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WIND LOADS ON STRUCTURES

BY

A. G. DAVENPORT

TECHNICAL PAPER NO. 88

OF THE

DIVISION OF BUILDING RESEARCH

NRC 5576

OTTAWA

MARCH 1960

PRICE $1.00

NATIONAL RESEARCH COUNCIL

CANADA

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PREFACE

Much attention has recently been given by the Building Structures Section of the Division of Building Research to the subject of loads on structures, with special reference to their treatment in the National Building Code of Canada. Improvements in the methods of designing structures must be paralleled by a more accurate assessment of the loads acting upon them, since a design can be no more accurate than the load assumption made for the calculation.

The Building Structures Section has begun a study of the action of wind on structures, in addition to carrying on an extensive study of snow loads on roofs. In view of the great variation in climate and topography across Canada and the great variation of shapes of structures, much simplification is needed in codifying wind loads for practical purposes. Part of the simplification is the inevitable consequence of lack of knowledge; there is still much room for improvement in the present wind load requirements.

Wind forces on structures depend on the velocity of the air on the one hand and the effect of the shape of the structure itself on the other, or in other words, on meteorological as well as engineering information. The basic information on wind speeds in the National Building Code (1953) consists of a map showing "computed maximum gust speeds" for all of Canada. This was prepared by the Meteorological Division of the Department of Transport with which the Division of Building Research is fortunate to maintain the closest liaison.

The present study on wind loads was begun with an extensive survey of the literature on the action of wind on structures, based upon which the author has made a number of recommendations which inevitably reflect to some degree his personal opinions. The author of the paper is A.G. Davenport who, from July 1957 to September 1958, was a member of the Building Structures Section, the head of which is W.R. Schriever. The author has since left the Division of Building Research to take up post graduate study at the University of Bristol.

Ottawa
March 1960

Robert F. Legget
Director

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A.G. Davenport

The purpose of this paper is to indicate the basis of the present design wind-load requirements of the National Building Code of Canada (172)*, to discuss their underlying assumptions, and to provide a survey of research dealing with the subject of wind loads. From this recommendations will be offered with regard to what improvements might be effected in the analysis and formulation of wind-load requirements and in the recording of meteorological data and, to this end, to suggest research needed.

1. PRESENT WIND LOAD REQUIREMENTS (Section 4.1.2.3, reference 172)

1.1 Introduction

Present wind-load requirements are based on:

(a) "computed maximum gust speeds", V_g, which are assumed to refer to a given datum level of 30 ft above ground.

(b) a coefficient C_h giving the increase in this velocity for heights greater than 30 ft. It is defined by the formula $C_h = \left(\frac{h}{30}\right)^{1/7}$ (or the one-seventh power law) where h is the height above ground, in feet.

(c) a set of shape coefficients C_s from which the wind pressures on different surfaces of a structure assumed uniformly distributed, and also internal pressures can be inferred.

The unit pressure q is then given by

$$q = C_s \times 1/2\ \rho\ (C_h\ V_g)^2$$

in which ρ, the density of air, is assumed to have a value such that if V_g is in mph,

$$q = 0.00258\ C_s .\ (C_h\ V_g)^2 \text{ lb/sq ft}$$

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* numbers in parentheses refer to list of reference material at the end of this paper.

This uniform pressure is assumed to be statically applied and to act normal to the surface. In general form this formula and the inherent assumptions in its application are similar to those used in many other building codes (169-178).

1.2 Shape Factors

The values of the shape factors given in the British Standard Code of Practice (171) which, in turn, were largely based upon investigations in a laminar flow wind tunnel at the National Physical Laboratory (103, 161) have been adopted by the National Building Code without substantial alteration. Recent studies appear to show the need for some revisions.

1.3 Increase of Velocity with Height

The one-seventh power law which has been adopted by the code to describe the increase in "maximum computed gust speed" with height is a familiar aerodynamic profile found in wind-tunnel investigations of turbulent flow over smooth boundaries at relatively high Reynolds numbers, (51, 85 (p.22), and 198) in neutrally stable conditions over open flat terrain. It should be noted, however, that the profile refers to the increase in mean velocity with height; the assumption implied in its adoption by the present code that it also describes the increase in "maximum computed gust speed" with height is not substantiated.

1.4 Design Wind Velocities

Present design wind velocities (231) were obtained basically from anemometer records. These records are available for about two hundred stations across Canada (220). Most of the information is used in connection with the meteorological services rendered by the Department of Transport to aviation and shipping and it is not urprising to find that the majority of stations are either at airports, or harbours and other places along the coastline.

The anemometers, usually situated on top of buildings, are of the Robinson cup type - the improved three-cup instrument having gradually replaced the less accurate four-cup instrument.

Records are available from these stations for periods ranging from one to thirty years. The records from these instruments are given in terms of the number of miles of wind passing the anemometer in an hour. To this a correction factor is sometimes applied which may be about 5 per cent at 60 mph (220), yielding the "true" hourly mileage. This correction factor is a function of the gustiness which is assumed to be constant in applying the correction. In the last few years no corrections have been applied.

During the preparation of the 1953 Code it was decided that design wind velocities for the requirements of the Code should consist of the maximum gust velocity measured by these particular instruments over a thirty-year period. At that time Dines anemometer records were available for five stations for two years. These instruments are of the pressure type and record on anemographs the average velocity of gusts whose duration is between about 2 and 10 sec (depending on the response time of the instrument).

Unfortunately, however, records from all other stations were readily available only in terms of mean hourly wind speeds. It was therefore decided to estimate the gust speeds at these stations by correlating maximum gust speeds with hourly wind mileages at the four stations for which there were simultaneous records of both velocities, for two years. In performing this correlation, gust speeds of over 65 mph occurring simultaneously with hourly mileages over 35 mph were used; all others were omitted from the computation.

To these records, shown in Fig. 1, a line was fitted by least squares relating the "most probable" gust velocity, V_g, with the mean hourly velocity, V_m, according to

$$V_g = 19 + 1.22\ V_m.$$

Parallel to this line and enveloping the data was fitted the line

$$V_g\ (\text{max}) = 25 + 1.22\ V_m.$$

This equation was used to determine the maximum gust velocities for all other stations. This was the first step in determining design velocities.

The second step was to examine the records of all stations across Canada and determine for each the maximum recorded hourly mileage.

The third step was to correct each of these maximum recorded mileages according to the number of years records had been kept. The following standardizing factors were obtained from a study of thirty years records at Toronto, Quebec, and Victoria:

Percentage of mileage to be added to bring a recorded maximum to a 30-year maximum						
Years on record	1	5	10	15	20	30
% to add	25	10	5	3	1	0

The maximum gust speeds for each station were then computed from the corrected maximum hourly mileages.

To these results, with more weight given to the records of longer period, were fitted the isotachs shown in Chart 9 of Part 2, Climate, of the National Building Code (Fig. 2).

The present design velocity requirements of the Code are defined (by this chart) for "ordinary terrain".

2. THE DETERMINATION OF BASIC DESIGN WIND VELOCITIES

2.1 Structure of Natural Wind

Winds are caused by the atmospheric pressure differentials which arise over the surface of the earth due to differences in the amount of heat that is received from the sun. The acceleration produced by these pressure differentials is affected, however, by another component of acceleration known as the geostrophic acceleration which is caused by the rotation and curvature of the earth. Thus a parcel of air moving on the northern hemisphere experiences force to the right (at right angles to the direction of motion) proportional to the speed of motion, called the coriolis force. If, firstly, the pressure system remains the same and sufficient time elapses for the wind to reach a steady-state condition and if, secondly, there is no friction, then the wind will flow at right angles to the pressure gradient (i.e. parallel to the isobars) with the lower pressure on the left. In other words, the coriolis force is just in equilibrium with the force due to the pressure differentia Above 1000 or 1500 feet these conditions are often nearly satisfied and the wind blows nearly parallel to the isobars. This wind, which is unaffected by friction, is called gradient wind and its velocity gradient velocity.

Under steady-state conditions the gradient velocity can be determined directly if the latitude, the radius of curvature of the isobars, and the pressure gradient (or spacing of the isobars) are known (35). For zones of cyclonic winds (associated with a low-pressure system and with storms and high winds) the gradient velocity V_g is given by:

$$V_g = r.w. \sin \lambda \left[\sqrt{\frac{\frac{dp}{dn}}{\rho r.w.^2 \sin^2 \lambda} + 1} - 1 \right]$$

where r = radius of curvature of isobars

w = rotational speed of the earth

λ = latitude

$\frac{dp}{dn}$ = pressure gradient

ρ = density of the air

The equation lends itself ideally to simple nomographic solution as shown by Humphreys (35), and the gradient velocity may be estimated directly from isobar charts such as that shown in Fig. 3.

Accurate estimates of gradient velocity, however, are difficult to obtain unless the grid of meteorological stations measuring surface pressures is close. This is not particularly the case in Canada and attempts made by the author to find some consistency in a comparison of surface velocities measured by an anemometer and simultaneous gradient velocity measured from the surface pressure charts were not successful, because of the above reasons as well as a variety of others.

The velocity of the gradient wind, however, is attained only at heights in the neighbourhood of 1000 to 2000 ft above the ground. Closer to the ground the wind is retarded by frictional forces and obstructions at the surface and viscous forces transmitted upwards by turbulence: its direction then is no longer parallel to the isobars. Turbulence also causes rapid fluctuations in the velocity over a wide range of frequencies and amplitudes. The velocity of the wind at lower levels is therefore expressed most usefully in terms of its mean speed and the deviations from this velocity (85).

The time or distance interval over which the mean is averaged depends upon the purpose for which the wind velocity is to be used. For many applications it is sufficient to know the average wind speed during a day or an hour (43). For the design of structures, however, it is necessary to know the mean wind speed during the peak of a storm which may last only a few minutes. The choice of a suitable time or distance interval for averaging mean velocities is dictated by considerations of the dynamic characteristics of the wind, the structure, and the anemometer. From a more detailed discussion, which is left until later, it is found that the fastest minute of wind or the fastest mile of wind represents roughly the optimum averaging interval for determining and defining basic design wind velocities.

2.2 Increase of Velocity with Height; the Effects of Surface Friction

One of the most important factors to be considered is the increase of the mean wind velocity with height; a corollary to this is the retarding effect of the surface friction on wind velocity nearer the surface.

Various empirical, semi-empirical, and theoretical formulae have been derived to represent the variation of wind velocity with height. Three of the more familiar forms are the spiral, (19, 59) logarithmic, (83) and exponential (5) profiles. For structural purposes the exponential or power law profile has been used most widely because of its simplicity. It can be stated as follows:

$$V_z = k\ z^{1/\alpha}$$

where V_z is the velocity at height z above ground and k and $1/\alpha$ are constants.

By suitable choice of exponent this expression can be made to correspond closely over a considerable range to the other forms of profile which are less empirical. The power law is applicable only in the layer extending from the ground up to the height at which the gradient velocity is first attained (usually in the range 1000 to 2000 ft). Above this height the wind velocity may be regarded as constant.

In his treatise on Physical and Dynamical Meteorology, Brunt (5) notes that "If the variation of wind with height be represented by a lower law z^p it is found that p is increased by an increase in either roughness or stability" (z being the height above ground). Here reference is made by Brunt specifically to a range of heights above roughly 10 metres (33 ft) which is of interest to structural engineers.

An attempt is now made to evaluate these influences of stability and surface roughness on first, the rate of increase of mean velocity with height and second, on the magnitude of the mean surface velocity, both of which are of prime importance in estimating basic design wind velocities for structures.

The stability of a storm is measured by the lapse rate or rate of temperature variation with height (5). In storm winds of long duration in which turbulence causes thorough mixing the lapse rate near the ground is invariably close to the adiabatic which corresponds to a state of neutral stability (78, 85). Geiger has stated that this condition is generally attained at velocities greater than 6 m/sec (13 mph) (23). Observations (40) of hurricanes Edna (1954) and Ione (1955) off the New England coast reported by Kessler (40) apparently confirm the general assertion that the stability of mature and large-scale storms, whether of the tropical or extra tropical variety, is close to being neutral.

Exceptions to this statement may be found however, in severe local storms such as thunderstorms and frontal squalls (and perhaps other larger storms such as hurricanes in their early incipient stages before full maturity is reached) which are notably unstable, air near the ground being warmer than that aloft. As a consequence of this instability violent thermal interchange takes place between the air near the surface and the faster-moving upper air which is not retarded by friction near the surface. Under circumstances of extreme instability the value of the exponent 1/ may attain the limiting value of zero corresponding to zero increase in velocity with height. At Agra, Barkat Ali (1) measured an exponent of 0.02 (1/50) in very unstable conditions and Sutton (85) suggests a value of 1/100.

From the foregoing remarks the following general inferences may be drawn with regard to the effects of stability on the profile.

(1) Severe local storms, such as thunderstorms, frontal squalls (and perhaps other storms such as hurricanes in their early incipient stages) are extremely unstable and consequently the increase of mean velocity with height is very small. The frictional characteristics of the ground surface may have almost negligible effects on the velocity profile.

(2) Large-scale mature storms of either tropical or extra-tropical description exhibit nearly neutral stability with no marked tendency for violent thermal interchange. The dominating influence on the velocity profile in these storms is not stability but the surface roughness.

The important question now arises how great an effect does surface roughness have on the wind velocity profile in storms of the latter category, which are probably far more important? In comparison to the vast amount which has been written in engineering papers on this general subject scant attention has been given to this particular question. It appears to be of paramount importance in the accurate evaluation of wind velocities.

It should first be emphasized that what is referred to in the term "surface roughness" is neither the shielding due to individual obstacles nor the orographic effects influencing the airflow in mountain regions but the cumulative statistical drag effect of many obstructions on the wind. The induced retarding forces on the airflow are due not only to the frictional drag at the surface but also to the much greater viscous forces associated with the turbulence which extends to a height far greater than the obstructions which caused it. The surface roughness is therefore characterized by the density, size and height of the buildings, trees, vegetation, rocks, etc. on the ground, around and over which the wind must flow. Surface roughness will be a minimum over the ocean and a maximum over a large city.

The measurement of wind velocity profiles has for some years been of interest in the fields of meteorology, aviation, agriculture and wind power as well as engineering (see References on Wind Structure, Nos. 1 to 100). A considerable amount of information has now accumulated from which it is possible to evaluate the influence of the surface roughness on the wind velocity profile.

In Fig. 4 the power laws corresponding to the accumulated experimental results of a number of observers (38) have been plotted on a comparative basis relating the ratio of mean velocities at a given height to that at 30 ft with the height above ground. The data from which these equivalent power law curves are derived are given in references 17, 22, 25, 39, 48, 67, 73, 79, 80, 90, 99, 100, and 156 (see also Appendix I). Where the results were not explicitly stated as a power law it was found that in each case a power law could be closely fitted to the data with only small deviations. It should be noted that the data refer specifically to mean velocity profiles prevailing above a height of 30 ft in strong winds, over flat ground surface at lapse rates which, if not explicitly stated, by the nature of the storms studied could not have differed greatly from the adiabatic. The data are therefore homogeneous with the exception that the nature of the ground roughness and the aggregate nature of obstructions vary widely from the smooth surface of the sea to the rough obstructed surface of a large city.

The exponent of the power law increase is seen to vary between roughly 1/9 and 1 depending only on the surface roughness characteristics. It is seen that curves 1 to 7 have exponents lying between 1/8.3 and 1/6.7 and in each case the surrounding terrain was characteristically flat and open. The average lies close to 1/7 which is a familiar exponent found at the boundaries of pipes and wind tunnels in which mechanical turbulence predominates and the fluid is neutrally stable. The exponents of curves 8 to 14 lie between 1/5 and 1/2.8 and average 1/3.5. For these curves the terrain corresponded to the rougher characteristics of rough coasts, treed and wooded farmland, towns, scrub trees, etc. Curves 15 and 16 are derived from records obtained in large cities, Paris and New York, which probably represent conditions of extreme surface roughness; the corresponding exponents are 1/2.0 and 1/1.6.

It should be noted at this stage that velocity itself may be expected to have a slight secondary effect on the profile in that the value of the surface friction increases slightly with wind velocity; as a consequence the rate of increase of mean wind velocity with height increases slightly with velocity. This is a familiar result in wind tunnel work (85).

Numerical evidence of the indirect influence of the velocity on the velocity-height relationship is furnished by Collins' investigations of nine storms, mainly at Brookhaven Laboratory, Long Island (11). In this it was found by examining the 5-minute mean velocities at different elevations up to 410 ft that the exponent of the power law profile increased by approximately 0.02 for every 10 mph increase in surface wind velocity, at 50 mph the value being 0.27 (i.e. 1/3.7) and the extrapolated value at 80 mph being 0.33 (1/3.0). These profiles, according to Collins, fitted the experimental records extremely well with the standard deviation equalling 1.26 mph.

These results indicate however, that the effect of wind velocity (over the range of maxima encountered) is not nearly so great as that due to the differences in surface roughness.

Bearing in mind the influence of the wind velocity on the rate of increase of wind velocity with height, it is now possible to suggest approximate values for power law exponents corresponding more or less qualitative descriptions of the surface roughness or aggregate influence of the surface obstructions, as follows:

Description of the Terrain	Power Law Exponent
For open country, flat coastal belts, small islands situated in large bodies of water, prairie grassland, tundra, etc.	1/7
For wooded countryside, parkland, towns, outskirts of large cities rough coastal belts	1/3.5
For centres of large cities	1/2.5

These figures refer to the mean wind velocity over level ground, to large-scale severe storms (which exhibit nearly neutral stability) and to heights between about 30 ft and the height at which the gradient velocity is first attained. If there are areas in which the highest probable velocities occur during severe local storms such as thunderstorms and frontal squalls (which does not seem likely) no increase in velocity with height would seem appropriate.

2.3 The Estimation of Extreme Mean Wind Velocity

If reliable long-term anemometer records were available for all areas exhibiting differing characteristics of surface roughness and incidence to severe storms the discussion could perhaps be left at this point: a basic wind velocity could be determined for each geographical location "based upon a statistical analysis of wind records over a period of 40 or 50 years", and this applied to a wind velocity profile appropriate to the surface roughness of the vicinity.

Unfortunately there are serious obstacles to this approach (at least in a sparsely populated country such as Canada) since meteorological records are not always as satisfactory as this approach would require for the following reasons.

Only a comparatively small number of stations would have records extending back a sufficient number of years (especially in Canada); at some locations the anemometer has been moved several times during the period of record, affecting the exposure and the homogeneity of the data: on other occasions severe storms have blown the anemometer away or rendered it inoperative thus losing crucial information. The anemometer may also give readings which are not representative of level country owing to the siting of the anemometer on or near a building. Dryden and Hill (117), for example, suggest that the well-exposed anemometer on top of the Empire State Building situated more than 200 ft above the roof, reads 23 per cent higher than the approaching flow due to presence of the building. If, at airports, anemometers were raised to a satisfactory height to be free from the influence of the buildings they might present a hazard to aircraft. In winter, the period of worst storms, Dines anemometers sometimes clog with blowing snow and ice accretions sometimes form on cup anemometers; anemometer readings taken in mountains, valleys, and coastal cliffs, are subject to orographic effects sometimes resulting in much higher velocities.

All these considerations add emphasis to Sherlock's recommendation, shared by many others, that design velocities should be based "on a statistical analysis of wind records over a period of 40 to 50 years" (77). But not only this, the results obtained from the records of one station should be related to the results from other neighbouring stations by a suitable numerical method; only in this way can spurious and systematic errors arising in the records of an individual station be minimized.

The next step, therefore, is to explore a suitable method whereby the accumulated total of all meteorological wind records might be correlated.

Since it is not desirable to restrict the admissible data solely to those obtained from anemometers situated in "open level country" (77) (this in certain regions of Canada would decimate the available data), it is first necessary to investigate more ... lly the effects of surface roughness on the velocities measured near the surface over level ground.

It has already been noted that there is some height at which the influence of the ground friction transmitted upwards through eddy viscosity, has a negligible effect on the velocity of the wind as it responds to the pressure gradient. If the velocity at this height is denoted by V_g (the gradient velocity) and the height at which this velocity is first attained by z_g then by reference to the power law increase of velocity with height,

$$V_z = V_g \left(\frac{1}{z_g} \right)^{1/\alpha} z^{1/\alpha} .$$

To determine the ratio of the velocity at height z above the ground to the gradient velocity it is first necessary to determine values of z_g corresponding to the various surface roughness categories already elaborated upon. This will now be attempted on the basis of such pertinent experimental data as are available.

Examination of Sherlock's investigations at Ann Arbor (73), indicate that for these conditions of flat open country the value of z_g (the height at which the gradient velocity is first attained) is of the order of 900 ft. This value agrees reasonably closely with that obtained by Taylor at Salisbury Plain (discussed by Pagon) (48) for similar terrain in which average values of z_g for strong winds were 1250 ft in summer and 885 ft in winter. An approximate value of 900 ft is therefore chosen for the value of z_g in flat open country.

In large cities Pagon (48) citing Taylor's studies at the Eiffel tower suggests a value of z_g for strong winds in a large city of 2020 ft in summer and 1420 ft in winter. An approximate value of 1700 ft is adopted for the present. The value of 1300 ft is chosen for the intermediate conditions of rough wooded country. These values of z_g of 900, 1300, and 1700 ft (together with the appropriate exponents) corresponding to the three types of roughness conditions give the three curves of Figs. 6 and 7.

At this stage no suggestion is intended that these curves are highly accurate -- indeed the qualitative aspects of the problem do not permit great exactness. It should be noted that in these curves the values of $1/\alpha$ are founded upon a relatively greater amount of information than are the values of z_g. The errors involved in the latter however (which may differ by 200 ft) are of less consequence.

Some affirmation of the rough accuracy of these curves is afforded by the comparison of the surface velocities over terrain of different surface roughnesses. In their study of the climate of Central Canada, Kendrew and Currie (224, p.150) observe "the mean (annual) wind speed in the prairies is between 12 and 16 mph .... The speed is appreciably less in parklands with means of 9-12 mph and again less in forests 5-9 mph; the increased friction among the trees is the main cause".

These reductions in wind velocity compared to the archetypal flat open country of the prairies are entirely compatible with those suggested by Fig. 7.

