BS 8100-1:
1986

Lattice towers and masts —

Part 1: Code of practice for loading

UDC 624.97.074.5

Committees responsible for this British Standard

The preparation of this British Standard was entrusted by the Civil Engineering and Building Standards Structures Committee (CSB/-) to Technical Committee CSB/40, upon which the following bodies were represented:

British Broadcasting Corporation
British Steel Industry
British Telecommunications plc
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Department of Energy (Electricity Division)
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Joint Radio Committee of the Nationalized Power Industries
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North of Scotland Hydro-electric Board
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Contents

Foreword

This Part of BS 8100 has been prepared under the direction of the Civil Engineering and Building Structures Standards Committee. BS 8100 is a standard combining a code of practice and a guide covering the loading and design of lattice towers and masts of metallic construction.

It comprises the following Parts:

— *Part 1: Code of practice for loading*;

— *Part 2: Guide to the background and use of Part 1 "Code of practice for loading"*.

DD 133 covers the strength assessment of members.

This Part of BS 8100 does not apply to other metallic structures. Other British Standards exist for some of those structures.

It has been assumed in the drafting of this British Standard that the execution of its provisions will be entrusted to appropriately qualified and experienced people.

The full list of organizations who have taken part in the work of the Technical Committee is given on the inside front cover. The Chairman of the Committee is Mr A Taylor and the following people have made a particular contribution in the drafting of the code.

Mr B W Smith	Consultant Drafter
Mr B P Wex OBE	Past Committee Chairman
Mr R L Clapp	
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Summary of pages

This document comprises a front cover, an inside front cover, pages i to iv, pages 1 to 64, an inside back cover 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.

Section 1. General

1.1 Scope

This Part of BS 8100 gives recommendations for procedures to be used to determine the loading for the design or appraisal of free-standing tower structures of lattice construction.

Whilst sections 3 and 4 are applicable to the loading of guyed masts, the gust response factors contained in section 5 should not be used for the design of such structures. Until such time that this Part of BS 8100 is extended to cover guyed masts, recourse should be made to published sources to determine their response to gust loading. In addition, the safety factors set out in section 2 may be inappropriate to guyed masts and should only be used for general guidance purposes.

The procedures given primarily apply to bolted, riveted, or welded metallic structures of up to 300 m height and composed of leg members and triangulated bracings, but they also encompass towers of geodetic form. Section 5 may be inappropriate for towers of Vierendeel form, due to their stiffness characteristics. The response of such towers should be determined using published sources.

This Part of BS 8100 covers dead loads and wind and ice loading. Other superimposed loads are not covered and, where appropriate, should be considered.

This Part of BS 8100 is applicable to structures in the UK for which specific meteorological data are given. Guidance is also provided on the use of this Part of BS 8100 for structures in other countries.

This Part of BS 8100 is based on the principles of limit state design and the partial safety factors given are appropriate only for towers designed in accordance with this standard.

A flow chart indicating the design procedure using this Part of BS 8100 is given on a foldout page following Appendix G.

NOTE The titles of the publications referred to in this standard are listed on the inside back cover.

1.2 Definitions

For the purposes of this Part of BS 8100 the following definitions apply.

1.2.1
wind resistance

the product of the drag coefficient and a reference projected area, forming the resistance to the flow of wind offered by the elements of a tower and any ancillary items that it supports

1.2.2
linear ancillary item

any non-structural component, extending over several panels, e.g. waveguides, feeders, ladders and pipework

1.2.3
discrete ancillary item

any non-structural component, concentrated within a few panels, e.g. dish reflectors, aerials, lighting and insulators

1.2.4
projected area

the shadow area of the element considered when projected on an elevation normal to the face of the tower under consideration. For wind blowing other than normal to the tower, a reference face is used for the projected area

NOTE For tower faces inclined to the vertical by more than 20°, due account may be taken of the horizontal projected area.

1.2.5
panel

a convenient division of a tower into vertical sections for the purposes of deriving projected areas and wind resistance, typically, but not necessarily, taken between intersections of legs and primary bracings

1.3 Symbols

1.3.1 Main symbols

The main symbols used in this Part of BS 8100 are as follows. For subscripts other than those given in this subclause, see **1.3.2**. Symbols are further clarified as appropriate in the text.

A	projected area
a	altitude
B	size factor
b	width of tower
C	drag coefficient
c	distance from leading edge of tower
D	diameter
d	depth of tower in direction of the wind
e	vertical dimension
F	force in a member
f	fraction or ratio
G	gust response factor
g	cost ratio
H	overall height
h	height
I	intensity
i	life
j	height factor
K	factor
k	coefficient
L	projected length
m	mass
N	number
n	frequency
P	wind load
R	wind resistance
R_e	effective Reynolds number
R^v	normalized co-spectrum
r	radial ice thickness
S^v	power spectrum
S	Strouhal number
s	separation
t	time; thickness
V	wind speed
$\overline{V}$	mean hourly wind speed
v	coefficient of variation
x	horizontal distance from crest of hill
z	height above ground level
z_o	terrain roughness parameter
α	power law index of variation of wind speed with height
β	slope
γ_m	partial safety factor on strength
γ_{DL}	partial safety factor on dead load
γ_v	partial safety factor on wind speed and ice thickness
Δ	deflection
δ	logarithmic decrement of damping
η	shielding factor
θ	angle of wind incidence to the normal in plan
μ	terrain profile index
υ	kinematic viscosity of air
ρ	density
Σ	sum or total
τ_i	time constant for anemometer
τ_o	dynamic volume/resistance constant
ϕ	solidity ratio
ψ	angle of wind incidence to longitudinal axis
ω	spacing ratio

1.3.2 Subscripts

Subscripts (and superscripts) where used in this Part of BS 8100, are as follows.

A	ancillary item
a	aerodynamic, air
B	basic
b	calibration
C	cable
c	circular-section members
cr	critical
d	wind direction
E	equivalent
e	effective, excitation
F	face
f	flat-sided members
g	gradient wind
H	tower top
h	hill
i	with ice
k	characteristic
L	length
m	bending moment
N	overall
n	single frame
o	in the absence of wind
q	shear
R	terrain roughness
r	reference
S	serviceability
s	structural
T	bare tower *or* tower only
t	time
w	with wind
W	in the direction of the wind
X	in the crosswind direction
x	horizontal distance x from site
z	height z above ground level
δ	damping
θ	angle of wind incidence
µ	terrain profile
$\overline{\ }$	due to mean hourly wind effects, e.g. $\overline{V}$, $\overline{P}$, $\overline{F}$
$'$	due to fluctuating wind effects, e.g. V', P', F'

Section 2. Performance requirements

2.1 Safety and service life

2.1.1 General

In order to select appropriate safety factors to be applied in design to the loadings defined in this code, consideration should be given to the reliability required of a tower during its intended period of service. The factors adopted should take into account the risk to life in the event of a collapse and the potential economic or strategic consequences of failure. They also depend on the quality of materials and workmanship specified and achieved in construction.

Appropriate values of the safety factor should be determined in accordance with **2.2** to **2.4**.

For the majority of towers, simple static analysis of the structure under the factored loads is sufficient to determine the loading effects. For towers which are likely to exhibit dynamic sensitivity, spectral analytical methods should be adopted. The criteria for when such methods are required are set out in **5.1.1**.

For modifications to existing structures, due account should be taken of any changed performance requirements since the original design of the tower.

2.1.2 Service life

For the purposes of this code, the design service life, i_s, in years, is taken as the intended period of service of the tower. A tower designed in accordance with this code may, if adequately maintained, provide consistent reliability beyond the period of its design service life, that period being used only as a basis for the economic assessment of the appropriate annual risk of failure. The fatigue life may, however, limit the period of reliable service; for temporary structures a minimum design life of 5 years should be adopted.

2.2 Classification of required reliability

2.2.1 General

The safety factors to be used, appropriate to the reliability required of a tower, should be selected on the basis of either of the following performance requirements:

a) the potential hazards resulting from failure of the tower, i.e. the environmental conditions near the tower (see **2.2.2**); *or*

b) the economic consequences of failure or the usage of the tower (see **2.2.3**).

2.2.2 Environmental conditions

The environmental category of the tower should be selected with due consideration of the potential risk to life in the event of the tower's failure. This risk will depend on the location and size of the structure in relation to inhabited buildings, railways or roads and on the possible contingent effects of collapse.

Figure 2.1 indicates categories which should be used to select safety factors within the range appropriate to the environment of the tower.

2.2.3 Economic consequences or usage

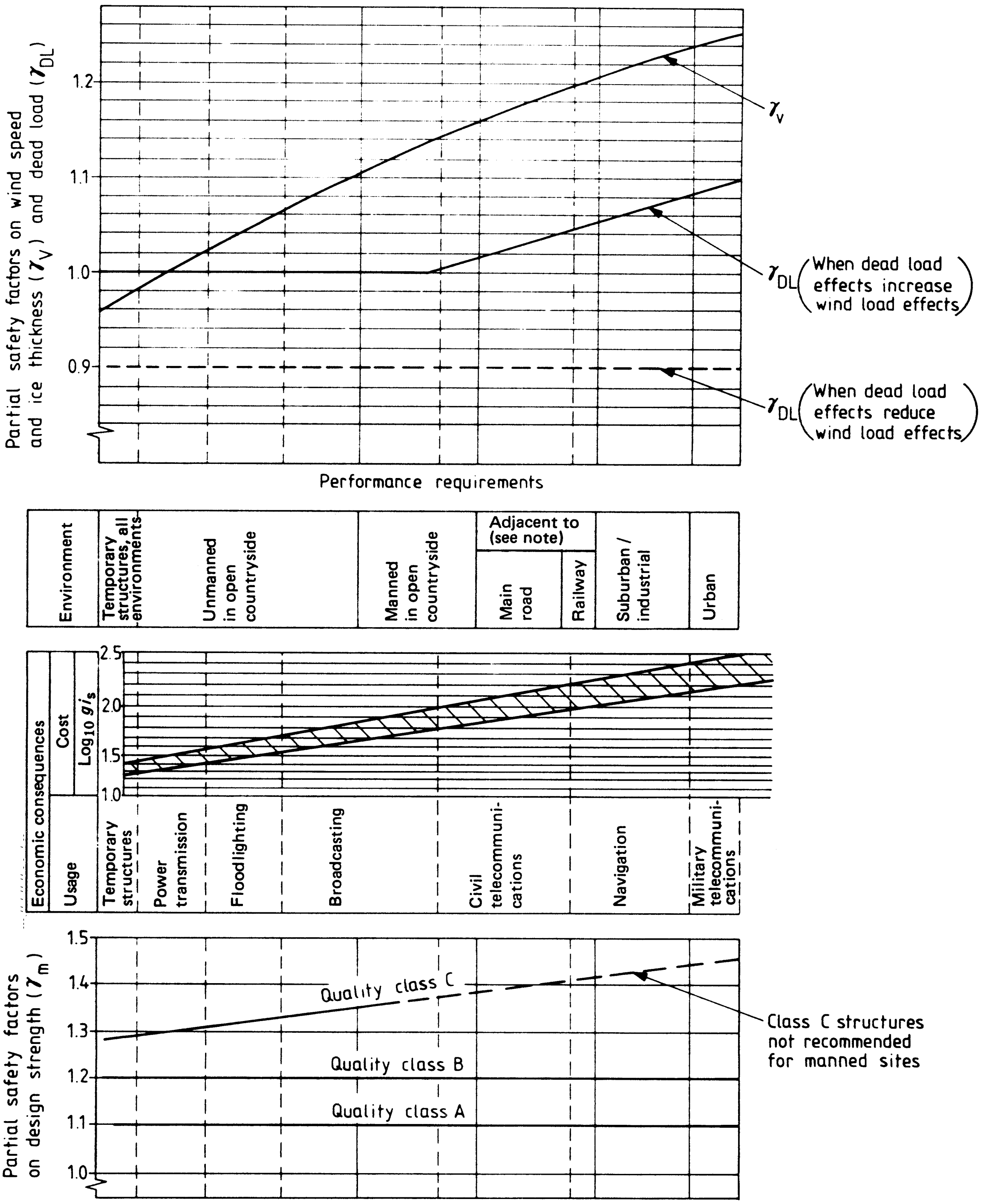
2.2.3.1 *General.* Where the potential risk to life is small, e.g. unmanned towers in open countryside, or the economic consequences of failure are great, e.g. a major link in a telecommunications network, the safety factors should be selected with regard to the potential cost in the event of collapse.

2.2.3.2 *Economic consequences of failure.* The potential total cost, at net present value, of failure within the design service life should be estimated. This should include the cost of removal and replacement of the tower and its ancillary attachments and all contingent costs such as loss of revenue, third-party claims and loss of amenity. The ratio, g, of this consequential cost to the initial cost of the tower should then be evaluated.

Figure 2.1 indicates categories of the potential economic consequences of failure, represented as the logarithm of gi_s where i_s is the design service period.

2.2.3.3 *Usage.* If the economic consequences of failure cannot be judged, the reliability should be selected on the basis of the usage of the tower as indicated in Figure 2.1.

NOTE The left-hand limit of the range shows the minimum factors of safety that should be used; higher factors than those given by the right-hand side of the range indicated may be appropriate.



NOTE Adjacent to is defined as within a distance less than the height of the tower.

Figure 2.1 — Partial safety factors on wind speed, ice thickness, dead load and design strength

2.3 Classification of quality

2.3.1 General

The reliability of a tower depends in part on the quality of the materials and workmanship used in its construction and on the adequacy of the maintenance after its erection. It also depends on the degree of control checking of the design and installation. The quality categories to be adopted for use with this code are given in **2.3.2** to **2.3.4**, inclusive. The method of assessment given in Appendix F may be used as a guide to determine the appropriate quality classification.

2.3.2 Class A towers

Towers may only be considered to be of quality class A if they comply with all of the following conditions.

a) The structural design is in accordance with this code and DD 133[1)].

b) The design and detailing are subject to a comprehensive independent appraisal or, alternatively, subjected to type testing followed by appropriate modifications to the design or details if failure occurs at loads below those given in this code.

c) The materials of construction and quality control testing are in accordance with the appropriate British Standards[1)]. Rigorous identification procedures should also be included to ensure that components are of the required grades.

d) The workmanship in fabrication and erection is in accordance with the appropriate British Standards[1)] and to the satisfaction of the designer or appraiser.

e) The components are subject to inspection after fabrication and their assembly checked independently of the fabricator on completion of erection and any parts rectified if they do not comply with the required level of workmanship.

f) The tower is subject to comprehensive inspection at agreed intervals to check for damage, bolt loosening, weld cracking, corrosion or any other deterioration and any deficiencies rectified in order to maintain the structures in good condition.

NOTE For classes A and B, the interval between inspections should not be greater than 2 years and 5 years, respectively.

2.3.3 Class B towers

Towers may be considered to be class B if they only comply with the conditions given in a) and c) to f) of **2.3.2**.

2.3.4 Class C towers

Towers should be considered to be class C if they only comply with the conditions given in a), c) and d) of **2.3.2**.

2.4 Safety factors

2.4.1 Wind speed and ice thickness factors

Partial safety factors γ_v should be applied to both wind speed and ice thicknesses in accordance with **3.1** and **3.5**, respectively. An appropriate value should be selected by reference to Figure 2.1 using the greater value obtained from the most onerous performance requirement, i.e. according to environment or economic consequences/usage.

When spectral analytical procedures are used in accordance with **5.4**, γ_v may be factored by 0.97.

2.4.2 Dead load and partial factors

2.4.2.1 *Dead load factors.* Partial safety factors γ_{DL} should be applied to the self weight of the structure including ancillary attachments (see **2.4.2.2**). These should be selected by reference to Figure 2.1 appropriate to the performance requirements for the wind speed and ice thickness factors (see **2.4.1**). When the dead load factors are additive to wind load effects, the factors are given by full lines. For conditions where the dead load effects reduce the wind load effects, the lower values given by the dotted lines should be used.

[1)] Towers designed in accordance with other relevant British Standards or other approved standards, may be deemed to be in accordance with this code, provided the factors of safety given in **2.5** are used in a), and compatible standards are used in a), c) and d).

2.4.2.2 *Calculation of dead loads.* The self weight of the tower, including ladders and platforms, should be based on nominal sizes of members, making an appropriate allowance for gussets, bolts and welds.

The self weight of ancillaries, including aerials, feeders, lighting, insulators, etc., should be assessed according to whether or not ancillary load effects increase or decrease wind load effects. Maximum ancillary dead loads predicted to occur during the design life of the structure should be used when such load effects are additive to the wind load effects. Minimum ancillary dead load effects should be used when their effects reduce the wind load effects.

2.5 Safety assessment

In assessing the safety of the design of a tower under extreme conditions, the tower loading derived from sections 3 to 5 should be used to determine the forces in each component of the tower. The forces so obtained should not exceed the design strength of any of the components for the required reliability to be achieved by the design.

The design strength should be taken as the characteristic strength as given in DD 133, divided by the partial safety factor γ_m.

The characteristic strength given in DD 133 is taken for the purposes of this code as the value of strength having a probability of 0.05 of not being achieved by the component in a class A quality tower.

The partial safety factors, γ_m, should be selected from Figure 2.1 appropriate to:

a) the quality class of the tower;

b) the performance requirements used for the derivation of partial safety factors on loading (see **2.4**).

Alternative methods for deriving design strengths of components not covered by DD 133, may only be used where the strengths can be shown to be characteristic strengths derived on the same basis as taken for DD 133.

Additionally, the structure should be designed such that fatigue damage will not lead to failure during the design service period; a basis for fatigue life assessment is given in **3.4**.

2.6 Serviceability

When the performance of any system supported by a tower is dependent on the magnitude of movement of the structure, reliability of service should be related to the fraction of the annual operating period during which the performance is predicted to be unsatisfactory. The loadings to be assumed in calculating deflections should be those defined in **5.2.5** and the periods of unserviceability due to excessive movement may then be estimated by application of **3.3**.

Section 3. Meteorological parameters

3.1 Wind speed data

3.1.1 General

The atmospheric environment in which a tower is to be built may be considered for the purposes of design to be of one of the following types.

Type (a). Regions that may be considered to be well-conditioned, i.e. temperate climates, in conditions of atmospheric neutral stability and where extremely localized and intense storms can be ignored in design [but see type (c)]. The wind speed to be used in determining the design wind forces to be applied at any level of a tower located in such an environment is dependent upon the basic wind speed, the wind direction, the terrain roughness and the variation of wind speed with height. The speed at the reference level at the site should be determined in accordance with **3.1.2** to **3.1.5**, and its variation with height in accordance with **3.2**.

Type (b). Regions subject to hurricanes or typhoons. Design wind speeds should be similarly determined as for type (a), except that reference speeds should only be derived from the records of appropriate meteorological stations in the same region used in combination.

Type (c). Regions where there is a risk of tornadoes or other local intense storms which need to be considered in design. Wind speeds cannot be reliably predicted by the procedures of **3.1.2** to **3.1.5** and they should be derived by assessment of local records which have included such winds.

NOTE The safety factors given in **2.4.1** may be inappropriate for types (b) and (c), where higher values may be necessary to achieve the required reliability.

3.1.2 Basic wind speed

The basic wind speed, $\bar{V}_B$, should be obtained from wind maps based on Meteorological Office data of the maximum mean hourly wind speed independent of direction at a height 10 m above level ground in assumed basic open terrain category III (see Table 3.1) at the site of the structure, and having an annual probability of occurrence of 0.02 (that is a return period of 50 years). The appropriate map for the UK is shown in Figure 3.1, adjusted for sea level. For each 100 m above mean sea level (AMSL), the map value should be increased by 10 % to obtain $\bar{V}_B$ at 10 m above the general ground level, i.e. excluding any significant topographic effects which are covered in **3.2.2**.

Alternatively, and in situations where no appropriate official Meteorological Office wind maps are available, $\bar{V}_B$ should be derived by statistical analysis assuming an extremal distribution based on at least 15 annual maximum mean hourly wind speeds recorded at suitable locations, as near as possible to the site (see Appendix A). When less than 15 years of records are available, $\bar{V}_B$ should be taken as equal to 1.5 times the average value of annual maximum mean hourly wind speeds obtained from at least 10 years of records. Guidance on the statistical analysis of such records and advice on their interpretation is given in Appendix A.

3.1.3 Wind direction factor

Where the structure provides resistance to the wind varying with wind direction or has marked variation in strength in different directions or when considering combinations of wind with ice [see **3.5.1** b)], allowance may be made for the variation of wind speed with direction by use of the factor K_d, which may be derived as follows.

a) For a site in the UK, K_d may be obtained from Figure 3.2 subject to the following.

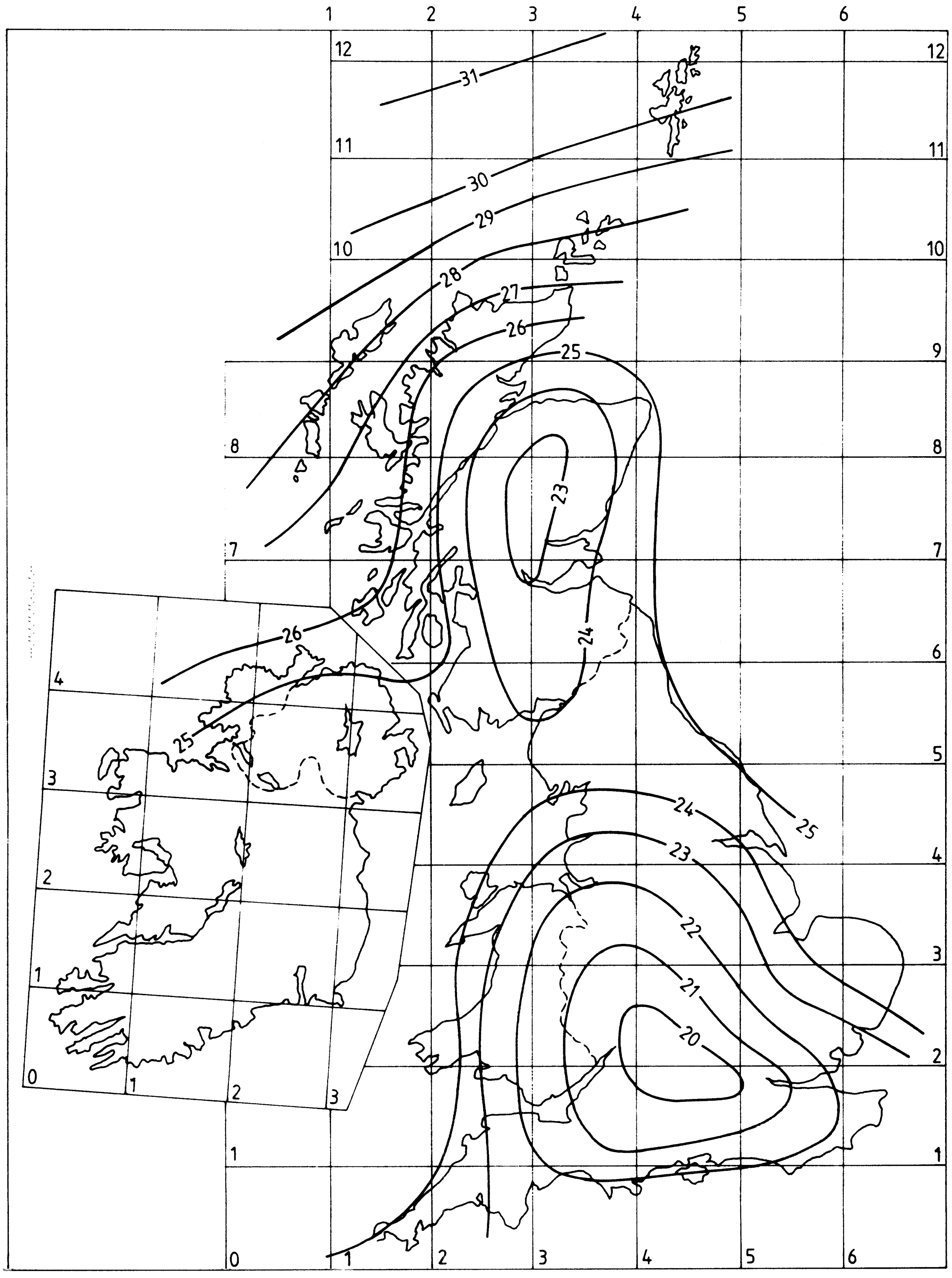
1) K_d = 1.0 for sites within 16 km of the east coast, for ice-free conditions.

