures’ allied topic in this volume, ADSWSL, 7-9 February , SERC, Chennai, 2007

Arunachalam, S., (**2007**) ‘ Wind Characteristics for structural Design’ and ‘ Analysis and design of tall chimneys and cooling towers for dynamic wind action’ allied topics in this volume, ADSWSL, 7-9 February , SERC, Chennai, 2007

Cook, N.J., “The designer’s guide to wind loading of buildings and structures Part 1”, Butterworths, London, **1985**.

Cook, N.J., "The designer's guide to wind loading of buildings and structures Part 2 Static Structures ", Butterworths, London, **1990**.
Davenport, A.G.,(**1961**)"Application of statistical concepts to wind loading of structures", Proceedings of Institution of Civil Engineers, Vol.19, August 1961, pp 449-472.
Davenport, A.G., (**1963**), The relationship of wind structure to wind loading, *Proc.wind effects on buildings and structures*' National Phy. laboratory, Middlesex, U.K, **1,** 54-102
Davenport, A.G.,(**1967**) "Gust loading factors", Journal of Structures Division, ASCE, Vol. 93, 1967, pp 11-34.
Davenport, A.G.,(**1992**) "Dynamic Gust Response factors for guyed towers", Journal of Wind engineering and Industrial Aerodynamics", Vol.43, pp 2247-2248.
Davenport, A.G. (**1977**), The prediction of response to gusty wind, *Int. Res. Seminar Safety of Structures, under dynamic loading,* Trondheim, Norway, **2,** Tapir, 1978, 257-284
Dyrbye, C. and **S.O Hansen**, *Wind Loads on structures,* John Wiley & Sons, NY, **1996**
ECCS-(1985) TC 6 *Fatigue – Recommendations for the fatigue design of steel structures,* Report No. 43 , European Convention for Constructional Steel, 1985
Gomathinayagam S., P.Harikrishna, B.Venkateswarulu and J.Shanmugasundaram, "Comparative Study of 3-D and Pole Model For Wind Induced Responses of Tall Lattice Towers", Proceedings of the National Symposium on Tall Structures, at REC, Tiruchirapalli, February 2-3, **1995**.
Gomathinayagam, P. Harikrishna, , J.Shanmugasundaram and N.Lakshmanan , Analysis and design of support towers for wind turbines – state of art and software, research report, SERC, Madras, March **1996**
Gomathinayagam S., P. Harikrishna, and J.Shanmugasundaram, Temporal Analysis of Towers to WINd effects – TATWIN – user guide –a Research report , SERC, Madras, August **1996**
Gomathinayagam S., J.Shunmugasundaram, P.Harikrishna, N.Lakshmanan, Design and Field testing of Support towers of Wind electric Generators, Proc. of II Natioanl Confernce on wind engineering , SERC,at Gazhiabad, U.P, pp 364-376 April , **1997**
Gomathinayagam, S., J. Shunmugasundaram, P. Harikrishna, N.Lakshmanan (**1999**), Field testing of Support towers of Wind electric Generators, Proceedings of International Conference on Structural Engineering, Institution of Engineers (India) at Gazhiabad, Published in Text book on Advances in Structural Engineering, Phoenix , 1999 September, 77-88
Gomathinayagam S., C.P.Vendhan, J.Shunmugasundaram, Dynamic Wind effects on offshore deck structures-Critical review of provisions and practices ,International Journal of Wind Engineering and Industrial Aerodynamics, vol 84 , n.3 pp345-367, Mar **2000 a**
Gomathinayagam S., J. Shanmugasundaram, P. Harikrishna, N. Lakshmanan and C. Rajasekaran (**2000 b**), Dynamic Response of a lattice Tower with Antenna under Wind Loading, *Journal of the Institution of Engineers (India)*, **81**, 37-44
Gomathinayagam, S., Lakshmanan, N., Harikrishna, P., Annadurai, A., and Sekar, N., " Measured fatigue loads of wind mill support tower with exteneded hub-height for enhanced power generation", National conference on wind engineering NCWE-04, Nagpur, Volume-II, Feb **2004** , pp 388-395
Gomathinayagam S., (**2005**) 'Dynamic wind loading and response of structures using full scale experiments in natural wind' –PhD-thesis- Ocean Engineering Department, Indian Institute of Technology, IIT, Madras , Chennai 2005
Gomathinyagam S., Lakshmanan, N., Vendhan, C.P., (**2005**), ' Random wind loading on structures close to ground – A field approach in natural wind', Proc. of National symposium on Structural dynamics, Random vibrations and earthquake engineering, ,Indian Institute of Science, Bangalore, pp 125-133, July 21-22, 2005
Gomathinayagam S., Harikrishna, P., A.Abraham, and N. Lakshmanan(**2006**) , ' Bi-modal spectral method for evaluation of along wind induced fatigue damage', Wind and Structures: An international Journal, 9(4), July 2006 , 255-270
Harikrishna, P., J. Shanmugasundaram, S. Gomathinayagam, and N. Lakshmanan (**1999**) Analytical and experimental studies on gust response of a 52 m tall steel lattice tower under wind loading , *Computers and Structures* , **70**, 149-160
Harikrishna, P.,(**2003**) Annadurai, A., Gomathinayagam, S. and Lakshmanan, N., "Full Scale Measurements of the Structural Response of a 50 m Guyed Mast under Wind Loading", Engineering Structures, Vol.25, Issue 7, 859-867, June, 2003
Harikrishna P., (**2007**) ' Post-disaster damage surveys and their implications to structural design' allied topic in this volume, ADSWSL, 7-9 February , SERC, Chennai, 2007
Holmes, J.D. (**1996**), Along-wind response of lattice towers – II Aerodynamic damping and deflection, *Engineering Structures*, **18, 7**, 483-488
Holmes, J.D., *Wind loading of Structures*, Spon Press NY **2001**

