A STUDY OF THE STATISTICAL APPROACH TO WIND LOADING.

A thesis submitted in partial fulfilment of the requirements for the degree of Master of Science in Engineering by :-

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TABLE OF SYMBOLS

a, b, c	-	Empirical Co-efficients in the coherence function
A	-	Surface area of structure
B_i	-	Breadth at x_i
$\check{C}(\tau)$	-	Auto-covariance for time lag, τ
$\check{C}ij(\tau)$	-	Cross covariance between x_i and $x'j$ for time lag, τ
C_D	-	Drag Co-efficient
C	-	Inertial Mass Co-efficient
D(t)	-	Fluctuating drag on a body
$g(\nu T)$	-	A probabalistic multiplier for the most likely extreme value.
H	-	Height
$J_j a^2$	-	The part of modal admittance expressing the variation of mean speed with height.
$J_{j\rho}^2(n)$	-	The part of modal admittance expressing gust correlation effects.
Kj	-	Generalised Stiffness for mode j
ℓ	-	Representative body dimension
Mj	-	Generalised mass for mode j
m(x)	-	Mass per unit length at x
n	-	Frequency
nj	-	Natural frequency for j th mode.
$\check{P}ij(n)$	-	Real part of cross-spectral density function or co-coherence
Pt(V), Pt(V)	-	Probability of occurence for variable (V)
q(x,t)	-	Displacement of point x on a line structure at time, t
$\check{Q}ij(n)$	-	Complex conjugate of cross-spectral density or quad-coherence
Qj	-	Generalised displacement for mode j
$\check{R}ij(n)$	-	Normalised cross spectral density or coherence at frequency n
Rt(V), rt(V)	-	Return period for variable (V)
rij	-	Separation distance between points xj and x^1j
$Sij^V(n)$	-	Velocity spectral density at frequency n
$S^P(n)$	-	Force spectral density at frequency n
$Sj^Q(n)$	-	Displacement response spectral density for mode j at frequency n
Vi	-	Wind velocity vector
Vg	-	Gradient Wind Speed
$\bar{V}(10)$	-	Mean wind speed at 10 metres
$\hat{V}$	-	Maximum gust wind speed

Z_g	-	Gradient Height
α	-	Power index for variation of Wind Speed with height
β_j	-	Influence functions for mode j
ξ	-	Reduced frequency, $2\pi n l/\bar{V}$
κ	-	Surface roughness co-efficient
$\mu_j(x)$	-	Normalised modal ordinate for mode j
σ	-	Standard Deviation
ν	-	A statistical factor
$\rho^v(\tau)$	-	Auto-correlation for time lag, τ
ρ	-	Density of air = 1,266 kg/m^3
τ	-	time lag
$\chi^2_{a_j}(n)$	-	Aerodynamic admittance for frequency n and mode j
$\chi^2_{m_j}$	-	Mechanical admittance for mode j
δ	-	logarithmic decrement
ω	-	circular frequency, $2\pi n$ for mode j
γ	-	Non-dimensional parameter expressing the variation of velocity, mode and breadth over the structure.

SYNOPSIS

This study is concerned with the prediction of the maximum probable response of structures in the along-wind direction. An outline of the theoretical basis of the approach is given and the effect of varying wind and structural parameters for a range of different structures is examined.

To put the study in context, a brief look is taken at the problems of formulating a more consistent approach to the overall problem of wind loading.

CHAPTER 1

INTRODUCTION.

1.1. The Need for A Study.

The need for this study arose out of the writer's involvement in the design of guyed masts and microwave towers. In particular, it was required to show that certain dished microwave reflectors would not exceed a specified displacement.

The first thing that became apparent in this instance was that such a specification was asking the impossible. The best that could be achieved was to show that this displacement had a certain likelihood of being exceeded in the design life of the structure.

Secondly, although it was clear that a non-deterministic approach was the right one - it seemed, at the time, that the subject was so clothed in unfamiliar terminology and assumed so many arbitrary parameters that it was difficult to have much confidence in the result.

In the event, the exigencies of the design precluded further pursuit of the subject and the problem was solved using static loading. However, subsequent design work on other wind-sensitive structures convinced the writer of the need for a non-deterministic approach that could be easily understood and applied to design problems.

1.2. Scope.

This study is, therefore, concerned with determining the maximum probable response of structures in high wind conditions in a direction coincident with the wind. This along-wind response ignores effects such as vortex shedding, galloping, flutter and other forms of aerodynamic instability which produce responses primarily out of the plane of the wind.

1.3. Current Treatment of Wind Loading.

The current treatment of wind loading on structures based on British Standard Code of Practice No. 3 Chapter 5 (2) involves the selection of a basic wind speed from isopleth maps giving the maximum gust speed, at a height of ten metres above the ground, likely to be exceeded not more than once in fifty years. This is, then, modified to take into account local topographic influences, surface roughness of the environment, gust duration appropriate to building size, height and design life of the structure.

With this modified velocity, the wind pressure and, hence, the total wind load acting on the structure can be calculated. This method has two main limitations. Firstly, although allowing a lower gust speed to be used for buildings exceeding 50 metres by increasing the averaging time to fifteen seconds rather than five seconds, no attempt is made to quantify the extent to which the size effects the overall wind load. Secondly, no account is taken of the dynamic effects of gustiness on the structure or the likelihood of a sequence of gusts striking the structure and the consequences.

For a flexible structure, these effects may be very significant indeed.

1.4. The Statistical Approach.

The statistical approach, on the other hand, describes the wind speed in terms of mean and fluctuating parts and treats the fluctuations as a random stochastic process.

Then using the already well developed techniques for the analysis of structures subjected to random loading, the maximum probable response can be predicted taking into account both dynamic effects and the spatial properties of the gusts related to the size of structure.

This method borrows extensively from theory of random signals in communications and control engineering and uses terms and concepts largely unfamiliar in civil engineering.

The development of the method owes much to the early work of Davenport (3) in 1961 who showed that the fluctuating part of wind velocity and the resulting pressures on a structure in turbulent flow can be treated as a stationary stochastic process. The difficulty, however, has been the satisfactory derivation of the many empirical relationships required and developing of a consistent theory to describe the nature of atmospheric turbulence.

Once the maximum probable response has been found it is convenient to express it as a ratio of the mean response and such a ratio is known as a gust factor. Gust factors were first proposed by Sherlock (13) and later developed by Davenport (6), Vellozzi and Cohen (12) and others.

1.5. The Purpose of the Study.

It is the intention of this study :

(a) to summarize the theoretical basis of the statistical approach and outline some of the recent developments in the field. Chapter two concerns the description of the mean wind and chapters three to five, the effects of the fluctuations in the wind speed.

(b) to compare the various published relationships for the essential empirical function - the coherence, and the effect on the gust factor.

(c) to examine the effect of varying the design parameters over a range of possible solutions to test the sensitivity of the gust factor to such parameters as the structure size, damping, wind speed profile and surface roughness of the environment.

Finally, it is hoped to demonstrate that, despite the admittedly uncertain nature of many of the assumptions, the statistical approach offers a more rational treatment of the wind loading problem in general and the design of tall buildings and wind-sensitive structures in particular. Moreover, since it allows wind loading to be assessed on a probabalistic basis the method fits in with the current limit-state philosophy.

5./......

CHAPTER 2

THE MEAN WIND.

2.1. THE NATURE OF WIND :

Wind is a natural phenomenon which is caused by differences in atmospheric pressure resulting from variations in the solar heat absorption over the earth's surface. Additional forces are produced also by the effects of the curvature and rotation of the earth. In meteorology, lines of equal barometric pressure which cause winds in the same direction are known as isobars which are given on the familiar weather maps indicating the location and distribution of complete pressure systems.

At ground level, the atmospheric wind is retarded by surface friction, causing the formation of a turbulent boundary layer. The height at which the wind remains unaffected by this turbulence is known as the gradient height $\bar{Z}g$ and the corresponding wind velocity - the gradient velocity $\bar{V}g$. The gradient height normally varies between 300 and 600 m above the ground.

If a mass of air which is in thermal equilibrium with its surroundings rises, it will undergo an adiabatic expansion and a consequent fall in temperature. If the temperature falls at the same rate as that of its surroundings, it is said to be in a state of neutral stability with an adiabatic temperature lapse rate.

This state of neutral stability is a necessary condition for the turbulent boundary layer to consist only of mechanical stirring effects and not convection currents due to heat exchange between the air masses.

6./......

The assumption is usually made that, in high winds (the major concern in structural loading problems), the effects of these convection currents are negligible and that atmospheric conditions are stable.

Within the earth's boundary layer or the planetary boundary layer, the surface roughness gives rise to turbulent fluctuations in the wind velocity.

The scale of this turbulence varies over wide limits ranging from major eddies several thousand metres in extent and lasting several minutes to the eddy caused by some local obstruction which may only last a fraction of a second. These small eddies are usually surperimposed on the larger ones with the result that wind speeds vary greatly from place to place and from moment to moment.

A physical model of a high wind is therefore one of a mean flow which results from atmospheric pressures, on which are superimposed fluctuations generated in the turbulent boundary layer. Examination of these fluctuations shows them to be entirely random in nature.

The assumption of the randomness of the variations of wind speed is fundamental to the statistical approach. It enables the wind to be characterised by the statistical properties that constitute a stochastic process. However,two important simplifications are made :

(a) The process is stationary, that is, the average properties do not vary with time.

(b) The process may be described by a probability distribution making it possible to discuss the wind speed in terms of probability of occurrence.

2.2. MEAN HOURLY WIND SPEED :

The concept of a mean wind speed introduces the problem of a suitable time period for averaging the wind speed values. Because the fluctuations in the wind speed vary over such a wide scale, the result of measurements to determine the mean will depend on the duration over which the sample is taken. A large duration will allow the effects of the large eddy and even changes in the complete weather system, while a short duration may only include the small scale eddies. The choice of such an "averaging" time, therefore, depends on several factors :

(a) The mean speed should be reasonably constant for the assumption of a stationary process to remain valid. In practice, this means that the period must be short enough to be unaffected by atmospheric pressure variations and changes in the weather regime and long enough not to be influenced by gust variations in the averages. An analysis undertaken by Van der Hoven (14) shows that the intensity of wind variations with time has two distinct peaks, one at about four days corresponding to the passage of complete isobar system and the other at about one minute - the variations due to turbulence. Further, the variations are least between about 15 minutes and two hours and this period is known as the spectral gap because the effects associated with each of the peaks are neatly seperated by this period of low variation intensity.

(b) The period should be short enough to contain the peak effects in a storm of an isolated train of gusts. Davenport (3) states that effects such as thunder storms and squalls usually last for 5 to 10 minutes.

(c) It should be long enough for the structure to attain a steady-state of response. Fundamental frequencies of most structures are usually less than ten seconds and they have ample time to reach a steady state in 5 to 10 minutes. (A side effect of this is that transient response need not be considered in assessing the overall structural response.)

(d) The necessary data must be available and it must be such that the assumptions of randomness remain valid.

Although Davenport (3) has suggested an averaging time of between 10 and 15 minutes as being ideal, most meteorological stations have wind speed data averaged over one hour. It can be seen that all the above factors apply to an hourly averaging period and, particularly, that it falls within Van der Hoven's spectral gap. The result is that choosing an averaging time of one hour leads to statistically stable averages.

Assuming conditions of neutral stability, this mean hourly wind speed has been found to follow a simple power law variation with height. (In fact, meteorologists (7) use a more complicated logarithmic law derived from the equations of motion of a fluid and the effects of momentum in a turbulent boundary layer but, for structural purposes, the simplified law would seem to be justified (7a)). Thus :

$$\frac{\bar{V}(z_1)}{\bar{V}(z_2)} = \left(\frac{z_1}{z_2}\right)^{\alpha} \qquad - (2.1)$$

where $z1$ and $z2$ are different heights above the ground and $\bar{V}(z_1)$ & $\bar{V}(z_2)$ are the respective mean wind velocities.

Relating this to the gradient height or a standard height of 10 m above the ground we have :

$$\frac{\bar{V}(z)}{\bar{V}(z_g)} = \left(z/z_g\right)^{\alpha}$$

$$\text{or} \quad \frac{\bar{V}(z)}{\bar{V}(10)} = \left(z/10\right)^{\alpha} \qquad \text{—— (2.2)}$$

where $\bar{V}(10)$ in the mean hourly wind velocity at a reference height of ten metres.

Values for the power law index, α, can be determined from measurements of the mean wind speed profile variation with height, and at the same time, a surface drag co-efficient, K, can be defined as a dimensionless measure of the roughness of the terrain relating the mean wind speed to the shear stress near the ground.

Values for these parameters given by Davenport (6) are listed in Table I. Based of such data, Davenport gives the following values :

TABLE I

	Power Law Index,	Gradient Ht Z(g)m	Surface Drag Co-efficient, K
Open Grassland	0.16	300	0.003-0.005
Woodland, Suburbs	0.28	430	0.015-0.030
Urban Centres, Broken Country	0.40	560	0.030-0.050

10./......

The figures given for the power law index in open grassland and woodland or suburbs are well substantiated by a wide range of experimental evidence but, for urban centres, the power law variation is less well-defined. It is not certain whether it is more satisfactory to assume a higher value for the power law index as Davenport (6) recommends or whether to use a lower value and assume it starts from a level above the average obstruction height of the buildings.

It is doubtful whether either the power law index and the mean wind is very meaningful in these circumstances as the wind speed is very much determined by such factors as the nature of the buildings in the surroundings, the orientation of the streets and other obstructions.

The power law variation with height can, therefore, only be regarded as a satisfactory approximation if :

(a) There is reasonably level terrain with uniform roughness.

(b) There is a steady flow condition (high wind) at the gradient height with neutral stability.

(c) The index, α, is varied to take into account the surface roughness.

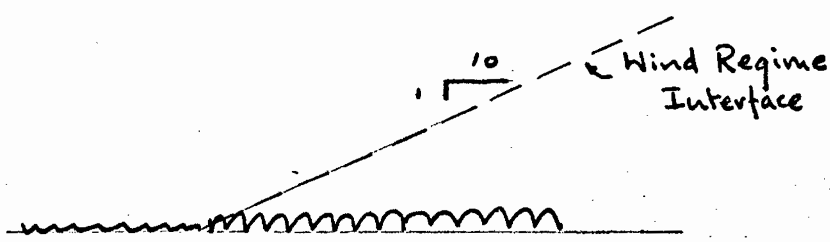
Thus the power law is unsuitable in the presence of hills and for the effects caused by funnelling down valleys or other topographical features. On the tops of hills, often the sites of tall towers, the effect of streamlining can cause an increase in wind speed immediately above the ground which may be followed by a decrease of velocity with increasing height and ultimately a reversion to the standard profile.

11./......

C.P. 3, Chap. V (1970) (2) gives a table of factors which modify the wind speed to take into account topographical influences and a method based on the French Code of Practice (16) to define the effective height of a building close to the tops of hills or escarpments.

Another problem is the satisfactory description of the wind profile when there is a sudden change in surface roughness such as may occur between open country and urban centres. It is generally recognised that an interface exists between the two turbulent regimes. Davenport (15) suggests that this interface slopes at approximately 1:100 but more recent research (7a) suggests that 1:10 might be more appropriate.

Under these circumstances, the designer should ascertain the height at which the regimes change in relation to the height of the structure (see Fig. 1).



The Effect of a change in Terrain Roughness

FIG. 1

The direction of the mean hourly wind does not remain constant over the gradient height. The effect of the rotation of the earth causes a direction change which may vary from 30^{o} to 45^{o} depending on the terrain roughness. This change of wind direction with height is known as the Ekman Spiral but, except for very large structures, its effect is usually ignored.

To summarise, it is convenient to average the wind speed data from a continuous record over one hour and such an average is known as the mean hourly wind speed. This mean speed is assumed to vary up to the gradient height according to a simple power law which depends on the roughness of the terrain over which the wind passes. This assumption is inadequate in dense urban areas, where the roughness changes suddenly or when topographical features alter the wind speed profile. In these cases, the designer should be aware of the problems and make suitable allowances.

There is a tendency to regard the mean hourly wind speed as a well defined phenomenon and it is hoped this brief description has served to underline its basically unpredictable nature.

2.3. The Analysis of Mean Wind Speed.

Mean hourly wind speed data have been recorded in this country for periods of between fourteen and twenty-one years and one way of finding a wind speed to use as a basis for design is to simply take the highest appropriate value recorded.

This approach, however, gives no indication of risk of the actual wind speed exceeding this value and in order to treat the wind speed in terms of a given probability of occurence the techniques of extreme value statistics are used.

If $P_t(V)$ is the probability of the actual wind speed not exceeding V then a return period is defined such that

$$R_t(V) = \frac{1}{1 - P_t(V)} \qquad (2.3)$$

It is generally accepted (9) that the statistical population of annual mean hourly maxima conforms to the Fisher-Tippett Type 1 Distribution.

$$P_t(v) = \mathrm{Exp}(-\mathrm{Exp}(-a(V-u))) \quad — (2.4)$$

where a is a measure of the dispersion of the extreme and u is the mode of the distribution.

For each site, the highest recorded mean hourly wind speed is tabulated for each year a record is available. This data is then ranked such that each value, m, is arranged in ascending order (m = 1 for the smallest) up to the total of N values. Plotting positions, pt are then found such that :

$$pt = \frac{m}{N+1}$$

This value corresponds to the probability of obtaining the associated highest mean hourly wind speed in any one year with a return period :

$$rt = \frac{1}{1-pt}$$

A straight-line relationship is then obtained by plotting each value for the mean hourly wind speed against the reduced variate y such that :

$$y = -\log_e(-\log_e pt)$$

From the plot, the mean hourly wind speed with a given probability of occurence within the required return period can be derived. Such a plot is given for the D.F. Malan Weather Station, Cape Town in Fig. (2). It is usual to draw up isotach maps for the mean hourly wind speeds likely to be exceeded once in 50 years which can be used to find the design values for a particular area.

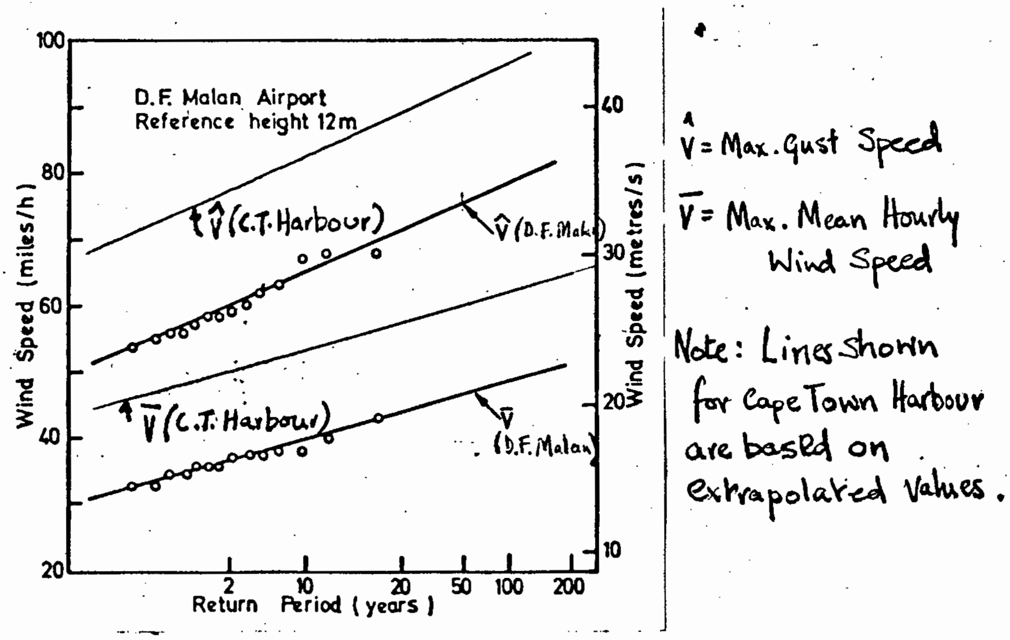

D.F. Malan Airport
Reference height 12m
100
80
60
40
20
Wind Speed (miles/h)
40
30
20
10
Wind Speed (metres/s)
V̂ (C.T. Harbour)
V̂ (D.F. Malan)
V̄ (C.T. Harbour)
V̄ (D.F. Malan)
2
10
20
50
100
200
Return Period (years)
V̂ = Max. Gust Speed
V̄ = Max. Mean Hourly Wind Speed
Note: Lines shown for Cape Town Harbour are based on extrapolated values.

Fig. 2

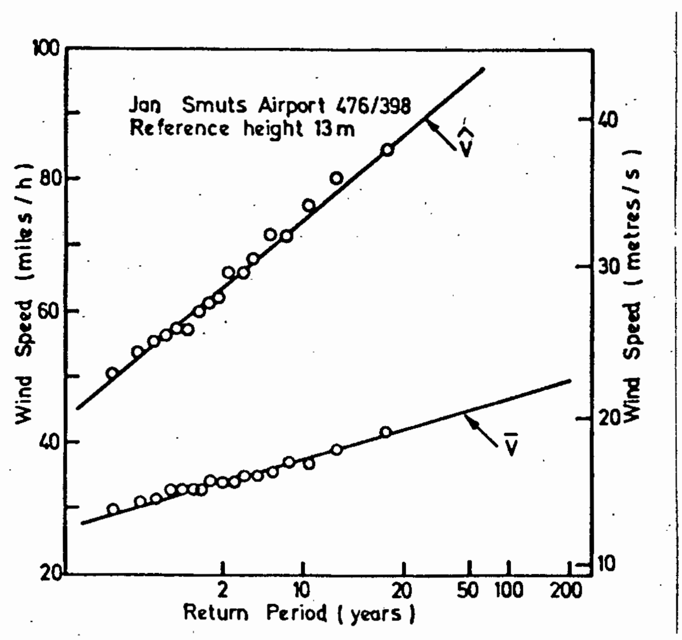

Jan Smuts Airport 476/398
Reference height 13m
100
80
60
40
20
Wind Speed (miles/h)
40
30
20
10
Wind Speed (metres/s)
V̂
V̄
2
10
20
50
100
200
Return Period (years)

Fig. 3

CHAPTER 3

THE ANALYSIS OF GUSTS.

3.1. THE WIND VELOCITY VECTOR AND THE VARIANCE OF THE FLUCTUATIONS

As has already been stated, it is convenient in the statistical approach to separate the properties of the wind into mean and fluctuating components.

Thus the wind velocity, Vi, can be defined in terms of a scalar mean speed and a fluctuating gust vector :

$V = \bar{V} + vi$ where $\bar{V}$ is the mean hourly speed and vi is the gust vector as defined in figure 4.

For most applications, only the along wind component of Vi is used and it is, in fact, only comparatively recently that instruments have been developed which are capable of measuring the other two components.

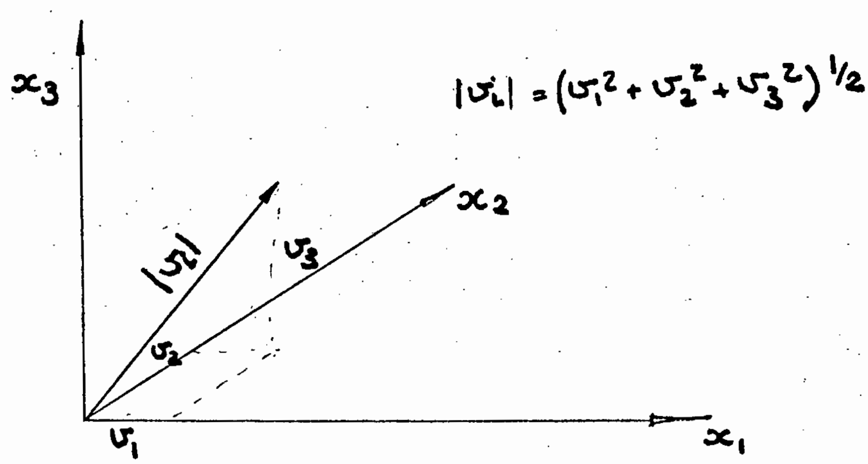


FIGURE 4.

It should be noted that the mean value of vi is, by definition, equal to zero.

Thus :

$$\frac{1}{T}\int_0^T v_i \, .dt = 0 \text{ where } T = 1 \text{ hour}$$

and the variance is given by $\sigma^2(v_i) = \frac{1}{T}\int_0^T v_i^2 . dt$ - (3.1)

Hence $\sigma^2(v_i) = \frac{1}{T}\int_0^T (v_1^2 + v_2^2 + v_3^2)\, dt$

$$= \sigma^2(v_1) + \sigma^2(v_2) + \sigma^2(v_3) \quad - (3.2)$$

At ground level : $\sigma(v_1) \sim 3\,\sigma(v_2)$ and $\sigma(v_2) > \sigma(v_3)$

so that it is usual to approximate $\sigma(v_i)$ by $\sigma(v_1)$.

Experimental results show that the standard deviation of the gust variations decreases slowly with height and, to fit into the concept of a planetary boundary layer, it should tend to zero at the gradient height Zg. However, for structural design purposes, the assumption is usually made (7) that the standard deviation is invariant with height so that :

$$\sigma(v_i) \cong \sigma(v_1) = 2{\cdot}58\, K^{1/2}\, \overline{V}(10) \quad - (3.3)$$

thus

$$\sigma(v_i) \cong 2{\cdot}58\, K^{1/2} \left(10/z_g\right)^{\alpha} \overline{V}g$$

and the intensity of turbulence :

$$\sigma(v_i)/\overline{V}(x_3) = 2{\cdot}58\, K^{1/2} \left(10/x_3\right)^{\alpha} \quad - (3.4)$$

expressed in a consistent convention (see fig. 4).

If Davenport's values for the surface drag co-efficient and gradient height are assumed, $\sigma(v_i)$ and $\sigma(v_i)/\overline{V}(10)$ are given in Table 2.

TABLE 2.

	$\sigma(v_i)$	$\dfrac{\sigma(v_i)}{\bar{V}(10)}$
Open Country	0.106 $\bar{V}g$	0.18
Woodland	0.110 $\bar{V}g$	0.32
Urban centres	0.115 $\bar{V}g$	0.58

It can be seen that $\sigma(v_i)$ is almost invariant not only with height but also with the surface roughness co-efficient, K, and a reasonable value is :

$$\sigma(v_i) \doteqdot 0.11\,\bar{V}(G)_? \doteqdot 0.19\,\bar{V}(10)$$ (see Harris, Ref. 7)

It is interesting to express the maximum short duration gust speed which is usually used in the calculation of wind pressures on structures in terms of the mean value and the variance.

Harris (7) gives the ratio of the 2-second gust speed to the mean speed at ten metres height as 1.6.

Since $\dfrac{\sigma(v_i)}{\bar{V}(10)}$ = 0.18 in open country, it follows

that : $V_{2s}(10) = \bar{V}(10) + 3.3\,\sigma(v_i)$ - (3.5)

inconsistent notation $\bar{V}(10)$ not $\bar{V}_{10}$

and for the variation of the 2-second gust speed with height :

$$V_{2s}(x_3) = \left(\frac{x_3}{10}\right)^{\alpha} \bar{V}_{10} + 0.6\,\bar{V}(10) \quad - (3.5a)$$

Figure 5 gives the ~~cumulative~~ normal probability density function ~~distribution~~ with a mean hourly wind speed of 20 m/s and an intensity of turbulence corresponding to the open country site.

