International Research Seminar

WIND EFFECTS ON BUILDINGS AND STRUCTURES

Ottawa, Canada, 11-15 September 1967

PROCEEDINGS, VOLUME I

National Research Council of Canada
University of Toronto
University of Western Ontario
Meteorological Branch, Dept. of Transport

UNIVERSITY OF TORONTO PRESS

THE DEPENDENCE OF WIND LOADS ON METEOROLOGICAL PARAMETERS

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Abstract

This paper first defines the meteorological requirements for defining statistically the response of a structure to wind. The requirements are specified under the climatic aspects (defining the windiness of a site) and the wind structure as defined by the turbulence and mean velocity profile. Representative statistical properties of the turbulence (spectra and cross-correlograms) and the mean velocity profiles are estimated from experimental results for different types of terrain, particular attention being paid to the city environment.

The general problem of defining the climate of wind from weather recrods is discussed with regard to interpretation, reliability and influence of exposure on the results. A relationship between the parent distribution of wind speeds, which are assumed to be Weibull distribution, and extreme values is developed and the implications for structural design discussed.

Résumé

L'auteur commence par préciser quelles sont les données météorologiques nécessaires pour la définition statistique de la réponse d'un ouvrage au vent. Ces données sont d'ordre climatique (ventosité locale ou caractère de fréquence des vents) et structural (profils de turbulence et de vitesses moyenne du vent). Il déduit des données expérimentales les caractéristiques statistiques représentatives de la turbulence (courbes de fréquence et diagrammes de la fonction de corrélation) et les profils de vitesses moyennes du vent pour différents types de terrains, en particulier les aires urbaines.

L'auteur étudie les questions générales de définition d'un régime éolien à l'aide des statistiques météorologiques, et particulièrement des questions d'interprétation, de fiabilité et d'influence de l'orientation sur les résultats. L'auteur établit une relation entre la distribution des diverses vitesses du vent, qu'il suppose être celle de Weibull, et la courbe des vitesse maximales; il étudie ensuite les conséquences qui en découlent pour le calcul des ouvrages.

Introduction

Since the last International Seminar in 1963 at the National Physical Laboratory in England, there have been a number of significant developments in the meteorological aspects of the wind loading problem. In part, these are due to research in meteorology and in part due to new demands for meteorological information created by advances in structural design.

I would first like to refer briefly to one particular advance in structural design which I think is particularly significant. For some years, the damaging effects of repeated loads on structures have been recognized from research results and histories of failure: for some time, cognizance of these effects has been taken in the design of aircraft.[1] More recently, there have been attempts to design civil structures in a deliberate way to resist the repeated loading action due to wind[2,3]. This is in contradistinction to the more conventional design approach which is concerned with the static application of a single large load. Experience suggests that unserviceability due to repeated loading effects is a more likely occurrence than unserviceability or collapse from the single application of an exceptionally large load.

The action of repeated loading by wind can cause a variety of forms of structural unserviceability. The first of

these is fatigue damage. There are many examples of fatigue failure due to wind amongst smaller structures such as towers[4], lamp standards, chimney stacks, and even among bridges[5]. With the greater use of higher strength steels, the threat of this form of failure increases.

The second form of unserviceability induced by repeated loading is foundation settlement. There are several indications to suggest that repeated loading due to wind can induce settlement at an accelerated rate: one of the classic examples is that of the Tower of Pisa. Interesting recent research by Colonetti[6] on the full scale structure and in the wind tunnel suggests that repeated reversals of loading may have accelerated the tilting.

The third action of repeated loads is to cause excessive deflections. These may frequently impair the performance of a structure at loads which are far below those which would cause failure. Examples of this are the cracking of masonry elements and plaster in tall buildings, and the deflection of television and microwave towers as well as radio telescopes and radar antennas.

A fourth action of repeated loads is to induce motion in tall buildings affecting the comfort of people , if not actually causing anxiety. The importance of this problem has been ably commented upon already by Robertson[3] and he will probably elaborate further on this question at this conference.

The need to meet restrictions on the effects of repeated loads considerably braodens the demand for meteorological information.

At this point, we should turn our attention to briefly describing an approach to the problem of repeated load. Fig. 1 illustrates a typical response history of a structure (a long span suspension bridge) under the action of wind. Shown are the expected annual number of cycles of operation (from .01 - 10^6 cycles per annum) for a range of response levels. Also shown are various design limitations on fatigue and deflection. Clearly, the structure will remain serviceable, provided the performance curve lies within the criteria. It should be noted that the performance curve for fewer than 1 cycle per annum approaches the extreme value distribution for the largest annual maximum response; the return period in years then is approximately the reciprocal of the number of cycles.

The method of arriving at such performance curve has been described in some detail elsewhere[2]. It depends amongst other things on the prediction of the statistical distributions of the response. This can be done in a manner outlined in Fig. 2. This indicates typically the prediction of the statistical distribution of the deflection, stress and glass pressure desirable in the design of a tall building. To derive these distributions, two types of information are required: first, the aerodynamic responses, and second, the distribution of reference wind speeds. Let

us describe more specifically what we mean by these terms.

The aerodynamic response is a mapping of the response of the structure -- whether this be stress, deflection or glass pressure -- over the entire range of wind directions and speeds likely to be encountered. This "map" can be derived either from wind tunnel tests or, in some case, theoretical analysis. If the response is dynamic, the aerodynamic response will be characterized by two or more diagrams reflecting mean and fluctuating response.

The probability distribution of the reference wind speed is taken the same as the relative frequency of wind speeds observed at the site of the structure -- inferred to have occurred there. This is based on the premise that past statistics of the wind will be representative of the future. The particular reference velocity chosen is partly one of convenience. This question is an important one; it is referred to again in a discussion below, from which it appears the preferred value for a reference velocity at the site of a structure is a mean velocity averaged over a period of 10 - 30 minutes. The reference velocities used in development of the aerodynamic response and in the probability distribution should, of course, correspond to one another.

From this general idea of a design approach to repeated loading, we can now proceed to develop the meteorological information required. One the one hand, we need to be able to define the statistics of the wind velocity indicating how windy the site is; and on the other hand, we need to be able to define the structure of the wind, its turbulence and mean speed profile, so as to determine the aerodynamic response under flow conditions appropriate to the actual site. The evaluation of these properties of the wind forms the major topic of this paper. Unfortunately, the problem of the development of reliable information is not always straightforward.

Properties of the wind

Let us imagine that prior to the building of a structure, measurements of the wind are made at the site over a long period. Such a record can perhaps be construed from Fig. 3. This shows the speed and the vertical and horizontal direction changes. Such a record can, of course, be described by a statistical distribution giving the relative frequencies of occurrence of each velocity component and a spectrum, describing the amplitude of the contributions from the different frequencies characterizing the fluctuations.

The spectrum of such a record generally reveals a readily observable fact that the wind contains motions of two vastly different time scales; on the one hand, motions on the scale of the weather systems themselves and on the other hand, gusts. A somewhat similar situation is found with the sea. The sea surface rises and falls with the tide as well as the waves. While the tide cycles in several hours, a wave lasts only a second or so. A

difference is that the movement of the weather systems is far less regular than the tides. Some of the earliest quantitative data, on these fluctuations, is furnished by van der Hoven's analysis of the wind speed spectrum over a frequency range extending from approximately 1 cycle per month to 1 cycle per second. This is shown in Fig. 4, as well as other spectra[9,10,11,12] covering the low cycle range.

The principle features of these spectra are:

1) the peak of energy at a period of 4 days, (synoptic fluctuations);
2) the peak of energy at a period of approximately 1 minute (gusts); and
3) the pronounced gap centered at a period of approximately half an hour. (the meteorological gap).

Other measurements by van der Hoven[8] pinpoint the frequency at the center of the spectral gap at several locations as follows:

TABLE 1

Characteristics of the Micrometeorological Gap

Location	Ht. m	Wind vel. m/sec.	Period min.	Amplitude $(m/sec.)^2$
Brookhaven	108	6.2	60	.20
Brookhaven	125	7.8	20	.15
Pennsylvania	30	1.8	30	.15
Oak Ridge	100	4.1	10	.10

Location	Ht. m	Wind vel. m/sec.	Period min.	Amplitude $(m/sec.)^2$
Oak Ridge	100	5.2	20	.15
Idaho Falls	76	12.8	12	.20
Idaho Falls	76	8.9	7	.10

An explanation for the gap given in terms of the different dissipation ratio of the quasi - horizontal synoptic fluctuations and the gusts have been given by Kolesnikova and Monin[13]. A major significance of the spectral gap is that it enables the wind to be described conveniently in terms of 1) a mean velocity, reflecting only the synoptic variations in wind speed, and 2) the superimposed gusts.

This brings us at once to the choice of a suitable averaging period for defining mean wind speeds. Several factors bear on the choice, and the more important can be cited as follows:

1) The period should be chosen to minimize the non-stationarities (i.e. trends) within the period.
2) The period should be short enough to reflect the maximum effect of a relatively short duration wind storm.
3) The period should be long enough to allow steady state response of the structure to develop.
4) The period should, if possible, conform to a standard meteorological observation.

General considerations suggest that a good averaging period is in the range 5 - 30 minutes and at best about 10 - 15 minutes. Reasons for this choice are that:

1) This period lies near the centre of the spectral gap and this is a good assurance that in general trends will not be strong;
2) The period is generally short enough to reflect sharp sudden storms such as thunder storms which usually last 5 - 10 minutes;
3) Natural frequencies of structures range from approximately .1 cycle/sec. (for tall buildings and long bridges) and higher. Within a period of 15 minutes, at least 90 cycles of oscillation will therefore occur, and this is normally quite adequate for the development of steady state conditions.
4) The standard meteorological measurements made vary throughout the world. A one hour averaging period is common: Japan uses a 10 min. averaging period; U.S.A. used to use a 5 min. averaging period.

For the present, we shall define a mean velocity as one averaged over a period of approximately 10-20 minutes. In practice, it is often necessary to work with wind speed measurements made over different averaging periods. The scatter introduced by using different averaging periods will be discussed at a later juncture. For the present, we shall assume that the consequences of using different averaging periods, provided these periods lie within the spectral gap, are not very significant.

We will now turn our attention to the specific information about the wind at a building site, referred to earlier, required for structural design; namely,

1) The wind structure; and
2) Statistics of the wind climate.

THE STRUCTURE OF THE WIND

General

Two properties of the wind are of particular importance in structural design:

1) The mean velocity profile; and
2) The properties of turbulence.

Both questions were discussed in some detail in a paper[14] presented at the previous International Symposium. We will discuss below the results of these earlier findings in the light of recent research.