2) K_d is not greater than 0.85 when considering combinations of wind and ice.

3) K_d appropriate to any required wind direction should be taken as that within ± 30° of the direction assumed in order to allow for local deviations at particular sites.

4) Consideration should be given to the use of an increased value of K_d where the terrain adjacent to the site contains steep-sided valleys or excavations which may cause funnelling of the wind from certain directions.

b) K_d may be derived from the statistical analysis of records taken at the site for wind direction. See guidance in Appendix A.



NOTE 1 Values are at sea level in basic open terrain, category III (see Table 3.1).

NOTE 2 Add 10 % per 100 m altitude AMSL to obtain $\overline{V}_B$.

NOTE 3 Values are in metres per second.

Figure 3.1 — Basic mean hourly wind speeds for UK, $\overline{V}_B$ (metres per second)

3.1.4 Terrain roughness factor

The terrain roughness factor, K_R, which allows for the general roughness of the ground at the site and its environs, should be derived in either of the following ways.

a) From Table 3.1, appropriate to the category of the site. Consideration should be given to foreseeable alterations to the environs of the site which could change the terrain characteristics. The site reference wind speed, $\overline{V}_r$, is to be assumed to apply at a level above ground of $(10 + h_e)$ metres where h_e is the effective height of surface obstructions appropriate to the terrain as given in Table 3.1.

b) K_R may be derived from the statistical analysis of records taken at the site in accordance with Appendix A.

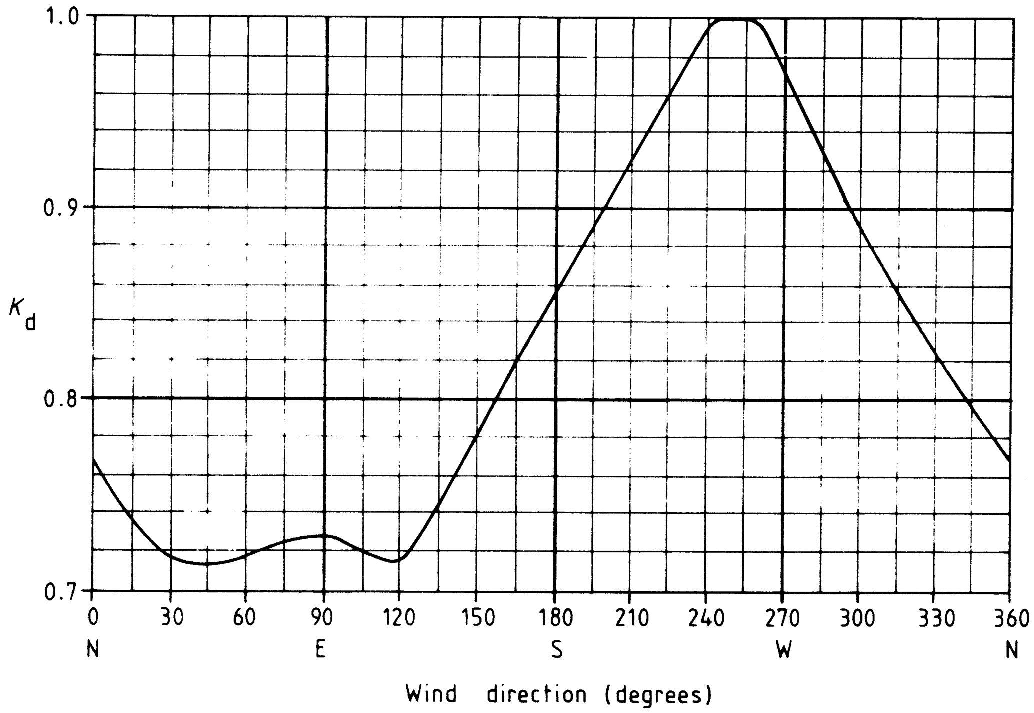


Figure 3.2 — Wind direction factor, K_d

3.1.5 Site reference wind speed

The site reference wind speed, $\overline{V}_r$, is defined as the mean hourly wind speed at the site at a level of 10 m above the effective height of surface obstructions appropriate to the site terrain (see Table 3.1).

It is given by:

$$\overline{V}_r = \gamma_v K_d K_R \overline{V}_B$$

where

$\overline{V}_B$ is the basic wind speed, determined in accordance with **3.1.2**;

γ_v is the partial safety factor on wind speed to be determined from Figure 2.1, appropriate to the type of structure (see **2.4.1**);

K_d is the wind direction factor, determined in accordance with **3.1.3**;

K_R is the terrain roughness factor, determined in accordance with **3.1.4**.

In situations where the basic wind speed, $\overline{V}_B$, cannot be derived in accordance with **3.1.2** and where the nearest meteorological stations are remote from the site, the site reference wind speed should be derived from the gradient wind speed $\overline{V}_g$ in accordance with Appendix C. This procedure is approximate and should not normally be used for sites in the UK where wind map data is available.

For serviceability requirements (see **3.3**) the characteristic wind speed, $\bar{V}_k$, is required, which is defined as:

$$\bar{V}_k = K_R \bar{V}_B$$

Table 3.1 — Terrain characteristics

Category	Terrain description	Terrain roughness factor, K_R	Power law index of variation of wind speed with height, α	Effective height, h_e
				m
I (z_o = 0.003 m)	Snow-covered flat or rolling ground without obstructions; large flat areas of tarmac; flat coastal areas with off-sea wind	1.20	0.125	0
II (z_o = 0.01 m)	Flat grassland, parkland or bare soil, without hedges and with very few isolated obstructions	1.10	0.14	0
III (z_o = 0.03 m)	*Basic open terrain* Typical UK farmland, nearly flat or gently undulating countryside, fields with crops, fences or low hedges, or isolated trees	1.00	0.165	0
IV (z_o = 0.10 m)	Farmland with frequent high hedges, occasional small farm structures, houses or trees	0.86	0.19	2
V (z_o = 0.30 m)	Dense woodland, domestic housing typically covering 10 % to 20 % of the plan area	0.72	0.23	10

NOTE 1 z_o is the terrain aerodynamic roughness parameter (see Appendix B).

NOTE 2 The lower (smoother) of any two possible categories should be adopted where the environs of the site are difficult to define or may change.

NOTE 3 The terrain descriptions should apply to environs extending several kilometres upwind from the site.

NOTE 4 Higher (rougher) categories that occur within only a few kilometres upwind from the site, may not be sufficiently extensive to develop an equilibrium wind profile and should not generally be used as a basis for determining the terrain category.

NOTE 5 In urban areas ($z_o \simeq 0.8$ m), where towers rise above the general level of the surrounding buildings, category V should be adopted. Specialist advice should be sought where considerations of local accelerations from adjacent high buildings could affect the tower design.

3.2 Variation of wind speed with height

3.2.1 Sites on level terrain

For all sites on level terrain, i.e. other than those on hills which are covered by **3.2.2**, the mean wind speed, $\bar{V}_z$, at a height z metres above the site ground level should be taken as:

$$\bar{V}_z = \bar{V}_r \left(\frac{z - h_e}{10}\right)^{\alpha} \quad \text{for } z \geqslant 10 + h_e$$

$$\bar{V}_z = \frac{\bar{V}_r}{2}\left(1 + \frac{z}{10 + h_e}\right) \text{for } z < 10 + h_e$$

where

- $\bar{V}_r$ is the site reference wind speed, determined in accordance with **3.1.5**;
- α is the power law index of variation of speed with height to be obtained from Table 3.1, appropriate to the site terrain;
- h_e is the effective height of surface obstructions to be obtained from Table 3.1, appropriate to the site terrain.

For sites of intermediate roughness, α and h_e should be interpolated on the basis of the value of K_R by reference to Figure 3.3.

NOTE For simplicity, the power law variation of wind speed with height has been used, the log law variation being correctly wind speed dependent. The differences between the two are negligible for designs within the height constraints of this code.

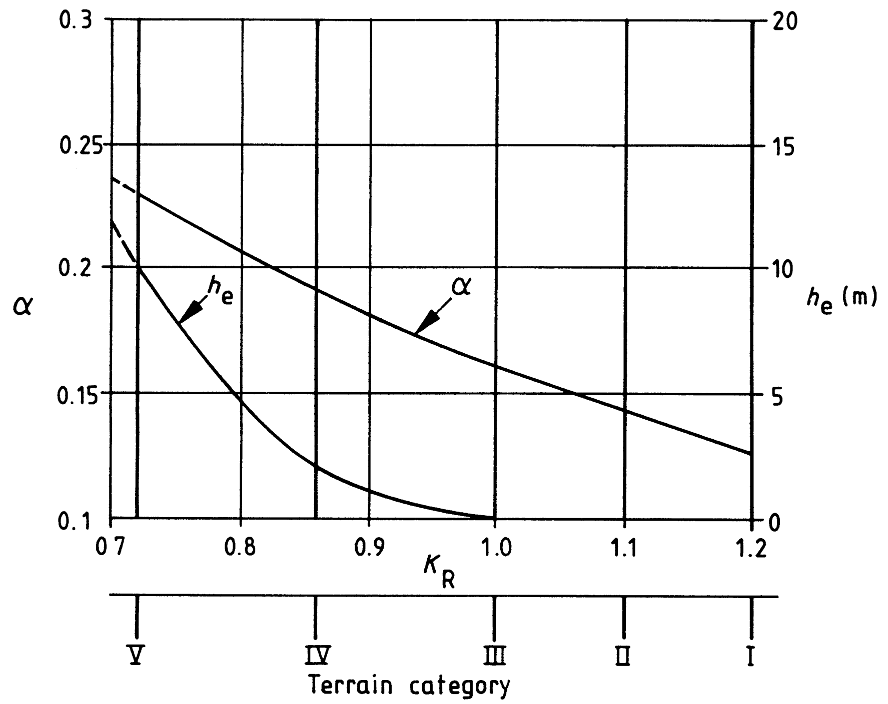


Figure 3.3 — Variation of power law index and effective height with terrain roughness

3.2.2 Sites on hills

3.2.2.1 *Sites on tops of hills.* For sites on the tops of hills of height greater than one-twentieth of the horizontal distance from the hill top to the general level of surrounding terrain in the upwind direction, the mean wind speed, $\bar{V}_z$, at a height z metres above the site ground level should be taken as:

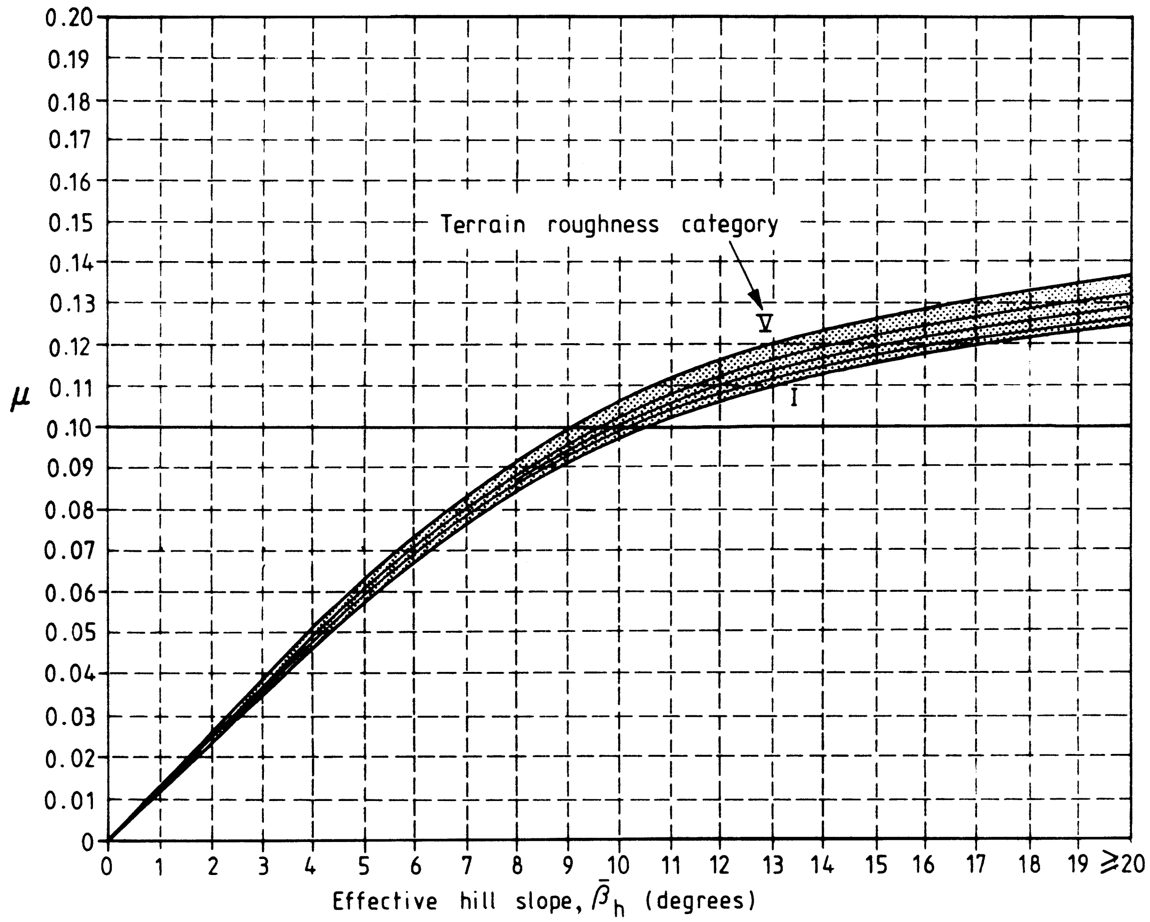
$$\bar{V}_z = \bar{V}_r K_\mu \left(\frac{z - h_e}{10}\right)^{\alpha - \mu} \quad \text{for } z \geqslant 10 + h_e$$

$$\bar{V}_z = \frac{\bar{V}_r}{2} K_\mu \left(1 + \frac{z}{10 + h_e}\right) \quad \text{for } z < 10 + h_e$$

where

- $\bar{V}_r$ is the site reference wind speed, determined in accordance with **3.1.5**;
- μ is a terrain profile index to be obtained from Figure 3.4, appropriate to the mean inclination to the horizontal, $\bar{\beta}_h$, of the upwind slope of the hill and the terrain roughness category;
- K_μ is a terrain profile factor to be obtained from Figure 3.5, appropriate to the height, H_h, of the hill above the general level of the upwind terrain and the terrain roughness category;
- α and h_e are as defined in **3.2.1**.

NOTE Where the contours of a hill are such that its mean slope in any direction radial to the site does not differ by more than 5° within ± 90° of the considered wind direction, the value for K_μ may be reduced by 10 %, but not to a value less than 1.0.



NOTE For basis of curves see Appendix G.

Figure 3.4 — Terrain profile index, μ, for hills

3.2.2.2 *Sites downwind from the crest of a hill.* For sites located downwind from the crest of a hill by a distance x not exceeding 18 H_h, the mean wind speed, $\bar{V}_z$, at height z metres above the site ground level should be taken as:

$$\bar{V}_z = \bar{V}_r K_{\mu x} \left(\frac{z - h_e}{10}\right)^{\alpha - \mu_x} \quad \text{for } z \geqslant 10 + h_e$$

$$\bar{V}_z = \frac{\bar{V}_r}{2} K_{\mu x} \left(1 + \frac{z}{10 + h_e}\right) \quad \text{for } z < 10 + h_e$$

where

$K_{\mu x}$ is a factor given by:

$$K_{\mu x} = K_\mu - \frac{x}{18H_h} (K_\mu - 1);$$

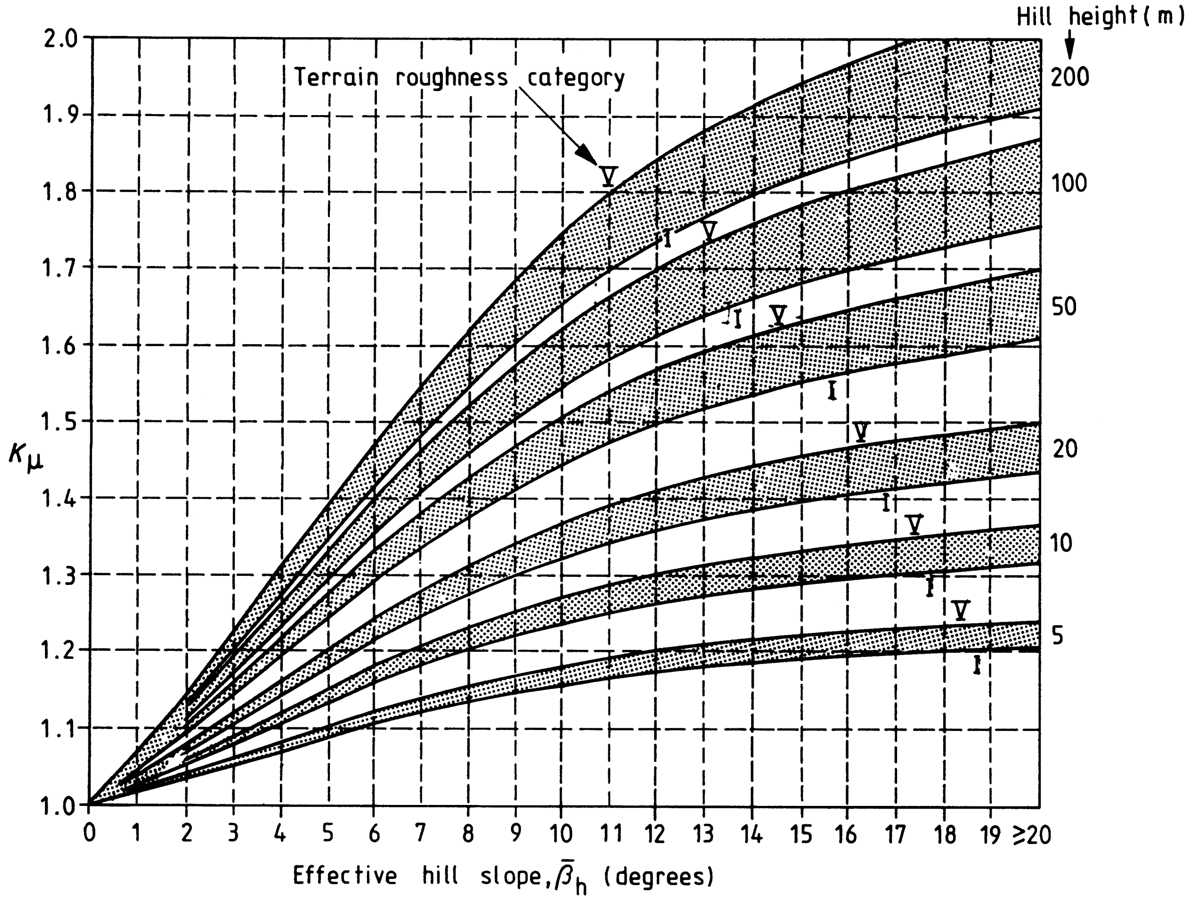
μ_x is the terrain profile index at a distance x from the crest of the hill given by:

$$\mu_x = \mu \left(1 - \frac{x}{18\,H_h}\right);$$

$\overline{V}_r$, μ, K_μ are as defined in **3.2.2.1**;

α and h_e are as defined in **3.2.1**.

Sites located at a distance more than 18 H_h from the crest are unaffected by the accelerated flow over the hill and the variation of wind speed with height in accordance with **3.2.1** should be adopted.



NOTE For basis of curves see Appendix G.

Figure 3.5 — Terrain profile factor, K_μ, for hills

3.2.2.3 *Sites located on the upwind side of a hill.* For sites located on the upwind side of a hill, the mean wind speed, $\overline{V}_z$, should be derived in accordance with **3.2.2.1**, but H_h should be taken as the height of the site above the surrounding terrain on the upwind side of the hill irrespective of the wind direction under consideration.

3.2.2.4 *Sites located in undulating terrain.* For sites located in regions of undulating terrain, methods of assessing the height, H_h, the mean inclination to the horizontal of the hill, $\bar{\beta}_h$, and the distance downwind from the crest, x, are given in Appendix D.

3.3 Serviceability

3.3.1 Distribution of wind speeds

For the purpose of assessing the duration of unserviceability due to excessive deflection and for the estimation of fatigue life in accordance with **3.4**, the procedure set out in **3.3.2** should be adopted for sites in the UK.

Alternatively, where appropriate records are available the statistical distribution of mean hourly wind speeds appropriate to the site should be obtained by analysis of the wind records over a period of at least 15 years obtained in open terrain as near as possible to the site. Guidance on the statistical analysis of such records is given in Appendix A.

3.3.2 Downwind deflections

When assessing the compliance of the design with any limitations on downwind deflection, the serviceability limit mean wind speed, $\bar{V}_s$, expressed as a ratio of the characteristic wind speed, $\bar{V}_k$, independent of direction should be derived as follows:

$$\frac{\bar{V}_S}{\bar{V}_k} = \gamma_v \sqrt{\frac{\Delta_S}{\Delta}}$$

where

$\bar{V}_k$ is as defined in **3.1.5**;

γ_v is to be obtained from Figure 2.1 (see **2.4.1**);

Δ is the deflection, determined in accordance with **5.2.5**;

Δ_S is the specified serviceability limit deflection.

When the limiting deflection applies independently of direction, the corresponding annual duration of unserviceability may then be obtained from Figure 3.6 as the number of hours that the serviceability limit speed is expected on average to be exceeded.