Hence : $\sigma(v_L) = 0.18 \times 20 = 3.6$ m/s

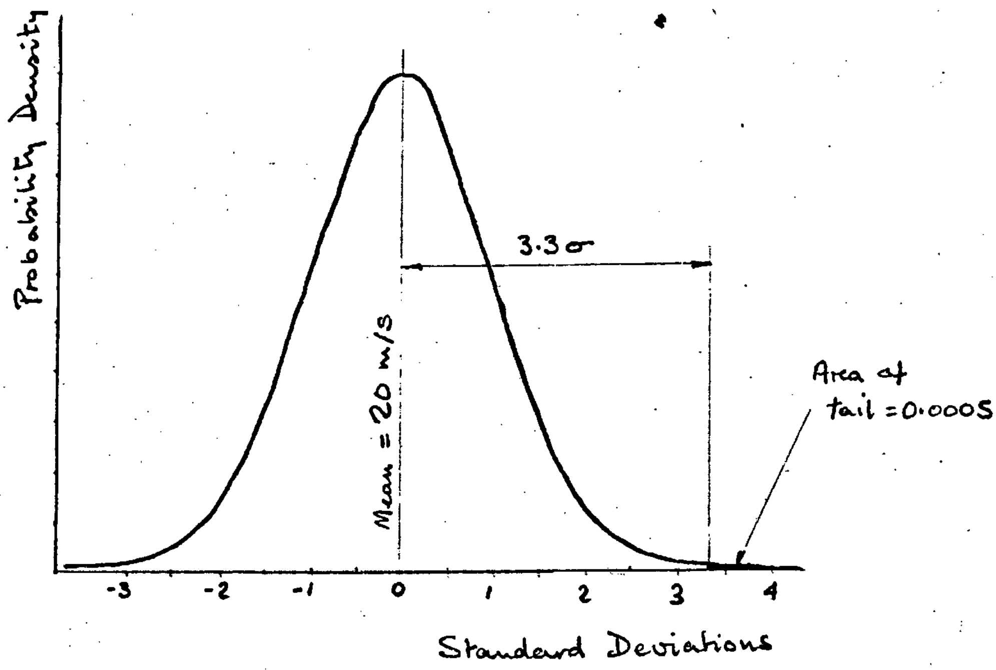

Probability Density
3·3σ
Mean = 20 m/s
Area of tail = 0·0005
-3
-2
-1
0
1
2
3
4
Standard Deviations

Fig. 5

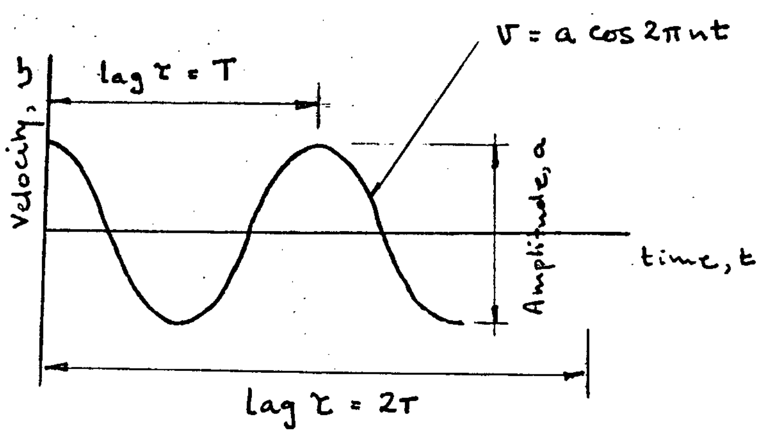

v = a cos 2πnt
lag τ = T
Velocity, v
Amplitude, a
time, t
lag τ = 2T

Fig. 6

It can be seen that the probability of the 2-second gust speed being exceeded is 0·0005 or one in 2000 .

3.2. CORRELATIONS AND GUST SPECTRA:

It is not, however, sufficient for structural purposes to describe the effects of gusts solely in terms of the variances of the fluctuations. For a large structure, their effect may not cover the whole surface and, in some instances, may be extremely localised. Clearly, a gust will have definite dimensions and variations both in time and space must be described. This is done by using the statistical theory of correlations and spectral analysis.

Consider a point in space defined by the Cartesian co-ordinates, x_i. Then, if pairs of wind velocity measurements $v_i(t)$ and $v_i(t + \tau)$ are taken with time lag, τ, at this point, the auto-covariance function is defined as

$$C^v(\tau) = \overline{v_i(t)\ v_i(t+\tau)} \qquad -(3.6)$$

where the bar denotes a time average over time t.

If the time lag, τ, is reduced to zero, then

$$C^v(0) = \overline{v_i^2(t)} = \sigma^2(v_i) \qquad -(3.7)$$

The normalized auto-covariance function is called the auto-correlation $\rho^v(\tau)$ and is defined by :-

$$\rho^v(\tau) = C^v(\tau)/C^v(0) = C^v(\tau)/\sigma^2(v_i) \qquad -(3.8)$$

$\rho^v(\tau)$ is dimensionless and $\rho^v(0) = 1$. As τ tends to infinity, $\rho^v(\tau)$ becomes asymtotic to zero.

Note also that the function is symetrical about $\tau = 0$

i.e. - (3.9)

$$\rho^v(-\tau) = \rho^v(\tau) \quad \text{and} \quad C^v(-\tau) = C^v(\tau)$$

The auto-correlation function is a measure of relation of two gust velocities seperated by a time lag, τ. As τ becomes longer, the relation becomes less and a time scale, T, is defined such that :

$$T = \int_0^{\infty} \rho^v(\tau)\, d\tau$$

- (3.10)

T is sometimes called the 'memory' time since if $\tau < T$, the gusts are said to be well correlated and if $\tau > T$, badly correlated.

The power spectrum of wind velocity is used to describe the frequency range and energy content of the gusts occurring in the averaging time. The gust signal is not periodic; it consists of a continuous range of frequencies which can be described as being made up of simple harmonic contributions using the Fourier Series. If one of these contributions has the cosine form $v = a\cos 2\pi nt$ and its period T_1 is $1/n$, (See Fig. 6) then the covariance function for this contribution is

$$C^v(\tau) = \overline{v(t)\, v(t+\tau)}$$

and if τ is taken as T_1:

$$C^v(T_1) = \overline{a^2 \cos^2 2\pi n T_1} = a^2/2$$

Where a is the amplitude and n is the frequency. Now the variance of the cosine form is also $a^2/2$ and the variance of the whole range of frequencies must be equal to the sum of the variances of the contributions :

$$\sigma^2(v) = \sum_{i=1}^{N} \frac{a_i^2}{2} \qquad - (3.11)$$

The power spectrum is, then, defined as the total contribution to the variance of all harmonic components so that

$$\sigma^2(v) = \int_0^\infty S^v(n)\,dn \qquad - (3.12)$$

and $S^v(n)$ is the spectrum of gust velocity at frequency n.

$C^v(n)$ and $S^v(n)$ are Fourier Transform pairs:

$$S^v(n) = 4\int_0^\infty C^v(\tau)\cos 2\pi n\tau\,d\tau \qquad - (3.13)$$

$$C^v(\tau) = \int_0^\infty S^v(n)\cos 2\pi n\tau\,dn \qquad - (3.14)$$

It is necessary now to determine the spectrum in terms which can be used for design purposes. This has been done by various authors who have suggested an empirical relation on the basis of experimental results.

Harris (7) suggests the following relationship :

$$n\,S^v(n) = 4\,k\,\bar{V}_{10}^2\;\tilde{n}\Big/(2+\tilde{n}^2)^{5/6} \qquad - (3.15)$$

where $\tilde{n} = \frac{nL}{\bar{V}_{10}}$ and L is a length constant taken as 1800 m.

Thus it should be noted that the gust spectrum is defined in terms of surface roughness and the mean hourly wind speed is 10 m/S

Using this equation and the Fourier Transform relationship, it can be shown that the auto-correlation function (7) can be calculated by :

$$\rho^{v}(\tau) = \frac{2}{\Gamma(1/3)} \left(\frac{\tilde{\tau}}{2}\right)^{1/3} K_{1/3}(\tilde{\tau}) \qquad - (3.16)$$

where $\tilde{\tau} = 2\sqrt{2}\,\pi\,\bar{V}_{10}\,\tau/L$, $\Gamma(1/3) = 2.679$ and $K_{1/3}(\tilde{\tau})$

is a modified Bessel function of order 1/3.

The time scale, T, in the same way is given by :

$$T = \sqrt{2}\,\Gamma(5/6)/\sqrt{\pi}\,\Gamma(1/3)\,L/\bar{V}_{10} = 0.084\,L/\bar{V}_{10} \qquad - (3.17)$$

where $\Gamma(5/6) = 1.129$

3.3. SPATIAL CORRELATIONS :

The terms auto-covariance and auto-correlation refer to the time averaged properties of gusts at a single point (hence the prefix, auto-) but, in structural problems, it is also necessary to describe the spatial as well as the time averaged properties of gusts.

To do this rigorously requires a knowledge of the relationship of each three velocity components at a point x_i, to the corresponding components at another point x'_j It is thus necessary to use tensor notation for the systematic treatment of these functions. However, although the derivation of some of these functions could be given in general terms, this study is primarily concerned with the along wind component. Further, the knowledge existing of the relationships between all the other components is very slight and it is usual in terms of the empirical relationships derived from experiments to deal only with this single component.

Consider two points xi and x'_j , then the cross-covariance function could be written as either :

$$C^v_{ij}(x_i, x'_j; \tau) = \overline{v_i(x_i; t)\, v_j(x'_j; t+\tau)}$$

or $$C^v_{ij}(\tau) = \overline{v_i(t)\, v_j(t+\tau)}$$

-(3.18)

and $$C^v_{ii}(0) = \sigma^2(v_i)$$

The normalised cross-covariance or cross-correlation is :

$$\rho^v_{ij}(\tau) = C^v_{ij}(\tau) / \sqrt{C^v_{ii}(0) . C^v_{jj}(0)} \qquad -(3.19)$$

It can be seen that $\rho^v_{ij}(\tau)$ is dimensionless, lies between -1 and +1 and is a measure of the relation of the gust velocities at the two points lagging by a time, τ . It is not, however, symmetrical i.e.

$$C^v_{ij}(\tau) \neq C^v_{ij}(-\tau)$$

Dividing $C^v_{ij}(\tau)$ into symmetrical and anti-symmetrical parts :

$$C^v_{ij}(\tau) = \tfrac{1}{2}(C^v_{ij} + C^v_{ji}) + \tfrac{1}{2}(C^v_{ij} - C^v_{ji}) \qquad -(3.20)$$

This leads directly to the cross spectrum :

$$S^{u}_{ij}(n) = P^{u}_{ij}(n) - iQ^{u}_{ij}(n) \qquad -(3.21)$$

where $P^{u}_{ij}(n)$ and $Q^{u}_{ij}(n)$ are defined by the symmetrical and anti-symmetrical parts of the cross-covariance. The complex quantity is introduced because the cross-covariance is not a symmetrical function of τ.

Thus $$P^{u}_{ij}(n) = 2\int_0^\infty (C^{u}_{ij} + C^{u}_{ji}) \cos 2\pi n\tau \, d\tau \qquad -(3.22)$$

and $$Q^{u}_{ij}(n) = 2\int_0^\infty (C^{u}_{ij} - C^{u}_{ji}) \cos 2\pi n\tau \, d\tau \qquad -(3.23)$$

Unlike the power spectrum, the cross- spectrum has no easy physical interpretation and, therefore, is a much more difficult concept to understand. It leads to a very important quantity in spectral analysis : the coherence, $R^{u}_{ij}(n)$ which is defined as the normalised cross-spectrum.

Thus $$R^{u}_{ij}(n) = S^{u}_{ij}(n) / \sqrt{S^{u}_{ii}(n) S^{u}_{jj}(n)} \qquad -(3.24)$$

Like the cross-spectrum, it is also a complex quantity and is used to describe the spectral loading on a structure subjected to a random load. It can be interpreted physically as the filtered (of frequency n) cross-correlation between xi and x'_j for zero lag.

At the present time, most of the relationships for the cross-spectrum and coherence are derived on an empirical basis from wind tunnel tests and observation of the behaviour of full scale structures on a great variety of different sites. Harris (7) has given a consistent theoretical approach based on two assumptions about the nature of turbulence in the planetary boundary layer. These are :-

(a) Taylor's Hypothesis :

Based on wind tunnel experiments on the turbulence generated by a grid of bars, Taylor suggested that the cross-covariance between any components of turbulence at two points separated by a distance S1 in the downstream direction is equal to the cross-covariance measured at one point for a lag τ given by $\tau = S1/\bar{V}$ where $\bar{V}$ is the mean wind speed in the wind tunnel.

i.e.

$$C^{v}_{ij}(x_1, x_2, x_3, x_1+S1, x_2', x_3'; \tau) = C^{v}(x_2, x_2'; \tau+(S1/\bar{V}(x_3))) \quad \text{— (3.25)}$$

The implication is that the turbulence does not change substantially over a short distance in the downstream direction.

(b) Homogeneous Isotropic Turbulence.

Homogeneous isotropic turbulence (H.I.T.) is a theoretical concept representing the turbulence developed in an infinite region with no boundaries and a uniform rate of generation of turbulent energy per unit mass. It follows that the variances in all three component directions must be equal.

i.e. $\sigma^2(v_1) = \sigma^2(v_2) = \sigma^2(v_3)$

Clearly, at ground level this assumption is wrong since the turbulence is highly anisotropic but it becomes progressively better with increasing height.

This approach leads to highly complex mathematical forms involving Modified Bessel functions of fractional order. Moreover, it is doubtful whether the H.I.T. assumption is not too drastic to be universally applicable. For these reasons, the theoretical derivation will not be given and the co-herence and cross-correlation functions used in this study will all be of an empirical nature.

3.4 Coherence Functions.

Expressions derived by various authors (4,7,17) for the coherence function and the phase angle (introduced because of the complex nature of the function) are given in Table 3. Almost all these expressions apply only to horizontal or vertical lattice structures. This is because, for these structures, the magnitude of the turbulent fluctuations caused by the structure is small compared with its size and, therefore, experimental results for a free wind stream apply. However, both Davenport and Vickery have suggested that their results may be used for solid structures. Of Shiotani's results, the case of anisotropic homogeneous turbulence is the more general, more applicable close to the ground and will be used in this study.

But it should be noted that Shiotani's results were measured in areas subjected to violent tropical storms and are applicable to typhoon conditions. In these conditions, it was found that there is little difference between the lateral and vertical scales of turbulence some distance from the ground and probably, the most accurate model is one of anistropic turbulence at low heights with the isotropic condition developing with increasing height. However, it is possible that Davenport's results may be more applicable to South African conditions and it should be emphasized that all these empirical results apply primarily to extreme conditions in the Northern Hemisphere and the applicability to the more temporate regions of the Southern Hemisphere has not yet been verified.

Thus, it is proposed to compare a modified Davenport coherence function for bluff structures with those given in Table 3.

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TABLE 3

PUBLISHED COHERENCE FUNCTIONS.

Author	Applicability	Coherence	Phase Angle Ø
Davenport	Lattice	$R^v_{ij}(n) = e^{-cn r_{ij}/\bar{V}}$ where C = 16 for horizontal structures and C = 8 for vertical structures	NIL
Vickery	Lattice Vertical	$R^v_{ij}(n) = e^{-1.5 n(\theta/2\pi)}$ where $\theta/2\pi = \frac{r_{ij}}{2\pi L_1}\left[1 + \left(\frac{2\pi n L_1}{\bar{V}}\right)^2\right]^{1/2}$ and L_1 is the along-wind integral scale	$\cos 1.4\pi(\theta/2\pi)$
Shiotani	Bluff Solid Structures : Isotropic and Anisotropic turbulence. Horizontal and vertical.	$R^v_{ij}(n) = e^{-\left[ab \frac{2\pi n r_{ij}}{\bar{V}} / (b^2\cos^2\theta_{ij} + a^2\sin^2\theta_{ij})^{1/2}\right]}$ where θ_{ij} is the angle between ij and the horizontal For Isotropic Homogeneous Turbulence : a = b = 1.27 and c = 0 For Anisotropic Homogeneous turbulence : a = 3.82 b = 1.27 c = 0.8	$\cos\left(c\,\frac{2\pi n r_{ij}}{\bar{V}}\right)$

This modified function uses the fact that the constant C for horizontal separations is twice that for vertical seperations and, hence, the proposed function is :

$$R^{v}_{ij} = e^{\left(-8n\,\frac{((2x_1)^2 + x_3^2)^{1/2}}{\bar{V}m}\right)}$$

where n is the frequency

x1 is the horizontal separation between xi and x$'$j

x3 is the vertical separation between xi and x$'$j

and $\bar{V}m$ is $\frac{\bar{V}\ (x3)\ +\ \bar{V}'\ (x3)}{2}$

A further problem concerns the height at which the mean velocity may be assumed to act when points are seperated vertically. Davenport in his original formulation used 10 metres regardless of the actual height but it has been found preferable to take a speed corresponding to the mean height between the points.

CHAPTER 4

WIND FORCES ON STRUCTURES IN TURBULENT FLOW.

The problem of wind generated forces is complicated, by the presence of a structure which itself modifies the flow in its vicinity. In broad terms, the difficulty is to express the fluctuating forces on a body in turbulent flow in terms of the power spectrum of fluctuating velocities. Such an expression is known as a transfer function and called the aerodynamic admittance.

Consideration will now be given to the formulation of these functions in general terms and their application to different types of structure.

4.1. Point Correlations.

The drag force D(t) on a body in turbulent flow can be represented by the equation :

$$D(t) = \tfrac{1}{2}\rho V(t)^2 l\, C_D + \rho l^3 \frac{dV(t)}{dt} . C_m \qquad -(4.1)$$

where ρ is the fluid density

l is the representative body dimension.

C_D is the drag co-efficient

and Cm is the inertial or virtual mass co-efficient.

$\rho l^3 C_m$ is known as the inertial mass since the inertial force, Di(t), resulting from the additional accelerations in the flow due to the presence of the body, equals $\rho l^3 C_m \frac{dV(t)}{dt}$

If this expression is divided into mean and fluctuating parts such that :

$$V(t) = \bar{V} + v_1$$

Then, if the fluctuating part of the drag force is p :

then $\bar{D} + p = \tfrac{1}{2}\rho(\bar{V}^2 + v_1^2 + 2\bar{V}v_1)\, l^2 C_D + \frac{d(\bar{V}+v_1)}{dt}\rho l^3 C_m \qquad -(4.2)$

But, for the fluctuating force only, v_1^2, is small and hence :

$$p = \rho \bar{V} L^2 C_D + \frac{dv_1}{dt} \rho L^3 C_m \qquad -(4.3)$$

The auto-covariance function for the fluctuating load, $C^P(\tau)$ is :

$$C^P(\tau) = \overline{p(t)\,p(t+\tau)}$$

$$= \rho^2 \bar{V}^2 L^4 C_D^2\, \overline{v_1(t) v_1(t+\tau)} + \rho^2 L^6 C_m^2\, \overline{\frac{dv_1(t)}{dt}\frac{dv_1(t+\tau)}{dt}}$$

$$+ \rho^2 \bar{V} L^5 C_D C_m \left[\overline{v_1(t)\frac{dv_1}{dt}(t+\tau)} + \overline{v_1(t+\tau)\frac{dv_1}{dt}(t)}\right]$$

$$-(4.4)$$

Since the process is stationary and the function symmetrical, the third term is equal to zero and :-

$$\overline{\frac{dv_1(t)}{dt}\frac{dv_1(t+\tau)}{dt}} = -\frac{d^2}{d\tau^2}\left(\overline{v_1(t)v_1(t+\tau)}\right) \qquad -(4.4a)$$

so that

$$C^P(\tau) = \rho^2 L^4 \left(\bar{V}^2 C_D^2\, \overline{v_1(t)v_1(t+\tau)} - L^2 C_m \frac{d^2}{d\tau^2}\left(\overline{v_1(t)v_1(t+\tau)}\right)\right)$$

$$-(4.5)$$

Using the Fourier Transform relationship :

$$S^P(n) = 4\int_0^\infty C^P(\tau)\cos 2\pi n\tau\, d\tau$$

Then :

$$S^P(n) = 4L^4\rho^2 \int_0^\infty \left(\bar{V}^2 C_D^2\, C^V(\tau)\cos 2\pi n\tau - L^2 C_m^2 (2\pi n)^2 C^V(\tau)\cos 2\pi n\tau\right) d\tau$$

$$-(4.6)$$

Therefore :

$$\frac{S^{P}(n)}{\frac{1}{4}\rho^{2}L^{4}\bar{V}^{4}} = S^{C_D}(n) = 4C_D^2 \frac{S^{V}(n)}{\bar{V}^2}\left[1 + \frac{C_m^2}{C_D^2}\left(\frac{2\pi nL}{\bar{V}}\right)^2\right] \qquad -(4.7)$$

where $S^{C_D}(n)$ is the power spectrum for the fluctuating drag co-efficient C_D.

Thus relationship between fluctuating drag and velocity is

$$\chi_a^2 = 4C_D^2\left[1 + \frac{C_m^2}{C_D^2}\left(\frac{2\pi nL}{\bar{V}}\right)^2\right] \qquad -(4.8)$$

where χ_a^2 is the aerodynamic admittance.

The term, $\frac{2\pi nL}{\bar{V}}$, is referred to as the reduced frequency. It can be seen that the drag and inertial mass co-efficients are functions of both the reduced frequency and their position on the structure. Hence the expression for the aerodynamic admittance must be modified to take into account spatial correlations of the fluctuating velocity.

Small wave length gusts have only a localised effect and therefore are less well correlated than larger gusts. At some stage the wave size must become so small that it no longer has a perceptible effect.

4.2. Fluctuating Drag Including Spatial Correlation.

In order to assess the effects of spatial correlation of velocities or pressures over a structure, it is necessary to break its surface up into a number of infinitesimal sub-areas over which the correlation is assumed to be total.

Thus, from equation (4.2), the fluctuating drag over the whole surface becomes :

$$p(t) = \int_0^A \left(\rho\,\bar{V} v_1 C_D + \rho C_m L \frac{dv_1}{dt}\right) dA \qquad -(4.9)$$

so that, following the derivation as in equation (4.4), the covariance of fluctuating drag force may be written:

$$C^p(\tau) = \overline{p(t)\,p(t+\tau)}$$

$$= \rho^2 \int_0^A\!\!\int_0^A \Big[\bar{V}_k \bar{V}_l C_{D_k} C_{D_l} \overline{v_k(t).v_l(t+\tau)}$$

$$+ l_k l_l C_{m_k} C_{m_l} \overline{\frac{d}{dt} v_k(t) . \frac{d}{dt} v_l(t+\tau)}$$

$$+ l_k C_{m_k} \bar{V}_l C_{D_l} \overline{v_l(t). \frac{d}{dt} v_k(t+\tau)}$$

$$+ l_l C_{m_l} \bar{V}_k C_{D_k} \overline{v_k(t+\tau).\frac{d}{dt} v_l(t)} \Big] dA_k\, dA_l$$

$$= \rho^2 \int_0^A\!\!\int_0^A \Big[\bar{V}_k \bar{V}_l C_{D_k} C_{D_l} C^v_{kl}(\tau)$$

$$- l_k l_l C_{m_k} C_{m_l} \frac{d^2}{d\tau^2} C^v_{kl}(\tau)$$

$$+ l_k C_{m_k} \bar{V}_l C_{D_l} \left\{ -\frac{d}{d\tau} C^v_{kl}(\tau) \right\}$$

$$+ l_l C_{m_l} \bar{V}_k C_{D_k} \frac{d}{d\tau} C^v_{kl}(\tau) \Big] dA_k\, dA_l$$

— (4.10)

$C^v_{kl}(n)$ is the velocity cross-covariance function between any two points k and l on the structure.

The power spectrum, $S^p(n)$ can now be written by using the Fourier transform relationship:

$$S^p(n) = 4 \int_0^\infty C^p(\tau) \cos 2\pi n \tau \, d\tau$$

$$\& \quad S^v_{kl}(n) = 4 \int_0^\infty C^v_{kl}(\tau) \cos 2\pi n \tau \, d\tau$$

Thus :

$$S^p(n) = \rho^2 \int_0^A\!\!\int_0^A \Big[\bar{V}_k \bar{V}_l C_{D_k} C_{D_l} S^v_{kl}(n) + l_k l_l C_{m_k} C_{m_l} (2\pi n)^2 S^v_{kl}(n)$$

$$+ l_k C_{m_k} \bar{V}_l C_{D_l}\, 2\pi n\, i\, S^v_{kl}(n) + l_l C_{m_l} \bar{V}_k C_{D_k}\, 2\pi n\, i\, S^v_{kl}(n) \Big] dA_k\, dA_l .$$

— (4.11)

This expression can be reduced by noting that the power spectral density function must be real so that :

$$P^{V}_{kL}(n) = S^{V}_{kL}(n) \cos \phi_{kL}(n) \qquad -(4.12)$$

where $P^{V}_{kL}(n)$ is the co-coherence defined in equation (3.22) and $\phi_{kL}(n)$ is the phase angle. $\phi_{kL}(n) = \tan^{-1}\left(\frac{Q^{V}_{kL}(n)}{P^{V}_{kL}(n)}\right)$

Thus the normalized cross-spectrum, $R^{V}_{kl}(n)$, is given by :-

$$P^{V}_{kL}(n) = R^{V}_{kl}(n)\sqrt{S^{V}_{k}(n)S^{V}_{L}(n)}\cos \phi_{kL}(n)$$

and :-

$$S^{P}(n) = \rho^{2}\int_{0}^{A}\int_{0}^{A}\Big[\bar{V}_{k}\bar{V}_{L}\left(C_{Dk}C_{DL} + \xi_{k}\xi_{L}C_{mk}C_{mL}\right) R^{V}_{kL}(n)\sqrt{S^{V}_{k}(n)S^{V}_{l}(n)}\cos\phi_{kl}(n)\Big]dA_{k}\,dA_{L}$$

-(4.13)

where ξ_k, ξ_L are the reduced frequencies :

$\frac{2\pi n L_k}{\bar{V}_k}$ and $\frac{2\pi n L_L}{\bar{V}_L}$ respectively.

The validity of the assumptions which lead to Equation(4.13) has been examined by Vickery and Davenport (18). The velocity pressure relationship as given by equation (4.1) is adequate for open lattice structures where the flow is not substantially altered by the structure itself and the magnitude of the velocity disturbances is small compared with the structure size. In the case of large bluff structures, however, the validity of the relationship is more dubious since the disturbed velocity field bears little relation to the undisturbed one.

This is largely because the velocity field is no longer substantially made up of the along-wind component. As other velocity components become more significant the aerodynamic co-efficients C_D and C_M no longer hold and it becomes necessary to define the spectra in terms of more general aerodynamic admittance functions.

For lack of experimental data concerning these functions, it is generally assumed (10) that the assumptions of equation (4.1) are valid even for bluff buildings.

It should be mentioned here that most researchers also assume that the pressure fluctuations on the windward face and the suction fluctuations on the leeward face are practically uncorrelated (10) and that the total load can be satisfactorily assumed to be the total mean pressure (both pressure and suction) together with the fluctuating pressure on the windward face only.

Recent work by Simiu (11), however, shows that this may lead to significant errors in the prediction of the maximum response and suggests a method to include fluctuations in the suction on the leeward face.

