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Chapter V-2:
1972

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Code of

Basic data for the design of buildings —

Chapter V: Loading —

Part 2: Wind Loads

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Contents

Foreword

Part 1 of the current edition of this Code of Practice, which deals with dead and imposed loads other than wind loads, was published in 1967; Part 2 deals with wind loads.

In 1970 a revision of the 1952 edition was published as Part 2 in which pressure coefficients and force coefficients for a limited range of building shapes were given. The present edition extends this range by the addition of pressure coefficients for all building shapes for which there is sufficient information available. It also gives force coefficients for unclad structures and additional appendices on ice formation and wind forces on circular sections.

The committee responsible for the preparation of this Code appreciates that there are many building shapes not covered, but the necessary data are not available for these to be included in a code and there is need for additional research, particularly on the relationship between wind tunnel conditions and those in reality, to enable figures to be given for further shapes.

As compared with the 1952 edition, this edition treats wind loading in more detail because changes in building design and construction have, for some types of buildings, increased the influence of wind loading in relation to other imposed loads. This edition also uses wind data that have become available since 1952.

An important change in this edition from the 1952 edition is the adoption of gust loadings as the basis for design in place of the mean load averaged over one minute, which had been the basis hitherto. There is evidence to justify this change which will lead to a more realistic assessment of the wind loads. At the same time the opportunity has been taken to assess the probable wind speeds on a statistical basis in accordance with the current advice of the Meteorological Office.

In this Code numerical values have been given in terms of SI units, details of which are to be found in BS 3763, "*The International System of units (SI)*". For the convenience of those who wish to use them, approximate equivalents are also given in metric technical units and in imperial units; more accurate conversions can be obtained from BS 350, "*Conversion factors and tables*". A wind speed and pressure conversion chart (SI: metric: imperial) is given in Figure 3. For the purposes of this Code, the principal difference between SI units and metric technical units lies in the measurement of force, load and load intensity. The SI unit of force and load is the newton (symbol, N); it is that force which, when acting on a mass of 1 kg, imparts to it an acceleration of 1 m/s^2. Thus, under conditions of standard gravity, the following relationships apply:

1 kgf = 9.806 65 N (exact value)

1 lbf ≡ 4.448 N (approximate value).

This Code of Practice represents a standard of good practice and therefore takes the form of recommendations. Compliance with it does not confer immunity from relevant statutory and legal requirements.

Summary of pages

This document comprises a front cover, an inside front cover, pages i to iv, pages 1 to 48 and a back cover.

This standard has been updated (see copyright date) and may have had amendments incorporated. This will be indicated in the amendment table on the inside front cover.

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1 Scope

This Part of this Code gives methods for calculating the maximum gust wind loads that should be taken into account when designing buildings or components thereof.

It does not apply to buildings that are of unusual geometric shape, or have unusual site locations warranting special consideration, the loading for which should be obtained by experimental methods.

It does not apply to buildings whose structural properties (weight, stiffness, frequency or damping) make them susceptible to dynamic excitation. These should be assessed using published sources, examples of which are given in Appendix J, or by experimental methods. It should be noted that if a building is susceptible to excitation by vortex-shedding or other aerodynamic instability the maximum dynamic response may occur at wind velocities lower than the maximum.

For the purposes of this Code, "experimental methods" include properly-conducted wind tunnel tests which meet the provisions given in Appendix K. These experimental methods are also recommended when wind loading data are required in more detail than is given in this Code, or when the form of the building can be changed in response to the test results in order to give an optimal design.'

2 Definitions

For the purposes of this Code, the following definitions apply:

2.1
breadth[1)]

the dimension of the building normal to the direction of the wind

2.2
depth[1)]

the dimension of the building measured in the direction of the wind

2.3
height

the height of a building above the ground adjacent to that building

2.4
length[2)]

the greater horizontal dimension of a building above the ground adjacent to that building; or the length, between supports, of an individual structural member

2.5
width[2)]

the lesser horizontal dimension of a building above the ground adjacent to that building; or the width of a structural member across the direction of the wind

2.6
height above ground

the dimension above the general level of the ground to windward

2.7
element of surface area

the area of surface over which the pressure coefficient is taken to be constant

2.8
effective frontal area

the area normal to the direction of the wind or "shadow area"

2.9
dynamic pressure of wind

the free stream dynamic pressure resultant from the design wind speed

[1)] Breadth and depth of a building are dimensions related to the direction of the wind.
[2)] Length and width are dimensions related to the plan form.

2.10
pressure coefficient

the ratio of the pressure acting at a point on a surface to the dynamic pressure of the incident wind

2.11
force coefficient

a non-dimensional coefficient such that the total wind force on a body is the product of the force coefficient multiplied by the dynamic pressure of the incident wind times an appropriate area, as defined in the text

2.12
topography

the nature of the earth's surface as influenced by the hill and valley configurations

2.13
ground roughness

the nature of the earth's surface as influenced by small-scale obstructions such as trees and buildings (as distinct from topography)

3 Symbols

A = element of surface area
A_e = effective frontal area
b = breadth
C_f = force coefficient
C_{fn} = normal force coefficient
C_{ft} = transverse force coefficient
C_t = frictional drag coefficient
C_p = pressure coefficient
C_{pe} = external pressure coefficient
C_{pi} = internal pressure coefficient
d = depth
D = diameter
F = force
F_n = normal force
F_t = *transverse force*
F' = frictional force
h = height
H = height above ground
j = width of member as indicated in diagram
$j\alpha$ = width of member across direction of wind
k = a constant
K = reduction factor
l = length
p = pressure on surface
P_e = external pressure
P_i = internal pressure
P = total load intensity
q = dynamic pressure of wind (stagnation pressure)
R_e = Reynolds number
S_1 = topography factor
S_2 = ground roughness, building size and height above ground factor

S_3 = a statistical factor
V = basic wind speed
V_s = design wind speed
w = width of building
w' = bay width in multi-bay buildings
α = wind angle (from a given axis)
β = aerodynamic solidity ratio
η = shielding factor
υ = kinematic viscosity
ϕ = geometric solidity ratio

4 Outline of procedure for calculating wind loads on structures

4.1 The wind load on a structure should be calculated for:

1) the structure as a whole,

2) individual structural elements such as roofs and walls, and

3) individual cladding units and their fixings.

4.2 It is important to note that the wind load on a partially completed structure will be dependent on the method and sequence of construction and may be critical. It is reasonable to assume that the maximum design wind speed V_s will not occur during a short construction period, and a reduced factor S_3 can be used to calculate the probable maximum wind. The graphs of Figure 2 should not, however, be extrapolated to a period of less than two years.

4.3 The assessment of wind load should be made as follows:

1) *The basic wind speed V* appropriate to the district where the structure is to be erected is determined in accordance with **5.2**.

2) The basic wind speed is multiplied by factors S_1, S_2 and S_3 to give *the design wind speed* V_s for the part under consideration (see **5.3**, **5.4**, **5.5** and **5.6**)

$$V_s = V S_1 S_2 S_3.$$

3) The design wind speed is converted to *dynamic pressure q* using the relationship

$$q = kV_s^2.$$

Table 4, Table 5 and Table 6 give corresponding values of q and V_s in the three systems of units. See also Figure 2.

4) a) The dynamic pressure q is then multiplied by an appropriate pressure coefficient C_p to give the *pressure p exerted at any point on the surface* of a building.

$$p = C_p q.$$

If the value of the pressure coefficient C_p is negative this indicates that p is a suction as distinct from a positive pressure.

Since the resultant load on an element depends on the difference of pressure between opposing faces, pressure coefficients may be given for external surfaces C_{pe} and internal surfaces C_{pi}. The resultant wind load on an element of surface acts in a direction normal to that surface and then:

$$F = (C_{pe} - C_{pi})qA$$

where A is the area of the surface.

A negative value for F indicates that the resultant force is outwards. The *total wind load* on a structure may be obtained by vectorial summation of the loads on all the surfaces.

4) b) There is, however, another and shorter method of finding the *total wind load on the building as a whole* by using a force coefficient C_r, where such is available.

The total wind load F is then given by:

$$F = C_f q A_e$$

where A_e is the effective frontal area of the structure.

The direction in which the force acts is specified in the tables for the force coefficients.

Pressure coefficients and force coefficients are given in **7** for a range of building shapes. Force coefficients are given in **8** for unclad structures.

5 Design wind speed

5.1 General. The design wind speed V_s should be calculated from

$$V_s = V S_1 S_2 S_3$$

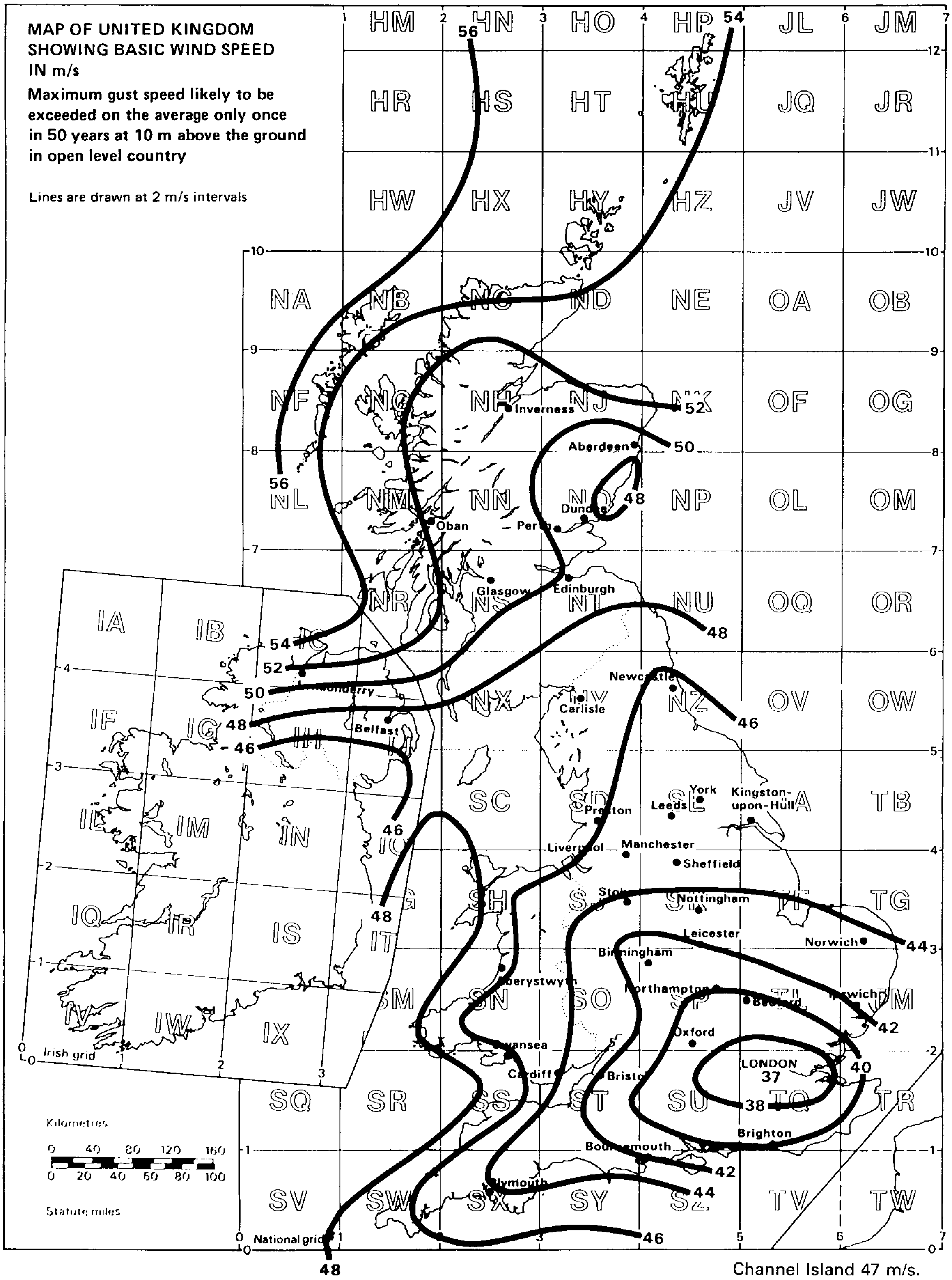
where V is the basic wind speed (see **5.2**), and S_1, S_2, S_3 are design wind speed factors (see **5.3** to **5.6** inclusive).

5.2 Basic wind speed

5.2.1 The basic wind speed V is the 3-second gust speed estimated to be exceeded on the average once in 50 years. This speed has been assessed for the United Kingdom by statistical analysis of the continuous wind records from the meterological stations after adjusting them, as necessary, to a common basis. The values are given as isopleths (lines of equal wind speed) drawn at 2 m/s intervals on the map in Figure 1. Values from this map represent the 3-second gust speed, at 10 m (33 ft) above ground in an open situation, that is likely to be exceeded on the average only once in 50 years.

5.2.2 The basic wind speed V is greatest from the direction of the prevailing winds, that is from the southwest to west in the United Kingdom. For buildings, whose design may be directionally dependent, a reduced windspeed may be used for other directions, at the designers discretion, by the use of Appendix L.

Table 1 deleted



Based on information provided by the Meteorological Office.

Figure 1 — Basic wind speed

5.3 Wind speed factors

5.3.1 The basic wind speed has been assessed to apply to open level country with no obstructions and should be adjusted to take account of variations from this standard. The map necessarily incorporates the effects of the general level of the ground above mean sea level. A topography factor, S_1, described more fully in **5.4**, takes account of large local variations in the ground surface; it deals with the effects of hills and the sheltering in valleys. The ground roughness, building size and variation of strong winds with height modify the basic speed and have been combined in the factor S_2 (see **5.5**). Finally, the period of years over which there will be exposure to the wind should be considered in modifying the basic speed. For this the factor S_3 (see **5.6**) may be used.

5.4 Topography factor S_1. The basic wind speed, V, given in Figure 1, takes account of the general level of the site above sea level. This does not allow for local topographic features such as hills, valleys, cliffs, escarpments or ridges, which can significantly affect the wind speed in their vicinity.

Near the summits of hills, or the crests of cliffs, escarpments or ridges the wind is accelerated. In valleys or near the foot of cliffs, steep escarpments or ridges, the wind may be decelerated. In all cases the variation of wind speed with height is modified from that appropriate to level terrain.

Where the average slope of the ground does not exceed 0.05 within a kilometre radius of the site, the terrain may be taken as level and the topography factor S_1 should be taken as 1.0.

In the vicinity of local topographic features the factor S_1 is a function of the upwind slope and the position of the site relative to the summit or crest, and will be within the range of $1.0 \leqslant S_1 \leqslant 1.36$. It should be noted that S_1 will vary with height above ground level, at a maximum near to the ground, and reducing to 1.0 at higher levels.

In certain steep-sided enclosed valleys, wind speeds may be less than in level terrain. Caution is necessary in applying S_1 values less than 1.0 and specialist advice should be sought in such situations.

A method for calculating the value of S_1 for values greater than 1.0 is given in Appendix D.

Table 2 deleted

5.5 Ground roughness, building size and height above ground, factor S_2. The factor S_2 takes account of the combined effect of ground roughness, the variation of wind speed with height above ground and the size of the building or component part under consideration.

In conditions of strong wind, the wind speed usually increases with height above ground. The rate of increase depends on ground roughness and also on whether short gusts or mean wind speeds are being considered. This is related to building size to take account of the fact that small buildings and elements of a building are more affected by short gusts than are larger buildings, for which a longer wind-averaging period is more appropriate.

5.5.1 *Ground roughness.* For the purposes of this Code the ground roughness is divided into four categories and buildings and their elements are divided into three classes as follows:

Ground roughness 1. Long fetches of open, level or nearly level country with no shelter. Examples are flat coastal fringes, fens, airfields and grassland, moorland or farmland without hedges or walls around the fields.

Ground roughness 2. Flat or undulating country with obstructions such as hedges or walls around fields, scattered windbreaks of trees and occasional buildings. Examples are most farmland and country estates with the exception of those parts that are well wooded.

Ground roughness 3. Surfaces covered by numerous large obstructions. Examples are well-wooded parkland and forest areas, towns and their suburbs, and the outskirts of large cities. The general level of roof-tops and obstructions is assumed at about 10 m, but the category will include built-up areas generally apart from those that qualify for category 4.

Ground roughness 4. Surfaces covered by numerous large obstructions with a general roof height of about 25 m, or more. This category covers only the centres of large towns and cities where the buildings are not only high, but are also not too widely spaced.

Further explanation of these ground roughness categories will be found in Appendix A.

5.5.2 *Cladding and building size*. The wind speed fluctuates from moment to moment and can be averaged over any chosen period of time. It has been found that the shortest time scale, 3 s, that is normally measured produces gusts whose dimensions envelop obstacles up to 20 m across. The longer the averaging time the greater is the linear length encompassed by the gust. For this reason three classes have been selected as described below.

Class A. All units of cladding, glazing and roofing and their immediate fixings and individual members of unclad structures (see **7.2**, Note).

Class B. All buildings and structures where neither the greatest horizontal dimension nor the greatest vertical dimension exceeds 50 m (165 ft).

Class C. All buildings and structures whose greatest horizontal dimension or greatest vertical dimension exceeds 50 m (165 ft).

The values of S_2 for variation of wind speed with height above ground for the various ground roughness categories and the building size classes are given in Table 3.

The height should be taken to the top of the structure or, alternatively, the height of the structure may be divided into convenient parts and the wind load on each part calculated, using a factor S_2 that corresponds to the height above ground of the top of that part. The load should be applied at the mid height of the structure or part, respectively. This also applies to pitch roofs.

Further explanation of the relationship between gusts and wind speed averaging time is given in Appendix B.

