

Lattice towers and masts —

Part 1: Code of practice for loading

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Committees responsible for this British Standard

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- British Steel Industry
- British Telecommunications plc
- Constructional Steel Research and Development Organisation
- Department of Energy (Electricity Division)
- Department of the Environment (Building Research Establishment)
- Electricity Supply Industry in England and Wales
- Home Office
- Independent Broadcasting Authority
- Institution of Civil Engineers
- Institution of Structural Engineers
- Joint Radio Committee of the Nationalized Power Industries
- Ministry of Defence
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- North of Scotland Hydro-electric Board
- Transmission and Distribution Association (BEAMA Ltd.)

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Foreword

This part of BS 8100 has been prepared under the direction of the Civil Engineering and Building Structures Standards Committee.

The start and finish of text introduced or altered by Amendment No. 1 is indicated in the text by tags **[A1]** **[A1]**.

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

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”.

Part 3: Code of practice for strength assessment of members of lattice towers and masts;

DD 133 has been superseded by BS 8100-3. Reference to DD 133 in the text has been replaced by reference to BS 8100-3.

Part 4: Code of practice for loading of guyed masts.

Part 1 only applies to the loading of self supporting towers. Since the publication of this part of BS 8100 considerable advances have been made and experience gained in wind engineering. These advances have been reflected in the UK wind loading code of practice BS 6399-2 first published in 1995 and incorporated in BS 8100-4:1995.

Resources have not been available to update BS 8100-1 to be compatible with BS 8100-4 and as a result, inconsistencies between the two documents will be produced in assessing the wind speeds at any given site. Generally, but not exhaustively, it has been found that BS 8100-1 provides higher values of wind speed at a given site than use of BS 8100-4.

However this is not always the case for the prediction of wind speeds at low levels [below $(10 + h_e)$ metres]. In addition no warning is given about the likely accelerated flows for towers mounted on or near to large buildings. Accordingly Amendment No. 1 has been issued to address these aspects. In view of the likely introduction of Eurocodes in this area in the not too distant future it has been accepted that use of BS 8100-1 should continue until that time.

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. As a code of practice, this British Standard takes the form of guidance and recommendations. It should not be quoted as if it were a specification and particular care should be taken to ensure that claims of compliance are not misleading.

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	
Mr D G Clow	
Dr A R Flint OBE	
Mr R R Gibbon	
Mr M V Huband	
Mr M J Lambert	
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Mr A Pardoe	
Mr J Short	
Mr H B Walker	
Dr T A Wyatt	

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Summary of pages

This document comprises a front cover, an inside front cover, pages i to v, a blank page, pages 1 to 69 and a back cover.

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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 Section 3 and Section 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  separate sheet provided with this standard .

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

1.3.2 Subscripts

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

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

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, 2.3 and 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