The formulation given above for the power spectrum of fluctuating drag is based on the work of Solnes and Sigbjörnson (10) which includes the inertial mass in the formulation. Although Davenport (3) gives some experimental values for the inertial mass co-efficient, C_M, applied to a flat plate in turbulent flow, they are not generally applicable and, for design purposes, inertial mass is ignored for lack of experimental back-up.

Parametric studies of wind loading using random simulation techniques (19) have shown that neglecting higher order terms (as in Eqn. 4.3) and non-linear and variable damping terms results in both the fluctuating forces and response tending to be non-Gaussian in their nature.

The error in neglecting these higher order terms could be much as 10% of the response. Also, terms involving C_M are dependent on the nature and size of the structure, and errors of 15% are possible if these terms are neglected.

CHAPTER 5

THE CALCULATION OF STRUCTURAL RESPONSE.

The fluctuating pressures resulting from the turbulent wind velocity produce certain structural responses. The relative importance of these responses will depend on the design criteria applied. For instance, in a tall building where occupier comfort is a major consideration, the accelerations at each floor level are likely to be a limiting factor in the design - whereas, a microwave tower will certainly have a maximum allowable displacement.

In this study, since modal analysis is used, the variance of the displacement for each mode is calculated and from this, the variance of the required response can be formed.

The structure is assumed to have reached a steady-state of vibration and the response is linear. Further, although it is likely that in these conditions a low cycle fatigue or cumulative plastic damage would lead to failure, little is known about these effects in relation to non-deterministic analysis and, therefore, the so-called "first passage approach" is used whereby failure is deemed to have taken place when a given stress or response level is exceeded.

5.1. Modal Analysis.

In the calculation of the response spectrum, modal analysis is a natural choice. The resolution of response into natural modes of free vibration allows the contribution of each to the total response to be treated separately.

When a structure is vibrating in a single mode, the displacement at any point can be written :

$$q(t,x) = Q_j(t)\,\mu_j(x) \qquad \text{— (5.1)}$$

where $Q_j(t)$ is called the generalised displacement for the jth mode and $\mu_j(x)$ is the ordinate of the modal function. * $Qj(t)$ defines the amount of $\mu_j(x)$ present in the excitation and the acceleration at any point is :

$$\frac{dq^2}{dt^2} = \mu_j \frac{d^2Q}{dt^2} = -\omega_j^2 \mu_j Q_j \qquad -(5.2)$$

and the inertial force p(t) required to excite this acceleration is :

$$p\,(t) = m(x)\,\mu_j\,\omega_j^2\,Q_j \qquad -(5.3)$$

where m(x) is the mass per unit length and ω_j, the circular frequency.
Because of the orthogonality of natural modes whereby it is possible to write :

$$\int_0^l m(x)\,\mu_i(x)\,\mu_j(x)\,dx = 0 \quad \text{for } i \neq j$$

the generalised force Pj(t) can be written :

$$Pj(t) = \int_0^l p(x,t)\,\mu_j(x)\,dx$$

so that

$$P_j(t) = \int_0^l m(x)\,\mu_j^2(x)\,Q_j\,\omega_j^2\,dx$$

and the generalised stiffness Kj is given by

$$K_j = P_j/Q_j = \omega_j^2 \int_0^l m(x)\,\mu_j^2(x)\,dx \qquad -(5.4)$$

Hence by making the necessary transformation to generalised co-ordinates, the modal contribution to the response can be calculated from the equation :

$$M_j \frac{d^2Q_j}{dt^2} + C_j \frac{dQ_j}{dt} + K_j Q_j = P_j \qquad -(5.5)$$

* Note $\mu j(x)$ refers here to the normalised values obtained by scaling all the ordinates such that the maximum value is unity.

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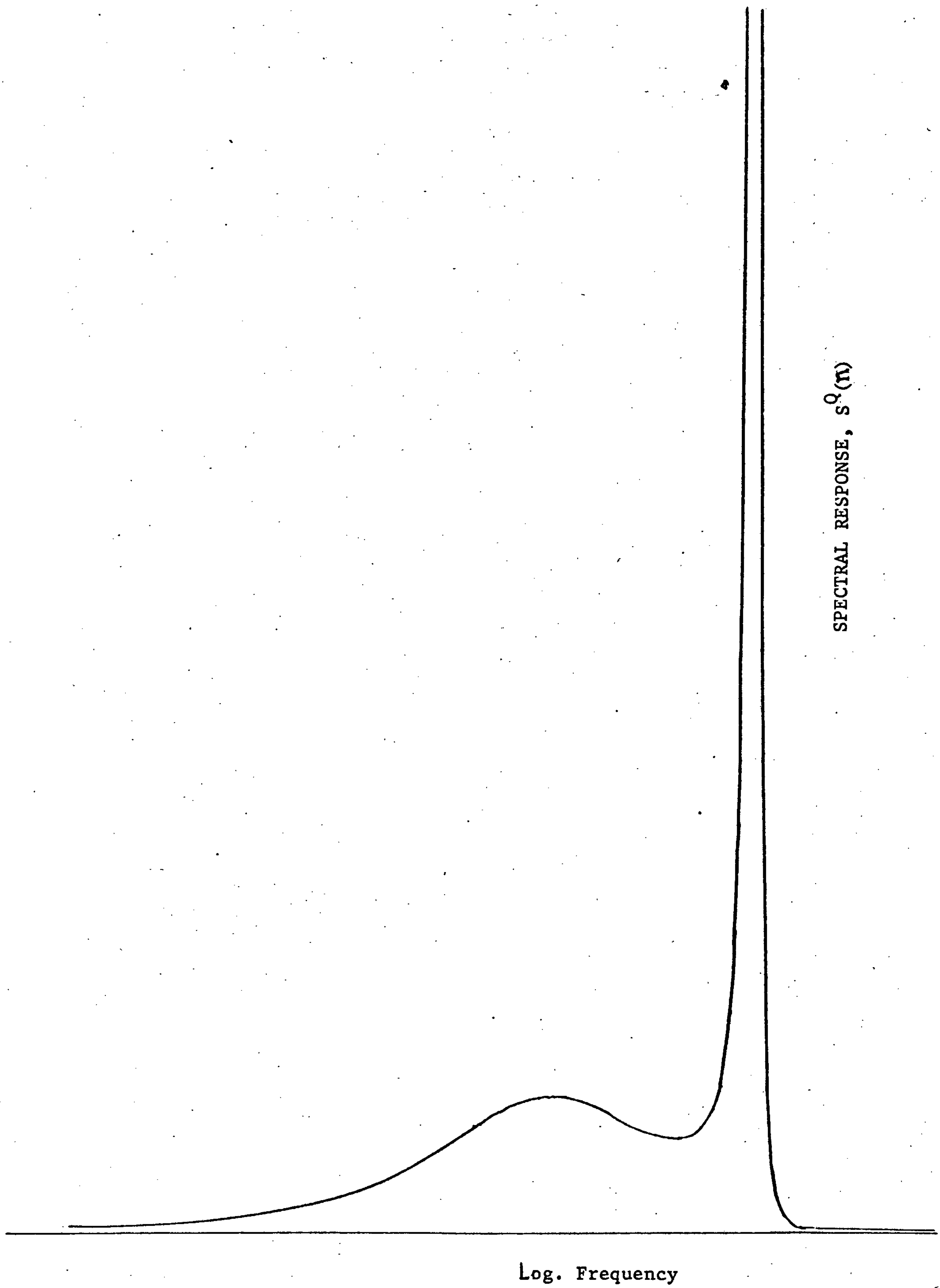

RESPONSE SPECTRUM FOR FIRST MODE DISPLACEMENT

FIG. 8

and the steady state response to a sinusoidal excitation of frequency n and amplitude P_j can be shown to be :

$$Q_j = \frac{P_j}{K_j} \frac{1}{\left[\left(1 - n^2/n_j^2\right)^2 + \left(\frac{\delta n}{\pi n_j}\right)^2\right]^{1/2}} \qquad -(5.6)$$

where δ is the logarithmic decrement (assuming damping is small) and n_j is the modal frequency.

From this, the spectral response can be shown to be :

$$S_j^Q(n) = \frac{S_j^P(n)}{K_j^2} \frac{1}{\left[\left(1 - n^2/n_j^2\right)^2 + \left(\frac{\delta n}{\pi n_j}\right)^2\right]} \qquad -(5.7)$$

and the mechanical admittance $X_{mj}^2(n)$ is defined by :

$$X_{mj}^2(n) = \frac{1}{K_j^2} \frac{1}{\left[\left(1 - n^2/n_j^2\right)^2 + \left(\frac{\delta n}{\pi n_j}\right)^2\right]}$$

Hence,

$$S_j^Q(n) = S_j^P(n)\, X_{mj}^2(n) \qquad -(5.8)$$

so that the spectrum of structural response for each mode can be found by considering each frequency in turn over the full range. Then, by definition, the variance of the response is the area under the spectral response curve :

$$\sigma^2(Q_j) = \int_0^\infty S_j^Q(n)\, dn \qquad -(5.10)$$

Finally, assuming there is no coupling between modes the standard deviation of the response can be calculated by summing the variances for each mode considered :-

$$\sigma_Q = \sqrt{\sigma_1^2 + \sigma_2^2 + \sigma_3^2 + \ldots + \sigma_n^2} \qquad -(5.11)$$

The spectral response curve for each mode has a very pronounced resonant peak corresponding to the appropriate natural frequency (See fig.8) under which most of the area is concentrated. In practice, it is only the spectral peaks of the first three or so modes which contribute and the inaccuracy involved in ignoring all but the fundamental mode is small.

Finally, given the modal displacements at any point in the structure, the stresses induced can be found by using the appropriate modal influence functions. For example, the modal influence function for shear force at a section x from the base of a line structure of height, H_1 is :

$$\beta_j^F = \omega_j^2 \int_0^H m(x)\,\mu_j(x)\,dx$$

and the corresponding moment influence function :

$$\beta_j^m = \omega_j^2 \int_0^H m(x)\,\mu_j(x)\,x\,dx$$

5.2. <u>The Modal Response Spectrum.</u>

Equation 4.13 can now be used to write the expression for the spectral response for each mode.

$$S_j^P(\omega) = \rho^2 \int_0^H \int_0^H \left[\bar{V}_k \bar{V}_L\, B_k B_L\, \mu_j(k)\mu_j(L)\, R_{KL}^V(n)\, S^V(\omega) \cos\phi_{KL}(n)\right] dH_k\, dH_L \qquad -(5.12)$$

(Considering, now, a vertical line structure and integrating over the height).

Bk and Bl are factors which contain the variation of breadth and the pressure coefficients C_D and C_M over the structure.

Note that the expression also contains the mode shape ordinates at k and l, $\mu_j(k)$ and $\mu_j(l)$. This results from the orthogonality property which allows the force spectrum for the jth mode to be expressed in terms of the mode shape. Also, it is assumed that the velocity spectrum is invariant with height, i.e.

$$S^v(n) = \sqrt{S^v_k(n)\,S^v_l(n)} \qquad -(5.13)$$

Now, it is convenient to define new non-dimensional parameters γ_k and γ_l such that :

$$\gamma_k = \frac{\bar{V}_k B_k \mu_j(k)}{\bar{V}_0 B_0}$$

where $\bar{V}_0$ and B_0 are taken at any convenient reference point (in this study, always the 10 m reference height).

For vertical structures, a further term must be added to equation 5.8 to account for the variation of the mean velocity with height.

Thus :

$$S_j^P(n) = \rho^2 \bar{V}_0^2 B_0^2 \bar{V}(10)^2 \int_0^H\!\!\int_0^H \left[\gamma_k \gamma_l R^v_{kl} \cos\phi_{kl}(n)\right] dH_k\, dH_l \; \frac{S^v(n)}{\bar{V}(10)^2} \qquad -(5.14)$$

Now if the mean wind load $\bar{P}_j$ is defined by :

$$\bar{P}_j = \tfrac{1}{2}\rho B_0 \bar{V}_0 \bar{V}(10) \int_0^H \gamma_k \frac{\bar{V}_k}{\bar{V}(10)}\, dH \qquad -(5.15)$$

then :

$$S_j^P(n) = 4\,\bar{P}_j^2\, Ja_j^2\, J_{pj}^2(n)\, \frac{S^V(n)}{\bar{V}_{(10)}^2} \qquad -(5.16)$$

where

$$Ja_j^2 = \left\{ \frac{1}{\int_0^H \gamma_k\, \bar{V}_k / \bar{V}_{(10)}\, dH} \right\}^2 \qquad -(5.17)$$

and

$$J_{pj}^2(n) = \int_0^H \int_0^H \left[\gamma_k \gamma_L\, R_{kL}(n) \cos\phi_{kL}(n) \right] dH_k\, dH_L \qquad -(5.18)$$

Ja_j^2 and $Jp_j^2(n)$ are called the modal admittances and Ja_j^2 expresses primarily the variation of the mean speed with height and $Jp_j^2(n)$ expresses the correlation of the gust effects over the structure. Further explanation of the modal admittance terms is necessary for the complete understanding of the method.

5.3. Modal Admittances.

Consider the vertical line structure shown in Figure 9.

It can readily be seen that the mean wind load for each mode on the structure can be found by summing the contributions at each level :

$$\bar{P}_j = \frac{1}{2}\rho\, \delta H \sum_{k=1}^{6} B_k \bar{V}_k^2 = \frac{1}{2}\rho\, \delta H\, B_o \bar{V}_o \bar{V}_{(10)} \sum \frac{\bar{V}_k B_k}{\bar{V}_o B_o} \cdot \frac{\bar{V}_k}{\bar{V}_{(10)}}$$

Where B now contains the breadth and the pressure co-efficients C_D and C_M and δH is the height interval. Since the variation with height is a continuous function, then

$$\bar{P}_j = \frac{1}{2}\rho\, \bar{V}_o\, \bar{V}_{(10)}\, B_o \int_0^H \gamma_k \frac{\bar{V}_k}{\bar{V}_{(10)}} \cdot dH$$

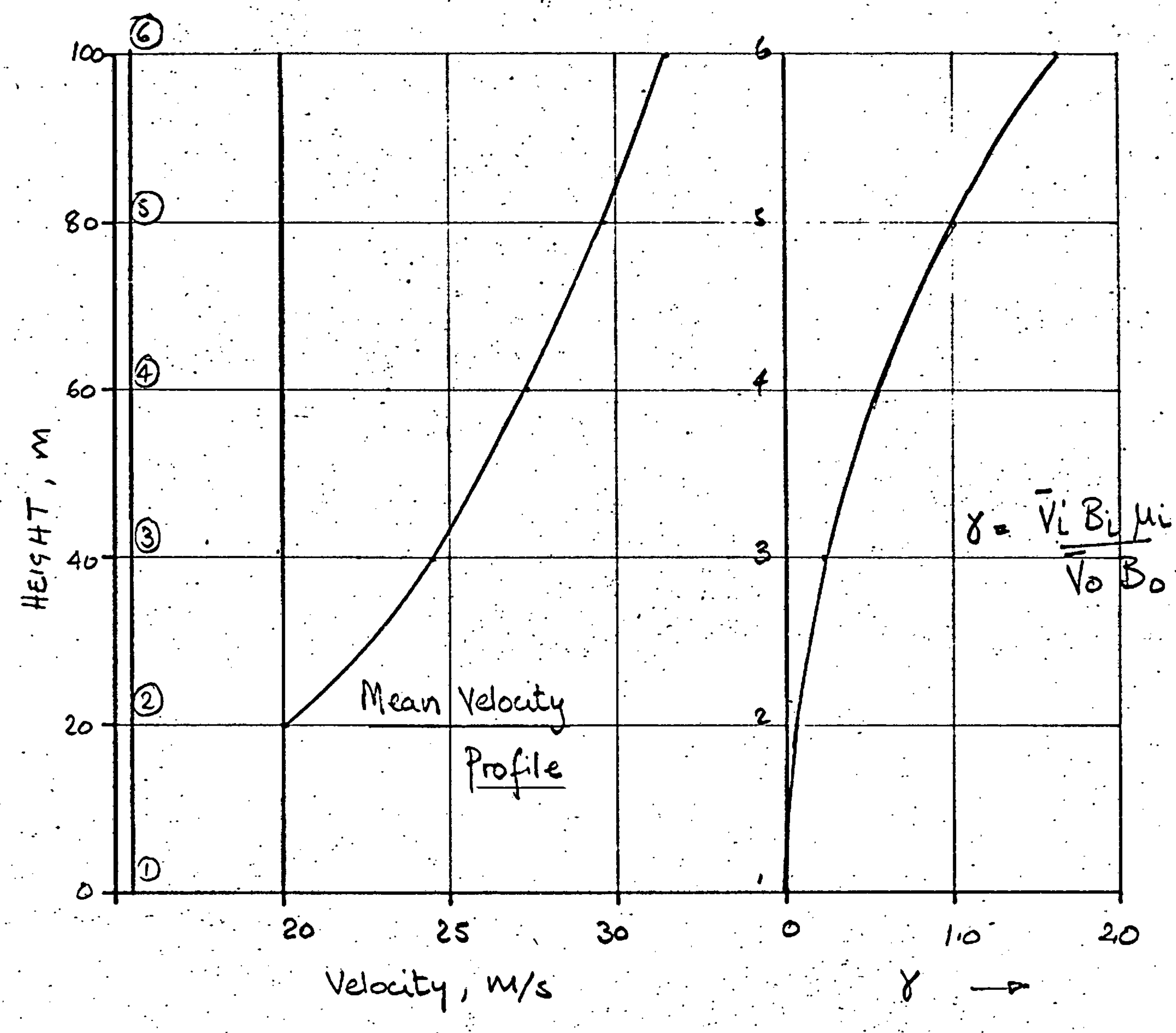

Fig. 9

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The admittance function, $J^2_{\rho_j}(n)$ expresses the sum, over all the elements of the structure, of the effects of the wind velocity contributions made to the spectral load for the frequency n :-

i.e.

$$J^2_{\rho_j}(n) = \sum_{k=1}^{6} \sum_{l=1}^{6} \gamma_k \gamma_L \, e^{-8\frac{nr}{\bar{V}}}$$

(using Davenport's coherence function).

Considering Figure 9 again, if, for example, we assumed a frequency of 1 HZ, we would have the sum of the terms in a matrix of the form :-

$$\begin{aligned} J^2_{\rho(n)} = {} & \gamma_1 \gamma_1 e^{-\frac{8\times 0}{V}} + \gamma_1 \gamma_2 e^{-\frac{8\times \delta H}{V}} + \gamma_1 \gamma_3 e^{-\frac{8\times 2\delta H}{V}} + \gamma_1 \gamma_4 e^{-\frac{8\times 3\delta H}{V}} \ldots \\ & + \gamma_2 \gamma_1 e^{-\frac{8\times \delta H}{V}} + \gamma_2 \gamma_2 e^{-\frac{8\times 0}{V}} + \gamma_2 \gamma_3 e^{-\frac{8\times 2\delta H}{V}} + \ldots \\ & + \gamma_3 \gamma_1 e^{-\frac{8\times 2\delta H}{V}} + \gamma_3 \gamma_2 e^{-\frac{8\times \delta H}{V}} + \gamma_3 \gamma_3 e^{-\frac{8\times 0}{V}} + \ldots \\ & + \gamma_4 \gamma_1 e^{-\frac{8\times 3\delta H}{V}} + \ldots \\ & + \gamma_5 \gamma_1 e^{-\frac{8\times 4\delta H}{V}} + \ldots \\ & + \gamma_6 \gamma_1 e^{-\frac{8\times 5\delta H}{V}} \ldots \end{aligned}$$

Computations are facilitated by noting that, in such a matrix, the diagonal, and lines parallel to it, have equal seperation distance r, so that the matrix becomes :

0	0	0	0	0	0
0	0.0007	0.0007	0.0006	0.0006	0.0004
0	0.0007	0.048	0.052	0.046	0.032
0	0.0006	0.052	0.278	0.246	0.172
0	0.0006	0.046	0.246	1.077	0.755
0	0.0004	0.032	0.172	0.755	2.62

→ k ; ↓ L

$r = 100$ m, $r = 80$ m, $r = 60$ m, $r = 40$ m, $r = 20$ m, $r = 0$

$\bar{V} = 20$ m/s

Since this function is also continuous, it clearly results in a volume integral of the kind shown in figure 10.

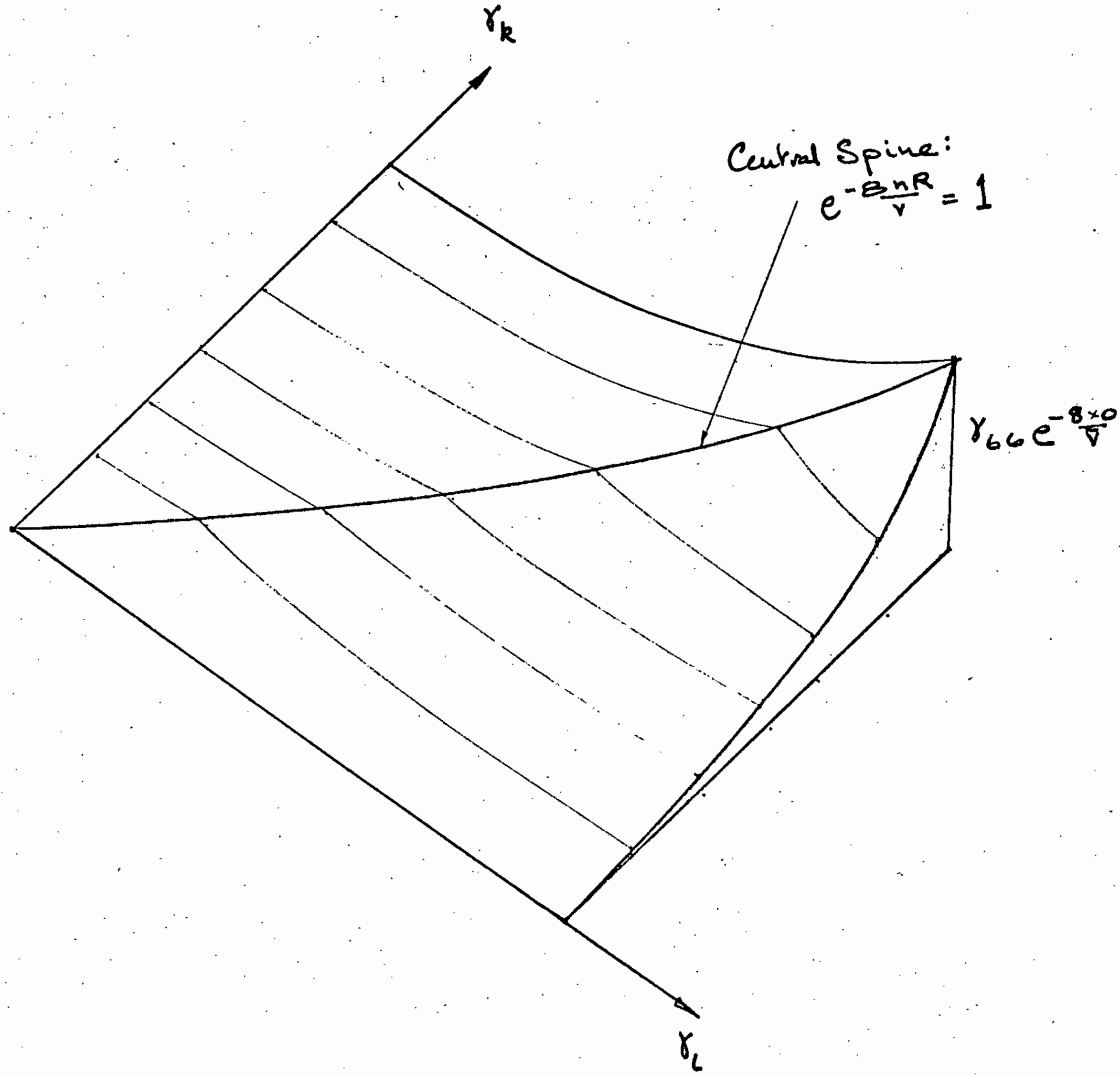


FIGURE 10

Some authors treat the problem simply by the discrete formulation indicated in Equation 5.? However, particularly at high frequencies, the "cusping" associated with the central spine ($k = l$ and $R^{v}_{ij}(\omega) = 1$) becomes very acute and it is recommended (20) that a curve fitting approach such as Simpson's Rule is used for the integral.

Bluff structures can be handled in a similar manner by dividing them up into vertical strips corresponding approximately in width to the vertical divisions as in Figure 11.

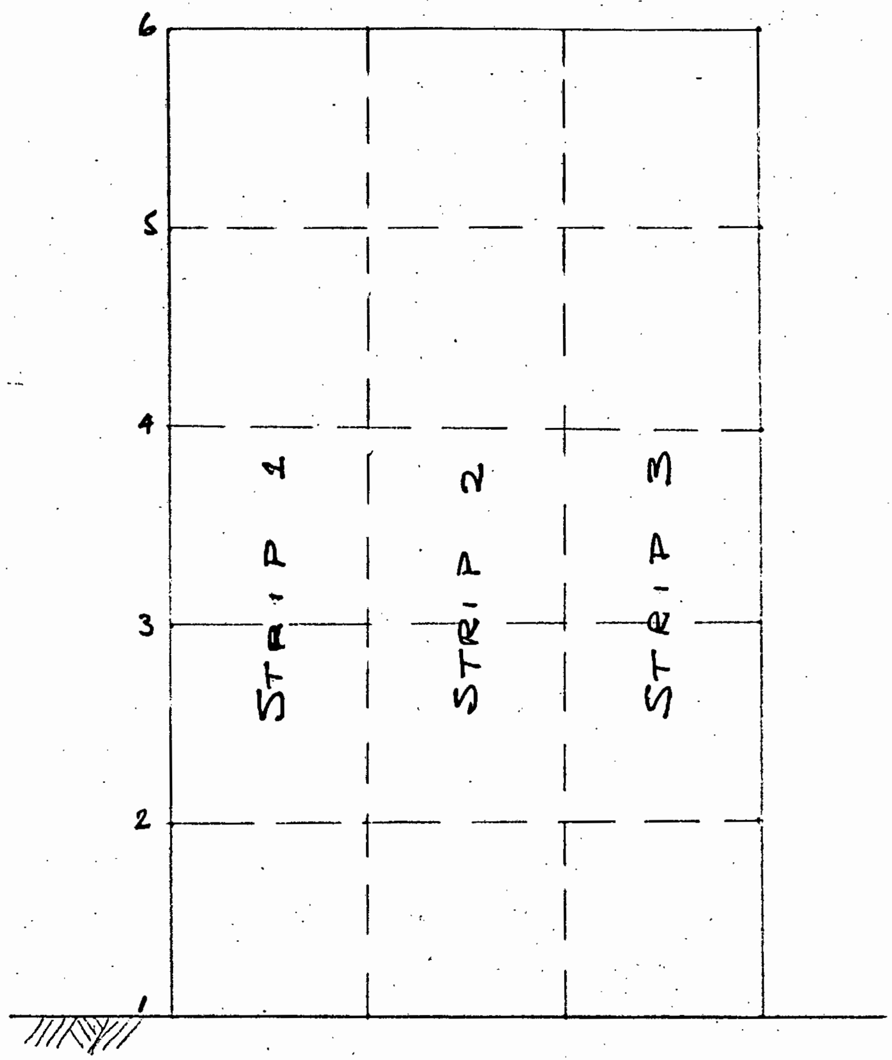


FIGURE 11

Normally, the natural frequency analysis for such a building would have been done on the basis of a simple line-like form and no torsion modes would have been considered. In this case, each vertical strip has the same modal ordinates and can be treated in the same way as the line structure, only cross-correlation between strips must be considered. Thus a symmetrical admittance matrix of the following form can be set up :

Strip 1	Strip 2	Strip 3	
11	12	13	Strip 1
21	22	23	Strip 2
31	32	33	Strip 3

Where 11,22, 33 are line correlations and 12, 23, 13 etc. express the cross-correlation between strips.