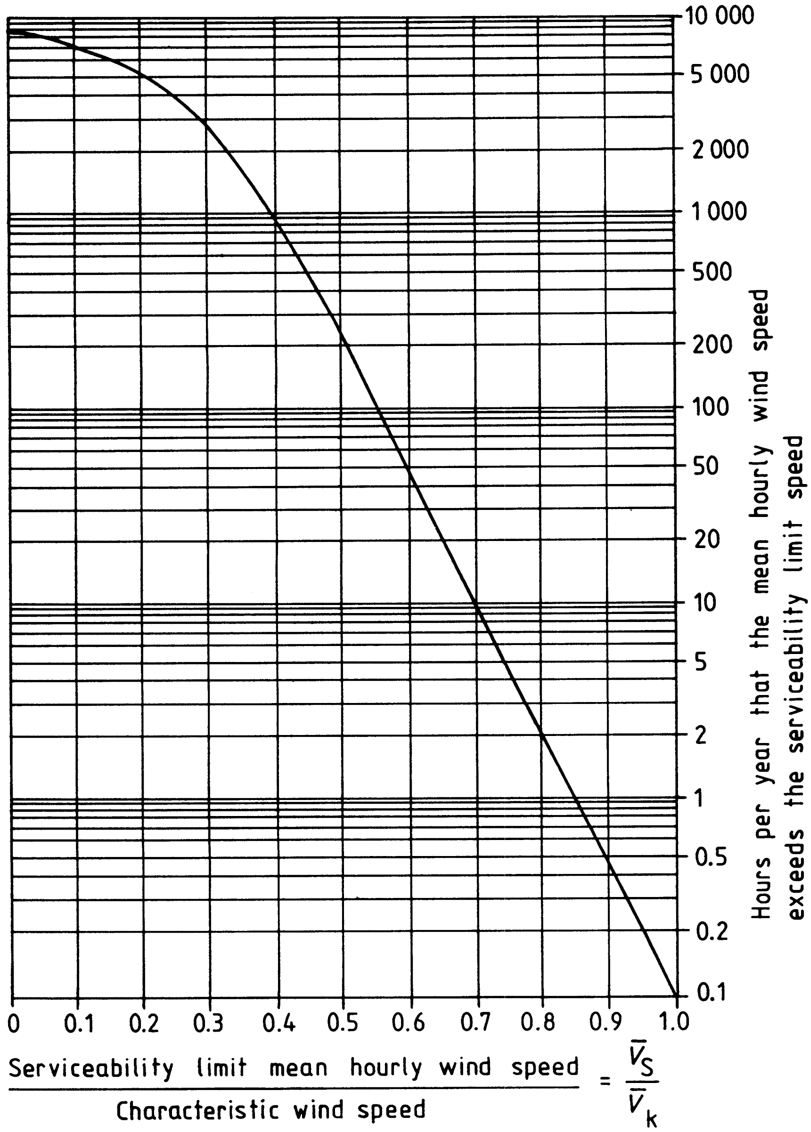


Figure 3.6 — Hours per year that the mean hourly wind speed exceeds the serviceability limit

When the deflection limitation is confined to a certain direction, the serviceability limit wind speed within each 30° sector producing a deflection of Δ_S in that direction, should be calculated and a corresponding annual duration of unserviceability for each sector found from Figure 3.6. Each period should then be multiplied by the duration factor associated with this sector from Figure 3.7, the summation of these factored periods gives the total annual duration of unserviceability in the specified direction.

3.3.3 Crosswind deflections (vortex shedding)

The serviceability limit mean wind speeds for assessing performance when crosswind vibrations due to vortex excitation are predicted should be taken as the critical wind speed, V_{cr}, defined in **5.5**. If the predicted amplitudes of movement are in excess of the limits of satisfactory performance, the duration of unserviceability should be estimated by reference to Figure 3.8. This shows the annual periods within any 30° sector in which the wind speed is within ± 2.5 % of a given ratio of the serviceability wind speed, $\bar{V}_s$, to the characteristic wind speed, $\bar{V}_k$, for any assumed direction of wind.

Alternatively, when vibrations may occur in any direction at the same critical wind speed and the serviceability requirements are independent of direction, the total annual period for the same speed may be taken as 6.5 times the corresponding period in the 240° segment obtained from Figure 3.8.

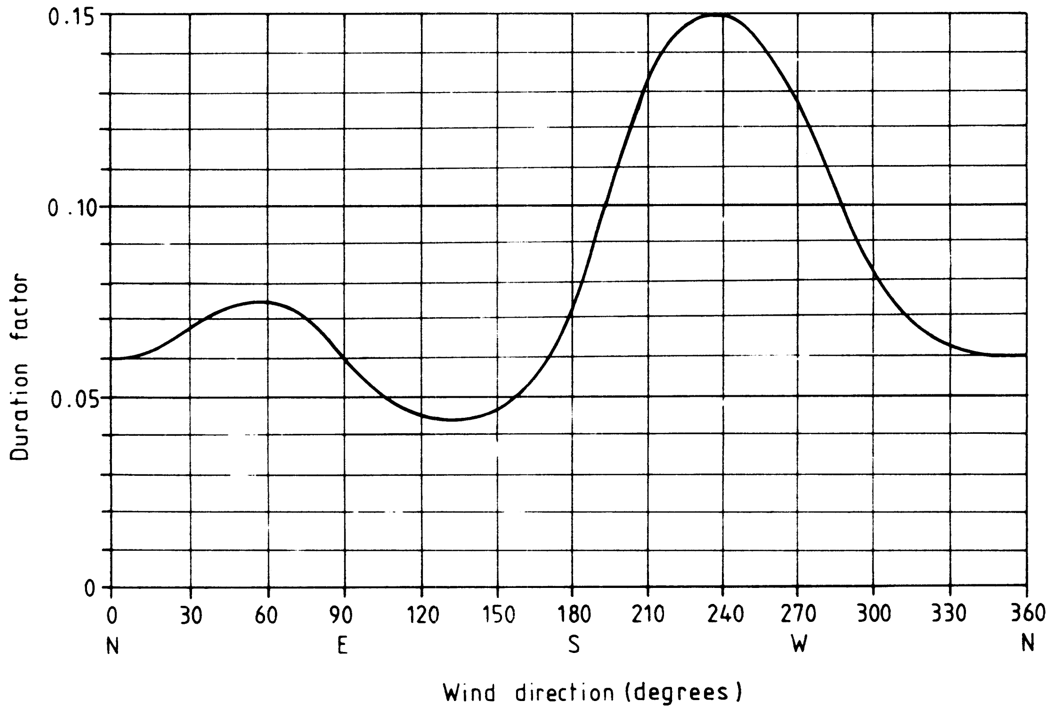


Figure 3.7 — Wind speed duration factor

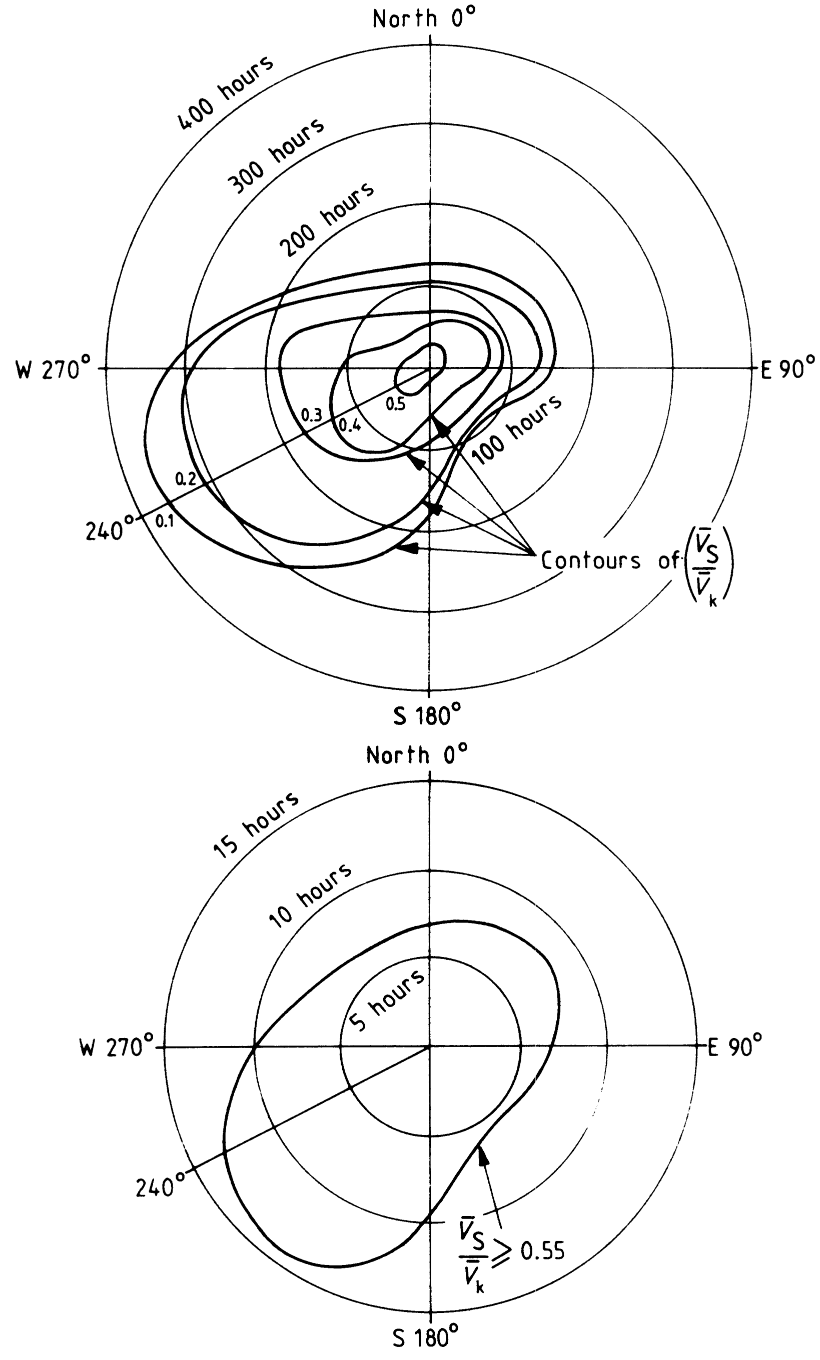


Figure 3.8 — Expected average annual occurrence of mean wind speeds as a function of direction

3.4 Fatigue life assessment

3.4.1 In-line vibrations

For towers entirely of bolted or riveted construction of steel of up to 450 N/mm^2 yield stress, the fatigue life under in-line wind loading in the absence of vortex-excited vibration (see **5.5**) may be assumed to exceed 50 years.

For towers containing welded details, the fatigue life should be evaluated when either:

a) the design life is greater than 30 years; *or*

b) the tower is constructed of steel of yield stress greater than 355 N/mm^2.

However, caution should be exercised in the design of all welded structures and due account taken of the welded details adopted.

For towers constructed of other metallic materials, specialist advice should be sought with regard to fatigue life assessment.

The fatigue stress history due to wind gusts may be evaluated by using Figure 3.6 and Figure 3.7 to determine the annual durations of different mean wind speeds from different directions. The fluctuations about the mean values may then be assumed to have a statistically normal distribution with a standard deviation in stress corresponding to $G/4$ times the stress due to the mean speed, where G is the appropriate gust response factor derived in accordance with section 5.

3.4.2 Crosswind vibrations

3.4.2.1 *Overall response.* Crosswind amplitudes should be calculated in accordance with **5.5** for towers supporting cylindrical aerials or stacks. For each mode considered, the duration of such vibrations should be estimated as described in **3.3.3**. The resulting duration of each mode of vibration, within the design life of the tower, should then be used to assess the accumulated fatigue damage, using *S*/*N* curves appropriate to the class of detail, in accordance with DD 133.

3.4.2.2 *Individual member response.* For structures composed of welded circular sections, consideration should be given to the possibility of crosswind excitation of individual members. The critical wind speed and amplitude of the fundamental mode of vibration may be calculated in accordance with **5.5** and the fatigue assessment undertaken as for overall response.

3.5 Ice loading

3.5.1 General

For design against ice loading, two combinations should be considered using the appropriate reference ice thickness and density.

a) *Extreme icing in the absence of wind.* The additional weight of ice should be determined in accordance with **3.5.2** to **3.5.4**.

NOTE This will seldom govern the design of a free-standing tower.

b) *Combination of ice with wind.* The additional weight of ice should be determined in accordance with **3.5.2** to **3.5.4**. The wind resistance (see **4.8**) should be based on projected areas increased to allow for a uniform coating of the reference ice thickness, r_r, determined in accordance with **3.5.2** and **3.5.3**.

The site reference wind speed, $\bar{V}_i$, to be used with ice when determining the design wind loading should be taken as:

$$\bar{V}_i = 0.8\ \bar{V}_r$$

where

$\bar{V}_r$ is the site reference wind speed determined in accordance with **3.1.5**, but with due allowance for the limiting value of 0.85 of the wind direction factor, K_d, in accordance with **3.1.3** a), if appropriate.

The variation of $\bar{V}_i$ with height should be determined in accordance with **3.2** but using $\bar{V}_i$ in place of $\bar{V}_r$.

NOTE At sites where severe icing is known to occur, caution should be exercised in the choice of both wind speed and ice thickness if local records indicate that the specified values are less than those experienced in practice.

3.5.2 Basic ice thickness

3.5.2.1 *Ice thickness in the absence of wind.* The basic ice thickness, r_B, in the absence of wind, for the UK, should be taken as:

$$r_B = k_i \left\{ r_o + \left(\frac{a - 200}{25} \right) \right\} \quad \text{but not less than } k_i\, r_o$$

where

k_i is a coefficient that is *either*:

a) 1.0 for structural sections other than round bars, circular tubes or cables; *or*

b) $\left(\frac{2}{3} + \frac{4}{D}\right)$ but not more than 1.2 for round bars, circular tubes and cables, where D is the diameter of the member (in mm);

a is the altitude of the tower top above sea level (in m);

r_o is the radial ice thickness in the absence of wind to be obtained from Figure 3.9 (in mm), appropriate to the position of the site. Alternatively, r_o may be derived from a statistical analysis assuming an external distribution based on records of the annual maximum thicknesses of ice formation on components of form and size similar to those to be used in the tower or its attachments at the latitude and altitude of the site and having an annual probability of occurrence of 0.02.

3.5.2.2 *Ice thickness in conjunction with wind.* The basic ice thickness, r_B, in conjunction with wind for the UK, should be taken as:

$$r_B = k_i \left\{ r_w + \left(\frac{a - 200}{25}\right) \right\} \quad \text{but not less than } k_i\, r_w$$

where

k_i and a are as defined in **3.5.2.1**;

r_w is the radial ice thickness in conjunction with wind to be obtained from Figure 3.9 (in mm), appropriate to the position of the site. Alternatively, r_w may be derived from records, (see **3.5.2.1**) but having an annual probability of occurrence of 0.5.

3.5.3 Reference ice thickness

The reference ice thickness, r_r, to be considered for design should be taken as:

$r_r = \gamma_v\, K_c\, r_B$

where

γ_v is the partial safety factor to be obtained from Figure 2.1, appropriate to the type of structure (see **2.4.1**);

r_B is the basic radial thickness of ice, determined in accordance with **3.5.2**;

K_C is a cable factor which should be taken as *either*:

a) 1.0 for all tower members and ancillaries; *or*

b) $\frac{N_C + 0.3}{1.3 N_C}$ for all cables supported by the tower, where N_C is the number of cables.

3.5.4 Ice weight

The weight of ice deposited on the tower should be calculated assuming all structural sections and ancillary parts to be uniformly coated in ice thickness r_r, and:

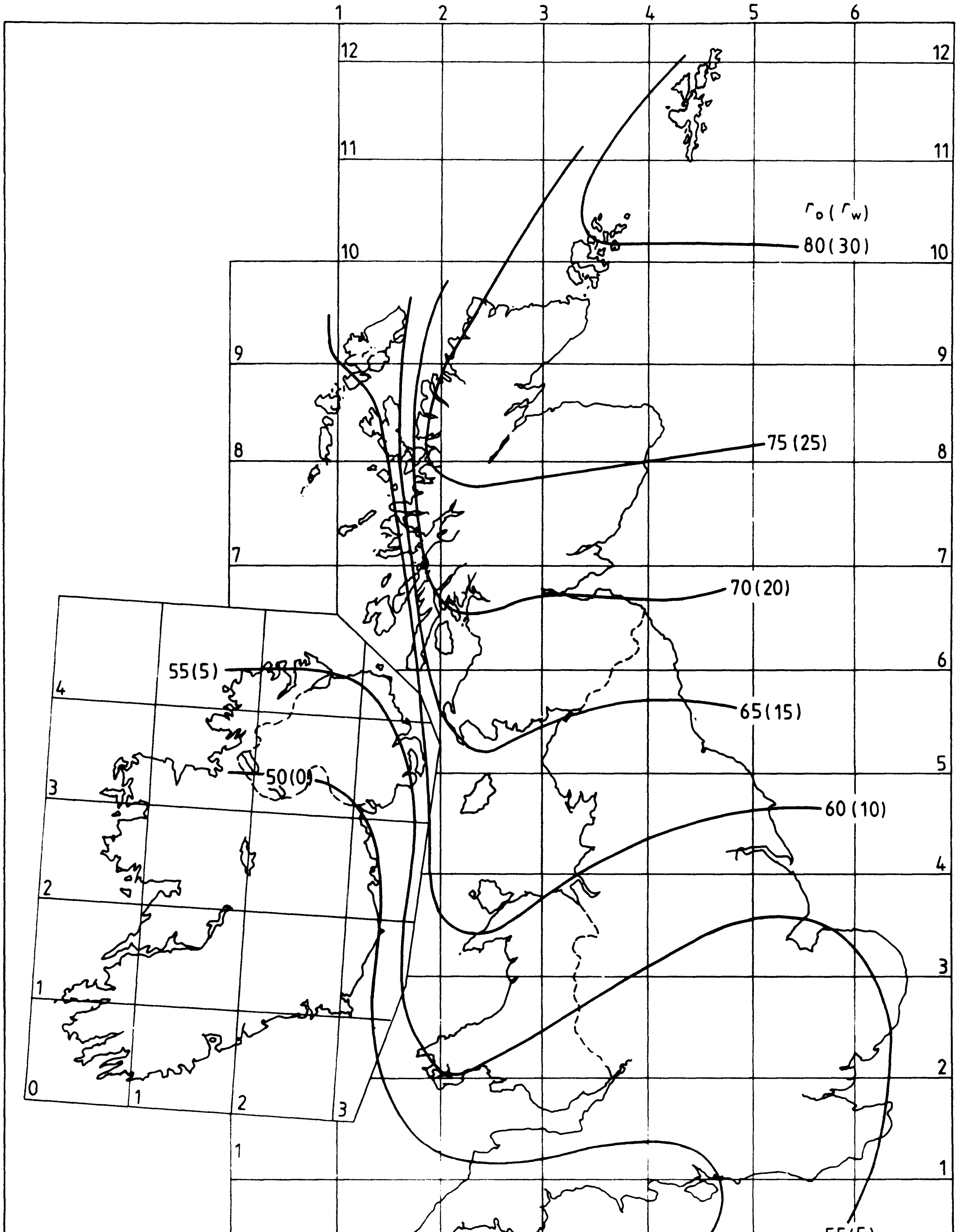
a) the unit weight of ice, in the absence of wind should be taken as 5 kN/m^3;

b) the unit weight of ice, in conjunction with wind, should be taken as:

1) 9 kN/m^3 for design against compression;

2) 5 kN/m^3 for design against uplift;

c) the weight of ice deposited should include an allowance for gaps of less than 75 mm completely filled with ice in accordance with **4.8**.



All thicknesses are in millimetres.

Figure 3.9 — Ice thickness for UK, r_o and r_w (in millimetres)

Section 4. Wind resistance

4.1 General

4.1.1 Method of derivation

The resistance to the flow of wind offered by the assembled components of a tower and by any elements which it supports should be derived from either:

a) the coefficients given in this section; *or*

b) tests on models in wind tunnels under conditions simulating those appropriate to the site and within the range of predicted reference wind speeds, either in smooth flow or, preferably, in scaled turbulent flow.

For the purposes of calculating wind resistance, a tower should be divided into a series of panels, where a panel is taken between intersections of legs and primary bracings and includes any horizontal bracing. Projections of bracing members from faces parallel to the wind direction, and plan and hip bracing, should be ignored in determining the projected area of the structure. A tower should generally be divided into a minimum of 10 panels but in cases where there are a high number of panels, when so divided, panels may be aggregated into heights not exceeding 0.05 of the total height.

4.1.2 Symmetrical towers without ancillaries

For panels in towers of square or triangular form in the absence of ancillary items, the basis of derivation is given in **4.2**.

4.1.3 Symmetrical towers with limited ancillaries

For panels in towers containing ancillary items, the basis for derivation is given in **4.3** provided that for each panel under consideration:

a) the total projected area of those ancillary parts adjacent to the face under consideration is less than the effective area of the structural members in that face (see Figure 4.1);

b) the total projected area normal to any face of the tower of any single internal or external ancillary is less than half the gross area of the face of the panel;

c) any ancillary does not extend more than 10 % beyond the total width of the tower at that level.

NOTE If the ancillaries are reasonably identical on each face, i.e. the projected areas are within 10 % of each other, and are of circular or flat-sided profile, then they may be treated as appropriate structural members and the overall drag factors calculated in accordance with **4.2**.

4.1.4 Other cases

In all other cases, the resistance should be calculated in accordance with **4.4**.

4.2 Method for symmetrical towers without ancillaries

4.2.1 Calculation of total wind resistance

The total wind resistance should be determined in the direction of the wind and in the crosswind direction in accordance with a) and b), respectively, as follows.

a) The total wind resistance, ΣR_W, in the direction of the wind over a panel height of the structural components of a lattice tower of square or equilateral triangular cross section, having equal areas for each face, may be taken as that of the bare tower, R_T, given by:

$$R_T = K_\theta \, C_N \, A_s$$

where

C_N is the overall drag (pressure) coefficient, determined in accordance with **4.2.2**;

A_s is the total area projected normal to a face of the structural components within one panel height of the tower at the level concerned (see Figure 4.1) including icing when appropriate;

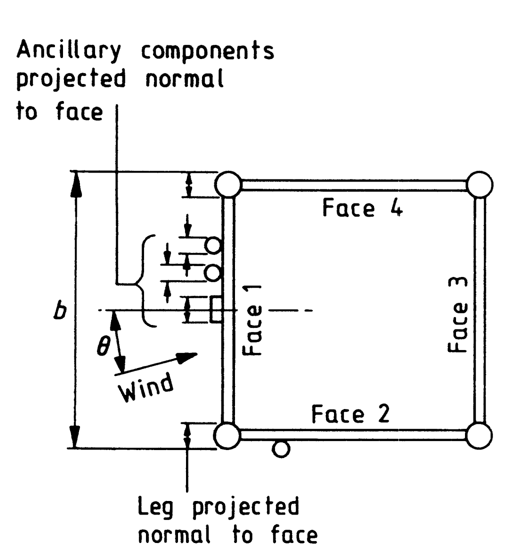
K_θ is the wind incidence factor given in Figure 4.2 for commonly used values of θ. For other values of θ refer to formulae in **G.4.1**, using $A_F = A_s$;

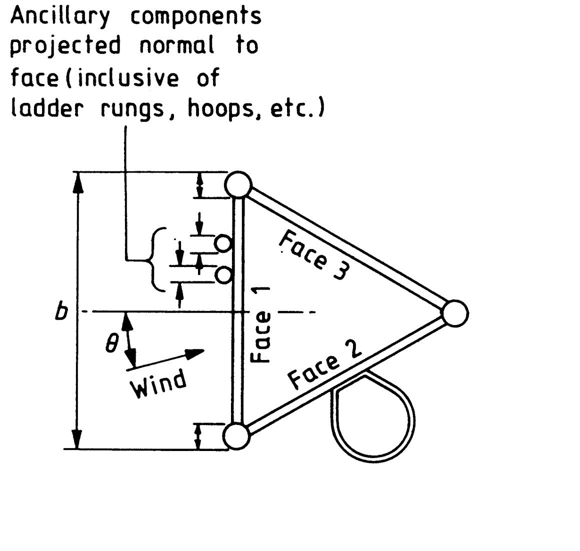
θ is the angle of incidence of the wind to the normal to face 1, in plan; face 1 should be taken as the windward face (see Figure 4.1):

where

A_f is the total projected area, when viewed normal to the face, of the flat-sided section members in the face;

ϕ is the ratio of the total projected area within a panel height of the structural components in the windward face (A_s) visible when viewed normal to the face, to the area enclosed over the panel height by the boundaries of the frame projected normal to the face, both at the level considered (see Figure 4.1).