Mean Wind Speed Profile

In the previous paper[14], it was concluded that a power law velocity profile was a simple and adequate expression for the mean velocity profile under most strong wind (or neutral) conditions over all types of terrain, provided it was relatively level. A convenient form was,

$$\frac{V_z}{V_G} = \left(\frac{z}{z_G}\right)^{\alpha} \quad (1)$$

where V_z is the velocity at height z;
V_G is the gradient velocity first attained at height z_G; and
α is the power law exponent.

The gradient velocity is a useful reference wind speed, since it is independent of the local terrain roughness. Values of the parameters α and Z_G suggested previously for various terrains were,

TABLE 2

Average Parameters of Power Law Mean Wind Speed Profiles

	α	Z_G (Ft)
Flat open country	.16	900
Rough wooded country, city suburbs	.28	1300
Heavily built up urban centres	.40	1400

A selection of mean speed velocity profiles from different localities is given in Fig. 5. These include data given in the original paper as well as a number of more recent observations. The source material is given in Appendix A. The exponents derived from these curves appear in general agreement with the values quoted above in Table 2.

The profiles of particular importance in the design of tall buildings are those for city centres. Unfortunately, these are conditions for which the least information is known since most meteorological research effort has been expended on studies over open terrain. More information has been acquired recently and Table 3 contains a summary of the power law exponents found in ten cities.

TABLE 3

Values of Power Law Exponents in Cities

City	Reference	Upper limit of investigation Ft.	Exponent	Comment
Paris	Eiffel (1900)[15]	1000	.45	mean speeds of 3 storms recorded at Eiffel tower 18/6/1897, 3/3/1896 and 12/11/1896
Leningrad	Ariel and Kliuchnikova (1960)[16]	490	.41	Tower measurements
New York	Rathbun (1940)[17]	1250	.39	measurements at Empire State Bldg. U.S.W.B. and N.Y. City Obsty.
Copenhagen	Jensen (1958)[18]	240	.38	Tower measurements
London (U.K.)	Shellard (1967)[19]	600	.36	P.O. Tower measurements (avge).
London (Ont.)	Davenport (1964)[20]	137	.36	Microwave tower in suburbs
Kiev	Ariel and Kliuchnikova (1960)[16]	590	.35	Tower measurements
Tokyo	Shiotani, and Yamamoto[21]	200	.34	Near Royal Palace
Tokyo	Soma (1964)[22]	820	.33	Typhoon measurements - city outskirts
Montreal	-	(984)	.28	On tower in Bot. Gdns. Upper level on Mount Royal
St. Louis	-	455	.25	T.V. Tower

The scatter is to be expected. These data support the suggested value of .40, but also indicate that a value of .35 may be representative of some built-up regions.

The value of the other parameter, Z_G, for rough terrain can be found from the ratio of surface to gradient velocity. Observed values of this ratio are given in Table 4.

TABLE 4

Observed Ratios of Surface to Gradient Wind

City	Obs'n Ht(Ft)	Obs'd exponent	$\frac{V_{obs}}{V_g}$	Ref
Washington⁺	100	-	.45	Graham and Hudson (1960)[23]
Kiev⁺	66	.34	.40	Ariel and Kliuchnikova (1960)[16]
Leningrad⁺	492	.41	.70	" "
Brookhaven*	355	.32	.61	Davenport (1965)[24]

⁺City Centre *Wooded Terrain

These observations are plotted in Fig. 6B. Taken together, the results fall on a line having an exponent of .36 and intercepting the gradient velocity at 1600 ft. This value is in agreement with the values suggested previously, given in Table 2.

Another check on the surface velocities in cities was obtained by more devious means, using some data published previously[7] (and kindly provided by Mr. H.C.S. Thom) on extreme wind speeds observed at city centres and airports in the United States. Values are given in Table 5.

TABLE 5

Once-in-50-year Wind Speeds in U.S. Cities

City	City Office		Airport	
	Anem.Ht. Ft.	Wind speed mph.	Anem.Ht. Ft.	Wind speed mph
Boston	188	72	63	103
New Haven	155	60	42	74
Chicago	-	57	38	70
S.S. Marie	52	63	33	85
Kansas City	181	63	76	95
Omaha	121	65	68	91
Knoxville	111	57	71	89
Nashville	191	73	42	86
Spokane	110	51	29	78

In spite of the anemometer at the City Office being in all cases higher than at the Airport, the wind speed is lower. These data are plotted in a somewhat different form in Fig. 6A. In this, the City Office wind speeds are normalized by a gradient wind speed found from the Airport wind speeds assuming the latter fall on a 1/7 power law profile ($\alpha = .143$) with $Z_G = 900\ ft$. Plotted in this way, the city data tends to cluster about a completely different profile to the airport data.

The New Orleans data is taken from a study by Graham and Hudson[23] of 132 occasions of high winds occurring at the

Airport and City Office. The anemometers were 53 ft and 85 ft above ground respectively. In spite of the higher City Office instrument, the wind speed in the city was found to average 60% of that at the airport. No significant directional effect was apparent.

A representative profile fitted to the city data is $\alpha = .36$, $Z_G = 1300\ ft$. These results appear to be quite consistent with previously assumed values given in Table 2.

Recent theoretical investigations

A number of promising theoretical investigations have been offered in recent years by Lettau[25], Blackadar[26] and others. These investigations set out to deduce the theoretical Ekman wind profiles over different terrain taking into account both the longitudinal and transverse shear (introduced by the convergence in the Ekman layer induced by the geostrophic layer). These depend on assumptions regarding the coefficients of eddy viscosity, and at this point in the analysis, some empiricism is required. In view of the fact that the numerical definition of these coefficients is not straightforward at the present time, there do not appear to be any advantages for practical applications in the use of these theoretical profiles in preference to the much simpler power law profiles, suggested above.

The theoretical profiles depend on the Rossby number which are for any roughness of surface slightly dependent on the latitude and wind speed. Within the important ranges of wind speed and latitude the departures from the power law profiles do not appear large.

Effect of changes in roughness on the mean wind velocity profiles

When the wind blows from a smooth to a rough surface (or vice versa) a change in the wind speed profile takes place. It is frequently useful to determine the fetch required to build up the mean velocity profile characteristic of the new surface. This is useful both assuming the likely wind velocity profile at a structure built near the outskirts of a city surrounded by open country and in estimating the influence of terrain at an anemometer site with a non-homogeneous exposure.

In the previous paper[14], an approach by Taylor[27] was reported. Since that time, a somewhat more satisfactory approach has been put forward by Townsend and Panofsky[28]. This indicates somewhat steeper interface layers than Taylor and suggests that the new profile establishes itself at roughly a 1/10 slope. Since significant differences in velocity exist mostly in the first 500 ft., a mile downwind of the change in roughness should be sufficient to produce most of the significant changes to the profile. The change in roughness takes place slightly faster when the wind blows from a smooth surface to a rough than in the opposite direction.

Although the profile corresponding to the new surface establishes itself quite rapidly according to the Townsend-Panofsky theory, accelerations in the flow are still

noticeable 10^5 roughness lengths downstream of the change in roughness.

Properties of turbulence

A knowledge of the turbulence properties is required both for the analytical determination of the dynamic response of structures to gusts and for the correct wind tunnel modelling of turbulence. For some years, structural engineers have made allowances for gusts in design by using maximum gust velocities in their designs and assuming that these act in a quasi-static manner. While not infrequently, this method leads to effective pressures which are not unreasonable, in fact, the approach appears inadequate in several respects. A major objection is that it embodies a physically unrealistic concept of how gusts really affect structures.

Structures respond far less to the intensity of an individual gust than they do to the energy contained in sequences of gusts, in particular those fluctuating components of the gusts which are resonant with the structure. A further important aspect of gusts is their spatial organization. It is a well recognized fact that the high velocities associated with a gust prevail only over local regions of the structure.

To describe these properties of gusts, we cannot do better than to make use of the statistical theories of turbulence. According to this, we can describe the fluctuations using:

1) The spectrum of turbulence;
2) The cross-correlations (or cross-spectra) of the velocity fluctuations at different points; and
3) The probability distributions of the velocity components.

From the viewpoint of wind loading of structures, probably the most important power spectrum is that of the longitudinal component since this gives rise mainly to the fluctuations in drag. However, in tall structures[7], the lateral component can also contribute to the lateral fluctuations and in bridge decks the vertical component of velocity can give rise to an important and somewhat unexpected lift force on the deck[29,30].

The forms of the various spectra were discussed in the previous paper[14]. An empirical form was suggested for the spectrum of wind speed:

$$\frac{n\ S(n)}{K\ \overline{V}^2_{10}} = 4\ \frac{x^2}{(1+x^2)^{4/3}} \qquad (2)$$

in which $S(n)$ is the power spectral density at frequency n;

K is the surface drag coefficient referenced to the velocity at 10 m height;

$\overline{V}_{10}$ is the reference mean velocity at 10 m height; and

x is a non-dimensional frequency.

We define x by

$$x = \frac{nL}{\overline{V}_{10}} \qquad (4)$$

where L is a scale length found to be of the order of 4000 ft.

Another quite similar (and possibly better expression for the spectrum) has been put forward by Harris[31] in which the right hand side is of the form:

$$constant\ \frac{x}{(1+x^2)^{5/6}}$$

Under some circumstances, this may be a less attractive form for integration; it does, however, yield a finite value for the power spectral density at zero frequency.

More recently, it seems that for a wide range of heights in the boundary layer there are some advantages to using a more flexible expression for the spectrum involving the variance σ^2 rather than the square of the friction velocity V_*^2 (equal to $K\overline{V}^2_{10}$), and using a flexible scale length. A tentative form is:

$$\frac{n\ S(n)}{\sigma_z^2} = \frac{2}{3}\ \frac{x^2}{(1+x^2)^{4/3}} \qquad (4)$$

$$\text{where } x = \frac{n\ L_z}{\overline{V}_z} \qquad (5)$$

The peak of this spectrum is at $x = \sqrt{3}$; thus, the required wave length L_z is equal to $\sqrt{3}$ times the wave length at the peak. A study by Berman[32] (which made use of material in the earlier paper) indicated that

$$\lambda_{peak} = \left(\frac{\overline{V}_z}{n}\right)_{peak} = 200\ z^{.25}\ metres \qquad (6)$$

(In fact, since the mean velocity varies with height approximately according to $z^{.25}$; this expression implies that if a standard reference velocity is used in defining the peak -- for example, that at 10 m height -- the peak wave length is more or less invariant with height. This result was concluded previously.)

Although much additional information has since become available on the wind spectrum (and, in fact, is to be discussed at this conference), space does not permit a lengthy discussion. However, there are some results of particular relevance to the wind load problem, namely, information obtained of the wind speed spectra in cities. These results are shown in Fig. 7 and 8, and include New York, Montreal and St. Louis.