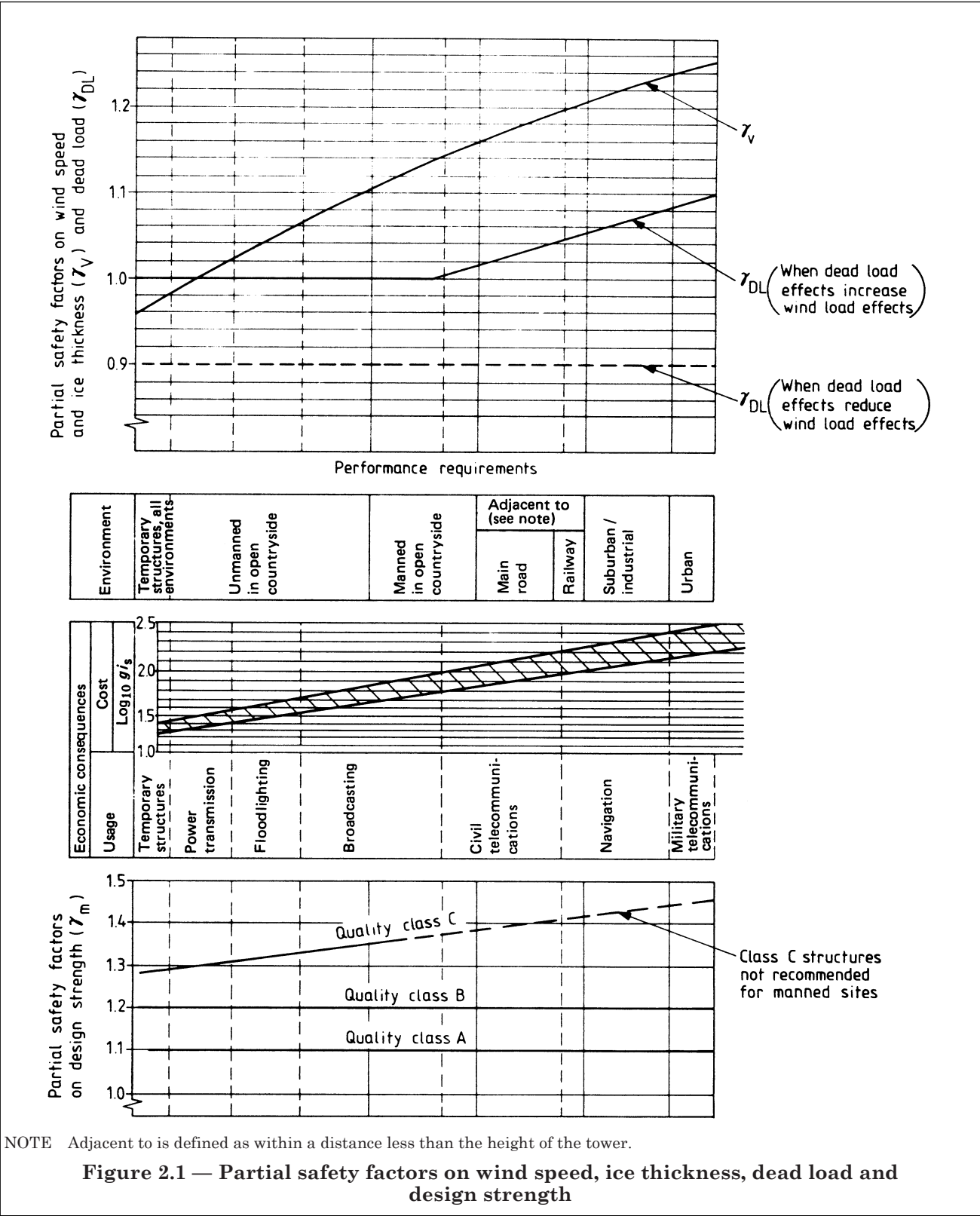
The potential total cost, at net present value, of failure within the design service life should be estimated. This should include the cost of removal and replacement of the tower and its ancillary attachments and all contingent costs such as loss of revenue, third-party claims and loss of amenity. The 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.



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, 2.3.3 and 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 $\boxed{A_1}$ BS 8100-3 $\boxed{A_1}$ 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 Standards¹⁾. 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 Standards¹⁾ 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.

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 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.

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 Section 3, Section 4 and Section 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 $\square{A_1}$ BS 8100-3 $\square{A_1}$, divided by the partial safety factor γ_m .

The characteristic strength given in $\square{A_1}$ BS 8100-3 $\square{A_1}$ 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:

- the quality class of the tower;
- 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 $\square{A_1}$ BS 8100-3 $\square{A_1}$, may only be used where the strengths can be shown to be characteristic strengths derived on the same basis as taken for $\square{A_1}$ BS 8100-3 $\square{A_1}$.

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, 3.1.3, 3.1.4 and 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, 3.1.3, 3.1.4 and 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