5.4. Calculation of Maximum Probable Response.

One of the major advantages of the statistical approach to wind loading is that it provides the designer with a means of assessing the risks of failure during the life of the structure.

In order to do this, it is necessary to know the distribution of peak values occurring within a given time.

Davenport has suggested that for the stationary random variable x during an averaging time of T (see ref. 5) :

$$x_{peak} = \bar{x}\left[1 + g(\nu T)\,\frac{\sigma(x)}{\bar{x}}\right] \qquad -(5.19)$$

where $$g(\nu T) = \sqrt{2\log_e \nu T} + \frac{0.5772}{\sqrt{2\log_e \nu T}} \qquad -(5.20)$$

and

$$\nu = \frac{\sigma'(x)}{\sigma(x)} = \left[\frac{\int_0^\infty n^2 S^x(n)\,dn}{\int_0^\infty S^x(n)\,dn}\right]^{1/2} \qquad -(5.21)$$

The averaging time T is usually taken as one hour.

Thus the most likely maximum response in a given averaging time is, therefore :

$$Q_{max} = \bar{Q}\left[1 + g\,\frac{\sigma(Q)}{\bar{Q}}\right] \qquad -(5.22)$$

Hence, the gust factor, GF, is :

$$GF = 1 + g\,\frac{\sigma(Q)}{\bar{Q}} \qquad -(5.23)$$

Note that this formulation for the maximum expected response assumes that this response is normal or Gaussian. It has been shown (19) that this is, in fact, not the case because of neglecting higher order and non-linear terms.

CHAPTER 6

THE APPLICATION OF THE STATISTICAL APPROACH.

In order to illustrate the use of the method and afford some comparison with conventional design, the effects of varying the required parameters will be examined for a bluff building. At the same time, there are a number of parameters such as the surface roughness co-efficient and the damping for which accurate assessment cannot be made. A knowledge of the sensitivity of the response to variations of these parameters is, therefore, essential if the statistical approach is to be used with confidence.

There are two solution techniques which are usually used. One is a static approach with the addition of dynamic effects close to the resonant frequency - the so-called "narrow-band" response. The other is the dynamic or frequency approach which uses modal analysis to evaluate the spectral response for the range of exciting frequencies. In this study, only the dynamic approach is used.

In much of the following, the gust response factor will be used as an indication of the sensitivity the structure to the fluctuating effects. In this context, the gust factor is taken to be the ratio of the extreme response to the mean response for the first modal displacement only. It will be shown that for many common applications this is a satisfactory assumption and one that is commonly made in simplified approaches.

6.1. NOTES ON COMPUTATIONAL METHODS :

A suite of computer programmes under the general title of DYNWIND has been written for the solution of a range of wind loading problems by the statistical approach.

It is not the purpose of this study to emphasize the computational aspect. The programming is straight forward and there are no doubt more efficient solution techniques. The writer makes no claims for exclusivity in this respect. For this reason, only a summary of the general aspects and a short resumé of each programme will be given.

6.1.1. Input : All the programmes require terrain and wind speed profile data, structural data and the mode shapes. Detailed input requirements are given in Appendix B.

6.1.2. Output: Each programme prints the velocity, force and response spectra, together with the admittance function, for each mode. The mean and standard deviation of the modal displacement are then given, followed by the gust factor for that mode.

6.1.3. Basic Programme :

The programme first computes the parameters that are not frequency dependent and then computes the spectral force contribution for a frequency range of 0.00001 to 10 HZ (i.e. a range of gusts having period of between 10000 and 0.1 seconds) from the relation (Eqn. 5.16):

$$S_j^P(n) = 4 \; \overline{P}j^{\,2} \; Ja^2 \; Jp^2(n) \; \frac{S^V(n)}{\overline{V}^2(10)}$$

$S^V(n)$ and $\overline{V}_{(10)}$ are known and $4 \, \overline{P}j^2 \, Ja^2$ is constant for each frequency, leaving $Jp^2(n)$ which is found as described in section 5.3.

Then, $S^Q(n)$ is calculated from (Eqn. 58) :

$$S^Q(n) = Sj^P(n) \; X^2mj$$

The necessary integrations are done using Simpsons's Rule with the exception of the integration of the response spectrum where the trapezoidal rule is used since the frequency increments were sufficiently small.
Note that in all the figures the admittance function, Jp(n), has been normalized by dividing by $\left[\int_0^A \gamma_k . dA\right]^2$

A listing of each programme is given in Appendix C.

6.1.4. DYNWIND/BUILDING allows the gust factor and modal spectral response for a rectangular bluff structure to be found. Any one of four different coherence functions may be chosen and the surface roughness co-efficient and the damping may be varied between prescribed limits. Only the overall dimensions and the number of vertical and horizontal divisions need be input. It is assumed that the mode shape has a strip form that only a small alteration would be required to include torsion modes as well.

6.1.5. DYNWIND/LATTICE allows the response of wind sensitive vertical lattice structures to be found. No horizontal correlation effects are considered although the choice of coherence functions remains.

6.1.6. DYNWIND/TOWER computes the response of transmission line towers. Horizontal correlation effects of the wind on the transmission wires are considered.

6.2. SUMMARY OF ASSUMPTIONS :

The following is a summary of the assumptions made for the purposes of the programme :

(a) The basis of the method is that the fluctuations in wind speed are a stationary random process.

(b) The wind velocity spectrum used is after Harris (7) as given in equation 3.15.

(c) The response is linear and a steady state has been attained.

(d) There is no coupling between modes.

(e) Shiotani's coherence function for anisotropic homogeneous turbulence has been used because it has the most general applicability.

6.3. THE EFFECT OF VARYING THE STRUCTURE SIZE :

Varying the structure size for a particular turbulent régime while holding all the other parameters constant illustrates the effect of the correlations on the gust factor.

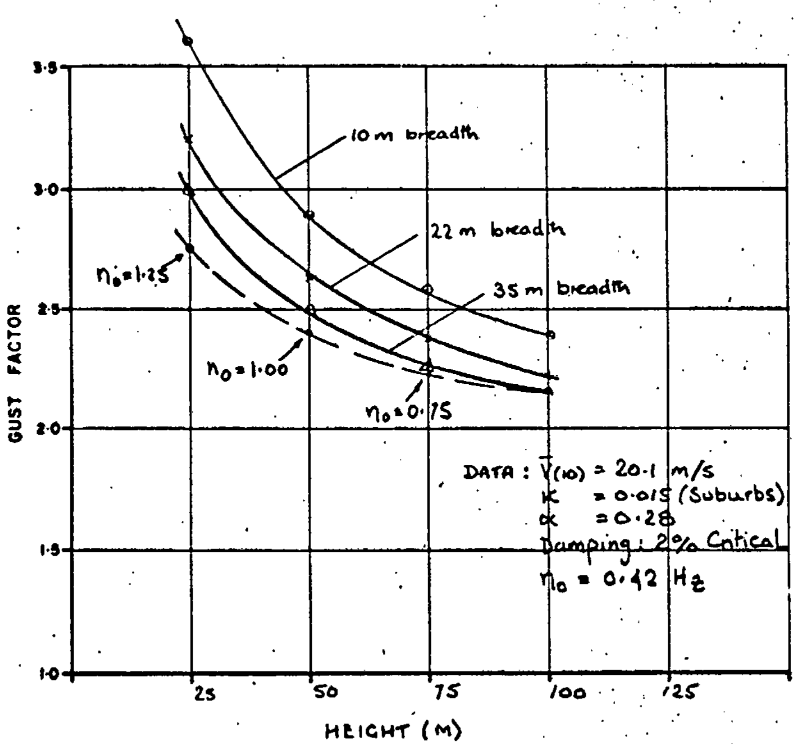
A bluff building is considered with a height that varies from 25 m to 100 m and a breadth of between 10 m and 35 m. (The fundamental mode only is considered for the purposes of these comparisons). It can be seen from figure 12 that the effect on the gust factor is considerable and that it is greater for narrow structures than for broad structures - for the 10 m width the reduction in the gust factor is 33% and for the 35 m width only 28%.

Intuitively, it is easy to see that long period gusts (say, 1000 seconds) will envelop the whole structure and that the effect of very short period gusts will be negligible because of their size in relation to the structure. This approach enables this effect to be quantified.

From Eqn. 5.16 it is seen that the admittance, $Jp^2(n)$, amounts to a frequency dependent factor which relates the spectral values of velocity and force. This factor varies according to the size of the structure. A decrease in size, increases the range over which the admittance factor is significant, that is to say - it is totally or partially correlated for a greater range of gust frequencies.

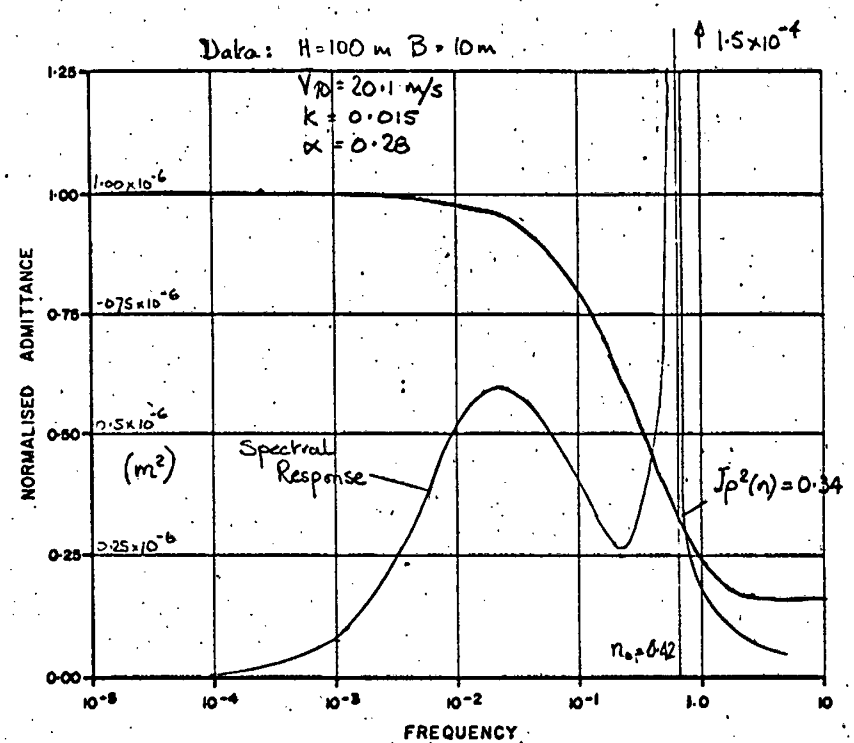
As can be seen from fig. 13 a large part of the area of the response spectrum (and, therefore, the variance) is concentrated close to the resonant peak.

Hence, if the gusts are significantly correlated at resonance, the result is the variance and, consequently, the gust factor is increased.



GUST FACTOR VARIATION WITH SIZE FOR BLUFF BUILDINGS

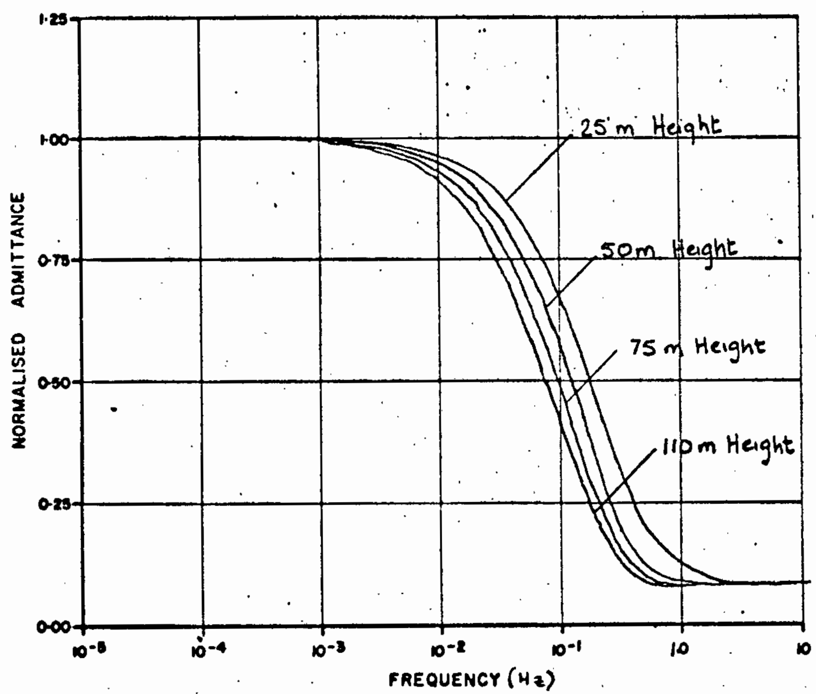
Fig. 12



Admittance and Spectral Response Curves

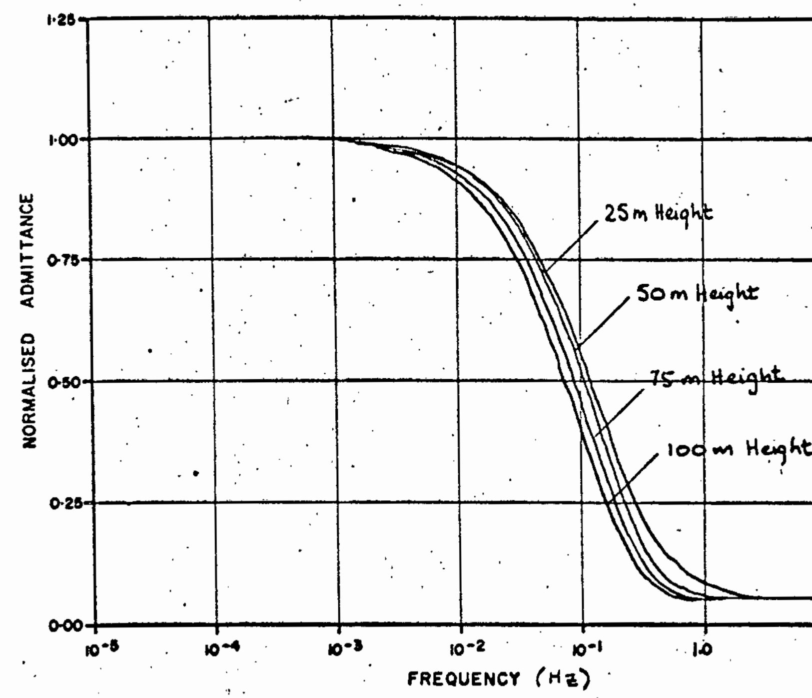
For 100 m High Building (B=10m)

Fig. 13



Admittance Function For Bluff Buildings (B=22m)

Fig. 15



Admittance Function For Bluff Buildings (B=35m)

Fig. 14

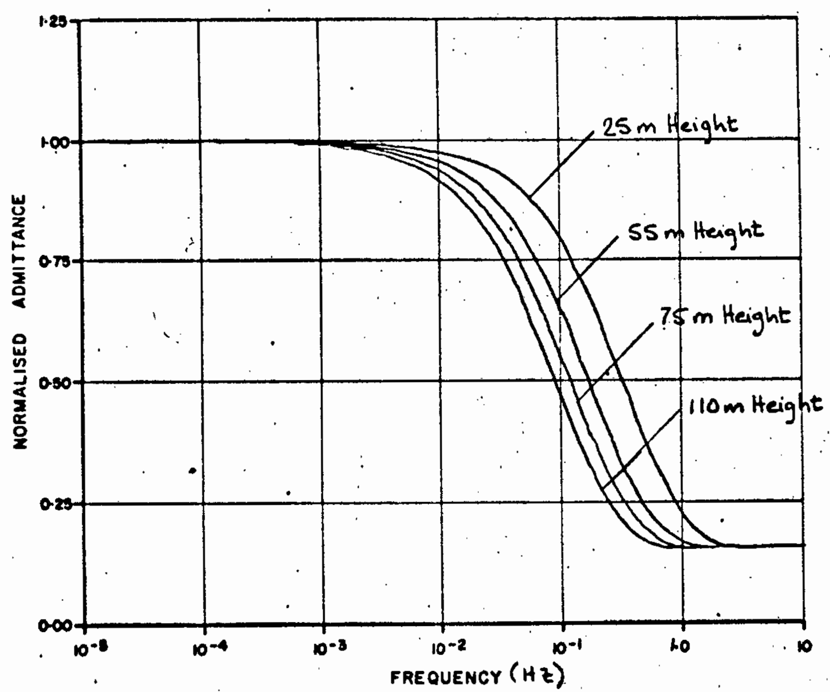
For example, referring to fig. 16 , a 25 m high by 10 m wide concrete tower subjected to gusts with a period of 100 seconds ($n = 10^{-2}$) has an admittance factor of 0.95 whereas, at the resonant frequency $Jp(n_o)^2$ is 0.34 which is still significant for the spectral peak. Thus, for this structure the gust factor is 3.6 compared with 2.4 for a 110 m height. (See figs. 14-16 for the variation of admittance with size.)

In practice, however, a 25 m high building would have a considerably higher fundamental frequency which will reduce this effect. This can be seen from the dotted line on figure 12 which represents the effect of simultaneously varying the fundamental frequency and the height.

6.4. NATURAL FREQUENCY, MODE SHAPE AND DAMPING :

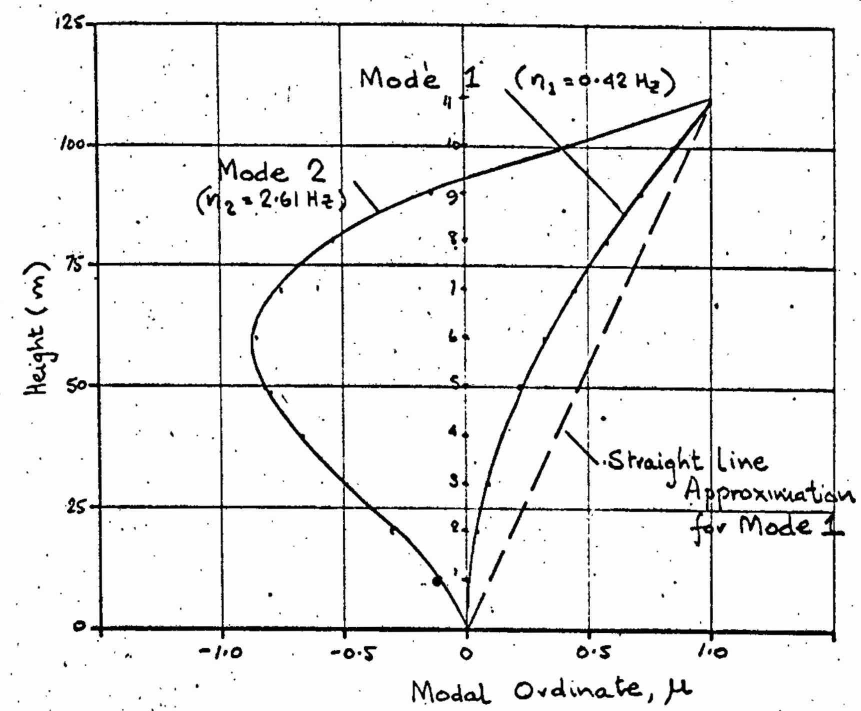
The dynamic characteristics of the structure effect the response and an important feature of the statistical approach is that it allows these effects to be taken into account.

There are many ways of finding the natural frequencies and mode shapes of structures (1, 4, 8). Buildings, however, tend to be more rigid than calculated because of the bracing effect of partitions and other redundants which are usually ignored. A study by Ward (31) of an eleven storey office building in Ottawa has shown that it is difficult to calculate the natural frequencies exactly and that the fundamental mode shape is close to a straight line about both the long and the short axes regardless of the presence of bracing shear walls. This would tend to show that bending effects do not predominate to the extent usually assumed in this type of structure and the assumption of a linear fundamental mode is the most reasonable regardless of the calculated shape. The values used for this study are taken from those calculated for a twenty-three storey concrete cored tower block (Reference 7) and as can be seen from figure 17 the bending effect predominates. However, the effect of using a linear mode shape instead has been investigated and the results are given in Table 4.



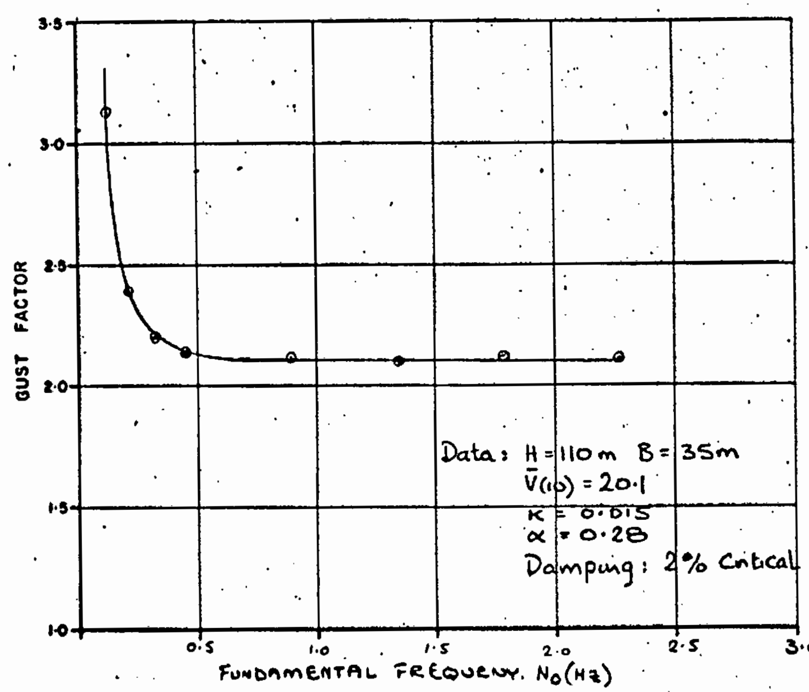
Admittance Function for Bluff Buildings (B = 10m)

Fig. 16



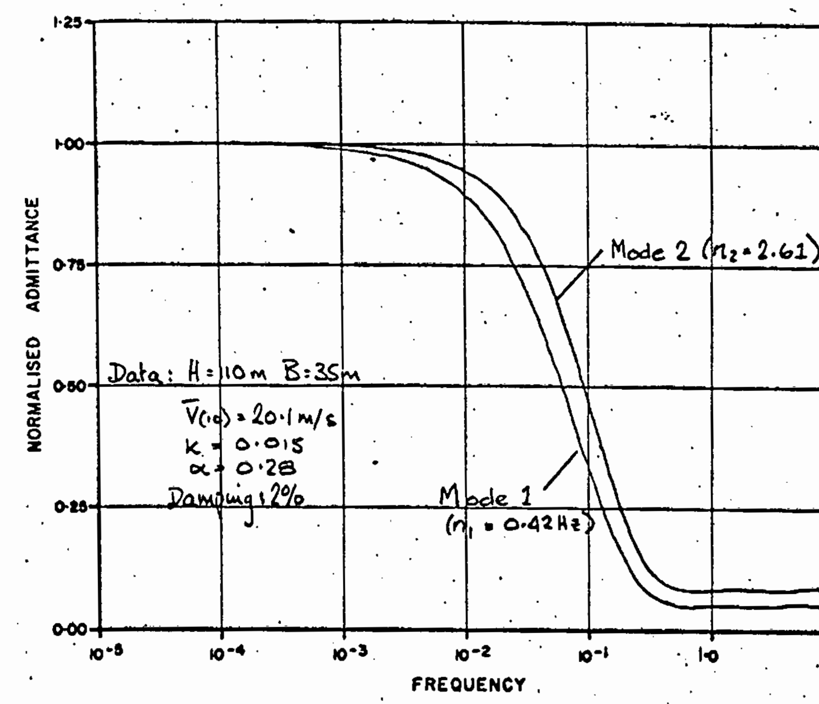
Mode Shapes For 110 m High Concrete-Cored High-rise Building (Masses Lumped at eleven equally spaced points)

Fig. 17



GUST FACTOR VARIATION WITH FUNDAMENTAL FREQUENCY

Fig. 18



The Effect of Higher Modes on the Admittance

Fig. 19

It can be seen that the effect on the gust factor is neglible.

TABLE 4 .

EFFECT OF STRAIGHT LINE MODE APPROXIMATION (B = 35m)

Building Height (m)	Gust Factor (Bending Mode)	Gust Factor (Straight line Mode)
25	2.998	3.005
50	2.506	2.504
75	2.279	2.275
100	2.141	2.136

Figure 18 shows the variation of the gust factor with fundamental frequency and that the gust factor is constant over a substantial range of the fundamental frequencies considered. This implies that so long as the gusts are sufficiently uncorrelated close to the resonant frequency, then dynamic magnification will be negligible.The interesting thing is that the structure has to be very flexible indeed before these effects become significant.

For most design applications it is necessary to consider the effect of higher modes. This depends on the type of structure but for the cantilever building considered with natural frequencies generally in the range 1:6:17:34 (characteristic of uniform cantilevers) and 1:3:5:7 (shear frames) - the effect of the second and higher modes can be ignored. The form of the second mode admittance function is given in fig. 19.

The effect on the gust factor can be found from :-

1st Mode : $\overline{Q}_1 = 0.023$ m $\sigma_1 = 0.007$ m

2nd Mode : $\overline{Q}_2 = 0.0003$ m $\sigma_2 = 0.0001$ m

and
$$G.F. = \frac{\overline{Q}_1 + \overline{Q}_2 + g \times \sqrt{\sigma_1^{\,2} + \sigma_2^{\,2}}}{\overline{Q}_1 + \overline{Q}_2}$$

thus $G.F. = 0.0227 + 3.75 \times 0.00701$

$G.F. = 2.162$ c.f. 2.141 (Ist Mode only)

Thus the increase in the gust factor resulting from considering the second mode in this case is less than 1%.

Damping is another important consideration which cannot be assessed with any degree of accuracy. The following figures for the mechanical damping percentage critical damping in different types of structure are suggested by Davenport (6) :

Concrete structures	1% - 2%
Steel structures	0.5%- 1%

This mechanical damping is not only a function of the material used but will also depend on the type of construction, intensity of excitation, joint rigidity and the foundations.

At the same time, there is a degree of aerodynamic damping which will take place as a result of dissipation of energy through the induced force fluctuations in the airflow. An estimate of this effect has been made by Davenport (4) but this has not been fully substantiated experimentally (7d).

Figure 19 shows the variation of the gust factor with increasing critical damping ratios. It can be seen that the estimation of damping ratios becomes critical for tall flexible structures, but for normal buildings the effect is slight.

52./......

6.5. THE STRUCTURE OF THE WIND WITH REFERENCE TO SITES IN SOUTH AFRICA :

The wind structure for a given location depends on the mean wind speed and the surface roughness co-efficient.

Both these parameters are interrelated since the degree of roughness effects the mean speed at ground level. Thus, it may be unrealistic to take a mean wind speed as applying to a particular area regardless of the terrain.