The New York data, which is obtained on two buildings at heights of 580 ft. (N.Y. Telephone Building) and 920 ft. (40 Wall St. Buildilng) above ground are taken over probably the most heavily built up city region in the world. In spite of this, the spectra are relatively consistent with one another, and as is seen in Fig. 9 to agree in general form with the empirical spectrum suggested above. One distinct difference of these spectra is the peak wave length. A comparison of the observed peak wave number and that predicted according to the above relationship in Equation (6), is given below:

Comparison of peak wave length of wind speed spectra in New York

	Ht. ft.	Ht. m	Peak wave length (m) Observed	Peak wave length (m) Predicted Eq. (6)
N. Y. Tel. Bldg.	580	177	4000	720
Wall St. Tower	920	280	5000	820

In contrast, the well defined peak of the Montreal data in Fig. 8 coincides closely with the predicted peak found for other spectra. The St. Louis data is not sufficiently well defined to consider in detail, although it indicates a somewhat longer peak wave length.

Although in the vertical spectrum the peak wave length, as discussed in the previous paper[14], appears to have a definite dependence on the height, the longitudinal spectrum does not appear to have any consistent dependence. What then is the longitudinal scale dependent on? This is a baffling question, but it appears to have been partially answered by some recent interesting work by Faller[34].

Following laboratory experimental work on the stability of a laminar Ekman layer, Faller has stated that:

> "Large eddies in a turbulent Ekman layer should take the form of stationary (or slowly moving) horizontal roll vortices oriented in the general direction of the wind, but at a small angle to the left of the geostrophic flow (northern hemisphere). Their horizontal wave length should be 1 km or greater dependent on the geostrophic speed and latitude."

The governing parameter in Faller's analysis is the turbulent Reynolds number:

$$Re_t = \frac{U}{\sqrt{\Omega\ \nu_t}} \qquad (7)$$

where U is the geostrophic wind speed;

Ω is the effective angular velocity at the latitude due to the earth's rotation ($7.27 \times 10^{-5} \sin\phi$ rad/sec at latitude ϕ);

ν_t is the effective turbulent eddy viscosity.

The wave length of the eddies is found to be approximately 11D where D is $\sqrt{\nu_t \Omega}$. From this, it is clear that the wave length will be larger for localities for which the eddy viscosity is greater -- that is in rougher regions. To some extent also, the scale is dependent on latitude. While these observations still require further investigation, it would seem here that there are some interesting indications as to the source of the very large longitudinal eddy scales in the atmosphere (many times the height) and some explanation as to the range in sizes of the scale. Another quotation from Faller[34] indicates the relation between the motions in the atmospheric and wind tunnel boundary layers.

> "From experimental studies, large eddies are a rather general characteristics of 2-dimensional turbulent shear flows. Inasmuch as boundary layer instability in rotating systems is similar to that of 2-dimensional flows, it may be expected that the turbulent boundary layer of the atmosphere will have some form of large-eddy structure. It is suggested ... that the large eddies will have characteristics similar to those of the vortex motions which occur when the corresponding type of laminar flow becomes unstable."

The second aspect of turbulence that is of importance is the correlation of velocity over different spatial separations. This is sometimes expressed by the cross spectrum (real and imaginary parts) and sometimes by the cross-correlation at individual wave numbers. The latter form seems more manageable for wind loading applications. This cross correlation (again it has real and imaginary components) is expressed as:

$$R(x,x';\frac{n}{V}) = \frac{iQu(x,x';\frac{n}{V}) + Co(x;x';\frac{n}{V})}{\sqrt{S(x;\frac{n}{V})\ S(x';\frac{n}{V})}} \qquad (8)$$

where $n/\overline{V}$ is the wave number;

x, x' are two spatial coordinates;

Qu is the quadrature spectrum (out-of-phase component) of the cross-spectrum;

Co is the co-spectrum (in-phase component) of the cross spectrum; and

S is the point spectrum

For reasons of symmetry the quadrature spectrum between similar velocity components is usually zero for points in the same horizontal plane. For vertical separations, however, Qu is non-zero, although not usually as significant as Co. A useful indication of the cross correlation is given by the coherence which is defined by

$$\sqrt{coherence} = \left|R(x,x';\frac{n}{V})\right| \qquad (9)$$

Information on the form of the cross-correlation spectrum is to be presented at this conference. Present approaches are that the vertical correlation can be expressed by the relation

$$\sqrt{coherence} = e^{-k_z \frac{n\Delta z}{\overline{V}_z}} \qquad (10)$$

or

$$= e^{-k_z \frac{n\Delta z}{\overline{V}_1}} \qquad (11)$$

where Δ_z is the vertical separation;

n is the frequency;

and k_z, k_1 are coefficients defined according to whether the mean velocity $\overline{V}_z$ or $\overline{V}_1$ (a reference velocity is used).

In Fig. 10, recent results obtained in a rough urban area, St. Louis, are presented.

A summary of various measured values is given in Table 6 below.

The third property of turbulence which is of interest is the probability distribution. No results are presented here, but measurements by Shiotani[35], Singer[37] and others have generally confirmed the widely held assumption that these are normally distributed.

TABLE 6

Coefficient of Exponential Decay of $\sqrt{coherence}$

Definition of k_z and k_{10} $\sqrt{coherence} = exp(-k_z \frac{n\Delta z}{\overline{V}_z})$

$$= exp(-k_{10} \frac{n\Delta z}{\overline{V}_{10}})$$

	α	Ht.ft.	k_z	k_{10}	Reference
Brookhaven	.30	300		6	Davenport[24]
Tokyo tower	.33	830	10 →	(5)	Soma[22]
St. Louis	.25	455		6	-
Savannah River	.17	800	9 →	(6)	-
London, Ont. (CFPL Tower)	.17	700		6	-
Honshu/Shikoku	.17	495		6	Shiotani[35]
Sale	.16	500		7.7	Davenport[33]
Wind tunnel	-	-		8	Davenport[36]

THE CLIMATE OF MEAN WIND STATISTICS

General

As remarked earlier, the second major question relating to the wind is associated with establishing the statistics of the wind climate at the site of the structure. In this context, it is assumed the wind structure is established and what is required is the properties of a reference mean wind velocity.

Two types of statistics are of interest; those relating to the total population of wind speeds and second the properties of the extremes. In the following, we will discuss both these questions and attempt to find a relation between the two.

Direction and speed distributions of wind can occasionally be obtained from published meteorological data for observing stations; in other instances, these can be computed from past records. An example of such a distribution is shown in the upper part of Fig. 11.

While information of this kind is in a form well suited for integrating with wind tunnel studies, it is frequently more feasible to deal with the speed distribution alone. Justification for this simplification can be found to the extent that in the analysis the response of the structure is integrated with respect to direction.

The speed distribution corresponding to the speed and direction is shown below in Fig. 11. The distribution is compared with a Rayleigh distribution. The reason for the selection of this distribution is now considered.

The simplest model that might be considered is to regard the large scale atmospheric motions causing the wind as a two dimensional turbulent motion. Initially it is assumed that the wind is isotropic and there is no "prevailing" wind.

Let the components of the velocity in the east-west and north-south directions be u and v respectively. Assume that the distributions of these components have a Gaussian

distribution (characteristic of most turbulent motion) so that the probability of obtaining a velocity having the components between u and $u + du$ and $v + dv$ is:

$$P(u,v).du.dv = \frac{1}{2\pi\sigma_x\sigma_y} \; exp - \{\frac{u^2}{2\sigma_x^2} + \frac{v^2}{2\sigma_y^2}\} \; du \; dv \qquad (12)$$

where σ_x and σ_y denote the standard deviations of the velocity components. Because it has been assumed that the wind is isotropic:

$$\sigma_x = \sigma_y = \sigma \; (say) \qquad (13)$$

So
$$P(u,v) \; du \; dv = \frac{1}{2\pi\sigma^2} \; exp - \{\frac{u^2 + v^2}{2\pi^2}\} \; du \; dv \qquad (14)$$

This distribution is a curve the shape of the crown of a Panama hat with its axis through the centre as shown in Fig. 12. Now consider the magnitude of the wind speed V. This is given by:

$$V^2 = u^2 + v^2 \qquad (15)$$

If we now consider the distribution of speed in the region V and $V + dV$ and in the sector between θ and $d\theta$, the probability of obtaining a velocity between V and $V + dV$ is:

$$P(V) \; dV = \frac{1}{2\pi\sigma^2} \int_0^{2\pi} e^{-\frac{V^2}{2\sigma^2}} .V \; dV. \; d\theta$$

$$= \frac{1}{\sigma 2} \; V \; e^{-\frac{V^2}{2\sigma^2}} \; dV \qquad (16)$$

This is a Rayleigh distribution and has the shape indicated in Fig. 11.

The probability distribution is:

$$P(V) = \int_0^\infty P(V)\, dV = e^{-\frac{V^2}{2\sigma^2}} \qquad (17)$$

This model is somewhat oversimplified. The peak of the distribution of velocity in Fig. 12 is likely to be shifted by a "prevailing wind speed". Furthermore, the assumption of isotropy of the wind is not likely to be realized, partly due to geographical influences. If these directional characteristics are allowed for, the probability density can be modified to the form:

$$P(V)\, dV = A \int_0^{2\pi} V\, e^{-V^2 f(\theta,\, V)}\, dV\, d\theta \qquad (18)$$

where A is a constant and $f(\theta,V)$ is a function defining the anisotropy of the wind distribution. The influence of the mean velocity and directional characteristics are, it appears, usually slight. As a consequence, it is possible to use another distribution, the Weibull distribution, which is of similar form to the Rayleigh.

$$P(>V) = e^{-(\frac{V}{c})^k} \qquad (19)$$

The modified exponent allows additional flexibility in the distribution which appears to be adequate to take care of the influence of mean velocity and directional characteristics. To demonstrate the suitability of this theoretical distribution a number of observed distributions (including examples from Tagg[38] and Lappe[39] et al) have been plotted in Fig. 13. The double logarithmic transformation has been used to indicate conformity by a straight line.

$$\frac{1}{k} \ln\{-\ln P(V)\} = \ln V - \ln c \qquad (20)$$

It is noticed in the diagram that the exponent in all cases, is close to 2 indicating that the Rayleigh distribution is approximately correct. The linearity of the distributions in all cases appears excellent, so is consequently the conformity with the Weibull distribution.

Extreme Values of Wind Speed

At this point, it is worthwhile to consider the relation between the distribution of the extreme values of wind speed with the parent distribution. We shall assume the parent distribution is Weibull.

A straightforward approach results from extreme value theory. Suppose the wind is broken down into N separate values in a year (or some other convenient time interval). The distribution of the largest of these N values is as follows. Since the Weibull distribution is of the exponential type, it is readily shown the extreme value distribution is of the double exponential type.

$$P(V) = e^{-e^{-a(V-U)}} \qquad (21)$$

in which a and U are parameters found below.

