Zeitschrift: IABSE congress report = Rapport du congrès AIPC = IVBH Kongressbericht

Band: 8 (1968)

Artikel: The design of multi-story buildings against wind

Autor: Davenport, A.G.

DOI: https://doi.org/10.5169/seals-8808

IIIc

The Design of Multi-Story Buildings against Wind

Dimensionnement de bâtiments élancés par rapport aux efforts du vent

Bemessung von Hochhäusern auf Wind

A.G. DAVENPORT
Professor
Director, Boundary Layer Wind Tunnel Laboratory
the University of Western Ontario
Faculty of Engineering Science
London, Ontario, Canada

Submitted as a discussion of the paper "Dynamic Effects of Wind and Earthquake", by D. Sfintesco.

The author's paper reminds us of the similarities and differences in the approach to the wind and earthquake design of tall buildings. Taken together with the papers by Ferry Borges and also by Newmark and Hall, a fairly comprehensive survey of the subject is presented.

The historical allusion made by Sfintesco to the notable work by Gustav Eiffel and Sir Benjamin Baker reminds us of their insight into the action of the wind on structures. Their recognition of the influence of the size of the structure on the response and its dynamic response to wind, in many senses is clearer than several contemporary viewpoints.

In this discussion, the writer draws attention to two approaches to designing tall buildings against wind which perhaps answer some of these questions posed by Eiffel and Baker regarding both the size effect and the resonant response. These approaches are:

a) a Gust Factor approach; and
b) the use of wind tunnel modelling.

A design approach embodying gust factors has been described in several papers. It is already in use in the Danish Standards and is currently under consideration for incorporation in the National Building Code of Canada.

DESIGN CRITERIA FOR WIND LOADING

Sfintesco refers to most of the significant wind effects on tall buildings; namely, collapse, damage to masonry and finishes, damage to windows and cladding, fatigue damage, and comfort of occupants.

Some tentative design criteria for these effects are as follows:

1) Collapse: Current design conceives of the structure withstanding a wind having a recurrence interval of about 30 years, with a safety factor on the minimum stress of roughly 2.0. In fact, this may be the least critical requirement in most tall buildings. It might be more logical to use a far more improbable wind speed and a lower safety factor; for example, a once-in-500-year wind speed and a safety factor of 1.1 might give a more rational evaluation of risk.

2) Damage to masonry and finishes: Masonry and plaster appears to become sensitive to cracking under racking loads when the story deflection is in the range 1/8" - 1/2". This corresponds to an average building drift limitation of the order of 1/250 to 1/1000. If the average interval for redecorating is 3 years and if a 10% risk that damage would be done within this period was acceptable, an average recurrence interval of 30 years would be appropriate. The actual deflection criterion should properly be related to the kind of partition and masonry or other elements used.

3) Windows and cladding: Cladding and window lights today represent a very large proportion of the total cost of tall buildings. An acceptable breakage rate of 1 light per building every ten years might be acceptable: unacceptable deflections on the windows should probably not be permitted to occur more often than once every 5 years.

4) Fatigue: This is the most common cause of failure of structures damaged by wind. It can probably best be evaluated by use of cumulative damage laws. Procedures for its evaluation have been described by Davenport. It is likely to arise whenever dynamic stress amplitudes are high. These circumstances indicate the desirability of wind tunnel tests.

5) Comfort of occupants: It appears that the threshold of perception

of human beings to horizontal vibration occurs when the maximum acceleration is roughly in the range 0.5 - 1.5% of gravity: 1.5 - 5.0% of gravity may be annoying.

Of course, all of the above must be regarded as opinions rather than inflexible yardsticks: the subject matter concerned is essentially statistical, and the decision making cannot be made without some uncertainty. The suggested criteria are summarized in Table 1.