5.6 Factor S_3. The factor S_3 is based on statistical concepts, given in Appendix C, which take account of the degree of security required and the period of time in years during which there will be exposure to wind. Whatever wind speed is adopted for design purposes, there is always a probability, however small, that it may be exceeded in a storm of exceptional violence; the greater the period of years over which there will be exposure to the wind, the greater is this probability. Figure 2 shows values of S_3 equivalent to a period of exposure of 50 years plotted against this period of years.

Normally wind loads on completed structures and buildings should be calculated at $S_3 = 1$ with the following exceptions.

1) Temporary structures.

2) Structures where a shorter period of exposure to the wind may be expected.

3) Structures where a longer period of exposure to the wind may be required.

4) Structures where greater than normal safety is required.

For these special cases both the period of exposure to the wind and the probability level may be varied according to circumstances. In no case is the period of exposure to be taken as less than 2 years. Some examples are given in Appendix C.

Table 3 — Ground roughness, building size and height above ground, factor S_2

H	(1) Open country with no obstructions			(2) Open country with scattered windbreaks			(3) Country with many windbreaks; small towns; outskirts of large cities			(4) Surface with large and frequent obstructions, e.g. city centres		
	Class			Class			Class			Class		
	A	B	C	A	B	C	A	B	C	A	B	C
m												
3 or less	0.83	0.78	0.73	0.72	0.67	0.63	0.64	0.60	0.55	0.56	0.52	0.47
5	0.88	0.83	0.78	0.79	0.74	0.70	0.70	0.65	0.60	0.60	0.55	0.50
10	1.00	0.95	0.90	0.93	0.88	0.83	0.78	0.74	0.69	0.67	0.62	0.58
15	1.03	0.99	0.94	1.00	0.95	0.91	0.88	0.83	0.78	0.74	0.69	0.64
20	1.06	1.01	0.96	1.03	0.98	0.94	0.95	0.90	0.85	0.79	0.75	0.70
30	1.09	1.05	1.00	1.07	1.03	0.98	1.01	0.97	0.92	0.90	0.85	0.79
40	1.12	1.08	1.03	1.10	1.06	1.01	1.05	1.01	0.96	0.97	0.93	0.89
50	1.14	1.10	1.06	1.12	1.08	1.04	1.08	1.04	1.00	1.02	0.98	0.94
60	1.15	1.12	1.08	1.14	1.10	1.06	1.10	1.06	1.02	1.05	1.02	0.98
80	1.18	1.15	1.11	1.17	1.13	1.09	1.13	1.10	1.06	1.10	1.07	1.03
100	1.20	1.17	1.13	1.19	1.16	1.12	1.16	1.12	1.09	1.13	1.10	1.07
120	1.22	1.19	1.15	1.21	1.18	1.14	1.18	1.15	1.11	1.15	1.13	1.10
140	1.24	1.20	1.17	1.22	1.19	1.16	1.20	1.17	1.13	1.17	1.15	1.12
160	1.25	1.22	1.19	1.24	1.21	1.18	1.21	1.18	1.15	1.19	1.17	1.14
180	1.26	1.23	1.20	1.25	1.22	1.19	1.23	1.20	1.17	1.20	1.19	1.16
200	1.27	1.24	1.21	1.26	1.24	1.21	1.24	1.21	1.18	1.22	1.21	1.18

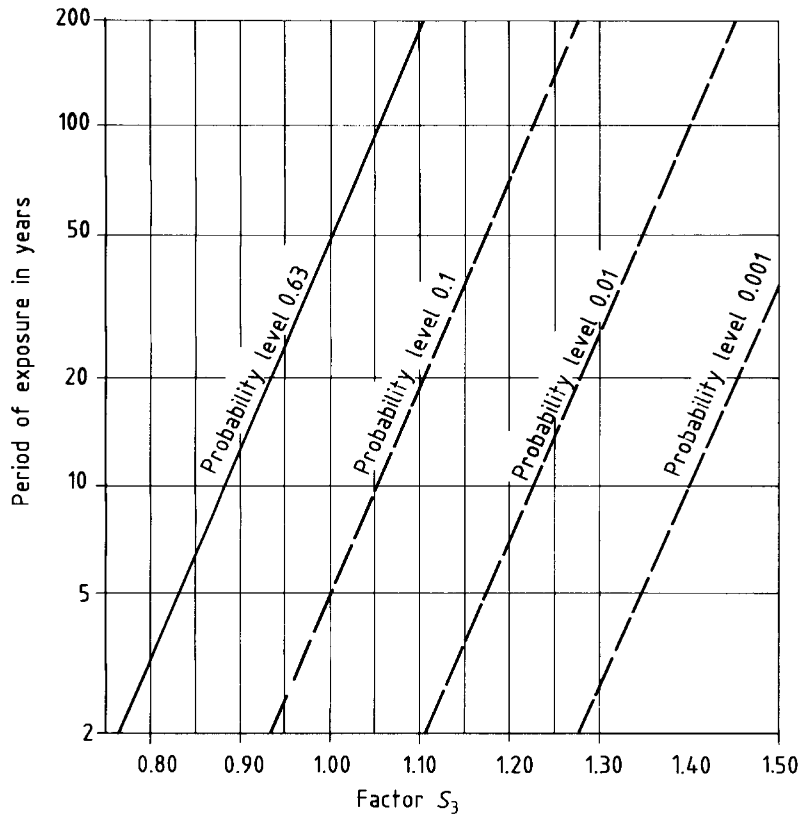


Figure 2 — A statistical factor S_3

6 Dynamic pressure of the wind

From the value of the design wind speed V_s obtained from 5 the dynamic pressure of the wind q above atmospheric pressure is obtained from Table 4, Table 5 or Table 6 according to the units being used; these are derived from the equation:

$q = kV_s^2$ [3)].

Values of k are as follows for the various units used in this Code:

k = 0.613 in SI units (N/m² and m/s)

k = 0.062 5 in metric technical units (kgf/m² and m/s)

k = 0.002 56 in imperial units (lbf/ft² and mile/h).

Any vertical line traversing the scales on Figure 3 indicates approximately the direct conversion between velocity and pressure on all three systems of units.

[3)] The design wind speed and therefore q depends on S_2, which varies with height (see **5.5.2**).

Table 4 — Values of *q* in SI units (N/m²)

V_s m/s	0	1.0	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0
10	61	74	88	104	120	138	157	177	199	221
20	245	270	297	324	353	383	414	447	481	516
30	552	589	628	668	709	751	794	839	885	932
40	981	1 030	1 080	1 130	1 190	1 240	1 300	1 350	1 410	1 470
50	1 530	1 590	1 660	1 720	1 790	1 850	1 920	1 990	2 060	2 130
60	2 210	2 280	2 360	2 430	2 510	2 590	2 670	2 750	2 830	2 920
70	3 000									

Table 5 — Values of *q* in metric technical units (kgf/m²)

V_s m/s	0	1.0	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0
10	6.3	7.6	9.0	10.6	12.3	14.1	16.0	18.1	20.3	22.6
20	25.0	27.6	30.3	33.1	36.0	39.1	42.3	45.6	49.0	52.6
30	56.3	60.1	64.0	68.1	72.3	76.6	81.0	85.6	90.3	95.1
40	100	105	110	116	121	127	132	138	144	150
50	156	163	169	176	182	189	196	203	210	218
60	225	233	240	248	256	264	272	281	289	298
70	306									

Table 6 — Values of *q* in imperial units (lbf/ft²)

V_s mile/h	0	1	2	3	4	5	6	7	8	9
30	2.3	2.5	2.6	2.8	3.0	3.1	3.3	3.5	3.7	3.9
40	4.1	4.3	4.5	4.7	5.0	5.2	5.4	5.7	5.9	6.1
50	6.4	6.7	6.9	7.2	7.5	7.7	8.0	8.3	8.6	8.9
60	9.2	9.5	9.8	10.2	10.5	10.8	11.2	11.5	11.8	12.2
70	12.5	12.9	13.3	13.6	14.0	14.4	14.8	15.2	15.6	16.0
80	16.4	16.8	17.2	17.6	18.1	18.5	18.9	19.4	19.8	20.3
90	20.7	21.2	21.7	22.1	22.6	23.1	23.6	24.1	24.6	25.1
100	25.6	26.1	26.6	27.2	27.7	28.2	28.8	29.3	29.9	30.4
110	31.0	31.5	32.1	32.7	33.3	33.9	34.4	35.0	35.6	36.3
120	36.9	37.5	38.1	38.7	39.4	40.0	40.6	41.3	41.9	42.6
130	43.3	43.9	44.6	45.3	46.0	46.7	47.3	48.0	48.8	49.5
140	50.2	50.9	51.6	52.3	53.1	53.8	54.6	55.3	56.1	56.8
150	57.6									

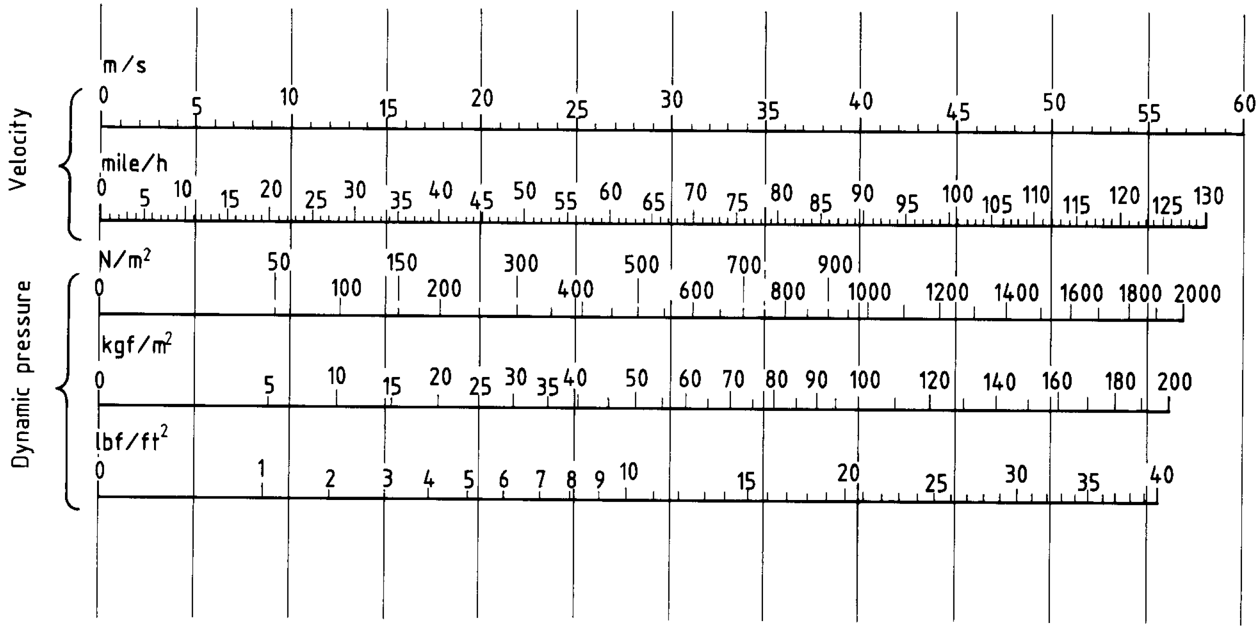


Figure 3 — Conversion chart for wind speed and dynamic pressure

7 Pressure coefficients and force coefficients

7.1 General. In the preceding clauses the method of assessing the dynamic pressure q is given. In order to determine the forces on a building or structure, or part thereof, the pressure has to be multiplied by a coefficient that is dependent on the shape of the building or structure and by the area of the building or structure, or part thereof.

There are two types of coefficient. These are as follows:

1) pressure coefficients, C_p;

2) force coefficients, C_f.

Values of these coefficients for some building shapes are given in Table 7 to Table 15. These tables may be used for other buildings of generally similar shape.

These coefficients are of necessity obtained mainly from measurements on models in wind tunnels, and the great majority of data available have been obtained in conditions of relatively smooth flow. Where sufficient field data exist as in the case of rectangular buildings, values have been adjusted to allow for turbulent flow.

7.2 Pressure coefficients. Pressure coefficients are always given for a particular surface or part of the surface of a building. The area of that surface or part of the surface when multiplied by the pressure coefficient and the dynamic pressure q gives the wind load acting in a direction normal to that particular surface or part thereof. The total wind load on a building can then be obtained by vectorial summation of the loads acting on each of the surfaces or parts of the surfaces of the building.

Average values of the pressure coefficients are given in the tables for critical wind directions in one or more quadrants. In order to determine the maximum wind load on the building the total load should be calculated for each of the critical directions shown from all quadrants.

Where considerable variation of pressure occurs over a surface it has been subdivided and mean pressure coefficients given for each of its several parts.

In addition, areas of high local suction frequently occurring near the edges of walls and roofs are separately shown. Coefficients for local effects should only be used to calculate the loads on these local areas. They should not be used for calculating the load on entire structural elements such as roof, walls or the structure as a whole.

NOTE For the design of cladding or its fixings to a structural member, Class A should be used with the pressure coefficient applicable to the particular area in which the cladding lies.

For the design of a structural member carrying the cladding, Class B or C should be used with the pressure coefficient applicable to the area in which the member lies. In considering the design against high local pressures of the structural member carrying the cladding, the secondary effects such as distribution due to the stiffness of the cladding should be taken into account.

For main structural members the design should be to Class B or C using the normal coefficients for the whole area.

When calculating the wind load on individual structural elements such as roofs and walls, and individual cladding units and their fixings, it is essential to take account of the pressure difference between opposite faces of such elements or units. For clad structures it is therefore necessary to know the internal pressure as well as the external pressure. The following distinguishing pressure coefficients are therefore used:

1) external pressure coefficient, C_{pe};

2) internal pressure coefficient, C_{pi}.

The load F acting in a direction normal to the individual structural element or cladding unit therefore is

$$F = (C_{pe} - C_{pi})qA^{4)}$$

where A is the surface area of the structural element or cladding unit. A negative value for F indicates that the resultant force is outwards.

Values of C_{pe} are given in Table 7 to Table 15 and a method of assessing the values of C_{pi} is given in Appendix E.

[4] The design wind speed and therefore q depends on S_2, which varies with height (see **5.5.2**).

7.3 Force coefficients. Force coefficients, when given, apply to a building or structure as a whole, and when multiplied by the effective frontal area A_e of the building or structure and by the dynamic pressure q give the total wind load on that particular building or structure thus:

$F = C_f q A_e$[5)]

where F is the force acting in a direction specified in the table and C_f is the force coefficient for the building.

It should be noted that the value of the force coefficient differs for the wind acting on different faces of a building or structure. In order to determine the critical load, the total wind load should be calculated for each wind direction.

If the wind load is calculated by dividing the area into parts, the value of C_f applied to each part should be that for the building as a whole.

7.4 Frictional drag. In certain buildings of special shape a force due to frictional drag should be taken in addition to those loads calculated from **7.2** and **7.3**. For rectangular clad buildings this addition is necessary only where the ratio $\frac{d}{h}$ or $\frac{d}{b}$ is greater than 4. The frictional drag F' in the direction of the wind is given by the following.

If $h \leqslant b$, $F' = C_{f'} q b(d - 4h) + C_{f'} q 2h(d - 4h)$, or

if $h \geqslant b$, $F' = C_{f'} q b(d - 4b) + C_{f'} q 2h(d - 4b)$.

The first term in each case gives the drag on the roof and the second the drag on the walls. The terms are given separately to allow for the use of different values of $C_{f'}$ and q on the different surfaces.

$C_{f'} = 0.01$ for smooth surfaces without corrugations or ribs across the wind direction.

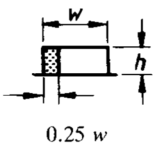
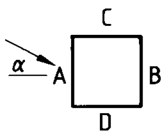
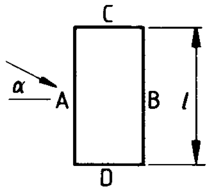
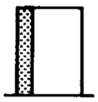
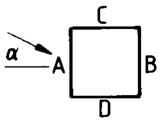
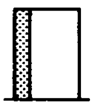
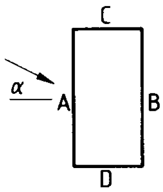
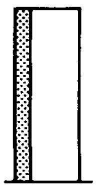
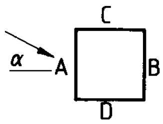
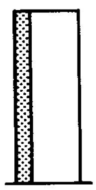
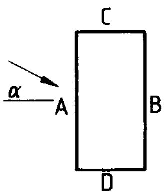
$C_{f'} = 0.02$ for surfaces with corrugations across the wind direction.

$C_{f'} = 0.04$ for surfaces with ribs across the wind direction.

For other buildings the frictional drag will be indicated, where necessary, in the tables of pressure coefficients and force coefficients.

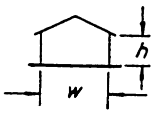
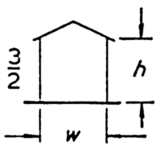
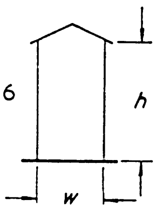
5) The design wind speed and therefore q depends on S_2, which varies with height (see **5.5.2**).