The modal value U is the expected value and given by:

$$P(U) = \frac{1}{N} \tag{22}$$

or on substituting for $P(U)$:

$$U = c\,\{ln\ N\}^{1/k} \tag{23}$$

The value of the dispersion factor $1/a$ is given by

$$1/a = \frac{dU}{d\,ln\ N} = \frac{c}{k}\,\{ln\ N\}^{1/k-1} \tag{24}$$

For the Rayleigh distribution $c = \sqrt{2}\sigma$ and $k=2$

So $$U = \sigma\sqrt{2 ln N} \tag{25}$$

$$\frac{1}{a} = \frac{\sigma}{\sqrt{2 ln N}} \tag{26}$$

A difficulty found in adopting this approach is that extreme value theory assumes that the N sample values are independent of one another. In the case of wind, this is not altogether true since the speeds in one sample period are not uncorrelated from one another. The effective number of uncorrelated sample periods is, therefore, somewhat smaller than the total number. To assess this difficulty, we can consider the distribution of extremes from a different viewpoint -- by considering the wind speed as a continuous random process the parent distribution function of which is Rayleigh.

It is shown in the Appendix B that, in this case, the largest values of such a process in period T is again of the double exponential form:

$$P(>V) = e^{-e^{-a(V-U)}} \tag{27}$$

where in this instance:

$$U = \sqrt{2 ln \nu T}\,\{1 + \frac{ln\sqrt{\pi ln \nu T}}{2 ln \nu T - 1}\}\sigma \tag{28}$$

and

$$\frac{1}{a} = \{\sqrt{2 ln \nu T} - \frac{1}{\sqrt{2 ln \nu T}}\}^{-1}\sigma \tag{29}$$

In this ν is the effective cycling rate of the process and given by

$$\nu = \frac{\sigma'}{\sigma} \tag{30}$$

$$= \left[\frac{\int_0^\infty n^2\, S(n)\, dn}{\int_0^\infty S(n)\, dn}\right]^{1/2} \tag{31}$$

wherein $S(n)$ is the power spectrum of the process at frequency n.

We can obtain a value for ν (which only requires to be approximately correct) from the spectra of wind speed over the very low frequency range. Referring to the spectrum by Singer and Raynor in Fig. 4, it is found that for 15 minute averaged wind speed $\nu = .10$ *c.p.h.*

Comparing equations (25) and (26) with equations (28) and (29), it is seen that for long time periods (or large numbers of cycles) that the effective number of samples:

$$N_{eff} = \nu T \qquad (32)$$

If we choose T as one year, (8760 hours) the effective number of samples is then 876 compared to the total number of 15 minute samples of 35,000.

For the Rayleigh distribution, using the value of 876 for N_{eff} we find:

	exact(eq. 28/29)	*approx.(eq. 25/26)*	*Ratio: exact/approx.*
$(\frac{U}{\sigma})$	4.11	3.68	1.12
$(\frac{1}{a})/\sigma$	.294	.272	1.08
$\frac{1}{a}/U$	.072	.074	1.03

It appears, therefore, that approximate methods underestimate the extremes by approximately 10%. Unfortunately, to carry out an exact analysis for the Weibull process is more difficult. However, in view of the similarity of the Weibull form to the Rayleigh a similar adjustment to that used for the Rayleigh would appear justifiable.

For the New York data given in Fig. 13, the values of c and k are 25.5 mph and 1.87 respectively; these lead to the following values for U and $\frac{1}{a}$:

	Approximate	*Adjusted*
U mph	71	78
$\frac{1}{a}$ mph	5.6	6.2
$\frac{1}{a}/U$	.08	.08

Figures are not available for an exact comparison of these predicted extreme wind parameters with observations of extremes taken from the same data. A comparison can be made, however, using results obtained from an analysis carried out by the writer (in collaboration with the structural engineers Skilling, Helle, Christiansen, and Robertson) on extreme winds at gradient height on the Atlantic Coast of New England and the Canadian Maritimes. In this instance, extreme gradient wind speeds were estimated from anemometer observations at the surface and extrapolated upwards by means of a roughness factor (K_A) evaluated from a qualitative description of the terrain at the anemometer station. This approach was described by Davenport[40]. The results of this analysis are given in Tables 7 and 8.

TABLE 7

Estimation of Extreme Gradient Wind Speeds* at Canadian Maritime Stations

Station	Roughness Category	Anemometer Ht. Ft.	K_A	U mph	K_AU mph	(1/a)/U
Goose Bay	B-C	47	1.8	39.1	70.5	.094
Cartwright	B	29	1.75	43.0	75.5	.063
Gander	B-C	61	1.75	48.2	84.0	.107
Torbay	B	45	1.6	54.0	86.5	.121
Cape Race	A	45	1.35	61.4	83.0	.122
St. John, N.B.	B-C	41	1.80	38.1	68.5	.096
St. Paul Is.	A-B	43	1.50	54.0	81.0	.063
Sable Island	A-B	40	1.5	55.1	82.6	.088
Moncton	B-C	80	1.70	45.3	77.0	.137
Sydney	B	65	1.55	49.3	76.5	.095
Charlottetown	B-C	72	1.70	44.3	75.4	.121
Halifax	B-C	35	1.70	39.9	67.8	.180
				Averages:	79 mph	.010

*Hourly Average Wind Speeds

TABLE 8

Estimation of Extreme Gradient Wind Speeds* at East Coast Stations of United States

Station	Roughness Category	Ht. Ft.	K_A	U mph	$K_A U$ mph	$(1/a)/U$
Eastport, Maine, City	B	76	1.6	53.7	87	.131
Portland, Me., City	D-C	117	2.3	43.2	100	.113
Portland, Me., Airport	B-C	55	2.0	51.2	103	.207
Boston, Mass., City	D	188	2.3	42.0	97	.123
Boston, Mass., Airport	C-B	62	1.8	55.1	99	.220
Providence, R.I., City	C	251	1.6	57.8	93	.145
Providence, R.I., Airport	B-C	60	2.1	49.2	103	.167
Nantucket, Mass., City	B-C	90	1.7	58.2	99	.175
Nantucket, Mass., Airport	B	35	1.7	56.7	97	1.48
Block Island, R.I., City	B	46	1.6	59.6	95	.126
La Guardia, N.Y., Airport	B-C	82	1.6	59.3	95	.094
Trenton, N.J., City	C-D	107	2.4	42.7	102	.138
Atlantic City, N.J., City	B-C	172	1.5	63.3	96	.123

*Fastest Mile Wind Speeds

It should be noted that observations for the Canadian Stations are hourly average velocities while those for the New England Coast are "fastest mile". While only small differences between hourly average and 15 min. averages should be expected (Durst's[41] results suggest about 2%) "fastest Miles" are generally 30% higher.

With these remarks in mind, it is seen that the extremes predicted from the parent population are in agreement with the estimates made from surface observations, except for the

somewhat greater scatter of the "fastest mile" extremes. The latter is to be expected both because of the short duration of the record and because the estimation of "fastest mile" speeds is prone to inaccuracy.

While these approaches still require further study, the indications given above are promising. If extremes can be successfully predicted from a knowledge of the parent population it implies that estimates of more frequently occurring winds are also likely to be reasonable.

There are other advantages. A primary one is the possibility of estimating extremes from much shorter periods of observation: two or three years record is frequently sufficient to establish the parent distribution of wind speeds. With planning this would enable useful observations to be made within the normal period available for the design of a major structure. A further advantage is that greater use can be made of balloon data obtained routinely at airports over the last few years. Since these balloon observations are free from the very marked and often indeterminate influence of local terrain, it is to be hoped the properties of these observations would provide consistent results over a very much wider region.

One further interesting fact emerges from the above analysis, this concerns the form of the extreme value distribution. Two types of extreme value distributions are in use -- the Fisher Tippett type I and type II. The former is the double exponential form given in equation (27) and the second is similar but with *lnV* written for *V*; in fact, it turns out it is equivalent to the Weibull distribution. The type I has been used by Boyd[43] in Canada, Shellard[44] in England, Whittingham[45] in Australia, Court[50] in the U.S.A., and Johnson[51] in Sweden. The Type II has been preferred by Thom[46,47] in the United States and Anapolskaia and Gandin[48] in the U.S.S.R.

Thom's reason for preferring the latter distribution is that in theory, the Type I distribution assumes positive as well as negative values while wind speed observations can only assume positive values.

On the whole, the distributions are similar for relatively frequent occurrence intervals and because of the scatter observations do not appear particularly to favour either distribution. Projected to higher wind speeds, however, the Type II distribution leads to significantly higher wind speeds than the Type I.

The reason for favouring the Type II distribution put forward by Thom has been commented on by Gumbel[49] who states:

> "From a practical standpoint this argument is not as strong as it looks offhand. Many observations of a positive variate are usually and successfully analyzed by the normal distribution, which is unlimited in both directions. This procedure is legitimate provided that the probabilities for negative values (which do not exist) are so small, say of the order 10^{-7}, that their occurrence within the possible number of observations is not to be expected.
>
> "The same argument holds for extreme values. Here experience has shown that the first asymptotic distribution can successfully be used for the analysis of floods, although floods are positive variates. This may also hold for extreme wind speeds."

Gumbel has also stated elsewhere[42]:

> "The asymptotic distributions of the largest values depend exclusively upon the behaviour of the initial distributions toward large values of the variate. The properties of the initial distributions about the median or the small values of the variate are irrelevant."

A more positive reason for preferring the Type I distribution has been given in this paper. It has been proposed that there are fundamental arguments for assuming that the distribution of wind speed is of the Rayleigh or Weibull form -- both of which are exponential. The extreme values of such distributions are of the Fisher-Tippett, Type I[42].

<u>Comparisons of wind speeds over different periods of time</u>

It has been suggested above that an optimum averaging period for wind speed measurements are not always available and use has to be made of measurements over other time intervals.

Studies by Durst[41], Deacon[52], Mitsuta[53], Shellard[44] and others made between extremes of wind speed over different averaging periods.

As Deacon[52] has indicated (and as argued elsewhere by Davenport[54]) the ratio of short term gust speeds to mean speeds are functions both of terrain and anemometer response. For open terrain, the service anemometers (having distance constants of the order of 20 ft.) indicate maximum gusts about 1.5 times the hourly average velocities at standard heights. In the city, factors double this value can be obtained.

Because of the low spectral values in the micrometeorological gap the relation of mean velocities are smaller. The maximum 15 min. average speeds are likely to be only 2-5 percent of excess of the hourly average.

Because of the doubtful validity of designing explicitly for maximum gust speeds, it is believed these ratios are useful only for obtaining insight into the climatological factors.

Influence of intense local storms

In formulating wind statistics, the question has been posed as to whether intense local storms including tornadoes and thunderstorms conform to the wind structure of large scale storms. This is an important question. The treatment of tornadoes is the most serious in terms of potential damage.