A comparison of mean wind speed in nine Canadian cities (220) and at airports on their outskirts indicates that the speed in the city is 65 per cent of that near the outskirts. (The value suggested by Fig. 7 is 59 per cent.) Elevation, shielding, and siting of the anemometers and periods of observation vary in every case but the trend is obvious.

A study of hurricane winds at Lake Okeechobee (Florida) by the U.S. Corps of Engineers (97) indicated that the wind off the land (everglades, covered with scrub cypress, etc.) averaged 60 per cent of the wind over the water when the latter was 50 mph and 74 per cent when the latter was 80 mph.

An article in "The Structural Engineer" by Ferrington (181, p.514) discussing the wind velocities found in Great Britain (in general terms a terrain characterized by treed, rolling country and straggling urban areas) contains the following interesting note. "On one occasion when the whole of the British Isles was covered with parallel isobars running nearly west to east, all stations on the western side gave the wind as force 8 (42 mph) while those on the eastern side gave force 5 (21 mph) so that the velocity was reduced by one-half in consequence of the "friction" of the land. If the velocity at the exposed western stations be taken at two-thirds the velocity of the wind free from friction, we get the following interesting result which is probably correct enough for practical use: one-third of the velocity is lost by the sea friction on the western side, and one-third more by the land friction of the country between west and east."

These again are very close to the values suggested by Fig. 7.

It is now possible to return to the problem of determining design velocities appropriate to different geographical regions.

The prediction of probabilities and return periods of extreme wind velocities has been suggested many times. In 1932 Wing (203) used the normal distribution curve to obtain the distribution curves for extreme wind velocities in several large American cities. A more recent writer Johnson (202) has remarked, however, that this type of distribution "gives a significant deviation from observed values". Since the date of Wing's correspondence the extreme value distribution (due largely to Gumbel (201)) has been developed; Johnson analyzed the anemometer records for both extreme indicated gusts and hourly mileages at 13 stations in Sweden and six in the British Isles and states that "the results obtained do not contradict the assumption that the distribution of extreme values of type No. 1 is in close agreement with the distribution of the actual wind velocities" (202, p.119). Court (199) analyzed the records of 25 weather stations in the United States having 37 years of satisfactory records according to the same theory and states "all of the wind data seems to follow the theory".

The form this distribution function takes is

$$\phi(x) = e^{-e^{-y}}$$

The reduced variate $y = a\ (x-u)$

where a is the scale factor and u the mode of the extreme value data.

Suppose that all anemometer records are analyzed to obtain the parameters a and u (in terms of which the return period of extreme surface velocities can be completely determined according to the extreme value theory). Then an estimate of the distribution of extreme gradient velocities at this location is given by the parameters ka and ku where k is a "roughness coefficient" defined by

$$k = \left(\frac{z_g}{z_a}\right)^{1/\alpha}$$

z_a being the height of the anemometer and z_g and $1/\alpha$ being given by Fig. 7.

The object now is to try to correlate these estimates of the gradient velocities to reduce the systematic errors which may have occurred in the anemometer records, improve the estimate of extreme wind velocities occurring at stations with shorter periods of records, and minimize the subjective elements involved in the determination of design wind velocities.

Suppose that anemometer "n" situated at latitude ϕ_n and longitude λ_n possesses records extending back a period of N_n years which, by extreme value analysis, yields values for the scale factor and mode of a_n and u_n. Suppose that the best estimate of the "roughness factor" is k_n, then the corresponding values of scale factor and modal value referring to the gradient velocity at this point are $k_n a_n = a'_n$ and $k_n u_n = u'_n$.

It is now assumed that these parameters follow some m^{th} degree contour surface (where m is less than the total number of records being analyzed) of the forms

$$a^1 = \sum_{j=o}^{m} \sum_{i=o}^{j} A_{ij} \lambda^i \phi^{j-i}$$

$$u^1 = \sum_{j=o}^{m} \sum_{i=o}^{j} B_{ij} \lambda^i \phi^{j-i}$$

The values of the coefficients A_{ij} and B_{ij} are now obtained by fitting the observed values to these surfaces by the method of least squares. This problem of course is one suited to electronic computation. In fitting the data it would be appropriate to weight the values first by the factor $\sqrt{N_n}$ which puts more reliance on records of longer period and second by a factor Q determined, unavoidably in most cases, by a subjective evaluation of the quality of the records, having regard to the siting of the anemometer, the number of times it has been moved, the possible amplification effects of mountains, valleys, etc. For example a first-class weather station at which the anemometer is situated on level ground well away from shielding and has not been moved might be given a weighting of 9 or 10 (out of a possible 10) and an anemometer situated close to the roof of a building, in a shielded area or in a valley, might only be weighted by a factor of 1 or 2. The least squares process minimizes the errors in a' and u' owing to the many causes cited (on the assumption that the weighted errors are normally distributed).

This process leads directly to the contours of a' and u' for the territory considered. If now it is wished to erect a structure at a certain location to last a period of T years ($T > 10$) with a risk of q (dictated by considerations of public liability, the use and occupancy of the structure, replacement costs, etc.) that the basic design wind velocity V is exceeded within this time, the return period R of this wind velocity V is given by

$$R = \frac{-T}{\log_e(1-q)}$$

$$\approx \frac{T}{q} \qquad \text{if } q \text{ is small.}$$

This represents a probability of $\frac{1}{R}$. By extreme value theory the value of the required gradient velocity

$$V_g = \frac{1}{a'}\left[-\log_e\left(-\log_e\frac{1}{R}\right)\right] + u'$$

where a' and u' are determined from the contours.

The values of the velocity nearer the surface corresponding to this gradient velocity are determined from Fig. 7 according to the appropriate roughness conditions.

If the structure is to be erected on a hill or in a valley a suitable amplification factor should be used. (Examples of these given by Pagon (48) and Putnam (53) are tabulated in Appendix II.) Often the only way to evaluate this would be by actual observation of wind velocities at the site for a short period and comparing their value with those at a nearby anemometer on level terrain.

2.4 Choice of a Suitable Averaging Interval

It has already been noted that owing to the wide fluctuations in wind velocity which take place over periods ranging from fractions of a second to many centuries it is necessary to measure wind velocity statistically in terms of a mean value and the deviations from the mean. The time or distance interval for which the mean is obtained depends upon the purpose for which it is required. To calculate basic wind velocities for the design of structures certain fundamental considerations determine which interval is most appropriate. These may be stated as follows:

(1) The interval should coincide as far as possible with some natural periodicity of the wind.

(2) The interval should be "long" compared to both the natural frequency of the structure and to the response time of the instrument: in this way there will be no dynamic interaction between the structure and the mean wind and measured wind velocities will be "true".

(3) The interval should be short enough to record the "peaks" of severe storms.

(4) The interval should correspond to a body of air of sufficient size to completely envelop a structure and its vortex regions.

It should be noted that in Canada the only wind velocity statistics recorded on a routine basis are the mean hourly mileages recorded by cup anemometers and the gust speeds (roughly a 3-sec average speed) recorded by a small number of fairly recently established Dines anemometers. Neither of these averaging intervals is suitable for obtaining basic design wind velocities for structures as can be argued from the fact that the hourly average does not satisfy conditions 1 and 3 and the 3-sec average does not satisfy conditions 1, 2, and 4.

On the other hand, it can be argued that the "mile of wind" or the "minute of wind" both represent ideally suitable, if not optimum, intervals for measuring high wind velocities for purposes of structural design. The reasons may be stated as follows in corresponding order to the stipulated conditions.

(1) By means of correlation coefficients, Durst (25) found that in storm winds major groups of eddies, thermal in origin, had wavelengths of about 4000 to 6000 ft - corresponding closely to the mile interval or the minute interval in winds of 60 mph.

(2) The natural period of most structures is of the order of 0.1 to 3 sec (235) with that for the Empire State Building of 8.14 sec (156). With the damping present in most structures fluctuations corresponding to one mile in extreme winds would have infinitesimal dynamic action.

Sherlock and Stout, referring to the response time of commerical anemometers, wrote in 1937 "that because of the inertia of moving parts of the instruments the records could only be accepted as accurate if they were averaged over 10 seconds or more" (72). Thus even at 150 mph, the mile of wind (or the minute of wind) satisfies the second requirement.

(3) The mile of wind also represents a body of air far larger than most structures, so that static pressures at least equivalent to this average speed can be anticipated.

(4) The mile of wind will be of sufficiently short duration to record the peak of a sudden severe local thunderstorm or squall.

These arguments justifying the use of the extreme mile or minute of wind as basic design velocities are endorsed further by the fact that the mile of wind has been recommended for use in the United States (77, 170) (where records are available for many years) as the basis for design wind velocities and the minute of wind in the British Isles (171). The modifications to the anemometers at present in use in Canada which would be necessary to convert from readings of hour averages to mile averages would involve little more than the replacement of the present recorder to one of faster chart speed. In view of the fact that at least 13 or 14 years of record should elapse before the wind velocity data are useful for statistical analysis, the modification of the present instruments would appear to be of utmost urgency. The cost of these modifications would be small compared to the greater safety and economy which could thereby be effected in structures throughout the country.

3 THE DETERMINATION OF WIND PRESSURES ON STRUCTURES

3.1 Wind Tunnel Studies and Shape Coefficients

Most of the knowledge relating to shape coefficients and the distribution of pressures on structures has been acquired from numerous and often extensive tests on models in wind tunnels. A selection of the published reports of these investigations originating from many countries and covering a wide variety of shapes is given in references 101 to 168.

These shapes include elementary geometrical figures (120, 123, 124, 139, 140, 141, 165) (cylinders, cones, flat plates, spheres), walls, (114) elementary pitch roof structures, (113, 114, 139, 140, 149, 157) rectangular block structures (103, 114, 139, 140, 157) including the effects of parapets (147), clerestoried structures (119,157), gambrel roofs (157), shed roofs (157), saw-toothed roofs (157), valley roofs (142,157), arched roofs (102, 114, 115), gas tanks (125), cooling towers (133), skyscrapers (117), bridge sections (110), trusses for towers and other structures (155), and numerous other shapes (146). Internal pressures (135, 139, 143) and shielding (103, 136) have been the subjects of other studies.

Figures 8a, b, and c show the characteristic two-dimensional flow patterns around a 45° pitch roof structure, a flat roof block structure and a semi-circular arch structure each standing on a flat ground surface. It is noted that the flow is divided into three distinct regions - a windward vortex region (A), a leeward vortex region (B), separated from the nonrotational flow (C) by a vortex layer (V) which envelopes the structure like an imaginary membrane, and in contact with the structural shape at the separation point (S). The windward vortex region may be thought of as an extension of the boundary layer. In regions in which the vortex layer is convex (with respect to the region of nonrotational flow) the pressure is positive (to windward) and where concave, is negative (to leeward). The static pressure on either side of the vortex layer is the same (141). Much of the kinetic energy within the vortex regions is dissipated into heat by turbulence. The pressures on surfaces of the structure are equal to that at the vortex layer modified by the degree of turbulent activity remaining. High suction prevails just behind the point of separations. In curved structures the position of S varies and is dependent on the Reynolds number $\left(\frac{\text{velocity x characteristic dimension}}{\text{kinematic viscosity of fluid}}\right)$, the surface roughness of the body, the turbulence of the oncoming flow and the upstream velocity profile. Hence the pressures also depend on these factors. On sharp-edged structures, however, the separation found normally occurs at one of the sharp edges, and the flow patterns and hence the pressures are largely independent of Reynolds number and surface roughness, but not of the upstream velocity profile. Exceptions to this statement are found, however, when the vortox layer approaches tangency to one of the surfaces, in which case a further turbulent zone may be set up over this surface with high suctions. In this case Nøkkentved obtained direct proof of a strong Reynolds number dependence for a definite type of building, broad in proportion to height and with a flat roof slope, particularly marked with a roof slope of 20° (149). The same situation can arise when the wind is incident upon the walls of a structure at small angles.

The presence of the ground surface exerts a considerable stabilizing influence on the pressure distributions which, except just to the lee of the separation point, are for all practical purposes static if the flow is steady. If the ground plane is not present (Fig. 9), as is effectively the case in tall slender structures such as smoke stacks, marked instability can arise in the pressure distribution: this case requires special consideration and will be discussed in section 4.3 which deals with the vibration of structures.

3.2 The Validity of Wind Tunnel Results

The results of wind tunnel tests on models are generally valid for full-scale prototypes provided that three conditions are fulfilled (196):

(a) that there is geometrical similarity between the model and prototype

(b) that there is equality of Reynolds numbers

(c) that there is kinematic similarity in the approaching flows.

Provided that only over-all effects are being studied the first condition presents no problem. The second condition, however, is not so easy to satisfy since equality of Reynolds numbers requires that the velocity and characteristic dimension for the model and prototype be kept in inverse ratio. That is, if the model is 1/100th scale, the velocity in the wind tunnel should be one hundred times as great as that to which the prototype is exposed, provided, of course, the fluid used in both cases is air at atmospheric temperature and pressure. The size of the model, in turn, is determined by the fact that the cross-sectional area should not exceed about 5 per cent of the cross-sectional area of the wind tunnel. The construction effects which result if this figures is exceeded can be quite large (157). These practical restrictions generally prohibit equality of Reynolds numbers being achieved.

Fortunately, however, for most sharp-edged structures inequality of Reynolds numbers between model and prototype are inconsequential as has already been described. Curved structures such as cylindrical stacks, arched roofs, etc., normally exhibit a fairly constant pressure distribution between Reynolds numbers of 10^4 and 2×10^5 at which point a transition occurs producing a different but again practically constant pressure distribution for Reynolds numbers in excess of 5×10^5. The borderline between these two patterns is primarily a viscous function and transition from one flow pattern to the other can be artificially stimulated by roughening the surface of the body or by increasing the turbulence of the approaching flow by means of mesh screens.

In this way the effects of inequality of Reynolds numbers for models and prototypes with curved surfaces can be kept to a minimum as in the case of sharp-edged structures.

The third requirement for the validity of model tests, namely, that there is kinematic similarity in the approaching flows, is far more difficult to achieve, and in only one or two tests has this requirement been even partially satisfied. It is therefore important to indicate the consequences of kinematic dissimilarity in the approaching flows insofar as the pressures on full-scale structures might differ from those in the wind tunnel.

3.3 The Consequences of Kinematic Dissimilarity

The kinematic properties of the natural wind which require simulation in the wind tunnel are, basically, the increase of mean velocity with height and the turbulence.

It is normal in wind tunnel tests to test models as far as possible in a uniform velocity. This is achieved by mounting the models on smooth base plates and supporting these near the centre of the tunnel away from the frictional effects near the wall. The shape coefficient, C_s, at a point on the surface is thus given by

$$C_s = \frac{P - P_o}{1/2 \rho V^2}$$

where $P - P_o$ represents the difference between the pressure at the point on the surface of the structure and the static pressure in the upstream flow and $1/2 \rho V^2$ is the velocity pressure of the upstream flow.

Simulation of any given rate of increase of velocity with height in the wind tunnel can be accomplished, however, by using graduated mesh screens placed in the upstream flow or by making use of the natural profile occurring at the walls of the wind tunnel. One such test along these lines was carried out by Bailey and Vincent (103); in this the model was placed on the floor of the tunnel where the rate of velocity increase corresponded closely with the 1/7th power law. The shape coefficients were referenced to the velocity pressure at a scale height of 40 ft above ground, at which point the velocity pressure was some 40 per cent less than that occurring away from the walls. In spite of the increase in velocity with height "the analysis of the results ... showed that in all cases the effect is practically equivalent to a uniform pressure over the whole projected height".

This statement is distinctly at variance with the assumption commonly used in practice, that the pressure on a structure at any given height is directly proportional to the velocity pressure of the oncoming flow at the same level. This assumption may be true for tall, slender structures of high aspect ratio such as radio towers, but is evidently not true for the more usual building shapes. The total pressures in lb/sq ft acting on the various structural shapes tested by Bailey and Vincent are given in Fig. 10 (in terms of an 80-mph wind velocity at the reference height of 40 ft). It is noted that these pressures increase with the height H of the structure and that all the experimental results lie close to the line (B) expressed by

$$q \propto H^{\frac{1}{1.33}} .$$

Compared with this result is the increase of pressure with height given by the practical rule already mentioned; namely, that the pressure on a structure at any height is proportional to the velocity pressure of the wind at the same height. For the 1/7th power law increase in height this corresponds to

$$q \propto H^{2/7} .$$

This relationship is given by line (B) in which the shape coefficient of 1.5 and the velocity of 80 mph at 40 ft are assumed. The disparity between the two curves is immediately noticed and suggests that where the wind velocity profile corresponds to the 1/7th power law the assumption that the pressure on a structure increases at the same rate as the velocity pressure may considerably over-estimate the total horizontal drag forces on a structure less than about 100 ft and under-estimate these forces on a structure greater than this height.

These findings of Bailey and Vincent concerning the effect of the increase of velocity with height on the pressure distribution should also be considered in conjunction with other experiments in which the effect of turbulence on the pressure distribution is also studied.

Turbulence is the other factor leading to kinematic dissimilarity between the flows of the wind tunnel and the natural wind, it being usual to use laminar flow in the model tests.

In the natural wind, eddies of a wide range of size a superimposed upon one another. Some eddies will be very much smaller than the structure itself: these may be expected to alter the static pressure on structures (especially in rounded structures where the pressures are sensitive to variations in the position of the points of separation) but may have little or no dynamic effect. The second type of eddy, of the same order of size as the structure itself, may be responsible for causing local dynamic changes in pressure while the third type of eddy, much larger than the structure and its surrounding vortex layers (and probably more commonly referred to as gusts), may be responsible for large dynamic pressure changes affecting the structure as a whole.

Only the effects of small-scale turbulence are discussed immediately, the other two categories being discussed under the heading of Gust Action.

One of the very few published reports of tests under turbulent flow is that by Kamei (143) in Tokyo. In these ·bulence was induced in the flow by lattice screens; in some of the tests mesh grids were used to simulate an increase of velocity with height as well.

The results indicated that windward pressures were slightly less than in laminar flow and the reductions in leeward pressure considerably less (up to 50 per cent) for both rounded and sharp-edged structures. Comparative tests on full-scale prototypes evidently yielded pressures similar to those found in a turbulent wind tunnel, i.e. the windward pressures on full-scale structures were slightly less than in laminar flow wind tunnel tests and the reductions in leeward pressure up to per cent less.

These results are contrary to Bailey's findings concerning the pressures on a shed (105) which resulted in the following comment in the paper by Bailey and Vincent (103).

"Experiments on wind pressures on a full scale building under natural conditions and on its counterpart in a wind tunnel ... indicate that whilst the general form of pressure distribution is similar, there is a greater reduction of pressure (up to 50 per cent more) on the leeward side of the full scale building than would be estimated from the model tests. Similar evidence on this point was obtained by Stanton".

Only one other investigation, relevant to this discussion, comparing the pressures on a prototype structure and its wind tunnel model appears to have been reported: this was on the Empire State Building (54). One of the observations of this paper was:

"A comparison of the pressures on the model and those on the building shows clearly that the natural wind movements are not at all like those in a wind tunnel."

In these full-scale tests the pressures were measured by manometers at ten points at each of the 36th, 55th, and 75th floors. The external pressure readings were referenced to an unknown internal pressure inside a closet on each floor and therefore only the total pressure difference of the wind on the building was of any meaning. In spite of the seemingly haphazard distribution of the pressures (due largely probably to the wide spacing of the pressure tubes and shielding from other buildings) two trends seem to be apparent. First, that on the whole the pressures are much less (up to 50 per cent) than wind tunnel results would suggest at the measured wind velocities. Second, that in spite of the very rapid increases of velocity with height to be found in the city (1/2 power law) the pressures at the 36th floor do not differ greatly and are sometimes greater than those found at the 75th. There is no suggestion that the pressure increases at the same rate as the measured velocity pressure. This agrees entirely with the findings of Bailey and Vincent discussed earlier.

Apart from the incomplete information to be derived from these few investigations it appears that no comprehensive study of the consequences of the marked dissimilarity of the flow in the wind tunnel and the natural wind has hitherto been published. In practical application this lack of experimental or other data has been met by certain assumptions which may be stated as follows:

1. The differences of pressure between prototype structures and models tested in steady flow which may arise due to turbulence in the natural wind are not significant.

2. The increase of velocity with height found in the natural wind (but not normally simulated in the wind tunnel) can be accounted for by an increase in pressure on the full-scale structure with height proportional to the increase in velocity pressure.

The experimental evidence referred to indicates that both these assumptions are highly tenuous: the small-scale turbulence may have a significant effect on the shape factors describing the distribution of pressure on a structure; the horizontal pressures on broad bluff buildings are effectively uniform even if the velocity increases rapidly with height; the magnitude of this uniform pressure increases with the height of the building in a manner which is not simply related to the increase of velocity pressure.

In view of the large number of detailed wind-tunnel studies on a wide variety of structural shapes it is perhaps remarkable that such important factors as those mentioned above, upon which the validity of the application of the wind-tunnel results must depend, should have received so little attention.

3.4 Shielding Effects

A structure lying to the lee of another structure will in general experience substantially reduced pressures. Various studies have been conducted on this subject (103,135, 136,144) including that due to Bailey and Vincent at the National Physical Laboratories in which the following comment appears with regard to their experimental findings on the effects of shielding.

"The general conclusion to be drawn from these tests is that when a building is shielded by another of the same order of height on the windward side, a reduction in the wind-pressure factor is permissible, the amount depending on the distance apart; if the shielding building is relatively low only a small reduction, if any, is permissible.

"These tests have been carried out with only one or two models to the windward of the test-model, but the shielding effects do not appear to be much affected by the number of models and it appears reasonable to suppose that the results would be applicable to a normal built-up area.

"It is clear that for general design purposes it would not be practicable to treat each case separately and allow for the shielding effects of existing surrounding buildings, partly because this would be an unnecessarily complicated procedure but mainly because the conditions might be varied after the building was erected. On the other hand, the results of the tests show that in a built-up area, even with buildings quite large distances apart, there is a substantial shielding effect and it is unnecessary therefore to allow for the fully exposed loading....

"A practical method of making this allowance would be to take reduced values for the lower portions of a building up to some specified height, say 100 feet, above which the fully-exposed values would be taken."