Table 7 — Pressure coefficients C_{pe} for the walls of rectangular clad buildings

Building height ratio	Building plan ratio	Elevation	Plan	Wind angle α	C_{pe} for surface A	C_{pe} for surface B	C_{pe} for surface C	C_{pe} for surface D	Local C_p (shaded)
				degrees					
$\frac{h}{w} \leqslant \frac{1}{2}$	$1 < \frac{l}{w} \leqslant \frac{3}{2}$			0 90	+ 0.7 – 0.5	– 0.2 – 0.5	– 0.5 + 0.7	– 0.5 – 0.2	– 0.8
	$\frac{3}{2} < \frac{l}{w} < 4$			0 90	+ 0.7 – 0.5	– 0.25 – 0.5	– 0.6 + 0.7	– 0.6 – 0.1	– 1.0
$\frac{1}{2} < \frac{h}{w} \leqslant \frac{3}{2}$	$1 < \frac{l}{w} \leqslant \frac{3}{2}$			0 90	+ 0.7 – 0.6	– 0.25 – 0.6	– 0.6 + 0.7	– 0.6 – 0.25	– 1.1
	$\frac{3}{2} < \frac{l}{w} < 4$			0 90	+ 0.7 – 0.5	– 0.3 – 0.5	– 0.7 + 0.7	– 0.7 – 0.1	– 1.1
$\frac{3}{2} < \frac{h}{w} < 6$	$1 < \frac{l}{w} \leqslant \frac{3}{2}$			0 90	+ 0.8 – 0.8	– 0.25 – 0.8	– 0.8 + 0.8	– 0.8 – 0.25	– 1.2
	$\frac{3}{2} < \frac{l}{w} < 4$			0 90	+ 0.7 – 0.5	– 0.4 – 0.5	– 0.7 + 0.8	– 0.7 – 0.1	– 1.2

NOTE *h* is the height to eaves or parapet, *l* is the greater horizontal dimension of a building and *w* is the lesser horizontal dimension of a building.

Table 8 — Pressure coefficients C_{pe} for pitch roofs of rectangular clad buildings

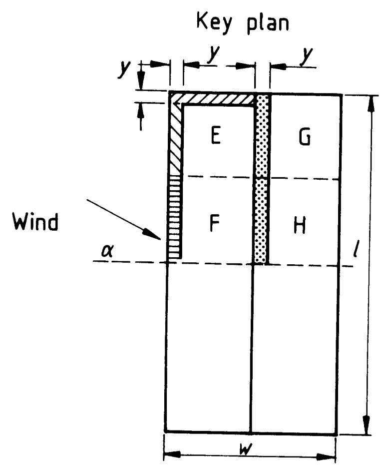
Building height ratio	Roof angle	Wind angle α 0° EF	Wind angle α 0° GH	Wind angle α 90° EG	Wind angle α 90° FH	Local coefficients (diagonal hatching)	Local coefficients (reverse diagonal hatching)	Local coefficients (horizontal hatching)	Local coefficients (dotted)
	degrees								
$\frac{h}{w} \leq \frac{1}{2}$ 	0	– 0.8	– 0.4	– 0.8	– 0.4	– 2.0	– 2.0	– 2.0	—
	5	– 0.9	– 0.4	– 0.8	– 0.4	– 1.4	– 1.2	– 1.2	– 1.0
	10	– 1.2	– 0.4	– 0.8	– 0.6	– 1.4	– 1.4		– 1.2
	20	– 0.4	– 0.4	– 0.7	– 0.6	+ 1.0			– 1.2
	30	0	– 0.4	– 0.7	– 0.6	– 0.8			– 1.1
	45	+ 0.3	– 0.5	– 0.7	– 0.6				– 1.1
	60	+ 0.7	– 0.6	– 0.7	– 0.6				– 1.1
$\frac{1}{2} < \frac{h}{w} \leq \frac{3}{2}$ 	0	– 0.8	– 0.6	– 1.0	– 0.6	– 2.0	– 2.0	– 2.0	—
	5	– 0.9	– 0.6	– 0.9	– 0.6	– 2.0	– 2.0	– 1.5	– 1.0
	10	– 1.1	– 0.6	– 0.8	– 0.6	– 2.0	– 2.0	– 1.5	– 1.2
	20	– 0.7	– 0.5	– 0.8	– 0.6	– 1.5	– 1.5	– 1.5	– 1.0
	30	– 0.2	– 0.5	– 0.8	– 0.8	– 1.0			– 1.0
	45	+ 0.2	– 0.5	– 0.8	– 0.8				
	60	+ 0.6	– 0.5	– 0.8	– 0.8				
$\frac{3}{2} < \frac{h}{w} < 6$ 	0	– 0.7	– 0.6	– 0.9	– 0.7	– 2.0	– 2.0	– 2.0	—
	5	– 0.7	– 0.6	– 0.8	– 0.8	– 2.0	– 2.0	– 1.5	– 1.0
	10	– 0.7	– 0.6	– 0.8	– 0.8	– 2.0	– 2.0	– 1.5	– 1.2
	20	– 0.8	– 0.6	– 0.8	– 0.8	– 1.5	– 1.5	– 1.5	– 1.2
	30	– 1.0	– 0.5	– 0.8	– 0.7	– 1.5			
	40	– 0.2	– 0.5	– 0.8	– 0.7	– 1.0			
	50	+ 0.2	– 0.5	– 0.8	– 0.7				
	60	+ 0.5	– 0.5	– 0.8	– 0.7				

NOTE 1 h is the height to eaves or parapet and w is the lesser horizontal dimension of a building.

NOTE 2 The pressure coefficient on the underside of any roof overhang should be taken as that on the adjoining wall surface.

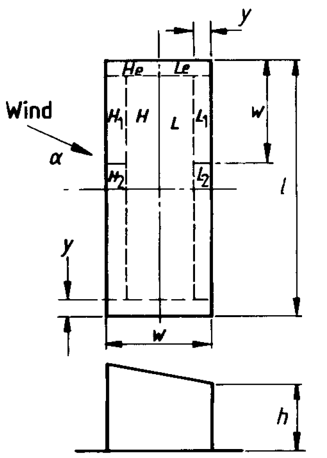
Where no local coefficients are given the overall coefficients apply.

NOTE 3 For hipped roofs the local coefficient for the hip ridge may be conservatively taken as the appropriate ridge value.



$y = h$ or $0.15w$, whichever is the lesser.

Table 9 — Pressure coefficients C_{pe} for monopitch roofs of rectangular clad buildings with h/w < 2



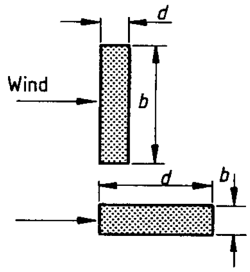
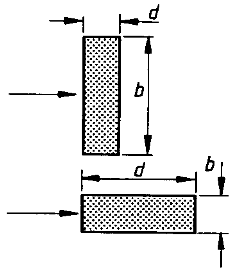
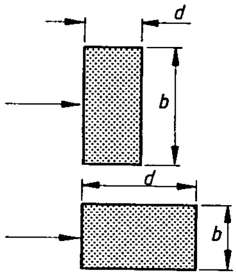
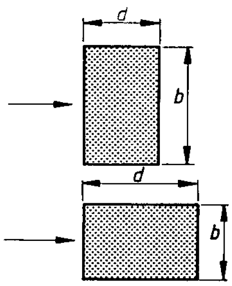
$y = h$ or $0.15\ w$, whichever is the lesser.

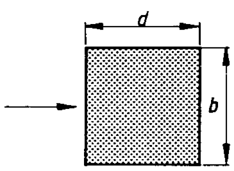
NOTE Area *H* and area *L* refer to the whole quadrant.

Roof angle	Wind angle α										Local C_{pe}					
	0°		45°		90°		135°		180°							
degree	*H*	*L*	*H*	*L*	*H* & *L*	*H* & *L*	*H* (Applies to length $w/2$ from windward end)	*L* (Applies to remainder)	*H*	*L*	H_1	H_2	L_1	L_2	H_e	L_e
5	– 1.0	– 0.5	– 1.0	– 0.9	– 1.0	– 0.5	– 0.9	– 1.0	– 0.5	– 1.0	– 2.0	– 1.5	– 2.0	– 1.5	– 2.0	– 2.0
10	– 1.0	– 0.5	– 1.0	– 0.8	– 1.0	– 0.5	– 0.8	– 1.0	– 0.4	– 1.0	– 2.0	– 1.5	– 2.0	– 1.5	– 2.0	– 2.0
15	– 0.9	– 0.5	– 1.0	– 0.7	– 1.0	– 0.5	– 0.6	– 1.0	– 0.3	– 1.0	– 1.8	– 0.9	– 1.8	– 1.4	– 2.0	– 2.0
20	– 0.8	– 0.5	– 1.0	– 0.6	– 0.9	– 0.5	– 0.5	– 1.0	– 0.2	– 1.0	– 1.8	– 0.8	– 1.8	– 1.4	– 2.0	– 2.0
25	– 0.7	– 0.5	– 1.0	– 0.6	– 0.8	– 0.5	– 0.3	– 0.9	– 0.1	– 0.9	– 1.8	– 0.7	– 0.9	– 0.9	– 2.0	– 2.0
30	– 0.5	– 0.5	– 1.0	– 0.6	– 0.8	– 0.5	– 0.1	– 0.6	0	– 0.6	– 1.8	– 0.5	– 0.5	– 0.5	– 2.0	– 2.0

NOTE *h* is the height to eaves at lower side, *l* is the greater horizontal dimension of a building and *w* is the lesser horizontal dimension of a building.

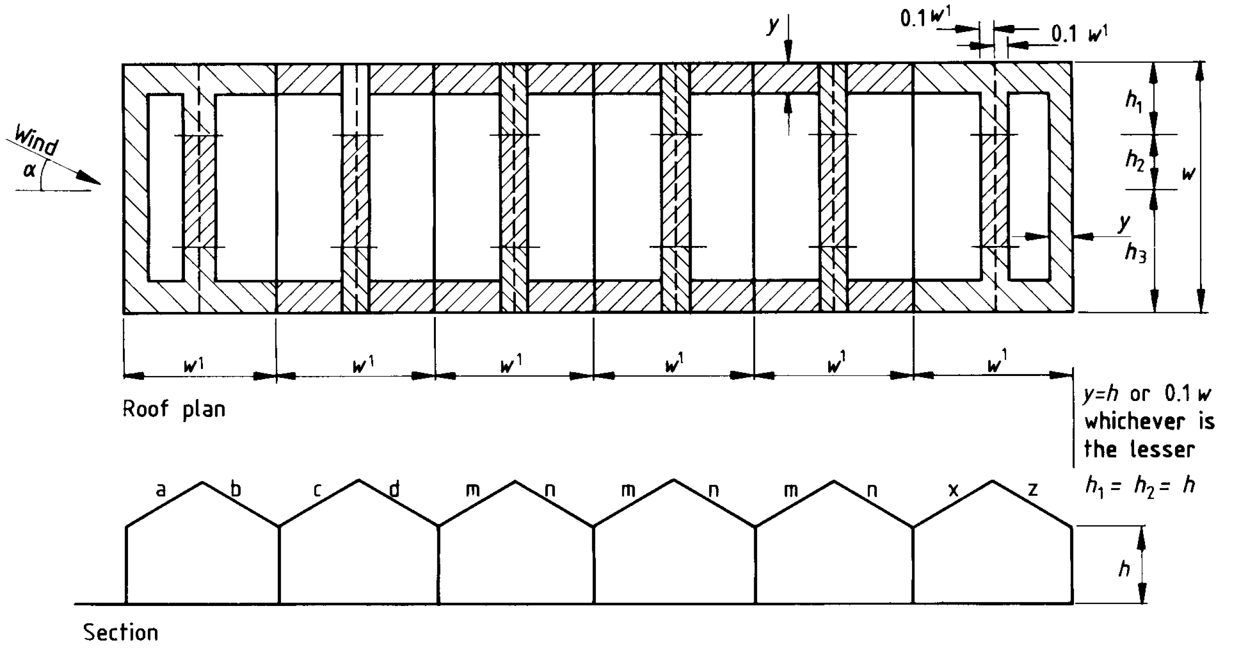
Table 10 — Force coefficients C_f for rectangular clad buildings with flat roofs (acting in the direction of the wind)

Plan shape	$\frac{l}{w}$	$\frac{b}{d}$	C_f for height/breadth ratio:				
			Up to ½	1	2	4	6
	⩾ 4	⩾ 4	1.2	1.3	1.4	1.5	1.6
		⩽ ¼	0.7	0.7	0.75	0.75	0.75
	3	3	1.1	1.2	1.25	1.35	1.4
		⅓	0.7	0.75	0.75	0.75	0.8
	2	2	1.0	1.05	1.1	1.15	1.2
		½	0.75	0.75	0.8	0.85	0.9
	1½	1½	0.95	1.0	1.05	1.1	1.15
		⅔	0.8	0.85	0.9	0.95	1.0

Plan shape	$\frac{l}{w}$	$\frac{b}{d}$	C_f for height/breadth ratio:						
			Up to ½	1	2	4	6	10	20
	1	1	0.9	0.95	1.0	1.05	1.1	1.2	1.4

NOTE *b* is the dimension of the building normal to the wind, *d* is the dimension of the building measured in the direction of the wind, *l* is the greater horizontal dimension of a building and *w* is the lesser horizontal dimension of a building.

Table 11 — Pressure coefficients C_{pe} for pitch roofs of multi-span buildings (all spans equal) with $h \ngtr w'$



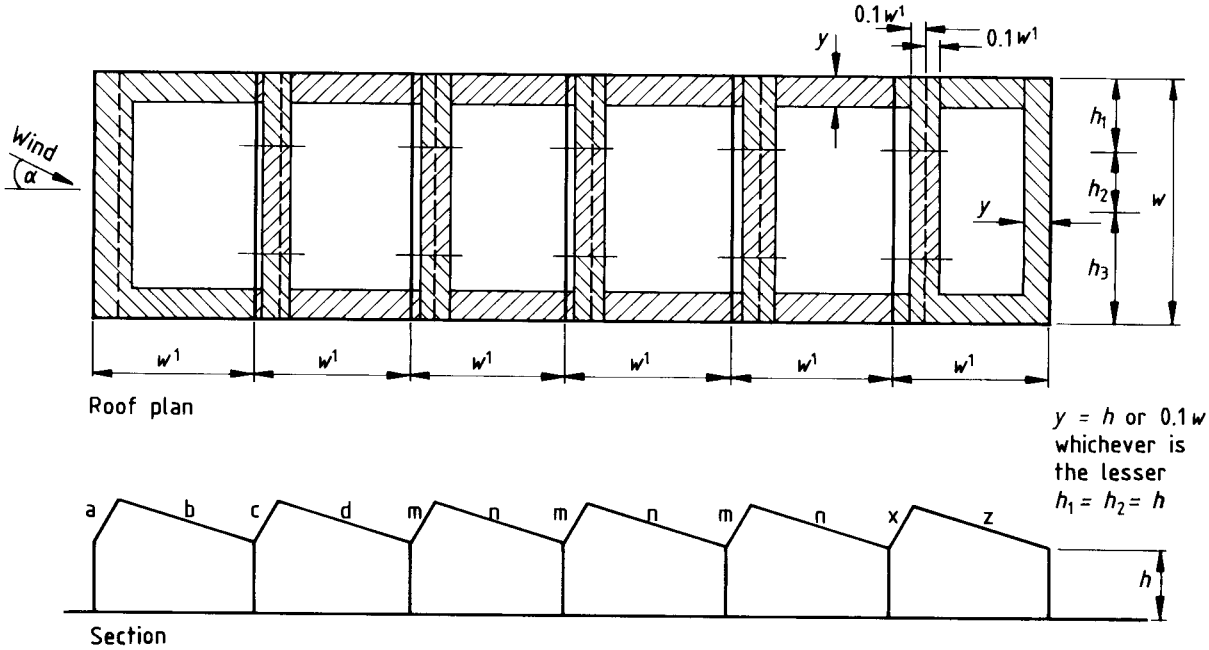
Roof angle	Wind angle α	First span		First intermediate span		Other intermediate span		End span		Local coefficient	
		a	b	c	d	m	n	x	z	(cross-hatched)	(hatched)
degrees	degrees										
5	0	– 0.9	– 0.6	– 0.4	– 0.3	– 0.3	– 0.3	– 0.3	– 0.3		
10		– 1.1	– 0.6	– 0.4	– 0.3	– 0.3	– 0.3	– 0.3	– 0.4		
20		– 0.7	– 0.6	– 0.4	– 0.3	– 0.3	– 0.3	– 0.3	– 0.5	– 2.0	– 1.5
30		– 0.2	– 0.6	– 0.4	– 0.3	– 0.2	– 0.3	– 0.2	– 0.5		
45		+ 0.3	– 0.6	– 0.6	– 0.4	– 0.2	– 0.4	– 0.2	– 0.5		

Roof angle	Wind angle α	Distance h_1	h_2	h_3
degrees	degrees			
Up to 45	90	– 0.8	– 0.6	– 0.2

Frictional drag: when wind angle α = 0° horizontal forces due to frictional drag are allowed for in the above values;
when wind angle α = 90° allow for friction drag in accordance with **7.4**.

NOTE Evidence on these buildings is fragmentary and any departures from the cases given should be investigated separately.

Table 12 — Pressure coefficients C_{pe} for saw-tooth roofs of multi-span buildings (all spans equal) with $h \not> w'$



Wind angle α	First span		First intermediate span		Other intermediate span		End span		Local coefficient	
	a	b	c	d	m	n	x	z		
degrees										
0	+ 0.6	– 0.7	– 0.7	– 0.4	– 0.3	– 0.2	– 0.1	– 0.3	– 2.0	– 1.5
180	– 0.5	– 0.3	– 0.3	– 0.3	– 0.4	– 0.6	– 0.6	– 0.1		

Wind angle α	Distance h_1	h_2	h_3
degrees			
90	– 0.8	– 0.6	– 0.2
270	Similarly, but handed		

Frictional drag: when wind angle α = 0° horizontal forces due to frictional drag are allowed for in the above values;
when wind angle α = 90° allow for frictional drag in accordance with **7.4**.

NOTE Evidence on these buildings is fragmentary and any departures from the cases given should be investigated separately.