Thom[55] has conducted a notable statistical study of the paths and occurrences of tornadoes in the tornado belt of the United States. This suggests the probability of a tornado occurring in a one degree square is approximately once in a 1000 years in an intense part of this belt. This is a probability on order of magnitude different than other storm winds. A design approach based on fail-safe concepts is probably indicated by these recurrence intervals.

The question of thunderstorms is less clear cut. In certain parts of the world, it appears that a significant proportion of maximum gusts arise from thunderstorms. Whittingham[45] in his analysis of Australian wind conditions shows that as much as 50% of maximum winds occur in thunderstorms. These storms may last 5-10 minutes and subside rapidly during which time severe convective turbulence may induce strong gusts. Unfortunately, little is known concerning the abnormalities of the turbulence structure during these storms.

From the design point of view the question is probably best treated by adopting an approach in which the mean velocities are obtained for intervals (10 minutes or so as suggested above) short enough to reflect the higher winds prevalent in the thunderstorm and assume turbulence response characteristic of other major storms.

A pointer which provides some reassurance that this approach is satisfactory is given in Fig. 6B comparing once-in-50-year "fastest miles wind speeds" at city and airport stations (e.g. Omaha, Kansas City) are exposed to high thunderstorm activity yet these extreme speeds exhibit the same wind structure characteristics regarding the effect of terrain as other cities less affected.

Eventually, it may be possible to treat thunderstorms separately and if significantly different properties are found and prove important design accordingly. Thom[47] has treated hurricane statistics separately -- although more from the viewpoint of improving the statistical reliability.

Determination of the wind speed at a site

The above remarks refer namely to the evaluation of wind statistics at a place for which records are available. The problem of translating these statistics to the site of the structure is frequently not straightforward. Highly significant differences can arise due to differences in terrain as, for example, in the design of a structure in a city when wind speed data is derived from airport observations. The modifications required are clear from remarks made above. Another striking example of the influence of terrain, at the site of a major suspension bridge, is indicated in Fig. 14, and is taken from the study by Graham and Hudson[23].

Fig. 14 compares hourly average wind speeds at Chesapeake Bay Bridge with speeds at Baltimore Friendship Airport. The exposure of the anemometer at the Airport station is "more or less uniform rolling terrain in all directions

... with no particular obstruction from any one direction so the underlying frictional surface was assumed to be the same for all directions"[23]. This study indicates that the transverse winds pertinent to the design of the deck, the wind velocities assumed should be 1.6 times the velocity at the airport meteorological office and the longitudinal wind velocities pertinent to the tower design .8 times the airport velocity. Thus the mean speeds appropriate for the design of the deck are 4 times those for the tower.

There appear to be two methods available for evaluating these terrain effects at the site (apart from the use of judgment). These are;

1) site observations;
2) topographic measurements in the wind tunnel.

Observations of the first type have occasionally been carried out, but their full potential has not always been realized. An example of the results of a wind tunnel investigation on a topographic model are indicated in Fig. 15. This model enabled a relationship between the wind speeds at points of observation and at the site of the structure to be determined as well as the wind speed profiles for different wind fetches. It appears to produce consistent and reasonably convincing results in a terrain flanked by high hills.

Finally, it is believed that the establishment of wind speeds at gradient level are a means for bridging between different places without introducing the conflicting effects of terrain.

Concluding Remarks

A few of the observations made in this paper are summarized below.

1) It is believed that the most important action of the wind on structures is associated with the action of repeated loads. It is necessary to derive the wind parameters on which these loads depend.

2) The wind can be defined conveniently in terms of mean wind speeds and gusts related to the mean wind. There are physical grounds for preferring an averaging period between 10-30 minutes.

3) Climatic statistics should, if possible, be expressed in terms of these mean velocities. The influence of gusts is best determined by means other than the use of maximum gust speeds.

4) Mean wind speed profiles are strongly dependent on terrain. Power law profiles suggested previously appear to be supported by more recent research.

5) Turbulence structure is also dependent on terrain. Turbulence spectra appear to be reasonably well expressable in terms of forms suggested previously. The question of scale is still not clarified and the origins of the large eddies not fully explained. Research on the stability of the boundary layer is beginning to shed light on this question.

6) It appears possible to relate the distribution of extreme winds at a site with the overall distribution of winds. This offers promise in terms of defining wind extremes from comparatively short periods of record.

7) The modifying influence of surrounding terrain on the winds prevailing at the site of a structure is highly significant. For large structures, the use of site observations or wind tunnel tests on topographic models yield significant information.

Acknowledgments

The writer is considerably indebted to a number of people who have provided data which has been used in this report. In particular, the following should be mentioned. Dr. R. E. Munn, Dept. of Transport of Canada provided the original wind data for Montreal; Mr. Donald Boyd of the Dept. of Transport provided data on extreme winds at the Canadian Maritime Stations; Mr. H. C. S. Thom provided the observations of extreme winds in the U.S. The assistance of Mr. L. E. Robertson, Dr. Ben Davidson and Mr. I. A. Singer in respect of New York observations is also acknowledged. Mr. Alan Billings assisted in the analysis of results.

APPENDIX A

Sources of data used in Mean Velocity Profiles in Fig. 5

Profile #	Reference
1	16
2	17
3	18
4	56
5	21
6	57
7	58
8	40
9	40
10	40
11	40
12	40
13	4
14	35
15	59
16	60
17	60,40
18	61
19	40,60
20	62
21	40
22	40
23	40
24	40
25	40

N.B. Reference #40 summarizes original data appearing elsewhere.

Appendix B

Distribution of the largest values of a Rayleigh Process

Assume we have a horizontally isotropic wind regime with velocity components u and v with a joint probability density distribution:

$$p(u;v) = \frac{1}{2\pi\sigma^2} e^{-\frac{u^2+v^2}{2\sigma^2}} \qquad \text{(A1)}$$

Suppose also these have rate of change velocity u'_x and u'_y with a probability distribution:

$$q(u';v') = \frac{1}{2\pi\sigma'^2} e^{-\frac{u'^2+v'^2}{2\sigma^2}} \qquad \text{(A2)}$$

The scalar magnitudes of velocity and rate of change of velocity are:

$$V = \sqrt{u^2+v^2}$$

and (A3)

$$V' = \sqrt{u'^2+v'^2}$$

The distribution of V and V' are both Rayleigh and given by:

$$P(V) = \frac{V}{\sigma^2} e^{-\frac{V^2}{2\sigma^2}} \qquad \text{(A4)}$$

$$P(V') = \frac{V'}{\sigma'^2} e^{-\frac{V'^2}{2\sigma'^2}} \qquad \text{(A5)}$$

It can be shown[63] that the expected number of crossings of the value V of a stationary random process is given by:

$$N_V = \int_0^\infty V' \, p(V;V') \, dV' \qquad \text{(A6)}$$

If the velocity and rate of change of velocity are uncorrelated (which is usually the case):

$$p(V;V') = \frac{V}{\sigma^2}\ \frac{V'}{\sigma'^2}\ exp-(\frac{V^2}{2\sigma^2} + \frac{V'^2}{2\sigma'^2}) \qquad (A7)$$

Thus

$$N_V = \frac{V}{\sigma'^2\sigma^2}\ e^{-\frac{V^2}{2\sigma^2}} \int_0^\infty V'^2\ e^{-\frac{V'^2}{2\sigma'^2}}\ dV' \qquad (A8)$$

$$= \sqrt{\frac{\pi}{2}}\ \frac{\sigma'}{\sigma}\ \frac{V}{\sigma}\ e^{-\frac{V^2}{2\sigma^2}} \qquad (A9)$$

Put $\frac{\sigma'}{\sigma} = \nu$ (the effective frequency). (A10)

Following a previous approach in predicting extreme values[54] assume the occurrence of very large values are Poisson.

The probability distribution of the larges value $F(V)$ in a period T is then:

$$F(>V) = e^{-N_V T} \qquad (A11)$$

$$= exp\ -\{\nu\ T \sqrt{\frac{\pi}{2}}\ \frac{V}{\sigma}\ e^{-V^2/2\sigma^2}\} \qquad (A12)$$

Put $\frac{V}{\sigma} = x + \sqrt{2 ln \nu T}$

If the value of νT is large, $x \ll \sqrt{2 ln \nu T}$; that is the distribution is narrow in comparison to the average value of the largest distribution. It then follows we can simplify and write:

$$F(>V) = exp - \ \ exp\ -\{(\frac{V}{\sigma} - \sqrt{2 ln\ \nu T})(\sqrt{2 ln \nu T} - \frac{1}{\sqrt{2 ln \nu T}}) \qquad (A13)$$

$$- ln\ \sqrt{\pi\ ln\ \nu T}\}$$

This is of the form

$$F(>\ ) = e^{-e^{-a(V-U)}} \qquad (A14)$$

where $U = \sqrt{2 ln \nu T}\ \{1 + \frac{ln\sqrt{\pi\ ln\ \nu T}}{2 ln \nu T - 1}\}\sigma$ (A15)

$$\frac{1}{a} = \{\sqrt{2 ln \nu T} - \frac{1}{\sqrt{2 ln \nu T}}\}^{-1}\sigma \qquad A16)$$

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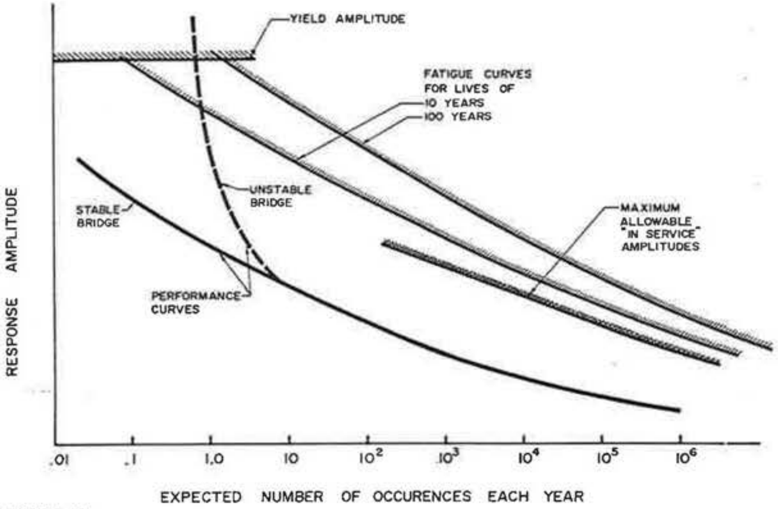