Although, as is indicated by this conclusion, shielding generally reduces pressures on a structure, in one p ticular instance the pressures can be significantly increased. If a structure stands in the lee of a shielding obstacle negative pressures will sometimes be produced over all surfaces of the structure. The effect of this is to expose all roofs, including those with very steep pitches, to the possibility of uplift. It would therefore be advisable to design all roofs for an uplift shape coefficient of -1.0 particularly in built-up areas where the vagaries of the airflow are highly unpredictable. In all other circumstances the shape coefficients in the unshielded condition yield the maximum pressures.

The general over-all allowance for shielding in built-up areas suggested by Bailey and Vincent in the last paragraph of the above quotation is to a large degree provided for by the reduction in surface wind velocities in cities and towns advocated in Section 2.2 of this paper.

The only other circumstance under which allowance c ld be conveniently made is in the case of bridges, tower trusses and other structures in which the shielding is afforded by other components of the same structures. This is dealt with in most papers dealing with wind loads on these structures (154, 166).

3.5 Formulation of Shape Coefficients

The formulation of design shape coefficients is governed largely by consideration of accurate reproduction of the pressures and their resultant moments and shears, of simplicity and of universality of application. Until the present the last two considerations would appear to have been given substantially greater weight in the formulation of the wind-load requirements to be found in most codes with the result that the design-shape coefficients advocated, sometimes bear little resemblance to the original wind-tunnel experiments. The requirement of the National Building Code of Canada is a case in point and is not unlike many other codes. In this only wind actions along the major axes of the building are specified; distinctly nonuniform pressures are considered uniform; the influence of building height on the roof pressures of pitched roofed structures is not noted; and the treatment given to circular structures is inaccurate.

A departure from this trend which brings engineering specifications more into line with the results of wind-tunnel studies is provided by the Swiss standards which appears in translation as an appendix to this paper (see also reference 159).

4 DYNAMIC EFFECTS OF WIND

1 Causes of Dynamic Pressures

Wind produces dynamic changes in pressure on structures in two ways. First by fluctuations in the flow velocity, more commonly known as gusts, and second by periodic variations in pressure which are due to instability in the flow patterns formed around certain bluff shapes, to deflections of the structure itself inducing a different and greater pressure distribution, or to a combination of both. The first type of dynamic pressure change (which in general is aperiodic) is discussed under the heading "Gust Action and Gust Coefficients" and the second (which in general is periodic), under the heading "Vibration of Structures".

4.2 Gust Action and Gust Coefficients

The fundamental problem related to gust action is to determine what equivalent static pressures, in excess of those given by the basic design wind velocity (fastest "mile" ("minute") in conjunction with the appropriate shape coefficient, are exerted by gusts of higher velocity striking a structure suddenly.

It is convenient at this stage to discuss the problem in relation to what will be termed a gust coefficient, defined as the ratio of the maximum equivalent static pressure exerted on a structure to the pressure exerted by the basic design wind velocity. (This is different from the gust factor now used extensively which is the ratio of the peak gust to the concurrent mean wind velocity. This quantity, based purely upon peak wind velocities without regard to the duration of the gust or of the dynamic effects, and which is largely dependent on anemometer sensitivity, gives a false estimate of the equivalent static pressures to be expected.)

It is to be expected that the gust coefficient so defined will depend on:

(a) the intensity of the gust relative to the mean wind velocity

(b) the size and duration of the gust relative to the size of the structure

(c) the dynamic response of the structure and its constituent materials to dynamically applied loading.

(It will be noted that the basic design wind velocity based on the fastest mile or minute was chosen specifically in order that the basic wind pressures should be independent of the above factors.)

It should be stated at once that it would appear that the experimental data necessary for an evaluation of gust coefficients, which would take these factors into account, are lacking. There has nevertheless been a considerable amount of work done in the field of natural air turbulence and gust action; although it is not possible to describe all the results, an attempt is made to summarize those which, it is thought, will be of importance when eventually a more complete determination of gust coefficients is possible.

The properties of natural air turbulence have been studied for a wide number of purposes by many investigators (see references on wind structure). This turbulence arises through two main causes, thermodynamic interchange between eddies of air at different temperatures or pressures and mechanical deflection around obstacles near the earth's surface.

Eddies of the first kind are thermal in origin and tend to be much larger than mechanical eddies. Durst (25) found by means of correlation coefficients, evidence for a major group of thermal eddies about 4000 to 6000 ft long and [illegible] ft wide and Heywood (210) states in typhoon winds at Hong Kong that the wave length of large-scale thermal eddies was of the order of 1000 ft. On the other hand, Durst (25) found that mechanical eddies were about 50 ft in diameter or about the same size as the objects which caused them; Sherlock found a similar result (71).

Both Sherlock (71) and Giblett (25) suggest that surface velocities in the larger thermal eddies can attain that of the gradient wind. In general the distribution of gust velocities follows a Maxwellian law (27, 99).

The gustiness, i.e. the range over which velocities fluctuate, decreases with height, that is to say, at higher levels the distribution of velocities lies much closer to the mean velocity (see Wax's results (99) Fig. 11). The gust velocity decreases with an increase in the interval over which gust velocities are averaged. These general results have been corroborated by others (15, 79).

The rates of change of velocity that can be measured depend largely on the sensitivity of the instrument. Using an anemometer far more sensitive than those currently used in practice for meteorological purposes, Wax (99) recorded (at hourly mean wind velocities of about 40 mph) changes of more than 5 mph at rates in excess of 200 mph per second; in one case a velocity change from 52 to 84 mph in 0.25 sec was recorded. The impactive effect this gust could have on a small structure might be very large.

Several investigators have been able to compare the velocity across a front of wind with the velocity measured at a point. Stanton (161) found from the records of six anemometers mounted on 70 ft poles at intervals across a 420 ft horizontal front that "in only eight percent of the records did the average (velocity) pressure exceed 90 per cent of the (velocity) pressure at the reference point, and in 75 per cent of the records the average pressure was below 80 per cent of the corresponding value at the reference point" (161 p. 131). Similar results were found over a very much wider front by his successors Bailey and Vincent (104) at the site of the Severn Bridge.

These results suggest that the additional pressures caused by gusts will be greater on a small structure than on a large. In his notable experiments at the Forth Bridge, Sir Benjamin Baker (106, 107) corroborated this expectation, finding that the pressures on a wind-gauge board 300 sq ft in area were some 50 per cent less than those on a board 1 1/2 sq ft in area. Sherlock (71) found by studying the force exerted on a 120 ft conductor wire that the effective velocity at any one moment averaged over the whole of this length was 3.1 per cent less at 37 mph and 8.2 per cent less at 45 mph than the velocity at a point. The velocity pressures were correspondingly 6.2 and 16 per cent less. In comparing Sherlock's results with those of Baker, it should be remembered that in the former case the reductions in pressures refer to a line and in the latter, to an area.

These rough indications reveal the significant effect the size of a structure may have on the gust pressures which effectively act upon it. This reduction in effective gust pressure arises through two general causes: namely, the peak velocity averaged over a large frontal area of wind generally less than that over a small and also the lapse of time required for the full gust pressure to build-up over a large structure, is longer than that over a small.

This last factor may be understood from the action of gusts on aircraft wings (188, 189) in which it is found that an aerofoil must penetrate a sharp-edged gust six chord lengths before even 95 per cent of the full lift forces build up. Although the build-up of drag pressures due to gusts on sharp-edged structures is not altogether analogous to the build-up of lift forces on an aerofoil, it is reasonable to assume that structures which, together with their adjacent vortex regions, are large will experience substantially lower pressures and less rapid build-up rates than smaller structures. (Consideration of this factor has prompted Sherlock (77) to recommend a gust factor of 1.3 for sign boards and small residences and 1.1 for structures having a horizontal dimension of 125 ft.)

It has already been noted that these questions concerning the intensity and rate of build-up of pressures on a structure in relation to the fluctuations, intensities, sizes, and durations of incident gusts cannot at present be answered owing to the almost complete lack of experimental evidence on this particular aspect.

In this respect, however, one series of experiments deserves special note, namely, Stanton's measurements of the wind pressure on the high level footways of the Tower Bridge in London (161).

In these experiments pressure tubes were mounted both on the windward and leeward faces of the footway girders at intervals across the span. These were coupled to mechanically linked aneroid pressure chambers in such a way as to yield the total average pressure acting on the footways.

This average pressure and the concurrent "point" pressure (measured across a single section of the span) were recorded continuously on a chart drum.

These records appear to be the only continuous measurements ever made of the wind pressures exerted on a structure. Unfortunately it was not possible to record simultaneously the velocity of the incident wind giving rise to these pressures and the valuable opportunity to correlate the fluctuation of the pressures on a structure with the corresponding fluctuations in the wind was lost.

The records, however, do indicate that the pressures across a single section of the span correspond closely with the average pressures acting on the span as a whole (both in respect to the intensities and the fluctuations in pressure). This can be seen from the fact that the total areas under the two simultaneous chart traces of the "point" and average pressures did not differ by more than 4 per cent, indicating that even if velocities are appreciably higher over one portion of the span the pressures on the structure (within the vortex regions) may still reflect only the average pressure over the vortex layer. Stanton ignored this possible explanation of the close correspondence between "point" and average pressures and suggested that it was due simply to the fact that the velocity variation across the front must have been small.

If pressure traces such as those obtained by Stanton were available for typical structural shapes together with the high speed velocity traces of the incident wind the first steps in the process of extracting gust coefficients would have been made.

The study of the correlation coefficients of the pressure traces and the concurrent velocity trace would probably shed much light on the relation between the intensity of the pressures acting and the gust intensity (relative to the mean wind), duration, and size relative to the size of the structure. A study of this nature, it may also be hoped, would enable the relationship to be inferred for structures other than those tested.

The further problem also exists of determining what equivalent static pressure gives rise to the same deflection stresses, etc. as the dynamically applied gust pressures - thus determining the gust coefficient. In this respect it is perhaps fortunate that analytically the dynamic action of fluctuating gust pressures is little different from that caused by the horizontal components of seismic forces. The advanced analytical techniques (269) already derived for predicting the dynamic amplification resulting from the latter (by use of autocorrelation coefficients) would also be useful for present purposes.

Since the inception of the plastic method for the design of structures, it is logical to give further consideration to the behaviour of the structure in the plastic region before collapse occurs. In an article by Horne (183), one of the chief proponents of the plastic design method, a method is described whereby it is possible to calculate the time for which any constant load must be applied to a portal frame to bring about collapse. It is shown that the collapse load increases substantially if the time of application decreases. In this paper, consideration is also given to a property of most materials, that the yield stress increases with the rate of load application. (Recent tests for example, on structural grade steel ST 37 showed that for this material which has an average static yield stress of 34,000 psi the yield stress under impulsive loading was 50,000 psi.)

These investigations by Horne, however, suffer from lack of any information on the manner in which gust pressures build up. This prompted the following comment in suggesting future research:

"The pressures exerted by natural winds on structures have been investigated by Stanton, Bailey and Vincent, Rathbun, and others, and information has been sought regarding the distribution of these pressures at any given instant. In general, however, the pressure points have been too far apart for a sufficiently detailed picture of the pressure distribution to be obtained, and the methods of recording have not been sufficiently sensitive to follow the rapid changes of pressure which occur. A really satisfactory investigation would be difficult and expensive to carry out, but is essential if a true picture of these pressures is to be obtained."

The observations of this section have indicated no basis from which even an approximately correct value for gust coefficients may be evaluated. The conclusion must be reached that this will prove impossible until such time that research along the lines suggested by Horne and also in this paper has been carried out.

As Horne (270) has remarked:

"The damage inflicted on a structure by winds can only be assessed by considering the effect of the highest gusts, the short duration of which necessitates the consideration of dynamic behaviour. Suitable wind pressures for use in design ought, therefore, to be based on an assessment of such damage, not as hitherto merely on the mean wind velocity, or on the highest momentary velocity which happens to be recorded by a particular type of instrument."

4.3 Vibration of Structures

Intimately connected with the subject of dynamically applied gust loads is the vibration of structures (see references numbers 232 to 266).

Serious vibration, generally implying vibration at one of the natural frequencies of the structure, can arise in two ways:

(a) Self excitation in which the form of the structural element is such that deflection due to an applied wind force increases the wind force. This type of aerodynamic instability was true, for instance, of the section used in the deck of the Tacoma Narrows bridge.

(b) Forced vibration due to instability in the flow pattern induced by the shape of the structure itself or due to a natural periodicity in the air flow itself.

Often both (a) and (b) occur simultaneously.

Normally in the case of structures (b) is more important except in the case of sign boards (150), suspension bridges, and certain shapes of suspended roofs where both are important. Forced vibration is exceptionally dangerous when resonance occurs with one of the natural frequencies of the structure. In wind, resonance can arise in two ways. First a resonance with periodic gustiness can occur.

Wax (99) investigated the periodicities in natural gusty wind by scanning anemometer traces photoelectrically at steadily decreasing speeds and passing the output through a narrow fixed frequency band-pass filter. The results indicated an amplitude of 0.07 mph at a frequency of 1/2 cps decreasing steadily to about 0.01 mph for a 25 second period, at an elevation of 88 ft above ground. At 118 ft above ground the maximum amplitude was 0.04 mph at 1/2 cps. This is an indication that such periodicities do exist. Rathbun discussing the Empire State Building states "a gusty wind will set up vibrations with amplitudes that vary with the strength and character of the storm".

With little information on the way in which gust pressures build up on a surface it is difficult to know exactly to what structures and at what wind speeds serious vibration of this nature may occur. The phase difference between the build-up of pressure on the windward and leeward vortex regions and surfaces may alleviate much of this threat especially for structures having considerable depth in the direction of the wind, for which this phase difference will be greatest. Durst (25) has observed that a large family of frictional eddies are about 50 ft in diameter. This is perhaps corroborated by Wax's results which indicate the largest amplitudes of fluctuation at frequencies of 1/2 cps when the wind velocity was about 100 ft per sec (70 mph). This may mean that a shallow structure with a natural frequency of 1 cps may be highly susceptible to vibration in wind speeds of the order of 50 cps. The author has noticed this type of occurrence in the case of tall cantilevred arc lamp standards at a football stadium. The natural frequency of these structures was about 1 cps; in a wind of 30 to 35 mph they would sway alarmingly - far more than the designer could ever have contemplated if he had given due regard to the disquieting influence the oscillations could have on spectators sitting beneath the swaying standards.

These factors would cast some doubt on the soundness of a comment by Irminger and Nøkkentved (140) discussing gust resonance that "it is modern practice with, for instance, radio masts to calculate the oscillation period which should not be allowed to exceed 3 to 4 seconds".

Another type of forced vibration affecting structures is the phenomenon associated with the Karman Vortex Street (Fig. 9(b)).

For any <u>bluff</u> cylinder between certain Reynolds numbers, the symmetrical airflow around it becomes unstable: the air prefers to flow first round one side of the structure and then around the other. The unbalanced circulatory momentum this produces at the end of each cycle causes a trail of vortices to be left behind the cylinder in a stepping-stone pattern. This unbalanced alternating flow first round one side and then the other is, by Bernoulli's theorem, also attended by a lift force on the side of the cylinder. This is the cause of the familiar phenomenon of a stick, shaking in water at right angles to the flow. It has also been responsible for damage to several chimneys, circular television antennae and other structures (235). The same forces however may be just as much at work on tall bluff buildings particularly of squarish section. Damping is one of the factors controlling the amplitudes that will arise owing to this action. Almost all tall buildings at present incorporate large quantities of concrete and masonry which are highly effective damping agents. If in the future similar structures are erected in which the frame and the cladding are all metal the damping would be far less and this dynamic force might conceivably become of considerable significance.

The effects of vibration are not only related to structural safety but also to the comfort of occupants. During the hurricane which struck Miami in 1948 (216) this note appears in the Miami Herald of October 6th of that year. "County jail prisoners high up in Dude County Skyscraper Courthouse were disturbed Tuesday night as the building swayed in the high winds. The chief deputy reported that some of the prisoners kept track of the swaying by watching the movement of water run into washbowls." In the article recording the research on the Empire State Building (54) other manifestations are recorded of the discomfort caused by vibrations. In one instance, in a dance hall, (not at the Empire State Building), the vibrations caused nausea and in another the building became commercially untenable because of discomfort caused by vibrations.

Various expressions have been put forward as defining the limits of personal comfort in the case of ridges (268). These are given in terms of the amplitude, a, and frequency, f, by

$af^3 < 2$ for frequencies 1 to 6 cps,

$af^2 < 1/3$ for frequencies from 6 to 20 cps.

The subject of vibration has not been one which has concerned building codes very much anywhere in the world. The increase in the number of buildings being erected, however, which are metal and devoid of the damping effects of masonry and concrete puts the subject of vibration, both from its structural and psychological effects, further to the fore.

5 RECOMMENDATIONS

The following recommendations are based directly on the findings of this report.

5.1 Basic Design Wind Velocities

5.1.1 Basic design wind velocities should be based on long-term anemometer records of extreme velocities averaged over an interval of one mile (or one minute). Owing to the length of time necessary for sufficient records to accumulate, facilities for obtaining these results should be established without delay so that the obtaining of accurate wind load records shall not be further prolonged.

5.1.2 Special consideration should be given to the siting of anemometers in order that they shall be free from possible amplification and shielding effects of buildings and other obstacles. They should be situated wherever possible on level terrain. At airports, for example, the anemometer should be situated near the centre of the field in preference to the top of a building.

5.1.3 Basic design wind velocities should be determined from a statistical analysis of long-term anemometer records and should refer to level country.

5.1.4 The effects of surface friction should be taken into account in the statistical determination of design wind velocities.

5.1.5 Designers should be advised of the possible amplification effects of mountains and valleys on these basic design wind velocities for level country.

5.1.6 The basic design wind velocity which a particular structure shall be capable of resisting should have a probability of occurrence directly related to the risks contingent upon this wind velocity being exceeded during the structure's anticipated life. This risk is determined by consideration of the siting of the structure, (i.e. whether it is built in a heavily or lightly populated area), the use and occupancy of the structure, the seriousness of any resulting damage, and the cost of replacement.

5.2 Increase of Wind Velocity with Height

5.2.1 The effects of ground roughness should be taken into account in the specification of the increase of basic design wind velocity with height.

5.2.2 Experimental research should be conducted to determine the influence of the increase in velocity with height on the pressures (and shape coefficients) effective on a structure. Until such time as this investigation has been carried out it would be cautious to assume in calculation of all design pressures that the velocity pressure acting at the highest point of the building is effective at all levels.

5.3 Shape Coefficients

5.3.1 Shape coefficients should be formulated in such a manner as to indicate all significant variations in pressure distribution likely to arise over the surfaces of elementary structures because of variations in shape, orientation to the wind, Reynolds number, etc. The formulation of shape coefficients adopted in the Swiss Standards would seem to satisfy all these requirements and could for the present be adopted in their entirety (see Appendix III).

5.3.2 Experiments should be carried out to determine the effects of small-scale turbulence in the airflow on the distribution of pressure on a structure.

5.3.3 The attention of designers should be drawn to the possible effect of shielding in producing uplift forces on all roofs. (A shape coefficient of -1.0.)

5.4 Gust Coefficients

5.4.1 Experiments should be conducted to determine the rate of build-up of gust pressures on sharp-edged structures and the influence of the size of the structure on the resulting pressures.

5.4.2 Experiments should be conducted on a variety of full-scale structures to determine the correlation between fluctuations in wind velocity and fluctuations in the actual pressures developed on the various structures.

5.4.3 These records of pressure fluctuations should be analyzed (in a manner similar to that used in analysis of seismic effects) to determine what range of dynamic amplification factors might be expected in typical structures.

5.4.4 Until such time as these experiments have been completed, gust coefficients of 1.7 for small signboards and residences and 1.2 for structures having a minimum horizontal dimension of 125 ft might be adequate (although there is no justification for putting very much confidence in these values).

These gust coefficients (suggested by the gust factors recommended by Sherlock) are applicable only to the fastest mile or minute velocities.

5.4.5 The attention of designers should be drawn to certain structures such as smoke stacks in which instability of the airflow may give rise to serious vibration.

5.5 The pressure on the surface of a structure should be designated by the equation:

$$q = C_g \cdot C_s \cdot C_a \cdot 1/2 \rho V_h^2 \left(\frac{H}{h}\right)^{2/\alpha}$$

where C_g = the gust coefficient dependent on the size of the building

C_s = the shape coefficient

ρ = air density

C_a = an amplification factor due to possible orographic or funnelling effects

V_h = the velocity at height h

H = the height of the structure

$1/\alpha$ = an exponent for velocity increase with height determined by the surface roughness in the vicinity of the site.

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TABLE I

The influence of surface roughness on values of the exponent $1/\alpha$ in the power law variation of mean wind velocity with height under conditions of strong wind or adiabatic lapse rate between heights of 30 ft and 1000 ft.