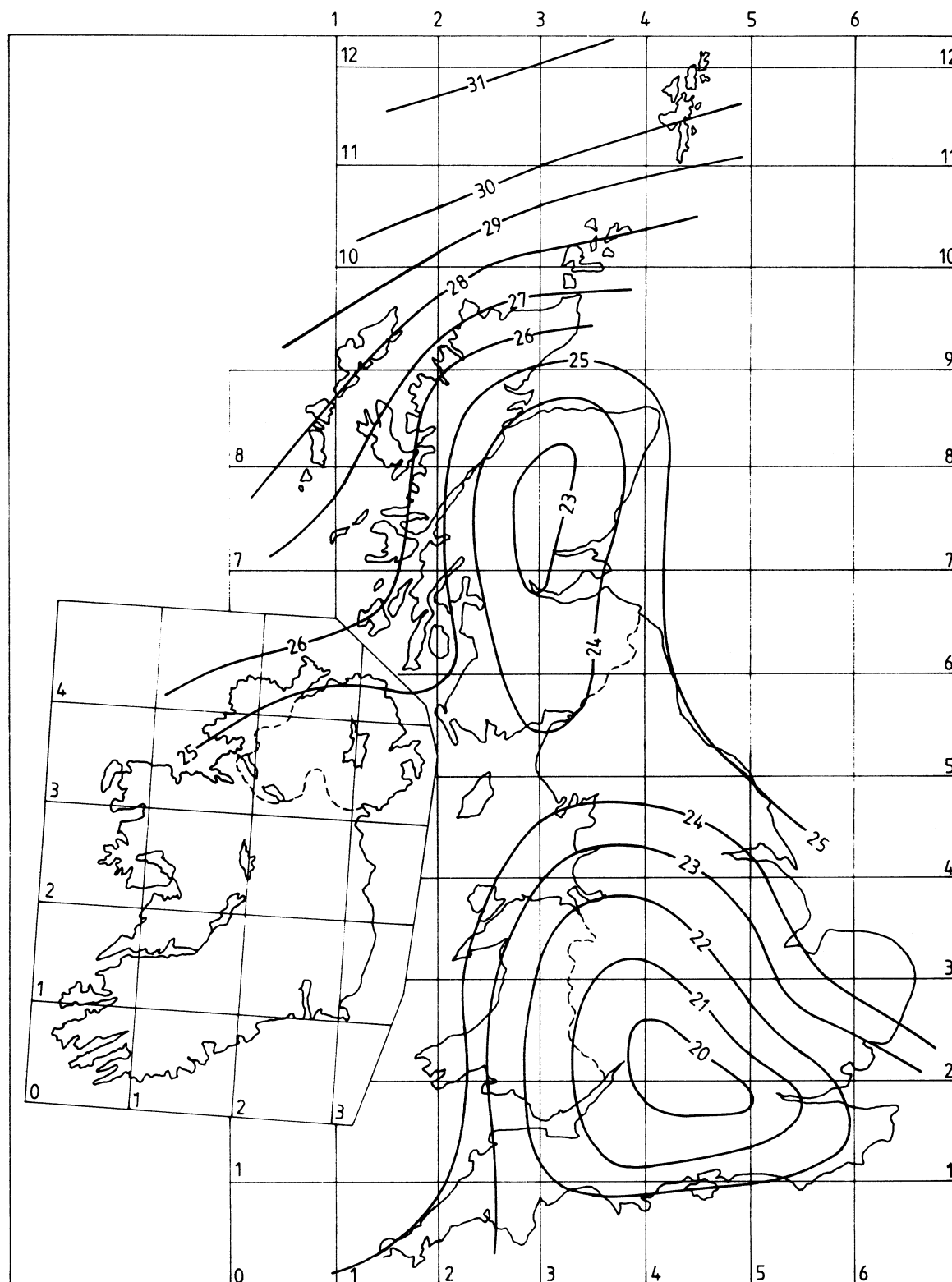
The basic wind speed, $\bar{V}_B$, should be obtained from wind maps based on Meteorological Office data of the maximum mean hourly wind speed independent of direction at a height 10 m above level ground in assumed basic open terrain category III (see Table 3.1) at the site of the structure, and having 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 3.5.1b)], 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 $\pm 30^\circ$ 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 $\bar{V}_B$.

NOTE 3 Values are in metres per second.

Figure 3.1 — Basic mean hourly wind speeds for UK, $\bar{V}_B$ (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.

- 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, $\bar{V}_r$, 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.
- K_R may be derived from the statistical analysis of records taken at the site in accordance with Appendix A.