FIGURE 1
COMPARISON OF RESPONSE HISTORY AND DESIGN LIMITATIONS FOR SUSPENSION BRIDGE

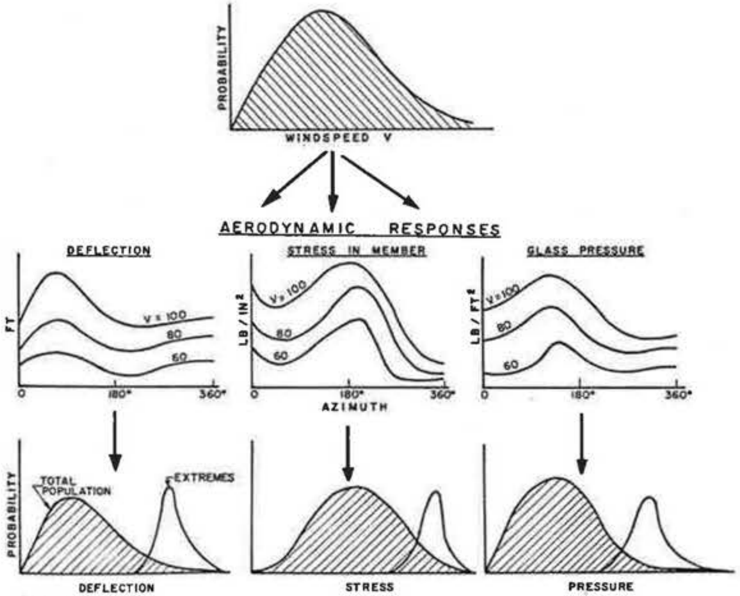


FIGURE 2
DETERMINATION OF PROBABILITY DISTRIBUTIONS OF STRUCTURAL RESPONSE

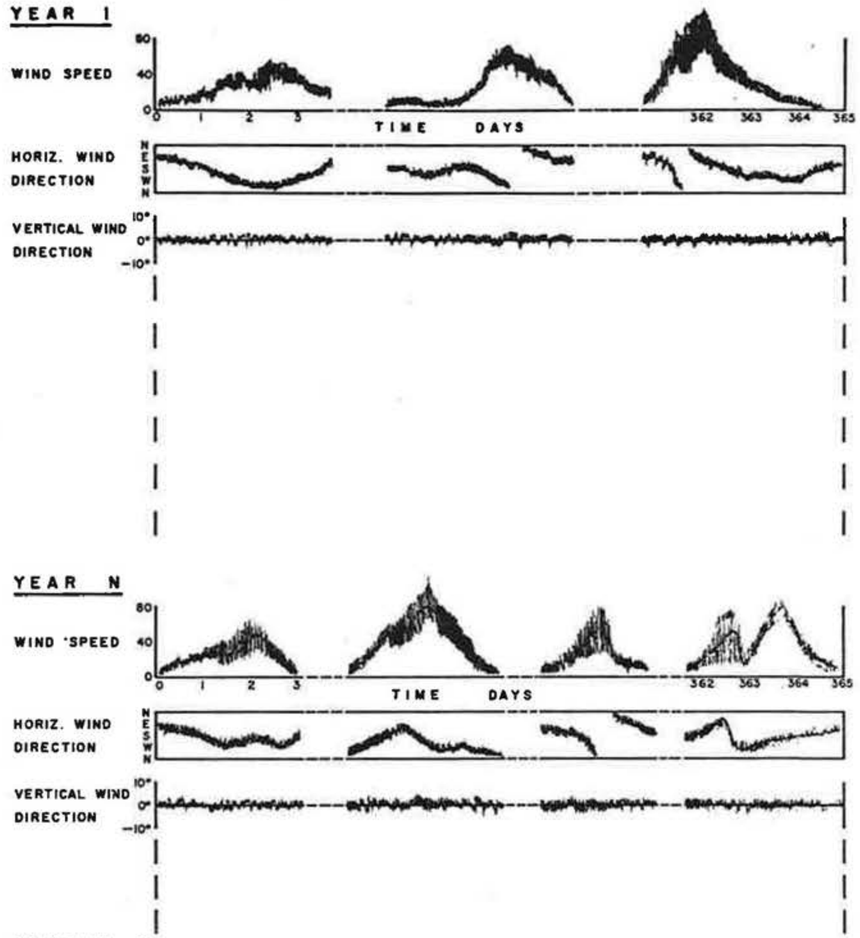


FIGURE 3

A RECORD OF WIND AT THE SITE OF A STRUCTURE

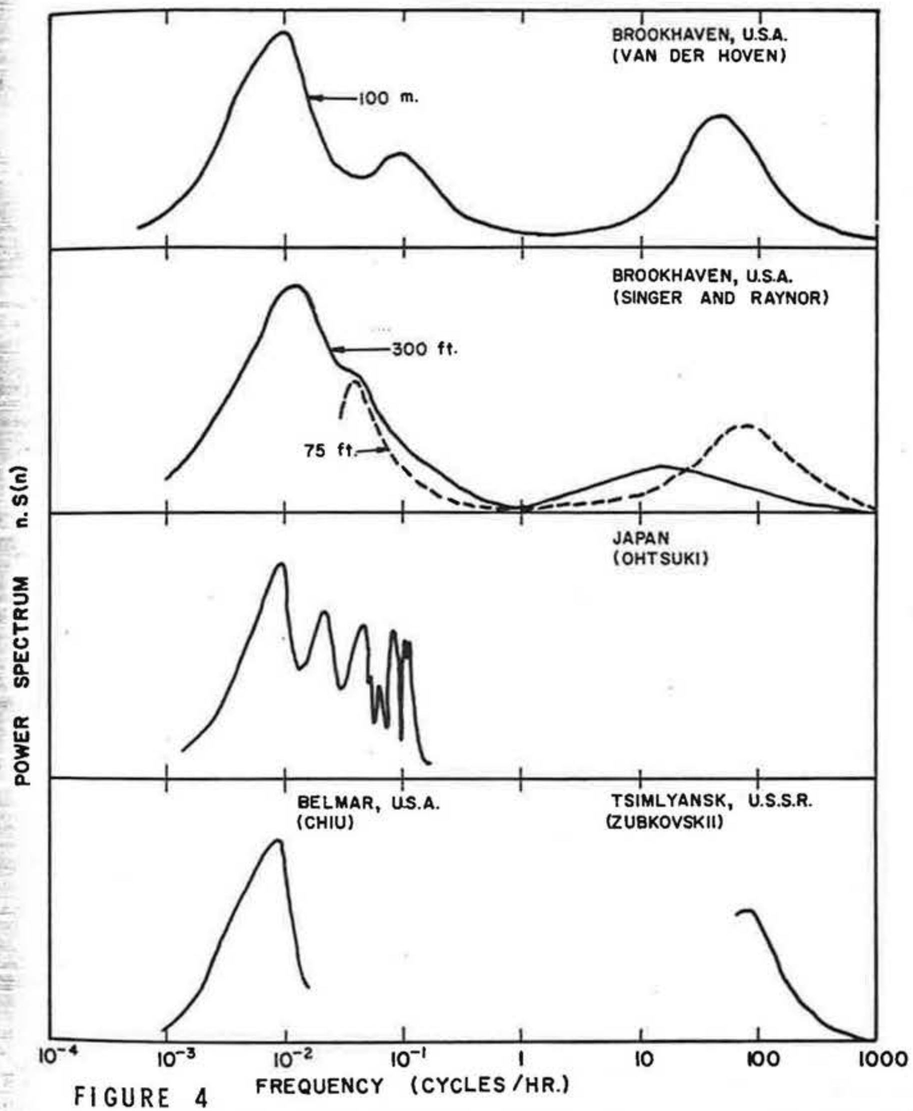


FIGURE 4

SPECTRUM OF WIND SPEED OVER EXTENDED FREQUENCY RANGE

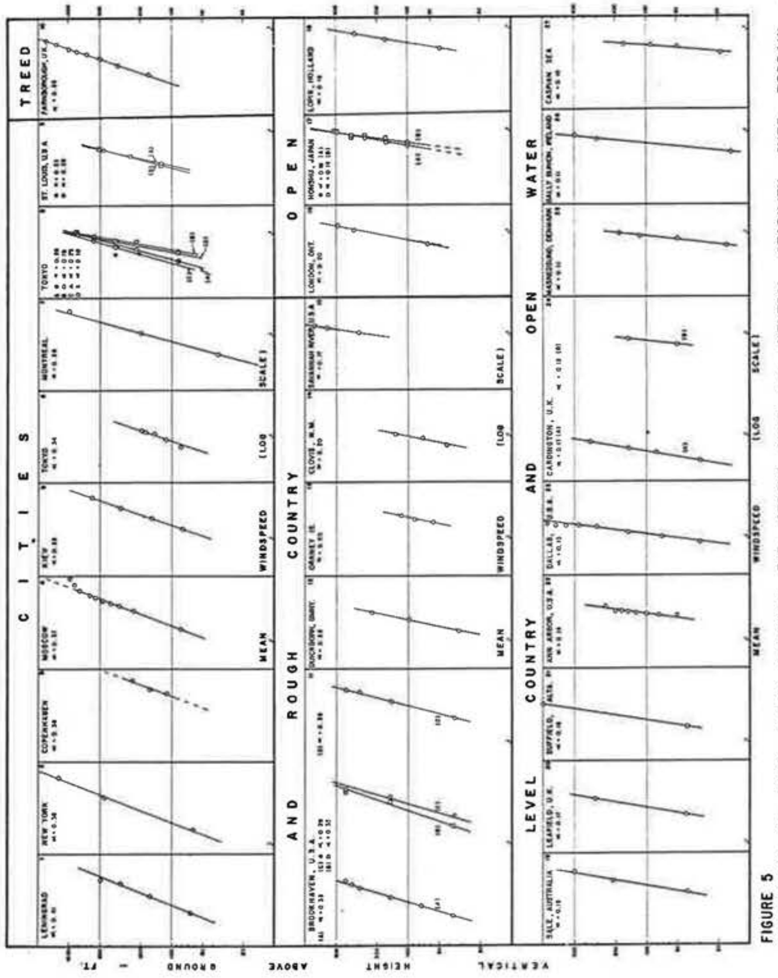
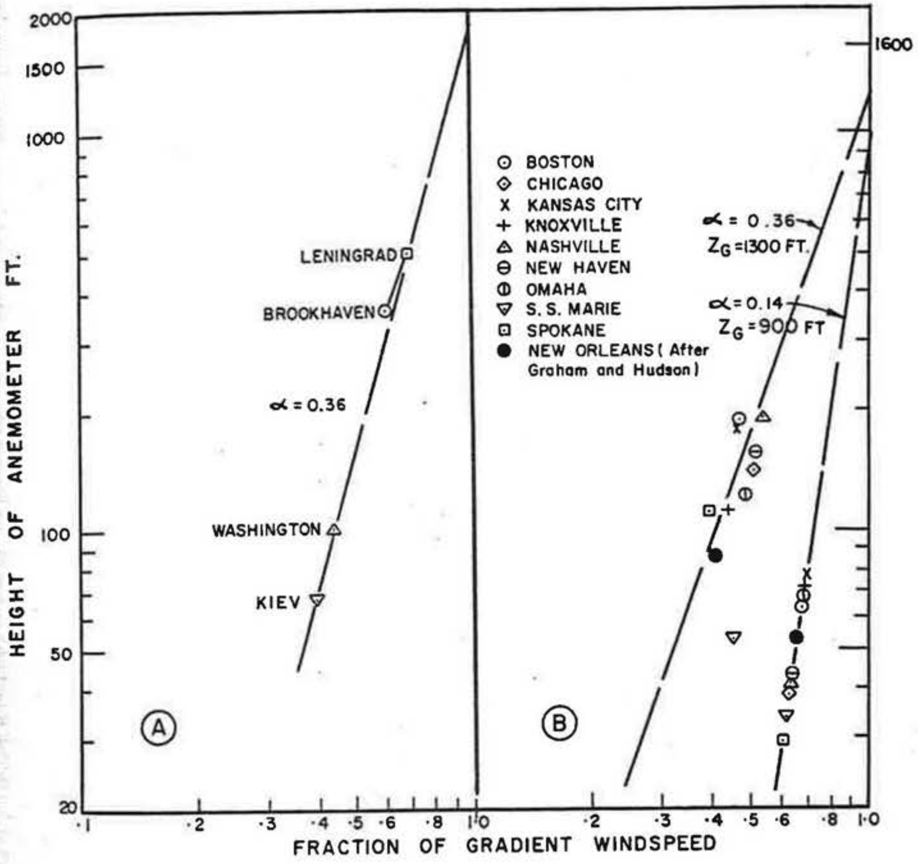