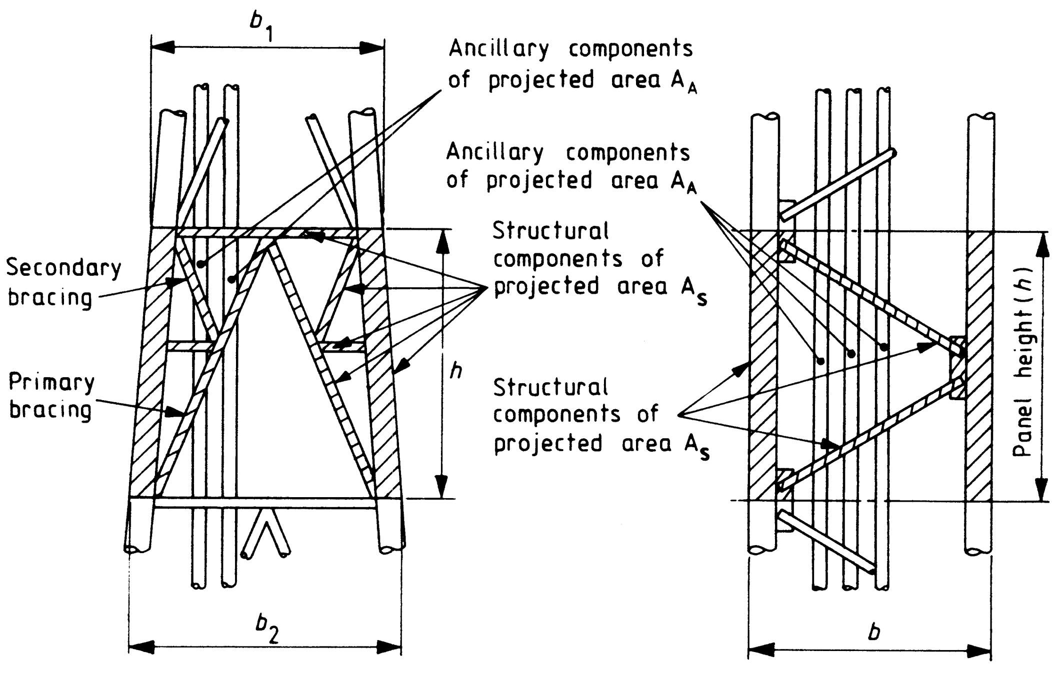


NOTE Face 1 is to be taken as the windward face such that $-45° \leqslant \theta \leqslant 45°$.

(a) Plan on square tower

NOTE Face 1 is to be taken as the windward face such that $-60° < \theta < 60°$.

(b) Plan on triangular tower



(c) View normal to face (square or triangular tower)

(1) For panel with inclined legs

For **4.2** and **4.3**: solidity ratio, $\phi = \dfrac{2A_s}{h(b_1 + b_2)}$

For **4.4**: solidity ratio, $\phi = \dfrac{2(A_s + A_A)}{h(b_1 + b_2)}$

(2) For panel with parallel legs

For **4.2** and **4.3**: solidity ratio, $\phi = \dfrac{A_s}{hb}$

For **4.4**: solidity ratio, $\phi = \dfrac{A_s + A_A}{hb}$

NOTE Structural components of front face shown hatched of projected area A_s, which is equal to A_F in **4.2**.

Figure 4.1 — Projected panel area used to calculate solidity ratio, ϕ

Circular-section members should be assumed to be in a subcritical regime when the effective Reynolds number $R_e \leqslant 4 \times 10^5$ or when iced, and may be assumed to be in a supercritical regime for higher values of R_e only when ice free.

The value of R_e is given by:

$$R_e = \frac{1.5\bar{V}_z D}{\nu}$$

where

$\bar{V}_z$ is the wind speed relevant to the height z from the ground to the centre of the member, determined in accordance with **3.2** (in m/s);

D is the member diameter (in m);

ν is the kinematic viscosity of air = 1.46×10^{-5} m²/s for sites in the UK.

Where supercritical flow is assumed for any or all members, it should be checked that greater loading does not result under a reduced windspeed corresponding to $R_e < 4 \times 10^5$.

b) The total wind resistance crosswind over a panel height, ΣR_X, should be taken as R_T.

4.2.2 Overall drag coefficients

Values of overall normal drag (pressure) coefficients, C_N, applicable to the structural framework of square or equilateral triangular section towers may be derived from Figure 4.3 for square towers and triangular towers, in which the solidity ratio, ϕ, is as defined in **4.2.1**. For such towers composed of both flat-sided and circular-section members, the overall normal drag (pressure) coefficient is given by:

$$C_N = C_{Nf}\frac{A_f}{A_F} + C_{Nc}\frac{A_c}{A_F} + C_{Nc'}\frac{A_{c'}}{A_F}$$

where

C_{Nf}, C_{Nc} and $C_{Nc'}$ are the drag coefficients for towers composed of flat-sided, subcritical circular- and supercritical circular-section members, respectively, to be obtained from Figure 4.3;

A_f is as defined in **4.2.1**;

A_c is the total projected area when viewed normal to the face of the circular-section members in the face in subcritical regimes;

$A_{c'}$ is the total projected area when viewed normal to the face, of the circular-section members in the face in supercritical regimes;

A_F is the total projected area normal to a face, given by:

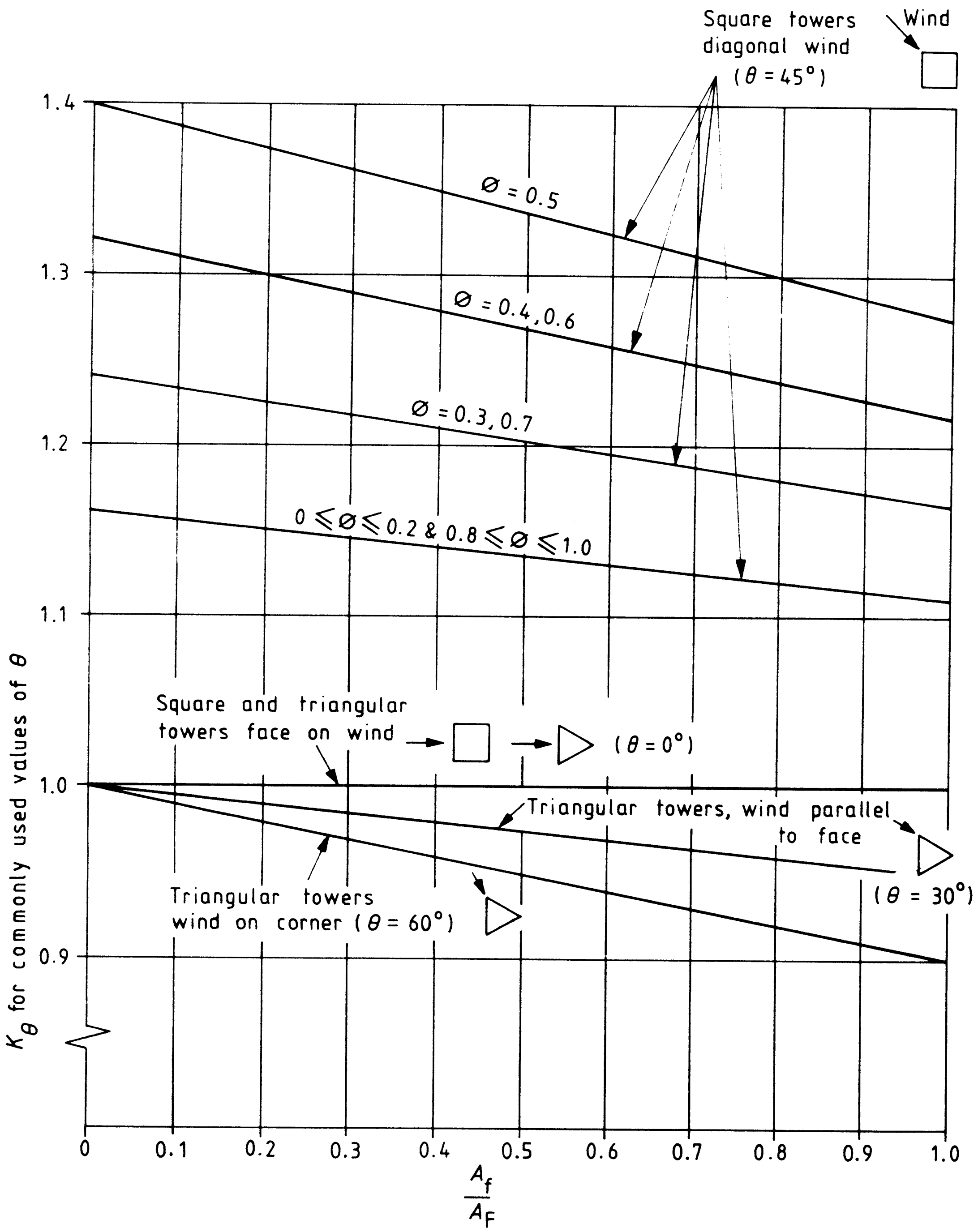
$A_F = A_f + A_c + A_{c'}$

= A (as defined in **4.2.1**) when all components are treated as structural members.

4.3 Method for symmetrical towers with limited ancillaries

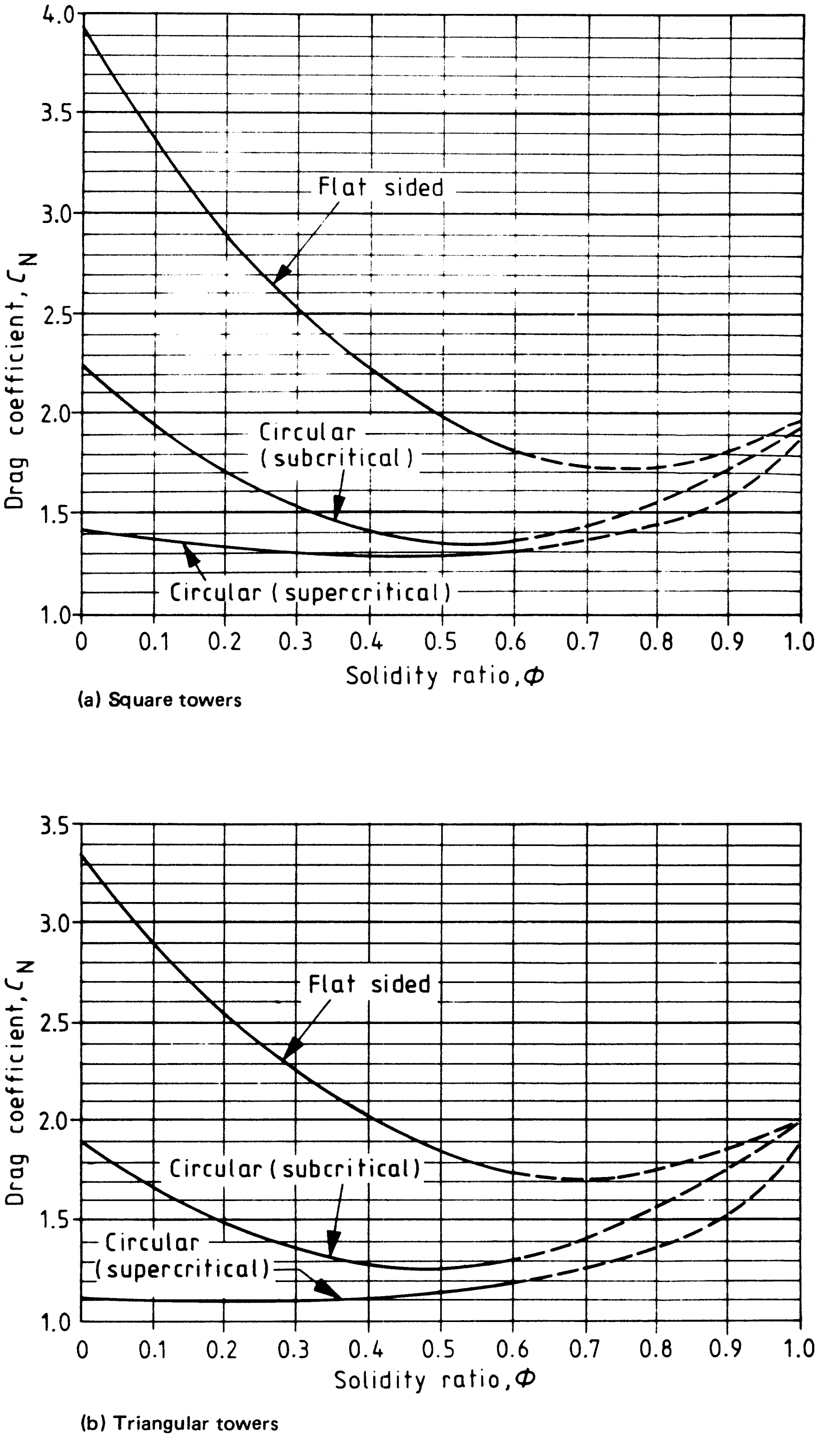
4.3.1 Constraints for simplified method

The calculation of total wind resistance may be determined from **4.3.2** only when the tower panel includes ancillaries that are within the constraints of **4.1.3**. For other cases, the general method given in **4.4** should be used.



NOTE For basis of curves see Appendix G.

Figure 4.2 — Wind incidence factor, K_θ



NOTE 1 For basis of curves see Appendix G.

NOTE 2 For towers with $\phi > 0.6$, the drag coefficients (shown thus – – –) should be verified by model testing in a wind tunnel.

Figure 4.3 — Overall normal drag coefficients, C_N, for square and triangular towers

4.3.2 Calculation of total wind resistance

The total wind resistance should be determined in the direction of the wind and the crosswind direction in accordance with a) and b) respectively, as follows.

a) The total wind resistance, ΣR_W, in the direction of the wind over a panel of a tower should be taken as:

$$\Sigma R_W = R_T + R_{AW}$$

where

R_T is the resistance of the bare tower panel, determined in accordance with **4.2** using the solidity ratio, ϕ, appropriate to the bare structure;

R_{AW} is the wind resistance of the ancillaries, determined in accordance with **4.5** and **4.6**, as appropriate.

b) The total crosswind resistance, ΣR_X, where required, over a panel, should be taken as:

$$\Sigma R_X = R_T + R_{AX}$$

where

R_{AX} is the wind resistance in the crosswind direction of the ancillaries, determined in accordance with **4.5** and **4.6**, as appropriate.

4.4 General method for towers containing ancillaries or unsymmetrical towers

4.4.1 Calculation of total wind resistance

The total wind resistance should be determined in the direction of the wind and the crosswind direction in accordance with a) and b), respectively, as follows.

a) The total wind resistance, ΣR_W, in the direction of the wind over a panel height of a square or triangular tower containing parts or ancillaries outside the constraints of **4.1.3** or of a tower of rectangular unequal sided cross section should be taken as:

$$\Sigma R_W = R_{1e} \cos^2 \theta_1 + R_{2e} \sin^2 \theta_1 \text{ for square and rectangular towers}$$

$$\Sigma R_W = R_{1e} \cos^2 \left(\frac{3\theta_1}{4}\right) + R_{2e} \sin^2 \left(\frac{3\theta_1}{4}\right) \text{ for triangular towers}$$

where

R_{1e} is an effective resistance given by:

$$R_{1e} = (R_1 + \eta_1 R_3)\, K\theta_1 \text{ for square and rectangular towers}$$

$$R_{1e} = \left\{R_1 + \frac{\eta_1}{2}\left(R_2 + R_3\right)\right\} \text{ for triangular towers}$$

R_{2e} is an effective resistance given by:

$$R_{2e} = (R_2 + \eta_2 R_4)\, K_{\theta 2} \text{ for square and rectangular towers}$$

$$R_{2e} = \left\{R_2 + \frac{\eta_2}{2}\left(R_1 + R_3\right)\right\} K_{\theta 2} \text{ for triangular towers}$$

where

R_1, R_2, R_3 and R_4 are wind resistances given by:

$$R_1 = A_{s1}\, C_{n1} + R_{AW1}$$

$$R_2 = A_{s2}\, C_{n2} + R_{AW2}$$

$$R_3 = A_{s3}\, C_{n3} + R_{AW3}$$

$$R_4 = A_{s4}\, C_{n4} + R_{AW4}$$

where

A_{s1}, A_{s2}, A_{s3} and A_{s4} are the areas projected normal to faces 1, 2, 3 and 4, respectively, of the components treated as structural members within the same panel height of faces 1, 2, 3 and 4 including icing, where appropriate, (see Figure 4.1);

C_{n1}, C_{n2}, C_{n3} and C_{n4} are the drag coefficients appropriate to faces 1, 2, 3 and 4, respectively, of the components treated as structural members which may be determined in accordance with **4.4.2**;

R_{AW1}, R_{AW2}, R_{AW3} and R_{AW4} are the wind resistances appropriate to faces 1, 2, 3 and 4, respectively, for the ancillary items not treated as structural members, determined in accordance with **4.5** or **4.6**, as appropriate;

η_1 and η_2 are the effective shielding factors for faces 1 and 2, respectively, including both structural and ancillary components and are to be taken as:

$\eta' + 0.15\ (\omega - 1)\ (\phi - 0.1)$ for square and rectangular towers, but not to be taken as greater than 1.0;

$\frac{2}{3}\ \eta' + 0.15\ (\omega - 1)\ (\phi - 0.1)$ for triangular towers, but not to be taken as greater than 1.0;

where

$\eta' = \eta_f\ (A_f + 0.83A_c + 2.1A_{c'} + A_A)/(A_s + A_A)$ but not greater than 1.0;

η_f is obtained from Figure 4.4 applicable to faces 1 or 2, as appropriate using ϕ as defined in this subclause;

A_f, A_c, $A_{c'}$ are as defined in **4.2.2** applicable to faces 1 or 2, as appropriate;

A_A is the projected area normal to the face of the ancillary items not treated as structural members applicable to faces 1, 2, 3 or 4, as appropriate;

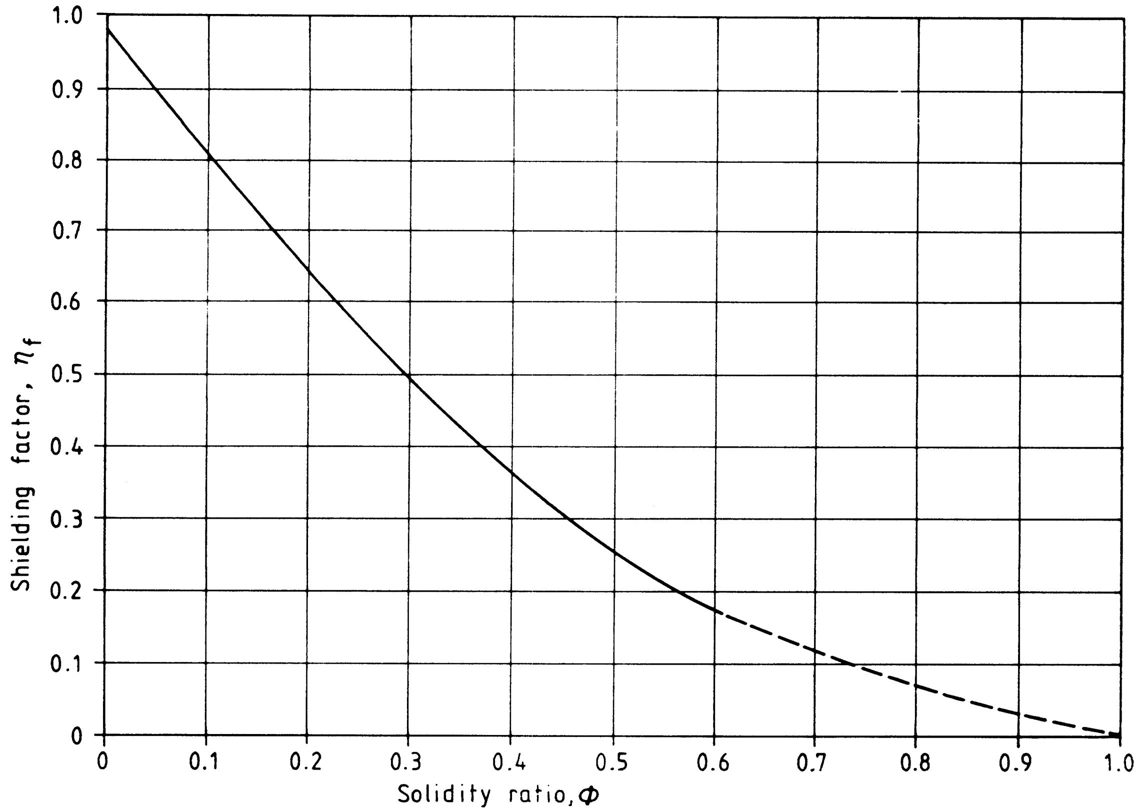
$A_F = A_s + A_A$ where $A_s = A_f + A_c + A_{c'}$;

ϕ is the solidity ratio appropriate to faces 1 and 2, respectively, as defined in **4.2.1**, but including both structural and ancillary components (see Figure 4.1);

ω is the spacing ratio, equal to the distance between the face considered and that parallel to it divided by the width of the face considered at the level of the centroid of the panel area but not to be taken as less than 1.0 and to be taken as 1.0 for triangular towers;

$K_{\theta 1}$ and $K_{\theta 2}$ are to be obtained from Figure 4.2, applicable to faces 1 or 2, as appropriate, using A_F, A_f and ϕ as defined in this subclause and where θ is the plan angle of incidence of wind to the normal to face 1.

b) The total wind resistance crosswind over a panel, R_X, should be calculated according to a) taking the reference direction as normal in plan to the mean wind direction.



NOTE 1 For basis of curves see Appendix G.

NOTE 2 For towers with $\phi > 0.6$ the shielding factors (shown thus – – – –) should be verified by model testing in a wind tunnel.

Figure 4.4 — Shielding factor, η_f, for single frames composed of flat-sided members

4.4.2 Drag (pressure) coefficients for single frames

Normal drag (pressure) coefficients, C_n, for single frames are given in Figure 4.5.

Normal drag (pressure) coefficients for single frames composed of both flat-sided and circular-section members may be taken as:

$$C_{nf}\frac{A_f}{A_s} + C_{nc}\frac{A_c}{A_s} + C_{nc'}\frac{A_{c'}}{A_s}$$

where

C_{nf}, C_{nc} and $C_{nc'}$ are the drag (pressure) coefficients for flat-sided, subcritical circular- and supercritical circular-section members, respectively, to be obtained from Figure 4.5, appropriate to the solidity ratio of the face considered, as defined in **4.4.1**;

A_s is as defined in **4.2.1**;

A_f, A_c and $A_{c'}$ are as defined in **4.2.2**.

4.5 Linear ancillaries

The wind resistance of linear ancillaries should be determined in the direction of the wind and the crosswind direction in accordance with a) and b), respectively, as follows.

a) The wind resistance, R_{AW}, in the direction of the wind of any linear ancillary part (including waveguides, feeders, etc.) within a panel height should be taken as:

$$R_{AW} = C_N K_A A_A \sin^2 \psi$$

where

C_N is the overall normal drag coefficient appropriate to the item and its effective Reynolds number, values of which are given in Table 4.1 for common isolated individual members and may be determined in accordance with **4.4.2** for parts composed of single frames;

K_A is a reduction factor to take account of the shielding of the component by the tower itself, and is given by:

$$K_A = K'_A \left\{ 1 - \frac{c}{2d} + \frac{2}{3}\left(\frac{c}{d}\right)^2 \right\}$$ for angle members and circular sections in subcritical flow, provided the ancillaries comply with the constraints of **4.1.3**;

$K_A = 1.0$ for circular sections in supercritical flow;

$K_A = 1.0$ for ancillaries not complying with the constraints of **4.1.3**, i.e. when using **4.4**;

where

K'_A is the reduction factor given in Table 4.2;

c is the distance of the leading edge of the ancillary part from the front face of the tower along a line through the position of the ancillary in the wind direction (taken as zero for external parts);

d is the total depth of the tower in the wind direction, along the same line;

A_A is the area of the part visible when viewed in the wind direction including icing when appropriate. For cylinders with the strakes, A_A should be based on the overall width, including twice the strake depth;

ψ is the angle of wind incidence to the longitudinal axis of any linear member.

b) The crosswind resistance, R_{AX}, over a panel should be calculated in accordance with a) taking the reference direction as normal in plan to the mean wind direction.