TABLE 1

TYPICAL CRITERIA FOR DESIGN OF TALL BUILDINGS AGAINST WIND

Unserviceability Symptom	Acceptance Criteria	Recurrence Interval: Years
1) Collapse	Safety factor = 1.1	500
2) Cracking of masonry & finishes	Max. def'n. $< \frac{1}{250} \rightarrow \frac{1}{1000}$ of height	30
3) Windows and cladding:		
a) perceptible deflections;	a) dependent on size of light, colour and type of glass	a) 5
b) breakage	b) <1 breakage per building	b) 10
4) Fatigue	Cumulative damage <100%	500
5) Comfort of occupants	Max. acc'n. <.5 → 1.5%g	10

DESIGN APPROACH #1 -- GUST LOADING FACTOR

This approach consists of the following phases:

1) The prediction of extreme average wind speeds from long term meteorological records such as those indicated in Fig. 1.

2) The adjustment of these wind speeds obtained at the meteorological observation station to the terrain conditions and height of the structure by means of profiles such as those shown in Fig. 2.

3) The determination of mean pressures using pressure coefficients appropriate to the particular flow conditions and structural shape as illustrated in Fig. 3.

4) The determination of the gust amplification factor using the gust pressure factor G defined below and in Fig. 4.

The gust pressure factor is intended to take account of the superimposed dynamic effect of gusts. It is used in conjunction with the mean load so that the total wind loading at any point on the building is,

$$p(Z)_{max} = G\ \bar{p}(Z)$$

where $\bar{p}(Z)$ refers to the mean pressure at height Z and given by such pressure coefficients as those in Fig. 3.

The factor G is the gust factor given by,

$$G = 1 + gr\sqrt{B+R}$$

in which g = peak factor, r = roughness factor, B = excitation by background turbulence, and R = excitation by turbulence resonant with structure.

The quantity,

$$R = \frac{S\ F}{\beta}$$

in which F = gust energy ratio, s = size reduction factor, and β = damping factor.

An explanation of these factors follows. In all cases, the mean velocity $\bar{V}$ is the velocity at the roof level. Graphs of g, r, B, F and s are shown in Fig. 4.

1) The peak factor g is the ratio of the peak dynamic response to the RMS response of the structure. It is a function of the average fluctuation rate of the response and the averaging period of the mean T. T should be between 5 min. and 1 hour. An expression for ν is

$$\nu = n_o \sqrt{\frac{R}{B+R}}$$

where R and B are defined below. For a peaked response, the value of $\frac{R}{R+B}$ is near to unity and $\nu \simeq n_o$, n_o being the natural frequency.

2) The expression $r\sqrt{B+R}$ is in fact the RMS response of the structure to gusts. r is a roughness factor dependent on the terrain. r^2B is the contribution to the variance (mean square) response due to "background excitation", while r^2R is the contribution to the variance from the resonant response of the structure at its natural frequency.

3) The significant effect of size of the structure in reducing the dynamic load is seen both in B and the size reduction factor s.

4) The gust energy ratio F reflects the distribution of energy with frequency in the wind and hence the energy available to excite resonance.

5) The critical damping ratio β should include contributions to the damping from both mechanical and aerodynamic origin. For tall buildings, however, neglect of the aerodynamic damping is generally not significant. Suggested values of the mechanical damping are as follows:

Concrete	β = .010 - .020
Steel	β = .005 - .010

If the deflected shape of the structure both in the fundamental mode of vibration and under the action of steady wind is approximately rectilinear, as usually is the case with tall prismoidal buildings, an approximate expression for the maximum deflection and acceleration amplitudes can be derived. To do so, it is necessary to define an effective stiffness K which is the base bending moment per radian of rectilinear rotation of the structure. Knowing the base bending moment M under either inertia loading (dynamic) or the static wind loading either deflections or acceleration amplitudes may be found. Expressed in radians, the amplitude will then be simply M_o/K.

It is convenient to express the base bending moment in terms of the aerodynamic coefficient C_M so that,

$$C_M = \frac{1}{bh^2} \int_A Z \, C_p \, dA$$

where dA is an element of the projected frontal area, C_p is the local pressure coefficient on the front or rear surfaces at position Z and the integral is taken over front and rear surfaces.