Curve No.	Source	Location	Upper Limit of Investigation (ft)	Description of Terrain in Site Locality	$\frac{1}{\alpha}$	Remarks
1	Goptarev	Caspian Sea (off Apsheron Penin.)	166	Coastal waters of inland sea	$\frac{1}{10.5}$	
2	Juul	Masnedsund, Denmark	182	Flat shore on "Ocean of Small Islands"	$\frac{1}{8.3}$	
3	Scrase	Salisbury Plain, England	43	Open grassland without hedgerows or trees	$\frac{1}{7.7}$	
4	Wing	Ballybunion, Ireland	492	Flat treeless grassland, Atlantic Ocean ½ mile distant	$\frac{1}{7.4}$	
5	Sherlock	Ann Arbor, Michigan, U.S.A.	250	Open slightly rolling farm land	$\frac{1}{7}$	
6	Taylor	Salisbury Plain, England	-	Open grassland without hedgerows or trees	$\frac{1}{7}$	
7a	Giblett	Cardington, Beds., England	150	Open level agricultural land with only isolated trees	$\frac{1}{7.8}$	
7b	Frost	Cardington, Beds., England	350	Same as above	$\frac{1}{6.9}$	
7c	Frost	Cardington, Beds., England	1000	Same as above	$\frac{1}{6.7}$	
8	Deacon	Sale, Victoria, Australia	503	Gently rolling grazing land with few trees	$\frac{1}{6.25}$	
9	Heywood	Leafield, Oxfordshire, England	313	Open fields divided by low stone walls and hedges	$\frac{1}{5.9}$	
10	Kamei	Japan	-	Rough coast	$\frac{1}{5}$	Observations not necessarily typical of level country
11	Wax	Orkney Islands	118	Flat topped grass hill, 1/3 mile inland from high cliff overlooking the sea	$\frac{1}{4.6}$	
12	Huss and Portman	Akron, Ohio, U.S.A.	352	Gently rolling country with many bushes and small trees	$\frac{1}{4.55}$	
13	Francken-berger and Rudloff	Quickborn, Germany	230	Relatively level meadow land but with numerous hedges and trees around the small fields	$\frac{1}{4.35}$	
14a	Smith	Upton, Long Island, N.Y., U.S.A.	410	Level country uniformly covered with scrub oak and pine to a height of 30 ft	$\frac{1}{4}$	13 winter storms
14b	Panofsky	Upton, Long Island, N.Y., U.S.A.	410		$\frac{1}{3.85}$	
14c	U.S. Weather Bureau	Upton, Long Island, N.Y., U.S.A.	410		$\frac{1}{3.3}$	Hurricanes Carol and Edna 1954: (fastest 1 min. mean speed)
14d	U.S. Weather Bureau	Upton, Long Island, N.Y., U.S.A.	410		$\frac{1}{2.9}$	(fastest 6 min. mean speed)
15	Kamei	Japan	-	Three Japanese towns	$\frac{1}{3}$	
16	Dines	Farnborough, England	1650	Wooded and treed farm land	$\frac{1}{2.8}$	
17	Jensen	Copenhagen, Denmark	242	Centre of a large city	$\frac{1}{2.1}$	
18	Taylor	Paris, (Eiffel Tower)	900	Centre of a large city	$\frac{1}{2}$	
19	Rathbun	New York (Empire State Building)	1263	Centre of a large city	$\frac{1}{1.6}$	

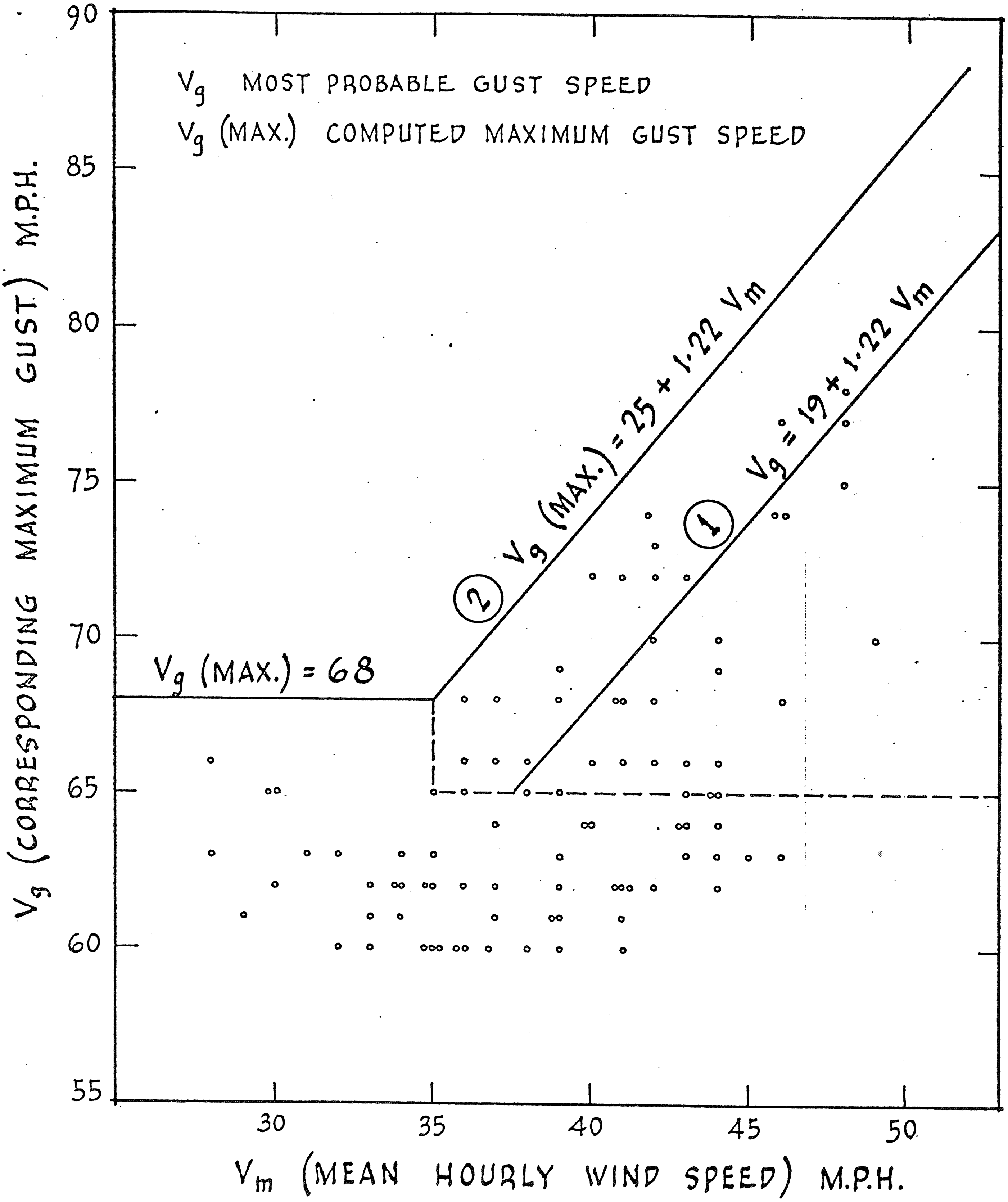
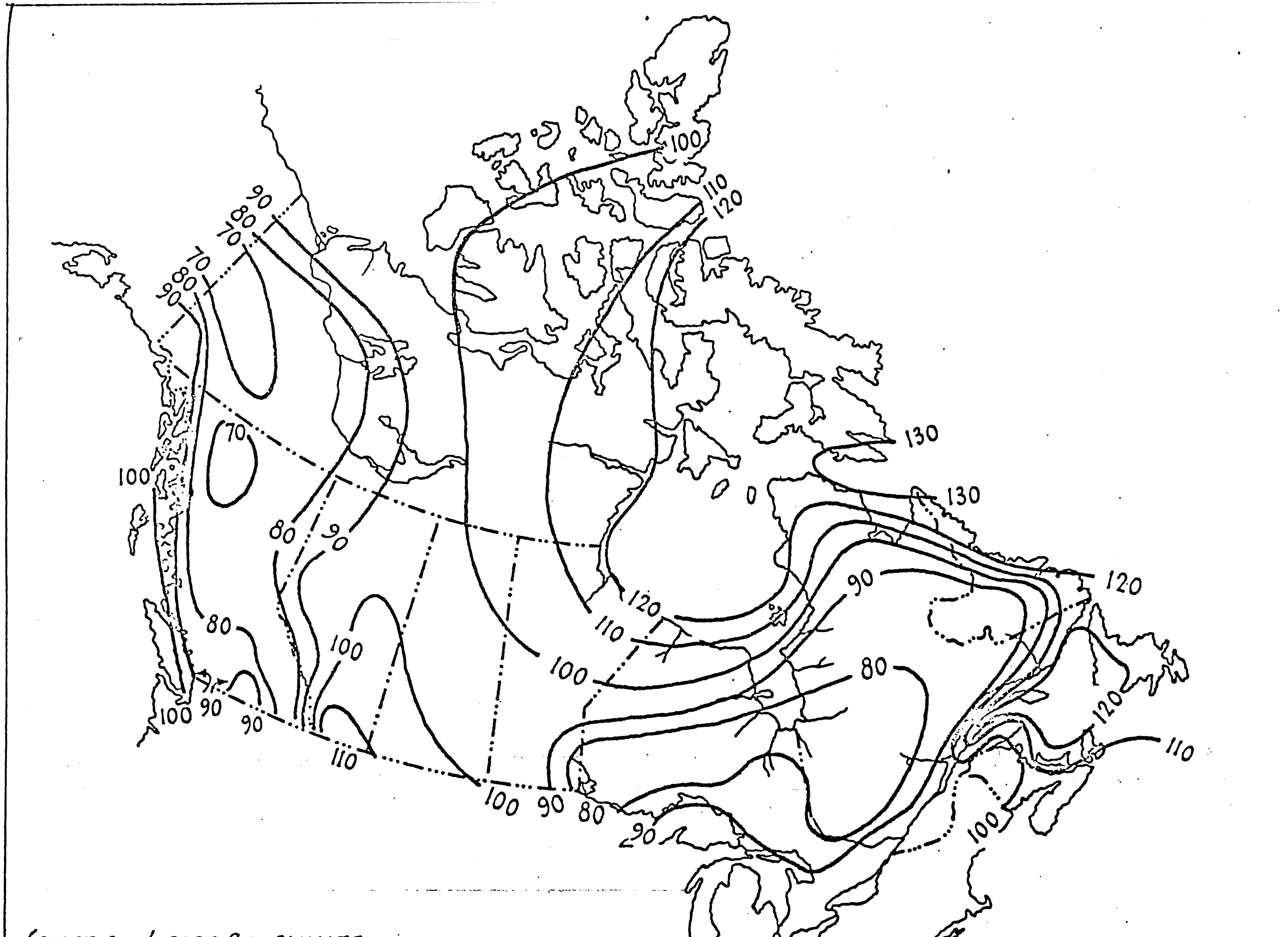


FIGURE 1

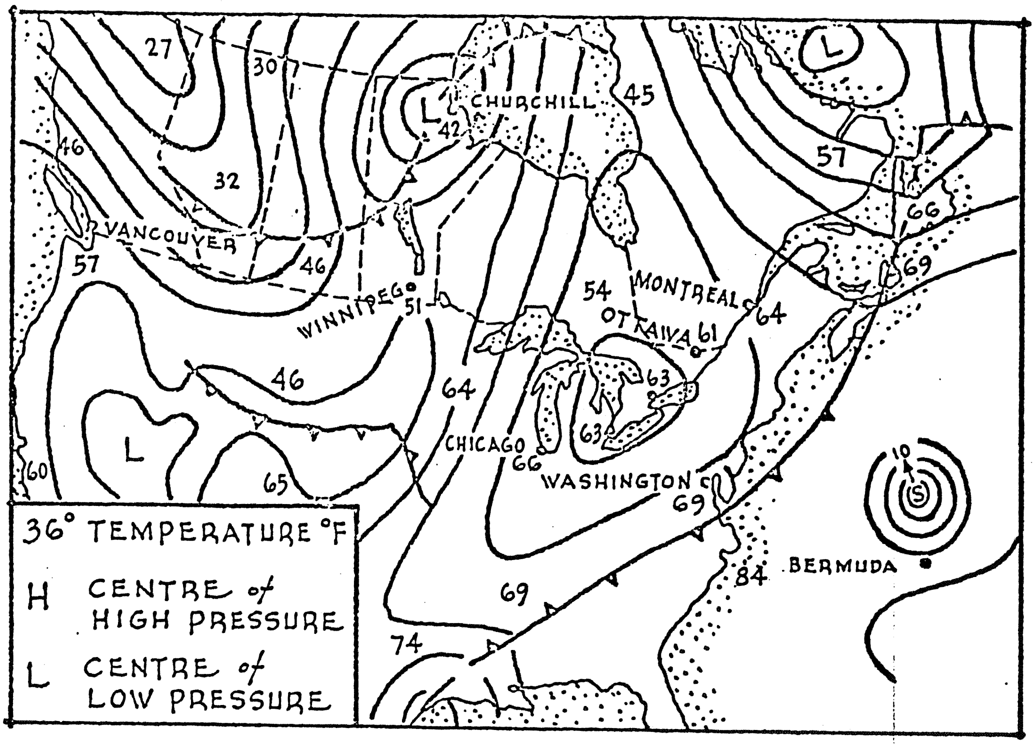
RELATIONSHIP BETWEEN GUST SPEEDS of OVER 60 M.P.H. AND MEAN HOURLY SPEEDS (FROM M.K. THOMAS)



(CHART 9 of PART 2: CLIMATE,
NATIONAL BUILDING CODE, CANADA, 1953)

FIGURE 2 COMPUTED MAXIMUM GUST SPEED (M.P.H.)

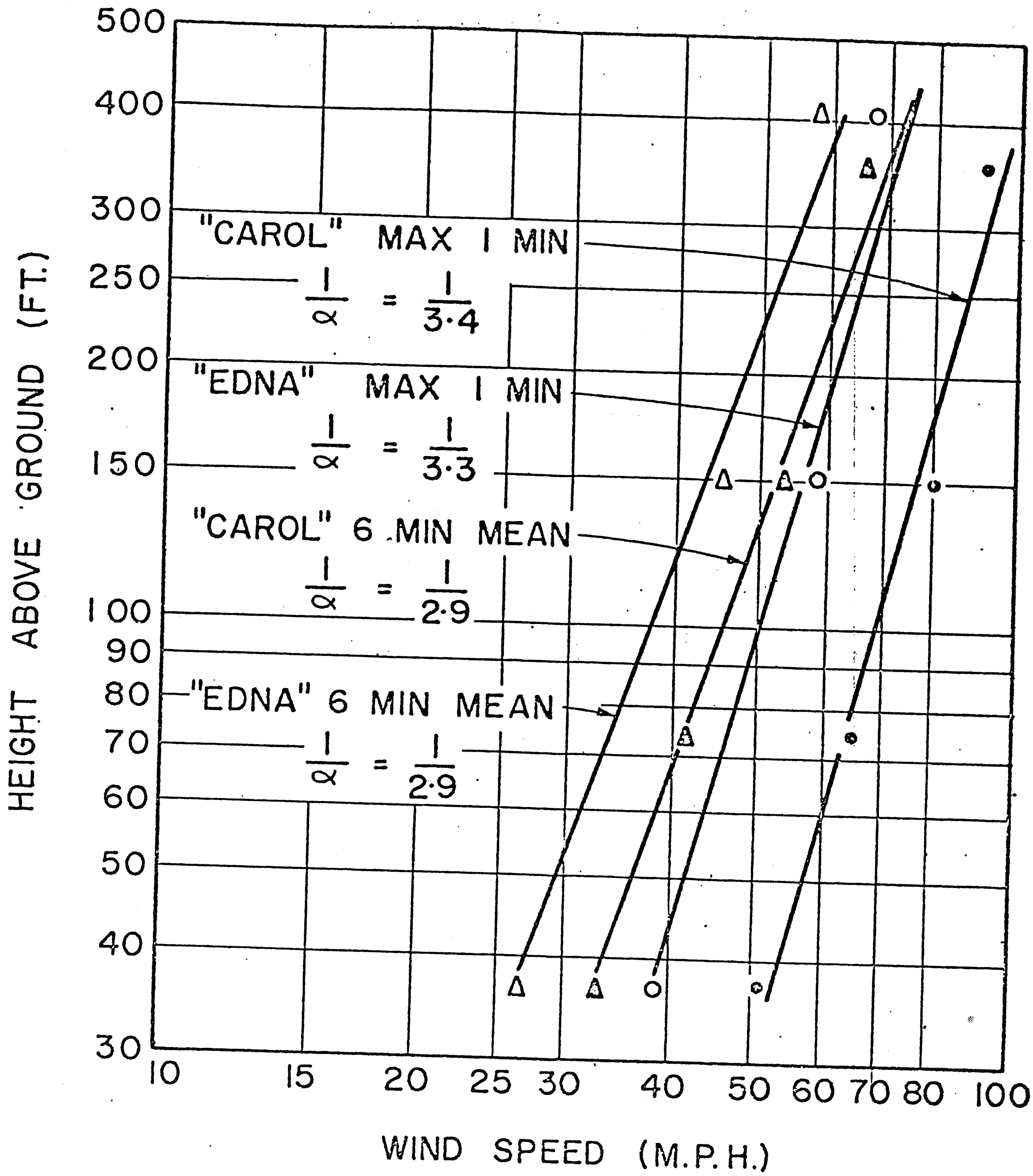
BR 2062-2



CONTINUOUS LINES (ISOBARS) JOIN PLACES HAVING THE SAME AIR PRESSURE

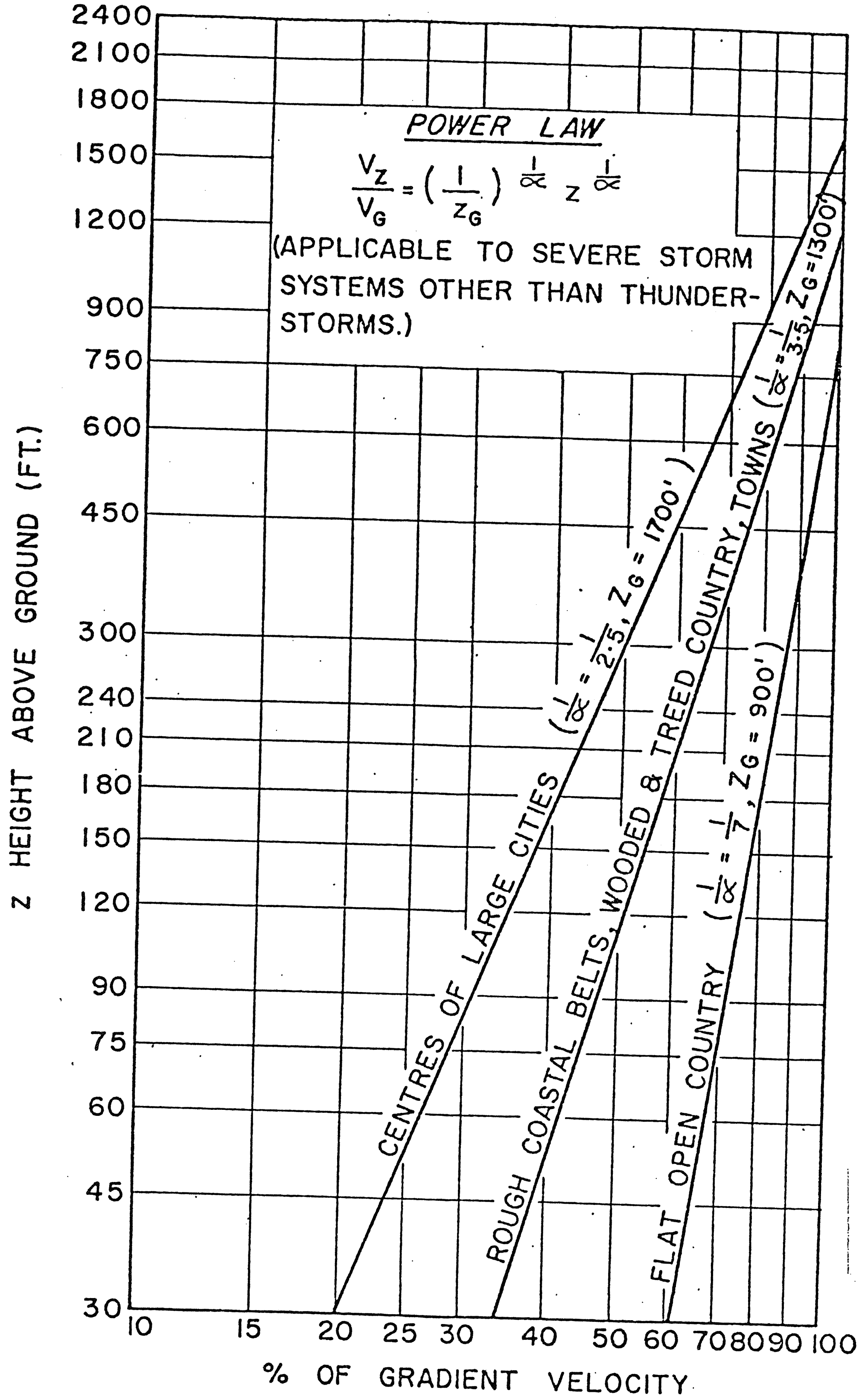
MAP SHOWING ISOBARS FOR SEPT 17, 1957
HURRICANE CARRIE IS SHOWN DRIFTING
NORTHWARDS OFF BERMUDA