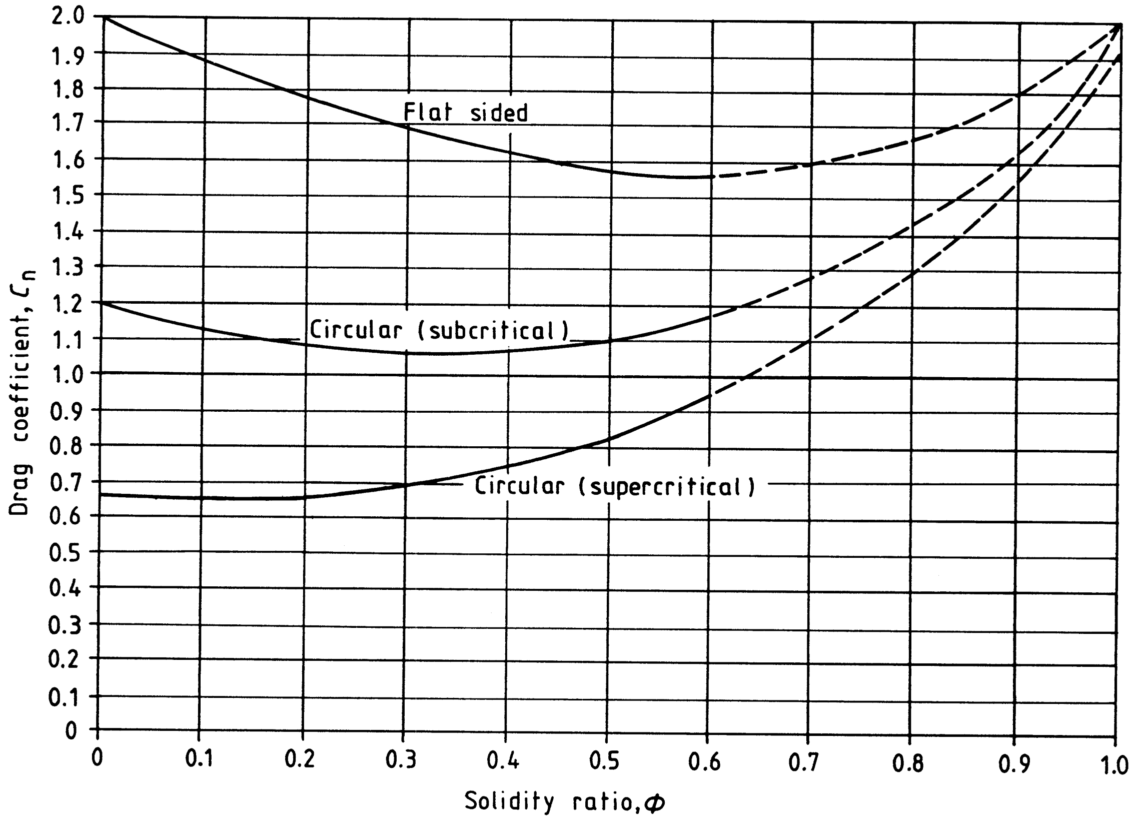
Table 4.1 — Typical drag (pressure) coefficients for individual components

Member type	Effective Reynolds number $R_e = 1.5\ \dfrac{\bar{V}_z D}{\nu}$	Drag (pressure) coefficient C_N Ice free	Drag (pressure) coefficient C_N Iced
a) Flat-sided sections and plates	All values	2.0	2.0
b) Circular sections and smooth wire	$\leqslant 2 \times 10^5$	1.2	1.2
	4×10^5	0.6	1.0
	$> 10 \times 10^5$	0.7	1.0
c) Fine stranded cable, e.g. aluminium core steel round conductor locked coil ropes spiral steel strand with more than seven wires	*Ice free:* $\leqslant 6 \times 10^4$ $\geqslant 10^5$	 1.2 0.9	
	Iced: $\leqslant 1 \times 10^5$ $\geqslant 2 \times 10^5$		 1.25 1.0
d) Thick stranded cable, e.g. small wire ropes round strand ropes spiral steel strand with seven wires only (1 × 7)	*Ice free:* $\leqslant 4 \times 10^4$ $> 4 \times 10^4$	 1.3 1.1	
	Iced: $\leqslant 1 \times 10^5$ $\geqslant 2 \times 10^5$		 1.25 1.0
e) Cylinders with helical strakes of height up to 0.12D	All values	1.2	1.2

NOTE 1 For intermediate values of R_e, C_N should be obtained by linear interpolation.
NOTE 2 D is the member diameter (in m).
$\bar{V}_z$ is the factored wind speed relevant to the height z from ground level to the centre of the member (see **3.2.1**) (in m/s);
ν is the kinematic viscosity of air (see **4.2.1**) (in m^2/s).

Table 4.2 — Reduction factor, K'_A, for additional ancillary items

Position of additional ancillaries	Reduction factor, K'_A Square and rectangular section towers	Reduction factor, K'_A Triangular section towers
Internal to the section	0.6	0.5
External to the section	0.7	0.6



NOTE 1 For basis of curves see Appendix G.
NOTE 2 For towers with ϕ > 0.6 the drag coefficients (shown thus – – – –) should be verified by model testing in a wind tunnel.

Figure 4.5 — Normal drag coefficient, C_n, for single frames

4.6 Discrete ancillaries

The total wind resistance of any discrete ancillary item should be determined in the direction of the wind and in the crosswind direction in accordance with a) and b), respectively, as follows.

a) For any discrete item such as a dish reflector, the total wind resistance, R_{AW}, in the direction of the wind, should be taken as:

$R_{AW} = C_A K_A A_A$

where

C_A is the drag coefficient for the item appropriate to the wind direction and wind speed and should be obtained from wind tunnel tests;

A_A is the face area of the item projected in the same direction;

K_A is as defined in **4.5**.

b) The crosswind resistance, R_{AX} should be calculated in accordance with a) taking the reference direction as normal in plan to the mean wind direction.

4.7 Cables

The wind resistance, R_{CW}, in the direction of the wind of any external cable attached to a tower may be taken as:

$$R_{CW} = C_C\, D_C\, L_C \sin^3 \psi$$

where

C_C is the overall normal drag coefficient appropriate to the effective Reynold's number, the values of which are given in Table 4.1 for both ice-free and iced conditions;

L_C is the chord length of the cable;

D_C is the diameter of the cable;

ψ is the angle of wind incidence to the chord.

4.8 Icing

In considering the wind resistance of a tower and ancillaries under iced conditions, each element of the structure and of the supported parts should be considered to be coated on all sides, with a thickness of ice equal to that given in **3.5**.

Where the gap between components when not iced, is less than 75 mm, this should be considered to be completely filled by ice under icing conditions.

For drag (pressure) coefficients of individual members see Table 4.1.

Section 5. Structural response to wind

5.1 Procedure

5.1.1 General

The maximum forces to be used in the design of tower components and foundations should be calculated with due allowance for the response to wind turbulence. Such forces should represent the resultant effect of an equivalent static loading due to wind of speed equal to the appropriate mean hourly value, acting only in the wind direction, and fluctuating loading both downwind and crosswind due to gustiness.

Two methods of determining the maximum forces in the members of a tower are provided. The equivalent static method (see **5.1.2**) should only be used if:

$$\frac{7m_T}{\rho_s R_{WT}\sqrt{d_B \tau_o}}\left(\frac{5}{6}-\frac{h_T}{H}\right)^2 < 1$$

where

R_{WT} is the sum of the panel resistances, commencing from the top of the tower, such that R_{WT} is just less than one-third of the overall summation ΣR_W for the whole tower (in m^2);

ρ_s is the density of the material of the tower structure (in kg/m^3);

m_T is the total mass of the panels making up R_{WT} (in kg);

H is the height of the tower (in m);

h_T is the total height of the panels making up R_{WT} but not greater than $H/3$ (in m);

τ_o is a volume/resistance constant taken as 0.001 m;

d_B is the depth in the direction of the wind, equal to:

base, d, for rectangular towers (in m);

0.75 × base width for triangular towers (in m).

Otherwise the spectral analytical method (see **5.1.3**) should be followed.

NOTE The equivalent static method includes an allowance for the dynamic amplification of response which is typical of the majority of towers that will be constructed in accordance with this code. The complexity of undertaking a full analysis of dynamic response, however, is such that the check for applicability of the static procedure should be considered for guidance only. Dynamic augmentation generally increases in successively higher panels of any tower but, particularly when supporting large concentrations of ancillary items or when using a concave outline profile (Eiffelization), the designer is recommended to exercise caution in applying the static procedure to towers where these effects are considerably more than those typically encountered.

5.1.2 Equivalent static method

For towers constructed of leg members with triangulated bracings and free from ancillaries, or only containing ancillaries which comply with the constraints of **4.1.3**, the maximum member forces should be derived in accordance with **5.2**.

For towers constructed of leg members with triangulated bracings and containing ancillaries which do not comply with the constraints of **4.1.3**, the maximum member forces should be determined in accordance with **5.3**.

5.1.3 Spectral analytical method

Alternatively, maximum member forces may be determined more accurately by spectral analytical methods, adopting the basis set out in **5.4** and the parameters given in Appendix E.

5.1.4 Deflections

Deflections will normally only be required to satisfy serviceability requirements. The criteria and procedures are given in **5.2.5**.

5.1.5 Vortex-excited vibrations

When towers support large bluff bodies of cylindrical form or may be expected to become heavily blocked by icing, consideration should be given to their susceptibility to vortex-excited vibration, in accordance with **5.5**.

5.2 Wind loading for symmetrical towers

5.2.1 General

For towers free from ancillaries or containing ancillaries complying with the constraints of **4.1.3**, the maximum mean wind load in the direction of the wind per panel height of the tower body, $\bar{P}_{TW}$, should be taken as:

$$\bar{P}_{TW} = \frac{\rho_a}{2} \bar{V}_z^2 \, \Sigma R_W$$

The maximum fluctuating load due to turbulence in the direction of the wind, P'_{TW}, should be taken as:

$$P'_{TW} = G \, \bar{P}_{TW}$$

The maximum fluctuating load due to turbulence in the crosswind direction, where required, P'_{TX}, should be taken as:

$$P'_{TX} = K_X \left(\frac{\Sigma R_X}{\Sigma R_W} \right) P'_{TW}$$

where

- G is a gust response factor appropriate to the bending moment or shear force, determined in accordance with **5.2.3** or **5.2.4**, as appropriate;
- ρ_a is the density of the air at the reference temperature and pressure (ρ_a = 1.22 kg/m^3 for the UK when determining P in newtons and with $\bar{V}$ in metres per second);
- $\bar{V}_z$ is the mean wind speed at the level of the centre of area of the panel at a height z metres above the site ground level, determined in accordance with **3.2**;
- ΣR_W is the total wind resistance of the structure (and any ancillaries if present) in the direction of the wind over the panel height concerned, determined in accordance with **4.2** or **4.3**, as appropriate;

  NOTE ΣR_W is taken as the wind resistance of the partially-shielded tower body, R_{TE}, when using **5.3**.
- K_X is a factor allowing for crosswind intensity of turbulence and should be taken as 0.5;
- ΣR_X is the corresponding crosswind resistance over the panel height.

These loads should be taken as acting at the level of the centre of area of the faces (including ancillaries if present) within a panel height.

5.2.2 Basic gust response factor

5.2.2.1 *General.* In order to calculate the gust response factor, G, the basic gust response factor G_B, should be derived in accordance with **5.2.2.2** or **5.2.2.3**.

5.2.2.2 *General method.* The basic gust response factor G_B should be taken as:

$$G_B = Bj$$

where

- B is a size factor to be obtained from Figure 5.1 as appropriate to the terrain category defined in **3.1.4** (interpolating for intermediate terrain categories);
- j is a height factor to be obtained from Figure 5.2;

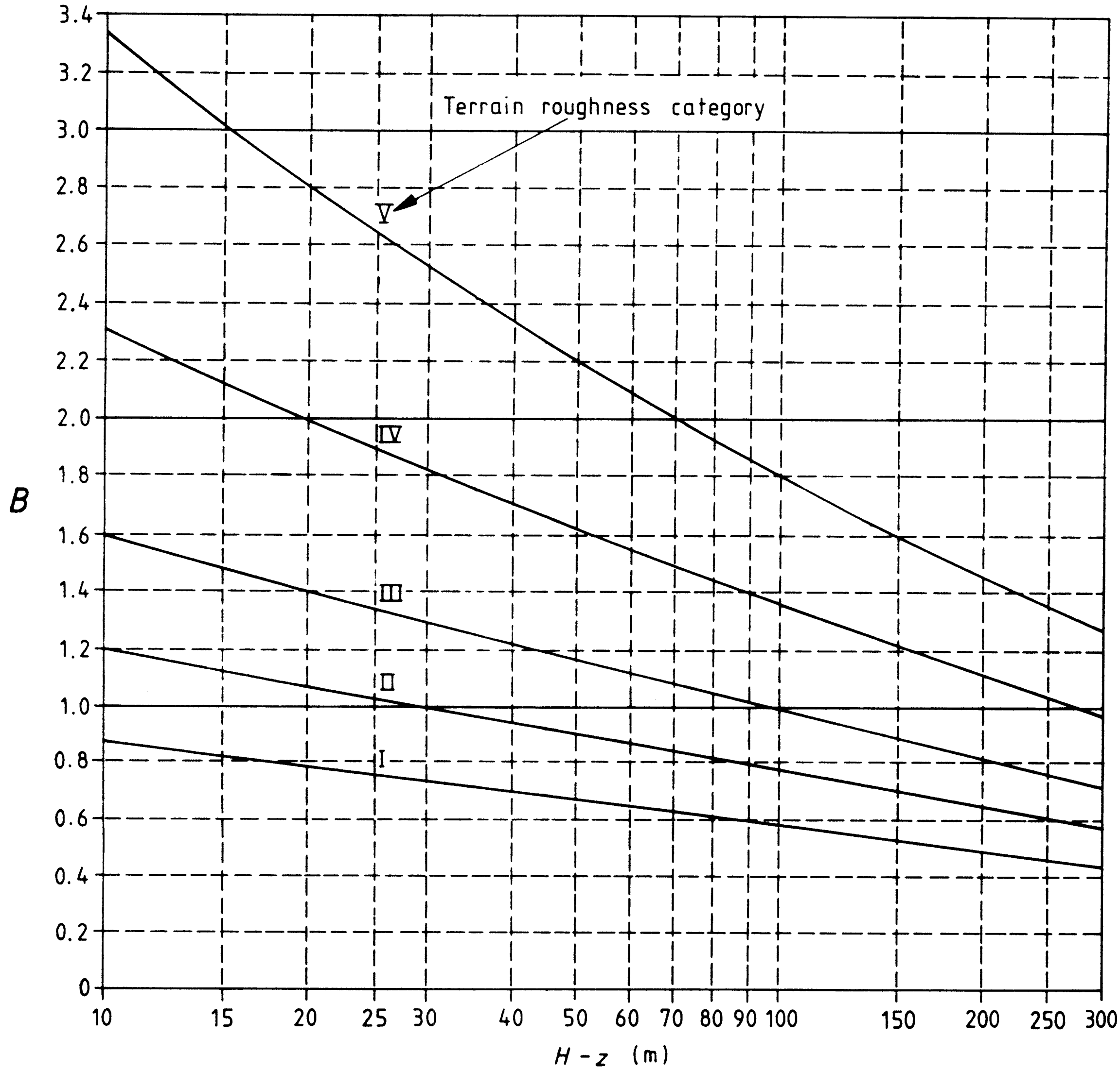
  where

  - z is the height above ground at which bending moments or shear force are required;
  - H is the overall tower height.

  NOTE $H - z$ should not be taken as less than 10 m when $H \leqslant$ 100 m and should not be taken as less than 0.1H when $H >$ 100 m, in the combined use of Figure 5.1 and Figure 5.2.

5.2.2.3 *Simplified method.* For simplification, the basic gust response factor, G_B, may be obtained directly from Figure 5.3 which applies to any value of z as appropriate to the terrain category defined in **3.1.4** (interpolating for intermediate terrain categories) where z is as defined in **5.2.2.2**.

NOTE This simplified alternative provides values of G_B which are conservative by up to 10 % particularly for parts near the top of short towers (H < 100 m) in rough terrain or for those near the base of tall towers (H > 100 m) in smooth terrain.



NOTE For basis of curves see Appendix G.

Figure 5.1 — Size factor, *B*

5.2.3 Loading for calculating bending moments

The gust response factor, G, used in the calculation of overall bending moments in the design of leg members and foundations should be taken as:

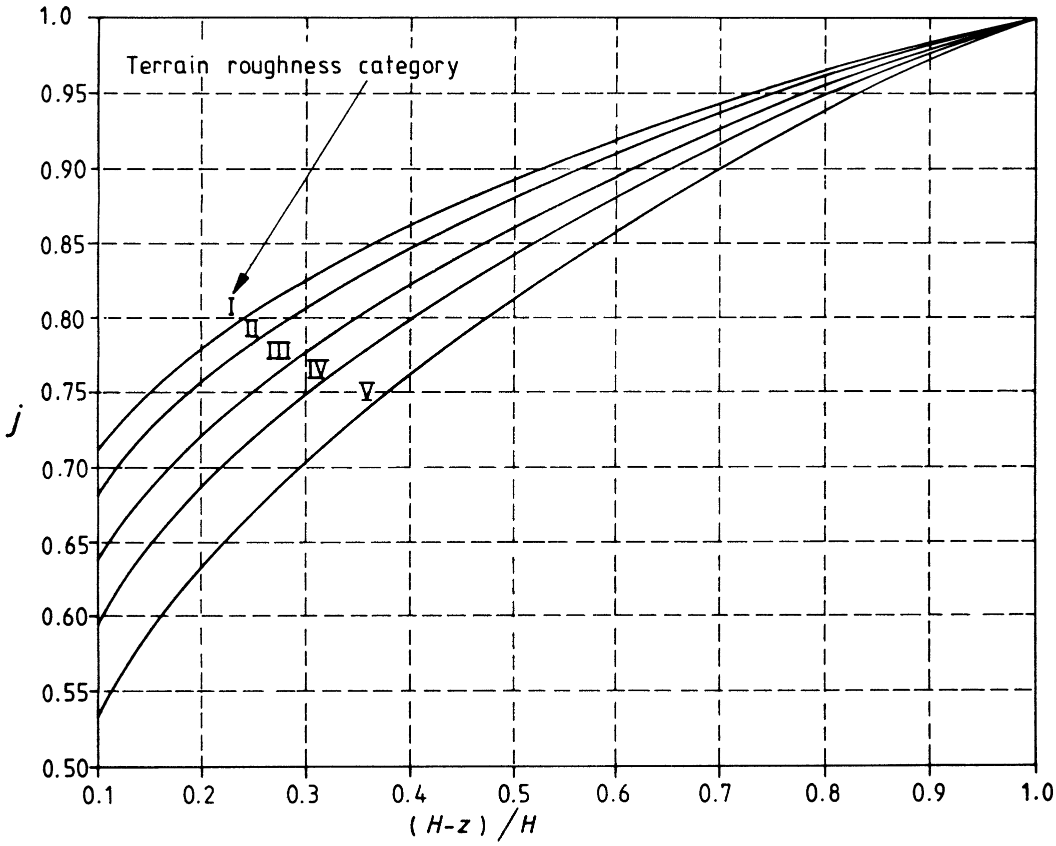
$$G = G_B \left\{1 + 0.2\left(\frac{z_m}{H}\right)^2\right\}$$

where

G_B is the basic gust response factor determined in accordance with **5.2.2** taking $z = z_m$;

z_m is the height above ground at which the bending moment is required;

H is the overall tower height.



NOTE For basis of curves see Appendix G.

Figure 5.2 — Height factor, *j*

5.2.4 Loading for calculating shear forces

The gust response factor, G, used in the calculation of overall shear loading above a given level to be carried by bracings or foundations should be taken as:

$$G = K_q\, G_B \left\{1 + 0.2\left(\frac{z_q}{H}\right)^2\right\}$$

where

G_B is the basic gust response factor, determined in accordance with **5.2.2** taking $z = z_q$;

z_q is the height above ground at which the shear is required;

K_q is a factor to be obtained from Figure 5.4, appropriate to the value $1/|f_q|$, where $|f_q|$ is the modulus of the ratio of the shear force carried by the bracing to the total shear force at the level, z_q, due to the tower wind loads under the mean wind loading;

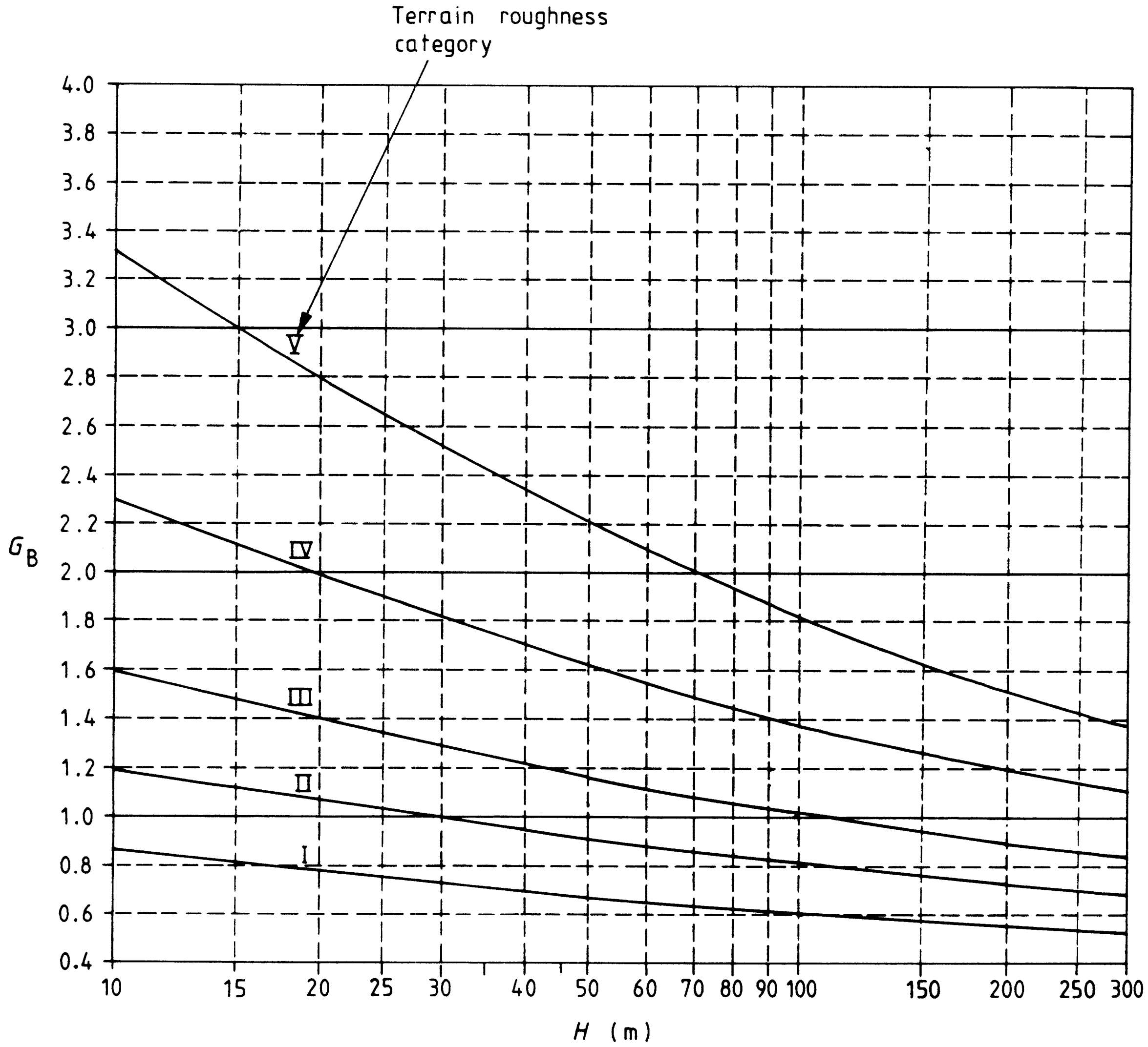
H is the overall tower height.