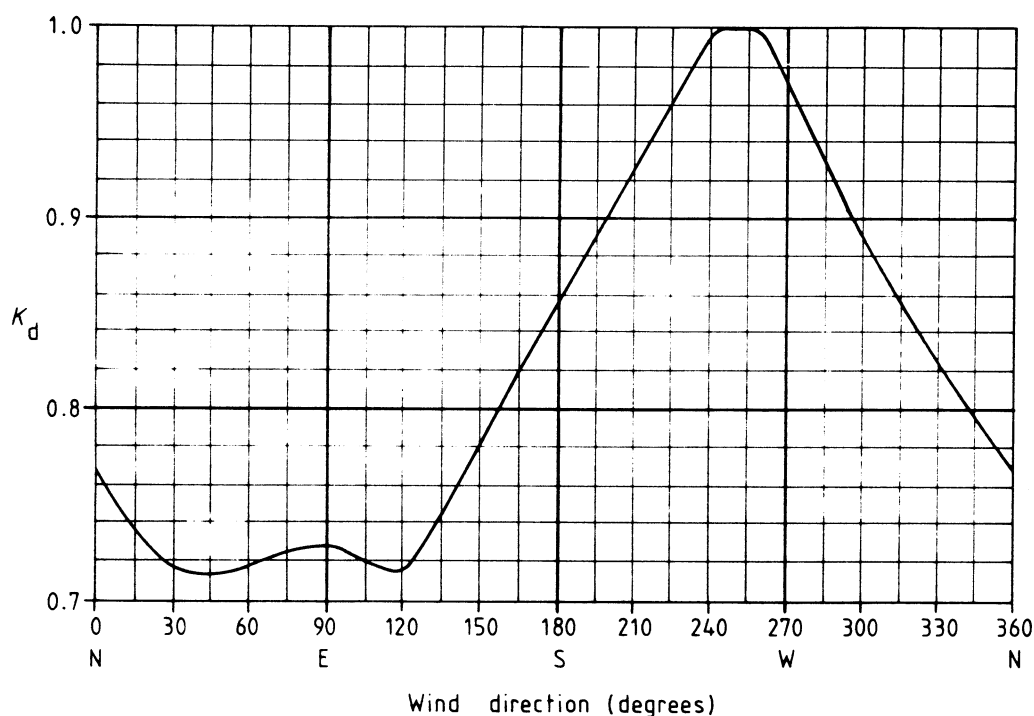


Figure 3.2 — Wind direction factor, K_d

3.1.5 Site reference wind speed

The site reference wind speed, $\bar{V}_r$, 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:

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

where

$\bar{V}_B$ 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, $\bar{V}_B$, 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 $\bar{V}_g$ 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 ($z_0 = 0.003$ 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 ($z_0 = 0.01$ m)	Flat grassland, parkland or bare soil, without hedges and with very few isolated obstructions	1.10	0.14	0
III ($z_0 = 0.03$ m)	<i>Basic open terrain</i> 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 ($z_0 = 0.10$ m)	Farmland with frequent high hedges, occasional small farm structures, houses or trees	0.86	0.19	2
V ($z_0 = 0.30$ m)	Dense woodland, domestic housing typically covering 10 % to 20 % of the plan area	0.72	0.23	10

NOTE 1 z_0 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 ($z_0 \approx 0.8$ m), where towers rise above the general level of the surrounding buildings, category V should be adopted.

NOTE 6 Wind close to obstruction level ($z < 10 + h_e$) might be subject to significant variation due to local features. Care should be exercised and specialist advice might need to be sought when evaluating wind effects for towers situated close to the base of tall buildings or for those mounted on roof-tops. A1

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:

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

$$\bar{V}_z = \bar{V}_r \left[\left(\frac{0.25}{10 + h_e} \right) z + 0.75 \right] \text{ for } z < 10 + h_e \quad \text{A1}$$

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.