FIGURE 3



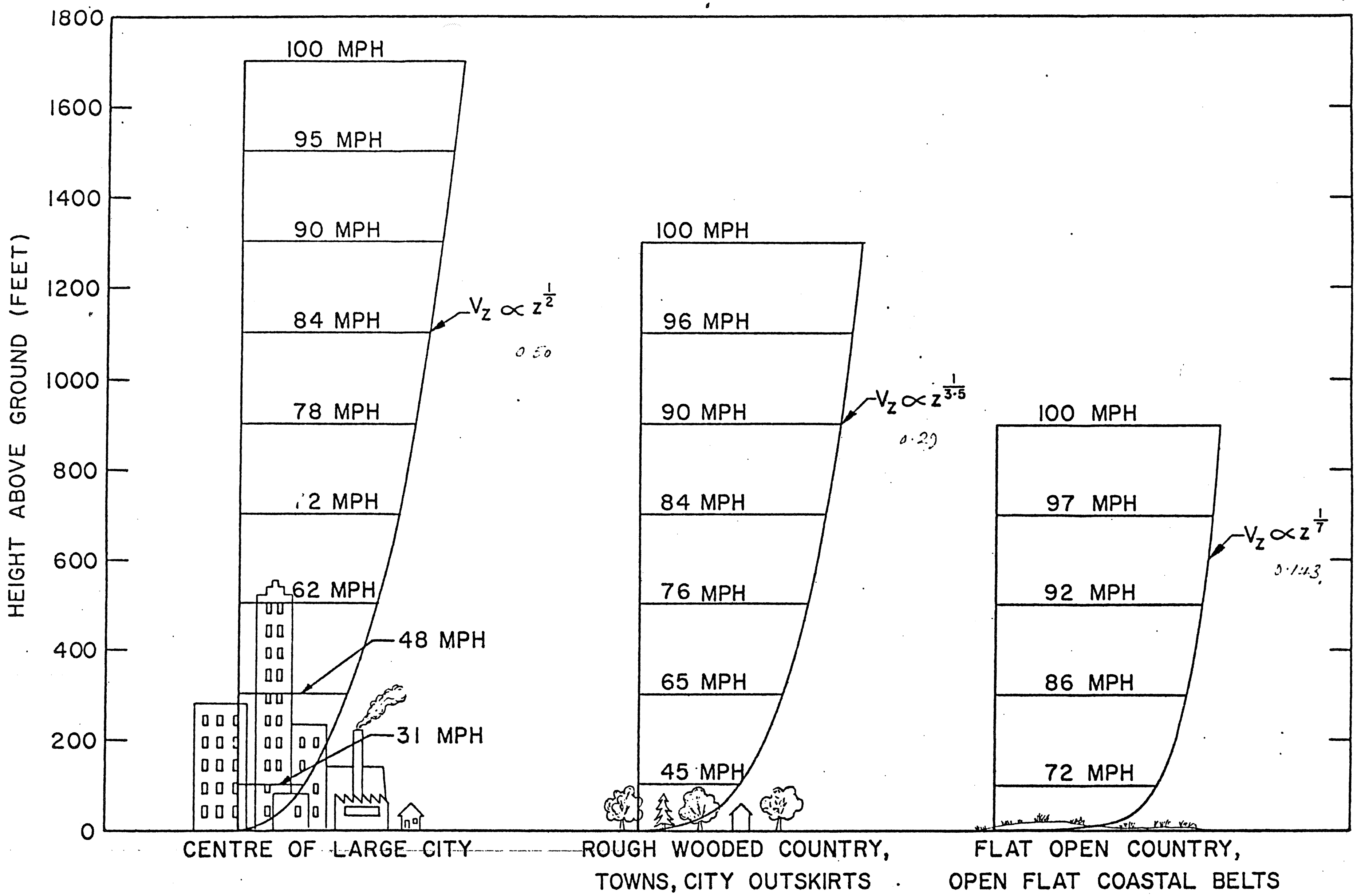
OBSERVATIONS OF HURRICANES "CAROL" & "EDNA" (1954) AT BROOKHAVEN LONG ISLAND.
(TERRAIN: FLAT COUNTRY WITH SCRUB TREES)

FIGURE 5



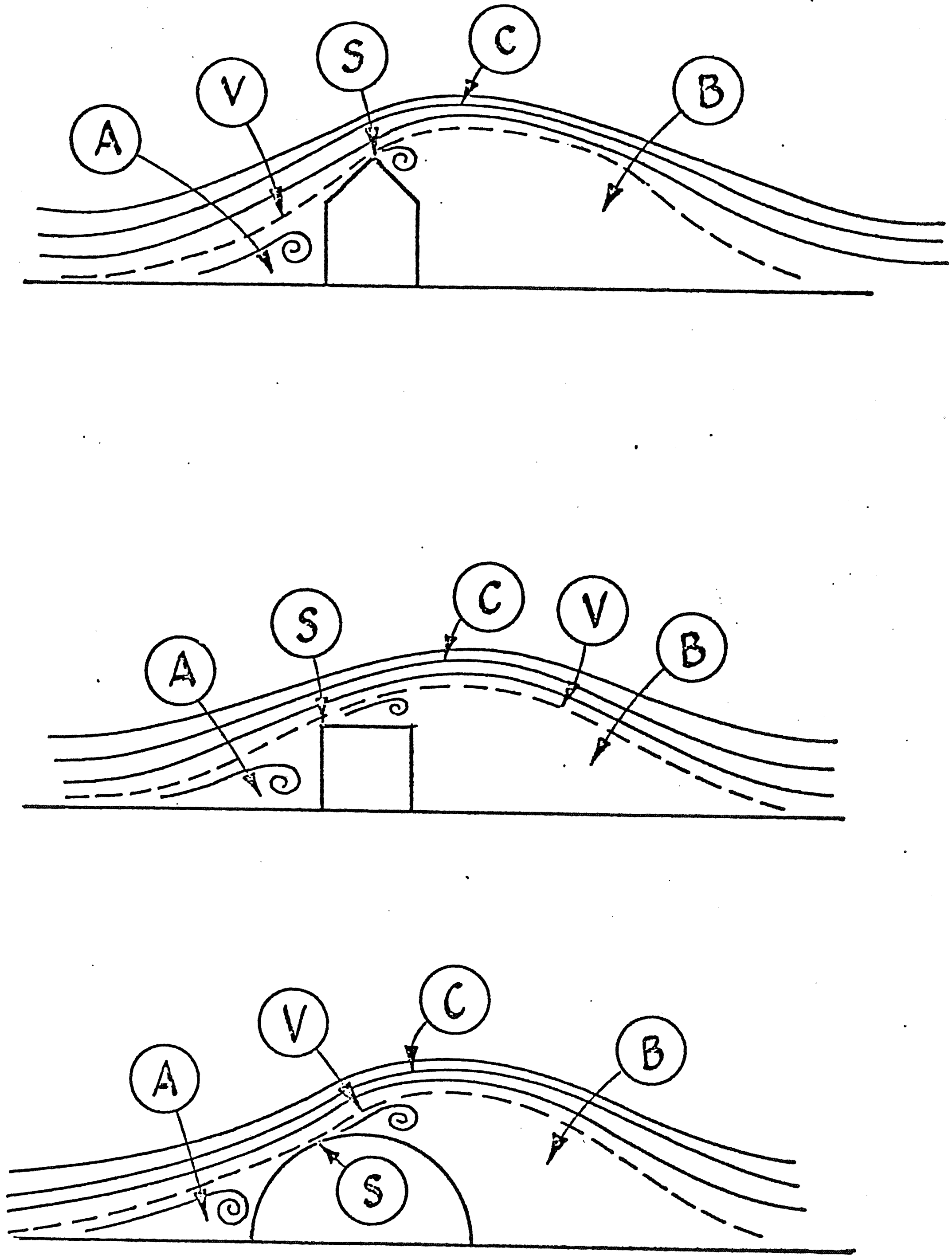
INCREASE OF VELOCITY WITH HEIGHT OVER LEVEL GROUND FOR THREE DIFFERENT TYPES OF SURFACE ROUGHNESS ACCORDING TO THE POWER LAW.

FIGURE 6



VELOCITY PROFILES OVER TERRAIN WITH THREE DIFFERENT ROUGHNESS CHARACTERISTICS FOR UNIFORM GRADIENT WIND VELOCITY OF 100 MPH.

FIGURE 7



(a)

(b)

(c)

Ⓐ WINDWARD VORTEX REGION
Ⓑ LEEWARD " "
Ⓒ NON-ROTATIONAL FLOW
Ⓥ VORTEX LAYER
Ⓢ SEPARATION POINT

FIGURE 8

FLOW AROUND THREE TYPICAL BUILDING SHAPES

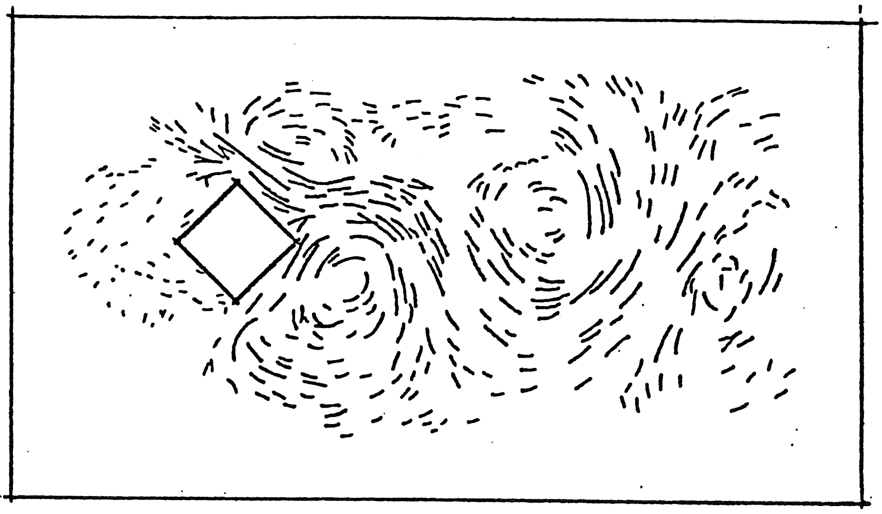


FIGURE 9 (a)

STREAM FLOW PATTERN BEHIND SQUARE CYLINDER (AFTER THE PHOTOGRAPH BY IRMINGER & NOKKENTRED)

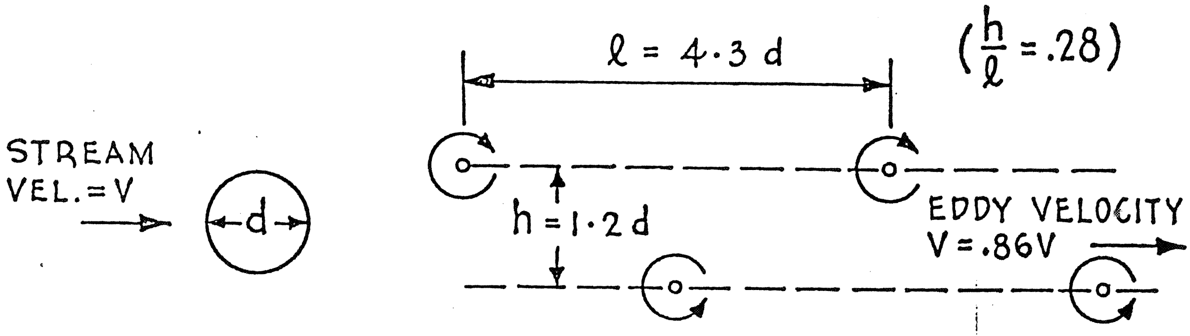


FIGURE 9 (b)

SCHEMATIC DIAGRAM OF VORTEX STREET BEHIND CIRCULAR CYLINDER

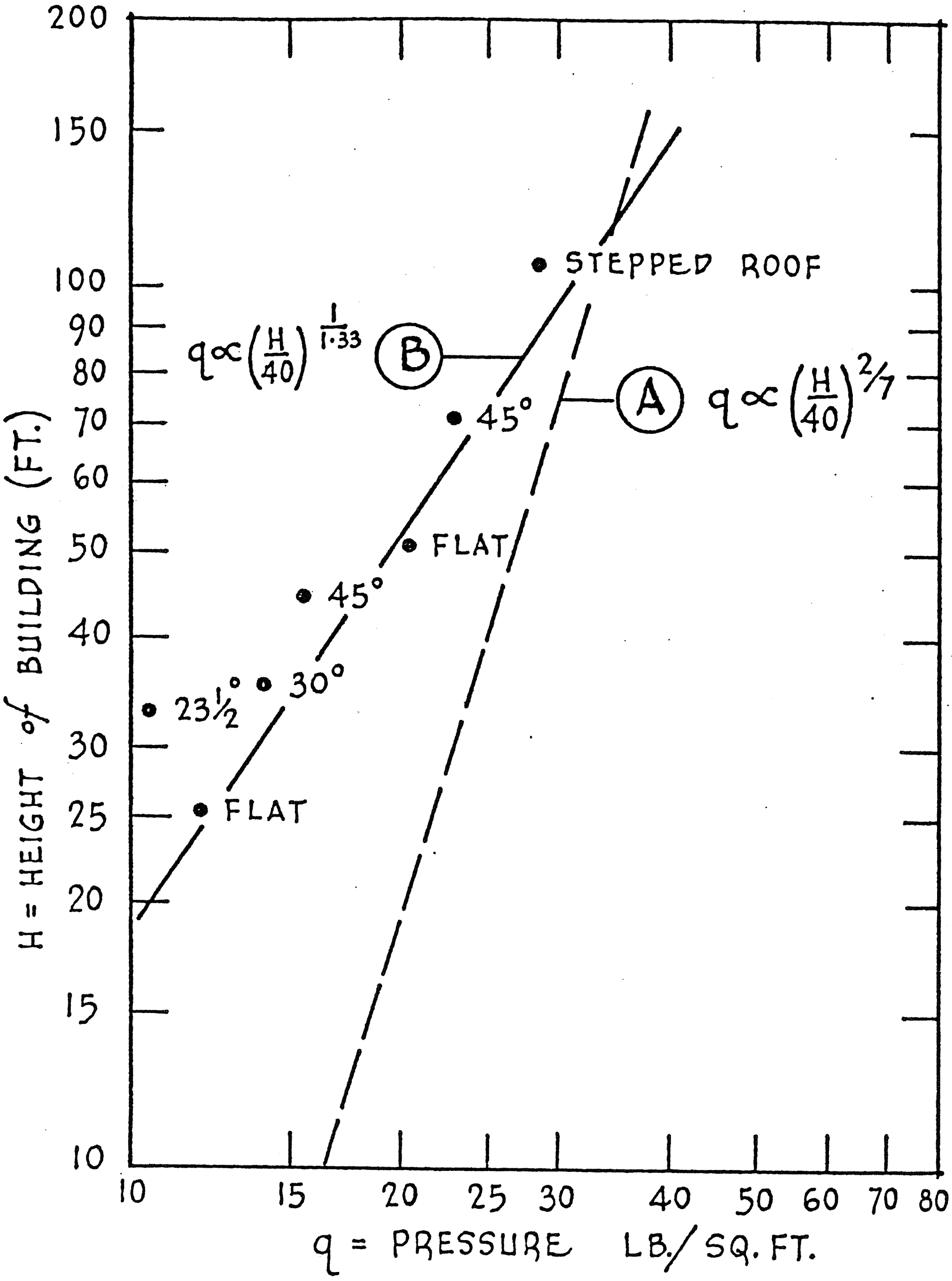


FIGURE 10

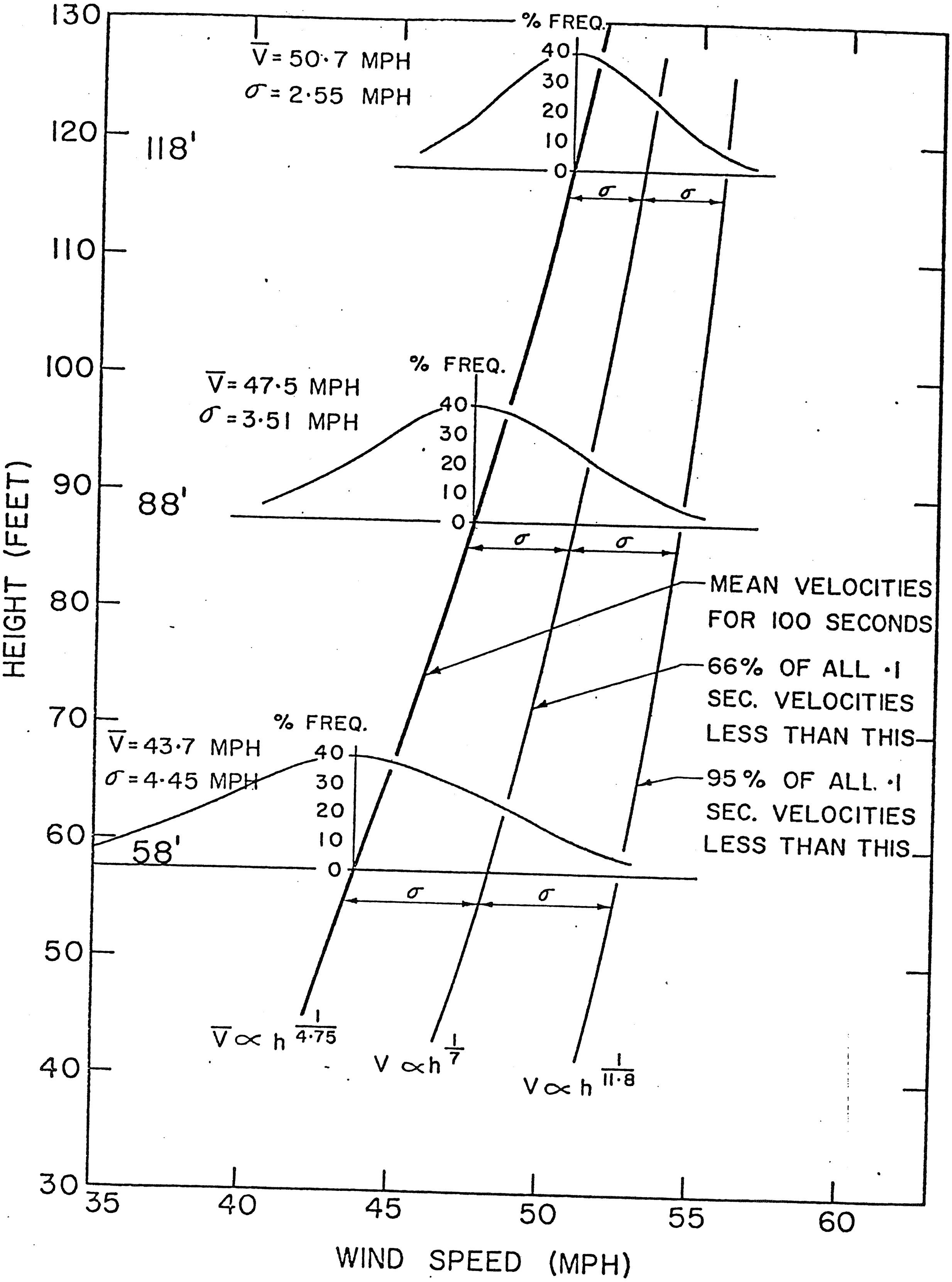
INCREASE of TOTAL PRESSURE ON BUILDINGS of DIFFERENT HEIGHT DUE TO INCREASE of VELOCITY WITH HEIGHT.

LINE (A) ACCORDING TO THE ASSUMPTIONS USED IN PRACTICE (LINE A)

LINE (B) ACCORDING TO THE EXPERIMENTAL RESULTS of BAILEY AND VINCENT (• & LINE B)

NOTE 1 *THE PRESSURES CORRESPOND TO AN ASSUMED VELOCITY of 80 M.P.H. AT A REFERENCE HEIGHT of 40 FT.*

2 *THE INCREASE of VELOCITY WITH HEIGHT CORRESPONDS CLOSELY TO THE $1/7^{TH}$ POWER LAW.*



WAX'S RESULTS SHOWING INCREASE OF MEAN VELOCITY AND DECREASE OF GUSTINESS WITH HEIGHT. - ORKNEY ISLANDS.

FIGURE 11

APPENDIX I

A1 RECORDS OF WIND VELOCITY PROFILES

A1.1 JUUL'S DATA

Location: Masned sund Denmark
Nature of surface: Flat level coast of "ocean of small islands" (off sea wind)
Reference: (38)

Height metres	Height ft	Wind Velocity metres/sec	Wind Velocity mph
5	16.5	14.2	32
15	49.5	16.5	37
35	115.5	17.2	39
55	182.0	18.8	42

A1.2 GIBLETT'S DATA

Location: Cardington, England
Terrain: Large flat open airfield
Reference: (25)

The ratio of wind velocities recorded by an anemometer at 150 ft to those at 50 ft under neutrally stable conditions and wind velocities in excess of 20 mph was found to be 1.18 which corresponds to a power law increase with an exponent of 1/7.8.

A1.3 SCRASE'S DATA

Location: Salisbury Plain
Nature of Terrain: Open grass covered plain
Reference: (67, 82 p. 238)

Reported profile in adiabatic conditions given by power law with exponent of 0.13 (1/7.7). Wind measurements were made between heights of 3 and 13 metres (approximately 10 and 43 ft).

A1.4 WING'S DATA

Location: Ballybunion, Ireland
Nature of Terrain: Low level plain. Atlantic Ocean 1/2 mile distant beyond low level sand dunes
Reference: (100)

Height (ft)	Wind Velocity (mph)
15	24
300	32
492	35

(These data refer to curve (1) of Fig. 8 of this paper "which is a curve which most of the high winds there very nearly approach" (p. 9).)

A1.5 SHERLOCK'S DATA

Location: Ann Arbor, Michigan
Terrain: Open farmland
Reference: (76)

Fastest 1 minute mean velocities

Height (ft)	Wind Velocity (mph)
50	44.5 *
75	45 *
100	45.5 *
125	47
150	47.5
175	48.5
200	48.5
225	54

* these values may be slightly higher than would otherwise be obtained owing to the fact that measurements were made near the brow of a slight rise.

A1.6 TAYLOR'S DATA

These data were taken from Pagon's paper (48) in which Taylor's data are summarized in terms of the following Ekman spiral parameters.

		Cities	Open Country
θ		45°	20°
Z_g (ft)	summer	2020	1250
	winter	1420	885

where θ = angle between directions of surface and gradient winds
Z_g = height at which gradient velocity is first attained.

Taylor's experiments (90) were conducted at Salisbury Plain and the Eiffel Tower.

The Ekman spirals specified by the above parameters correspond closely with power laws with exponents of 1/2 and 1/7 (for "cities" and "open country" respectively).

A1.7 FROST'S DATA

Location: Cardington, England
Nature of surface: Flat open airfield

This was obtained from Sutton (82, p. 238) who states:

"Layers 5 to 400 ft and 4 to 1000 ft have been investigated by R. Frost using instruments suspended from a captive balloon over the airfield at Cardington, Southern England. His results indicate that in these layers the profile can be represented by power laws in all conditions of temperature gradient .... From a further examination of extended profiles from 4 to 1000 ft Frost concludes that the value of (the power law exponent) in conditions of adiabatic lapse rate is .149 (1/6.7) which is very close to the value 1/7 (.142) occurring in the generally accepted power-law profile for the turbulent boundary layer of a flat plate in a wind tunnel."

A1.8 KAMEI'S DATA

These were obtained from the study of wind velocity profiles in three Japanese towns and also along the coast and represent summarized results (reference 39).

The power law exponents suggested by these studies were 1/3 in towns and 1/5 at the coast.

A1.9 WAX'S DATA (Fig. 11

Location: Costa Hill, Orkney Islands
Surface: Rough coast, flat topped hill 1/3 mile inland from high cliff overlooking sea
Reference: (99)

Height (ft)	Wind Velocity mph (100 sec mean)
58	43.7
88	46.8 *
118	50.7

* average of two simultaneous readings of 46.1 and 47.5 mph.

A1.10 SMITH'S DATA

Location: Brookhaven, Long Island
Surface: Level country uniformly with scrub oak and pine
Reference: (79)

Reported average exponent for 13 winter storms was 0.25.