NOTE 1 The sign of f_q is negative if the shear force in the panel resulting from loading above the point of intersection of the longitudinal axes of the legs is greater than that from below the point of intersection.

NOTE 2 When $\frac{1}{|f_q|} > 8$, K_q may be determined from:

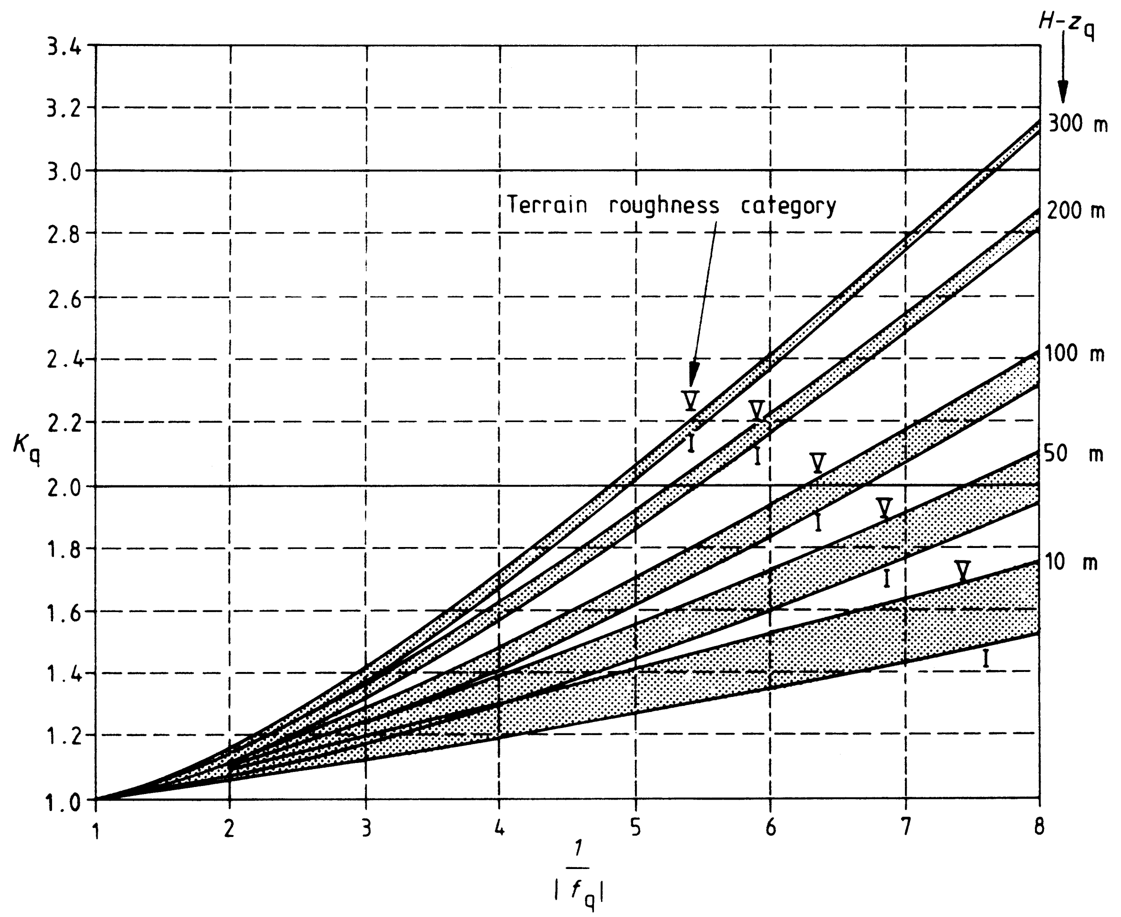
$$K_q = 1 + \frac{(H - z_q)^{0.4}}{23.6} \left(\frac{1}{|f_q|} - 3 \right)$$

Alternatively, consideration should be given to a separate calculation under patch wind loading (appropriate to the member under consideration) to derive the maximum total force in the member.



NOTE For basis of curves see Appendix G.

Figure 5.3 — Basic gust response factor, G_B



NOTE For basis of curves see Appendix G.

Figure 5.4 — Shear loading factor, K_q

5.2.5 Loading for calculating deflections

To calculate deflections, the criterion of serviceability has to be defined. Two categories may be considered as follows.

a) Towers for which during permitted periods of unserviceability, the specified limiting deflections may be only occasionally exceeded. In such instances, the deflections should be derived from both the mean hourly and the fluctuating components of wind load, using the gust response factor, G, appropriate to leg loading at the base of the tower (determined in accordance with **5.2.3**) for deriving the fluctuating component.

b) Towers for which during periods of unserviceability, the specified limiting deflections may be exceeded for 50 % of the time. For these, the fluctuating components of wind loading may be ignored and deflections under mean wind loading only need be considered.

5.2.6 Calculation of wind forces in tower members

The total force, ΣF_T, in any member due to wind should be taken as:

$$\Sigma F_T = (\bar{F}_T + F'_{TW})$$

where

$\bar{F}_T$ is the force calculated due to the maximum mean wind loads, $\bar{P}_{TW}$;

F'_{TW} is the force calculated due to the maximum fluctuating wind loads, P'_{TW};

$\bar{P}_{TW}$ and P'_{TW} are determined in accordance with **5.2.1**.

For towers within the constraints given in **4.1.3**, the forces due to maximum fluctuating crosswind loads, may be ignored. Thus the total force, ΣF_T, in any member may be calculated by the application of a total wind load per panel, ΣP_{TW}, in the direction of the wind only, of:

$$\Sigma P_{TW} = \frac{\rho_a}{2} \bar{V}_z^2 \, \Sigma R_W \, (1 + G)$$

where

ρ_a, $\bar{V}_z$, ΣR_W and G are as defined in **5.2.1**.

5.3 Wind loading for towers with complex attachments

5.3.1 General

For towers which contain large discrete and/or unsymmetrically placed linear ancillaries and/or cables, i.e. those not complying with **4.1.3**, the total forces due to the effects of wind load should allow for the combined action of wind on individual parts, both downwind and crosswind, when appropriate.

The loads due to wind on the large ancillary items should be determined in accordance with **5.3.2** using wind resistance R_{AW} determined in accordance with **4.5** and **4.6**, as appropriate, and the loads due to wind on any external attached cables should be determined in accordance with **5.3.3** using wind resistance R_{CW} determined in accordance with **4.7**.

The loads due to wind on the partially shielded tower body should be determined in accordance with **5.2.1** to **5.2.5** by using the wind resistance of the equivalent bare tower R_{TE}, in place of ΣR_W throughout, where:

$$R_{TE} = \Sigma R_W - (R_{AW} + R_{CW})$$

where

ΣR_W is the total wind resistance, determined in accordance with **4.4**;

R_{AW} is the wind resistance of large ancillary items, determined in accordance with **4.5** and **4.6**;

R_{CW} is the wind resistance of any external attached cables, determined in accordance with **4.7**.

Similarly when considering loads in the crosswind direction, ΣR_X should be replaced by a similar expression related to the appropriate resistances in the crosswind direction.

5.3.2 Loading on ancillary items

The maximum mean wind loading in the direction of the wind on large discrete items attached to a tower, such as dish aerials, $\bar{P}_{AW}$, should be taken as:

$$\bar{P}_{AW} = \frac{\rho_a}{2} \bar{V}_{zA}^{\,2} \, R_{AW}$$

The maximum fluctuating load due to gusts in the direction of the wind should be taken as:

$$P'_{AW} = G_A \, \bar{P}_{AW}$$

The maximum fluctuating load due to gusts in the crosswind direction should be taken as:

$$P'_{AX} = P'_{AW} \left(\frac{R_{AX}}{R_{AW}}\right) K_X$$

where

$\bar{V}_{ZA}$ is the mean wind speed at the level of the centre of area of the item, determined in accordance with **3.2**;

R_{AW} is the wind resistance of all ancillaries in the wind direction, determined in accordance with **4.5** and **4.6**;

R_{AX} is the wind resistance of all ancillaries in the crosswind direction, determined in accordance with **4.5** and **4.6**;

K_X and ρ_a are as defined in **5.2.1**;

G_A is the gust response factor for the ancillaries, given by:

$$B_A \, j_A \left\{ 1 + 0.2 \left(\frac{z_A}{H} \right)^2 \right\}$$

where

B_A is taken as B from Figure 5.1 with $H - z$ equal to the greater of:

a) e_A;

b) 10 m;

c) $0.1z_A$;

j_A is taken as j from Figure 5.2 with $\frac{H-z}{H}$ equal to the greater of:

a) $\frac{e_A}{z_A}$;

b) $\frac{10}{z_A}$;

c) 0.1;

but not greater than 1.0;

where

z_A is the height of the centre of wind resistance of the ancillary item above ground level (in m);

e_A is the vertical dimension of the ancillary item (in m).

5.3.3 Loading on cables

The maximum mean load applied to a tower in the direction of the wind due to wind action on an attached cable, $\bar{P}_{CW}$, should be taken as:

$$\bar{P}_{CW} = \frac{\rho_a}{4} \bar{V}_{zC}^{\,2} R_{CW}$$

The maximum fluctuating load in the same direction, P'_{CW}, should be taken as:

$$P'_{CW} = G_C \bar{P}_{CW}$$

where

$\bar{V}_{zC}$ is the mean wind speed at the level of the average height of the cable, determined in accordance with **3.2**;

R_{CW} is the wind resistance of the cable in the direction of the wind, determined in accordance with **4.7**;

G_C is the cable gust response factor to be taken as:

$$G_C = K_L K_z$$

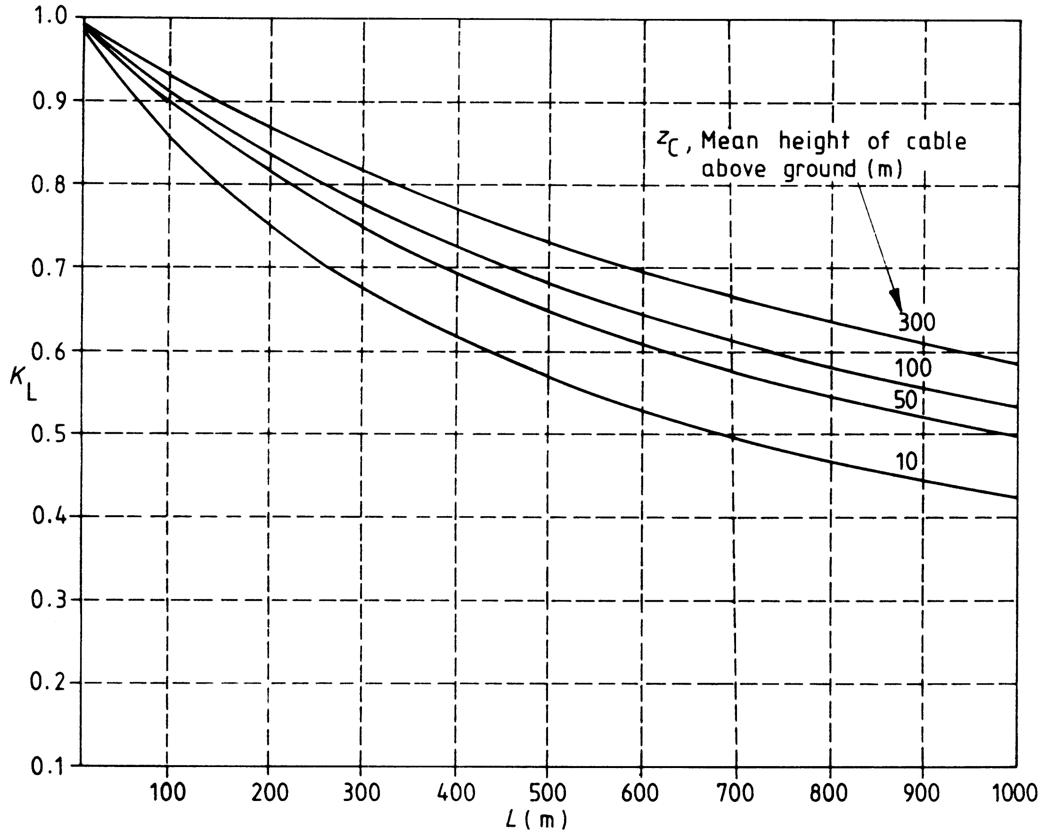
where

K_L is a length factor given in Figure 5.5, in which L is the total projected length of cable attached to the tower (see Figure 5.6);

K_z is a cable height factor to be obtained from Figure 5.7, as appropriate to the terrain roughness where z_C is the mean height of cable above ground;

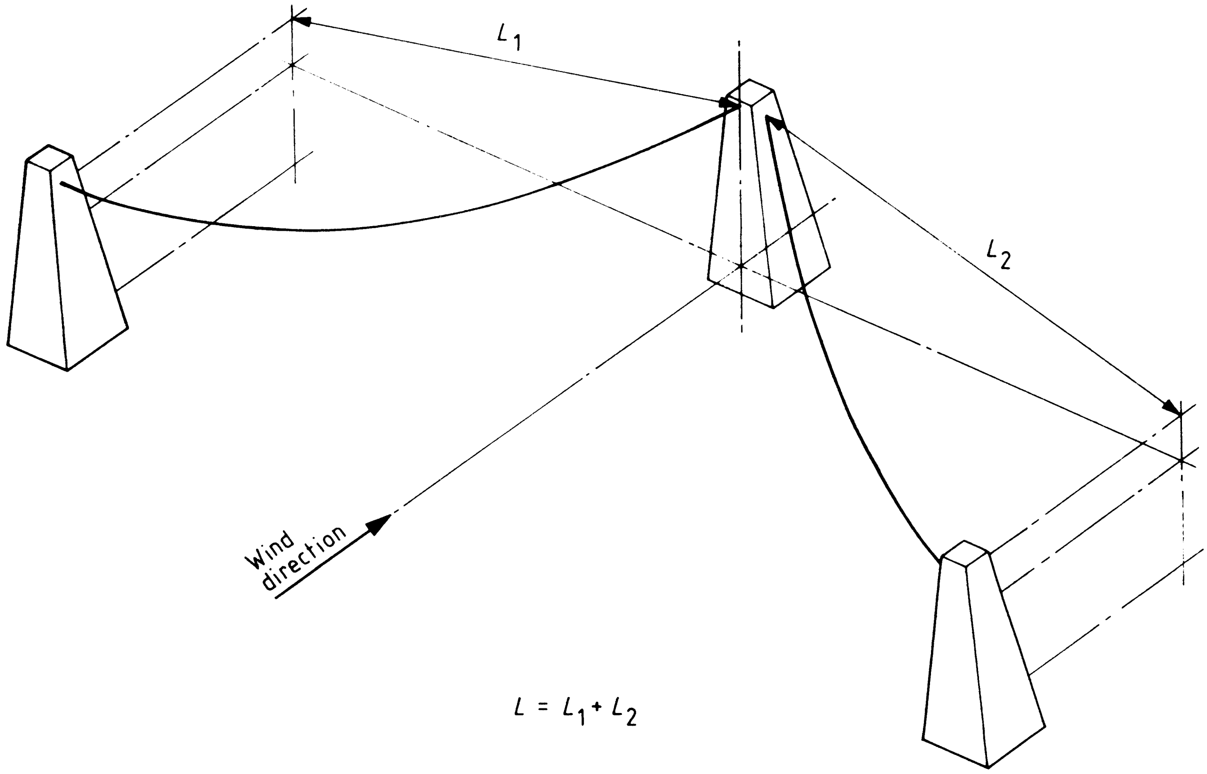
ρ_a is as defined in **5.2.1**.

The loading so obtained should be used directly to determine the forces due to wind in the attachments of a cable to a tower.



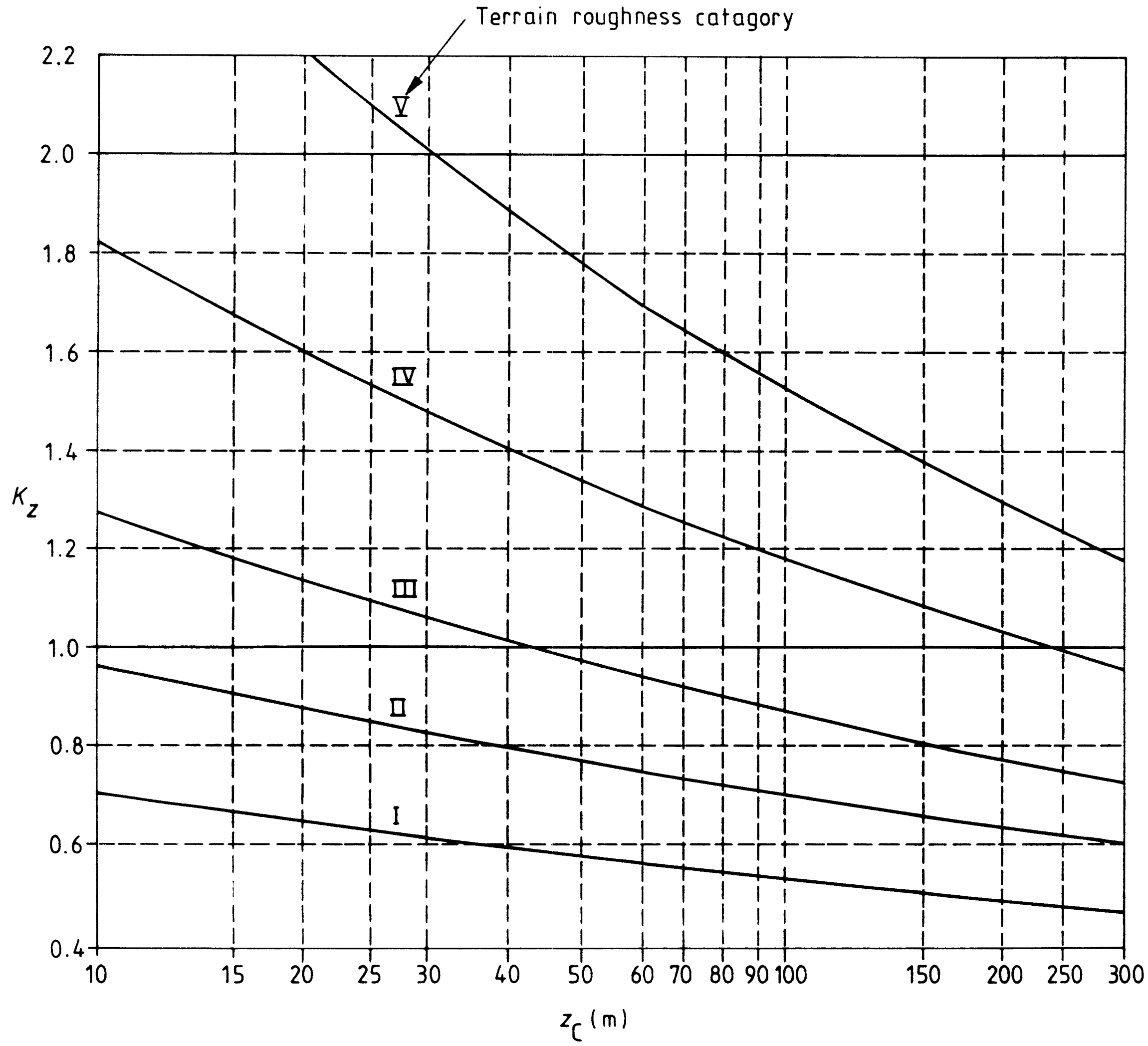
NOTE For basis for curve see Appendix G.

Figure 5.5 — Length factor, K_L, for cables



NOTE The bases of the three towers are set at different levels to show undulating terrain.

Figure 5.6 — Projected lengths of cables



NOTE For basis of curves see Appendix G.

Figure 5.7 — Cable height factor, K_z

5.3.4 Calculation of wind forces in tower members

The total force in a tower member due to wind loading, ΣF, is given by:

$$\Sigma F = \overline{F} + F'$$

where

$\overline{F}$ is the maximum force in the member due to the mean wind loading given by:

$$\overline{F} = \overline{F}_T + \overline{F}_A + \overline{F}_C$$

where

$\overline{F}_T$ is the force in the member calculated due to the maximum mean wind loads on the partially shielded tower body, $\overline{P}_{TW}$, determined from **5.2.1** to **5.2.5** in accordance with **5.3.1**;

$\overline{F}_A$ is the force in the member calculated due to the maximum mean wind loads, $\overline{P}_{AW}$, on all the ancillaries, determined in accordance with **5.3.2**;

$\overline{F}_C$ is the force in the member calculated due to the maximum mean wind loads, $\overline{P}_{CW}$, on all attached cables, determined in accordance with **5.3.3**;

F' is the maximum fluctuating force in the member due to turbulence and given by:

$$F' = \sqrt{(F'_{TW} + F'_{AW})^2 + (F'_{TX} + F'_{AX})^2 + F'^2_c}$$

where

F'_{TW} and F'_{TX} are the components of forces in the member due to downwind and crosswind fluctuating wind loads, respectively, on the partially shielded tower body, determined from **5.2.1** to **5.2.5** in accordance with **5.3.1**;

F'_{AW} and F'_{AX} are the components of forces in the member due to downwind and crosswind fluctuating wind loads, respectively, on all the ancillaries, determined in accordance with **5.3.2**;

F'_c is the component of force in the member due to downwind fluctuating wind loads on all attached cables, determined in accordance with **5.3.3**.

5.3.5 Loading for calculating deflections

The procedure given in **5.2.5** should be followed, but with due allowance for the mean hourly and fluctuating components of deflections resulting from attached ancillaries and cables, in **5.2.5** a) and b).

5.4 Spectral analytical method

When response to downwind forces is calculated by a spectral analytical method, the meteorological conditions to be assumed should be those defined in section 3, and the wind resistance taken as that given in section 4. In addition, the parameters defined in Appendix E should be adopted in the absence of more accurate information.

Response should be calculated for all modes of vibration having natural frequencies less than 2 Hz.

5.5 Crosswind response due to vortex excitation

5.5.1 Critical wind speed

A tower may vibrate transverse to the wind flow due to vortex excitation if it supports aerials, flues, shrouds or other bluff bodies of circular or prismatic cylindrical form, or when portions of the lattice framework are heavily iced.