Table 13 — Pressure coefficients C_p for canopy roofs with 1/4 < h/w < 1 and 1 < L/w < 3

The coefficients take account of the combined effect of the wind on both upper and lower surfaces of the canopy for all wind directions. Where the local coefficient areas overlap, the greater of the two given values should be taken.

The solidity ratio ϕ is equal to the area of obstructions under the canopy divided by the gross area under the canopy, both areas normal to the wind direction, $\phi = 0$ represents a canopy with no obstructions underneath, $\phi = 1$ represents the canopy fully blocked with contents to the downwind eaves. Values of C_p for intermediate solidities may be linearly interpolated between these two extremes, and apply upwind of the position of maximum blockage only. Downwind of the position of maximum blockage the coefficients for $\phi = 0$ may be used.

Canopies should be able to resist the maximum (largest + ve) and the minimum (largest – ve) pressures, the latter depending on the degree of blockage under the canopy. Duopitch canopies should additionally be able to support forces with one slope at the maximum or minimum and the other slope unloaded.

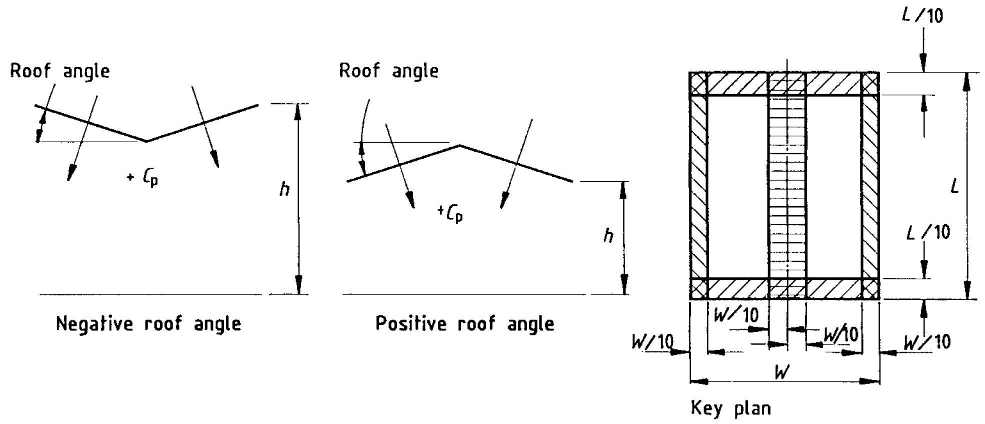
In addition to the pressures normal to the canopy, there will be horizontal loads on the canopy due to the wind pressure on any fascia at the eaves or on any gable between eaves and ridge on duopitch canopies, or to friction forces acting on top and bottom surfaces of the canopies. For any wind direction, only the greater of these two forces need to be taken into account. Fascia loads should be calculated on the area of the surface facing the wind using a pressure coefficient of 1.3 on the windward fascia/gable and 0.6 on the leeward fascia/gable acting in the direction of the wind. Frictional drag F should be calculated from

$$F = 0.025\, A_s q$$

where A_s is either the combined area of the top and bottom surfaces of an empty canopy or the area of the top surface only for a fully blocked canopy. Values of A_s for intermediate solidities may be linearly interpolated between these two extremes.

Table 13 — Pressure coefficients C_p for canopy roofs with 1/4 < h/w < 1 and 1 < L/w < 3

(a) Duopitch canopies

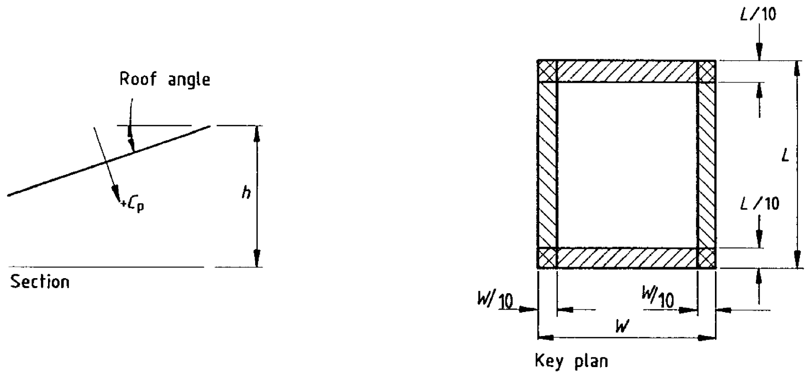


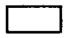
	Roof angle (degrees)	Solidity ratio	Maximum and minimum pressure coefficients: Overall coefficients	Local coefficients			
Maximum (largest + ve) pressures	– 20	All values of Φ	+ 0.7	+ 0.8	+ 1.6	+ 0.6	+ 1.7
	– 15		+ 0.5	+ 0.6	+ 1.5	+ 0.7	+ 1.4
	– 10		+ 0.4	+ 0.6	+ 1.4	+ 0.8	+ 1.1
	– 5		+ 0.3	+ 0.5	+ 1.5	+ 0.8	+ 0.8
	+ 5		+ 0.3	+ 0.6	+ 1.8	+ 1.3	+ 0.4
	+ 10		+ 0.4	+ 0.7	+ 1.8	+ 1.4	+ 0.4
	+ 15		+ 0.4	+ 0.9	+ 1.9	+ 1.4	+ 0.4
	+ 20		+ 0.6	+ 1.1	+ 1.9	+ 1.5	+ 0.4
	+ 25		+ 0.7	+ 1.2	+ 1.9	+ 1.6	+ 0.5
	+ 30		+ 0.9	+ 1.3	+ 1.9	+ 1.6	+ 0.7
Minimum (largest – ve) pressures	– 20	$\Phi = 0$	– 0.7	– 0.9	– 1.3	– 1.6	– 0.6
		$\Phi = 1$	– 1.3	– 1.5	– 2.4	– 2.4	– 0.6
	– 15	$\Phi = 0$	– 0.6	– 0.8	– 1.3	– 1.6	– 0.6
		$\Phi = 1$	– 1.4	– 1.6	– 2.7	– 2.6	– 0.6
	– 10	$\Phi = 0$	– 0.6	– 0.8	– 1.3	– 1.5	– 0.6
		$\Phi = 1$	– 1.4	– 1.6	– 2.7	– 2.6	– 0.6
	– 5	$\Phi = 0$	– 0.5	– 0.7	– 1.3	– 1.6	– 0.6
		$\Phi = 1$	– 1.3	– 1.5	– 2.4	– 2.4	– 0.6
	+ 5	$\Phi = 0$	– 0.6	– 0.6	– 1.4	– 1.4	– 1.1
		$\Phi = 1$	– 1.3	– 1.3	– 2.0	– 1.8	– 1.5
	+ 10	$\Phi = 0$	– 0.7	– 0.7	– 1.5	– 1.4	– 1.4
		$\Phi = 1$	– 1.3	– 1.3	– 2.0	– 1.8	– 1.8
	+ 15	$\Phi = 0$	– 0.8	– 0.9	– 1.7	– 1.4	– 1.8
		$\Phi = 1$	– 1.3	– 1.3	– 2.2	– 1.6	– 2.1
	+ 20	$\Phi = 0$	– 0.9	– 1.2	– 1.8	– 1.4	– 2.0
		$\Phi = 1$	– 1.3	– 1.4	– 2.2	– 1.6	– 2.1
	+ 25	$\Phi = 0$	– 1.0	– 1.4	– 1.9	– 1.4	– 2.0
		$\Phi = 1$	– 1.3	– 1.4	– 2.0	– 1.5	– 2.0
	+ 30	$\Phi = 0$	– 1.0	– 1.4	– 1.9	– 1.4	– 2.0
		$\Phi = 1$	– 1.3	– 1.4	– 1.8	– 1.4	– 2.0

For duopitch canopies the centre of pressure should be taken to act at the centre of each slope.

Table 13 — Pressure coefficients C_p for canopy roofs with $1/4 < h/w < 1$ and $1 < L/w < 3$

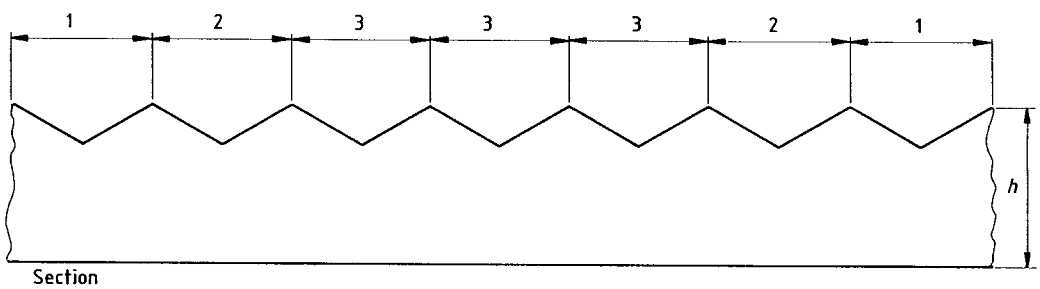
(b) Monopitch canopies



	Roof angle (degrees)	Solidarity ratio	Maximum and minimum pressure coefficients			
			Overall coefficients	Local coefficients		
						
Maximum (largest + ve) pressures	0 5 10 15 20 25 30	All values of Φ	+ 0.2 + 0.4 + 0.5 + 0.7 + 0.8 + 1.0 + 1.2	+ 0.5 + 0.8 + 1.2 + 1.4 + 1.7 + 2.0 + 2.2	+ 1.8 + 2.1 + 2.4 + 2.7 + 2.9 + 3.1 + 3.2	+ 1.1 + 1.3 + 1.6 + 1.8 + 2.1 + 2.3 + 2.4
Minimum (largest + ve) pressure	0	$\Phi = 0$ $\Phi = 1$	– 0.5 – 1.3	– 0.6 – 1.5	– 1.3 – 1.8	– 1.4 – 2.2
	5	$\Phi = 0$ $\Phi = 1$	– 0.7 – 1.4	– 1.1 – 1.6	– 1.7 – 2.2	– 1.8 – 2.5
	10	$\Phi = 0$ $\Phi = 1$	– 0.9 – 1.4	– 1.5 – 2.1	– 2.0 – 2.6	– 2.1 – 2.7
	15	$\Phi = 0$ $\Phi = 1$	– 1.1 – 1.4	– 1.8 – 1.6	– 2.4 – 2.9	– 2.5 – 3.0
	20	$\Phi = 0$ $\Phi = 1$	– 1.3 – 1.4	– 2.2 – 1.6	– 2.8 – 2.9	– 2.9 – 3.0
	25	$\Phi = 0$ $\Phi = 1$	– 1.6 – 1.4	– 2.6 – 1.5	– 3.2 – 2.5	– 3.2 – 2.8
	30	$\Phi = 0$ $\Phi = 1$	– 1.8 – 1.4	– 3.0 – 1.5	– 3.8 – 2.2	– 3.6 – 2.7

For monopitch canopies the centre of pressure should be taken to act at 0.25 w from the windward edge

(c) Multibay canopy



Loads on each slope of multibay canopies are determined by applying the following factors to the overall coefficients for isolated duo pitch canopies.

Bay	Location	Factors for all Φ	
		On maximum overall coefficient	On minimum overall coefficient
1	end bay	1.00	0.81
2	second bay	0.87	0.64
3	third and subsequent bays	0.68	0.63

Table 14 — Force coefficients C_f for clad buildings of uniform section (acting in the direction of the wind)

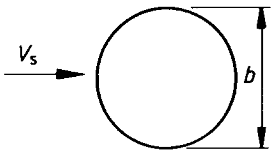
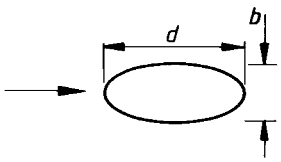
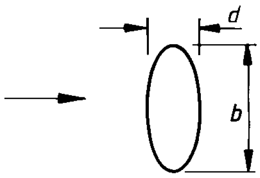
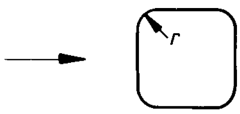
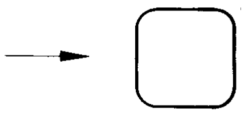
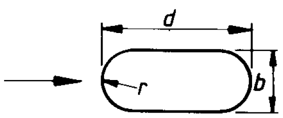
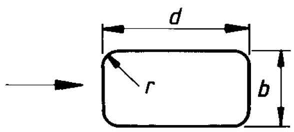
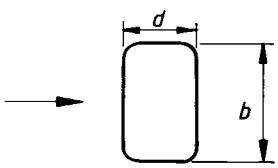
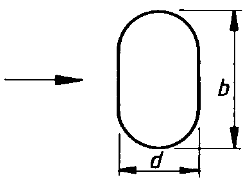
Plan shape		$V_s b$	C_f for height/breadth ratio: Up to ½	1	2	5	10	20	∞
	All surfaces	m²/s < 6	0.7	0.7	0.7	0.8	0.9	1.0	1.2
	Rough or with projections	⩾ 6							
See also Appendix G	Smooth	⩾ 6	0.5	0.5	0.5	0.5	0.5	0.6	0.6
	Ellipse $b/d = 1/2$	< 10	0.5	0.5	0.5	0.5	0.6	0.6	0.7
		⩾ 10	0.2	0.2	0.2	0.2	0.2	0.2	0.2
	Ellipse $b/d = 2$	< 8	0.8	0.8	0.9	1.0	1.1	1.3	1.7
		⩾ 8	0.8	0.8	0.9	1.0	1.1	1.3	1.5
	$b/d = 1$ $r/b = 1/3$	< 4	0.6	0.6	0.6	0.7	0.8	0.8	1.0
		⩾ 4	0.4	0.4	0.4	0.4	0.5	0.5	0.5
	$b/d = 1$ $r/b = 1/6$	< 10	0.7	0.8	0.8	0.9	1.0	1.0	1.3
		⩾ 10	0.5	0.5	0.5	0.5	0.6	0.6	0.6
	$b/d = 1/2$ $r/b = 1/2$	< 3	0.3	0.3	0.3	0.3	0.3	0.3	0.4
		⩾ 3	0.2	0.2	0.2	0.2	0.3	0.3	0.3
	$b/d = 1/2$ $r/b = 1/6$	All values	0.5	0.5	0.5	0.5	0.6	0.6	0.7
	$b/d = 2$ $r/b = 1/12$	All values	0.9	0.9	1.0	1.1	1.2	1.5	1.9
	$b/d = 2$ $r/b = 1/4$	< 6	0.7	0.8	0.8	0.9	1.0	1.2	1.6
		⩾ 6	0.5	0.5	0.5	0.5	0.5	0.6	0.6

Table 14 — Force coefficients C_f for clad buildings of uniform section (acting in the direction of the wind)

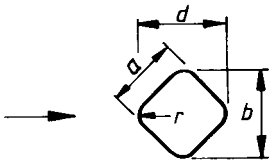
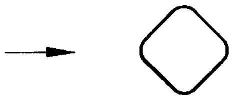
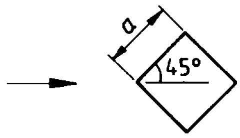
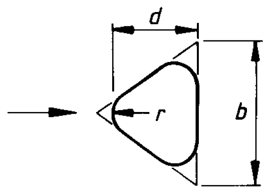
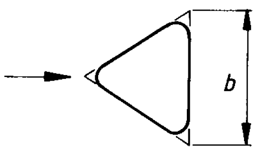
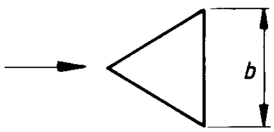
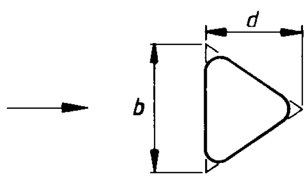
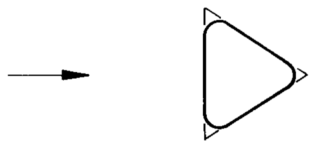
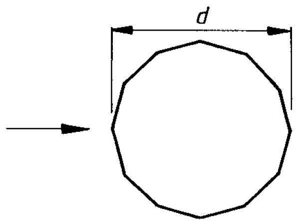
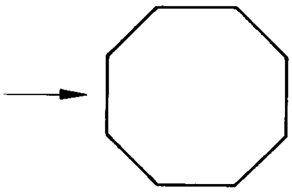
Plan shape		$V_s b$	C_f for height/breadth ratio: Up to ½	1	2	5	10	20	∞
	r/a = 1/3	m^2/s < 10	0.8	0.8	0.9	1.0	1.1	1.3	1.5
		⩾ 10	0.5	0.5	0.5	0.5	0.5	0.6	0.6
	r/a = 1/12	All values	0.9	0.9	0.9	1.1	1.2	1.3	1.6
	r/a = 1/48	All values	0.9	0.9	0.9	1.1	1.2	1.3	1.6
	r/b = 1/4	< 11	0.7	0.7	0.7	0.8	0.9	1.0	1.2
		⩾ 11	0.4	0.4	0.4	0.4	0.5	0.5	0.5
	r/b = 1/12	All values	0.8	0.8	0.8	1.0	1.1	1.2	1.4
	r/b = 1/48	All values	0.7	0.7	0.8	0.9	1.0	1.1	1.3
	r/b = 1/4	< 8	0.7	0.7	0.8	0.9	1.0	1.1	1.3
		⩽ 8	0.4	0.4	0.4	0.4	0.5	0.5	0.5
	1/48 < r/b < 1/12	All values	1.2	1.2	1.2	1.4	1.6	1.7	2.1
	12-sided polygon	< 12	0.7	0.7	0.8	0.9	1.0	1.1	1.3
		⩾ 12	0.7	0.7	0.7	0.7	0.8	0.9	1.1

Table 14 — Force coefficients C_f for clad buildings of uniform section (acting in the direction of the wind)

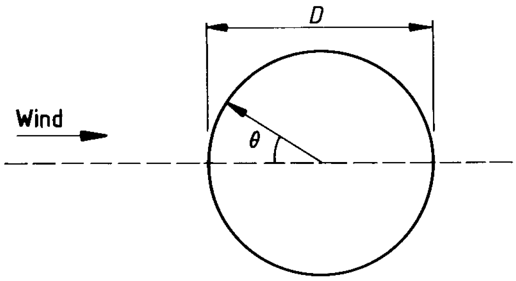
Plan shape		$V_s b$	C_f for height/breadth ratio:						
			Up to ½	1	2	5	10	20	∞
		m²/s							
	Octagon	All values	1.0	1.0	1.1	1.2	1.2	1.3	1.4

Where strakes are used, *b* may be taken as the breadth over the strakes. Structures that, because of their size and the design wind velocity, are in the supercritical flow regime may need further calculation to ensure that the greatest loads do not occur at some wind speed below the maximum when the flow will be subcritical.