For data from hurricanes Edna and Carol (1954) see Fig. 5.

A1.11 DINES' DATA

Location: Royal Aircraft Establishment, Farnborough, England
Surface: Treed and wooded farmland and fields
Reference: (17)

Height		Wind Velocity *	
metres	ft	metres/sec	mph
50	165	7.2	16.2
100	330	8.9	20.0
150	495	10.4	23.4
200	660	11.4	25.6
250	825	12.2	27.4
300	990	12.9	29.0
400	1320	14.5	32.6
500	1650	15.6	35.1

* Measured by balloons and theodolites: average of 25 experiments in which velocities were 10 metres/sec.

A1.12 RATHBUN'S DATA

Location: Centre of New York City
Surface: Heavily built-up city
Reference: (54)*

	Height (ft)	Wind Velocity (mph)
N. Y. City Met. Obs	62	28
U.S. Weather Bureau	454	58
Empire State Building	1263	90

* Storm of 22/3/36 (the most severe recorded)

APPENDIX II

Amplification factors for various ideal hills and ridges giving ratio of wind velocity at peak to oncoming flow over level ground.

h = height of hill

a = base width (in direction of wind)

Infinitely Long Elliptical Cylinders (48)

h/a	Amplification factor
1:4	1.25
1:3	1.33
1:2	1.50
1:1	2.00
2:1	3.00
3:1	4.00
4:1	5.00

Spheroids (48)

h/a	
1:4	1.08
1:3	1.12
1:2	1.21
1:1	1.50
2:1	2.12
3:1	2.74
4:1	3.38

Amplification Factor for Ridges (53)

h/a	
4:5	1.15
7:5	1.23
12:0	1.37
17:5	1.6

APPENDIX III

EXTRACT FROM THE STANDARDS OF THE SWISS ASSOCIATION OF ENGINEERS AND ARCHITECTS (S.I.A. NORMEN NO. 160, 1956) ON LOAD ASSUMPTIONS, ACCEPTANCE AND SUPERVISION OF BUILDINGS.

Translator's Note:

The following is a translation of the section on wind loads of the above Swiss code for buildings and bridges. The section consists of two parts, a text (p. 13 to 16) and a number of tables (p. 31 to 40) containing shape factors for a variety of different structures. The comprehensiveness of these tables and particularly the careful consideration of high local suctions would seem to make this specification worth careful consideration in any possible future revision of the section on wind loads of the National Building Code. (W.R. Schriever)

Article 20. Wind Loads

1. The wind loads on structures can be obtained either from information on the total wind effect or from the distribution of normal pressures on the various surfaces and subsequent integration. Plates I to VIII contain the appropriate shape factors and explanations. In general the frictional forces acting tangentially are small and can be neglected. Where this is not the case the tables give the appropriate details.

2. The basic quantity for all forces is the velocity pressure q

$$q = \frac{\gamma}{2g} v^2 \quad (kg/m^2) \qquad (1)$$

γ = specific gravity of air (kg/m^3)

$g = 9.81\ m/sec^2$ (gravitational acceleration)

v = wind velocity (m/sec)

Because in general the wind velocity increases with the height H above ground a gradation is made in the velocity pressures. The design calculations shall be based on the following pressures:

Height above ground (H) m.	ft.	Velocity pressure (q) kg/m²	lb/ft²
0 - 5	(0 - 16)	70	(14.6)
5 - 20	(16 - 66)	85	(17.4)
20 - 40	(66 - 133)	100	(20.4)
40 - 80	(133 - 266)	120	(24.5)
80 - 160	(266 - 532)	150	(30.6)
160 - 320	(532 - 1060)	180	(37.0)

For relatively low structures (less than 20 m or 66 ft in height) the whole structure shall be designed with the same velocity pressure, using the pressure valid for highest part of the structure. For high structures (e.g. towers) the decrease in pressure downwards may be taken into account according to the above table.

For bridges, aerial cableways, power lines and the like, a velocity pressure shall be used which is at least equal to the velocity pressure of the maximum height of the structure above ground or above water level.

3. A differentiation according to elevation above sea level is not required. But for exposed locations, such as mountain tops and areas of particularly high winds ("Foehn" or "chinook") the advice of meteorological authorities shall be obtained.

4. The pressures p (kg/m^2) and the forces K(kg) are calculated from the velocity pressure q, the pressure coefficients (or shape factors) c and the coefficients k, according to the following formulae:

$$p = c_p\, q \qquad (2a)$$

$K_n = c_n\, k\, q\, F$ normal to the surface considered (2b)

$K_t = c_t\, k\, q\, F$ tangential to the surface considered (2c)

The coefficients k take into account slenderness and shielding effects.

The pressures p can be positive or negative and are to be considered as differential pressures with regard to the normal atmospheric pressure (i.e. not as absolute pressures as used in physics). In the calculation of the total force according to (2b, 2c) the appropriate surface F shall be used as shown in the tables. The values of the coefficients c_p, c_n, c_t, k are shown in the Tables and are functions of the shape of the structure and the direction of the wind.

5. In calculating the forces from the pressure distributions both exterior and interior pressures must be considered, because a consideration of the exterior pressure alone would be based on the erroneous assumption that the interior pressure is always exactly equal to the exterior pressure at zero velocity.

In the Tables, therefore, in addition to the coefficients for the exterior pressure c_{pa}, the coefficients for the internal pressure c_{pi} have been listed for the more important cases, depending on the location and size of openings in the building (windows, doors, ventilation openings).

The consideration of the internal pressure in large open and semi-open buildings and halls (hangars, exhibition buildings, rooms with large show windows, etc.) is particularly important. For such cases the worst condition shall be determined in each case as a basis for calculation.

The pressure coefficients are usually given for various wind directions in a horizontal plane. If the structure is free standing and winds from all directions are to be expected all cases shall be considered. In special cases the direction of strongest winds is given by topographical and meteorological conditions. In such cases it is recommended to obtain the advice of meteorological authorities. Then, after the wind directions have been determined, a wind pressure diagram can be drawn by multiplying the velocity pressures q with the pressure coefficients c_p in order to obtain the pressures p (kg/m^2).

6. In the design of a structure the overall effect of the pressures on the stability of the whole building or its larger parts (roof, walls) shall be considered first. Experience has shown, however, that even with adequate stability local damages near the edge of roofs, lights, parapets, etc. may occur. The Tables show, therefore, the location of maximum forces for the more important cases (flat roofs and canopies) and the pressure coefficient maxima c_{pa} are indicated numerically. The corresponding shaded areas are shown to scale in the figures of the Tables.

When calculating the total wind forces the shaded areas need not be considered as additional forces. They serve merely for the determination of local suction forces. In such locations the construction must be adequate to resist the local forces (additional nailing or anchoring, careful gluing, the local weighting, etc.). Furthermore it should be noted that these local forces can act in a shaking manner and result in fatigue failures.

7. For some rounded structures (Table VI) it has been found that there is a considerable difference in the pressures depending on the Reynolds number. For practical purposes the Reynolds number can be taken as $d\sqrt{q}$, where d is in metres and q in kg/m^2.

The roughness of rounded structures may be of considerable importance. Common well laid brickwork without parging can be considered as having a "rough" surface (Table VI/26). Surface with ribs projecting more than 2 per cent of the diameter are considered as "very rough". In case of doubt it is recommended to use those c_p values which result in the greater forces. For cylindrical and spherical objects with substantial stiffening ribs, supports and attached structural members the pressure coefficients depend on the type, location and relative magnitude of these roughnesses.

8. In particularly long structures (buildings several hundred metres long, cables of aerial cableways and power lines, bridges, etc.) it is unlikely that the maximum forces will occur everywhere simultaneously. There are, however, to-day still insufficient data for a quantitative consideration of this. In such cases it is advisable to obtain the advice of meteorological authorities. It is also recommended that wind tunnel tests should be carried out on small models for shapes of structures which differ significantly from those shown in the Tables.

9. Structures subject to vibration due to the wind shall be investigated by theoretical and possibly experimental methods for the danger of dynamic over-loading and vibration at critical frequencies. This is the case particularly for: church steeples and sightseeing towers, high chimneys, high buildings, antenna towers, power lines, aerial cableways, aerial conveyors, cranes, etc. As a rough guide it may be said that caution should be used if the period for a full cycle is more than 1 second. The sum of static wind load plus dynamic forces may in certain cases be as high as double the static wind load.

For suspension bridges special investigations of the danger of aerodynamic instability is recommended.

10. With regard to combinations of wind and snow load Clause 19/8 is applicable.

In locations where the strongest winds and icing may occur simultaneously structures with trussed parts, rods, cables and ropes must be designed with an increased wind load due to icing according to the "Explanations" below.

11. The shape of a structure may change during erection. The wind loads may therefore be temporarily higher during erection than after completion of the structure. These increased wind loads shall be taken into account using the appropriate coefficients from the Tables.

12. All the coefficients of the Tables are given without any added safety factors and therefore relate to the actual loadings to be expected.

Explanations for Tables I to VIII

For closed sharp-edged buildings the pressure coefficients of Tables I to V and the appropriate clauses of Article 20 are to be used. For closed buildings in which the wind load does not exceed 25 per cent of the total load the following approximation can be used:

Valid range: For buildings with rectangular plan $\left(\frac{L}{B} = \frac{5}{2} \text{ to } \frac{2}{5}\right)$, Wind direction 0°, hip or single-sloped roofs $\left(\frac{B}{H} = \frac{3}{2} \text{ to } \frac{2}{3}\right)$, the coefficients for exterior pressure c_{pa} can be taken as:

Front wall A: $c_{pa} = +0.9$ Rear wall B: $c_{pa} = -0.5$

Side walls C & D: $c_{pa} = -0.7$ Rear roof slope F: $c_{pa} = -0.7$

Front roof slope E (slope α with horizontal):

$$0^\circ \leqslant \alpha \leqslant 20^\circ \qquad c_{pa} = -.10$$

$$20^\circ \leqslant \alpha \leqslant 50^\circ \qquad c_{pa} = \frac{1}{100}(5\alpha - 200)$$

$$50^\circ \leqslant \alpha \leqslant 90^\circ \qquad c_{pa} = +\frac{\alpha}{100}$$

The local pressure coefficient maxima c_{pa}^{*} listed in Article 20/6 must not be neglected in the above approximation.

The interior pressure coefficients c_{pi} shall be taken from Tables I to III, depending on the distribution of openings.

For closed objects with rounded surfaces the pressure coefficients of Table VI shall be used. Since the coefficients depend partly on roughness and other conditions the appropriate remarks of Article 20/7 shall be considered.

For trussed construction and plate like objects Tables VI, VII and VIII, the following explanations and Articles 20/9 and 20/10 are applicable.

Regarding Tables VI, VII and VIII:

If icing is to be taken into account according to Article 20/10 the increase in wind load is as follows: unless otherwise specified the areas of all structural members shall be calculated assuming a uniform ice covering of 12 mm. (1/2 inch thickness).

For icing in this form and thickness the same pressure coefficients may be used for sharp-edged sections for the condition with icing as for the condition without icing. For round profiles, however, the values for a "rough" surface shall be used for the iced condition according to Table VI/28.

Regarding Table VII/29:

This table contains coefficients which are valid for infinite length of the member considered. For finite lengths and projecting ends the values are smaller depending on the slenderness ratio k_{1p} given in Table VII/30. If one end of a member is located in a large plate or wall the slenderness ratio shall be calculated for twice the actual length. If a member terminates with both ends in large plates or walls the slenderness ratios for infinite length shall be used. This applies correspondingly to the coefficients for trussed construction, Table VII/31 and the slenderness factors k_{1F} according to Table VII/32.

Regarding Table VII/33:

For members which are located behind each other in the direction of the wind the shielding effect may be taken into account. The windward member and those parts of the leeward member which are not shielded shall be designed with a reduced pressure q_x according to Table VII/33. For trusses and plate girders h shall be taken as the girder height h_T, for individual sections as h_a.

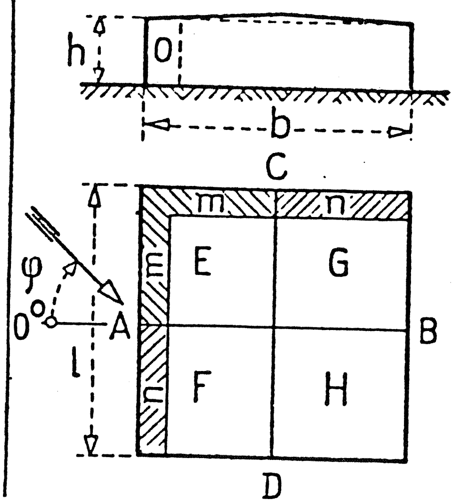
For constructions made from circular sections with $d\sqrt{q} < 1.5$ and $F_s/f \leqslant 0.3$ the shielding factors can be taken by approximation from Table VII/33. If $d\sqrt{q} > 1.5$ the shielding effect is small and the fullness degree $F_s/f \leqslant 0.3$ can be taken into account by a constant shielding factor of $k = 0.95$.

For unloaded truss and plate girder bridges the horizontal and vertical forces can be calculated from the coefficients of Table VII/31, 32, 33 and Table VIII/34 (Case 1). For bridges loaded by a line of railway cars or trucks the forces shall be calculated separately according to Table VIII/34 (Case 2) and added to the values calculated for Case 1 (without the line of cars).

For important large structures it is recommended to determine the forces more accurately with and without a line of cars by measurements in a wind tunnel.

I

1 GABLE ROOFS 0÷3°



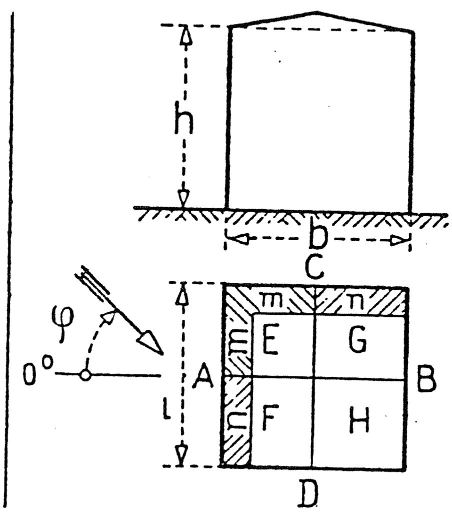
EXT. PRESS. COEFF. c_{pa} FOR h : b : l = 1 : 4 : 4.

φ	A	B	C	D	E	F	G	H
0°	+0,9	−0,3	−0,4	−0,4	−0,8	−0,8	−0,3	−0,3
15°	+0,8	−0,3	−0,1	−0,5	−0,7	−0,8	−0,2	−0,3
45°	+0,5	−0,4	+0,5	−0,4	−0,9	−0,6	−0,6	−0,3
15°	FOR SECTION „0" (SIDE C) c_{pa}^{*} = −0,8							
45°	FOR SECTION „m" c_{pa}^{*} = −2,0 „n" c_{pa}^{*} = −1,0							

CLOSED LOW SQUARE BLDG

INT. PRESS. COEFF. c_{pi} FOR φ =	0°	15°	45°
OPENINGS UNIFORMLY DISTR.	±0,2	±0,2	±0,2
" " ON SIDE A PREDOMINATING	+0,8	+0,7	+0,4
" " " B " "	−0,2	−0,3	−0,4
" " " C " "	−0,3	−0,2	+0,4

2 GABLE ROOFS 0÷10°



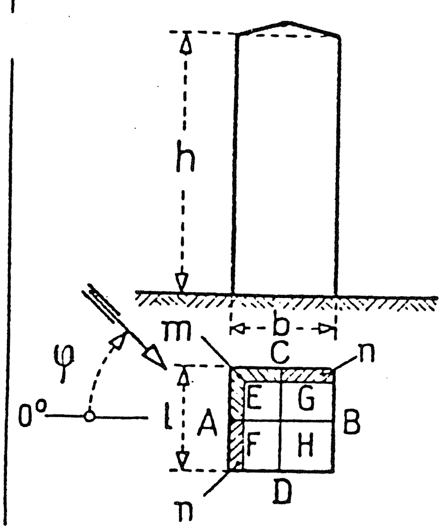
INT. PRESS. COEFF. c_{pa} FOR h : b : l = 1 : 1 : 1

φ	A	B	C	D	E	F	G	H
0°	+0,9	−0,5	−0,6	−0,6	−0,7	−0,7	−0,5	−0,5
15°	+0,8	−0,5	−0,7	−0,5	−0,7	−0,6	−0,5	−0,6
45°	+0,5	−0,5	+0,5	−0,5	−0,8	−0,5	−0,5	−0,4
45°	FOR SECTION „m" c_{pa}^{*} = −1,2 „n" c_{pa}^{*} = −0,8							

CLOSED SQUARE BLDG.

INT. PRESS. COEFF. c_{pi} FOR φ =	0°	15°	45°
OPENINGS UNIFORMLY DISTR.	±0,2	±0,2	±0,2
" " ON SIDE A PREDOMINATING	+0,8	+0,7	+0,4
" " " B " "	−0,4	−0,4	−0,4
" " " C " "	−0,5	−0,6	+0,4

3 GABLE ROOFS 0÷15°



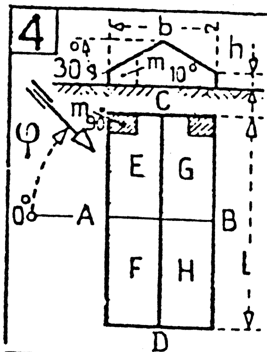
EXT. PRESS. COEFF. c_{pa} FOR h : b : l = 2,5 : 1 : 1

φ	A	B	C	D	E	F	G	H
0°	+0,9	−0,6	−0,7	−0,7	−0,8	−0,8	−0,8	−0,8
15°	+0,8	−0,5	−0,9	−0,6	−0,8	−0,8	−0,7	−0,7
45°	+0,5	−0,5	+0,5	−0,5	−0,8	−0,7	−0,7	−0,5
45°	FOR SECTION „m" c_{pa}^{*} = −1,0 „n" c_{pa}^{*} = −0,8							

CLOSED HIGH SQUARE BLDG

INT. PRESS. COEFF. c_{pi} FOR φ =	0°	15°	45°
OPENINGS UNIFORMLY DISTRIB.	±0,2	±0,2	±0,2
" " ON SIDE A PREDOMINATING	+0,8	+0,7	+0,4
" " " B " "	−0,5	−0,5	−0,4
" " " C " "	−0,6	−0,8	+0,4

4



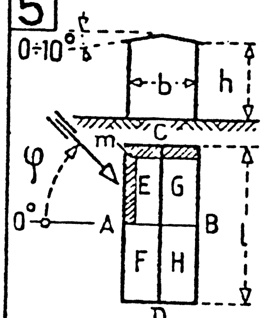
EXT. PRESSURE COEFF. c_{p_a} h:b:l = 1:8:16 INT. PRESS. COEFF. c_{p_i}

φ	A	B	C	D	E	F	G	H
0°	+0,8	-0,5	-0,5	-0,5	+0,2	+0,2	-0,6	-0,6
45°	+0,5	-0,5	+0,4	-0,3	+0,1	-0,1	-0,8	-0,5
90°	-0,3	-0,3	+0,9	-0,3	-0,5	-0,1	-0,5	-0,1
10° 90°	FOR SECTION „m" $c_{p_a}^* = -1,0$							

WIND DIRECTION φ =	0°	45°	90°
OPEN. UNIF. DISTRIB.	±0,2	±0,2	±0,2
SIDE A PREDOM.	+0,7	+0,4	-0,2
" B " "	-0,4	-0,4	-0,2
" C " "	-0,4	+0,3	+0,8

CLOSED SHED

5



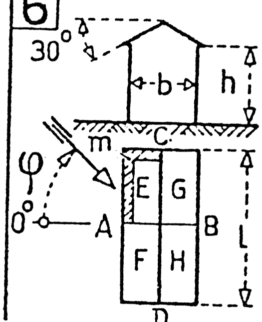
EXT. PRESS. COEFF. c_{p_a} h:b:l = 2,5:2:5 INT. PRESS. COEFF. c_{p_i}

φ	A	B	C	D	E	F	G	H
0°	+0,9	-0,5	-0,7	-0,7	-0,6	-0,6	-0,5	-0,5
45°	+0,6	-0,5	+0,4	-0,5	-0,9	-0,7	-0,6	-0,7
90°	-0,5	-0,5	+0,9	-0,4	-0,8	-0,2	-0,8	-0,2
45°	FOR SECTION „m" $c_{p_a}^* = -1,5$							

WIND DIRECTION φ =	0°	45°	90°
OPEN. UNIFORM. DISTR.	±0,2	±0,2	±0,2
SIDE A PREDOM.	+0,8	+0,5	-0,4
" B " "	-0,4	-0,4	-0,4
" C " "	-0,6	+0,3	+0,8

CLOSED HOUSE, FLAT ROOF

6



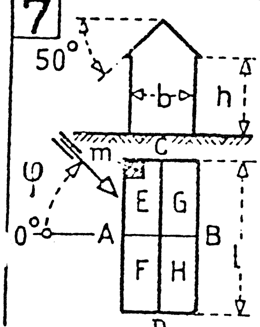
EXT. PRESS. COEFF. c_{p_a} h:b:l = 2,5:2:5 INT. PRESS. COEFF. c_{p_i}

φ	A	B	C	D	E	F	G	H
0°	+0,9	-0,5	-0,7	-0,7	-0,6	-0,6	-0,5	-0,5
45°	+0,6	-0,5	+0,4	-0,4	-0,4	-0,5	-0,6	-0,7
90°	-0,5	-0,5	+0,9	-0,4	-0,7	-0,2	-0,7	-0,2
45°	FOR SECTION „m" $c_{p_a}^* = -1,2$							

WIND DIRECTION φ =	0°	45°	90°
OPEN. UNIFORM. DISTR.	±0,2	±0,2	±0,2
SIDE A PREDOM.	+0,8	+0,5	-0,4
" B " "	-0,4	-0,4	-0,4
" C " "	-0,6	+0,3	+0,8

CLOSED HOUSE, MEDIUM ROOF

7



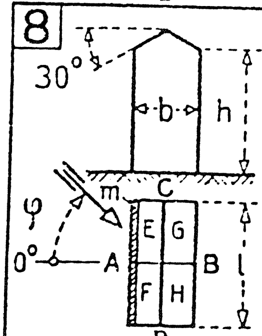
EXT. PRESS. COEFF. c_{p_a} h:b:l = 2,5:2:5 INT. PRESS. COEFF. c_{p_i}

φ	A	B	C	D	E	F	G	H
0°	+0,9	-0,5	-0,8	-0,8	+0,3	+0,3	-0,6	-0,6
45°	+0,6	-0,5	+0,4	-0,4	+0,3	-0,1	-0,5	-0,6
90°	-0,5	-0,5	+0,9	-0,4	-0,8	-0,2	-0,8	-0,2
75°	FOR SECTION „m" $c_{p_a}^* = -1,2$							

WIND DIRECTION φ =	0°	45°	90°
OPEN. UNIFORM. DISTR.	±0,2	±0,2	±0,2
SIDE A PREDOM.	+0,8	+0,5	-0,4
" B " "	-0,4	-0,4	-0,4
" C " "	-0,7	+0,3	+0,8

CLOSED HOUSE, STEEP ROOF

8



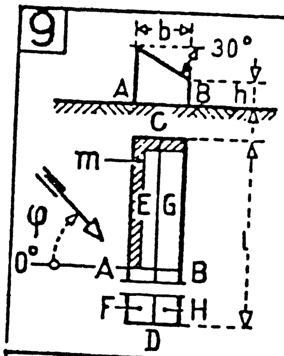
EXT. PRESS. COEFF. c_{p_a} h:b:l = 2:1:2 INT. PRESS. COEFF. c_{p_i}

φ	A	B	C	D	E	F	G	H
0°	+0,9	-0,5	-0,8	-0,8	-1,0	-1,0	-0,5	-0,5
45°	+0,6	-0,5	+0,4	-0,4	-0,3	-0,4	-0,5	-0,6
90°	-0,6	-0,6	+0,9	-0,4	-0,7	-0,5	-0,7	-0,5
0°	FOR SECTION „m" $c_{p_a}^* = -1,2$							

WIND DIRECTION φ =	0°	45°	90°
OPEN. UNIFOR. DISTR.	±0,2	±0,2	±0,2
SIDE A PREDOM.	+0,8	+0,5	-0,5
" B " "	-0,4	-0,4	-0,5
" C " "	-0,7	+0,3	+0,8

CLOSED HIGH HOUSE

9



EXT. PRESS. COEFF. c_{pa} FOR h:b:l = 1:24:12.