Unfortunately, although it may be possible to make an estimate of the roughness of the terrain for a particular site, it is usually difficult to assess with accuracy its effect on the mean wind speed.

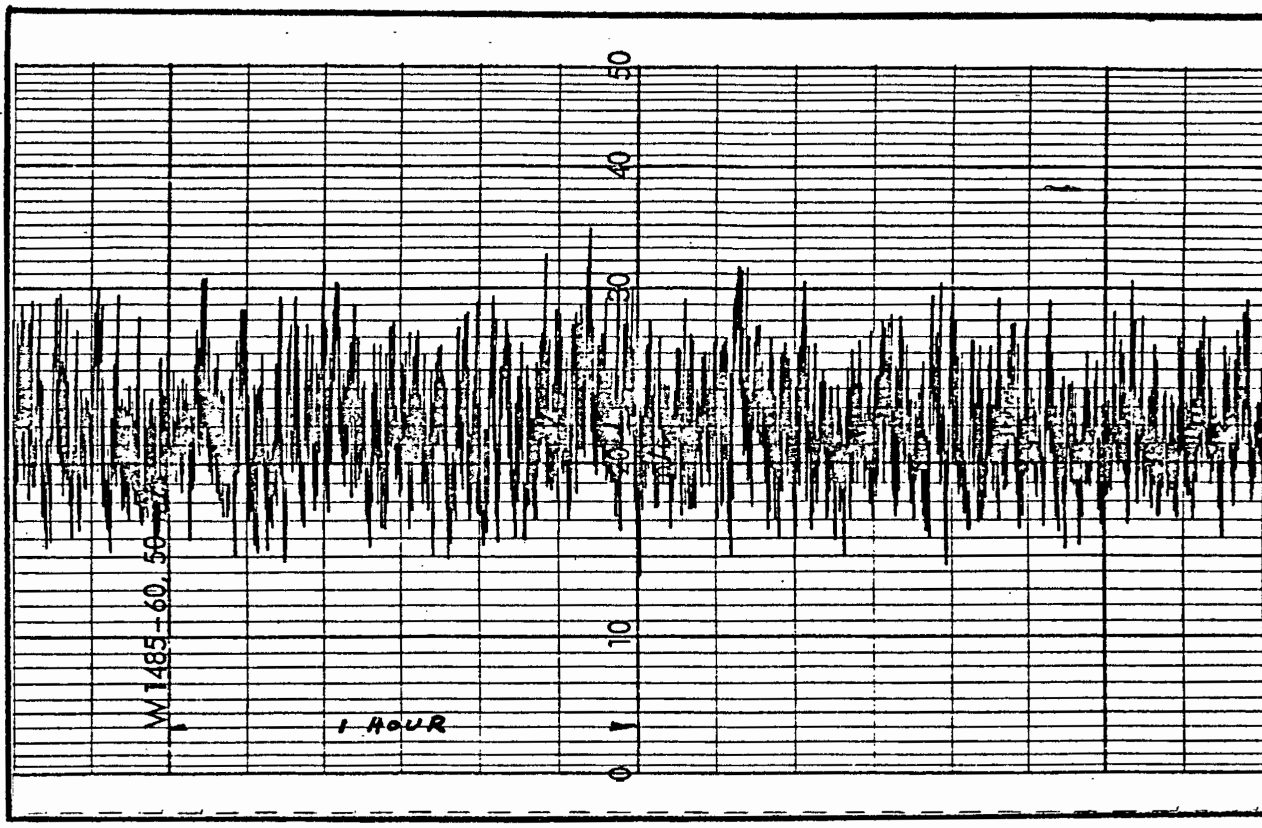
Since the wind speeds relating to a particular city in South Africa are normally measured at airports which tend to be in open flat terrain, the mean wind speeds recorded are usually higher than those in the built-up areas and the gust speeds will be lower.

The reverse is, however, true for Cape Town where, in the city centre, the funnelling and streamlining effects associated with the Table Mountain result in high mean speeds and intense gusting.

Fig. 21 shows an anomometer trace taken in Table Bay Harbour on the 18th November 1973. It can be seen that, for that record, the mean hourly wind speed was 23 metres per second and the extreme gust speed was 35 metres per second. This is the maximum mean hourly wind speed for that year and, from similar records, the 1974 maximum was 22 metres per second.

Unfortunately, prior to 1973, only maximum gust speeds were recorded by the South African Railways and Harbours and the anemometer traces are now unobtainable. It is difficult to draw any definite conclusions from just two statistics but calculating the return period as outlined in chapter 1 for each value from :

$$rt = 1/1 - pt$$



Anemometer Trace for 18th Nov. 1973 taken in Table Bay Harbour

Fig. 21

Then, using these points, an extrapolated line has been added to figure 2 showing the increased mean speeds in the harbour. This line is assumed to have the same slope as the line for the mean wind speed at D.F. Malan Airport which seems a reasonable assumption to make since the structure of the wind is not likely to have changed much in the ten kilometres or so between each location. From figure 2 the mean hourly wind and gust speeds for Table Bay Harbour can be read off as 27 and 41 metres per second respectively compared with 20,1 and 33 metres per second at D.F. Malan Airport.

It is interesting to note that the direction of the maximum mean hourly wind at the Harbour was consistently South East and, at D.F. Malan Airport, this was the case on only two occasions in fourteen years.

Where data is available, it is possible to estimate the relation between the two parameters from the difference between the maximum gust speed and the mean speed but this again depends on assumptions concerning the nature of the turbulence.

If the maximum two second gust speed and the mean hourly wind speed are known for a particular site, then the standard deviation of the wind velocity fluctuations may be calculated from :

$$V_{2s}(10) = \overline{V}_{(10)} + 3{\cdot}3\,\sigma_{v_1} \quad \text{(Eqn. 3.5)}$$

Although Equation 3.5 strictly only refers to an open country site ($\alpha = 0.16$), it seems reasonable to assume that the overall level of probability of occurence should not vary from site to site since this would lead to an inconsistent approach.

TABLE 5.

COMPARISON OF WIND-STRUCTURE PARAMETERS FOR THE MAIN CENTRES IN SOUTH AFRICA.

	CAPE TOWN	JOHANNES-BURG	DURBAN	BLOEMFONTEIN	PORT ELIZABETH	EAST LONDON	KIMBERLEY
Mean Hourly Wind, m/s (50 year return period)	20.1	19.4	24.7	22.7	21.5	23.8	19.7
Max. Gust Speed, m/s (50 year return period)	33.0	43.0	40.0	47.0	39.0	38.0	41.0
Standard Deviation of Gusts, m/s	3.9	7.15	4.63	7.36	5.3	4.3	6.45
Surface Roughness Co-efficient, K	0.005	0.02	0.005	0.015	0.01	0.005	0.016
Intensity of Turbulence, $\sigma_1/\bar{V}_{(10)}$	0.194	0.368	0.187	0.324	0.246	0.181	0.327

$\sigma_{v_1}/\bar{V}_{(10)}$ notation

Therefore, Equation 3.5 is assumed to relate to all sites regardless of the wind regime. This assumption gives figures for the intensity of turbulence at various sites in South Africa which correspond roughly to those given in Table 2 for Davenport's terrain roughness co-efficients. (Port Elizabeth is the only centre that falls into no specific category see Table 5.)

From equation 3.3, an estimate of the terrain roughness can now be made :

$$\sigma_{V_1} = 2.58\ K^{\frac{1}{2}}\ \overline{V} \quad (10)$$

The figures in Table 5 are based on wind speed data from various centres given in H.I. May's study : "Some wind speed data for estimating wind loads on structures in South Africa" (9). It should be noted that all but one of those on the coast have a surface roughness co-efficient corresponding to open grassland which points to influence of the sea on the wind structure in coastal areas.

The figures for the inland centres also show a remarkable degree of uniformity of terrain.

The significant fact about Table 5 is that in no way can the standard deviations of the turbulent fluctuations be taken as independent of terrain roughness at the various sites. This runs directly contractictory to the findings of Davenport and Harris (7) for sites in the Northern Hemisphere.

Predictably, as can be seen from Figure 22, the gust factor increases with the terrain roughness and this increase is most marked in the category applying to woodland, suburbs and small towns. It is this category that is most widely applicable in this country since the city centre areas are relatively small.

6.6. COMPARISON WITH C.P. 3 CHAP. V : (1970) :

In the conventional methods as set out in C.P. 3: Chap. V : 1970 (2) and the Standard Building Regualtions (21) the calculation of wind pressures is based on the maximum two second gust speed likely to be exceeded once in a given return period. Whereas, in the statistical approach, the maximum mean wind is used with a constant gust increment.

Apart from the essential differences between the two approaches referred to in the introduction, there is an inherent unreliability in using gust speeds. Wind recording instruments have become more sensitive in recent years which has resulted in records being broken. At the same time inertial effects cause the instrument to "overshoot" and, whereas both these effects tend to cancel out as far as mean speeds are concerned, they lead to cumulative positive errors in gust measurements. Mean wind speeds are, therefore, a more consistent measure on which to base design.

Based on the works of Deacon (22) in Australia, it has been found that the gust speed does not vary with height to the same exponent as the mean wind speed.

Deacon found that the power law exponent was 0.16 for the mean hourly wind speed and 0.085 for the average maximum gust. It is possible to compare the two approaches using Deacon's power law indices for the variation of the respective speeds with height and the use of a constant gust increment for the mean speed only.

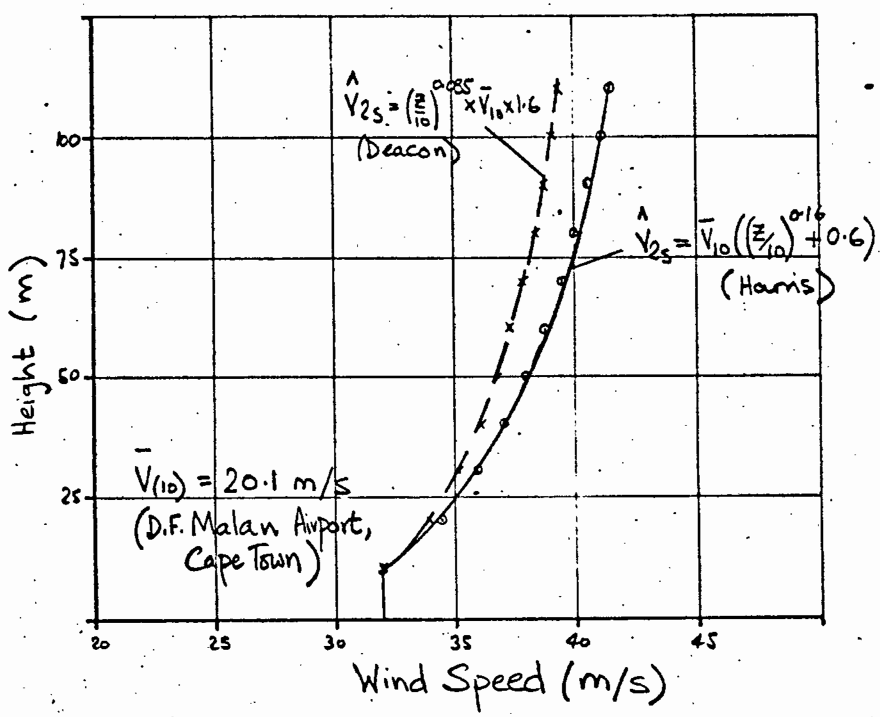
Thus, the two second gust speed may be expressed as :

$$\hat{V}_{2s} = \left(\frac{x_3}{10}\right)^{0.085} \times \bar{V}(10) \times 1.6$$

(assuming Harris's ratio of 1.6 for $\hat{V}_{2s}/\bar{V}(10)$)

or, using Equation 3.5 a :

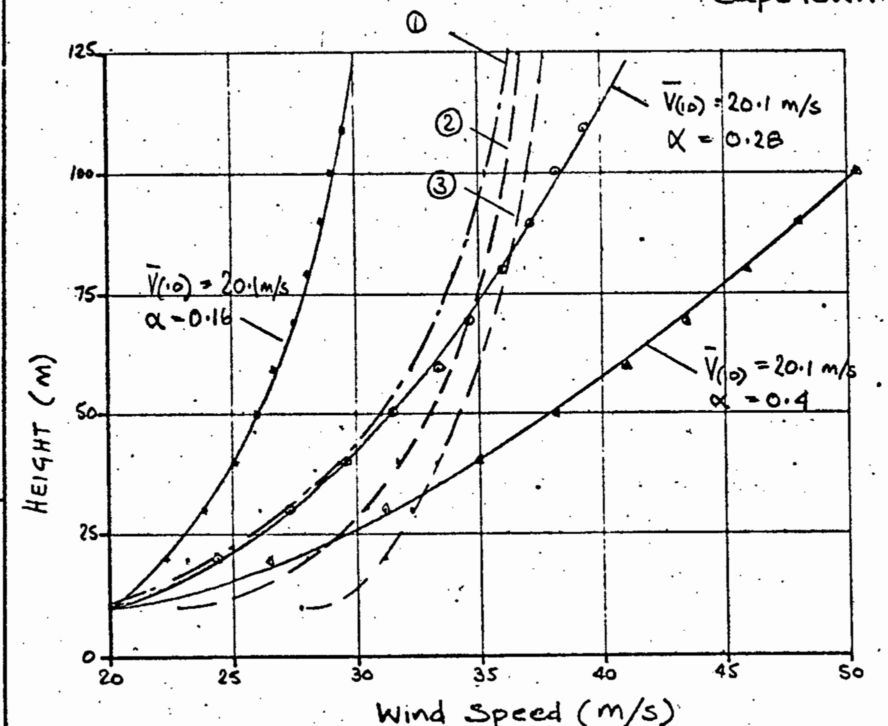
$$\hat{V}_{2s} = \bar{V}(10)\left[\left(x_3/10\right)^{0.16} + 0.6\right]$$



Comparison of the Power Law and the Gust Increment for the Variation of Gust Speed With Height

Fig. 23

Mean Hourly Wind speed and Maximum gust speed for 50 year return period at D.F. Malan Airport, Cape Town

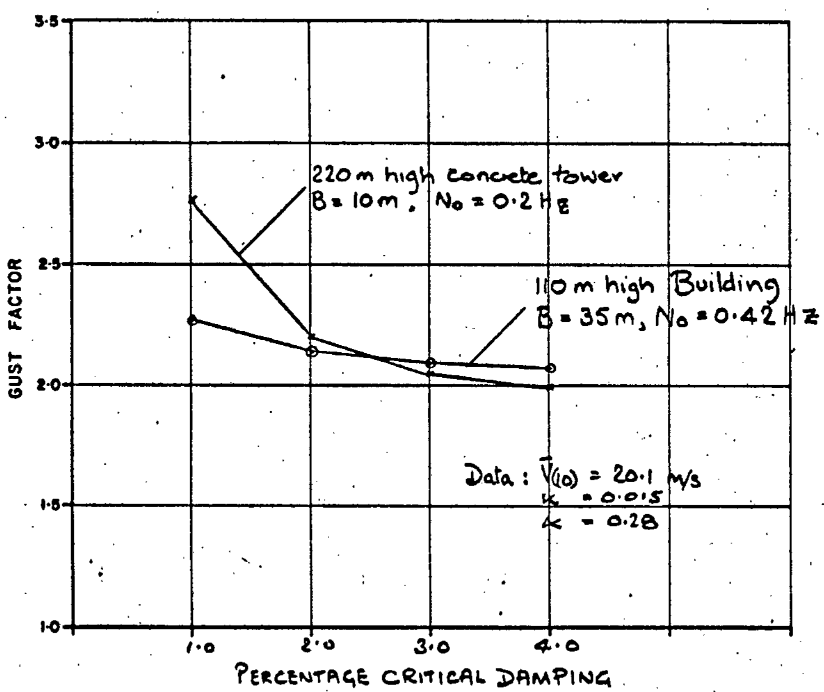


Key: ① City Centre
② Woodland/suburbs
③ Open grassland

C.P.3: Chap V: 1970 | 2 second gust speed $\hat{V}_{2s} = 33$ m/s

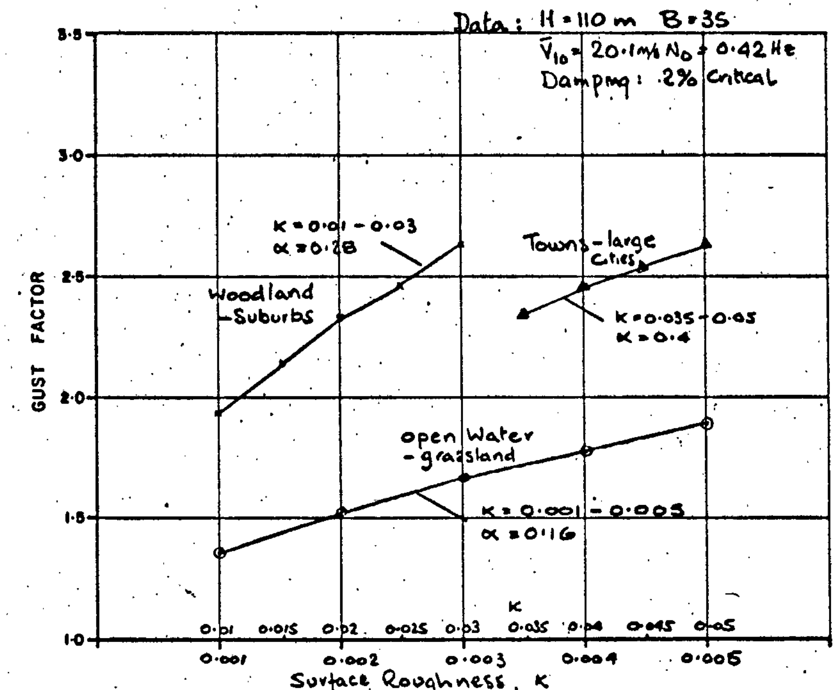
Comparison of Mean Hourly Wind Speed Profile and Gust Speed Profile from C.P.3: Chap V

Fig. 24



GUST FACTOR VARIATION WITH INCREASING DAMPING

Fig. 20



Variation of Gust Factor with terrain

Fig. 22

The two approaches have been plotted in fig. 23 and the constant gust increment results in velocities which are about 5% greater than the power index variation of 0.085.

Fig. 24 shows the variation with height of both the mean wind speed using Davenport's power law indices and the gust wind speed variation based on the indices given by the code for various different terrains (see Table 1).

The Code, C.P. 3 : Chap. V : , also allows the profile to begin at the average roof height which reduces the maximum gust speed slightly.

This, however, has not been done for the mean wind variation with height in this study.

At this stage, in order to look at the differences in response between the statistical approach and the conventional method, a quasi-static gust factor is defined equal to the ratio of the maximum response (based on the gust speed) to the mean response.

Now, for a uniform rectangular building, the moment at the base can be written :

$$M = 0.613\, B\, C_D\, V_H^2\, H^2 \int_0^1 \left(\frac{x}{H}\right)^{2\alpha} d\left(\frac{x}{H}\right)$$

$$= \frac{0.613\, B\, C_D\, V_H^2\, H^2}{2(1+\alpha)}$$

where B is the breadth

C_D is the drag co-efficient

V_H is the velocity at top of the building

H is the height

and α is the power law index.

Taking, as before, the 23 storey office building with the following parameters :

$H = 110$ m, $B = 35$ m, $C_D = 1.4$, $N_o = 0.42$ Hz

and values for the wind structure from table .

Then, if, for C.P. 3, Chap. V, Table 3 the terrain around D.F. Malan Airport is taken as category 2, at Jan Smuts Airport taken as category 3 and both are Class C structures, the response can be compared as follows :-

	D.F. Malan Airport	Jan Smuts Airport
Velocity Gust factor	1.64	2.21
Mean Moment, $\bar{M}$	1.4×10^8 N.M.	2.05×10^8 N.M.
Max. Moment, $\hat{M}$ (2s gust value)	2.25×10^8 N.M.	3.66×10^8 N.M.
Quasi-static Gust Factor, $\frac{\hat{M}}{\bar{M}}$	1.61	1.78
Statistical Gust Response Factor (see fig. 22)	1.84	2.32
Davenport's Gust Response Factor (see reference 6)	1.96	2.44

The following observations can be made about this comparison :

(i) The quasi-static gust factor is significantly less than the statistical. This means that for a "once in fifty year" probability the maximum wind moment would be underestimated by between 23 and 25 percent using C.P. 3 Chap. V.

(ii) The quasi-static gust factor is approximately the same for both sites despite the difference in the velocity gust factors. Clearly, this is unrealistic.

(iii) A small error in estimating the maximum gust speed results in a large difference in the moment. For example, assuming a gust speed of 45 m/s instead of 43 m/s increases the maximum moment by 20% and the quasi-static gust factor to 1.9.

(iv) It can be seen from fig. 22 that taking the terrain roughness as 0.025 instead of 0.02 at Jan Smuts, results in increasing the statistical gust factor to 2.47.

This would be equivalent to assuming :-

$$\sigma_{v_1} = 2{\cdot}58 \times 0{\cdot}025^{1/2} \times 19{\cdot}4 = 7{\cdot}9 \text{ m/s}$$

or

$$V_{2s} = 19{\cdot}4 + 3{\cdot}3 \times 7{\cdot}9 = 45{\cdot}5 \text{ m/s}$$

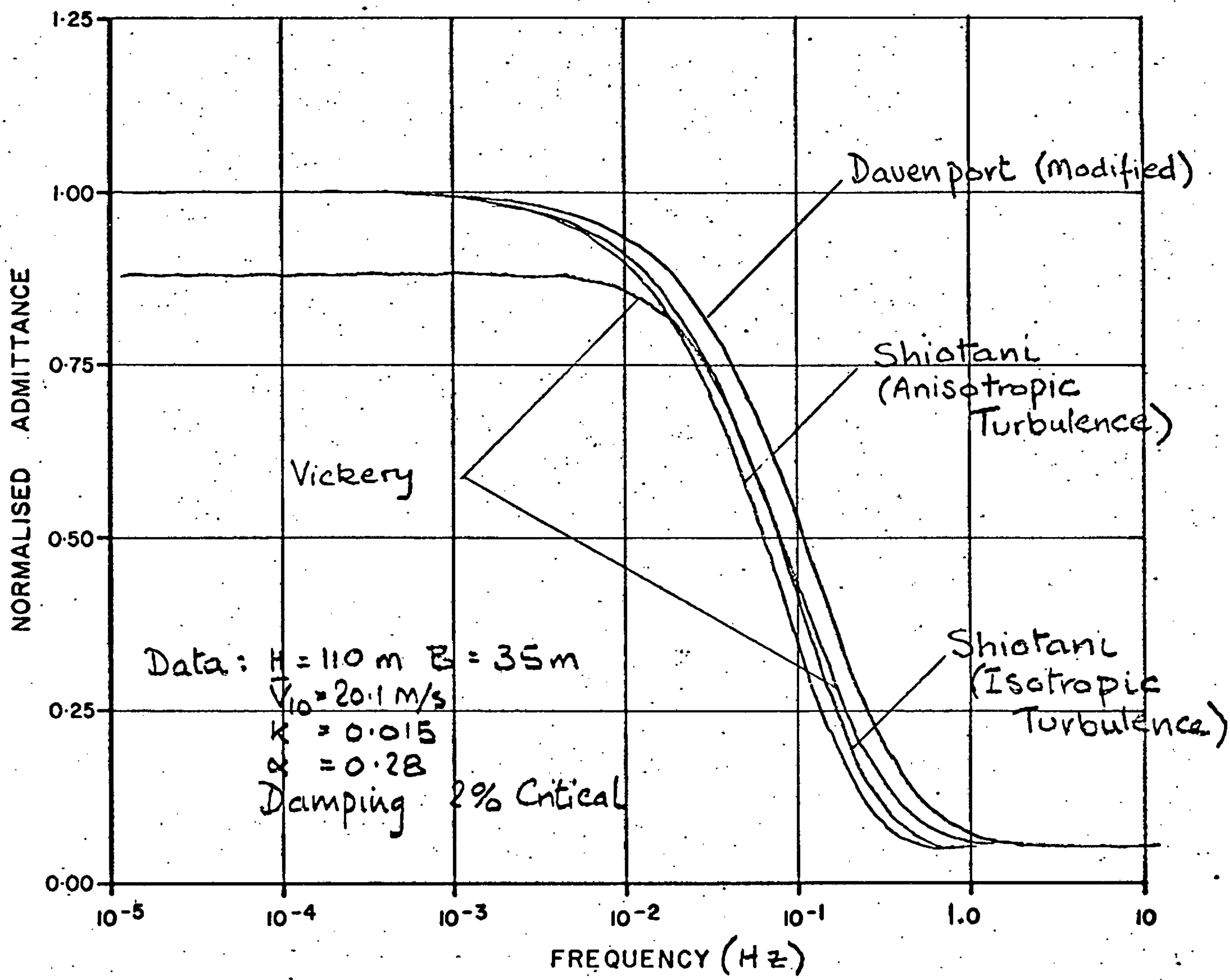
(v) Davenport's simplified gust factors have been calculated for comparison from Tables 4-8 in Reference 6. The discrepancy between the two figures results from using different coherence functions as can be seen in fig. 25.

It is to be expected (given the results in Table 4) that Davenport's method, which is based on a straight line mode shape for the fundamental mode, would yield closely similar results. However, the correspondence can not be extended beyond the case of simple cantilever type structures. Davenport's method can, therefore, be used for this type of structure only and is not applicable where :

(a) The shape or the stiffness of the structure alters with height.

(b) The mode shape differs significantly from a straight line.

59./......



Comparison of Published Coherence Functions

Fig. 25

(c) Several modes contribute to the response.

Finally, comparing the intensities of turbulence with intensities of response for the two sites :

	D.F. Malan	Jan Smuts
Intensity of Turbulence $\sigma_1/\overline{V}(10)$	0.19	0.37
Mean Response, $\overline{Q}1$ (Mode 1)	0.014 m	0.023 m
Standard deviation of response :	0.0031 m	0.0081 m
Intensity of response	0.22	0.35
Max. Response, Qmax	0.025 m	0.054 m

It can be seen that the maximum response figures for identical buildings on the two sites with similar mean wind speeds differ by a factor of two purely as a result of the nature of the surrounding terrain.

These figures differ by roughly the same proportion as the intensities of turbulence which correspond approximately to the response intensities.

6.7. THE COHERENCE FUNCTIONS :

The effect of using the different coherence functions given in Table 3 for a 110m office building and a 220m concrete chimney shield has been investigated. The details of each are as follows :

110 m Office Building

H = 110 m, B = 35 m

$\overline{V}(10)$ = 20.1 m/s, K = 0.015, α = 0.28

Damping : 2% critical.

<u>220 m Chimney Shield.</u>

H = 220 m, B = 10 m

$\overline{V}(10)$ = 20.1 m/s, K = 0.015, α = 0.28

Damping : 2% critical.

It can be seen that significant differences can result from the choice of the coherence function and the effect on the response is given in Table 6.

Using Davenport's coherence function which does not include a phase relationship results in a 8.5 and 16% respectively overestimation of the gust factor, for the two structures. (Assuming Shiotani's results to be more applicable) There is good agreement, however, between Shiotani's value for anisotropic turbulence and Vickery's value.

The normalized admittance functions have been plotted for comparison in Fig. 25 (Office Building only). It may be seen that Vickery's coherence function becomes asymtotic to 0.882 instead of 1.00 for decreasing frequency. This is strange because it implies that correlation is not total, no matter how long the period of the gusts. Experiments on flat plates carried out by Bearman (7) confirm that the function must be asymtotic to 1.00.