The coefficients are for buildings without projections, except where otherwise shown.

In this table $V_s b$ is used as an indication of the airflow regime.

Table 15 — Pressure distribution around cylindrical structures



Position on periphery θ	Pressure coefficient C_{pe}			
	Surface: rough or with projections		Surface: smooth	
degrees	$h/D = 10$	$h/D \ngtr 2.5$	$h/D = 10$	$h/D \ngtr 2.5$
0	+ 1.0	+ 1.0	+ 1.0	+ 1.0
10	+ 0.9	+ 0.9	+ 0.9	+ 0.9
20	+ 0.7	+ 0.7	+ 0.7	+ 0.7
30	+ 0.4	+ 0.4	+ 0.35	+ 0.35
40	0	0	0	0
50	– 0.5	– 0.4	– 0.7	– 0.5
60	– 0.95	– 0.8	– 1.2	– 1.05
70	– 1.25	– 1.1	– 1.4	– 1.25
80	– 1.2	– 1.05	– 1.45	– 1.3
90	– 1.0	– 0.85	– 1.4	– 1.2
100	– 0.8	– 0.65	– 1.1	– 0.85
120	– 0.5	– 0.35	– 0.6	– 0.4
140	– 0.4	– 0.3	– 0.35	– 0.25
160	– 0.4	– 0.3	– 0.35	– 0.25
180	– 0.4	– 0.3	– 0.35	– 0.25

For the purpose of calculating the wind forces that act in such a way as to deform a cylindrical structure the values of C_{pe} in Table 15 may be used. They apply only in supercritical flow (i.e. they should only be used where $D > 0.3$ m). They may be used for wind blowing normal to the axis of cylinders having their axis normal to the ground plane (i.e. chimneys, silos) and to cylinders having their axis parallel with the ground plane (i.e. horizontal tanks) provided that the clearance between the tank and the ground is not less than *D*.

h is the height of a vertical cylinder or length of a horizontal cylinder. Where there is a free flow of air around both ends, *h* is to be taken as half the length when calculating h/D. Interpolation may be used for intermediate values of h/D.

In the calculation of the load on the periphery of the cylinder, the value of C_{pi} shall be taken into account.

For open ended cylinders where $h/D \geqslant 0.3$ C_{pi} may be taken as – 0.8.

For open ended cylinders where $h/D < 0.3$ C_{pi} may be taken as – 0.5.

8 Force coefficients for unclad structures

8.1 General. This clause applies to permanently unclad structures and structural frameworks while temporarily unclad.

Structures that, because of their size and the design wind velocity, are in the supercritical flow regime may need further calculation to ensure that the greatest loads do not occur at some wind speed below the maximum when the flow will be subcritical.

8.1.1 *Icing.* Icing will affect the shape and loading on the structure, but at present it is thought unlikely to occur with extreme winds. Such information as is at present known is given in Appendix F.

8.2 Force coefficients of individual members. The coefficients refer to members of infinite length. For members of finite length the coefficients should be multiplied by a factor *K* that depends on the ratio $l/j\alpha$, where *l* is the length of the member and $j\alpha$ is the width across the direction of the wind. Values of *K* are given in Table 16.

Where any member abuts onto a plate or wall in such a way that free flow of air around that end of the member is prevented, the ratio $l/j\alpha$ should be doubled for the purpose of determining *K*. When both ends of a member are so obstructed, the ratio should be taken as infinity.

Table 16 — Values of reduction factor *K* for members of finite length and slenderness

l/j_{α} or l/D	2	5	10	20	40	50	100	∞
Circular cylinder, subcritical flow	0.58	0.62	0.68	0.74	0.82	0.87	0.98	1.0
Circular cylinder supercritical flow	0.80	0.80	0.82	0.90	0.98	0.99	1.0	1.0
Flat plate perpendicular to wind	0.62	0.66	0.69	0.81	0.87	0.90	0.95	1.0

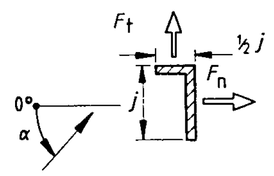
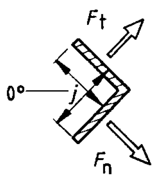
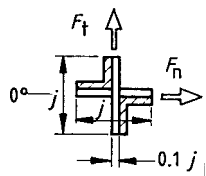
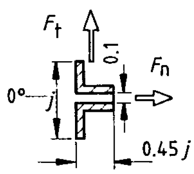
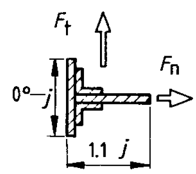
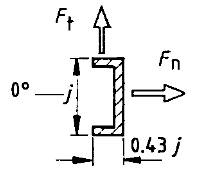
8.2.1 *Flat-sided members.* The force coefficients in Table 17 are given for two mutually-perpendicular directions relative to a reference axis on the structural member. They are designated C_{fn} and C_{ft} and give the forces normal and transverse, respectively, to the reference plane as will be apparent from the diagrams.

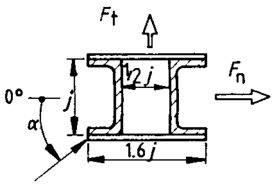
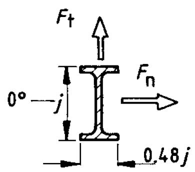
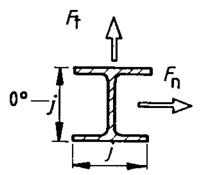
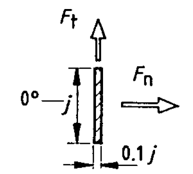
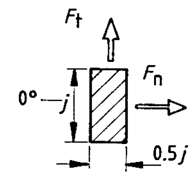
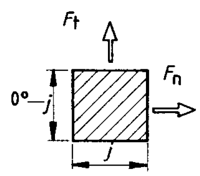
Force coefficients are for wind normal to the longitudinal axis of the member.

Normal force $F_n = C_{fn}\, qKlj$.

Transverse force $F_t = C_{ft}\, qKlj$.

Table 17 — Force coefficients C_{fn} and C_{ft} for individual structural members (flat sides) of infinite length

												
α	C_{fn}	C_{ft}	C_{fn}	C_{ft}	C_{fn}	C_{ft}	C_{fn}	C_{ft}	C_{fn}	C_{ft}	C_{fn}	C_{ft}
degrees												
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_{fn}	C_{ft}	C_{fn}	C_{ft}	C_{fn}	C_{ft}	C_{fn}	C_{ft}	C_{fn}	C_{ft}	C_{fn}	C_{ft}
degrees												
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

NOTE In this table the force coefficient C_f is given in relation to the dimension *j* and not, as in other cases, in relation to the effective frontal area A_e.

8.2.2 *Circular sections*. For circular sections, the force coefficients C_f, which are dependent upon values of DV_s, are given in Table 18. The values of C_f given in this table are suitable for all surfaces of evenly distributed roughness of height less than 1/100 diameter, that is, for all normal surface finishes, and for members of infinite length.

Force, $F = C_f\, qKlD$.

Table 18 — Force coefficients C_f for individual structural members of circular section and infinite length

Flow regime		Force coefficient C_f
Subcritical flow	$DV_s < 6$ m²/s (45 ft mile/h) $Re < 4.1 \times 10^5$	1.2
Supercritical flow	$6 \leqslant DV_s < 12$ m²/s $45 \leqslant DV_s < 90$ ft mile/h $4.1 \times 10^5 \leqslant Re < 8.2 \times 10^5$	0.6
	$12 \leqslant DV_s < 33$ m²/s $90 \leqslant DV_s < 240$ ft mile/h $8.2 \times 10^5 \leqslant Re < 22.6 \times 10^5$	0.7
	$DV_s \geqslant 33$ m²/s (240 ft mile/h) $Re \geqslant 22.6 \times 10^5$	0.8

A description of supercritical flow and Reynolds number (*Re* in above table) is given in Appendix G.

8.2.3 *Wires and cables*. For wires and cables the force coefficients C_f that are dependent upon values of DV_s are given in Table 19.

Table 19 — Force coefficients C_f for wires and cables ($l/D > 100$)

Flow regime	Force coefficient C_f for:			
	smooth surface wire	moderately smooth wire (galvanized or painted)	fine stranded cables	thick stranded cables
$DV_s < 0.6$ m²/s (4.5 ft mile/h)	—	—	1.2	1.3
$DV_s \geqslant 0.6$ m²/s (4.5 ft mile/h)	—	—	0.9	1.1
$DV_s < 6$ m²/s (4.5 ft mile/h)	1.2	1.2	—	—
$DV_s \geqslant 0.6$ m²/s (4.5 ft mile/h)	0.5	0.7	—	—

8.3 Single frames. Since the wind can come from any direction the most unfavourable load condition should be taken. In general, the wind load on a single frame should be calculated for the condition where the wind is at right angles to the frame unless it can be shown that another wind angle is appropriate.

The wind load acting on a single frame should be taken as

$$F = C_f q A_e$$

where A_e is the effective area of the frame normal to the wind direction;

q is the dynamic pressure of the wind;

C_f is the effective force coefficient.

The force coefficients for a single frame consisting of (a) flat-sided members or (b) circular section members in which all the members of the frame have DV_s values less than 6 m²/s or all members have DV_s values greater than 6 m²/s are given in Table 20.

Table 20 — Effective force coefficients C_f for single frames

Solidity ratio ϕ	Force coefficient C_f for:		
	flat-sided members	circular sections	
		subcritical flow $DV_s < 6$ m²/s (45 ft mile/h)	supercritical flow $DV_s \geqslant 6$ m²/s (45 ft mile/h)
0.1	1.9	1.2	0.7
0.2	1.8	1.2	0.8
0.3	1.7	1.2	0.8
0.4	1.7	1.1	0.8
0.5	1.6	1.1	0.8
0.75	1.6	1.5	1.4
1.0	2.0	2.0	2.0

The solidity ratio ϕ is equal to the effective area of a frame normal to the wind direction divided by the area enclosed by the boundary of the frame normal to the wind direction.

When single frames are composed of circular section members it is possible that the larger members will be in the supercritical flow regime (i.e. $DV_s \geqslant 6$ m²/s) and the smaller members will not (i.e. $DV_s < 6$ m²/s), there may also be some details fabricated from flat-sided sections.

In this situation the wind force acting on the frame should be calculated using an effective force coefficient equal to

$$Z.C_f(\text{super}) + (1 - Z)\frac{A(\text{circ. sub})}{\text{A(sub)}}.C_f(\text{sub}) + (1 - Z)\frac{A(\text{flat})}{A(\text{sub})}.C_f(\text{flat})$$

where C_f(super) is the force coefficient for single frames comprised of the subcritical circular sections from Table 20.

C_f(sub) is the force coefficient for single frames comprised of the subcritical circular sections from Table 20.

C_f(flat) is the force coefficient for single frames comprised of the flat-sided members from Table 20.

C_f(flat) is the force coefficient of the flat-sided members from Table 17;

A(circ. sub) is the effective area of the subcritical circular sections;

A(flat) is the effective area of the flat-sided members;

A(sub) = A(circ. sub) + A(flat);

$$Z = \frac{\text{Area of the frame in a supercritical flow}}{A_e}$$

8.4 Multiple frame structures. This section applies to structures having two or more parallel frames where the windward frame may have a shielding effect upon the frames to leeward. The windward frame and any unsheltered parts of other frames should be calculated as in **8.3**, but the wind load on the parts of frames that are sheltered should be multiplied by a shielding factor η, which is dependent upon the solidity ratio of the windward frame, the type of member comprising the frame and the spacing ratio of the frames. The values of the shielding factor are given in Table 21.

Where there are more than two frames of similar geometry and spacing, the wind load on the third and subsequent frames should be taken as equal to that on the second frame.

The loads on the various frames should be added together to obtain the total load on the structure.

Table 21 — Shielding factor η

Spacing ratio	Value of η for an *aerodynamic* solidity ratio β, of							
	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8 and over
up to 1.0	1.0	0.96	0.90	0.80	0.68	0.54	0.44	0.37
2.0	1.0	0.97	0.91	0.82	0.71	0.58	0.49	0.43
3.0	1.0	0.97	0.92	0.84	0.74	0.63	0.54	0.48
4.0	1.0	0.98	0.93	0.86	0.77	0.67	0.59	0.54
5.0	1.0	0.98	0.94	0.88	0.80	0.71	0.64	0.60
6.0 and over	1.0	0.99	0.95	0.90	0.83	0.75	0.69	0.66

The spacing ratio is equal to the distance, centre to centre, of the frames, beams or girders divided by the least overall dimension of the frame, beam or girder measured at right angles to the direction of the wind. For triangular framed structures or rectangular framed structures diagonal to the wind the spacing ratio should be calculated from the mean distance between the frames in the direction of the wind.

The aerodynamic solidity ratio used in Table 21 enables all cross sections of single members to be incorporated.

Aerodynamic solidity ratio β = solidity ratio (ϕ) x a constant

where the constant = 1.6 for flat-sided members;

= 1.2 for circular sections in the subcritical range and for flat-sided members in conjunction with such circular sections;

= 0.5 for circular sections in the supercritical range and for flat-sided members in conjunction with such circular sections.

8.5 Lattice towers

8.5.1 Lattice towers of square and equilateral triangular section constitute special cases for which it may be convenient to use an overall force coefficient in the calculation of wind load. The wind load should, for convenience, be calculated for the condition when the wind blows against any face.

The wind load F acting in the direction of the wind should be taken as

$F = C_f q A_e$

where A_e is the effective area of the face (see **8.3**);

q is the dynamic pressure of the wind (see **6**); and

C_f is the overall force coefficient (see Table 22, Table 23 and Table 24).

8.5.2 For towers with flat-sided members, the values of the overall force coefficient are given in Table 22.

Table 22 — Overall force coefficient C_f for towers composed of flat-sided members

Solidity ratio ϕ	Force coefficient C_f for:	
	square towers	equilateral triangular towers
0.1	3.8	3.1
0.2	3.3	2.7
0.3	2.8	2.3
0.4	2.3	1.9
0.5	2.1	1.5

For square lattice towers the maximum load occurs when the wind blows onto a corner. It may be taken as 1.2 times the load for the face-on wind.

For triangular lattice towers the wind load may be assumed to be constant for any inclination of the wind to face.

8.5.3 Since it is only in very few cases with lattice towers composed of members of circular cross section that all the members of a lattice tower are in the same flow regime, i.e. either subcritical or supercritical, wind force calculations should be carried out as described in **8.3** for single frames, due account being taken of the shielding factors given in **8.4**.

When it can be shown that all members of the tower are wholly in the same flow regime, the overall force coefficients C_f given in Table 23 and Table 24 may be used. These tables are based on actual measurements and give somewhat lower values than would be obtained using Table 20 and Table 21.

Table 23 — Overall force coefficient C_f for square towers composed of rounded members

Solidity ratio of front face ϕ	Force coefficient C_f for:			
	subcritical flow $DV_s < 6$ m²/s (45 ft mile/h)		supercritical flow $DV_s \geqslant 6$ m²/s (45 ft mile/h)	
	onto face	onto corner	onto face	onto corner
5	2.4	2.5	1.1	1.2
0.1	2.2	2.3	1.2	1.3
0.2	1.9	2.1	1.3	1.6
0.3	1.7	1.9	1.4	1.6
0.4	1.6	1.9	1.4	1.6
0.5	1.4	1.9	1.4	1.6

Table 24 — Overall force coefficient C_f for equilateral triangular towers composed of rounded members

Solidity ratio of front face ϕ	Force coefficient C_f for:	
	subcritical flow $DV_s < 6$ m²/s (45 ft mile/h)	supercritical flow $DV_s \geqslant 6$ m²/s (45 ft mile/h)
	all wind directions	all wind directions
0.05	1.8	0.8
0.1	1.7	0.8
0.2	1.6	1.1
0.3	1.5	1.1
0.4	1.5	1.1
0.5	1.4	1.2

Appendix A Ground roughness, building size and height above ground: the basis of the S_2 factors

A.1 Ground roughness — the four categories

Near the ground the wind encounters various obstacles in its path and the gustiness of the wind depends on the size, frequency and geometrical arrangement of these obstacles. If the obstructions are large and frequent the surface over which the wind flows is said to be rough; if the ground surface itself is level and there are no obstructions then the surface is said to be smooth. A rough surface will produce a wind flow with much gustiness whereas a smooth surface will not add appreciably to the gustiness already developed in the lower layers of the atmosphere. The gustiness can be defined numerically in a number of ways but the one adopted for the purposes of this Code is the ratio of the maximum gust, V, to the mean hourly wind speed $\overline{V}$, both having a probability of 0.02 of being exceeded only once in any one year. This ratio has been determined for all anemograph sites in the United Kingdom. The ratio $V/\overline{V}$ is found to vary from about 1.25 to about 2.4. Using the known site characteristics it is possible to classify site roughness into four categories each of which has an associated mean ratio $V/\overline{V}$.