The maximum deflection as a fraction of height is then computed from the expression

$$\frac{deflection}{h} = G \, \tfrac{1}{2}\rho V_o^2 \, C_M \, bh^2/K$$

The maximum acceleration amplitudes invariably occur at the natural frequency and an approximate expression for the peak acceleration in the wind direction is,

$$maximum\ sway\ acceleration = 4\pi^2 n_o^2 \, gr\sqrt{R} \, C_M \, \tfrac{1}{2}\rho V_o^2 \, bh^3/K$$

$$= gr\sqrt{R} \, C_M \, \tfrac{1}{2}\rho V_o^2 \, bh^3/I_o$$

where I_o is the moment of inertia of the building about the base, ie. $I_o = \sum m(Z) \, Z^2$, where $m(Z)$ is the mass in dynamic units at height Z.

Experience in the use of this approach generally indicates the following results:

1) Loading is on average in accordance with standard loadings used but the differentiation in loading between structures and between urban and rural terrains in significantly broader than standard approaches imply.

2) Tall slender structures with light damping incur relatively large dynamic gust factors (up to 3 times the mean load). Broad faced structures of relatively stiff construction incur relatively little dynamic amplification, perhaps only 30% greater than the mean load.

3) Structures in urban areas are affected more by turbulence than in rural area, but the mean loading is substantially lower.

4) Use of the correct velocity profile, wind tunnel testing conditions and dynamic gust factor are all highly significant and serious discrepancies can arise if this is not done.

DESIGN METHOD #2 -- BOUNDARY LAYER WIND TUNNEL MODELLING

Recently, strong and well justified criticism has been directed toward the use of aeronautical-type wind tunnels for investigation of pressures on models of structures. In some cases, the results of such tests can be highly misleading.

Seemingly a more promising development is the use of the boundary layer wind tunnel large enough to accommodate structural model testing. At present, only two or three such tunnels probably exist. That at the Boundary Layer Wind Tunnel Laboratory at The University of Western Ontario is of this type.

The application of this type of study is worthwhile in the investigation of large, important, structures exposed to the wind.

An outline of the possible phases of a wind tunnel study for the design of a tall building is given below in Table 2. The design procedure is illustrated diagrammatically in Figs. 10 and 11.

Perhaps the principal virtue of this approach is the understanding that evolves of the real way in which a structure is likely to behave in service; this understanding cannot really be duplicated by artificial formulation of wind loading parameters. While significant economy can be achieved by better tailoring the material in a structure to meet its actual behaviour, the greatest economy is achieved by recognition of problems at the design stage rather than after the structure is in service. The approach allows a number of problems which so far have been left unsettled to be studied; in particular, these problems include the question of maximum deflections, maximum acceleration, and the susceptibility of the structure to fatigue.

BIBLIOGRAPHY

A. G. Davenport "New approaches to the design of structures against wind action". Proceedings of Canadian Structural Engineering Conference, Toronto 1968. Published by Canadian Institute of Steel Construction.

A. G. Davenport "Gust loading factors". Jnl. of Str. Div., Proc. of Am. Soc. Civ. Eng., Vol. 93, pp 12-34, June 1967.

L. E. Robertson "On tall buildings" in "Tall Buildings". Ed. A. Coull, B. Stafford-Smith, Pergamon Press, 1967.

A. G. Davenport "The estimation of load repetitions on structures with application to wind induced fatigue and overload". Paper presented at the RILEM Symposium on the "Effects of Repeated Loading of Materials and Structures", Vol. 1, Mexico City, Sept. 15-17, 1966.

"Vejledning for fastsaettelse of vindbelastninger". Dansk ingeniorforenings standards for building construction, Teknisk Forlag, Kobenhavn, Denmark 1966.