FIGURE 5
COMPARISON OF MEAN SPEED PROFILES FOR STRONG WIND OR NEUTRAL STABILITY OVER TERRAIN OF DIFFERING ROUGHNESS



(A) OBSERVED RATIOS OF SURFACE TO GRADIENT WIND SPEED OVER ROUGH TERRAIN

(B) COMPARISON OF ONCE-IN-50-YEAR FASTEST MILE WINDSPEEDS AT CITY AND AIRPORT STATIONS FOR U.S. CITIES ASSUMING AIRPORT OBSERVATIONS LIE ON OPEN COUNTRY PROFILE

FIGURE 6
COMPARISON OF SURFACE AND GRADIENT MEAN WINDSPEED

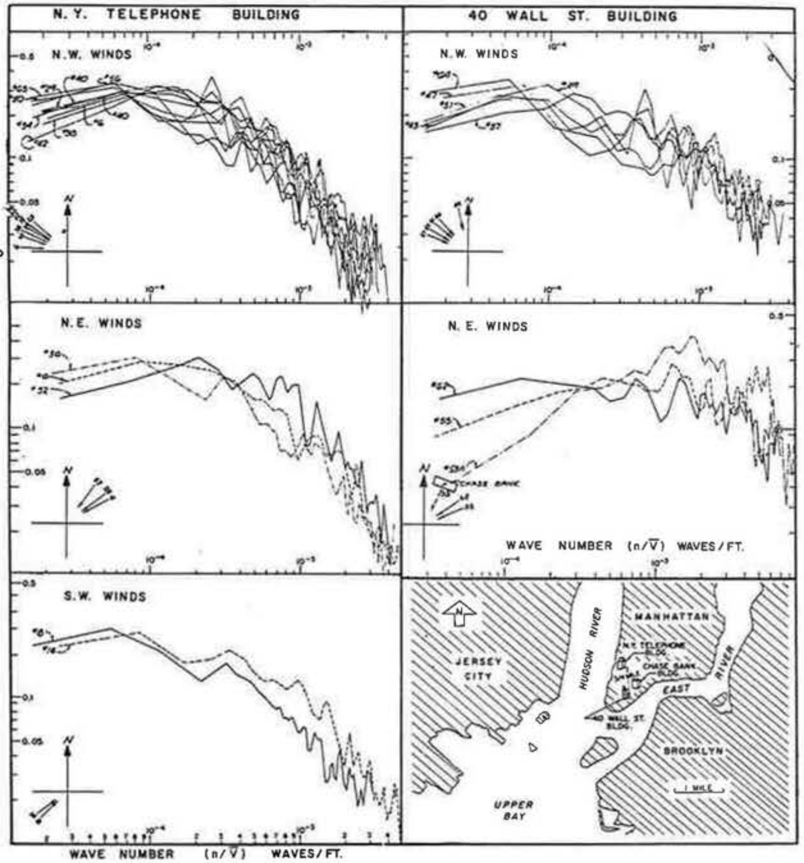


FIGURE 7 SPECTRA OF SPEED FOR STRONG WINDS AT TWO LOCATIONS IN MANHATTAN

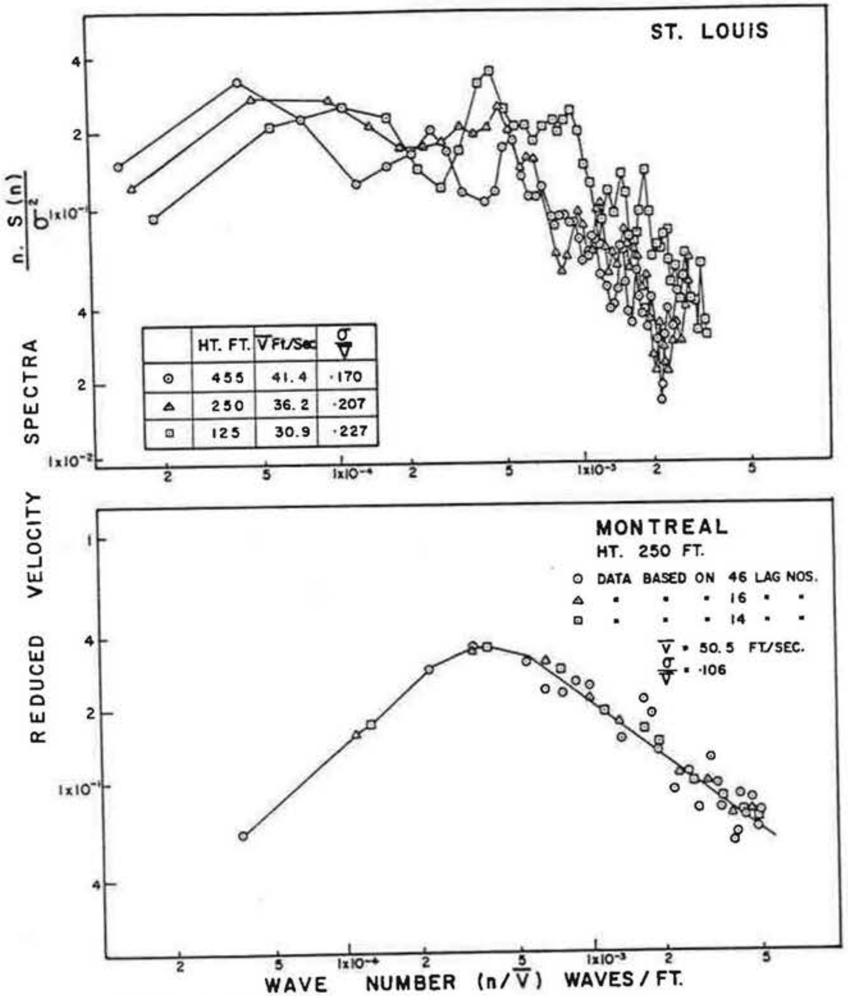


FIGURE 8
SPECTRA OF WIND SPEED IN CITIES IN MODERATELY STRONG WINDS

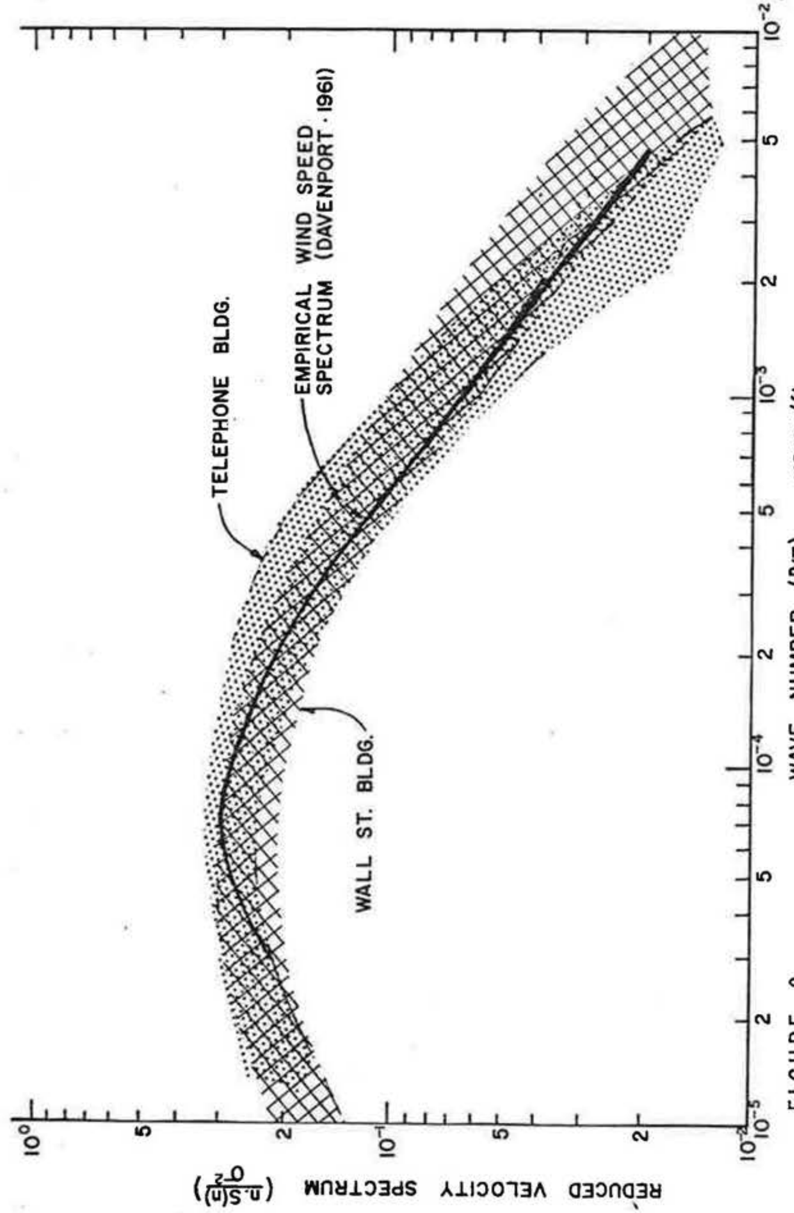


FIGURE 9
COMPARISON OF MEASURED WIND SPEED SPECTRA AT N.Y. TELEPHONE BLDG AND 40 WALL ST. BLDG WITH EMPIRICAL SPECTRUM

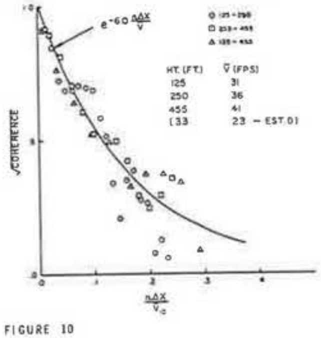
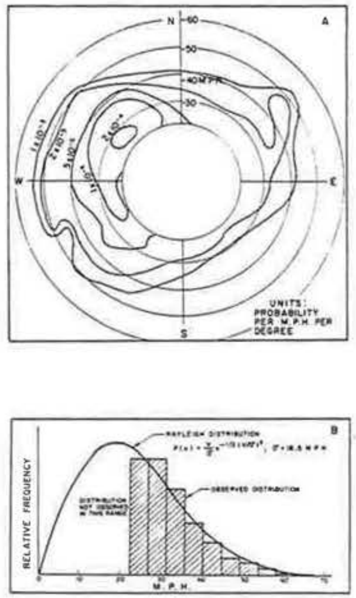