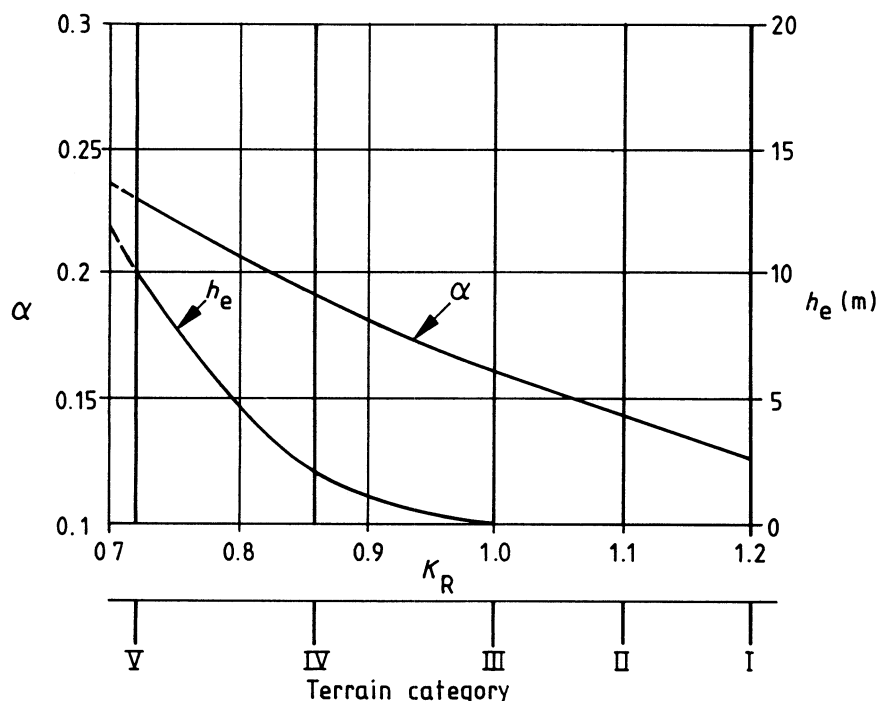


Figure 3.3 — Variation of power law index and effective height with terrain roughness

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:

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

$$\bar{V}_z = \bar{V}_r K_\mu \left[\left(\frac{0.25}{10 + h_e} \right) z + 0.75 \right] \quad \text{for } z < 10 + h_e$$

where

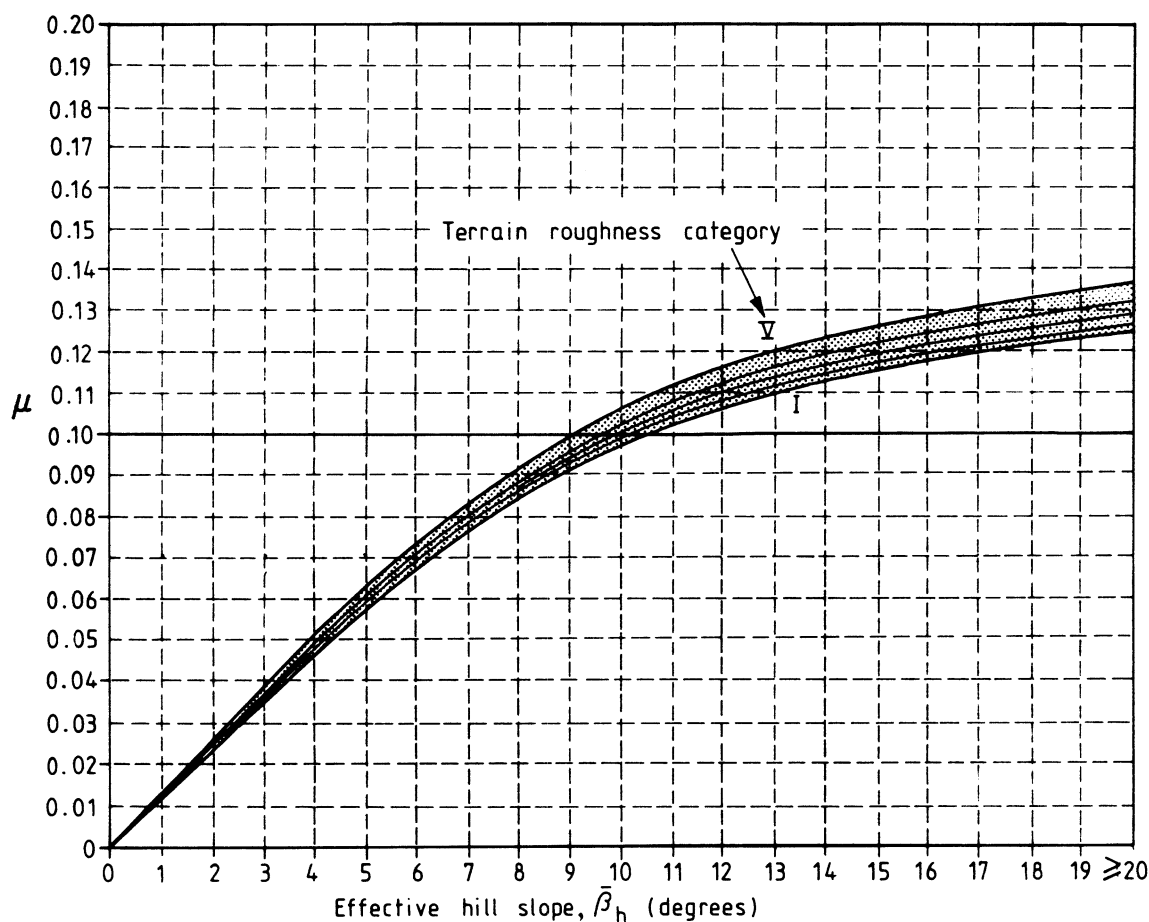
$\bar{V}_r$ is the site reference wind speed, determined in accordance with 3.1.5;