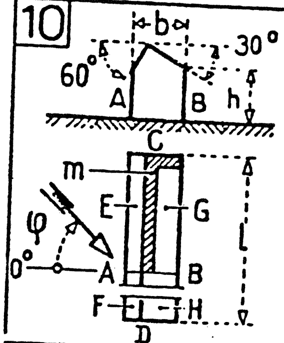
φ	A	B	C	D	E	F	G	H
0°	+0,9	-0,5	-0,6	-0,6	-0,5	-0,5	-0,5	-0,5
45°	+0,5	-0,6	+0,4	-0,4	-1,2	-0,7	-1,1	-0,7
90°	-0,4	-0,3	+0,9	-0,2	-0,3	0	-0,3	0
180°	-0,4	+0,8	-0,7	-0,7	+0,1	+0,1	+0,2	+0,2
45°	SECTION „m" $c_{pa}^{*} = -1,4$							

INT. PRESS. COEFF. c_{pi}

WIND DIRECT. φ =	0°	45°	90°	180°
OPEN UNIF. DISTR.	±0,2	±0,2	±0,2	±0,2
SIDE A PREDOM.	+0,8	+0,4	-0,2	-0,3
" B " "	-0,4	-0,5	-0,1	+0,7
" C " "	-0,5	+0,3	+0,8	-0,6
ROOF EF " "	-0,4	-0,8	0	0

BLDG WITH SINGLE SLOPE ROOF

10



EXT. PRESS. COEFF. c_{pa} FOR h:b:l = 1:1:5

φ	A	B	C	D	E	F	G	H
0°	+0,9	-0,5	-0,6	-0,6	+0,6	+0,6	-0,5	-0,5
45°	+0,5	-0,8	+0,4	-0,5	+0,2	-0,1	-1,0	-0,8
90°	-0,4	-0,4	+0,9	-0,3	-0,4	0	-0,4	-0
180°	-0,5	+0,9	-0,6	-0,6	-0,5	-0,5	-0,1	-0,1
45°	SECTION „m" $c_{pa}^{*} = -1,3$							

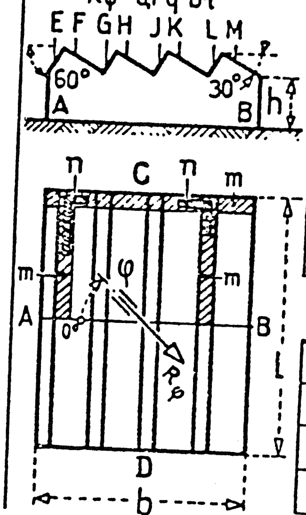
INT. PRESS. COEFF. c_{pi}

WIND DIRECT φ =	0°	45°	90°	180°
OPEN UNIF. DISTR.	±0,2	±0,2	±0,2	±0,2
SIDE A PREDOM.	+0,8	+0,4	-0,1	-0,4
" B " "	-0,4	-0,7	-0,1	+0,8
" C " "	-0,5	+0,3	+0,8	-0,5
ROOF EF " "	+0,5	0	-0,1	-0,4

BLDG WITH SHED ROOF

11

FRICTION IN WIND DIR.
$R_{\varphi} = 0{,}1 \cdot q \cdot b \cdot l$



EXT. PRESSURE COEFF. c_{pa} FOR h:b:l = 1:4:5

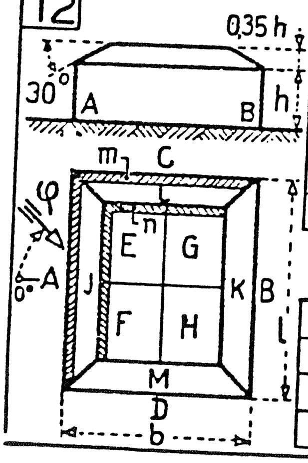
φ	A	B	C	D	E	F	G	H	J	K	L	M
0°	+0,9	-0,3	-0,4	-0,4	+0,6	-0,6	-0,6	-0,5	-0,5	-0,4	-0,3	-0,3
45°	+0,5	-0,4	+0,5	-0,3	+0,2	-0,8	-0,5	-0,4	-0,2	-0,4	-0,2	-0,5
90°	-0,4	-0,4	+0,9	-0,3	-0,3	-0,4	-0,4	-0,4	-0,4	-0,4	-0,4	-0,3
180°	-0,3	+0,9	-0,3	-0,3	-0,2	-0,3	-0,3	-0,4	-0,4	-0,6	-0,6	-0,1
0°-180°	SECTION „m" $c_{pa}^{*} = -1,3$. SECTION „n" $c_{pa}^{*} = -2.0$											

MULTIPLE SHED ROOF

INT. PRESS. COEFF. c_{pi} FOR WIND DIR.

φ =	0°	45°	90°	180°
OPENING UNIFORMLY DISTRIBUTED	±0,2	±0,2	±0,2	±0,2
" ON SIDE A PREDOMINATING	+0,8	+0,4	-0,3	-0,2
" " B " "	-0,2	-0,3	-0,3	+0,8
" " C " "	-0,3	+0,4	+0,8	-0,2

12



EXT. PRESS. COEFF. c_{pa} FOR h:b:l = 1:3:4

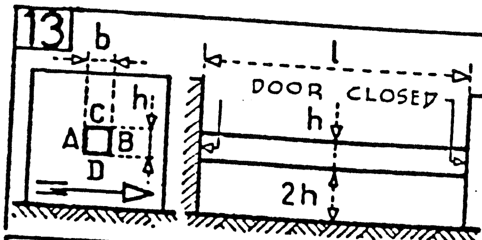
φ	A	B	C	D	E	F	G	H	J	K	L	M
0°	+0,9	-0,5	-0,6	-0,6	-0,8	-0,8	-0,4	-0,4	-1,0	-0,4	-0,5	-0,5
45°	+0,5	-0,5	+0,5	-0,4	-0,6	-0,5	-0,5	-0,5	-0,5	-0,5	-0,5	-0,5
90°	-0,5	-0,5	+0,9	-0,4	-0,8	-0,4	-0,8	-0,4	-0,4	-0,4	-1,0	-0,4
0°-90°	SECTION „m" $c_{pa}^{*} = -1,1$. SECTION „n" $c_{pa}^{*} = -1,5$											

CLIPPED FLAT ROOF

INT. PRESS. COEFF. c_{pi} FOR WIND DIR.

φ =	0°	45°	90°
OPENING UNIFORMLY DISTRIBUTED	±0,2	±0,2	±0,2
" ON SIDE A PREDOMINATING	+0,8	+0,4	-0,4
" " " " B " "	-0,4	-0,4	-0,4
" " " " C " "	-0,5	+0,4	+0,8

13



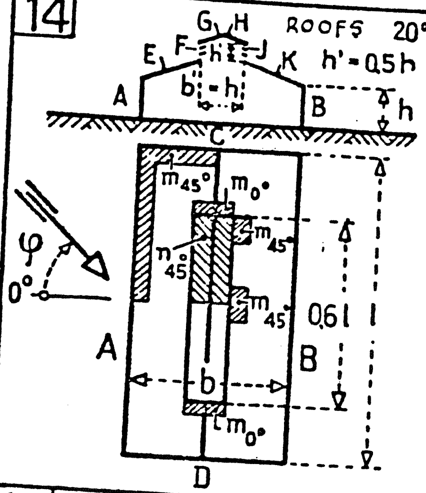
EXT. PRESS. COEFF. c_{pa} FOR OBJECTS $h:b:l = 1:1:10$ BETWEEN LARGE WALLS

φ	A	B	C	D
0°	+0,8	-1,2	-1,4	-1,5

INT. PRESS. COEFF. c_{pi} φ=	0°
OPEN UNIFORM DISTR.	-0,5
" ON A PREDOM.	+0,7
" B " "	-1,1
" C " "	-1,3

CLOSED CONNECTING PASSAGEWAY

14



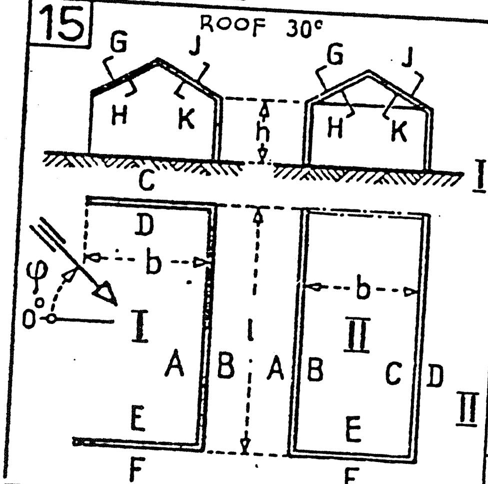
EXT. PRESS. COEFF. c_{pa} FOR $h:b:l = 1:4.8$

φ	A	B	C	D	E	F	G	H	J	K
0°	+0,8	-0,5	-0,7	-0,7	-0,2	+0,6	-1,0	-0,6	-0,5	-0,6
45°	+0,4	-0,5	+0,4	-0,5	-0,3	+0,2	-1,3	-1,4	-1,0	-0,7
90°	-0,4	-0,4	+0,8	-0,3	-0,4	-0,2	-0,3	-0,3	-0,2	-0,4
0 & 45°	SECTION „m" $c_{pa}^{*}=-1,2$ SECTION „n" $c_{pa}^{*}=-2,4$									

CLOSED BUILDING WITH ROOF VENT

INT. PRESS. COEFF. c_{pi} FOR φ =	0°	45°	90°
VENTS AT F & J CLOSED	±0,2	±0,2	±0,2
" " " F & J OPEN	-0,2	-0,5	-0,3
" " " F ONLY OPEN	+0,5	+0,1	-0,2
" " " J " "	-0,4	-0,9	-0,2

15



I PRESS. COEFF. c_p FOR $h:b:l = 1:2:4$ 1 LONG WALL „OPEN"

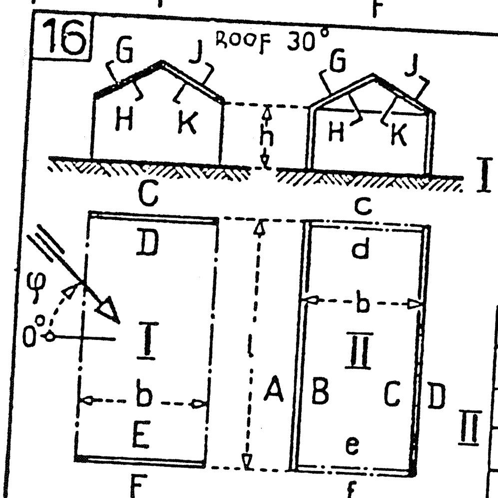
φ	A	B	C	D	E	F	G	H	J	K
0°	+0,8	-0,5	-0,7	+0,8	+0,8	-0,7	-0,3	+0,8	-0,4	+0,8
45°	+0,7	-0,6	+0,4	+0,6	+0,8	-0,4	-0,2	+0,6	-0,7	+0,7
60°	+0,3	-0,7	+0,7	+0,3	+0,4	-0,4	-0,3	+0,2	-0,6	+0,2
180°	-0,5	+0,9	-0,8	-0,5	-0,5	-0,8	-0,4	-0,5	-0,2	-0,5

BUILDING OPEN ON ONE SIDE

II PRESS. COEFF. c_p FOR $h:b:l = 1:2:4$ 1 END WALL „OPEN"

φ	A	B	C	D	E	F	G	H	J	K
0°	+0,9	-0,7	-0,7	-0,4	-0,7	-0,8	-0,2	-0,7	-0,4	-0,7
45°	+0,5	+0,7	+0,8	-0,5	+0,7	-0,4	-0,3	+0,7	-0,6	+0,8
60°	+0,1	+0,9	+0,9	-0,6	+0,9	-0,4	-0,3	+0,9	-0,7	+0,9
90°	-0,5	+0,8	+0,8	-0,5	+0,8	-0,3	-0,4	+0,8	-0,4	+0,8

16



I PRESS. COEFF. c_p FOR $h:b:l = 1:2:4$ 2 LONG WALLS „OPEN"

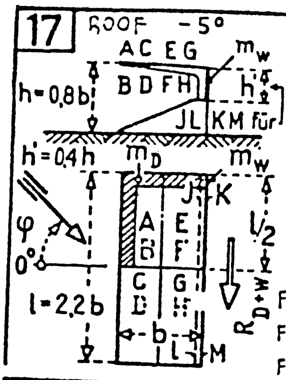
φ	C	D	E	F	G	H	J	K
0°	-0,2	-0,7	-0,7	-0,2	+0,4	-0,9	-0,5	-0,8
45°	+0,5	-0,4	+0,5	-0,4	0	-0,3	-0,6	0
60°	+0,7	-0,6	+0,5	-0,4	-0,3	-0,1	-0,7	-0,3

BUILDING OPEN ON TWO SIDES

II PRESS. COEFF. c_p FOR $h:b:l = 1:2:4$ 2 END WALLS „OPEN"

φ	A	B	C	D	G	H	J	K
0°	+0,9	-0,7	-0,7	-0,4	-0,2	-0,7	-0,4	-0,7
45°	+0,5	-0,4	-0,1	-0,8	-0,3	-0,4	-0,8	-0,3
60°	+0,3	-0,2	+0,1	-0,5	-0,3	-0,1	-0,8	+0,1
60°	END SECT. c = +0,7 d = -0,6 END SECT. e = +0,6 f = -0,8							

17 ROOF −5°



PRESS. COEFF. Cp FOR TOP & BOTTOM of ROOF

φ	A	B	C	D	E	F	G	H
0°	−1,0	+0,9	−1,0	+0,9	−0,7	+0,9	−0,7	+0,9
45°	−1,0	+0,7	−0,7	+0,4	−0,5	+0,8	−0,5	+0,3
135°	−0,4	−1,1	−0,7	−1,0	−0,9	−1,1	−0,9	−1,0
180°	−0,6	−0,3	−0,6	−0,3	−0,6	−0,3	−0,6	−0,3

Cp FRONT & BACK of WALL

φ	J	K	L	M
0°	+0,9	−0,5	+0,9	−0,5
45°	+0,8	−0,6	+0,4	−0,4
135°	−1,1	+0,6	−1,0	+0,4
180°	−0,3	+0,9	−0,3	+0,9

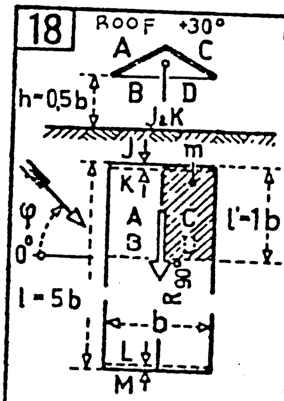
GRAND STANDS OPEN THREE SIDES

FOR φ=45° SECTION of ROOF „m_D" Cp^*_{TOP} = −2,0 Cp^*_{BOTTOM} = +1,0

FOR φ=60° SECTION of WALL „m_W" Cp^*_K = −1,0 Cp^*_J = +1,0

FOR φ=90° TANGENTIALLY ACTING FRICTION $R_D = 0{,}1 \cdot q\, b \cdot l$, $R_w = 0{,}1 \cdot q\, h' \cdot l$

18 ROOF +30°



PRESS. COEFF. Cp

φ=0&45° A÷D FULL LENGTH

φ=90° A÷D ONLY PART l'

φ	A	B	C	D
0°	+0,6	−1,0	−0,5	−0,9
45°	0,1	−0,3	−0,6	−0,3
90°	−0,3	−0,4	−0,3	−0,4

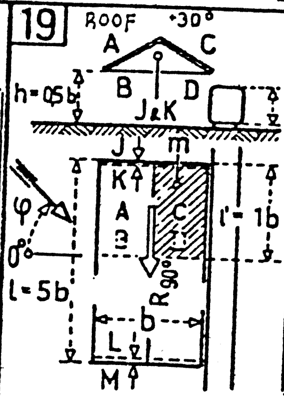
GABLE ENDS

φ	J	K	L	M
90°	+0,8	−0,4	+0,3	−0,3

FOR φ=45° SECTION „m" Cp^* = −1,0 Cp^* = −0,2

FOR φ=90° TANGENTIALLY ACTING FRICTION 0,05·q·l·b

19 ROOF +30°



PRESS. COEFF. Cp WITH EFFECT of TRAIN OR STORED MATERIAL

h'=0,8h

φ=0°,45°,180° A÷D FULL LENGTH

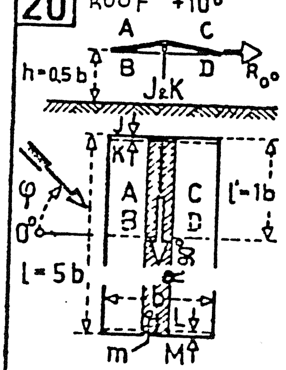
φ=90° A÷D ONLY PART l'

φ	A	B	C	D
0°	+0,1	+0,8	−0,7	+0,9
45°	−0,1	+0,5	−0,8	+0,5
90°	−0,4	−0,5	−0,4	−0,5
180°	−0,3	−0,6	+0,4	−0,6

FOR φ=45° SECTION „m" Cp^*_{TOP} = −1,5, Cp^*_{BOTTOM} = +0,5

FOR φ=90° GABLE AND FRICTION AS IN 18

20 ROOF +10°



Cp

φ=0&45° A÷D FULL LENGTH

φ=90° A÷D ONLY PART l'

φ	A	B	C	D
0°	−1,0	+0,3	−0,5	+0,2
45°	−0,3	+0,1	−0,3	+0,1
90°	−0,3	0	−0,3	0

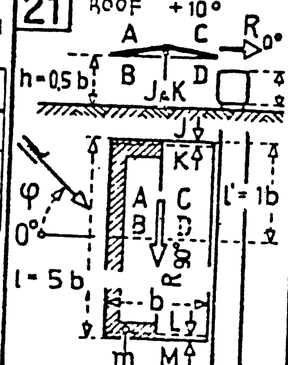
GABLE ENDS

φ	J	K	L	M
90°	+0,8	−0,6	+0,3	−0,4

FOR φ=0° SECTION „m" Cp^*_{TOP} = −1,0 Cp^*_{BOTTOM} = +0,4

FOR φ=0÷90° TANGENTIALLY ACTING FRICTION R=0,1·q·l·b

21 ROOF +10°



PRESS. COEFF. Cp WITH EFFECT of TRAIN OR STORED MATERIAL

h'=0,8h

φ=0°,45°,180° A÷D FULL LENGTH

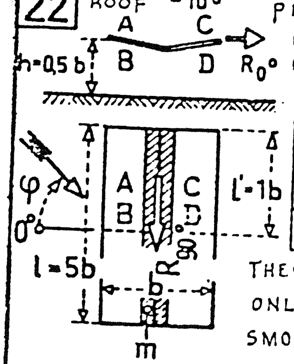
φ=90° A÷D ONLY PART l'

φ	A	B	C	D
0°	−1,3	+0,8	−0,6	+0,7
45°	−0,5	+0,4	−0,3	+0,3
90°	−0,3	0	−0,3	0
180°	−0,4	−0,3	−0,6	−0,3

FOR φ=0° SECTION „m" Cp^*_{TOP} = −1,6, $C_D{}^*_{BOT}$ = +0,9

FOR φ=0°÷180° GABLE AND FRICTION AS IN 20

22 ROOF −10°



PRESS. COEFF. Cp

φ=0°&45° A÷D FULL LENGTH

φ=90° A÷D ONLY PART l'

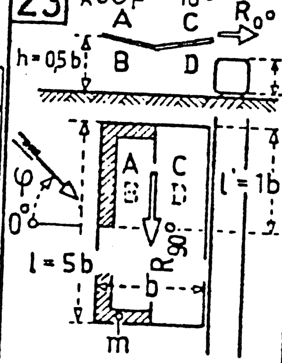
φ	A	B	C	D
0°	+0,3	−0,7	+0,2	−0,9
45°	0	−0,2	+0,1	−0,3
90°	−0,1	+0,1	−0,1	+0,1