M. Jensen, and N. Franck "Model scale tests in the natural wind, Part II", Danish Technical Press, Copenhagen, 1965.

International Conference on "Wind Effects on Buildings and Structures". NPL Teddington, England 1963, Pub. H.M.S.O., 1965.

International Seminar on "Wind Effects on Buildings and Structures". N.R.C., Ottawa, Canada, 1967, Proc. to be published by Univ. of Toronto Press.

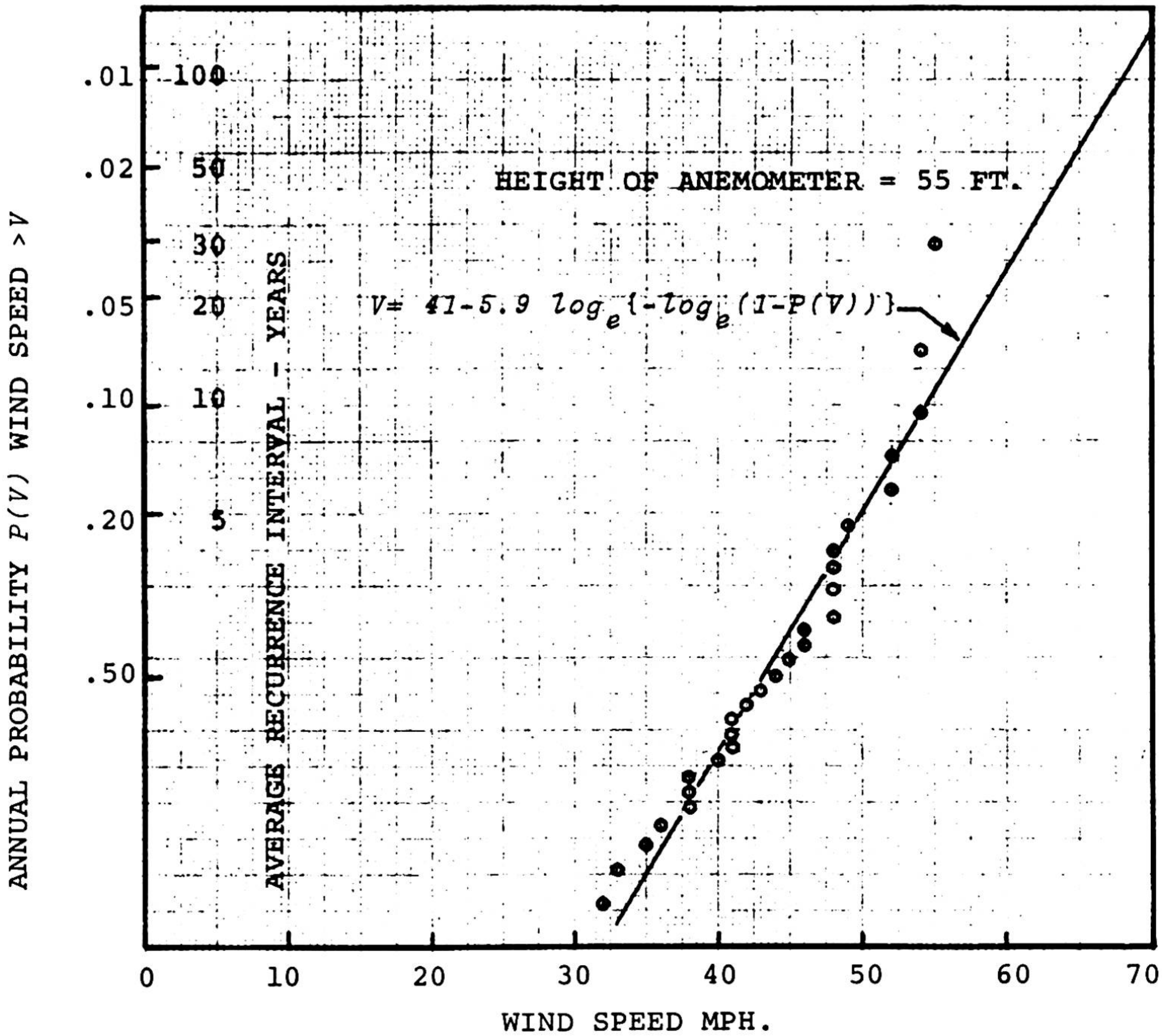


FIG. 1. EXTREME ANNUAL HOURLY AVERAGE WIND SPEEDS AT TORONTO MALTON AIRPORT (1939-1965).