Category 1. Open country with no obstructions	$V/\overline{V} = 1.5$
Category 2. Open country with scattered windbreaks	$V/\overline{V} = 1.7$
Category 3. Country with many windbreaks; small towns; outskirts of large cities	$V/\overline{V} = 1.9$
Category 4. Surface with large and frequent obstructions, e.g. city centres	$V/\overline{V} = 2.1$

A.2 Variation with height

Above an effective height of 10 m it is sufficient to assume that both mean hourly speeds and maximum speeds averaged over any selected interval of time between 1 h and 3 s vary with height according to a power law

$$V_H = V \left(\frac{H}{10}\right)^{\alpha}$$

where V_H is the maximum speed averaged over a given interval of time at height H metres,

V is the maximum speed averaged over the same interval of time at height 10 m and

α is the exponent for the power law that is specified for each averaging time and roughness category.

When $H = 10$ m, $\left(\frac{H}{10}\right)^{\alpha}$ becomes unity for all values of α, and $V_H = V$.

However, special experiments using three anemometers at the Post Office Tower in London have also shown that the general level of the rooftops may be taken into consideration and the power law modified. Suppose general rooftop level is Y metres and V_E is the maximum speed averaged over an appropriate time interval at a height of 10 m above Y, the power law becomes

$$V_H = V_E \left(\frac{H-Y}{10}\right)^{\alpha}$$

When $H = Y + 10 = E$, then $\left(\frac{H-Y}{10}\right)^{\alpha}$ becomes unity and $V_H = V_E$.

Introducing the suffix t to distinguish the different averaging times

$$V_{Ht} = V_{Et} \left(\frac{H-Y}{10}\right)^{\alpha}$$

A.3 Wind speed averaging times

It is shown in Appendix B that maximum wind speeds of a stated return period and averaged over periods of time equal to 3 s, 5 s and 15 s are required for design purposes. Because the anemograph in widespread use cannot be analysed to extract 5 s and 15 s maximum speeds, the results of special experiments have to be used to obtain the relationship between the maximum gust and the maximum speeds averaged over 5 s and 15 s intervals of time. It has been found that on the average at an effective height of *E* metres the relationship is:

5 s speed (V_5) = 0.95 × 3 s speed (V_3)

15 s speed (V_{15}) = 0.90 × 3 s speed (V_3)

Introducing these factors the maximum speed averaged over *t* seconds can be obtained at height *H* metres from a knowledge of the maximum speed averaged over 3 s at a height of *H* metres because

$$V_{Ht} = \left(\frac{V_{Et}}{V_{E3}}\right) V_{E3} \left(\frac{H - Y}{10}\right)^{\alpha}$$

A.4 Effective height

Finally, whatever the terrain, it has been assumed that the maximum speed averaged over any time interval is the same at gradient wind level. Allowances for the variation of gradient wind height with roughness of terrain cannot be made exactly but using the power law formula it is possible to calculate speed ratio at different heights and the factor $\left(\frac{V_E}{V_{10}}\right)$ has been introduced such that at certain heights, different for each ground roughness category, the 3-second gust speed is constant whatever the roughness of the underlying terrain.

It should be noted that *Y* represents a level corresponding to general rooftop or obstruction level. Thus in Category 1 it is assumed that the general obstruction height is zero, in Category 2 the mean level of obstruction is about 2 m, in Category 3 the mean level of obstruction is about 10 m and in Category 4 the obstructions have a general mean height of 25 m. Category 4 is thus seen to represent the centres of densely packed cities only where the buildings are tall (25 m or so) and the streets relatively narrow.

The effective height *E* is 10 m above the general rooftop or obstruction height *Y*.

For the 3-second gust these ratios are:

1) Speed at 12 m in Category 2 = 0.98 speed at 10 m in Category 1.

2) Speed at 20 m in Category 3 = 0.95 speed at 10 m in Category 1.

3) Speed at 35 m in Category 4 = 0.94 speed at 10 m in Category 1.

Introducing these ratios

$$V_{Ht} = \left(\frac{V_{Et}}{V_{E3}}\right) \left(\frac{V_{E3}}{V_{10,\,3}}\right) V_{10,\,3} \left(\frac{H - Y}{10}\right)^{\alpha},$$

where $V_{10,\,3} = V$, the basic 3-second gust at 10 m height in open country (map speed).

Defining S_2 as the number by which the maximum speed averaged over a time of 3 s at a height of 10 m above open level country (i.e. the basic speed) should be multiplied in order to obtain the maximum speed averaged over a time of *t* seconds at *H* metres above the different terrain where the general level of obstructions (or rooftop level) is *Y* metres,

$$S_2 = \left(\frac{V_{Et}}{V_{E3}}\right) \left(\frac{V_{E3}}{V_{10,\,3}}\right) \left(\frac{H - Y}{10}\right)^{\alpha},$$

Collecting the above factors and adding the values for the exponent, the values adopted from experiments, measurements and trial and error methods are:

Category	Y (metres)	E (metres)	$\left(\frac{V_{E3}}{V_{10,3}}\right)$	$\left(\frac{V_{Et}}{V_{E3}}\right)$			α		
				t (seconds)			t (seconds)		
				3	5	15	3	5	15
1	0	10	1.00	1.00	.95	0.90	0.080	0.090	0.100
2	2	12	0.98	1.00	.95	0.90	0.085	0.095	0.105
3	10	20	0.95	1.00	.95	0.90	0.090	0.100	0.110
4	25	35	0.94	1.00	.95	0.90	0.090	0.105	0.115

where V_{Ht} is the t second speed at height H in the appropriate category,

V is the 3-second speed at height 10 m in Category 1 (the map speed),

Y is the level corresponding to general rooftop or obstruction level,

E is the effective height 10 m above Y,

$\left(\frac{V_{E3}}{V_{10,3}}\right)$ is the conversion factor to ensure that at or near gradient wind height all 3 s speeds are the same whatever the roughness categories,

$\left(\frac{V_{Et}}{V_{E3}}\right)$ is the ratio of the t second to the 3-second wind at an effective height E, and

α is the selected power law exponent appropriate to the gust averaging time and the roughness categories.

The airflow near the ground between the obstructions will be a composite of wakes, deflections and channellings whose character cannot at present be codified owing to lack of observational data and a systematic theory. The pressure corresponding to a 3-second gust at 3 m height was fixed arbitrarily. The pressure at 10 m above general obstruction level was defined by the procedure described above. It has been assumed that the pressure increased linearly from the fixed value at 3 m to the defined value at 10 m above general obstruction level. From these definitions S_2 factors have been calculated.

A.5 Influence of change of roughness

Although classification of ground roughness into Categories 1 to 4 has been made, it should be recognized that the change from one ground roughness to another is necessarily a gradual process. The wind must traverse a certain ground distance before equilibrium is established in a new velocity profile. The change starts first in the layers of wind nearest the ground and the new profile extends to an increasingly deep layer as the fetch increases.

For practical purposes it may be assumed that a fetch of a kilometre or more is necessary to establish a different roughness category, but that within the actual roughness layer, i.e. below the general rooftop or obstruction level to windward, a lesser distance may apply as follows, depending on the density of buildings and other obstructions on the ground:

Ground coverage	Required fetch
$\nless$ 10 %	500 m
$\nless$ 15 %	250 m
$\nless$ 30 %	100 m

Shelter due to an individual building or obstruction should not be considered without expert guidance.

For a site where the ground roughness is different from different directions, the most severe grading should be used, or, exceptionally, appropriate gradings may be used for different wind directions. For example, the seafront of a coastal town would generally rank as ground roughness Category 1 (see also Appendix D if cliffs are involved).

Appendix B Explanatory notes on gusts and the wind-speed averaging time

The natural period of oscillation of most structures is only a few seconds or less and, since impulsive forces lasting only about a half-period of oscillation are effective in deflecting a structure, it follows that gusts of only a few seconds duration would produce significant wind loads if they developed simultaneously over the whole structure.

The incidence and spread of gusts over a building has been studied at the Building Research Station where it has been found that the gust loading on a tall rectangular office block is significantly greater than the mean load averaged over one minute as used in the 1952 edition of the Code.

Measurements suggest that the time interval over which maximum wind speeds should be averaged will depend on the size of the building or part of a structure under consideration. This averaging time varies from about 15 s for buildings of 50 m (165 ft) height or horizontal spread to 1 s or less for small elements such as windows, cladding units and roof coverings.

As a routine, at the network of anemograph stations maintained by the Meteorological Office in the United Kingdom, the mean hourly wind speed and the maximum gust of the day are extracted from the record. Because of the characteristics of the anemometer, the maximum gust speed represents the mean wind speed averaged over about 3 s. It is said to have an averaging time of 3 s and is often called the 3-second gust. Similarly the mean hourly wind speed has an averaging time of one hour.

Relatively little is known about the incidence and intensity of gusts averaged over periods of time shorter than 3 s, so that, although it is desirable to determine maximum wind loads on small units using gusts with shorter averaging times, it is not at present possible to specify magnitudes in strong winds. The standards adopted in the Code are, therefore, the maximum 3-second gust speeds for all units of glazing, cladding and roofing, whatever the size or proportion of the building concerned. A wind speed with 5 s averaging time is used for the structural design of buildings and structure where neither the greatest horizontal nor vertical dimension exceeds 50 m (165 ft). For buildings and structures whose greatest horizontal or vertical dimension exceeds 50 m (165 ft) a wind speed of 15 s averaging time is used.

for 115 stations in the United Kingdom, it is possible to analyse the series of annual maximum gust speeds statistically following the methods of Gumbel. A similar procedure is also adopted for the analysis of a series of annual maximum mean hourly wind speeds. However, in order to prepare a map of extreme values, the data must be homogeneous and refer to a standard datum level in a standard site. Extreme values for other sites and at other levels may then be estimated by the use of factors.

The basic wind speeds given in Table 1 and on the map in m/s have been adjusted as necessary from the results of the statistical computations. The maximum 3-second gust speed likely to be exceeded on the average only once in 50 years was chosen because

1) of the case for selecting the 3-second gust given above, and

2) the average lifetime of most buildings covered in the Code is near 50 years.

It should be noted that a value likely to be exceeded on the average only once in 50 years also has a probability of 0.02 of being exceeded only once in any year and is often called the 50 year value or the value having a return period of 50 years. An attempt has been made to enable the designer to choose speeds of different return periods by the use of suitable factors.

Appendix C A statistical factor S_3

Factor S_3 is based on statistical concepts and this appendix shows how the factor has been computed from the data and describes some of the uses.

The wind speed having a return period, T, of 50 years should more properly be called that speed which will be exceeded just once with a probability $p = 0.02$ in any one year. Return period and probability are connected by the relation $Tp = 1$.

Now the probability that a value less than or equal to wind speed x will occur in one year is $q = 1 - p$. In a period of N years the probability Q that a value less than or equal to x will occur is

$$Q = q^N.$$

The probability P that a value greater than x will occur at least once in a period of N years is

$$P = 1 - Q$$
$$= 1 - q^N$$
$$= 1 - (1 - p)^N$$
$$= 1 - \left(1 - \frac{1}{T}\right)^N .$$

For $N = 50$ and $T = 50$, $P = 0.63$. Therefore, there is only a probability of 0.63 that the once in 50 year wind speed will be exceeded at least once in a period of 50 years.

Factor S_3 has been obtained by selecting values of P and N, solving the equation for T and calculating values x_T, the wind speed of return period T, for all stations in the United Kingdom where observations are available. The value of S_3 for given P and N is $S_3 = x_T/x$, where x is the once in 50 years wind speed at the same place. For each pair of values of P and N the values of S_3 at places in the United Kingdom showed little variation with site. Therefore the mean of S_3 values at all places was taken for each pair of P and N, and some of them are plotted in Figure 2. On this diagram N is called the exposure period and S_3 the factor. The probability levels are the values of P. Figure 2 may be used in several different ways but normally $S_3 = 1.0$. The factor S_3 provides the designer with greater flexibility in choice of wind speed without referring to several different maps.

For the calculation of wind loads during construction or for calculation of wind loads on temporary structures whose probable life is short, wind speeds may be reduced using factor S_3. It is undesirable to consider an exposure period of less than 2 years in this context although the critical period may be only 2 weeks. As an example, with $P = 0.63$ the value of $S_3 = 0.77$ for an exposure period of two years. Therefore there is a probability of 0.63 that a speed which is 0.77 times the once in 50 year wind speed will be exceeded at least once in a period of 2 years. Because normally a probability level of 0.63 will be used for all design work the loads during construction may be calculated using a wind speed equal to 0.77 times the map speed. The calculations for the completed structure or building should use an appropriate value of S_3 (normally $S_3 = 10$).

The designer may also use Figure 2 to estimate wind loads which would result from choosing a different return period for the basic speed. For exposure periods greater than 10 years, with probability level equal to 0.63, the exposure period is roughly equal to the return period in years. The effect of multiplying a map speed by any given factor can be estimated in terms of return period. From Figure 2 the once in 100 years speed is roughly 1.05 times the once in 50 years speed.

The effect of greater safety can also be assessed. Suppose that, exceptionally, a probability level of 0.01 had been selected, then $S_3 = 1.35$. Although it cannot be seen from the diagram the application of this factor converts the once in 50 years wind to a once in 4 975 years wind. The probability that such a wind will be exceeded in any one year is 0.0002. It should be remembered that wind speeds having such low values of probability associated with them cannot be estimated satisfactorily from a record of even 50 years data and that there may be other factors which affect the value of such an extreme. For these reasons the use of probability levels other than 0.63 should be restricted to special cases.

When the wind load for a selected wind speed has been calculated it is common practice to apply a load or safety factor specified in other Codes of Practice. If it is assumed that the effect of this load factor is to increase the return period of the wind speed only, then the resulting change in the return period can be calculated from Figure 2. Suppose a load factor of 1.2 is applied. This would correspond to an increase of wind speed of $\sqrt{1.2}$ (= 1.1) over the design wind speed. Reference to Figure 2 will show that such an increase of wind speed, which would have been brought about by the use of a factor of 1.1, corresponds to the use of a return period of 200 years at the probability level of 0.63.

Appendix D Topography factors

D.1 General

The effect of local topography is to accelerate the wind near the summits or crests of hills, escarpments or ridges and decelerate the wind in valleys or near the foot of steep escarpments, or ridges.

The extent of this effect on gust wind speeds, as defined for the purposes of this Code by the topography factor S_1 in **5.4**, is confined to the range:

$1.0 \leqslant S_1 \leqslant 1.36$

Cases where the local topography is significant are defined in **D.2**. In such cases S_1 should be determined from **D.3**.

Where the local topography is not significant, as defined in **D.2**, S_1 should be taken as 1.0

For all cases outside the definitions provided in **D.2**, specialist advice should be sought (see Appendix H). All relevant wind directions should be considered.

D.2 Topographic definitions

D.2.1 *Terms*

L is the actual length of the upwind slope in the wind direction (see Figure 4).

Z is the effective height of the feature (see Figure 4).

ψ is the upwind slope Z/L, in the wind direction.

L_e is the effective length of the upwind slope, defined in Table 25.

s is a factor to be derived in accordance with **D.3**, appropriate to the height H above local ground level, and the distance x (see Figure 4) from the summit or crest relative to the effective length L_e.

D.2.2 *Significant slope.* For the purposes of this Code, local topography will be significant for a site when the upwind slope, ψ, is greater than 0.05.

D.2.3 *Type of feature.* The influence of the feature should be considered to extend 1.5 L_e upwind and 2.5 L_e downwind of the summit or crest of the feature.

If the zone downwind from the crest of the feature is level ($\psi < 0.05$) for a distance exceeding L_e, then the feature should be treated as an escarpment. If not, then the feature should be treated as a hill or ridge (see Figure 4).

In undulating terrain it is often not possible to decide whether the local topography of the site is significant in terms of wind flow. In such cases the average level of the terrain upwind of the site for a distance of 5 km should be taken as the base level from which to assess the height Z and the upwind slope ψ of the feature.