The lowest critical wind speeds at which such vibrations occur, V_{cr}, should be derived from:

$$V_{cr} = \frac{n_1 D_z}{S}$$

where

D_z is the diameter or crosswind width of the bluff body at any level z above the tower base;

S is the Strouhal number, to be taken as 0.2 for circular cylinders and as 0.15 for sharp-edged bodies;

n_1 is the fundamental natural frequency of the tower in crosswind vibration.

If transverse oscillation is to be avoided at all times, the value of V_{cr} should exceed 1.3 $\bar{V}_z$ at all levels z, where $\bar{V}_z$ is determined in accordance with **3.2**.

Alternatively, when occasional oscillations are permissible, the periods of unserviceability may be assessed by reference to **3.3.3**.

5.5.2 Excitation

When it is predicted that the critical velocities will be exceeded, either:

a) provision should be made to prevent vortex shedding by the use of aerodynamic spoilers in the form of strakes on circular cylinders or of perforated shrouds or other proven means; *or*

b) amplitudes of vibration and dynamic stresses should be calculated for all feasible modes of vibration.

Amplitudes may be estimated by dynamic analysis by application of fluctuating aerodynamic forces equal to:

$0.9\rho_a \bar{V}_e^2 k_e C'_a D_z \sin 2\pi nt$ per unit height of the bluff body

where

C'_a is the fluctuating aerodynamic lift coefficient and is equal to:

0.5 for sharp-edged profiles;
0.3 for circular cylinders;

t is the time;

n is the frequency of vibration of the mode under consideration;

ρ_a is as defined in **5.2.1**;

$\bar{V}_e$ is the mean wind speed at the mean height of the bluff body up to a maximum of $\bar{V}_z$, determined in accordance with **3.2**.

k_e is an excitation coefficient depending on the ratio $1.3\,\bar{V}_e/V_{cr}$ which may be determined from Figure 5.8 for circular or square cylinders, where V_{cr} is the critical wind speed.

Damping decrements may be obtained as the structural decrements from Appendix E ignoring aerodynamic damping.

Alternatively, crosswind response may be predicted by means of representative wind tunnel tests on an aeroelastic model or on sectional models used to obtain values for excitation for sections of more complex form.

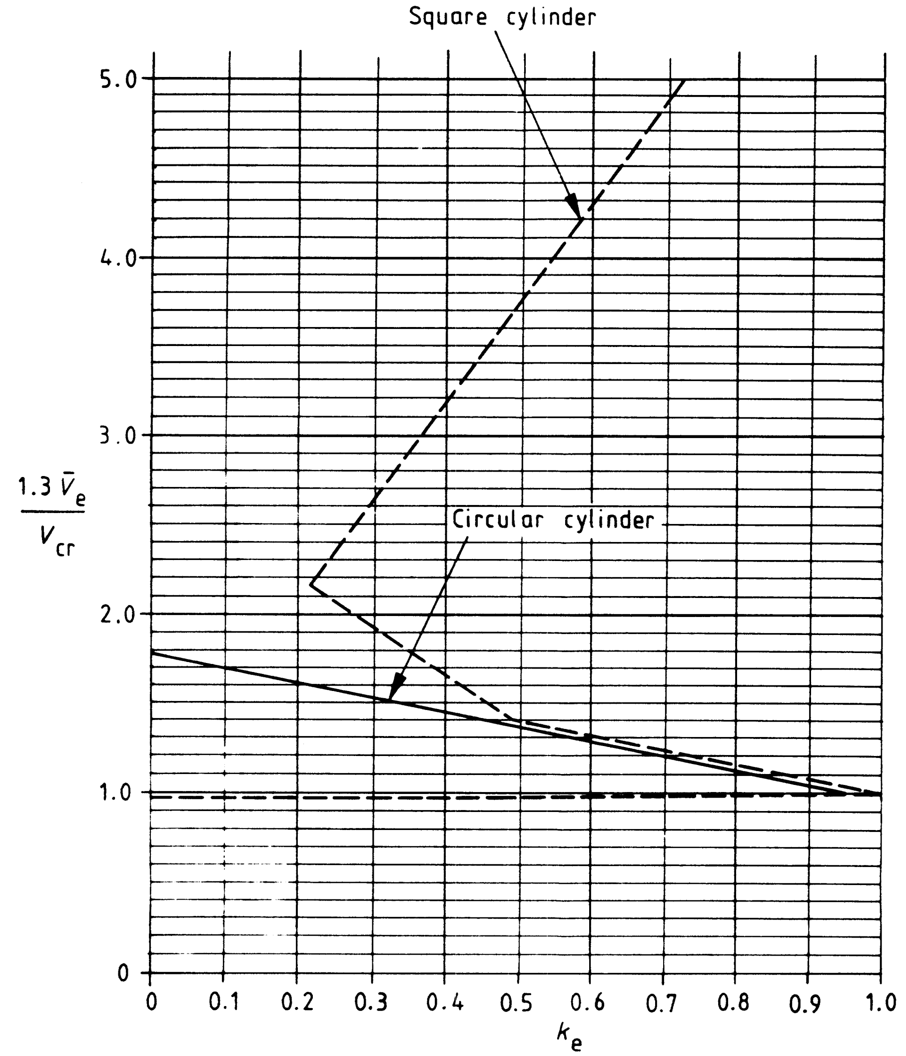


Figure 5.8 — Effective excitation coefficient, k_e

Appendix A Measurement and interpretation of wind data

A.1 Measurement and calibration

A.1.1 *Site for reference measurements*

Reference measurements of wind speed for use with this Part of BS 8100 should be made at sites and in locations such that they may be reliably interpreted. A suitable site for records should be:

a) in open terrain of category I, II or III as defined in Table 3.1;

b) not closer than 2 km to hills or mountains of height greater than 100 m above the level of the site.

A.1.2 *Calibration of site*

The wind flow characteristics for all directions of wind should be determined for the site. The terrain roughness factor K_R may be derived from the relationships plotted in Figure A.1 using the ratio:

$$\frac{V_t}{\bar{V}}$$

where

V_t is the maximum gust velocity of short duration, averaged over t seconds;

$\bar{V}$ is the mean hourly velocity for the same period.

These characteristics should be obtained for as wide a range of wind speeds and directions as possible. When calibrations such as these have not been carried out, the recording station should be assumed to have the terrain characteristics of category III (see Table 3.1). The value of K_R thus obtained should be checked for compatability with the observed or expected terrain characteristics.

A.1.3 *Location of measuring instrument*

The instrument used for speed measurement should be mounted at a level of at least 10 m above the ground level and well clear of obstructions interfering with the natural air flow. When it is necessary to mount the instrument at some greater level, records of the mean hourly speeds should be converted to equivalent values appropriate to the 10 m level in accordance with **3.2.1**.

A.1.4 *Response and averaging period*

The response characteristics of the measuring instrument and the recording equipment should be known and the effective averaging period for the recorded speeds defined. When this period is unknown, it should be assumed to be 2 s. A procedure for determining the averaging period is given in **A.2**.

Measurements of maximum speed over the effective averaging period for an instrument may be converted to equivalent mean hourly speeds, $\bar{V}$, by reference to Figure A.1 once the site has been calibrated. When records have been analysed to give equivalent mean speeds, V_t, averaged over t seconds at 10 m height, these may be converted to mean hourly values by division by the ratio V_t/V appropriate to the value of K_R for the site of the instruments given in Figure A.1.

A.1.5 *Statistical interpretation*

The maximum value of mean hourly speed at the reference height of 10 m above ground should be determined, independent of direction, for each year of records. These values should be analysed to determine the 50 year return maximum mean wind speed, $\bar{V}_{max}$, using the mean of these maxima and their coefficient of variation v assuming an external distribution, i.e. Gumbel (Fisher-Tippett type 1), when at least 15 consecutive years of records are available. Where fewer records exist, the mean value should be determined but the coefficient of variation should be taken as 0.2.

When predicting the extreme mean speeds of hurricanes or typhoons, reference should be made to the records of at least three stations in the region concerned to ensure that the extreme value distribution follows the Gumbel (Fisher-Tippett type 1) form.

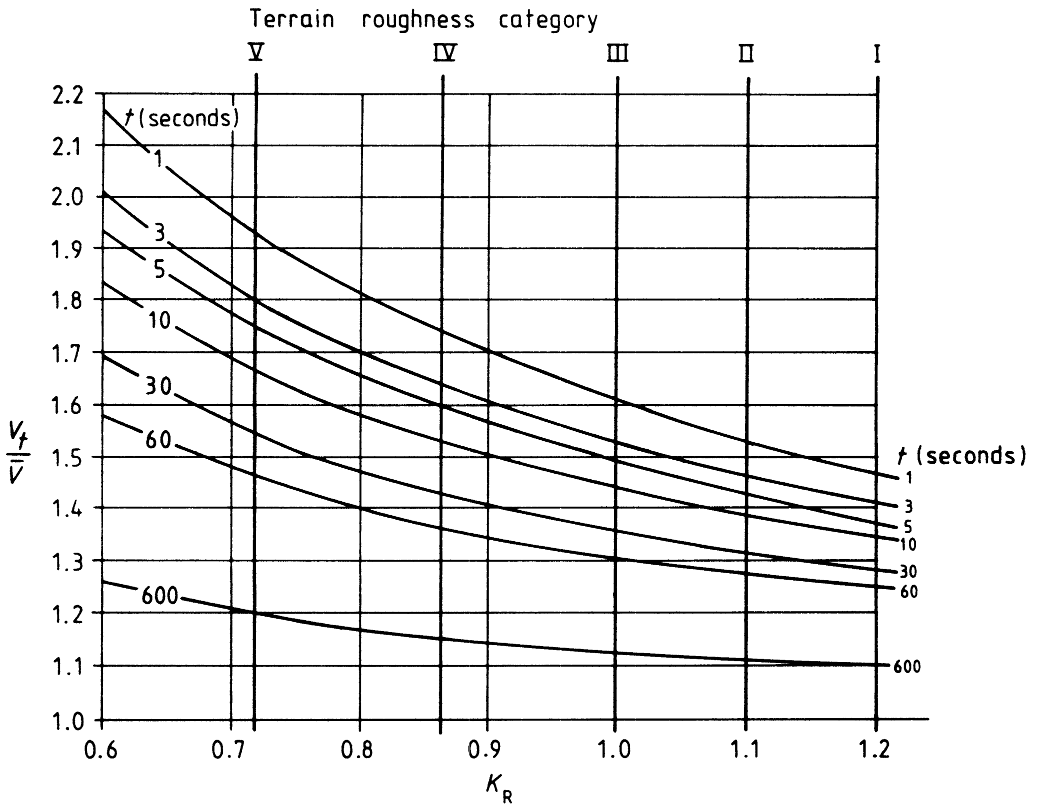


Figure A.1 — Derivation of terrain roughness factor, K_R, from wind speed records

A.1.6 ***Application of results to other sites***

In applying the values of the mean hourly speeds derived in accordance with **A.1.1** to **A.1.5** to sites other than those in the observations, due allowance should be made for any differences which may exist in altitude and surface roughness at the site. The values of $\bar{V}_B$ given in **3.1.2** are related to an equivalent open inland site having a terrain roughness of category III (see Table 3.1). When the site of the records has any other roughness category the values of $\bar{V}_{max}$, derived in accordance with **A.1.5** should be divided by K_R to obtain the basic wind speed, $\bar{V}_B$, to be used in **3.1.2**.

A.1.7 ***Velocity profile***

The index of variation of mean wind speed with height, may be taken to be related to the terrain roughness factor K_R in accordance with Figure 3.3.

A.2 Averaging period of an anemometer

The averaging period of an anemometer may be considered as the least duration of gust that can be reliably recorded by the instrument. For a rotating cup instrument, the averaging period should be taken as twice the time constant, τ_i. The time constant, τ_i, should be taken as the sum of the individual time constants of the instrument, τ_{i1}, and of its associated recorder, τ_{i2}.

The time constant τ_{i1} of the instrument should be obtained from that calibrated in a steady air flow, τ_{ib} this being equal to the time taken for the speed of rotation of the cups to reach 63 % of the maximum speed of rotation starting from a stationary condition in flow of velocity V_b. For any other velocity, V, the time constant may be taken as:

$$\tau_{i1} = \tau_{ib} \left(\frac{V_b}{V}\right)$$

The time constant for the associated recorder, τ_{i2}, may be found by applying a voltage step to the input terminals when the constant should be obtained as the time taken to record 63 % of the full voltage.

A.3 Advisory and references sources

A.3.1 ***General***

For interpretation of reference measurements and procedures for calibration of site wind speeds advice may be obtained from sources given in **A.3.2** and **A.3.3**.

A.3.2 ***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

A.3.3 ***Addresses of advisory offices of the Meteorological Office***

For England and Wales:

Meteorological Office

Met 0 3

London Road

Bracknell

Berkshire RG 12 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: 0232 28457

Appendix B Terrain parameters

B.1 Terrain roughness parameter, z_o

The terrain roughness parameter, z_o, is a measure of the retarding effect that the surface in question has on the wind speed near the ground. It is a parameter that changes by more than two orders of magnitude over the range of terrain found in nature. For the purposes of design, this range is divided into five categories, each with a given value of z_o, and each corresponding to an easily recognizable terrain type, as given in Table 3.1.

The values given are average values applicable to sites of comparatively uniform roughness where the general description of the terrain applies for a distance of at least several kilometres upwind. A sixth category with $z_o = 0.8$ m is referred to in other documents corresponding to city-centres, comprising mostly four-storey buildings or higher, typically between 30 % and 50 % of the terrain area for several kilometres upwind. Since almost no urban or suburban area exists in the UK of sufficient extent to enable the full benefit of the slower equilibrium profile to be obtained, this category VI should not be used as a basis for design, unless special studies of the wind profile at an urban site are made. For this reason category VI has not been included in Table 3.1.

There is a continuous gradation of aerodynamic roughness in nature, but these tabulated values should be sufficient to define the character of a site. If the roughness of a particular site appears to lie between two of these categories, then an intermediate value of z_o may be assumed; however, the adoption of the less rough category will ensure a conservative result.

The meteorological standard, or basic terrain in the UK, corresponds to the category of aerodynamic roughness z_o = 0.03 m. Even though the anemometers may be on airfield sites where the aerodynamic roughness is in the range 0.003 m < z_o < 0.01 m, the extent of this lesser roughness is small compared with the very much larger extent of the surrounding typical UK farmland, and not sufficient to modify the wind parameters.

The aerodynamic roughness of a large expanse of water is a special case, since the water surface responds to the wind by becoming rougher with increasing wind speed. The available data on wind/wave interaction have been correlated to produce the approximate relationship:

$$z_o \text{ (water)} = 4.9 \times 10^{-6}\ \bar{V}_B^2$$

where

$\bar{V}_B$ is the basic mean hourly wind speed (in m/s).

Giving:

$z_o = 0.002$ for $\bar{V}_B$ = 21 m/s;

$z_o = 0.004$ for $\bar{V}_B$ = 28 m/s;

$z_o = 0.006$ for $\bar{V}_B$ = 35 m/s.

The value of z_o = 0.003 m for terrain category I corresponds to the middle of the range of UK mean hourly wind speeds, $\bar{V}_B$, and would produce slightly conservative results for sites with higher basic wind speeds. It should therefore be sufficient for design at coastal sites in the UK. It should be emphasized, however, that this code is not applicable to offshore sites where account should be taken of the appropriate turbulence characteristics and effective heights to determine the relevant wind structure.

B.2 Effective height of surface obstructions, h_e

The flow between obstructions near ground level is very complex and no prediction of wind speed can be made with any certainty. The relationships given in this code apply only to heights above the general level of the surrounding obstructions and it is necessary to define a displacement plane to which all heights are referred. The height, h_e, of this zero plane above the ground is generally less than the general height, h, of surrounding buildings.

For open-country terrain where $z_o \leqslant 0.03$, the zero-plane displacement is not significant and may be taken as zero. Values of the zero-plane displacement which are typical of the rougher terrain categories in the UK are given in Table 3.1.

Appendix C Gradient wind speeds

C.1 General

Where meteorological records are not available, or the nearest meteorological stations are remote from the site, gradient wind speeds, $\bar{V}_g$, may be used as a basis for deriving the site reference wind speeds, $\bar{V}_r$, as detailed in **3.1.5** in accordance with **C.2** and **C.3**.

C.2 Gradient wind speed

The gradient wind speed, $\bar{V}_g$, should be obtained from official Meteorological Office wind maps of the average wind speeds over the period of 1 h, independent of direction, at the top of the boundary layer of air flow above the site of the structure, and having an annual probability of occurrence of 0.02. An approximate map for the UK is shown in Figure C.1, based on an assumed gradient wind height of 900 m.

C.3 Site reference wind speed

The site reference wind speed to be used for design, $\bar{V}_r$, should be based on a value of the basic wind speed, $\bar{V}_B$, derived from:

$$\bar{V}_B = K_g\ \bar{V}_g$$

giving:

$$\bar{V}_r = \gamma_v K_d K_R K_g \bar{V}_g$$

where

- γ_v is the partial safety factor on wind speed to be obtained from Figure 2.1, appropriate to the type of structure (see **3.1.5**);
- K_d is determined in accordance with **3.1.3**;
- K_R is determined in accordance with **3.1.4**;
- K_g is a gradient wind speed reduction factor dependent on the gradient wind height and the coastal distance upwind of the site to be obtained from Figure C.2;
- $\bar{V}_g$ is the gradient wind speed, determined in accordance with **C.2**.

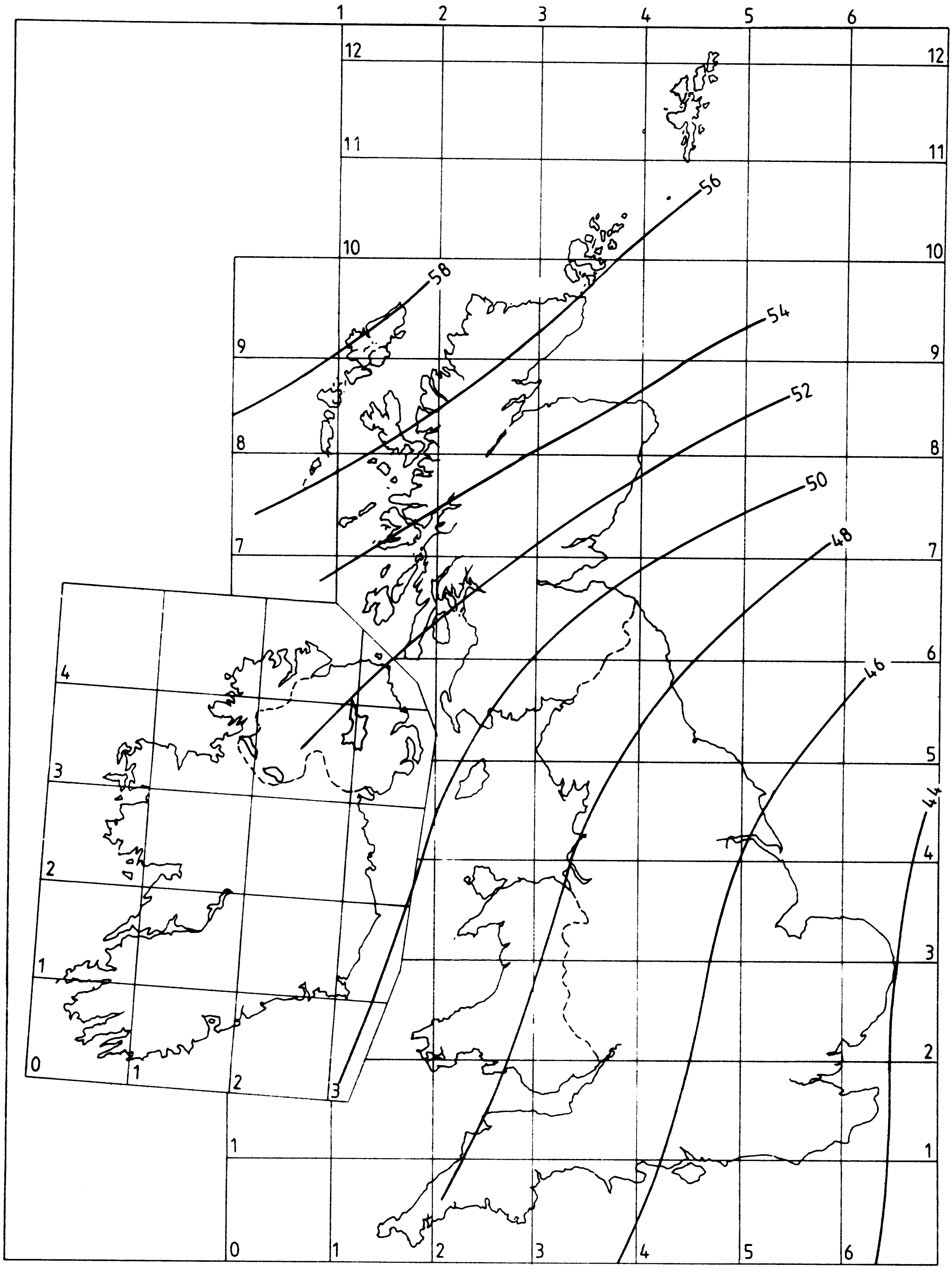


Figure C.1 — Mean hourly gradient wind speed, $\overline{V}_g$, for the UK (in metres per seconds)

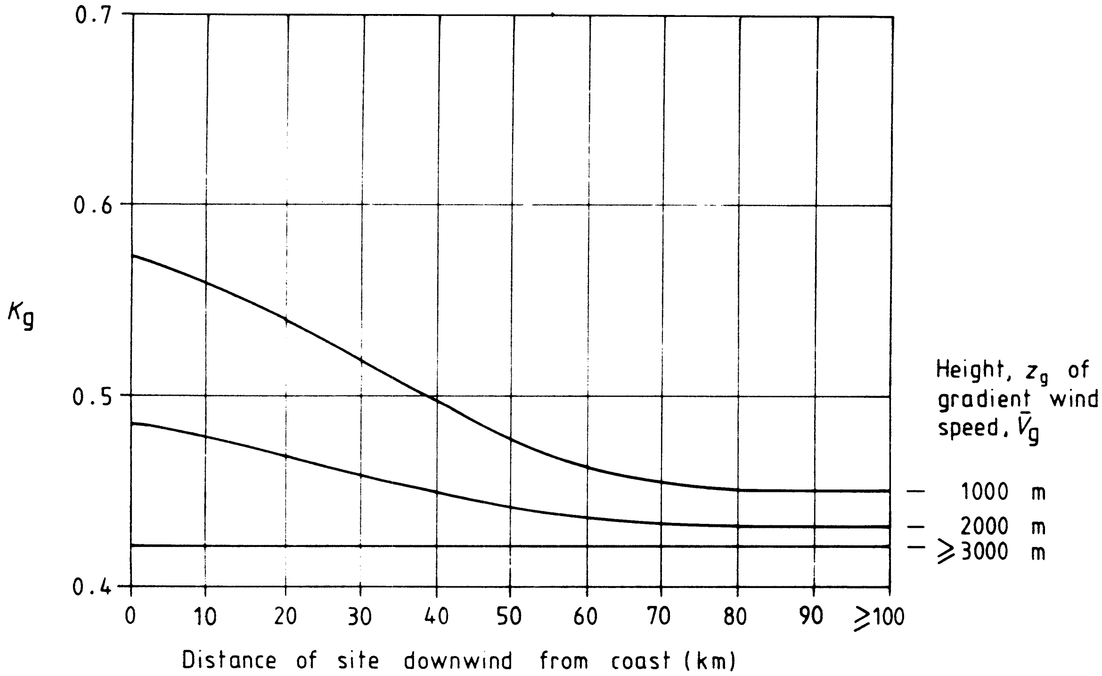


Figure C.2 — Gradient wind speed reduction factor, K_g

Appendix D Hill parameters in undulating terrain

The procedure for defining hill-site position and effective hill height, H_h, (see **3.2**) is shown in Figure D.1.