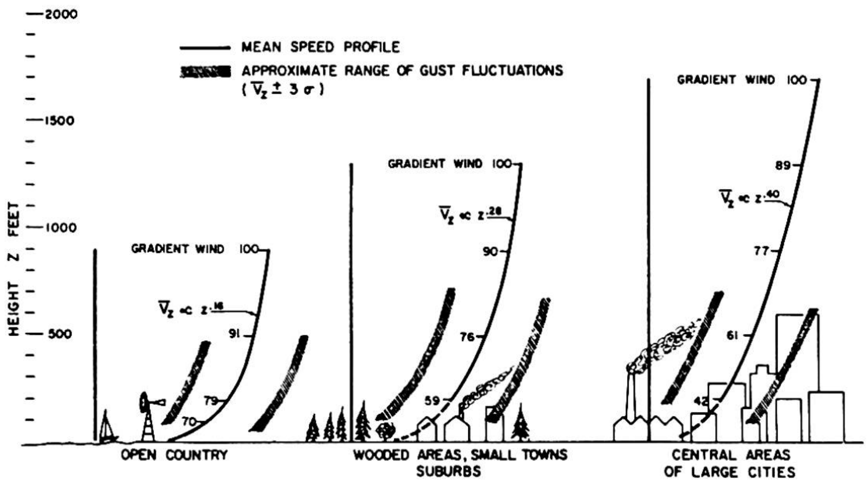
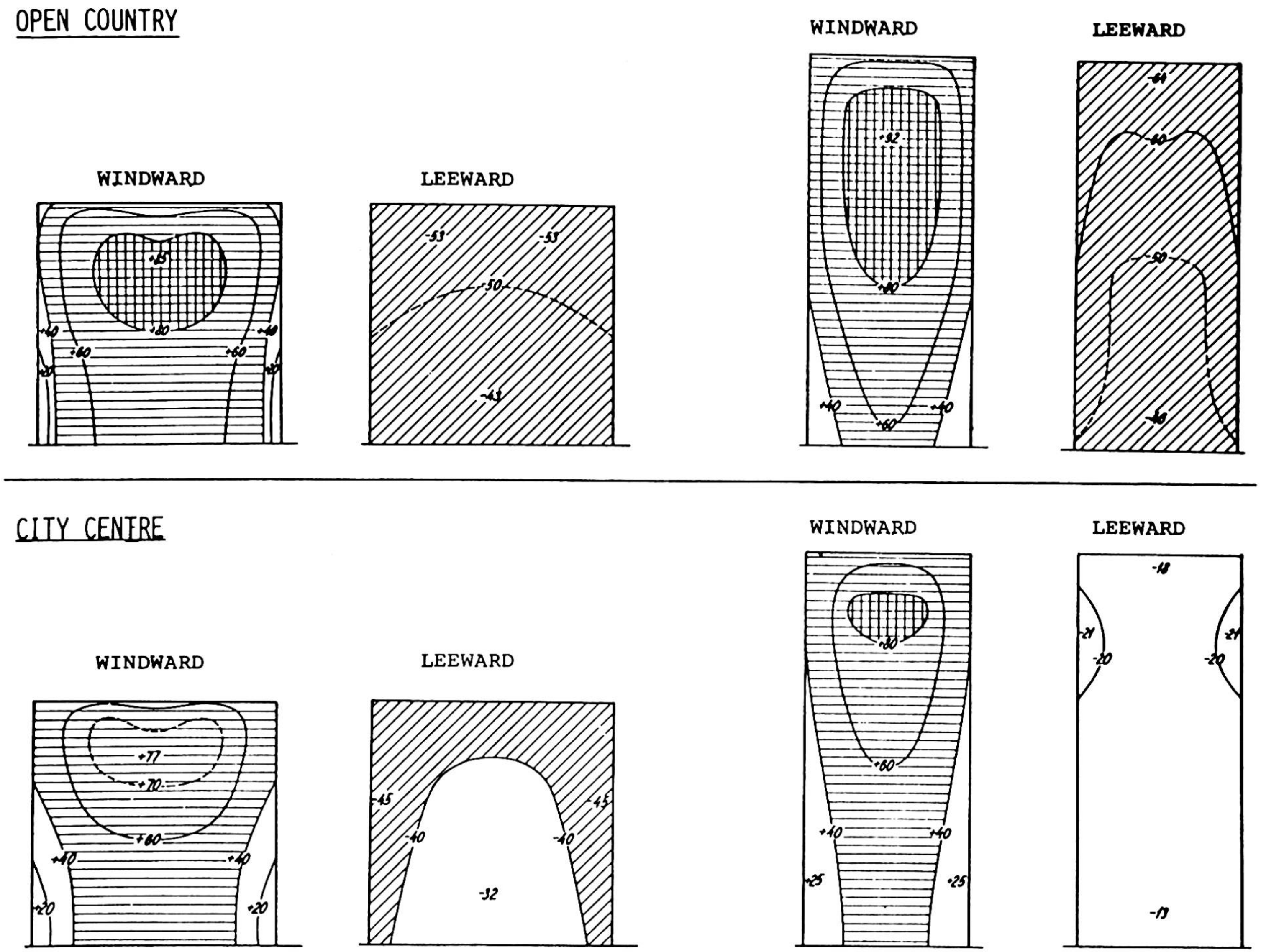