D.3 Topography factor S_1

S_1 should be obtained from Table 25 using the appropriate values for the slope of the hill ψ, the effective length L_e and the factor s which should be determined from:

a) Figure 5 for cliffs and escarpments: or

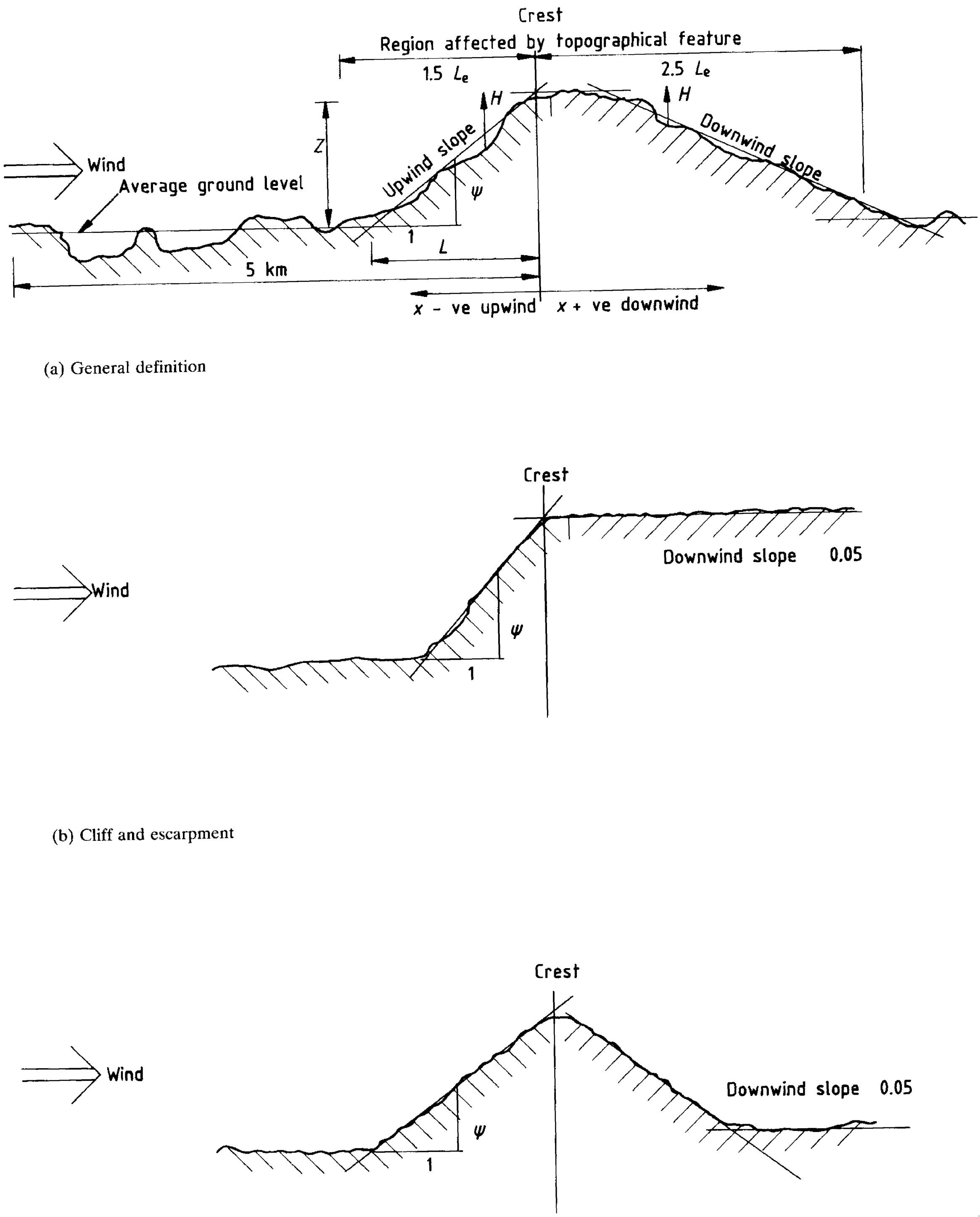
b) Figure 6 for hills and ridges.

NOTE 1 Where the downward slope of a hill or ridge is greater than 0.3 there will be large regions of reduced acceleration or even shelter and it is not possible to give general design rules to cater for these circumstances. Values of s from Figure 6 should be used as upper bound values.

NOTE 2 No differentiation is made in deriving S_1, between a three dimensional hill and a two dimensional ridge.

Table 25 — Values of L_e and S_1

Slope ($\psi = Z/L$)	
Shallow ($0.05 < \psi \leqslant 0.3$)	**Steep** ($\psi > 0.3$)
$L_e = L$ $S_1 = 1 + 1.2Zs/L$	$L_e = Z/0.3$ $S_1 = 1 + 0.36s$



(a) General definition

(b) Cliff and escarpment

(c) Hill and ridge

Figure 4 — Definition of topographical dimensions

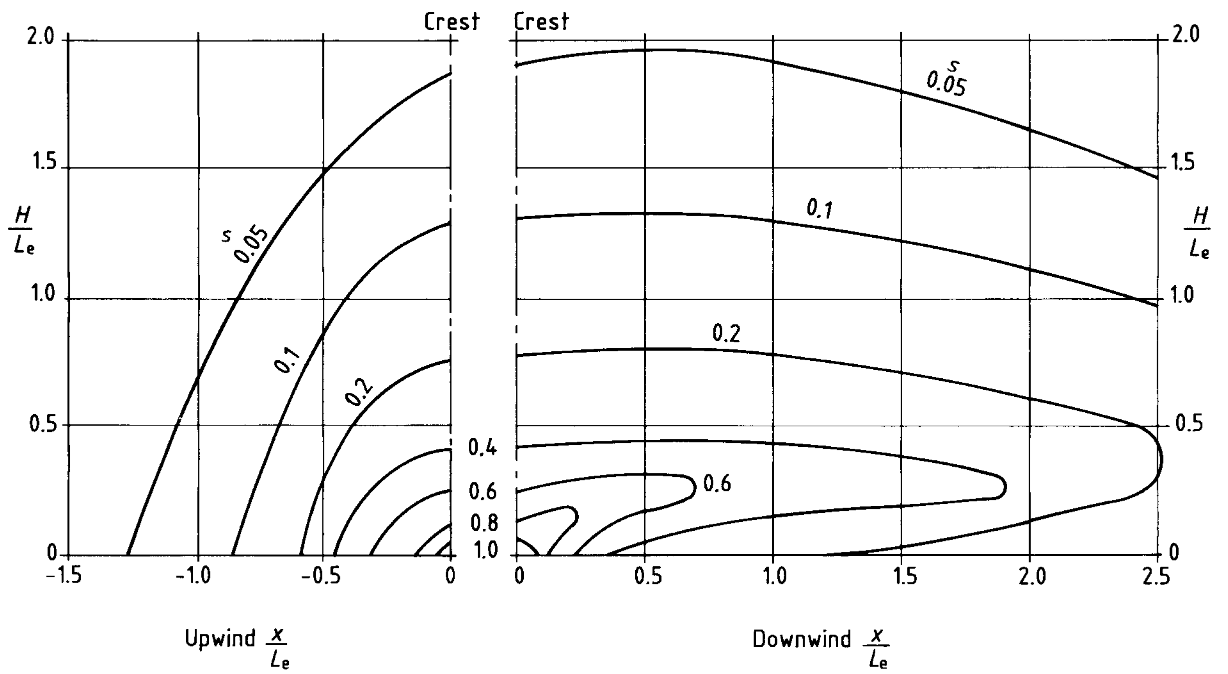


Figure 5 — Factor *s* for cliff and escarpment

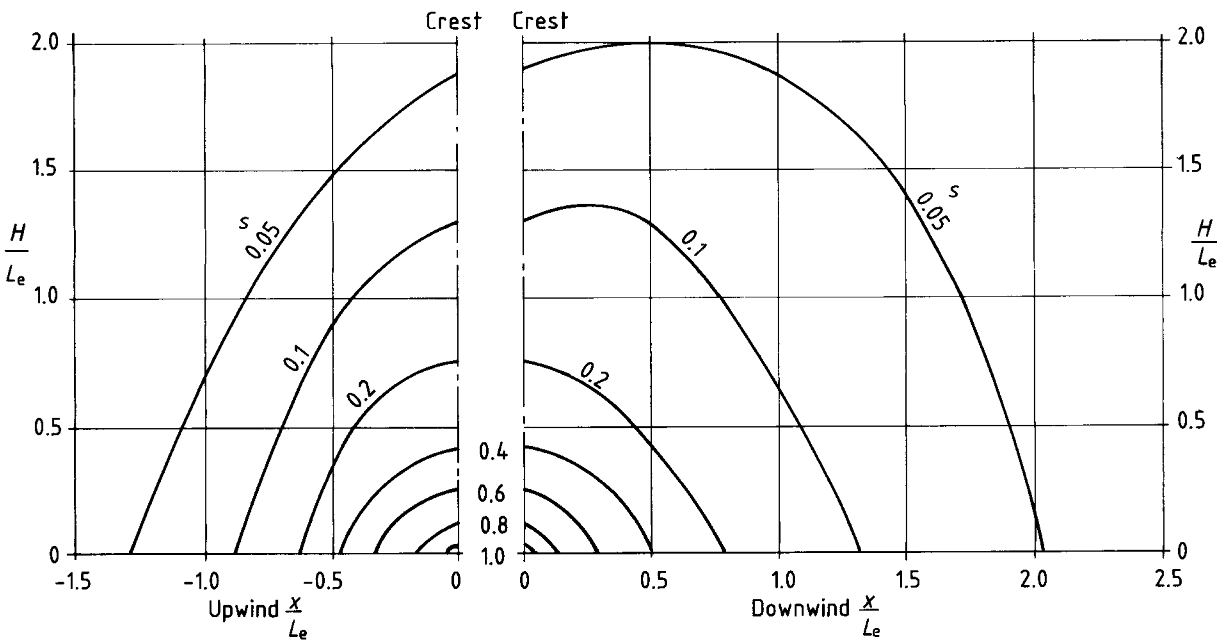


Figure 6 — Factor *s* for ridge and hill

Appendix E The estimation of internal pressure coefficients

The total wind force on a wall or roof depends on the difference of pressure between the outer and inner faces. Open doors, windows or ventilators on the windward side of a building will increase air pressure inside the building and this will increase the loading on those points of the roof and walls that are subjected to external suction (see Figure 7), and may affect the pressure on floors.

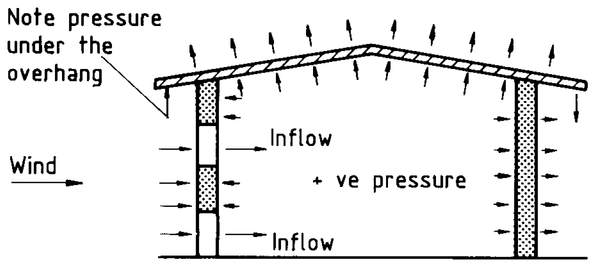


Figure 7 — Internal pressure coefficient (1)

Conversely, openings at positions that are experiencing external sunction will reduce the pressure inside the buildings thus increasing total loads on a windward wall, as shown in Figure 8.

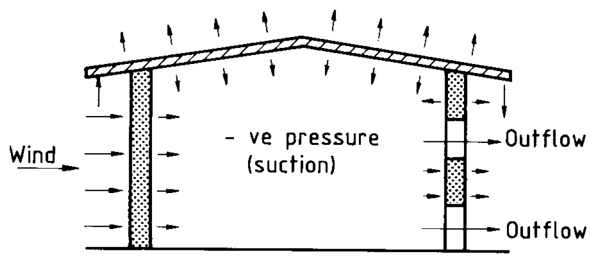


Figure 8 — Internal pressure coefficient (2)

In practice, conditions are generally not so simple. Most buildings have some permeability on each face, through windows, ventilation louvres, leakage gaps around doors and windows and to some extent through the cladding itself; and if there are chimneys, these can provide a low-resistance path for air flow. Permeability in this context is measured by the total area of such openings in a face. The problem is to determine the resulting internal balance of all the contributing leakage points for all critical wind directions, and, for design purposes, to assess the worst possible combination of external and internal pressures that may be developed on each wall or roof unit.

The following examples indicate approximately the values of C_{pi} that apply to a building with a reasonably open interior plan and are to be applied to the same values of q as the building in which they occur. If the interior is divided by relatively impermeable partitions the pressure difference between windward and leeward faces of the building will be broken down in steps, and will impose loads on the partitions.

	C_{pi}
1) Two opposite faces equally permeable; other faces impermeable	
a) wind normal to permeable face	+ 0.2
b) wind normal to impermeable face	– 0.3
2) Four faces equally permeable	– 0.3

	C_{pi}
3) With equal permeability on all faces except for a dominant opening on one or other face, of size and position as follows:	
a) on windward face, making the permeability of the windward face equal to the following proportions of the total distributed permeability of all the faces subject to suction.	
Proportion 1	+ 0.1
Proportion 1½	+ 0.3
Proportion 2	+ 0.5
Proportion 3 or more	+ 0.6
b) on leeward face: any dominant opening	– 0.3
c) on a face parallel to the wind	
i) any dominant opening not in an area of high local C_{pe}	– 0.4
ii) in an area of high local C_{pe}:	
if the area of the opening equals the following proportion of the total other distributed permeability of all the external faces subject to suction.	
¼ or less	– 0.4
½	– 0.5
¾	– 0.6
1	– 0.7
1½	– 0.8
3 or more	– 0.9

The distributed permeability should be assessed in each case as accurately as is practicable. As a guide it can be said that the typical permeability of a house or office block with all windows nominally closed is in the range of 0.01 % to 0.05 % of the face area, depending on the degree of draughtproofing.

Where it is not possible, or is not considered justified, to estimate the value of C_{pi} for a particular case, the coefficient should be based on one of the following paragraphs for any determination of wall or roof loading.

1) Where there is only a negligible probability of a dominant opening occuring during a severe storm, C_{pi} should be taken as the more onerous of + 0.2 and – 0.3.

2) For situations where a dominant opening is likely to occur, C_{pi} should be taken as 75 % of the value of C_{pe} outside the opening. The extreme conditions should be determined for the various wind directions that give rise to critical loadings and it should be noted that especially severe internal pressures may be developed if a dominant opening is located in a region of high local external pressure.

There is a further complication in a wall or roof element that comprises several layers. For example, a roof may be boarded and felted and covered with tiles. The pressure difference between outside and inside will then be broken down into steps, across each layer; these steps will depend on the relative permeability of the various layers and the access of air to the spaces between them. Each case needs careful study to ensure that the whole of the wind load is not accidentally transferred to a single membrane such as a thin metal sheet that may not be designed to carry it.

Control of internal pressure. The value of C_{pi} can sometimes be limited or controlled to advantage by the deliberate distribution of permeability in the walls and roof or by the deliberate provision of a venting device that can serve as a dominant opening at a position having a suitable external pressure coefficient. An example of such an application is a ridge ventilator on a low pitch roof which, under all directions of wind, will reduce the uplift force on the roof.

Appendix F Ice formation on structures

F.1 In winter or early spring each year at some place in the British Isles ice is deposited on structures and the designer would ideally like to know

1) what weight of ice can form on structures,

2) what is the shape of the ice deposit,

3) what is the density of the ice deposit,

4) what wind speeds are likely during and after ice deposition, and

5) how frequently do these conditions occur.

This appendix provides general guidance on icing conditions within the limitations set out below. In the discussion mean speeds are given because it is the build-up of ice that is important. An estimate of the maximum gust speed when the ice load has been established can be made by multiplying the appropriate mean speed by 1.5. The conditions leading to and after ice formation are not likely to be the same as those in which extreme gusts occur, so, if extreme gusts calculated by the method described in **5** are used to compute wind loads on iced structures an overestimate will be produced. Also, in strong winds ice will be blown off the structure and this may induce vibrations. No attempt has been made to discuss these vibration effects.

F.2 Ice may be deposited from three different types of precipitation.

1) Freezing rain or drizzle, as discussed in **F.3**, **F.4** and **F.5**.

2) Fog or cloud at temperatures below 0 °C, as discussed in **F.6**, **F.7** and **F.8**.

3) Melting snow as discussed in **F.9**.

Occasionally two or perhaps all three types may occur simultaneously or in sequence and this possibility is discussed in **F.10**. The wind forces or movements of the structure can be sufficiently strong to cause lumps of an otherwise even coating of ice to break off. Although, throughout the past 30 years or so, there have been reports of ice deposits in the meterological literature, most of the known occasions have been descriptively but not quantitatively analysed and as a result reliable statistics cannot be compiled. This appendix will, therefore, necessarily be fragmentary.

F.3 The most dense ice deposits arise when raindrops cooled to below 0 °C fall on an exposed structure and, perhaps after some running on the structure, freeze to form a clear ice deposit called glazed frost. The precipitation is known as freezing rain or freezing drizzle. Reports of freezing precipitation are often localized and occur once every few years in some parts of the country, being mainly confined to England and Wales. The most widespread glazed frost of recent years, perhaps of this century, occurred in January, 1940, and is described by Brooks and Douglas[6]. They reported deposits of 1 488 g of ice on a spray of beech twigs weighing 100 g and deposits of 50 mm diameter on telegraph wires. Another widespread glazed frost occurred between 11 and 15 March, 1947, but on this occasion the thickness of the deposit was generally less well reported. Vertical surfaces exposed to freezing precipitation are generally coated with ice on the side facing the wind. If the surface is flat and broad (e.g. a house side) the deposit has the form of a sheet of ice of more or less uniform thickness, rarely 40 mm to 50 mm thick. If the surface is curved and relatively narrow laterally (e.g. trees, telegraph poles) the deposit tends to build smoothly, but with a thicker deposit directly into wind and thinner deposits at the sides of the object producing a change of curvature. Thicknesses of up to 50 mm have been observed. If the surface is markedly curved and very narrow (e.g. cables, wire mesh fences) then the deposit, before freezing, may run to completely encase the exposed surface often embedding the support in the centre of the ice section but just as frequently building an oval section ice deposit with the wire roughly at the focus on the major axis furthest from the oncoming wind. If a cable or wire is at an angle to the vertical or if it is horizontal, the asymmetry of the load may induce a twisting moment. The coating of ice may then exhibit spiral effects with a very uneven surface but the absence of spirals cannot be taken to imply that twisting has not taken place. Glazed frost adheres strongly to most metallic, mineral or organic surfaces and is relatively dense; in the absence of measurements it must be assumed that the density is 0.92 g/cm^3, the density of pure water ice at 0 °C, because little or no air is trapped in the deposit (when air is trapped the deposit is white).

[6] BROOKS, C. F. P., and DOUGLAS, C. K. M. The glazed frost of January, 1940. H.M.S.O., *Geophysical Memoir* No. 98.