FIGURE 10
MEASUREMENTS OF VERTICAL COHERENCE OF WINDSPEED IN URBAN TERRAIN (ST. LOUIS)



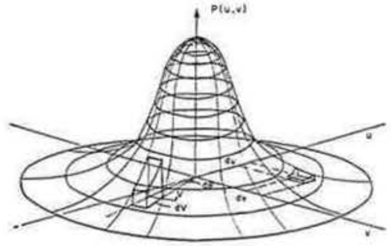
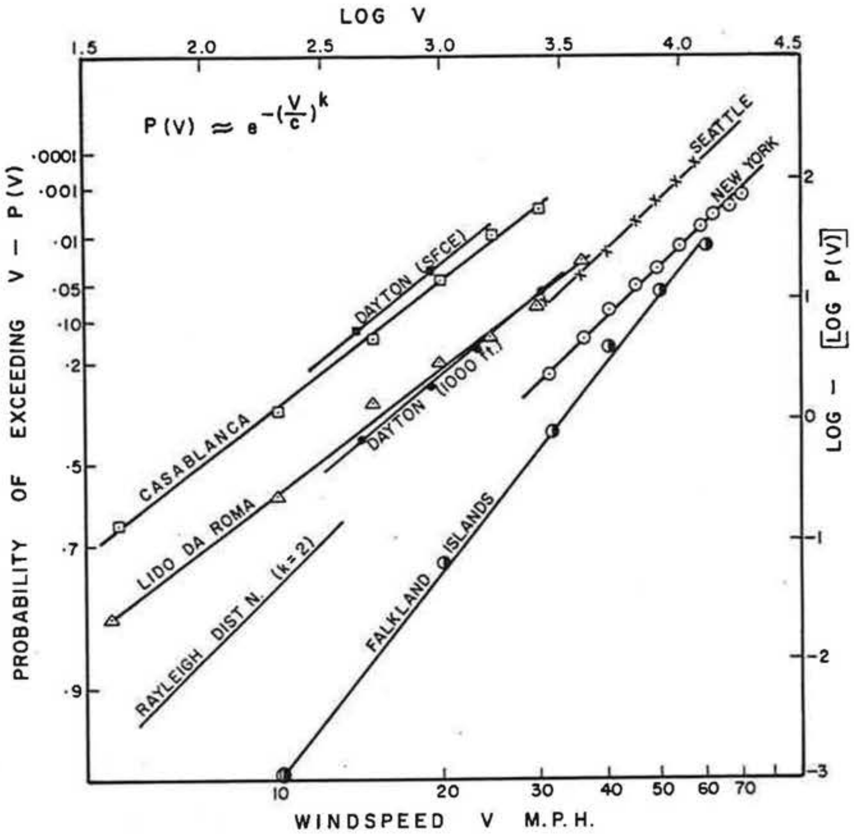


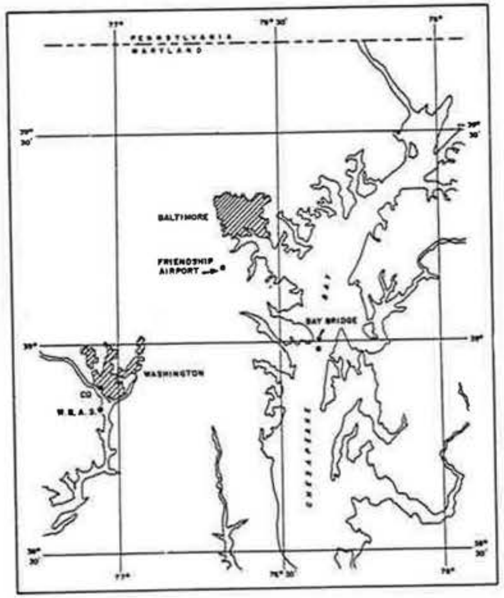
FIGURE 12
PROBABILITY DISTRIBUTION OF WIND VELOCITY IN A HORIZONTALLY ISOTROPIC WIND REGIME

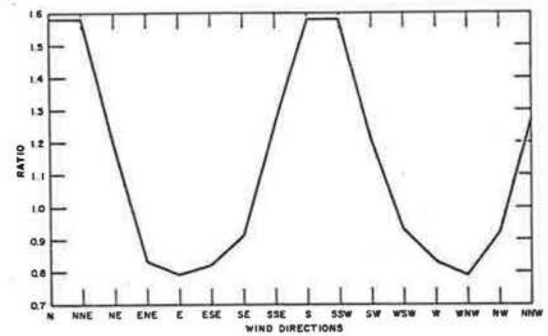
FIGURE 11
PROBABILITY DENSITY DISTRIBUTIONS OF WIND (> 22 M.P.H.) AT 500 M. AT J.F. KENNEDY AIRPORT, NEW YORK
A) WIND SPEED AND DIRECTION
B) WIND SPEED



	c(mph)	k
Dayton (Ohio) Surface	8.8	1.7
Dayton (Ohio) 1000 ft.	15.9	1.7
Casablanca	9.3	1.5
Falkland Islands	33.0	2.5
Lido da Roma	15.2	1.5
New York (J.F.K. airport - 500M)	25.5	1.87
Seattle airport (300M)	19.0	1.9

FIGURE 13
WEIBULL DISTRIBUTIONS OF WINDSPEED





RATIO OF HOURLY AVERAGE 30 FT. WIND-SPEED AT CHESAPEAKE BAY BRIDGE AND BALTIMORE FRIENDSHIP AIRPORT

FIGURE 14
INFLUENCE OF TERRAIN ROUGHNESS ON WIND SPEED AT SITE OF A MAJOR BRIDGE (AFTER GRAHAM AND HUDSON)

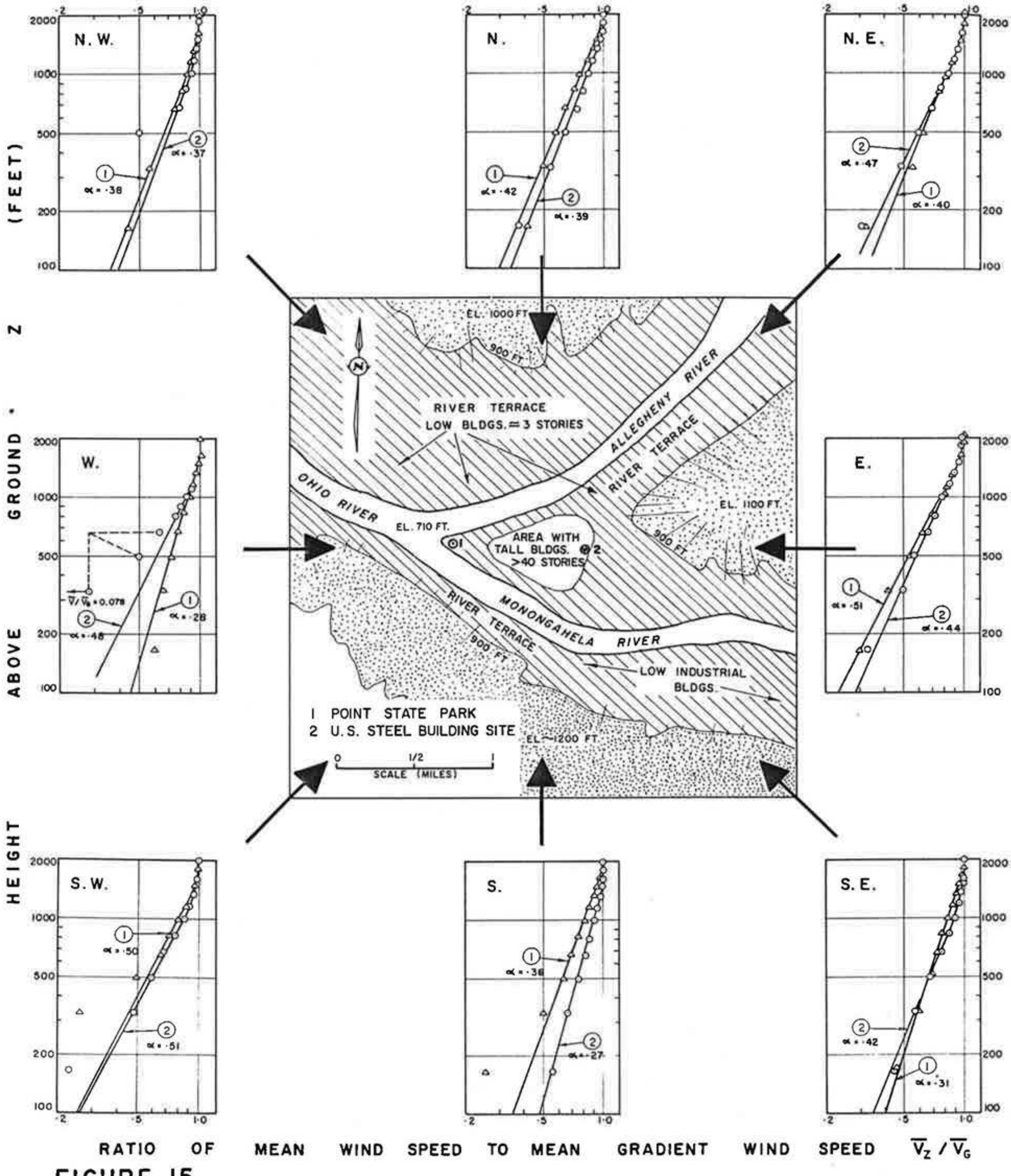


FIGURE 15
MEASUREMENTS OF MEAN VELOCITY PROFILES OVER 1 : 2000 TOPOGRAPHIC MODEL OF PITTSBURGH IN BOUNDARY LAYER WIND TUNNEL