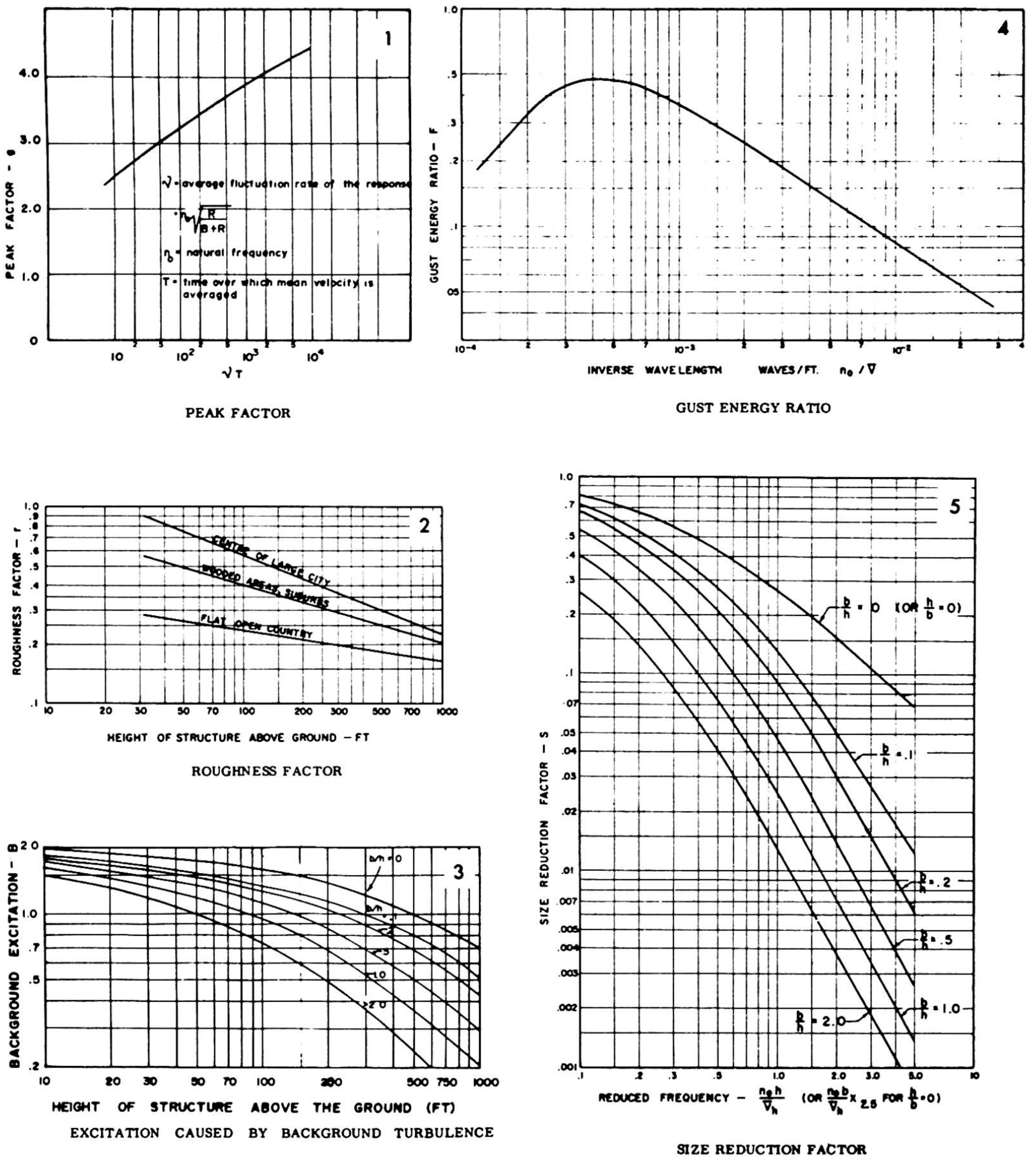
FIG. 2 MEAN WIND VELOCITY OVER LEVEL TERRAINS OF DIFFERING ROUGHNESS.