IASS: (**1981**), Recommendations for guyed masts Working group no.4, International Association for shell and spatial structures, Madrid, Spain.
IS:875(Part 3)-1987, "Indian Standard Code of Practice for Design Loads (other than Earthquake) for Buildings and Structures, Part 3: Wind Loads", Bureau of Indian Standards, New Delhi, **1989.**
Kaimal, J.C., J.C. Wyngaard, Y. Izumi and **O.R. Cote,** (1972), Spectral characteristics of surface-layer turbulence, *Journal of the Royal Meteorological Society,* **98**, 563-589
Keshava rao, M.N., Dharaneepathy, M.V., Gomathinayagam, S., Rama Raju, K., and Sudhesh, K.G.,(1991) " OSTA software – Offshore –Onshore structural analysis under UNIX system, International journal Computers and Structures, 38, (2), 185-201, 1991
Lakshmanan, N., S. Gomathinayagam, P. Harikrishna and A. Annadurai (**2002**), Full scale field experiments for cyclone disaster preparedness, *Proceedings of the DAE-Symposium on cyclone emergency Preparedness*, Paper CP-10, January 30-31, 2002 IGCAR, Kalpakkam, 207-213
Lakshmanan, N., Gomathinayagam, S**.**, Harikrishna, P., Annadurai, A., and Shakeer Mohamed, " Measured wind induced response and fatigue of steel lamp mast" Proc. Of 11th international conference on wind engineering ICWE-03 ,Lubbock, Texas, USA , June **2003**
Lakshmanan, N., Gomathinayagam, S., Annadurai, A., and Harikrishna, P., " Meteorological and measured wind speed probability distribution for fatgue life prediction", National conference on wind engineering NCWE- 04, Nagpur, Volume-II, Feb **2004** , pp 379-387
Mario Paz, (**1997**), Structural dynamics theory and computation , Van Nastrand Reinhold Co, USA, **1997**
Nigam N.C. and **S.Narayanan**, *Applications of Random Vibrations*, Narosa Publications, New Delhi, **1994**
Ramesh Babu, G., (**2007**) ' Wind tunnel testing of structures ' allied topic in this volume, ADSWSL, 7-9 February , SERC, Chennai, 2007
Selvi Rajan, S, (2007) ' Dynamic wind loads on bridges ' allied topic in this volume, ADSWSL, 7-9 February , SERC, Chennai, 2007
Shanmugasundaram, J., M. Arumugam, B.Venkateswarlu, P. Harikrishna and S**.** Gomathinayagam**,** (**1995**) Full scale field experiments on a 101m high lattice tower, *Proceedings of 9th International Conference on Wind Engineering (9 ICWE), New Delhi*, India, , January, 445-456
Shanmugasundaram, J., S. Gomathinayagam, P. Harikrishna and N. Lakshmanan, (**1996**), Fullscale Measurements of Dynamic Response of a Lattice Tower, Proceedings of the *International Seminar on structural assessment, The Role of Large and Full-scale Testing, Institution of structural Engineers/City University , London*, U.K, July 1996, Structural assessment, Publis. E & F SPON pp 363- 372
Shanmugasundaram J., P.Harikrishna, S.Gomathinayagam, N.Lakshmanan, Wind, terrain and structural characteristics under tropical cyclone condition, International Journal of Engineering Structures, vol.21, **1999**, pp1006-1014.
Shanmugasundaram J., N.Lakshmanan,S.Arunachalam,S.Gomathinayagam, Damage to buildings and structures in Gujarat Cyclone, International Journal of Wind Engineering and Industrial Aerodynamics, vol 84 , n.3 , March **2000**
Simiu, E. and **Scanlan,** R.H., "Wind effects on structures: Fundamentals and applications to design ", John Wiley & Sons, New York, 1978.
Vellozzi, J. and **Cohen**, E., "Gust response factors", Journal of Structures Division, ASCE, Vol.94, June 1968.
Venkateswarlu, B., Arunachalam, S., Shanmugasundaram, J. and Annamalai, G., "Variation of wind speed with terrain roughness and height", Journal of Institution of Engineers (I), Vol.69, January 1989, pp 228-234.
Venkateswarlu, B., Harikrishna, P., Selvi Rajan, S. and Sathis Ram Kumar, M., "Stochastic gust response of microwave lattice towers", Computers & Structures, Vol.52, **1994**, pp 1031-1041.
Vickery, B.J., "Along-wind loads and response", Chapter 5, Wind Engineering Course, Department of Mechanical Engineering, Monash University, Melbourne, Australia, **1992**.
Vickery, B.J., "On the reliability of gust loading factors", Proceedings of Technical Meeting Concerning Wind Loads on Buildings and Structures, BSS 30, National Bureau of Standards, Washington, D.C., pp 93-104.**1970**
Vickery B.J. and **P.J. Pike** (**1985**) An investigation of dynamic wind loads on offshore platforms, OTC paper 4955, OTC, 527 -542
Vickery B.J., (**1989**) Application of Wind engineering - principles to design of structures – short course notes by Davenport, A.G., and Vickery B.J., Hertig, J.A., Lansanne Switzerland, Feb 23-27, 1987.(repeated in course notes, Italy, 1989)-course notes made available during his visit as UNDP consultant at SERC

Web Resources : www.ttu.edu ; www.nd.edu ; www.sercm.org