TABLE 6

COMPARISON OF COHERENCE FUNCTIONS.

110 m High Office Building

	Shiotani Anisotropic	Shiotani Isotropic	Davenport	Vickery
Standard Deviation	.00704	0.00752	0.00802	.00707
Extreme Distribution Factor, g	3.751	3.795	3.818	3.765
Gust Response Factor (1st Mode only)	2.141	2.233	2.325	2.150

Chimney Shield

	Shiotani Anisotropic	Shiotani Isotropic	Davenport	Vickery
Standard Deviation	.0139	0.0150	0.0179	0.0142
Extreme Distribution Factor, g	3.714	3.722	3.737	3.719
Gust Response Factor (1st Mode only)	2.205	2.299	2.556	2.226

CHAPTER 7

SOME EXAMPLES OF THE STATISTICAL APPROACH

The application of the statistical approach to wind loading has been illustrated using large bluff buildings whose size in relation to the gusts is an important consideration.

They represent one end of the scale. At the other end, there are structures, inherently wind sensitive, but small enough to be excited by a much larger range of gusts.

The results of the analyses of three such structures are given in this chapter. Although very different, they have remarkably similar mode shapes and dynamic characteristics.

All are steel structures as opposed to the concrete of the previous chapter and the natural frequency analyses (details of which will not be given) were undertaken using the standard lumped mass approximations. They come from the writer's experience, were designed using static loadings, have been built and, in the absence of any reports to the contrary, are assumed to be functioning satisfactorily.

For the purposes, of comparison, the mean moments, the maximum gust moments (based on C.P. 3 Chap. V) and the maximum expected moment based on a 50 year return period using the statistical approach are given as well as the spectral analysis in each case. In addition, although further modes were considered, only the results for the first mode are given.

7.1. GROOTE SCHUUR HOSPITAL CHIMNEY :

Because of uncertainty over the nature of the wind structure in the area of the hospital, two different sets of wind parameters are considered and compared.

62./......

Wind Regime 1:

Wind parameters for the nearby D.F. Malan Airport are assumed to apply and no adjustment is made to these parameters resulting from the increased terrain roughness because of anticipated streamlining effects on the slopes of the mountain.

The parameters are :

$$\bar{V}(10) = 20.\ m/s \qquad \alpha = 0.16 \qquad k = 0.005$$

$$\hat{V}_{max} = 33\ m/s \text{ at 10 metres height.}$$

Wind Regime 2 :

Because of the average building height in the hospital confines, a rougher terrain is assumed coupled with a lower mean speed. To be consistent, however, the maximum gust speed is kept the same.

Thus, assuming $\frac{\hat{V}}{\bar{V}} = 2.2$ (see Reference 7a) then : -

$$\bar{V}(10) = \frac{33}{2.2} = 15\ m/s$$

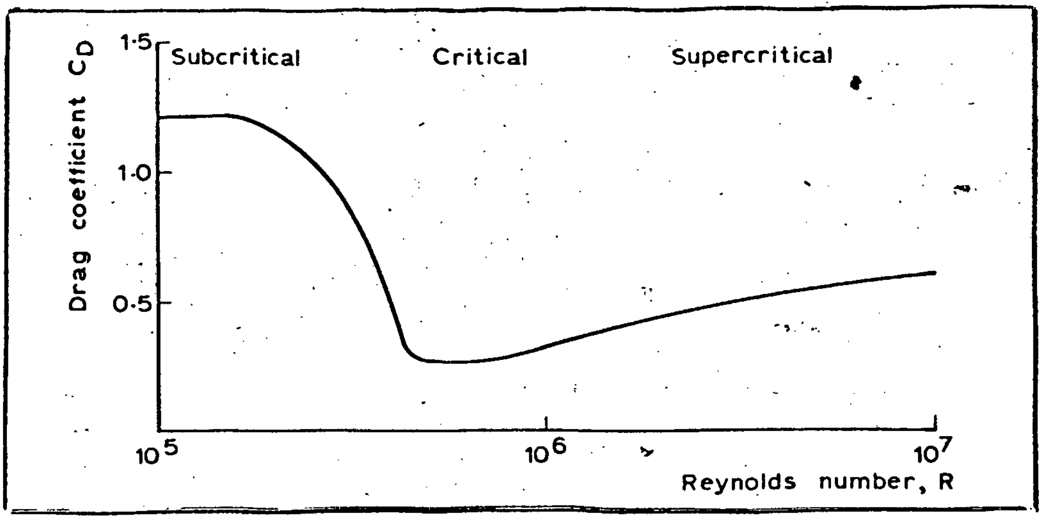
And $\alpha = 0.28$, $k = 0.015$

Details of the structure are given in Fig. 28 and the results listed in Tables 7 and 8.

The drag co-efficient for the circular cross-section of the chimney is taken from fig. 26 with a Reynold's number calculated from :-

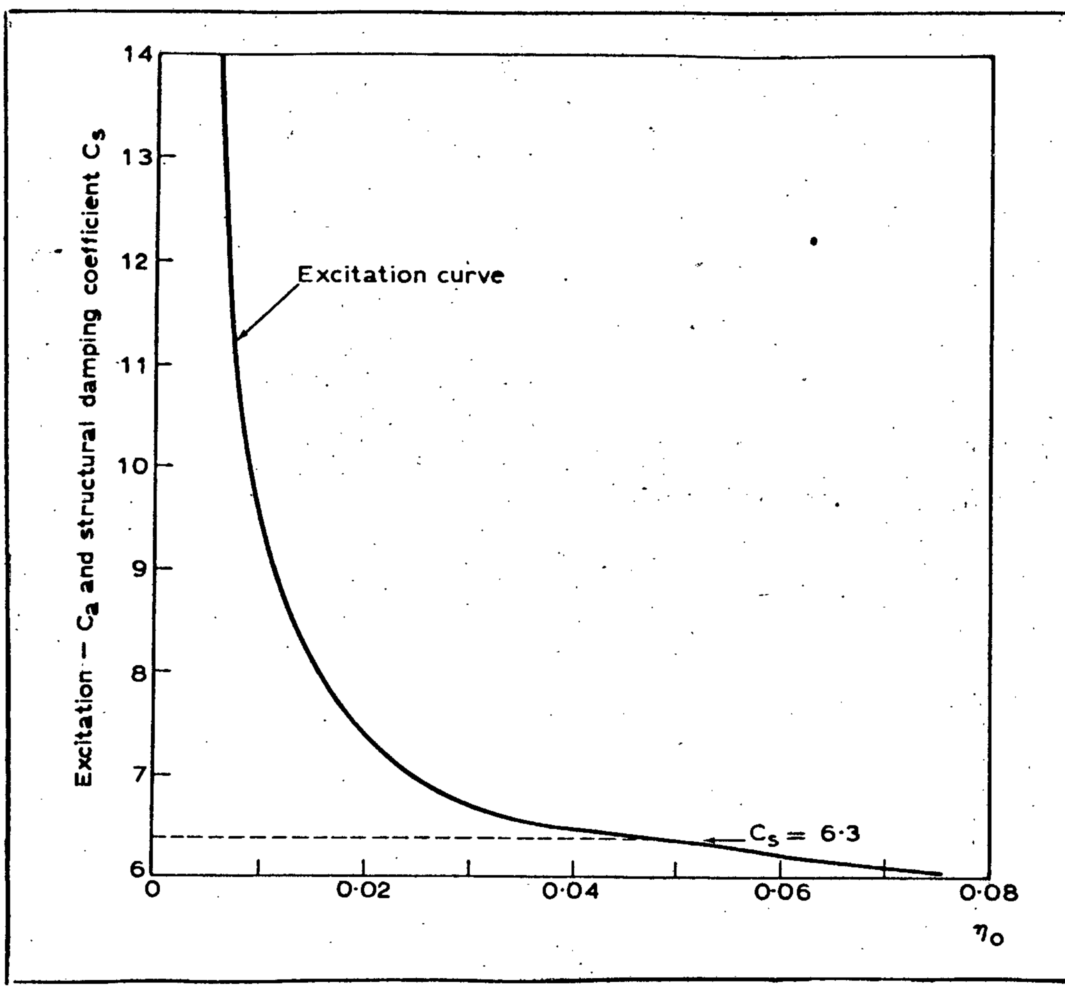
$$R_e = 69000\ VD = 1.04 \times 10^6$$

Thus $C_D = 0.35$



The Variation of Drag Co-efficient with Reynolds Number For Circular Section Structures

Fig. 26



The Amplitude of Vertex Shedding Oscillation

Fig 27.

Circular section structures are prone to vortex shedding excitation and this must be taken into account in the calculation of the maximum response.

The critical vortex shedding frequencies for the structure can be found from :-

Vcrit $\cong$ 5 ND (see Reference 7 : Paper 5)

(N is the natural frequency and D the diameter).

Thus V_1crit = 5.72 m/s

and V_2crit = 26,95 m/s

Following the method for calculating the amplitude of the vortex shedding oscillations in reference 7 (Example 3) :

$$C_S = \frac{2M\delta}{\rho D^2}$$ where M is the mass/ metre

so that C_S = 16.9 and from fig. 27 the R.M.S. amplitude of the oscillation, η_o = 0.01.

The standard deviation of the displacement due to vortex shedding can be calculated from :

$$\sigma_{Q\,vortex} = \eta_o D$$
$$= 0.01 \times 0.76 = 0.0076 \text{ m}$$

Since the response is likely to be sinusoidal, the maximum amplitude is :

$$QVortex = 2\sqrt{2} \times .0076 = 0.02 \text{ m}$$

So that the maximum displacement at the tip of the chimney is the vector sum of the along-wind and the across-wind components.

Assuming only the first three modes are significant :

From Table 7, $\bar{Q}max$ = 0.038

$$Qtotal = \sqrt{0.02^2 + 0.033^2} = 0.042 \text{ m}$$

or 42 mm

64./......

To calculate an approximate gust response factor using Davenport's Figs. 4 - 8 in Reference 6 :

$$g = 1 + g\gamma\sqrt{B + R} \qquad \text{and} \qquad R = \frac{SF}{\beta}$$

where g is the peak factor from Fig. 4 = 4.2

γ is the roughness factor from fig. 5,

for Regime 1, γ = 0.24

for Regime 2, γ = 0.42

B is the background excitation from fig. 6 = 1.6

S is the size reduction from fig. 7 = 0.05

F is the gust energy ratio from fig. 8 = 0.15

and β is the percentage critical damping = 0.0076

Therefore, for regime 1 : $G = 1 + 4.2 \times .24\sqrt{1.6 + \frac{0.15 \times .05}{0.0076}} = 2.61$

and for regime 2 : $G = 1 + 4.2 \times .42\sqrt{1.6 + \frac{0.15 \times .05}{0.0076}} = 2.82$

The second figure agrees well with that calculated by the programme if the difference in the coherence functions is borne in mind. The first figure is rather less than the programme value. This is because the roughness co-efficient used is at the extreme end of the scale applicable to open country and Davenport's method does not allow this sort of variation.

Nevertheless, it is to be concluded that Davenport's gust factor method is a satisfactory approximation for this type of structure.

A comparison of the moments in Tables 7 and 8 shows the maximum expected moment to be 47% and 26% higher than the maximum gust moments calculated from C.P. 3 Chap. V for regimes 1 and 2 respectively.

A general comment that can be made with regard to all of the results is the negligible effect of the second and higher modes.

7.2. LANTERN LIGHT STRUCTURE FOR ST. LUCIA BAY :

This lattice support structure for a lantern light is made of aluminium. The exposed nature of the site suggests a very low roughness co-efficient and the wind speed data for Durban taken from Reference 9 is assumed to be applicable.

The details are given in Fig. 29 and the results in Table 9.

There is no significant difference in the maximum gust moments calculated from C.P. 3 Chap. V and the maximum expected moment based on the statistical method.

The gust factor from Davenport's method is considerably more inaccurate for this structure which illustrates the strict limitations of the simplified approach.

7.3 132 KV TRANSMISSION LINE SUSPENSION TOWER :

The design of transmission line towers is a task usually only undertaken by specialists in this field.

Designs are used many times over and are, therefore, very economical. As a result, the Electricity Supply Commission and most municipalities usually require that full scale structures be tested to either two and half or one and a half times the specified working load depending on the loading conditions.

Few structural designers have the comfort of this sort of luxury and such tests provide a unique opportunity to learn from and refine designs.

The trouble is that the specified loadings are of a highly arbitrary and irrational nature. They have been fixed from experience at what is regarded as economic level and without any regard to the actual nature of the loads that will be acting on the structure.

Thus the designer is playing a kind of structural blindman's buff and there seems little point in doing expensive tests to specified loads if these are not related to reality.

Briefly, the towers are designed for a wind pressure of 0.717 kPa on 1.5 times the exposed area and the power lines for the same pressure on 0.6 times their exposed area. Even assuming that these wind pressures are acceptable, the loads take insufficient account for drag and lift effects which if calculated from the Standard Building Regulations (21) or Cohen and Perrin (23), could be nearly twice as high.

It must be admitted, however, that normal methods will lead to an over-designed solution because of the size effect in relation to the length of the line span and the dimensions of the gusts.

Clearly, a non-deterministic approach is required which will allow the horizontal correlation effects on the transmission lines to be quantified.

In this example, the details of which are given in fig.30 these horizontal correlation effects have been taken into account by assuming that the mode shape of the transmission lines is a sine function and the total correlation for each frequency is the product of the vertical correlations on the tower and the horizontal correlations on the lines.

The results given in Table 10 show that the reduction allowed by the correlation effects on the lines is not inconsiderable. A gust factor of 1.75 allows a 25% reduction on the gust moments calculated by C.P. 3 Chap. V.

The gust factor of 2.3 for the lantern light gives some indication of what the result would have been, had the horizontal correlation effects not been taken into account.

The maximum moment at the base calculated using the ESCOM specification is still, however, somewhat lower than that found by the statistical theory.

<u>WIND REGIME 1</u>

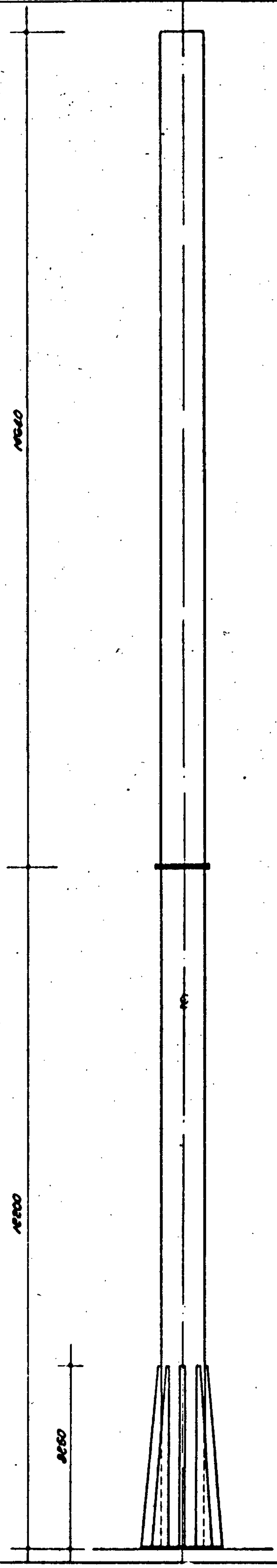
$\overline{V}(10) = 20$ m/s, $\alpha = 0.16$, $k = 0.005$

$\hat{V}$ max = 33 m/s at 10 metres

<u>WIND REGIME 2</u>

$\overline{V}(10) = 15$ m/s, $\alpha = 0.28$, $k = 0.015$

$\hat{V}$ max = 33 m/s at 10 metres.



<u>GROOTE SCHUUR HOSPITAL CHIMNEY</u>

<u>FIG. 28</u>

TABLE 7

THE SPECTRAL ANALYSIS OF GROOTE SCHUUR HOSPITAL CHIMNEY - WIND REGIME 1

MEAN WIND	ROUGHNESS	DAMPING	POWER INDEX	FUND.FREQ
20.000	.005	.050	.160	1.507

LEVEL	MASS/M	DRAG COEFF	BREADTH	MODE SHAPE	GAMMA	MEAN SPEED	GUST SPEED
.0	506.0	1.000	1.200	.000	.000	.000	.000
3.0	506.0	.600	1.200	.008	.004	16.496	26.179
6.0	190.0	.350	.780	.044	.009	18.430	27.768
9.0	190.0	.350	.780	.115	.026	19.666	28.741
12.0	190.0	.350	.780	.214	.050	20.592	29.453
15.0	114.0	.350	.760	.339	.080	21.341	30.017
18.0	114.0	.350	.760	.489	.119	21.972	30.486
21.0	114.0	.350	.760	.654	.163	22.521	30.888
24.0	114.0	.350	.760	.826	.211	23.007	31.240
27.0	114.0	.350	.760	1.000	.260	23.445	31.555

TABLE 7 CONTD.

SPECTRAL VALUES - MODE NUMBER 1

FREQUENCY	AERO.ADMITTANCE	VELOCITY	FORCE	MECH.ADMITTANCE	RESPONSE
.1000-03	.9998+00	.4041-01	.1587+03	.1000+01	.5486-07
.2000-03	.9996+00	.8081-01	.3174+03	.1000+01	.1097-06
.3000-03	.9993+00	.1212+00	.4759+03	.1000+01	.1645-06
.4000-03	.9991+00	.1615+00	.6342+03	.1000+01	.2192-06
.5000-03	.9989+00	.2019+00	.7924+03	.1000+01	.2738-06
.6000-03	.9987+00	.2422+00	.9503+03	.1000+01	.3284-06
.7000-03	.9984+00	.2824+00	.1108+04	.1000+01	.3829-06
.8000-03	.9982+00	.3226+00	.1265+04	.1000+01	.4372-06
.9000-03	.9980+00	.3627+00	.1422+04	.1000+01	.4915-06
.1000-02	.9978+00	.4027+00	.1579+04	.1000+01	.5456-06
.1100-01	.9757+00	.3188+01	.1222+05	.9999+00	.4224-05
.2100-01	.9541+00	.3613+01	.1355+05	.9996+00	.4683-05
.3100-01	.9331+00	.3336+01	.1223+05	.9992+00	.4231-05
.4100-01	.9126+00	.2989+01	.1072+05	.9985+00	.3709-05
.5100-01	.8926+00	.2686+01	.9420+04	.9977+00	.3263-05
.6100-01	.8731+00	.2437+01	.8359+04	.9967+00	.2398-05
.7100-01	.8541+00	.2232+01	.7492+04	.9956+00	.2601-05
.8100-01	.8356+00	.2063+01	.6775+04	.9942+00	.2355-05
.9100-01	.8177+00	.1921+01	.6173+04	.9927+00	.2149-05
.1100+00	.7849+00	.1706+01	.5262+04	.9894+00	.1836-05
.2100+00	.6393+00	.1122+01	.2819+04	.9615+00	.1013-05
.3100+00	.5315+00	.8678+00	.1813+04	.9172+00	.6829-06
.4100+00	.4519+00	.7209+00	.1280+04	.8575+00	.5159-06
.5100+00	.3925+00	.6236+00	.9616+03	.7841+00	.4238-06
.6100+00	.3475+00	.5535+00	.7559+03	.6992+00	.3736-06
.7100+00	.3131+00	.5003+00	.6156+03	.6054+00	.3514-06
.8100+00	.2864+00	.4583+00	.5157+03	.5057+00	.3524-06
.9100+00	.2653+00	.4241+00	.4422+03	.4038+00	.3785-06
.1010+01	.2487+00	.3956+00	.3866+03	.3035+00	.4401-06
.1220+01	.2255+00	.3527+00	.3126+03	.1341+00	.8057-06
.2200+01	.1847+00	.2355+00	.1709+03	.1280+01	.4614-07
.3200+01	.1831+00	.1834+00	.1320+03	.1231+02	.3703-08
.4200+01	.1859+00	.1530+00	.1118+03	.4580+02	.8433-09
.5200+01	.1873+00	.1327+00	.9771+02	.1190+03	.2838-09
.6200+01	.1878+00	.1180+00	.8708+02	.2536+03	.1186-09
.7200+01	.1878+00	.1068+00	.7884+02	.4764+03	.5719-10
.8200+01	.1878+00	.9796-01	.7228+02	.8184+03	.3052-10
.9200+01	.1878+00	.9073-01	.6694+02	.1315+04	.1759-10
.1020+02	.1878+00	.8470-01	.6249+02	.2008+04	.1075-10
.1120+02	.1878+00	.7958-01	.5871+02	.2941+04	.6898-11

MODAL RESPONSE PARAMETERS:

VARIANCE	STD.DEV	G FACTOR	MEAN RESP	MAX.RESPONSE
.3193-04	.5650-02	.4220+01	.1292-01	.3677-01

GUST FACTOR = 2.845

DAVENPORT'S APPROXIMATE GUST FACTOR = 2.61

TABLE 7 CONTD.

LEVEL	MEAN MOMENT	GUST MOMENT	MAX.MOMENT
.0	.3268+05	.6321+05	.9322+05
3.0	.2616+05	.4979+05	.7584+05
6.0	.2073+05	.3910+05	.5983+05
9.0	.1580+05	.2957+05	.4485+05
12.0	.1146+05	.2129+05	.3171+05
15.0	.7754+04	.1431+05	.2104+05
18.0	.4718+04	.8648+04	.1245+05
21.0	.2390+04	.4355+04	.6091+04
24.0	.8066+03	.1461+04	.1979+04
27.0	.0000	.0000	.0000

MAXIMUM RESPONSE (FIRST THREE MODES INCLUDED)

= 0.038 m

GUST RESPONSE FACTOR (FIRST THREE MODES INCLUDED)

= 2.85

TABLE 8

THE SPECTRAL ANALYSIS OF GROOTE SCHUUR HOSPITAL CHIMNEY - WIND REGIME 2.

MEAN WIND	ROUGHNESS	DAMPING	POWER INDEX	FUND.FREQ
15.000	.015	.050	.280	1.507

LEVEL	MASS/M	DRAG COEFF	BREADTH	MODE SHAPE	GAMMA	MEAN SPEED	GUST SPEED
.0	506.0	1.000	1.200	.000	.000	.000	.000
3.0	506.0	.600	1.200	.008	.003	10.707	26.022
6.0	190.0	.350	.780	.044	.009	13.001	27.697
9.0	190.0	.350	.780	.115	.025	14.564	28.726
12.0	190.0	.350	.780	.214	.051	15.786	29.480
15.0	114.0	.350	.760	.339	.084	16.803	30.078
18.0	114.0	.350	.760	.489	.128	17.683	30.575
21.0	114.0	.350	.760	.654	.178	18.463	31.003
24.0	114.0	.350	.760	.826	.234	19.167	31.377
27.0	114.0	.350	.760	1.000	.293	19.809	31.712

SPECTRAL VALUES - MODE NUMBER 1

FREQUENCY	AERO.ADMITTANCE	VELOCITY	FORCE	MECH.ADMITTANCE	RESPONSE
.1000-03	.9997+00	.9091-01	.2345+03	.1000+01	.8105-07
.2000-03	.9994+00	.1818+00	.4688+03	.1000+01	.1620-06
.3000-03	.9991+00	.2726+00	.7028+03	.1000+01	.2429-06
.4000-03	.9988+00	.3633+00	.9364+03	.1000+01	.3236-06
.5000-03	.9985+00	.4539+00	.1170+04	.1000+01	.4042-06
.6000-03	.9982+00	.5443+00	.1402+04	.1000+01	.4845-06
.7000-03	.9980+00	.6346+00	.1634+04	.1000+01	.5647-06
.8000-03	.9977+00	.7246+00	.1865+04	.1000+01	.6446-06
.9000-03	.9974+00	.8143+00	.2096+04	.1000+01	.7242-06
.1000-02	.9971+00	.9038+00	.2325+04	.1000+01	.8036-06
.1100-01	.9684+00	.5933+01	.1483+05	.9999+00	.5124-05
.2100-01	.9406+00	.5803+01	.1408+05	.9996+00	.4869-05
.3100-01	.9137+00	.5025+01	.1185+05	.9992+00	.4097-05
.4100-01	.8877+00	.4368+01	.1001+05	.9985+00	.3463-05
.5100-01	.8625+00	.3864+01	.8599+04	.9977+00	.2979-05
.6100-01	.8382+00	.3473+01	.7513+04	.9967+00	.2605-05
.7100-01	.8148+00	.3164+01	.6652+04	.9956+00	.2309-05
.8100-01	.7922+00	.2913+01	.5955+04	.9942+00	.2070-05
.9100-01	.7705+00	.2705+01	.5378+04	.9927+00	.1872-05
.1100+00	.7313+00	.2394+01	.4518+04	.9894+00	.1578-05
.2100+00	.5670+00	.1567+01	.2292+04	.9615+00	.8237-06
.3100+00	.4564+00	.1210+01	.1425+04	.9172+00	.5369-06
.4100+00	.3810+00	.1005+01	.9877+03	.8575+00	.3981-06
.5100+00	.3285+00	.8689+00	.7365+03	.7841+00	.3246-06
.6100+00	.2911+00	.7713+00	.5793+03	.6992+00	.2863-06
.7100+00	.2638+00	.6971+00	.4746+03	.6054+00	.2709-06
.8100+00	.2437+00	.6385+00	.4016+03	.5057+00	.2744-06
.9100+00	.2287+00	.5908+00	.3486+03	.4038+00	.2984-06
.1010+01	.2173+00	.5512+00	.3091+03	.3035+00	.3519-06
.1200+01	.2025+00	.4914+00	.2571+03	.1341+00	.6626-06
.2200+01	.1860+00	.3280+00	.1575+03	.1280+01	.4251-07
.3200+01	.1896+00	.2555+00	.1250+03	.1231+02	.3509-08
.4200+01	.1912+00	.2132+00	.1052+03	.4580+02	.7937-09
.5200+01	.1914+00	.1849+00	.9133+02	.1190+03	.2653-09
.6200+01	.1914+00	.1644+00	.8121+02	.2536+03	.1106-09
.7200+01	.1914+00	.1488+00	.7350+02	.4764+03	.5332-10
.8200+01	.1914+00	.1365+00	.6740+02	.8184+03	.2846-10
.9200+01	.1914+00	.1264+00	.6242+02	.1315+04	.1640-10
.1020+02	.1914+00	.1180+00	.5827+02	.2008+04	.1003-10
.1120+02	.1914+00	.1109+00	.5475+02	.2941+04	.6432-11

MODAL RESPONSE PARAMETERS:

VARIANCE	STD.DEV	G FACTOR	MEAN RESP	MAX.RESPONSE
.3097-04	.5566-02	.4205+01	.8532-02	.3194-01

GUST FACTOR = 3.744

DAVENPORT'S APPROXIMATE GUST FACTOR = 2.82

TABLE 8 CONTD.