HEIGHT: BREADTH: DEPTH = 5:5:1

HEIGHT: BREADTH: DEPTH = 2.4:1:1

FIG. 3 TYPICAL AERODYNAMIC PRESSURE COEFFICIENTS (AFTER JENSEN-1965)(WIND NORMAL TO FACE: COEFFICIENTS REFERENCED TO VELOCITY PRESSURE AT ROOF LEVEL)



GUST FACTOR: $G = 1 + gr\sqrt{B+R}$ $\quad R = \frac{SF}{\beta}$

FIG. 4

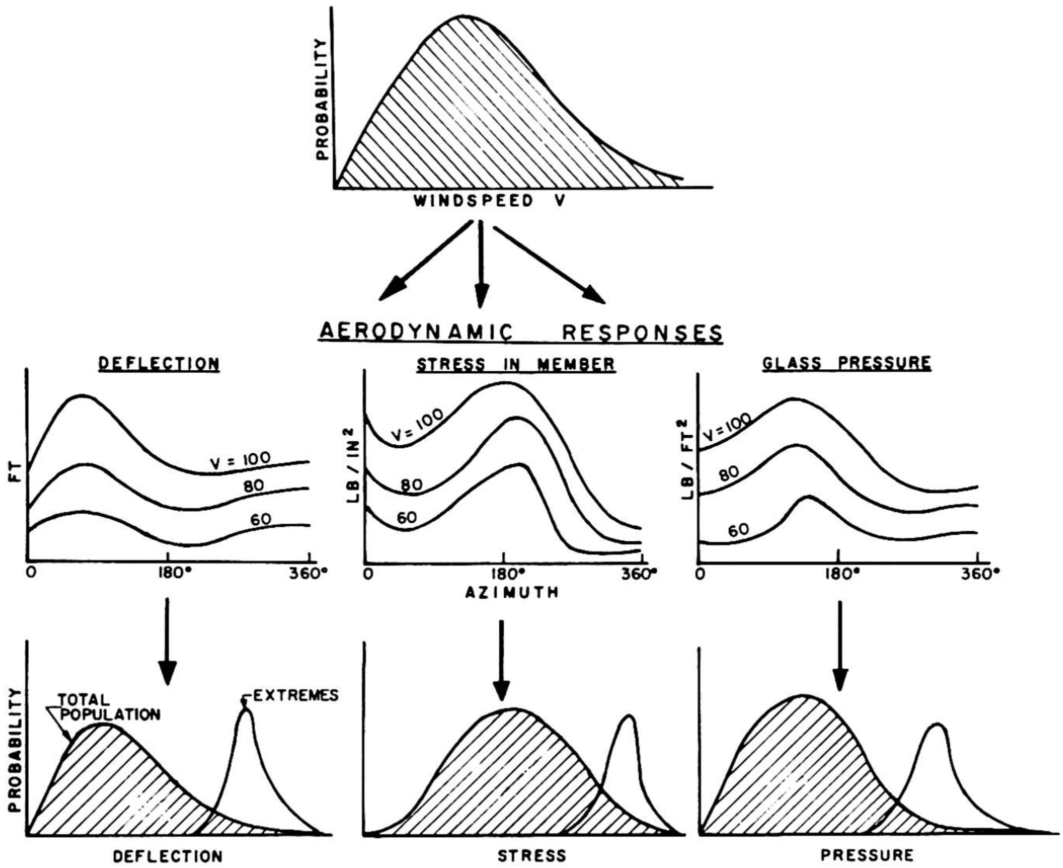


FIG. 5 DETERMINATION OF PROBABILITY DISTRIBUTIONS OF STRUCTURAL RESPONSE

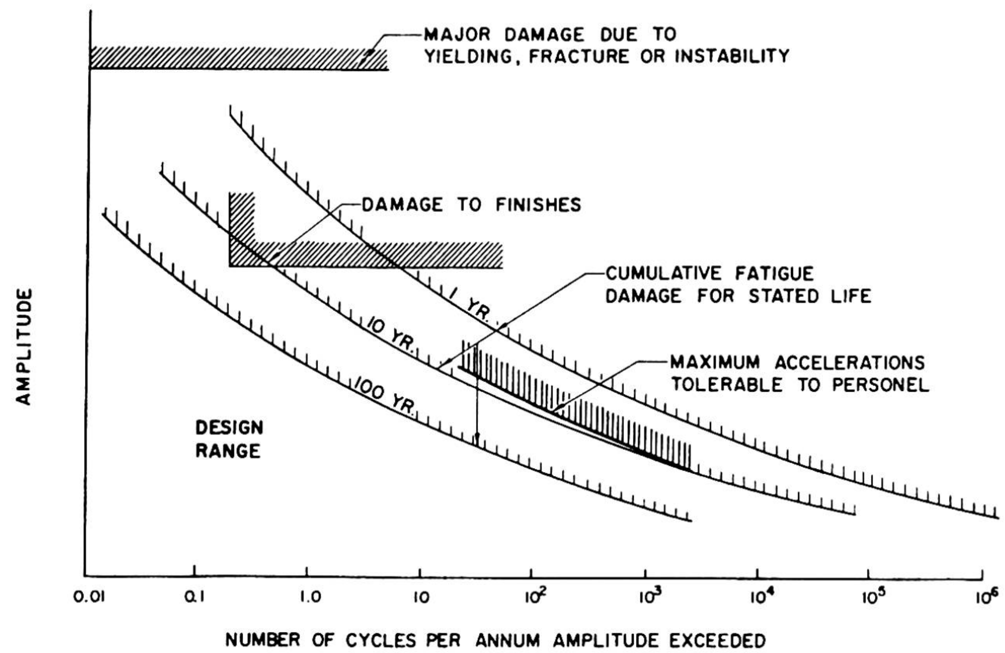


FIG. 6 ENVELOPE OF DESIGN LIMITATIONS