F.4 There are two types of meterological situation in which the conditions for glazed frost may be met. One is a steady situation in East or South-East winds with a narrow band of warm air overlying a very cold surface layer of air and with very cold air above so that snow falling from above is melted in the warm layer and then the drops or droplets are supercooled as they pass through the cooler underlying layer. This situation may persist for days and is mainly reported in England and Wales. The hourly mean wind speeds during formation are usually in the range from 6 m/s to 10 m/s and, exceptionally, speeds of 15 m/s may be experienced. The glazed frosts of January, 1940, March, 1947, and March, 1969, are typical examples in which deposits of 50 mm or so were recorded on trees, cables and house sides. More frequently, the meterological conditions are met for only a short time after a cold spell. On these occasions a period of 2 h or 3 h of freezing rain may be followed by very strong warm winds with mean hourly speeds of 20 m/s to 25 m/s, but melting of the ice takes place rapidly. Such an occasion was reported on 4 March, 1970. Because the deposit is formed in a short period of rain, thicknesses of 25 mm or so are unlikely to be exceeded except on rare occasions. This type of glazed frost, but with variations in deposit thickness, is reported in some part of the United Kingdom every year with wind speeds of up to about 15 m/s or so, mainly in Scotland but particularly on hills. While no detailed observations are available it is reasonable to assume that temperature and precipitation conditions at heights up to 200 m or so above ground will not vary greatly from conditions at the surface. However, it may also be assumed that wind speeds will increase with height according to a power law with exponent 0.17 for hourly mean speeds. Thus an hourly mean wind speed of 15 m/s at the surface becomes 25 m/s or so at 200 m.

F.5 It is suggested that for glazed frosts in England and Wales design criteria might be:

Maximum ice thickness	25 mm to windward
Ice density	0.92 g/cm^3
Hourly mean wind speeds during formation	10 m/s at the surface and increasing according to the power law to about 17 m/s at 200 m
Hourly mean wind speeds after formation	15 m/s at the surface and increasing with height to 25 m/s or so at 200 m.

Gust speeds may be computed as outlined in **F.1**.

In Scotland or on hill tops in England and Wales another, equally likely criterion might be:

Maximum ice thickness	15 mm to windward
Ice density	0.92 g/cm^3
Hourly mean wind speed during formation	15 m/s at the surface increasing according to the power law with exponent 0.17
Hourly mean speed after 2 or 3 h	20 m/s at the surface increasing according to the power law with exponent 0.17.

These two sets of criteria spring from different meterological situations and the probability of their occurrence cannot be assessed at the present time.

F.6 The least dense ice deposits are formed when fog or cloud is blown, usually at steady hourly mean speeds of 3–8 m/s, onto cold surfaces, freezing on impact to form a loose aggregate feather ice or rime ice that builds into wind in the shape of pointed icicles. Air is trapped between the frozen droplets that are white in appearance. The density of rime ice has been measured in controlled conditions by Macklin[7] who studied the influence of wind speed, ambient air temperature, droplet diameter and liquid water concentration, but no densities have been determined so far as is known from actual atmospheric samples. Macklin found densities of from 0.1 g/cm^3 to 0.9 g/cm^3 being dependent directly on droplet diameter and wind speed but inversely proportional to temperature of the accreting surface. For practical purposes a value of 0.5 g/cm^3 may be adopted because the very dense cases are associated with large droplet diameters. The ice is usually fragile, disturbances in the airflow and movements of the depositing surface being of sufficient force to result in lumps of ice breaking off. In steady wind conditions, on fairly solid objects like masts, rime ice will form a banner *into the wind*. If the object is thin enough, e.g. wires, there will be only one banner but flat narrow objects may acquire two banners, one at each edge facing the wind. These banners may grow to lengths of 600 mm[8]. Occasionally the banners may fill the spaces between adjacent upright members of a structure, the strength of the ice deposit being increased by the additional support and the tendency to fracture being much reduced.

F.7 During the period November to March in the British Isles rime ice may be experienced on structures rising from ground at 200 m or so above sea level, particularly on the windward slopes of hills. Usually banners of less than 200 mm length are recorded at lower levels but banners of 600 mm or more may occur at 1 000 m or so above sea level every year and once every few years at lower altitudes. However, the length of the banner will depend on the size of the depositing surface, its shape, the steadiness, particularly in direction, of the wind and the duration of the conditions of icing.

F.8 It is suggested that for this type of deposit, suitable design criteria might be:

Maximum ice length into wind	600 mm
Density of ice	0.5 g/cm^3
The hourly mean speed during and after formation	10 m/s at 200 m above sea level rising to 30 m/s at 1 000 m above sea level.

These criteria should be applied for all structures on hills at heights greater than 200 m above sea level.

F.9 Ice may also be deposited as melting snow on cold surfaces. The resulting deposit is whitish in appearance and is midway between glazed frost and rime ice both in appearance and density, the latter being about 0.7 g/cm^3 if the one measurement of 38 lb/ft^3 made on ice from the cables of the television mast at Emley Moor, Yorkshire, in March, 1969, is accepted. Because it is a midway case no design criteria are offered but hourly mean wind speeds of 15 m/s are fairly common, particularly in Scotland.

F.10 Combinations of the three methods of deposition in any order are possible but perhaps the sequence at Emley Moor during March 1969, will serve to illustrate the complexities. During the period 12–19 March, 1970, hourly mean speeds varied from 3 m/s to 9 m/s. Throughout the period fog was reported at temperatures near 0 °C to about – 2 °C. Superimposed on this more or less continuous accretion of rime ice were periods of melting snow and freezing rain. On the 19th of March ice up to 160 mm in thickness was measured on cables of about 30 mm diameter. On this occasion, however, hourly mean wind speeds did not exceed 10 m/s even on the 19th when rapid thawing took place with air temperatures between 1 °C and 3 °C, the higher temperatures being at 200 m to 370 m above the ground.

On lattice masts it is essential to assume that the interstices between members will be filled with a wall of ice. Because the ice will have been deposited by at least two processes, the density will be about 0.7 g/cm^3 in hourly mean wind speeds of 15 m/s. The thickness of the wall of ice will depend largely on the breadth of the surface facing into wind but values of 75 mm are not uncommon. Such icing will usually only be experienced at ground level, which is 200 m or more above sea level. Hourly mean wind speeds in the free air above the ground may be assumed to increase according to the power law with exponent 0.17 to heights up to 400 m above the ground; above this level there should be no increase of hourly mean wind speed. In very hilly terrain or on exposed hill tops rising 600 m or more above sea level, the hourly mean wind speed should be assumed constant at 28 m/s for reasons just given.

[7] MACKLIN,—. *Quarterly J. Royal Met. Soc*., 88, No. 375, Jan., 1962, pp. 3150.
[8] PAGE,—. Heavy glaze in Yorkshire — March, 1969. *Weather*, Dec., 1969, p. 8: See also photograph of rime ice on p. 497.

For design purposes for lattice masts it is suggested that the criteria should be:

1) Wall of ice filling the spaces between upright members	
2) Density of ice	0.7 g/cm^3
3) Hourly mean wind speeds during and after formation at heights from 200 m to 600 m above sea level	15 m/s rising according to the power law to about 28 m/s and constant at 28 m/s at all heights above 600 m.

Appendix G Wind forces on circular sections

The wind force on any object is given by

$F = C_f.A_e.q$

where C_f is the force coefficient (see **7** and **8**)

A_c is the effective area of the object normal to the wind direction, and

q is the dynamic pressure of the wind (see **6**).

For most shapes the force coefficient remains approximately constant over the whoe range of wind speeds likely to be encountered. However, for objects of circular cross section it varies considerably.

For a circular section the force coefficient depends upon the way in which the wind flows around it and is dependent upon the velocity and kinematic viscosity of the wind and diameter of the section. The force coefficient is usually quoted against a non-dimensional parameter, called the Reynolds number, which takes account of the velocity and viscosity of the flowing medium (in this case the wind) and the member diameter.

Reynolds number, $Re = \frac{DV_s}{\upsilon}$

where D is the diameter of the member,

V_s is the design wind speed, and

υ is the kinematic viscosity of the air, which is 1.46×10^{-5} m^2/s at 15 °C and standard atmospheric pressure.

Since in most natural environments likely to be found in the United Kingdom the kinematic viscosity of the air is fairly constant, it is convenient to use DV_s as the parameter instead of Reynolds numbers and this has been done in this Code of Practice.

The dependence of a circular section's force coefficient upon Reynolds number is due to the change in the wake developed behind the body.

At a low Reynolds number the wake is as shown in Figure 9 and the force coefficient is typically 1.2. As Reynolds number is increased the wake gradually changes to that shown in Figure 10, i.e. the wake width d_w decreases and the separation point, s, moves from the front to the back of the body.

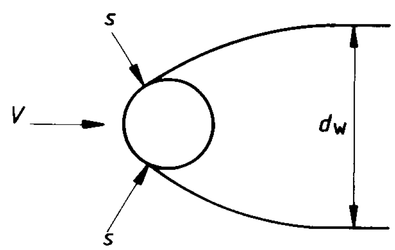


Figure 9 — Wake in subcritical flow

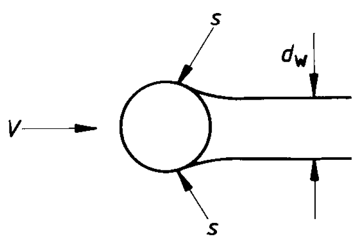


Figure 10 — Wake in supercritical flow

As a result the force coefficient shows a sudden drop at a critical value of Reynolds numbers, followed by a gradual rise as Reynolds number is increased still further.

The variation of C_f with the parameter DV_s is shown in Figure 11 for infinitely long circular cylinders having various values of relative surface roughness (ϵ/D)[9], when subjected to a wind having an intensity and scale of turbulence typical of built-up urban areas. The curve for a smooth cylinder ($\epsilon/D = 1 \times 10^{-5}$) in a steady airstream, as found in a low-turbulence wind tunnel, is shown for comparison.

It can be seen that the main effect of free-stream turbulence is to decrease the critical value of the parameter DV_s. For subcritical flows, turbulence can produce a considerable reduction in C_f below the steady airstream values. For supercritical flows, this effect becomes significantly smaller.

[9] For values of ϵ, see Data sheets of the Engineering Sciences Data Unit, 251-259 Regent Street, London WIR 7AD.

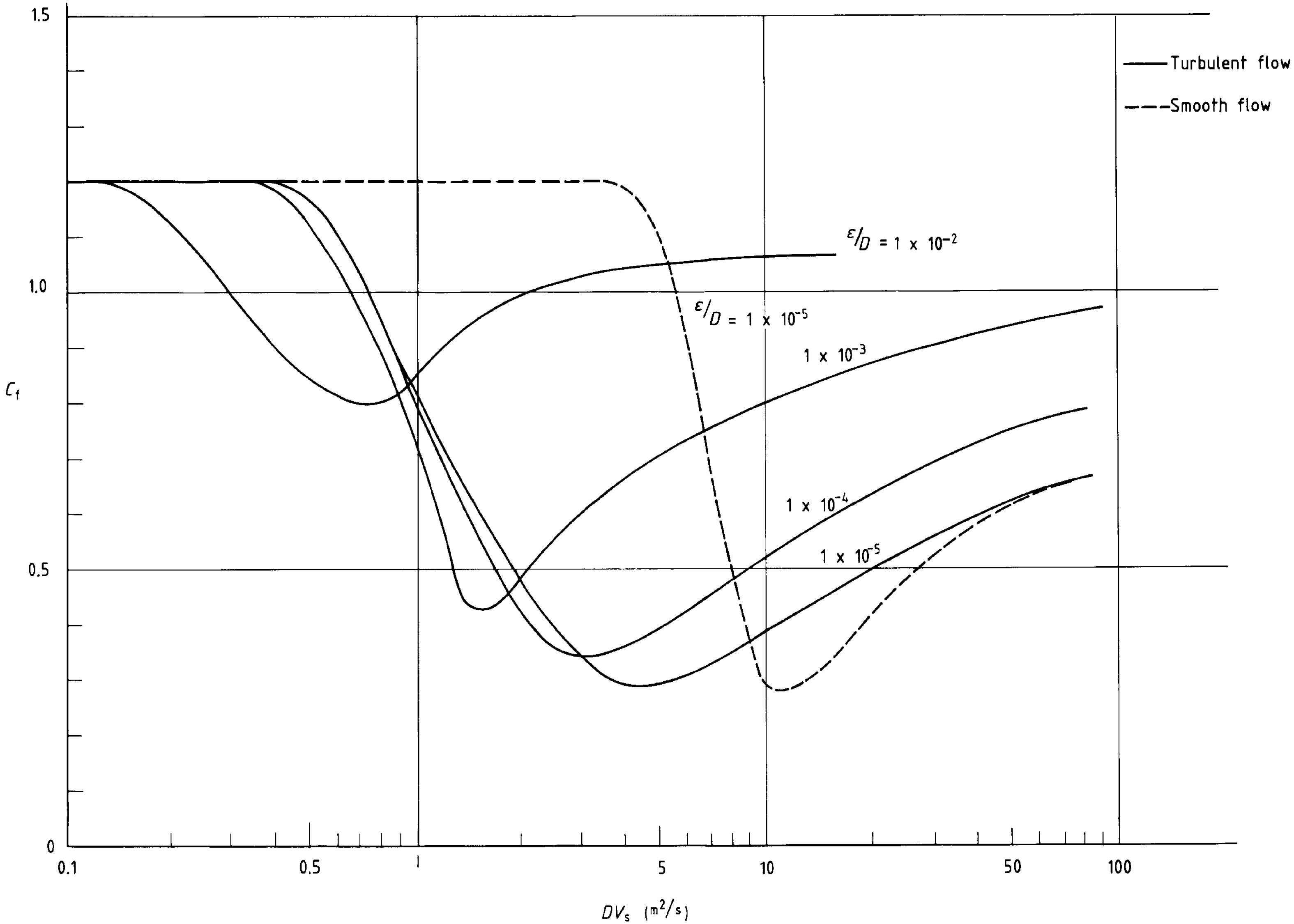


Figure 11 — Variation of force coefficient C_f with DV_s for infinitely long circular cylinders having various degrees of surface roughness

Appendix H Addresses of advisory offices

Addresses of advisory offices of the Meteorological Office

For England and Wales	Meteorological Office Met 0 3, London Road, Bracknell, Berkshire, RG12 2SZ Tel. 0344 420242 Extn. 2299
For Scotland	Meteorological Office 231 Corstorphine Road, Edinburgh EH12 7BB Tel. 031 344 9721 Extn. 524
For Northern Ireland	Meteorological Office Progressive House, 1 College Square East, Belfast BT1 6BQ Tel. Belfast 28457

Address of the advisory service of the Building Research Establishment

The Advisory Service
Building Research Station,
Bucknalls Lane,
Garston,
Watford WD2 7JR
Tel: 0923 67 6612

Appendix J Published sources for analysis of dynamic structures

Analytical methods for the response of dynamic structures to wind loading are given in the following documents:

1) Engineering Sciences Data, Wind Engineering Sub-series (4 volumes), London, ESDU International.

(NOTE A comprehensive index covering all items of Engineering Sciences Data is available on request from ESDU International, 251-259 Regent Street, London WIR 7AD. Tel. 01 437 4894.)

2) "Wind Engineering in the Eighties", London, Construction Industry Research and Information Association, 1981. (CIRIA, 6 Storey's Gate. London SW1P 3AU. Tel. 01 222 8891).

3) E. Simiu and R. H. Scanlan, "Wind Effects on Structures", New York, John Wiley and Sons, 1978.

4) Supplement to the National Building Code of Canada, 1980, NRCC, No. 17724, Ottawa, National Research Council of Canada, 1980.

Appendix K Necessary provisions for wind tunnel testing

Tests for the determination of wind loads on static structures should be considered properly conducted if:

1) the natural wind has been modelled to account for the variation of mean wind speed with height above ground appropriate to the terrain of the site;

2) the natural wind has been modelled to account for the intensity and scale of the turbulence appropriate to the terrain of the site;

3) the geometric length scale of the building model is not more than the following multiples of the geometric length scale of the simulated natural wind for the condition stated.

3 for overall values and 2 for cladding;

4) the response characteristics of the wind tunnel instrumentation are consistent with the measurements to be made;

5) the measurements yield the maximum wind loads expected on the building, commensurate with the recommendations of this Part of the Code.

Tests for the determination of the response of dynamic structures should be considered properly conducted only if:

a) the provisions for static structures, 1 to 5 above, are satisfied: and additionally,

b) the structural model is represented (physically or mathematically) in mass distribution, stiffness and damping in accordance with the established law of dimensional scaling.

Information to enable designers to make a considered judgment of the facilities offered when commissioning wind tunnel tests is available in "Wind Tunnel Modelling for Civil Engineering Applications" (T. A. Reinhold, Ed.), Cambridge, Cambridge University Press, 1982. In case of doubt, advice may be sought from the Building Research Establishment, Watford.

Appendix L Basic wind speed adjustment by direction

The basic wind speed may be adjusted to ensure that the risk of it being exceeded is the same for all directions. This may be achieved by an additional wind speed factor, the directional factor S_4, values of which are given in Table 26.

Wind direction is defined in the conventional manner, in that an easterly wind is a wind direction of 90° and blows from the east to the site.

When applying these values of S_4 the topography factor S_1 and the terrain roughness, building size and height above ground factor S_2, should be appropriately assessed for that direction.

The modified design wind speed $V_s = V S_1 S_2 S_3 S_4$

For local pressure coefficients which apply to upwind edges, use the highest value of V_s within the range of wind directions for which that edge is upwind. For overall pressure coefficients use the highest value of V_s within the range of wind directions ± 45° of the wind angle, α, under consideration.

Table 26 — Directional factor S_4

Wind direction (degrees)	N 0	30	60	E 90	120	150	S 180	210	240	W 270	300	330
General S_4	0.78	0.73	0.73	0.74	0.73	0.80	0.85	0.93	1.00	0.99	0.91	0.82
Coastal S_4	0.84	0.78	0.78	0.79	0.78	0.86	0.91	1.00	1.00	1.00	1.00	0.88

Coastal values of S_4 are applicable within 5 km of the coast for on-shore wind directions.

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