THESE COEFF ARE VALID ONLY FOR RELATIVELY SMOOTH UNDERSIDE B÷D

FOR φ=0° SECTION „m" $C_{D\,TOP}^*$ = +0,4, Cp^*_{BOTTOM} = −1,5

FOR φ=0÷90° TANGENTIALLY ACTING FRICTION R=0,1·q·l·b

23 ROOF −10°



PRESS. COEFF. Cp WITH EFFECT. of TRAIN OR STORED MATERIAL

h'=0,8h

φ=0°,45°,180° A÷D FULL LENGTH

φ=90° A-D ONLY PART l'

φ	A	B	C	D
0°	−0,7	+0,8	−0,6	+0,6
45°	−0,4	+0,3	−0,2	+0,2
90°	−0,1	+0,1	−0,1	+0,1
180°	−0,4	−0,2	−0,6	−0,3

FOR φ=0° SECTION „m" $C_{D\,TOP}^*$ = −1,1, Cp^*_{BOTTOM} = +0,9

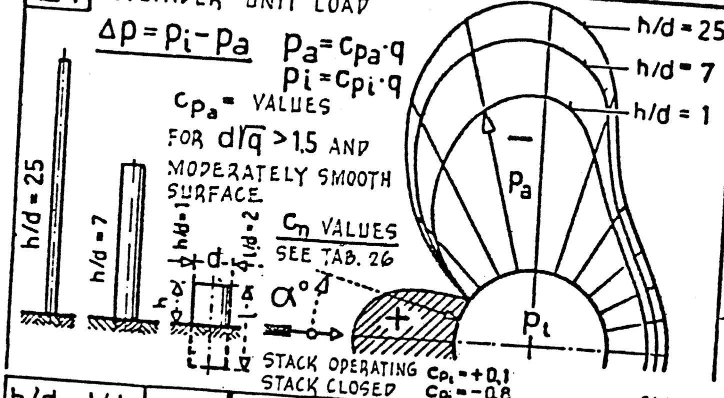
FOR φ=0÷180° TANGENTIALLY ACTING FRICTION R=0,1 q·l·b

24 CYLINDER - UNIT LOAD

$\Delta p = p_i - p_a$ $p_a = c_{pa} \cdot q$ $p_i = c_{pi} \cdot q$

c_{pa} = VALUES FOR $d\sqrt{q} > 1{,}5$ AND MODERATELY SMOOTH SURFACE

c_n VALUES SEE TAB. 26



SPHERE - UNIT LOAD

$\Delta p = p_i - p_a$ $p_a = c_{pa} \cdot q$

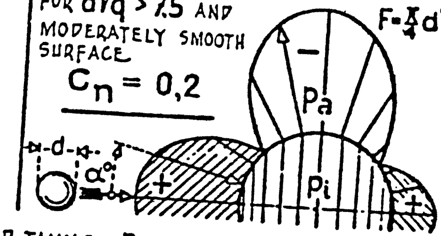
c_{pa} = VALUES FOR $d\sqrt{q} > 7{,}5$ AND MODERATELY SMOOTH SURFACE

$K_n = c_n \cdot q \cdot F$

$F = \frac{\pi}{4} d^2$

$c_n = 0{,}2$

STACKS TANKS



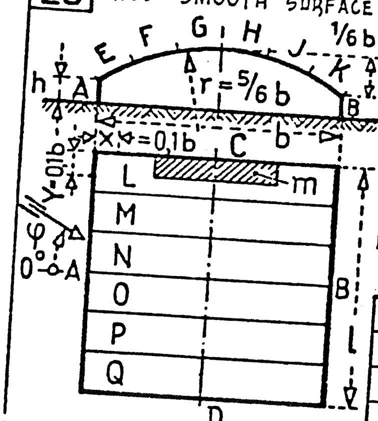
CLOSED TANKS p_i = WORKING PRESS IN kg/m^2

h/d	l/d	α =	0°	15°	30°	45°	60°	75°	90°	105°	120°	135°	150°	165°	180°
25	50	c_{pa} =	+1,0	+0,8	+0,1	-0,9	-1,9	-2,5	-2,6	-1,9	-0,9	-0,7	-0,6	-0,6	-0,6
7	14	c_{pa} =	+1,0	+0,8	+0,1	-0,8	-1,7	-2,2	-2,2	-1,7	-0,8	-0,6	-0,5	-0,5	-0,5
1	2	c_{pa} =	+1,0	+0,8	+0,1	-0,7	-1,2	-1,6	-1,7	-1,2	-0,7	-0,5	-0,4	-0,4	-0,4
		c_{pa} =	+1,0	+0,9	+0,5	-0,1	-0,7	-1,1	-1,2	-1,0	-0,6	-0,2	+0,1	+0,3	+0,4

25 MOD SMOOTH SURFACE

EXT. PRESS. COEFF. c_{pa} h:b:l = 1:12:12 RADIUS r = 5/6 b

HANGAR WITH CURVED ROOF



φ	A	B	C	D	E	F	G	H	J	K
0°	+0,7	-0,2	-0,3	-0,3	-0,1	-0,5	-0,8	-0,8	-0,4	-0,1
30°	+0,6	-0,3	+0,2	-0,4	-0,1	-0,4	-0,7	-0,9	-0,7	-0,4

φ	A	B	C	D	L	M	N	O	P	Q
90°	-0,3	-0,3	+0,9	-0,3	-0,8	-0,7	-0,5	-0,3	-0,1	-0,1
30°	SECTION „m" c_{pa} = -1,8 WITH c_{pa} MIN. = -2,5									

INT. PRESS. COEFF.

φ =	0°	30°	90°
OPENING, UNIFORMLY DISTRIB. (WINDOWS & DOORS CLOSED)	±0,2	±0,2	±0,2
" " SIDE A PREDOMENATING (GATE Y OPEN)	+0,4	+0,7	-1,0
" " " C " " (ALL GATES OPEN)	-0,1	+0,6	+0,8
" " " C " " (ONLY GATE X OPEN)	-1,5	+0,7	+0,4

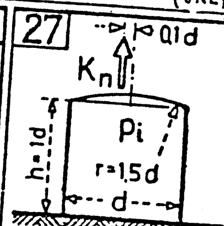
26 SLENDERNESS h/d =

VERTICAL CYLINDER $K_n = c_n \cdot q \cdot F$

c_n $d\sqrt{q} > 1{,}5$ $F = d \cdot h$

CROSS SECTION & ROUGHNESS	25	7	1
	c_n	c_n	c_n
MODERATELY SMOOTH (METAL, TIMBER, CONCRETE)	0,55	0,5	0,45
ROUGH SURFACE (ROUNDED RIBS h = 2 % d)	0,9	0,8	0,7
VERY ROUGH SURFACE (SHARP RIBS h = 8 % d)	1,2	1,0	0,8
SMOOTH & ROUGH SURFACE, SHARP EDGES	1,4	1,2	1,0

27 ROOF LOAD ON SMOOTH CLOSED TANK



$K_n = (p_i - p_a) \cdot F$

p_i = WORKING PRESS IN kg/m^2

$p_a = c_{pa} \cdot q$ $c_{pa} = -1{,}0$

$F = \pi/4\ d^2$

TANKS STACKS MESH

28 $K_n = c_n \cdot q \cdot F$ F = d·l

COEFF. FOR l/d > 100

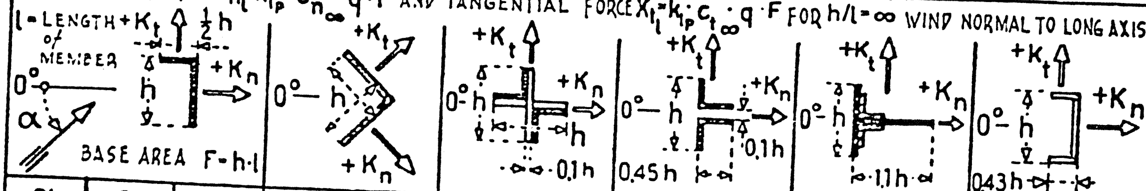
$d\sqrt{q}$	< 1,5	> 1,5
COEFF.	c_n	c_n
SMOOTH WIRES, RODS, PIPES	1,2	0,5
MOD. SMOOTH WIRES & RODS	1,2	0,7
FIVE WIRE CABLES	1,2	0,9
THICK WIRE CABLES	1,3	1,1

POLES ROPES WIRES

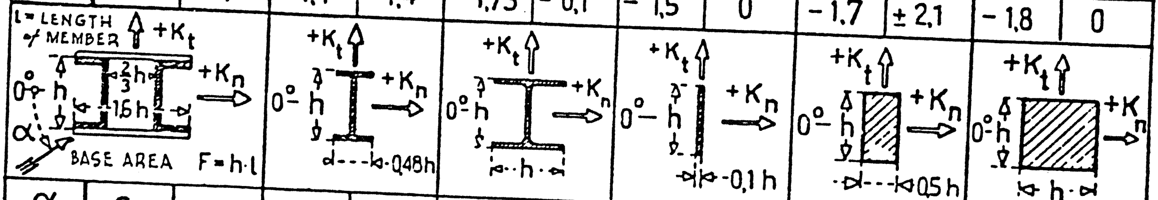
29 PRESS. COEFF. $C_{n\infty}$ & $C_{t\infty}$ FOR SINGLE AND ASSEMBLED SECTIONS

STRUCTURAL MEMBERS

· NORMAL FORCE $K_{n_l} = k_{l_p} \cdot c_{n_\infty} \cdot q \cdot F$ AND TANGENTIAL FORCE $X_{t_l} = k_{l_p} \cdot c_{t_\infty} \cdot q \cdot F$ FOR $h/l = \infty$ WIND NORMAL TO LONG AXIS



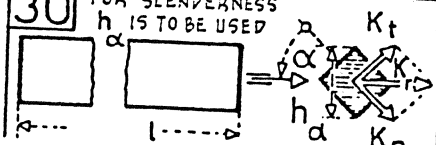
α	$C_{n\infty}$	$C_{t\infty}$	$C_{n\infty}$	$C_{t\infty}$	$C_{n\infty}$	$C_{t\infty}$	$C_{n\infty}$	$C_{t\infty}$	$C_{n\infty}$	$C_{t\infty}$	$C_{n\infty}$	$C_{t\infty}$
0°	+1,9	+0,95	+1,8	+1,8	+1,75	+0,1	+1,6	0	+2,0	0	+2,05	0
45°	+1,8	+0,8	+2,1	+1,8	+0,85	+0,85	+1,5	−0,1	+1,2	+0,9	+1,85	+0,6
90°	+2,0	+1,7	−1,9	−1,0	+0,1	+1,75	−0,95	+0,7	−1,6	+2,15	0	+0,6
135°	−1,8	−0,1	−2,0	+0,3	−0,75	+0,75	−0,5	+1,05	−1,1	+2,4	−1,6	+0,4
180°	−2,0	+0,1	−1,4	−1,4	−1,75	−0,1	−1,5	0	−1,7	±2,1	−1,8	0



α	$C_{n\infty}$	$C_{t\infty}$	$C_{n\infty}$	$C_{t\infty}$	$C_{n\infty}$	$C_{t\infty}$	$C_{n\infty}$	$C_{t\infty}$	$C_{n\infty}$	$C_{t\infty}$	$C_{n\infty}$	$C_{t\infty}$
0°	+1,4	0	+2,05	0	+1,6	0	+2,0	0	+2,1	0	+2,0	0
45°	+1,2	+1,6	+1,95	+0,6	+1,5	+1,5	+1,8	+0,1	+1,4	+0,7	+1,55	+1,55
90°	0	+2,2	±0,5	+0,9	0	+1,9	0	+0,1	0	+0,75	0	+2,0

30 FOR SLENDERNESS h_α IS TO BE USED

SLENDERNESS RATIO FOR SECTIONS



SLENDERNESS RATIO k_{l_p} FOR SECTIONS $k_{l_p} = c_{r_l}/c_{r_\infty}$ $\quad c_{r_\infty} = \sqrt{c_{n_\infty}^2 + c_{t_\infty}^2}$

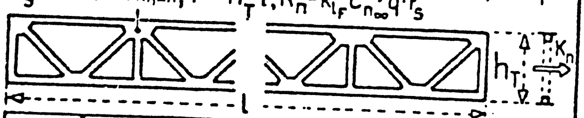
l/h_α	5	10	20	35	50	100	∞
k_{l_p}	0,60	0,65	0,75	0,85	0,90	0,95	1,00

COEFF. FOR TRUSSED CONSTRUCTION MADE FROM DIFFERENT SHARP-EDGED SECTIONS

PLANE TRUSSES

31 $C_{n\infty}$ FOR FULLNESS RATIO $F_s/F = 0 \div 1$ WIND NORMAL TO SURFACE $h_T \cdot l$

F_s = SECTION AREA, $F = h_T \cdot l$, $K_n = k_{l_F} \cdot c_{n_\infty} \cdot q \cdot F_s$



F_s/F =	0	0,1	0,15	0,2	0,3÷0,8	0,95	1,0
$C_{n\infty}$ =	2,0	1,9	1,8	1,7	1,6	1,8	2,0

32 SLENDERNESS RATIO k_{l_F} FOR TRUSSES

SLENDERNESS RATIO FOR TRUSSES

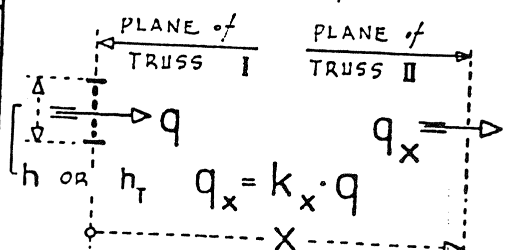
$k_{l_F} = c_{n_l}/c_{n_\infty}$

F_s/F =	0,25	0,5	0,9	0,95	1,0
l/h_T = 5	0,96	0,91	0,87	0,77	0,60
= 20	0,98	0,97	0,94	0,89	0,75
= 50	0,99	0,98	0,97	0,95	0,90
= ∞	1,0	1,0	1,0	1,0	1,0

33 SHIELDING FACTORS $k_x = q_x/q$ FOR TRUSSED CONST. MADE FROM IDENTICAL OR MIXED SHARP-EDGED SECTIONS

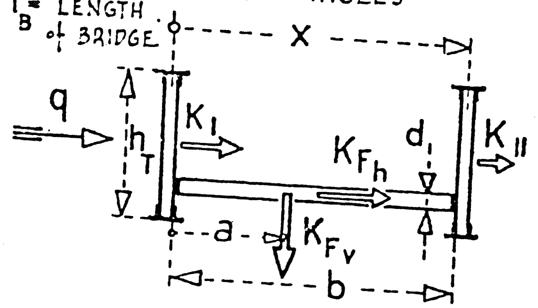
SHIELDING

F_s/F =	0,1	0,2	0,3	0,4	0,5	0,6	0,8	1,0
x/h = 1/2	0,93	0,75	0,56	0,38	0,19	0	0	0
= 1	0,99	0,81	0,65	0,48	0,32	0,15	0,15	0,15
" = 2	1,00	0,87	0,73	0,59	0,44	0,30	0,30	0,30
" = 4	1,00	0,90	0,78	0,65	0,52	0,40	0,40	0,40
" = 6	1,00	0,93	0,83	0,72	0,61	0,50	0,50	0,50



34 CALCULATION FOR TRUSS AND PLATE GIRDER BRIDGE

I. WITHOUT VEHICLES



WINDWARD GIRDER $K_I = c_{n_l} \cdot q \cdot F_S$

LEEWARD " $K_{II} = c_{n_l} \cdot k_x q \cdot F_S$

c_{n_l} FROM TABLE No 31 & 32 PLATE VII

F_S & k_x " " No 31 & 33 " VII

DECK, HORIZ. LOAD $K_{F_h} = 1{,}0 \cdot q \cdot d \cdot l_B$

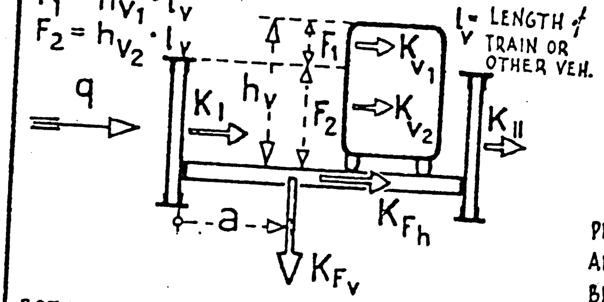
" VERT. " $K_{F_v} = 0{,}6 \cdot q \cdot b \cdot l_B$

LOCATION of K_{F_v} $a = 0{,}4 \cdot b$

II. WITH TRAIN

$F_1 = h_{v_1} \cdot l_v$

$F_2 = h_{v_2} \cdot l_v$



RAILWAY HIGHWAY AND PROTECTIVE BRIDGES

PEDESTRIAN AND LOADING BRIDGES

BOTH GIRDERS K_I & K_{II} = AS IN CASE I

TRAIN OR TRAFFIC LOAD:
- RAILWAY $h_v = 3{,}8$ $c_n = 1{,}5$
- HIGHWAY $= 3{,}0$ $= 1{,}2$
- PEDESTRIAN $= 1{,}7$ $= 1{,}0$

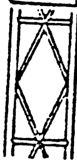
$K_{v_1} = c_n \cdot q \cdot F_1$ $K_{v_2} = c_n \frac{2}{3} q \cdot F_2$

DECK HORIZ. LOAD $K_{F_h} = 1{,}2 \cdot q \cdot d \cdot l_B$

" VERT. " $K_{F_v} = 0{,}8 \cdot q \cdot b \cdot l_B$

LOCATION of K_{F_v} $a = 0{,}4 \cdot b$

35 CALCULATION FOR THREE DIMENSIONAL TRUSS CONSTRUCTION WITH $F_S/F \leqq 0{,}3$



TOTAL LOAD IN WIND DIR. $K_{TOTAL} = \sum K_{MEMBER}$

UNSHIELDED MEMBERS $K_{St} = k_{l_p} c_{\infty\beta}\, q \cdot F \cos\beta$

SHIELDED MEMBERS $K_{St} = k_{l_p} c_{\infty\beta} \cdot k_x q \cdot F \cos\beta$

TRANSMISSION AND CABLEWAY TOWERS

CRANES

FOR TRUSSED CONST. WITH SHARPEDGED MEMB. $c_{\infty\beta} = k_\beta c_{\infty 90°}$, $c_{\infty 90°} = c_{n_\infty}$ & c_{t_∞} FROM TAB. 29

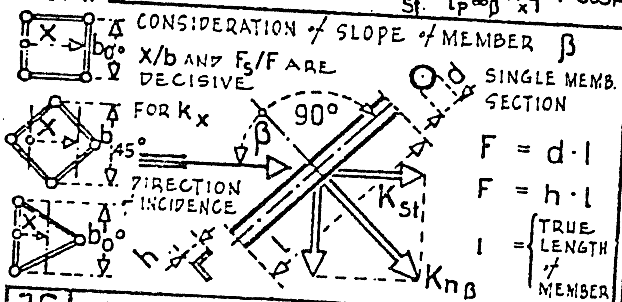
β	0°	15°	30°	45°	60°
k_β	1,00	0,98	0,93	0,88	0,80

k_x " " 33; k_{l_p} " " 30

CONSIDERATION of SLOPE of MEMBER β

X/b AND F_S/F ARE DECISIVE

FOR k_x



$F = d \cdot l$

$F = h \cdot l$

l = TRUE LENGTH of MEMBER

FOR TRUSSED COUPLE WITH ROUND MEMBERS WITH SMOOTH & ROUGH SURFACE : $d\sqrt{q} < 1{,}5$

β	0°	15°	30°	45°	60°
$c_{\infty\beta}$	1,20	1,16	1,04	0,85	0,60

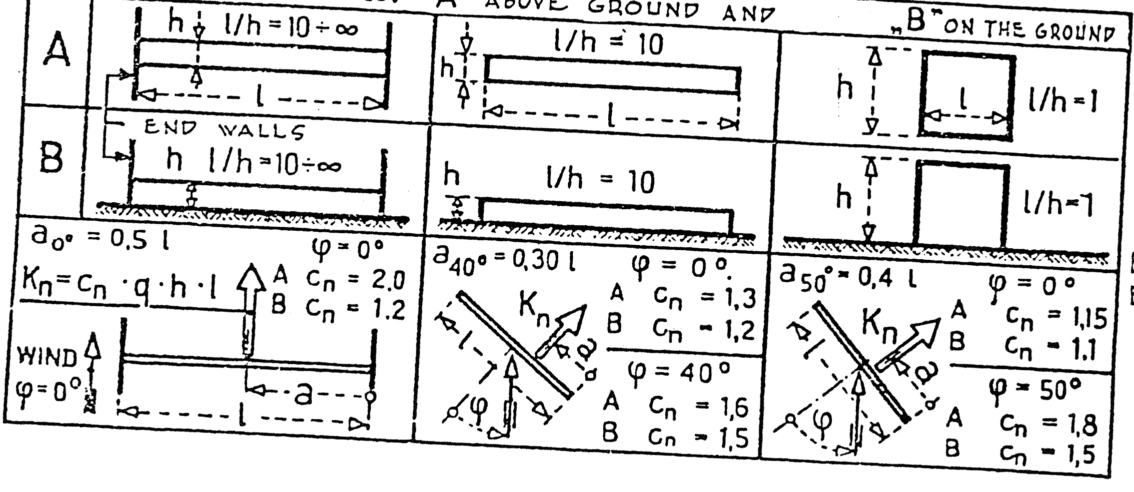
k_x FROM TABLE 33; k_l " " 30

FOR TRUSSED CONSTR WITH ROUND MEMBERS WITH MODERATLY SMOOTH SURFACE $d\sqrt{q} > 1{,}5$

β	0°	15°	30°	45°	60°
$c_{\infty\beta}$	0,6	0,58	0,53	0,42	0,28

$k_x = 0{,}95$; $k_l = 0{,}9$ FOR $l/d = 25$

36 FLAT PLATES AND WALLS "A" ABOVE GROUND AND "B" ON THE GROUND



FREE STANDING PROTECTIVE AND BLDG WALLS

BILL BOARDS