LEVEL	MEAN MOMENT	GUST MOMENT	MAX.MOMENT
.0	.2125+05	.6355+05	.8064+05
3.0	.1735+05	.5012+05	.6615+05
6.0	.1392+05	.3938+05	.5239+05
9.0	.1073+05	.2980+05	.3938+05
12.0	.7869+04	.2146+05	.2790+05
15.0	.5380+04	.1443+05	.1857+05
18.0	.3306+04	.8728+04	.1101+05
21.0	.1691+04	.4396+04	.5397+04
24.0	.5759+03	.1476+04	.1756+04
27.0	.0000	.0000	.0000

MAXIMUM RESPONSE (FIRST THREE MODES INCLUDED)

= 0.0325

GUST RESPONSE FACTOR (FIRST THREE MODES INCLUDED)

= 3.75

WIND PARAMETERS

$\bar{V}(10) = 24.8$ m/s, $\alpha = 0.16$, $k = 0.002$

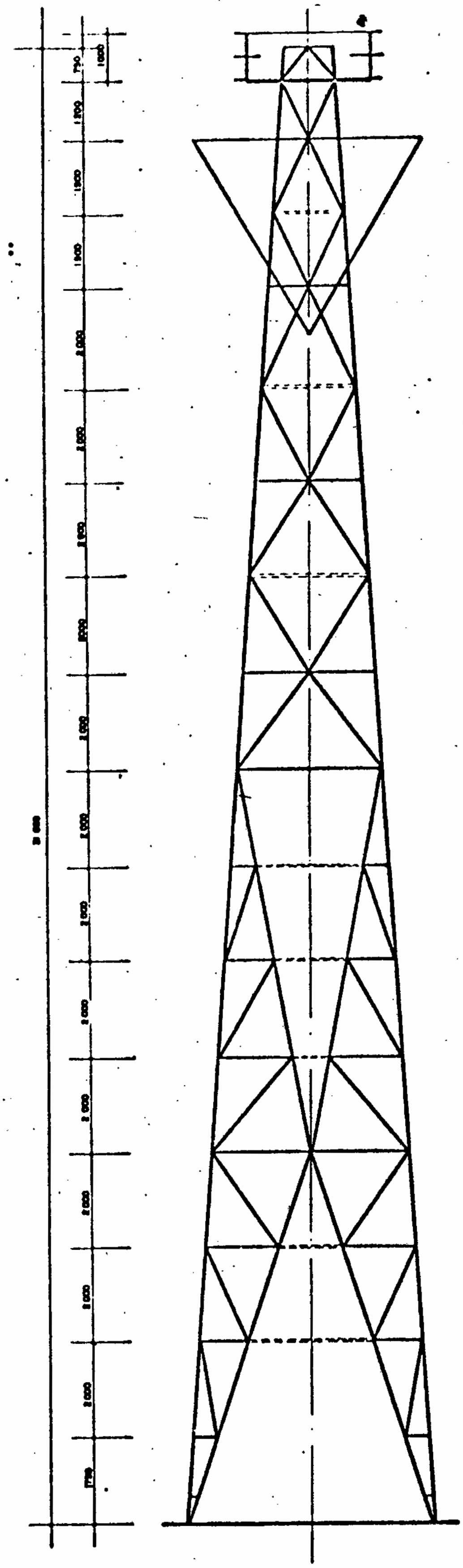
$\hat{V} = 38$ m/s at 10 metres

For calculation of Drag Co-efficients :
(See reference 23)

$C_D = 3.65 - 4.65\,\emptyset$

where $\emptyset$ = solidity ratio

$= \dfrac{\text{Projected area}}{\text{Enclosed Area}}$



LANTERN LIGHT STRUCTURE FOR ST. LUCIA BAY

FIG. 29

TABLE 9

THE SPECTRAL ANALYSIS OF ST. LUCIA BAY LANTERN LIGHT STRUCTURE

MEAN WIND	ROUGHNESS	DAMPING	POWER INDEX	FUND.FREQ
24.000	.002	.060	.160	1.100

LEVEL	MASS/M	DRAG COEFF	BREADTH*	MODE SHAPE	GAMMA	MEAN SPEED	GUST SPEED
.0	54.0	1.500	1.000	.000	.000	.000	.000
2.0	54.0	1.800	1.180	.001	.001	19.170	33.409
4.0	60.0	2.500	1.140	.002	.003	21.418	35.314
6.0	75.0	2.900	1.060	.007	.013	22.854	36.478
8.0	78.0	2.900	1.030	.020	.039	23.930	37.328
10.0	75.0	2.600	1.060	.050	.092	24.800	38.000
12.0	60.0	2.300	1.080	.100	.171	25.534	38.558
14.0	50.0	2.100	1.110	.200	.328	26.172	39.037
16.0	45.0	1.900	1.080	.300	.442	26.737	39.456
18.0	40.0	2.000	1.030	.400	.604	27.246	39.830
20.0	40.0	1.800	1.030	.450	.621	27.709	40.167
22.0	40.0	1.700	1.030	.600	.795	28.135	40.474
24.0	30.0	1.400	1.030	.700	.774	28.529	40.757
26.0	30.0	1.200	1.030	.800	.768	28.897	41.019
28.0	50.0	.870	1.030	.900	.634	29.241	41.263
30.0	50.0	.960	1.030	1.000	.786	29.566	41.491

* PROJECTED AREA/METRE OF HEIGHT.

TABLE 9 CONTD.

SPECTRAL VALUES - MODE NUMBER 1

FREQUENCY	AERO.ADMITTANCE	VELOCITY	FORCE	MECH.ADMITTANCE	RESPONSE
.1000-03	.9998+00	.2004-01	.5077+04	.1000+01	.3976-04
.2000-03	.9996+00	.4008-01	.1015+05	.1000+01	.7950-04
.3000-03	.9994+00	.6012-01	.1522+05	.1000+01	.1192-03
.4000-03	.9993+00	.8014-01	.2029+05	.1000+01	.1589-03
.5000-03	.9991+00	.1002+00	.2535+05	.1000+01	.1986-03
.6000-03	.9989+00	.1202+00	.3041+05	.1000+01	.2382-03
.7000-03	.9987+00	.1401+00	.3546+05	.1000+01	.2777-03
.8000-03	.9985+00	.1601+00	.4051+05	.1000+01	.3172-03
.9000-03	.9984+00	.1801+00	.4554+05	.1000+01	.3567-03
.1000-02	.9982+00	.2000+00	.5057+05	.1000+01	.3961-03
.1100-01	.9800+00	.1751+01	.4347+06	.9998+00	.3405-02
.2100-01	.9622+00	.2214+01	.5397+06	.9993+00	.4230-02
.3100-01	.9447+00	.2171+01	.5197+06	.9984+00	.4077-02
.4100-01	.9275+00	.2007+01	.4716+06	.9972+00	.3704-02
.5100-01	.9107+00	.1836+01	.4235+06	.9957+00	.3331-02
.6100-01	.8942+00	.1683+01	.3812+06	.9939+00	.3004-02
.7100-01	.8780+00	.1552+01	.3453+06	.9917+00	.2727-02
.8100-01	.8621+00	.1441+01	.3148+06	.9892+00	.2493-02
.9100-01	.8466+00	.1346+01	.2888+06	.9864+00	.2293-02
.1100+00	.8180+00	.1200+01	.2488+06	.9801+00	.1988-02
.2100+00	.6855+00	.7947+00	.1380+06	.9284+00	.1164-02
.3100+00	.5804+00	.6153+00	.9048+05	.8475+00	.8362-03
.4100+00	.4975+00	.5114+00	.6446+05	.7415+00	.6809-03
.5100+00	.4322+00	.4425+00	.4845+05	.6164+00	.6156-03
.6100+00	.3804+00	.3928+00	.3786+05	.4796+00	.6182-03
.7100+00	.3390+00	.3551+00	.3050+05	.3405+00	.7015-03
.8100+00	.3056+00	.3253+00	.2519+05	.2098+00	.9405-03
.9100+00	.2785+00	.3010+00	.2124+05	.9987-01	.1665-02
.1010+01	.2561+00	.2808+00	.1822+05	.2494-01	.5722-02
.1200+01	.2234+00	.2504+00	.1417+05	.3657-01	.3035-02
.2200+01	.1487+00	.1672+00	.6296+04	.9001+01	.5478-05
.3200+01	.1284+00	.1302+00	.4237+04	.5570+02	.5959-06
.4200+01	.1226+00	.1086+00	.3375+04	.1844+03	.1434-06
.5200+01	.1217+00	.9422-01	.2905+04	.4557+03	.4993-07
.6200+01	.1224+00	.8379-01	.2598+04	.9467+03	.2150-07
.7200+01	.1234+00	.7584-01	.2371+04	.1751+04	.1061-07
.8200+01	.1242+00	.6954-01	.2189+04	.2978+04	.5757-08
.9200+01	.1247+00	.6441-01	.2036+04	.4754+04	.3353-08
.1020+02	.1250+00	.6013-01	.1904+04	.7222+04	.2065-08
.1120+02	.1251+00	.5649-01	.1791+04	.1054+05	.1331-08

MODAL RESPONSE PARAMETERS:

VARIANCE	STD.DEV	G FACTOR	MEAN RESP	MAX.RESPONSE
.3372-01	.1836+00	.4167+01	.6215+00	.1387+01

GUST FACTOR = 2.231

DAVENPORT'S APPROXIMATE GUST FACTOR = 2.8

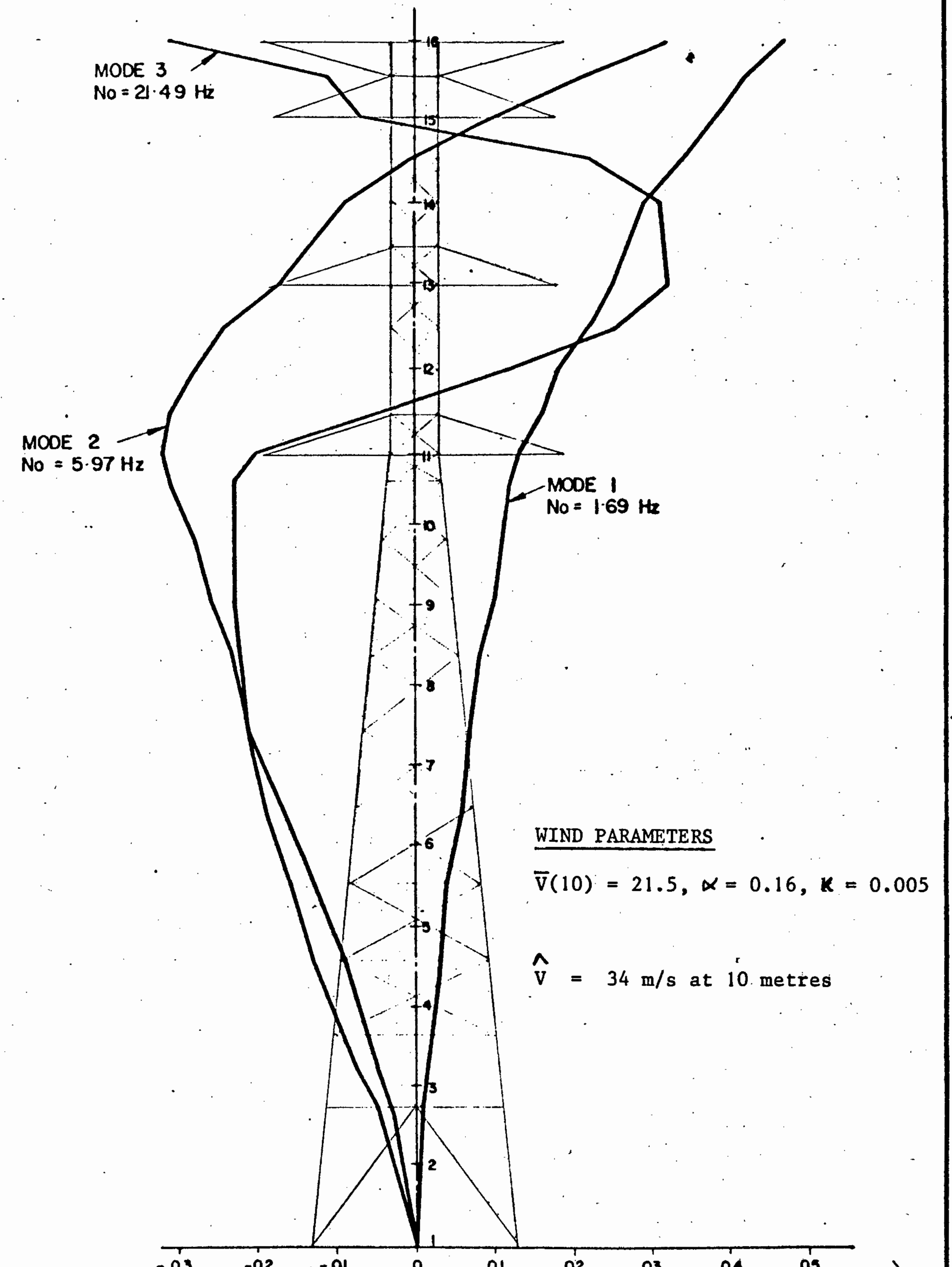
LEVEL	MEAN MOMENT	GUST MOMENT	MAX.MOMENT
.0	.3695+06	.7961+06	.8355+06
2.0	.3205+06	.6845+06	.7532+06
4.0	.2734+06	.5787+06	.6728+06
6.0	.2295+06	.4817+06	.5954+06
8.0	.1896+06	.3948+06	.5217+06
10.0	.1539+06	.3182+06	.4516+06
12.0	.1224+06	.2513+06	.3847+06
14.0	.9484+05	.1935+06	.3210+06
16.0	.7120+05	.1444+06	.2607+06
18.0	.5115+05	.1032+06	.2042+06
20.0	.3486+05	.6991+05	.1526+06
22.0	.2205+05	.4399+05	.1069+06
24.0	.1265+05	.2510+05	.6860+05
26.0	.6118+04	.1209+05	.3711+05
28.0	.2119+04	.4174+04	.1293+05
30.0	.0000	.0000	.0000

MAXIMUM RESPONSE (FIRST THREE MODES INCLUDED)

= 1.4 m

GUST FACTOR (FIRST THREE MODES INCLUDED)

= 2.30



182 KV TRANSMISSION LINE SUSPENSION TOWER

FIG. 30

TABLE 10

THE SPECTRAL ANALYSIS OF 132 KV TRANSMISSION LINE TOWER

****

MEAN WIND	ROUGHNESS	DAMPING	POWER INDEX	FUND.FREQ
21.500	.005	.060	.160	1.690

LEVEL	MASS/M	DRAG COEFF	BREADTH	MODE SHAPE	GAMMA	MEAN SPEED	GUST SPEED		
.0	105.0	2.500	.500	.000	.000	.000	.000	.000	.000
2.1	100.0	2.500	.500	.010	.008	16.749	30.039	.105	.000
4.2	95.0	2.500	.500	.020	.017	18.714	31.862	.208	.000
6.3	90.0	2.500	.500	.050	.046	19.968	32.979	.309	.000
8.4	80.0	2.700	.500	.070	.074	20.909	33.795	.407	.000
10.5	75.0	2.700	.500	.100	.109	21.668	34.443	.500	.000
12.6	70.0	2.700	.500	.130	.146	22.310	34.980	.588	.000
14.7	65.0	2.700	.500	.160	.184	22.867	35.442	.669	.000
16.8	60.0	2.700	.500	.190	.223	23.361	35.846	.743	.000
18.9	65.0	3.200	.300	.220	.187	23.805	36.207	.809	.000
21.0	1400.0	3.200	.300	.250	.216	24.210	36.533	.866	1257.522
23.1	65.0	3.200	.300	.350	.307	24.582	36.830	.914	1257.522
25.2	1400.0	3.200	.300	.500	.445	24.927	37.103	.951	3923.666
27.3	65.0	3.200	.300	.550	.496	25.248	37.357	.978	3923.666
29.4	1400.0	3.200	.300	.750	.684	25.549	37.593	.995	8125.103
31.5	270.0	3.200	.300	1.000	.923	25.833	37.814	1.000	11397.656

* PROJECTED AREA/METRE OF HEIGHT.

TABLE 10 CONTD.

SPECTRAL VALUES - MODE NUMBER 1

FREQUENCY	AERO.ADMITTANCE	VELOCITY	FORCE	MECH.ADMITTANCE	RESPONSE
.1000-03	.8360+00	.4344-01	.6026+05	.1000+01	.1181-05
.2000-03	.8296+00	.8687-01	.1196+06	.1000+01	.2344-05
.3000-03	.8233+00	.1303+00	.1780+06	.1000+01	.3488-05
.4000-03	.8170+00	.1737+00	.2355+06	.1000+01	.4615-05
.5000-03	.8109+00	.2170+00	.2920+06	.1000+01	.5723-05
.6000-03	.8047+00	.2604+00	.3477+06	.1000+01	.6814-05
.7000-03	.7987+00	.3036+00	.4024+06	.1000+01	.7807-05
.8000-03	.7927+00	.3469+00	.4563+06	.1000+01	.8942-05
.9000-03	.7868+00	.3900+00	.5093+06	.1000+01	.9980-05
.1000-02	.7810+00	.4331+00	.5613+06	.1000+01	.1100-04
.1100-01	.4219+00	.3559+01	.2491+07	.9999+00	.4883-04
.2100-01	.2747+00	.4188+01	.1909+07	.9997+00	.3742-04
.3100-01	.2005+00	.3942+01	.1311+07	.9993+00	.2572-04
.4100-01	.1574+00	.3565+01	.9313+06	.9988+00	.1827-04
.5100-01	.1299+00	.3221+01	.6941+06	.9982+00	.1363-04
.6100-01	.1110+00	.2931+01	.5400+06	.9974+00	.1061-04
.7100-01	.9744-01	.2691+01	.4351+06	.9965+00	.8558-05
.8100-01	.8729-01	.2491+01	.3608+06	.9954+00	.7103-05
.9100-01	.7946-01	.2321+01	.3061+06	.9942+00	.6034-05
.1100+00	.6870-01	.2064+01	.2353+06	.9915+00	.4650-05
.2100+00	.4473-01	.1360+01	.1009+06	.9694+00	.2041-05
.3100+00	.3507-01	.1052+01	.6123+05	.9338+00	.1285-05
.4100+00	.2906-01	.8741+00	.4215+05	.8858+00	.9326-06
.5100+00	.2485-01	.7561+00	.3118+05	.8262+00	.7395-06
.6100+00	.2175-01	.6712+00	.2422+05	.7565+00	.6276-06
.7100+00	.1939-01	.6067+00	.1952+05	.6782+00	.5641-06
.8100+00	.1754-01	.5558+00	.1618+05	.5934+00	.5344-06
.9100+00	.1607-01	.5143+00	.1371+05	.5043+00	.5329-06
.1010+01	.1486-01	.4798+00	.1183+05	.4134+00	.5610-06
.1200+01	.1311-01	.4277+00	.9303+04	.2460+00	.7411-06
.2200+01	.9145-02	.2856+00	.4334+04	.4831+00	.1758-06
.3200+01	.8191-02	.2225+00	.3024+04	.6685+01	.8864-08
.4200+01	.8025-02	.1856+00	.2471+04	.2680+02	.1808-08
.5200+01	.8082-02	.1610+00	.2159+04	.7170+02	.5900-09
.6200+01	.8170-02	.1431+00	.1941+04	.1552+03	.2450-09
.7200+01	.8231-02	.1296+00	.1770+04	.2942+03	.1179-09
.8200+01	.8262-02	.1168+00	.1629+04	.5082+03	.6281-10
.9200+01	.8274-02	.1100+00	.1511+04	.8200+03	.3611-10
.1020+02	.8277-02	.1027+00	.1411+04	.1255+04	.2203-10
.1120+02	.8277-02	.9651-01	.1326+04	.1842+04	.1410-10

MODAL RESPONSE PARAMETERS:

VARIANCE	STD.DEV	G FACTOR	MEAN RESP	MAX.RESPONSE
.1494-03	.1222-01	.4043+01	.6945-01	.1189+00

GUST FACTOR = 1.711

TABLE 10 CONTD.

LEVEL	MEAN MOMENT	GUST MOMENT	MAX.MOMENT
.0	.7065+06	.1572+07 *	.1361+07 *
2,1	.6424+06	.1426+07	.1251+07
4,2	.5792+06	.1283+07	.1142+07
6,3	.5171+06	.1143+07	.1034+07
8,4	.4565+06	.1006+07	.9269+06
10,5	.3974+06	.8743+06	.8213+06
12,6	.3400+06	.7465+06	.7171+06
14,7	.2845+06	.6232+06	.6145+06
16,8	.2308+06	.5045+06	.5135+06
18,9	.1792+06	.3905+06	.4143+06
21,0	.1290+06	.2799+06	.3162+06
23,1	.9091+05	.1968+06	.2241+06
25,2	.5438+05	.1172+06	.1339+06
27,3	.3067+05	.6601+05	.7371+05
29,4	.8604+04	.1844+05	.1619+05
31,5	.0000	.0000	.0000

* MAXIMUM BASE MOMENT CALCULATED USING ESCOM SPECIFICATION

= 1.02 x 106 N.M.

MAXIMUM RESPONSE (FIRST THREE MODES INCLUDED)

= 0.118 m

MAXIMUM GUST FACTOR (FIRST THREE MODES INCLUDED)

= 1.75

CHAPTER 8

THE OVERALL WIND LOADING PROBLEM.

No study of this nature would be complete without a discussion of the overall problem of wind loading and some of the difficulties involved in the formulation of a consistent and realistic approach to the design of structures subjected to wind excitation.

R.A. Waller in his paper "Design Techniques for Practical Wind Problems" (25) suggests the sort of errors that may arise in the estimation of wind-induced stresses and the confidence that can therefore be put into the various aspects of wind load calculations :

Errors in Stress Estimation (%)

	Wind Velocity	Pressure Co-efficients	Structural Model	Computational Model	Total
Static	50	20	20	10	60
Gusting	100	100	20	25	100
Vortex Shedding	50	200	25	10	200

Although questionable, these figures illustrate the areas where most uncertainty lies and that the degree computational sophistication is of minor significance. The assumption, for example, that mean wind speed at D.F. Malan Airport applies to the Cape Town docks would result in approximately a 50% underestimate in the maximum mean stress.

There is clearly a danger in extrapolating wind speed data from one location to another. The maximum wind velocity at ten metres for a given site depends on the roughness of the surrounding terrain, the topography and the gradient wind velocity. The most rational way of finding a design wind speed would be to base it on the maximum gradient wind velocity (not the extrapolated gradient wind velocities as given in reference 9). Unfortunately, measurements of the gradient wind are seldom if ever available on a consistent statistical basis. Another alternative, is the development of short term survey techniques to measure wind effects at a particular site and the evolution of the necessary statistical machinery to relate these to longer return periods.

8.1. WIND PARAMETERS AND C.P. 3 : CHAPTER V : PART 2 : 1970 :

There is a considerable body of opinion within the profession in Britain which feels that the latest wind code is based to a considerable degree on extrapolated data which results in unreasonably high design values. The adequacy of this code has been discussed in a number of publications (7,26) and the arguments for and against run roughly as follows :

The critics maintain that the revised code is

- based on extrapolated data.
- results in excessive wind loads compared with the 1952 code and is thus both uneconomic and unrealistic particularly for low-rise structures.
- is too complex to apply.
- gives little guidance on complex wind-sensitive structures while being too stringent for standard designs.
- Few, if any, cases occur where a structure fails through insufficient account having been taken of wind.
- there is inadequate research into the effect of these high winds on the maximum response.

The proponents of this code reply that it is :

- based on the latest research and that wind damage occurs in Britain, particularly to cladding costing £7 m per annum.

- the wind speeds in the code can occur and structures based on the 1952 code do not fail because of redundants which were not taken into account in the original design. However, this may not now be the case as structures become more sophisticated.

Clearly, much research is required before these conflicts are resolved but it does seem that, with the greater confidence a gust factor approach gives in the prediction of response, it will be possible to use a reduced safety margin for the effects of extreme winds. In order to do this, however, it will be necessary to predict the behaviour of structures in the non-linear range.

8.2. NON-LINEAR NON-DETERMINISTIC RESPONSE :

Implicit in the formulation outlined so far is the assumption of linearity. However, when considering the response to extremes with such low probabilities of occurrence it is unrealistic to use a "first-passage" approach. Certainly, to limit the response to the elastic range would be uneconomic but the problems of predicting the response using a frequency approach when some plastic deformation or cracking has occurred are formidable.

Hudson in his paper "Dynamic Properties of Full-Scale Structures Determined from Natural Excitations" (27) refers to a building in Peru for which the natural periods of vibration had been measured before and after an earthquake and it was found that the fundamental period had changed from 0.38 to 1.2 seconds as a result of cracking, plastic deformation and other non-linear effects. It was concluded that simple low-level wind excitation tests to measure the vibration characteristics (which show good correlation with calculated values) have limited applicability because of significant structural alterations take place under high load conditions.

At the same time, it has been shown (28) that as the intensity of the excitation increases the response is more complex.

Sub-dominant modes are increasingly prominent and the non-linear damping due to hysteresis effects in the stress/strain relationship becomes significant.

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With yielding, the non-linear damping tends to increase because of the large increase in energy absorption and this gives an additional, unquantifiable safety factor.

Modal analysis is not particularly well suited to the non-linear, non-deterministic solutions because of the need to find the natural modes and frequencies initially.

A possible solution technique might be to use the direct method as described by K.N. Handa (30) with an equivalent linearization (31) for the non-linear relationship.

Some work (29) has been done on the accumulation of plastic deformation in simple structures using step-by-step numerical simulation techniques.

This study concludes that a ductile structure has significantly greater margin of safety against wind action than a brittle structure with the same elastic strength. However, there is an increased risk that, for such structures, some permanent deformation may contribute to the damage of the cladding, etc. It is suggested that a nett load factor of 1.45 on the maximum gust effect can be used. This seems to mean an increase on the conventional wind load factor (1.4).

In general, however, although the theoretical framework exists (32) for the practical solution of non-linear problems on the probabalistic basis, the field remains largely untackled.

Finally, as has already been, mentioned, failure is likely to occur as a result of low cycle fatigue or cumulative plastic damage and these phenomena in themselves are not adequately understood at the present.

8.3. PRESSURE CO-EFFICIENTS :

Another field of uncertainty in the overall wind load problem is the determination of pressure co-efficients.

72./......

There have been many studies to determine the drag co-efficients for various shapes in wind tunnels, but there is evidence to show (26) that drag co-efficients determined from model studies are not the same as those obtained from full-scale research. Also, drag co-efficients in turbulent flow are higher than those measured in steady flow and these effects, together with those associated with the inertial mass co-efficient, C_M, and higher order terms in the formulation of fluctuating drag are usually ignored.

It is well known that structures themselves alter the nature of the flow in their vicinity and that many of the results applied to bluff buildings are taken from measurements in a free wind stream.

For this, reason more confidence can be placed in the pressure co-efficients determined for light lattice structures because the windstream disturbance is minimal.

For bluff structures, further full-scale research is required. Tentative results from studies made by Mackey (26) in Hong Kong on an instrumented, experimental building have shown that it responds to face-on winds at its natural frequency of about 1 HZ and also at a frequency of one seventh of this value. This is thought to be due to the presence of a cushion of stagnant air ahead of the building vibrating with its own natural frequency and "bleeding-off" into the wake around the sides. This research has also investigated the use of small-bore pipes on the face to bleed off high pressure areas into the suction wake.

8.4. VORTEX SHEDDING AND OTHER FORMS OF AERODYNAMIC INSTABILITY :

Having studiously avoided the difficulties of vibration out of the plane of the wind so far, it is necessary to mention them in this overall view of the problem.

In general the susceptability to vortex shedding and other forms of instability depends on the shape of the structure, its natural frequency and, more particularly, the Reynold's number for the flow concerned.

73./......