1. Plot profile for 8 km in front of site in windward direction
2. Draw typical slope, β_h, from hill on which tower is situated (through C). If site is behind brow of hill draw average slope in windward direction
3. Draw horizontal through minimum height level, h_{min}, within 8 km distance
4. Find average height, h_{av}, over distance between 8 km position (o) and intersection of slope line with minimum height line (D)
5. Draw horizontal height ($h_{av} - h_{min}$) above average height level

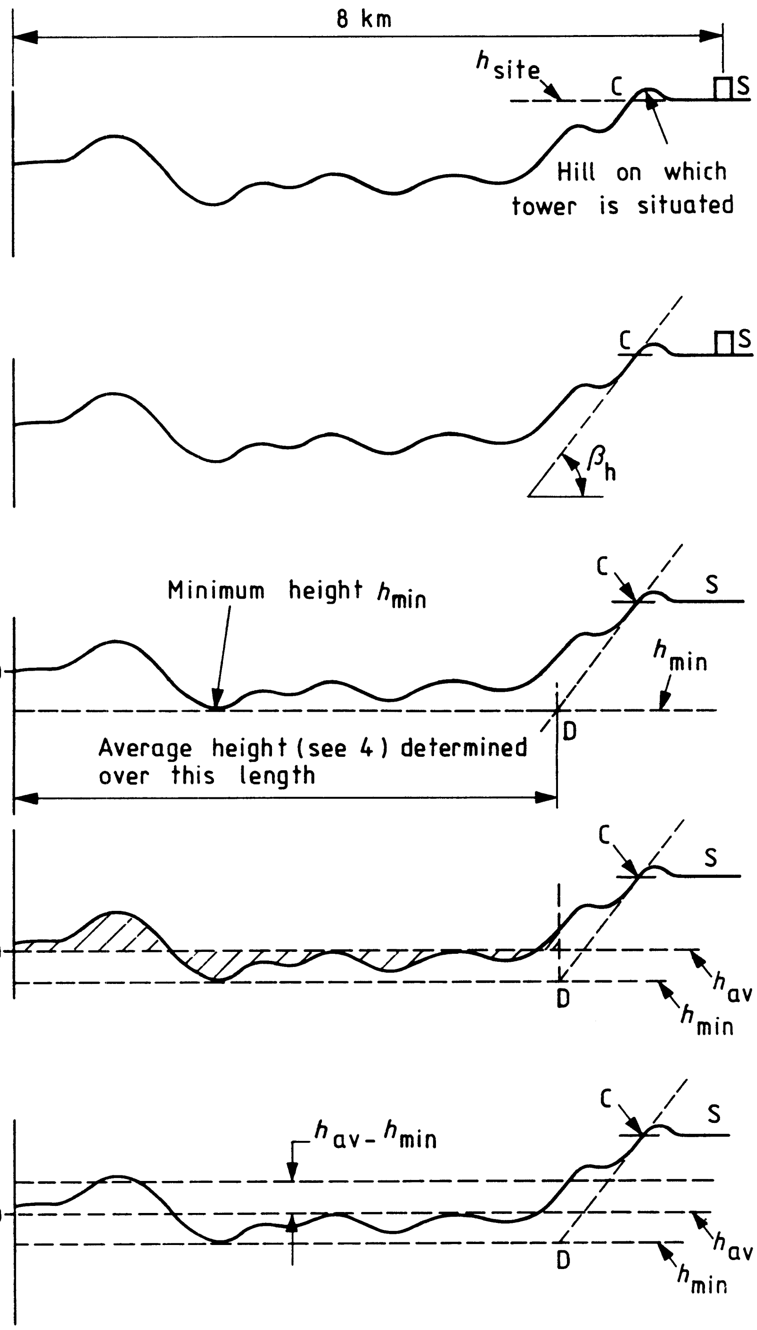
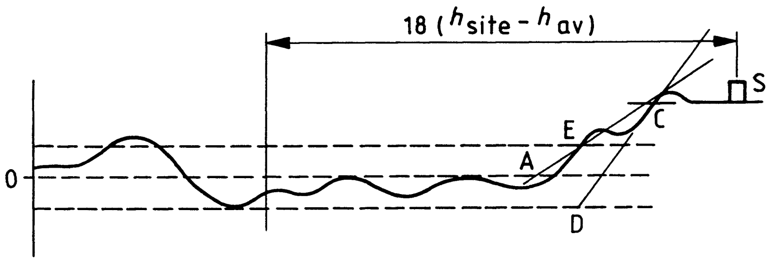
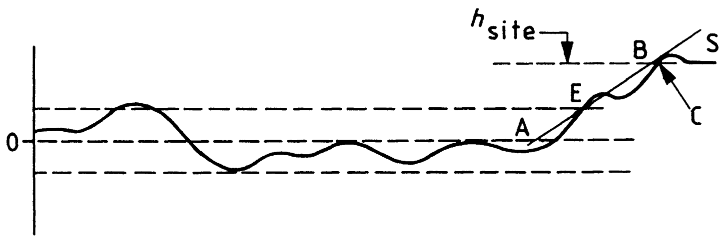


Figure D.1 — Hill parameters in undulating terrain

6. Where this line intersects profile within distance 18 (h_{site} – h_{av}) from site, draw slope through intersection point (E) to profile of hill on which tower is situated. This meets average level at A. (If more than one intersection use point furthest from tower within prescribed distance)



7. Draw horizontal through site level to intersect this slope, h_{site}, at B



8. The hill is then defined as OABS, with the effective hill parameters, for the purposes of **3.2** taken as:

$H_h = h_{site} - h_{av}$

$\sin \bar{\beta}_h = H_h/AB$

x = BS

where

$\bar{\beta}_h$ is the mean hill slope;

x is the distance of the site downwind from the crest.

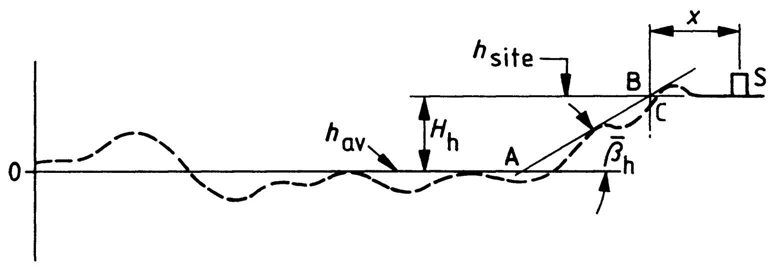


Figure D.1 — Hill parameters in undulating terrain

Appendix E Parameters for spectral analytical methods

E.1 Wind parameters

When spectral analytical methods are used (see **5.1.3** and **5.4**), the following parameters may be adopted in the absence of more accurate information.

a) *Longitudinal scale of turbulence.* The longitudinal scale of turbulence L_w (in m) may be taken as:

$$L_w = 150 \left(\frac{z - h_e}{10}\right)^{\alpha} \text{ or } 150\, K_\mu \left(\frac{z - h_e}{10}\right)^{\alpha - \mu} \quad \text{as appropriate for } z > 10 + h_e$$

for $z < 10 + h_e$, the value at $z = 10 + h_e$ should be adopted.

b) *Timescale of turbulence.* The timescale of turbulence may be taken as:

$$T_w = \frac{L_w}{\bar{V}_z} \text{ or } \frac{150}{\bar{V}_r}$$

c) *R.M.S. value of the gust velocity.* The r.m.s. value of the gust velocity in the direction of the wind, $\sigma(V)_w$ may be taken as:

$$\sigma(V)_w = \frac{\bar{V}_r}{3.3}\left(\frac{1.6}{K_R} - 1\right)$$

d) *Intensity of turbulence,* The intensity of turbulence, I_w, which relates $\sigma(v)$ to the mean hourly wind speed, $\bar{V}_z$, at any height, z, may be taken as:

$$I_w = \frac{\sigma(V)_w}{\bar{V}_z} \text{ for } z \geqslant 10 + h_e$$

For $z < 10 + h_e$, the value of I_w at $z = 10 + h_e$ should be adopted.

e) *Gust velocity spectrum.* The power spectrum of the longitudinal gust component, $S^v(n)$, may be taken as:

$$S^v(n) = \frac{55\left(\frac{1.6}{K_R} - 1\right)^2 \bar{V}_r}{\left\{1 + 0.5\left(\frac{1800\, n_1}{\bar{V}_r}\right)^2\right\}^{5/6}}$$

f) *Coherence.* The coherence, defining correlation, $R^v(s,n)$, may be taken as:

$$R^v(s,n) = e^{-(8 s n_1/\bar{V}_z)}$$

g) *Crosswind parameters.* Where crosswind parameters are required these may be taken as proportions of the longitudinal parameters as follows.

1) The crosswind scale of turbulence, $L_x = 0.5\, L_w$ [see a)].

2) The r.m.s. value of the crosswind gust velocity, $\sigma(V)_x = 0.7\, \sigma(V)_w$ [see c)].

3) The crosswind intensity of turbulence, $I_x = 0.7 I_w$ [see d)].

where

z is the height above site ground level (in m);

α is the power law index of wind speed variation with height determined in accordance with **3.2**;

$\bar{V}_z$ is the mean wind speed at height z above site ground level determined in accordance with **3.2** (in m/s);

$\bar{V}_r$ is the site reference wind speed determined in accordance with **3.1.5** (in m/s);

n_1 is the fundamental natural frequency of the tower (in Hz);

K_R is the terrain roughness factor determined in accordance with **3.1.4**;

s is the separation between two reference points in the z direction;

K_μ and μ are as defined in **3.2.2**.

E.2 Damping

E.2.1 *General*

The total damping available, $\Sigma\delta$, should be taken as:

$$\Sigma\delta = \delta_s + \delta_a$$

where

δ_s is the logarithmic decrement of structural damping, determined in accordance with **E.2.2**;

δ_a is the logarithmic decrement of aerodynamic damping, determined in accordance with **E.2.3**.

E.2.2 *Structural damping*

The structural damping should be assessed by reference to measurements in calm air on existing structures having size, form of construction and foundation conditions similar to the tower under consideration. Alternatively, the logarithmic decrement of structural damping, δ_s may be taken as:

$$\delta_s = K_\delta\, \delta_T$$

where

δ_T is the logarithmic decrement for the superstructure appropriate to the type of connection used, to be obtained from Table E.1;

K_δ is a factor to allow for the influence of foundation characteristics, to be obtained from Table E.2.

E.2.3 *Aerodynamic damping*

The logarithmic decrement of aerodynamic damping, δ_a, for vibrations in the plane of the wind should be calculated as:

$$\delta_a = \frac{\rho_a\, \Sigma R_{WT}\, \bar{V}_H}{2 n_1\, \Sigma m_T}$$

where

ΣR_{WT} is the sum of wind resistances in the direction of the wind, determined in accordance with section 4, of the panels in the top third of the height of the tower;

Σm_T is the total mass of the top third of the height of the tower;

$\bar{V}_H$ is the mean wind speed at the level of the top of the tower, determined in accordance with **3.1** and **3.2**;

n_1 is the fundamental natural frequency of the tower (in Hz);

ρ_a is as defined in **5.2.1**.

Table E.1 — Logarithmic decrements of damping, δ_T, for tower superstructure

Nature of structural connections	Surface finish at connections	Logarithmic decrement δ_T
All welded or all friction grip or fitted bolted	All finishes	0.015
Welded bracings: bolted flange plate connection to legs	All finishes	0.015
Welded bracings: black bolted gusset connection to legs	Cleaned, unpainted Gritblasted, metalsprayed Galvanized	0.06 0.045 0.03
Black bolted bracing: bolted flange plate connection to legs	Cleaned, unpainted Gritblasted, metalsprayed Galvanized	0.04 0.03 0.02
Black bolted bracing: black bolted gusset connection to legs	Cleaned, unpainted Gritblasted, metalsprayed Galvanized	0.08 0.06 0.04

Table E.2 — Damping factor, K_δ, for various types of foundation

Type of foundation	Factor, K_δ
Piled foundation or spread footing on stiff soil or rock	1
Spread footing on medium stiff soil	1.5
Spread footing on soft soil	3

Appendix F Method of assessment for classification of quality

F.1 Reliability of a tower

The reliability of a tower depends in part on the quality of the materials and workmanship used in its construction, and on the adequacy of the maintenance after its erection. It also depends on the degree of control, checking of the design and installation. The quality categories to be adopted for use with this code are set out in **2.3**, and the procedure detailed in **F.2** may be used as a guide to determine the appropriate quality classification.

F.2 Quality assessment

The material quality, fabrication and erection, design checking, inspection and maintenance should be individually rated in accordance with Table F.1. The quality category may then be determined from Table F.2 after summing the appropriate rating for each of the five items in Table F.1.

The material rating should take into account the control and testing procedures to be adopted with respect to material strength and to the methods of ensuring the correct identification of material used in construction. Workmanship ratings should allow for tolerances in member size and straightness and in fabricating connections, for the quality of any welding and for the standards required in erection, with due regard to the assumptions made in the design in respect of each of these items.

Table F.1 — Assessment of quality rating

Item	Quality	Rating
1. Design	a) In accordance with all relevant standard and codes, taking account of all load cases and condition of service and, where appropriate three-dimensional and dynamic effects	2
	b) In accordance with appropriate standards or codes, but utilizing simplified procedures with regard to secondary effects	1
	c) Simplified design procedure only	0
2. Design checking and testing	a) Type testing with design modification if failure occurs at below factored design loading	2
	b) Independent check of design	1
	c) No design checking	0
3. Material	a) In accordance with British Standard or equivalent	1
	b) Not in accordance with British Standard or equivalent	0
4. Workmanship and fabrications inspection	a) Very good, in accordance with British Standard or equivalent	2
	b) Good, in accordance with non-standard specification	1
	c) Adequate	0
5. Inspection and maintenance	a) Regular comprehensive inspection and rectification of deficiencies	2
	b) Selective inspection and maintenance	1
	c) Visual final; no maintenance	0

Table F.2 — Determination of quality classification

Quality class	Total rating
A[a]	7 to 9
B	4 to 6
C	0 to 3

[a] In addition, a class A structure should have a rating of at least 1 from each item 1 to 5 in Table F.1.

Appendix G Equations used for the production of the curves in the figures

G.1 General

As an alternative to obtaining values from the graphs given in the figures in this code, the equations given in **G.3** to **G.5** may be used for calculation of the required values; these equations should only be used within the bounds of the figures themselves.

G.2 Symbols

Some symbols used in this appendix are additional to those used in the main text. Some are used more than once in this appendix but only relate to the specific figure for which they are defined.

G.3 Meteorological parameters

G.3.1 *Figure 3.4: terrain profile index, μ*

The terrain profile index, μ, is given by:

$$\mu = \alpha - \bar{\mu}$$

where

α is as defined in **3.2.1**;

$$\bar{\mu} = \left\{ \left(144 + \frac{8}{\tan \beta'}\right) \alpha - 18 \right\} \Big/ \left(162 + \frac{8}{\tan \beta'}\right)$$

where

$\beta' = 1.221(e^{0.2176\bar{\beta}_h} - 1)$ where $\bar{\beta}_h$ is the mean inclination of the slope to the horizontal (in degrees) (see **3.2.2**).

G.3.2 *Figure 3.5: terrain profile factor, K_μ*

The terrain profile factor, K_μ, is given by:

$$K_\mu = \{ (1 + \alpha)/(1 + \bar{\mu})\}^\alpha (0.8\, H_h)^\mu$$

where

α is as defined in **3.2.1**;

μ, $\bar{\mu}$ are as defined in **G.3.1**;

H_h is the height of the hill above the general terrain level (in m) (see **3.2.2**).

G.4 Wind resistance

G.4.1 *Figure 4.2: wind incidence factor, K_θ*

The wind incidence factor, K_θ, is given by:

$K_\theta = 1.0 + K_1 K_2 \sin^2 2\theta$ for square towers

$K_\theta = \dfrac{A_c + A_{c'}}{A_F} + \dfrac{A_f}{A_F}(1 - 0.1 \sin^2 1.5\theta)$ for triangular towers

where

$$K_1 = \frac{0.55A_f}{A_F} + \frac{0.8\,(A_c + A_{c'})}{A_F}$$

K_2 = 0.2 for $0 \leqslant \phi \leqslant 0.2$ and $0.8 \leqslant \phi \leqslant 1.0$;

= ϕ for $0.2 < \phi < 0.5$;

= $1 - \phi$ for $0.5 < \phi < 0.8$;

where

ϕ is as defined in **4.2.1**;

A_f, A_F, θ are as defined in **4.2.1**;

A_c, $A_{c'}$ are as defined in **4.2.2**.

G.4.2 ***Figure 4.3: overall drag coefficients for square and triangular towers,*** C_N

The drag coefficients for towers composed of flat-sided members, C_{Nf}, subcritical circular-section members, C_{Nc}, and supercritical circular-section members, $C_{Nc'}$, are given by:

$C_{Nf} = 1.76\ C_1\ [1 - C_2\ \phi + \phi\ 2]$

$C_{Nc} = C_1\ (1 - C_2\ \phi) + (C_1 + 0.875)\ \phi^2$

$C_{Nc'} = 1.9 - \sqrt{\{(1 - \phi)\ (2.8 - 1.14C_1 + \phi)\}}$

where

C_1 is equal to:

2.25 for square towers;
1.9 for triangular towers;

C_2 is equal to:

1.5 for square towers;
1.4 for triangular towers;

ϕ is as defined in **4.2.1**.

G.4.3 ***Figure 4.4: shielding factor,*** η_f

The shielding factor, η_f, is given by:

$\eta_f = (1 - \phi)^{1.89}$

where

ϕ is as defined in **4.2.1**.

G.4.4 ***Figure 4.5: drag coefficients for single frames,*** C_n

The drag coefficients for single frames, C_n, is given by:

$$C_n = C_{nf} \left\{ \frac{A_f + (0.6 + 0.4\,\phi^2)\,A_c + (0.33 + 0.62\,\phi^{5/3})\,A_{c'}}{A_F} \right\}$$

where

C_{nf} is the drag coefficient for single frames equal to:

$1.58 + 1.05\ (0.6 - \phi)^{1.8}$ for $\phi \leqslant 0.6$;

$1.58 + 2.625\ (\phi - 0.6)^2$ for $\phi > 0.6$;

A_f, A_F, ϕ are as defined in **4.2.1**;

A_c, $A_{c'}$ are as defined in **4.2.2**.

G.5 Structural response to wind

G.5.1 *General*

In the equations throughout **G.5**:

H is the full height of tower (in m);

z is the height above the ground to the level at which the force is to be calculated (in m);

α is the power law index of wind speed variation with height (see **3.2.1**);

K_R is the terrain roughness factor (see **3.1.4**).

G.5.2 *Figure 5.1: size factor, B*

The size factor, B, is given by:

$$B = K_1 K_2 \left(\frac{3.976}{K_R} - 2.485\right)$$

where

$$K_1 = \left(1 + \frac{\alpha}{2}\right)\left(\frac{10}{H - z}\right)^{\alpha}$$

$$K_2 = \left\{\frac{2}{s} + \frac{2}{s^2}\left(e^{-s} - 1\right)\right\}^{1/2}$$

$$s = \left(\frac{H - z}{100.8}\right)\left(\frac{10}{H - z}\right)^{\alpha}$$

G.5.3 *Figure 5.2: height factor, j*

The height factor, j, is given by:

$$j = I_1^{\alpha}\{1 - (1 - I_1)^2\} / \{1 - (1 - I_1)^{\alpha + 2}\}$$

where

$$I_1 = \frac{H - z}{H}$$

G.5.4 *Figure 5.3: basic gust response factor, G_B*

The basic gust response factor, G_B, is given by:

G_B = maximum of G_1 or G_2

where

$$G_1 = K_1 K_2 \left(\frac{3.976}{K_R} - 2.485\right)$$

where

$$K_1 = \left(1 + \frac{\alpha}{2}\right)\left(\frac{10}{H}\right)^{\alpha}$$

$$K_2 = \left\{\frac{2}{s_1} + \frac{2}{s_1^{\,2}}\left(e^{-s_1} - 1\right)\right\}^{1/2}$$

$$s_1 = \left(\frac{H}{100.8}\right)\left(\frac{10}{H}\right)^{\alpha}$$

$$G_2 = K_3 K_4 K_5 \left(\frac{3.976}{K_R} - 2.485\right)$$

where

$$K_3 = \left(1 + \frac{\alpha}{2}\right)\left(\frac{10}{K_6}\right)^{\alpha}$$

$$K_4 = \left\{\frac{2}{s_2} + \frac{2}{{s_2}^2}\left(e^{-s_2} - 1\right)\right\}^{1/2}$$

$$K_5 = \left(\frac{K_6}{H}\right)^{\alpha}\left\{1 - \left(1 - \frac{K_6}{H}\right)^2\right\} \Big/ \left\{1 - \left(1 - \frac{K_6}{H}\right)^{\alpha + 2}\right\}$$

$$s_2 = \left(\frac{K_6}{100.8}\right)\left(\frac{10}{H}\right)^{\alpha}$$

$K_6 = \dfrac{H}{10}$ but not less than 10 m.

G.5.5 ***Figure 5.4: shear loading factor,*** K_q

The shear loading factor, K_q, is given by:

$$K_q = \frac{1}{f_q}\left(\frac{K_1}{K_2}\right)^{1/2}$$

where

$$K_1 = \left(1 - K_3 + \frac{{K_3}^2}{3}\right) - \frac{1}{s}\left\{1 - K_3 + \frac{{K_3}^2}{2} + e^{-s}(K_3 - 1)\right\} - \frac{{K_3}^2 e^{-s}}{s^2} + \frac{{K_3}^2(1 - e^{-s})}{s^3}$$

$$K_2 = \left(1 - \frac{1 - e^{-s}}{s}\right)$$

where

$$K_3 = (1 - f_q)\left(\frac{2 + \alpha}{1 + \alpha}\right)$$

where

$f_q = \bar{F}_q / \bar{F}$

where

$\bar{F}$ is the mean shear load, normal to the tower, on tower of height, H, at level z_q;

$\bar{F}_q$ is the mean shear load carried by the bracings at level z_q (in the same direction as $\bar{F}$).

$$s = \frac{H - z_q}{75}\left(\frac{10}{z_q}\right)^{\alpha}$$

G.5.6 ***Figure 5.5: length factor for cable,*** K_L

The length factor for cable, K_L, is given by:

$$K_L = \left\{\frac{2}{s} + \frac{2}{s^2}\left(e^{-s} - 1\right)\right\}^{1/2}$$

where

$$s = \left(\frac{L}{100.8}\right)\left(\frac{10}{z_c}\right)^{\alpha}$$

where

L is the length of cable (in m);

z_c is the mean cable height (in m).

G.5.7 *Figure 5.7: cable height factor,* K_z

The cable height factor, K_z, is given by:

$$K_z = 2.121 \left(\frac{10}{z_c}\right)^{\alpha} \left(\frac{1.6}{K_R} - 1\right)$$

where

z_c is the mean cable height (in m).

Publications referred to

BS 8100, *Lattice towers and masts.*
BS 8100-2, *Guide to the background and use of Part 1 “Code of practice for loading”.*
DD 133, *Strength assessment of members for towers of lattice construction.*

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