Indications are that when the vortex shedding frequency, which depends on these factors, approaches the fundamental frequency of the structure, there is a tendency to lock onto this frequency. This results in the amplitude of the response due to the component of the fluctuating force (lift component) out of the plane of the wind considerably exceeding the along-wind response. This is because the turbulent intensity of the fluctuating force is low at the natural frequency resulting in the sinusoidal fluctuations of the lift force dominating the response. The total effect can be formulated as a random process in a similar manner to the along-wind component but the necessary aerodynamic functions are very difficult to determine experimentally (see references 7c and 33).

CHAPTER 9

CONCLUSIONS.

In this study, a brief summary of the nature of wind and the theoretical background of the statistical approach to wind loading has been given. A wide range of examples have been looked at and the overall problem of wind loading discussed.

In a study of this nature, no definitive conclusions can be drawn. The basis, although as wide as possible, is still too narrow for general conclusions and too broad for specific conclusions. The best that can be offered is a few pointers to be added along the road to a more consistent approach to wind loading. These fall into three categories.

9.1. THE SPECIFIC EXAMINATION OF THE STATISTICAL APPROACH :

9.1.1. The variation of the gust response factor and the normalised admittance functions with size for a bluff building has been shown. For the range of sizes chosen the gust factor varied from 3.6 for a 25 m high by 10 m wide building to 2.2 for one which is 110 m high by 35 m wide. These sizes cover a comprehensive range of buildings, since the wind parameters taken apply generally to most urban centres in South Africa, fig. 12 can be used as a rough guide as to the size of the gust factor for most bluff buildings in the country.

9.1.2. The straight line mode was shown to be an excellent approximation for the fundamental mode shape in the examples chosen. This confirms the efficacy of Davenport's gust factor approach as set out in reference 6.

Dynamic effects only become significant in the calculation of the gust factor for cantilever-type, prismatic structures with fundamental frequencies below 0.5 HZ.

For this type of structure and all the examples given the effect of higher modes was neligible.

Damping was not a very significant factor for the normal range of relatively stiff structures considered.

9.1.3. The choice of a coherence function may make a significant difference to the gust factor but more research is required to establish which function if any, is actually applicable to the more temperate wind conditions in this country.

9.1.4. The examples of steel structures show Davenport's gust factor method to have a strictly limited applicability. In cases where it was clearly applicable, good agreement was found with the computed values for the gust factor.

The maximum moments given by the statistical approach did not differ substantially from those calculated from maximum gust speeds based on C.P. 3 Chap. V.

9.2. WIND PARAMETERS WITH REFERENCE TO SOUTH AFRICA :

9.2.1. There is a danger in assuming a set of wind data is generally applicable and extrapolating limited information.

9.2.2. The Davenport/Harris assumption that the standard deviation of the wind fluctuations is invariant with terrain roughness does not apply to South Africa.

9.2.3. For the statistical approach to be used with confidence in this country more wind speed data is needed and, in particular, short term measuring techniques must be developed which can then be related to the overall wind data to allow the wind parameters to be estimated for a particular site.

In the meantime, however, the statistical method allows structural response to be predicted with more confidence, not only because gust correlation and dynamic effects are taken into account, but, because even with reasonably arbitrary wind parameters, this response can be related to a specific probability of occurrence of the extreme wind values.

9.3. THE OVERALL WIND PROBLEM :

Clearly, much more research is required but it seems probable that a gust factor approach coupled with adjustable risk factors in assessing overall structural safety will ultimately lead to a more consistent approach to the problem of wind loading.

APPENDIX A - REFERENCES.

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APPENDIX B - DYNWIND : INPUT

The input required is common to all the DYNWIND programmes except DYNWIND/TOWER which requires two extra terms.

Line 1	:	V10, GUST, RK, DEL, ALPH, GALPH, IB, IH, RD, NO, ICOR, TRANS*
Line 2	:	(Repeated IH times).
		AM, CD, B, H, ATR*
Line 3	:	(Repeated NO times).
		RN, UM (1→IH)

<u>KEY</u>

V10	-	Mean wind speed for the site at 10 metres
GUST	-	Gust wind speed at 10 metres
RK	-	Roughness co-efficient
DEL	-	Logarthmic decrement for damping
ALPH	-	Power law index for mean wind speed.
GALPH	-	Power law index for gust wind speed.
IB	-	Required number of horizontal divisions for coherence function.
IH	-	Required number of vertical divisions (must correspon to the modal ordinates).
RD	-	Density of air
NO	-	Number of natural modes to be considered.
ICOR	-	Correlation function required :

ICOR = 1 - Shiotani (Anisotropic turbulence).

ICOR = 2 - Shiotani (Isotropic turbulence).

ICOR = 3 - Davenport (modified).

ICOR = 4 - Vickery

Repeat IH times

AM	-	Mass per metre at level H
CD	-	Drag co-efficient at level H
B	-	Breadth or area per metre for lattice structures at level H
H	-	Level in metres measured from the base.

Repeat NO times

FN	-	Natural Frequency for the mode considered.
UM (1→IH)	-	Modal ordinates at each level.

For DYNWIND/TOWER only* :

TRANS - Power line span in metres

ATR - Exposed area of power line at level H. (This should contain the drag co-efficient for a suspended cable for the wind blowing transversely onto the cables.

i.e. ATR = TRANS x cable diameter x Drag co-efficient.)

```
      DIMENSION  AM(25),CD(25),B(25),H(25),UM(25),G(25),V(25),HRO2(400),
     1RO(400),SV(400),SP(400),XA2(400),SQ(400),HS(25)  ,HR(25,25),GA(25),
     2FLUT(25),IBUF(2000),VG(25),
     3 VMO(25),GMO(25),VMM(25),BETA(25),SDE(5),GVT(5)
      CALL  PLOTS(IBUF,2000,17)
      IO=8
       IOUT=5
      GSD=0.
      QME=0.
      CALL  AXIS(-5.,0.,13HLOG.FREQUENCY,-13,6.,0.,-5.,1.)
      CALL  AXIS(-5.,0.,10HADMITTANCE,+10,5.,90.,0.,0.25)
      READ(IO,10)   V10,GUST,RK,DEL,ALPH,GALPH,IB,IH,RD,NO,ICOR
      DO 2 I=1,IH
      BETA(I)=0.
    2 READ(IO,10) AM(I),CD(I),B(I),H(I)
      DO 160 NON=1,NO
    3 READ(IO,10)   FN,(UM(I),I=1,IH)
      DO 160 IC=1,ICOR
      WRITE(IOUT,201)
  201 FORMAT(1H1,'        THE SPECTRAL ANALYSIS OF ',/6X,60(
     C  '*')//)
      WRITE(IOUT,17) V10,RK,DEL,ALPH,FN
   17 FORMAT(' MEAN WIND   ROUGHNESS  DAMPING  POWER INDEX FUND.FREQ '/
     1  ' ---- ----    ---------  -------  ----- ----- ---------'/
     2 5F10.3/)
      WRITE (IOUT,18)
   18 FORMAT('  LEVEL      MASS/M   DRAG COEFF   BREADTH  MODE SHAPE   GAM
     1MA MEAN SPEED  GUST SPEED'/'  -----       ------   ---- -----    -----
     2--   ---- ----- -----   ---- -----   ---- -----')
      DO    20   IL=1,IH
      VG(IL)=(H(IL)/10.)**GALPH*GUST
      V(IL)=(H(IL)/10.)**ALPH *V10
      G(IL)= V(IL)*B(IL)* CD(IL)* UM(IL)/B(1)/CD(1)/V10
      WRITE(IOUT,16) H(IL),AM(IL),CD(IL),B(IL),UM(IL),G(IL),V(IL)
     1,VG(IL)
   16 FORMAT(2(F8.1,2X),6F10.3 )
   20 GA(IL)=G(IL)/V10*V(IL)
      X1 =0
      X2=0
      X3=0.
      Y1=0.
      Y2=0
      Y3=0.
      BH=(B(1)+B(IH))/2/IB
      DH=H(IH)/(IH-1)
      NH=IH-2
      DO 30   IL=2,NH,2
      X3=X3+GA(IL)*4.
      X2=X2+4*G(IL)*G(IL)
   30 X1=X1+ 4* G(IL)
      NOH =IH-1
      DO 40   IL =3,NOH,2
      Y3=Y3+GA(IL)*2.
      Y2=Y2+2*G(IL)*G(IL)
   40 Y1= Y1 +   2 * G(IL)
      H2=(G(1)*G(1)+X2+Y2+G(IH)*G(IH))*DH*0.33333333
      H1 = (G(1) +X1 +Y1  +G(IH))* DH*0.3333333333
```

```
      H3=(GA(1)+X3+Y3+GA(IH))*DH*0.33333333333
      HE=H1**2/H2
      X1=0
      PJA=RD*B(1)*CD(1)*V10 *V10*H1/2
      PME=PJA/H1*H3
      Y1=0
      DO 50 IL =2,NH,2
   50 X1=X1 + UM(IL)*UM(IL)*AM(IL)
      DO 60 IL =3,NOH,2
   60 Y1=Y1 + UM(IL)*UM(IL)*AM(IL)
      GK=(UM(1)*UM(1)*AM(1)+4*X1+2*Y1+UM(IH)*UM(IH)*AM(IH))*DH*0.33333
      GK=GK*(2.*3.142*FN)**2
      WRITE(IOUT,14) NON
      RN=0.00000
      TN=.00001
      QQ=0.
      QQS=0.
      CALL PLOT(0.0,0.0,-3)
      DO 100 N=1,400
      IF(N.GT.100) TN=0.001
      IF(N.GT.199) TN=0.01
      IF(N.GT.299) TN=0.1
      RN=RN+TN
      IX=0
      DO 97 K=1,IB
      J=0
      IHJ=IH-J
   72 DO  78 I=1,IHJ
      IJ=I+J
      GO TO(71,73,76,75),IC
   76 HD=((J*DH)**2+(2*IX*BH)**2)**0.5
      HR(IJ ,I)=G(IJ )*G(I)*EXP(-  HD*8.*RN/(0.5*(V(IJ)+V(I))))
      HR(I,IJ)=HR(IJ,I)
      GO TO 78
   73 A=1.27
      D=1.27
      C=0
      GO TO 77
   71 A=3.82
      D=1.27
      C=0.8
   77 RMN=((J*DH)**2+(IX*BH)**2)**0.5
      TMN=1.57079-ATAN(IX*BH/J/DH)
      CONS=((D*COS(TMN))**2+(A*SIN(TMN))**2)**0.5
      EN=RN*2.*3.142
      HR(IJ ,I)=G(IJ )*G(I)*EXP(-A*D*EN*RMN/V10/CONS)*COS(C*EN*RMN/V10)
      HR(I,IJ )=HR(IJ ,I)
      GO TO 78
   75 AL1=151.2*(V(IJ)+V(I))/2./V10
      RAB=((J*DH)**2+(IX*BH)**2)**0.5
      THE=RAB/AL1*(1+(6.2832*RN*AL1/V10)**2)**0.5
      HR(IJ,I)=G(IJ)*G(I)*EXP(-1.1936*THE)*COS(0.7*THE)
      HR(I,IJ)=HR(IJ,I)
   78 CONTINUE
      IF(J-IH) 79,79,82
   79 J=J+1
      IHJ=IH-J
      GO TO 72
   82 HSUM=0.
      DO 85 I=1,IH
```

```
      HX=0
      HY=0
      DO 83 J=2,NH,2
   83 HX=HX+HR(I,J)
      DO 84 J=3,NOH,2
   84 HY=HY+HR(I,J)
   85 HS(I)=(HR(I,1)+4*HX+2*HY+HR(I,IH))*DH*0.3333333
      HX=0
      HY=0
      DO 87 I=2,NH,2
   87 HX=HX+HS(I)
      DO 88 I=3,NOH,2
   88 HY=HY+HS(I)
      HRO2(K )=(HS(1)+4*HX+2*HY+HS(IH))*DH*0.33333333
   97 IX=IX+1
      IT=1
      IK=0
      RO(N)=0
      DO 98 KN=1,IB
      RO(N)=RO(N)+HRO2(KN)*(IT*(IB-IK))
      IT=2
   98 IK=IK+1
      RO(N) = RO (N) /H1  ** 2/IB/IB
       AN = RN * 1800./ V10
   86 SV(N) = 4.*RK*V10* V10 *  AN /(2 +AN*AN)**0.83333
      SP(N) = 4.*PJA*PJA* RO(N)*SV(N)/V10/V10
      XA2(N)= (1- RN*RN/FN/FN) **2 +(DEL*RN/3.142/FN)**2
      SQ(N)= SP(N)/GK/GK/XA2(N)
      X=0.4343*ALOG(RN)
      Y=RO(N)*4.
      IF(N.EQ.1) IPEN=3
      IF(N.GT.1) IPEN=2
      CALL  PLOT(X,Y,IPEN)
      IF(N.EQ.1) TQ=SQ(1)
      IF(N.EQ.1) GO TO 99
      TQ=(SQ(N)+SQ(N-1))/2.
   99 QQS=QQS+TQ*RN*TN
      QQ=QQ+TQ/RN*TN
      TEN=N/10*10
      IF(N-TEN) 100, ,100
      WRITE(IOUT,19) RN,RO(N),SV(N),SP(N),XA2(N),SQ(N)
   15 FORMAT( 10F10.3/)
  100 CONTINUE
      SDE(NON) =QQ   **0.5
      VT=(QQS/QQ)**0.5
      QME=QME+PME/GK
      VTS=(2.*ALOG(VT*3600.))**0.5
      GVT(NON)=VTS+(0.5772/VTS)
      GSD=GSD+SDE(NON)*GVT(NON)
      QMAX=QME+GSD
      WRITE(IOUT,13) QQ,SDE(NON),GVT(NON),QME,QMAX
      UF=QMAX/QME
      GUF=GUF+UF
      TFL=0
      IFL=0
      DO 167 I1=1,IH
      FLUT(I1)=0.
      IFL=I1+1
      DO 166 I2=IFL,IH
  166 FLUT(I1)=FLUT(I1)+(6.284*FN)**2*DH*AM(I2)*UM(I2)*(H(I2)-H(I1))
```

```
167 BETA(I1)=BETA(I1)+FLUT(I1)*SDE(NON)*GVT(NON)
160 WRITE(IOUT,200) UF
    WRITE(IOUT,203)
    IFL=0
    DO 176 I1=1,IH
    VMO(I1)=0
    GMO(I1)=0
    IFL=I1+1
    DO 175 I2=IFL,IH
    VMO(I1)=VMO(I1)+0.613*CD(I2)*B(I2)*DH*V(I2)**2*(H(I2)-H(I1))
    GMO(I1)=GMO(I1)+0.613*CD(I2)*B(I2)*DH*VG(I2)**2*(H(I2)-H(I1))
175 VMM(I1)=VMO(I1)+BETA(I1)
    WRITE(IOUT,204)H(I1),VMO(I1),GMO(I1),VMM(I1)
176 CONTINUE
203 FORMAT(1H1,' LEVEL         MEAN MOMENT GUST MOMENT MAX.MOMENT
   1'/1X,6('-'),7X,3(14('-'))/)
204 FORMAT(F6.1,7X,3(E10.4,2X))
    CALL PLOT(11.,0.,999)
200 FORMAT('                    GUST FACTOR = 'F5.3/11X,19('-'))
 10 FORMAT( )
 13 FORMAT(/' MODAL RESPONSE PARAMETERS:'/26('-')/
   1' VARIANCE   STD.DEV     G FACTOR    MEAN RESP MAX.RESPONSE'/56('-')
   2/8(E10.4,2X))
 19  FORMAT(8(E10.4,2X))
 14 FORMAT(1H1,'       SPECTRAL VALUES - MODE NUMBER ',I2/6X,32('-')/
   1' FREQUENCY AERO.ADMITTANCE VELOCITY    FORCE   MECH.ADMITTANCE
   2RESPONSE'/73('-')/)
      CALL EXIT
     END
6:
```

```
      DIMENSION  AM(25),CD(25),B(25),H(25),UM(25),G(25),V(25),HRO2(400),
     1RO(400),SV(400),SP(400),XA2(400),SQ(400),HS(25)  ,HR(25,25),GA(25),
     2FLUT(25),IBUF(2000),VG(25),GC(25),CRO(25),ATR(25),
     3 VMO(25),GMO(25),VMM(25),BETA(25),SDE(5),GVT(5)
      CALL PLOTS(IBUF,2000,17)
      IO=8
       IOUT=5
      GSD=0.
      QME=0.
      CALL AXIS(-5.,0.,13HLOG.FREQUENCY,-13,6.,0.,-5.,1.)
      CALL AXIS(-5.,0.,10HADMITTANCE,+10,5.,90.,0.,0.25)
      READ(IO,10)   V10,GUST,RK,DEL,ALPH,GALPH,IB,IH,RD,NO,ICOR,TRANS
      DO 2 I=1,IH
      BETA(I)=0.
    2 READ(IO,10) AM(I),CD(I),B(I),H(I),ATR(I)
      DO 160 NON=1,NO
      ATMO=0.
      HTRANS=0.
    3 READ(IO,10)   FN,(UM(I),I=1,IH)
      DO 160 IC=1,ICOR
      WRITE(IOUT,201)
  201 FORMAT(1H1,'        THE SPECTRAL ANALYSIS OF ',/6X,60(
     C  '*')//)
      WRITE(IOUT,17) V10,RK,DEL,ALPH,FN
   17 FORMAT(' MEAN WIND   ROUGHNESS  DAMPING  POWER INDEX FUND.FREQ '/
     1  ' ---- ----   ---------  -------  ----- ----- ----------'/
     2 5F10.3/)
      WRITE (IOUT,18)
   18 FORMAT('  LEVEL      MASS/M   DRAG COEFF   BREADTH  MODE SHAPE   GAM
     1MA MEAN SPEED  GUST SPEED'/'  ------      ------    ---- -----    -----
     2--  ---- ----- -----  ---- -----  ---- -----')
      DO   20  IL=1,IH
      VG(IL)=(H(IL)/10.)**GALPH*GUST
      V(IL)=(H(IL)/10.)**ALPH *V10
      G(IL)= V(IL)*B(IL)* CD(IL)* UM(IL)/B(1)/CD(1)/V10
      GC(IL)=SIN(1.57079*(IL-1)/(IH-1))
      ATMO=ATMO+ATR(IL)*0.613*V(IL)**2*UM(IL)
      WRITE(IOUT,16) H(IL),AM(IL),CD(IL),B(IL),UM(IL),G(IL),V(IL)
     1,VG(IL),GC(IL),ATMO
   16 FORMAT(2(F8.1,2X),8F10.3 )
      HTRANS=HTRANS+GC(IL)
   20 GA(IL)=G(IL)/V10*V(IL)
      HTRANS=HTRANS*TRANS/2./(IH-1)
      X1 =0
      X2=0
      X3=0.
      Y1=0.
      Y2=0
      Y3=0.
      BH=(B(1)+B(IH))/2/IB
      DH=H(IH)/(IH-1)
      NH=IH-2
      DO 30  IL=2,NH,2
      X3=X3+GA(IL)*4.
      X2=X2+4*G(IL)*G(IL)
   30 X1=X1+ 4* G(IL)
      NOH =IH-1
```

```
      DO 40  IL =3,NOH,2
      Y3=Y3+GA(IL)*2.
      Y2=Y2+2*G(IL)*G(IL)
   40 Y1= Y1 +  2 * G(IL)
      H2=(G(1)*G(1)+X2+Y2+G(IH)*G(IH))*DH*0.33333333
      H1 = (G(1) +X1 +Y1  +G(IH))* DH*0.3333333333
      H3=(GA(1)+X3+Y3+GA(IH))*DH*0.33333333333
      HE=H1**2/H2
      X1=0
      PJA=RD*B(1)*CD(1)*V10 *V10*H1/2+ATMO
      PME=PJA/H1*H3
      Y1=0
      DO 50 IL =2,NH,2
   50 X1=X1 + UM(IL)*UM(IL)*AM(IL)
      DO 60 IL =3,NOH,2
   60 Y1=Y1 + UM(IL)*UM(IL)*AM(IL)
      GK=(UM(1)*UM(1)*AM(1)+4*X1+2*Y1+UM(IH)*UM(IH)*AM(IH))*DH*0.33333
      GK=GK*(2.*3.142*FN)**2
      WRITE(IOUT,14) NON
      RN=0.00000
      TN=.00001
      QQ=0.
      QQS=0.
      CALL PLOT(0.0,0.0,-3)
      DO 100 N=1,400
      IF(N.GT.100) TN=0.001
      IF(N.GT.199) TN=0.01
      IF(N.GT.299) TN=0.1
      RN=RN+TN
      DO 86 ITOW=1,2
      IF(ITOW.EQ.2) DH=TRANS/2./(IH-1)
      IF(ITOW.EQ.1) DH=H(IH)/(IH-1)
      IX=0
      DO 97 K=1,IB
      J=0
      IHJ=IH-J
   72 DO  78 I=1,IHJ
      IJ=I+J
      GO TO(71,76,73,75),ITOW
   76 HD=J*TRANS/(IH-1)
      HR(IJ ,I)=GC(IJ )*GC(I)*EXP(-  HD*16.*RN/V(IH))
      HR(I,IJ)=HR(IJ,I)
      GO TO 78
   73 A=1.27
      D=1.27
      C=0
      GO TO 77
   71 A=3.82
      D=1.27
      C=0.8
   77 RMN=((J*DH)**2+(IX*BH)**2)**0.5
      TMN=1.570796-ATAN(IX*BH/J/DH)
      CONS=((D*COS(TMN))**2+(A*SIN(TMN))**2)**0.5
      EN=RN*2.*3.142
      HR(IJ ,I)=G(IJ )*G(I)*EXP(-A*D*EN*RMN/V10/CONS)*COS(C*EN*RMN/V10)
      HR(I,IJ )=HR(IJ ,I)
      GO TO 78
   75 AL1=151.2*(V(IJ)+V(I))/2./V10
      RAB=((J*DH)**2+(IX*BH)**2)**0.5
      THE=RAB/AL1*(1+(6.2832*RN*AL1/V10)**2)**0.5
```

```
      HR(IJ,I)=G(IJ)*G(I)*EXP(-1.1936*THE)*COS(0.7*THE)
      HR(I,IJ)=HR(IJ,I)
   78 CONTINUE
      IF(J-IH) 79,79,82
   79 J=J+1
      IHJ=IH-J
      GO TO 72
   82 HSUM=0.
      DO 85 I=1,IH
      HX=0
      HY=0
      DO 83 J=2,NH,2
   83 HX=HX+HR(I,J)
      DO 84 J=3,NOH,2
   84 HY=HY+HR(I,J)
   85 HS(I)=(HR(I,1)+4*HX+2*HY+HR(I,IH))*DH*0.3333333
      HX=0
      HY=0
      DO 87 I=2,NH,2
   87 HX=HX+HS(I)
      DO 88 I=3,NOH,2
   88 HY=HY+HS(I)
      HRO2(K )=(HS(1)+4*HX+2*HY+HS(IH))*DH*0.33333333
   97 IX=IX+1
      IT=1
      IK=0
      RO(N)=0
      DO 98 KN=1,IB
      RO(N)=RO(N)+HRO2(KN)*(IT*(IB-IK))
      IT=2
   98 IK=IK+1
   86 CRO(ITOW)=RO(N)
      CRO(1) =  CRO(1) /H1  ** 2/IB/IB
      CRO(2)=CRO(2)/HTRANS/HTRANS
      RO(N)=CRO(1)*CRO(2)
       AN = RN * 1800./ V10
       SV(N) = 4.*RK*V10* V10 *  AN /(2 +AN*AN)**0.83333
      SP(N) = 4.*PJA*PJA* RO(N)*SV(N)/V10/V10
      XA2(N)= (1- RN*RN/FN/FN) **2 +(DEL*RN/3.142/FN)**2
      SQ(N)= SP(N)/GK/GK/XA2(N)
      X=0.4343*ALOG(RN)
      Y=RO(N)*4.
      IF(N.EQ.1) IPEN=3
      IF(N.GT.1) IPEN=2
      CALL  PLOT(X,Y,IPEN)
      IF(N.EQ.1) TQ=SQ(1)
      IF(N.EQ.1) GO TO 99
      TQ=(SQ(N)+SQ(N-1))/2.
   99 QQS=QQS+TQ*RN*TN
      QQ=QQ+TQ/RN*TN
      TEN=N/10*10
      IF(N-TEN) 100, ,100
      WRITE(IOUT,19) RN,RO(N),SV(N),SP(N),XA2(N),SQ(N)
   15 FORMAT( 10F10.3/)
  100 CONTINUE
      SDE(NON) =QQ   **0.5
      VT=(QQS/QQ)**0.5
      QME=QME+PME/GK
      VTS=(2.*ALOG(VT*3600.))**0.5
      GVT(NON)=VTS+(0.5772/VTS)
```

```
   GSD=GSD+SDE(NON)*GVT(NON)
   QMAX=QME+GSD
   WRITE(IOUT,13) QQ,SDE(NON),GVT(NON),QME,QMAX
   UF=QMAX/QME
   GUF=GUF+UF
   TFL=0
   IFL=0
   DH=H(IH)/(IH-1)
   DO 167 I1=1,IH
   FLUT(I1)=0.
   IFL=I1+1
   DO 166 I2=IFL,IH
66 FLUT(I1)=FLUT(I1)+(6.284*FN)**2*DH*AM(I2)*UM(I2)*(H(I2)-H(I1))
67 BETA(I1)=BETA(I1)+FLUT(I1)*SDE(NON)*GVT(NON)
60 WRITE(IOUT,200) UF
   WRITE(IOUT,203)
   IFL=0
   DO 176 I1=1,IH
   VMO(I1)=0
   GMO(I1)=0
   IFL=I1+1
   DO 175 I2=IFL,IH
   VMO(I1)=VMO(I1)+0.613*CD(I2)*B(I2)*DH*V(I2)**2*(H(I2)-H(I1))+
  CATR(I2)*0.613*(H(I2)-H(I1))*V(I2)**2
   GMO(I1)=GMO(I1)+0.613*CD(I2)*B(I2)*DH*VG(I2)**2*(H(I2)-H(I1))+
  CATR(I2)*0.613*(H(I2)-H(I1))*VG(I2)**2
75 VMM(I1)=VMO(I1)+BETA(I1)
   WRITE(IOUT,204)H(I1),VMO(I1),GMO(I1),VMM(I1)
76 CONTINUE
03 FORMAT(1H1,' LEVEL       MEAN MOMENT  GUST MOMENT  MAX.MOMENT
  1'/1X,6('-'),7X,3(14('-'))/)
04 FORMAT(F6.1,7X,3(E10.4,2X))
   CALL PLOT(11.,0.,999)
00 FORMAT('                 GUST FACTOR = 'F5.3/11X,19('-'))
10 FORMAT( )
13 FORMAT(/' MODAL RESPONSE PARAMETERS:'/26('-')/
  1' VARIANCE   STD.DEV    G FACTOR    MEAN RESP MAX.RESPONSE'/56('-')
  2/8(E10.4,2X))
19  FORMAT(8(E10.4,2X))
14 FORMAT(1H1,'      SPECTRAL VALUES - MODE NUMBER ',I2/6X,32('-')/
  1' FREQUENCY AERO.ADMITTANCE VELOCITY    FORCE  MECH.ADMITTANCE
  2RESPONSE'/73('-')/)
     CALL EXIT
    END
:
```

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