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:

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

$$\boxed{A_1} \bar{V}_z = \bar{V}_r K_{\mu x} \left[\left(\frac{0.25}{10 + h_e} \right) z + 0.75 \right] \quad \text{for } z < 10 + h_e \quad \boxed{A_1}$$

where

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

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

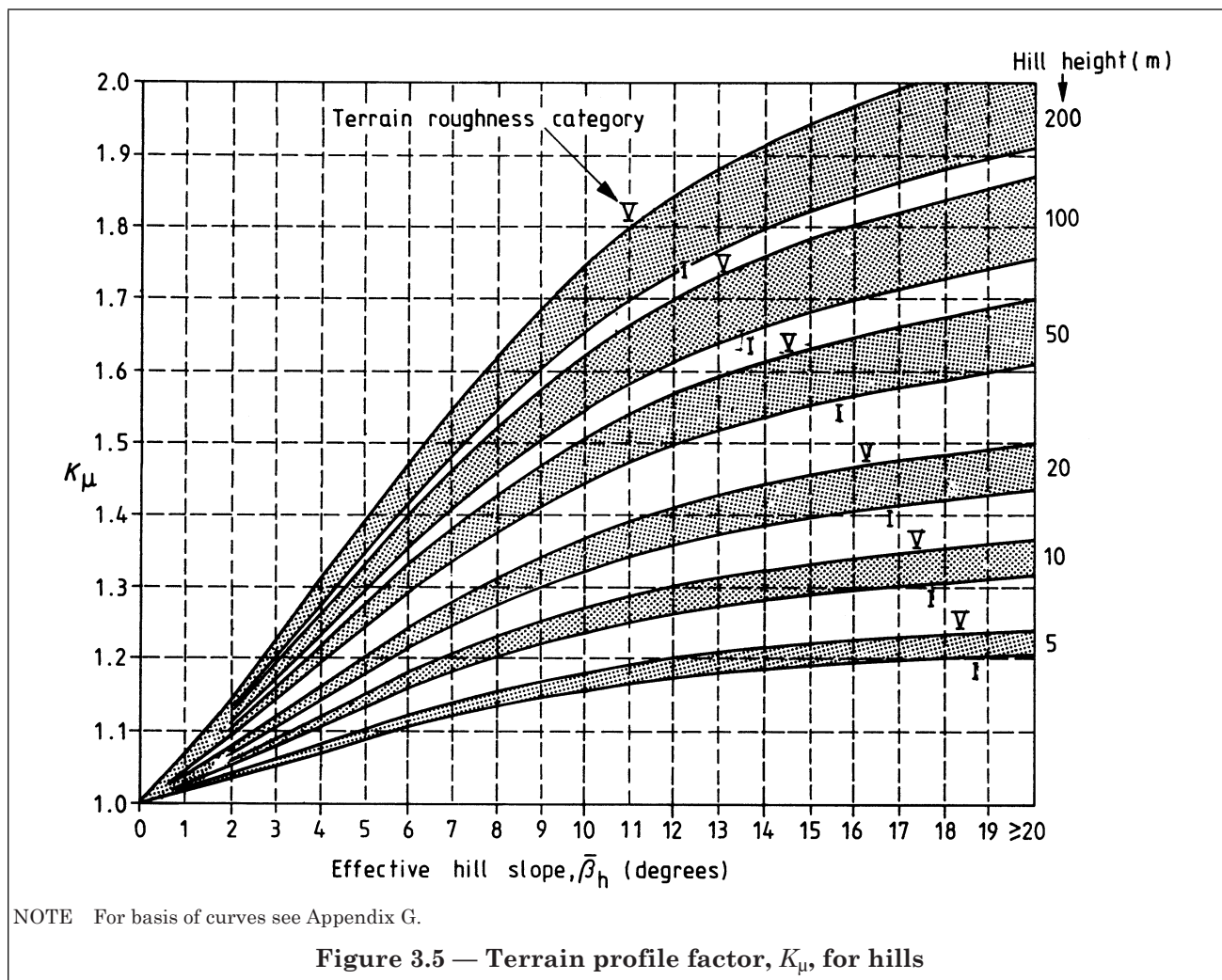
μ_x is the terrain profile index at a distance x from the crest of the hill given by:

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

$\bar{V}_r, \mu, K_\mu$ 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.



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, $\bar{V}_z$, 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.

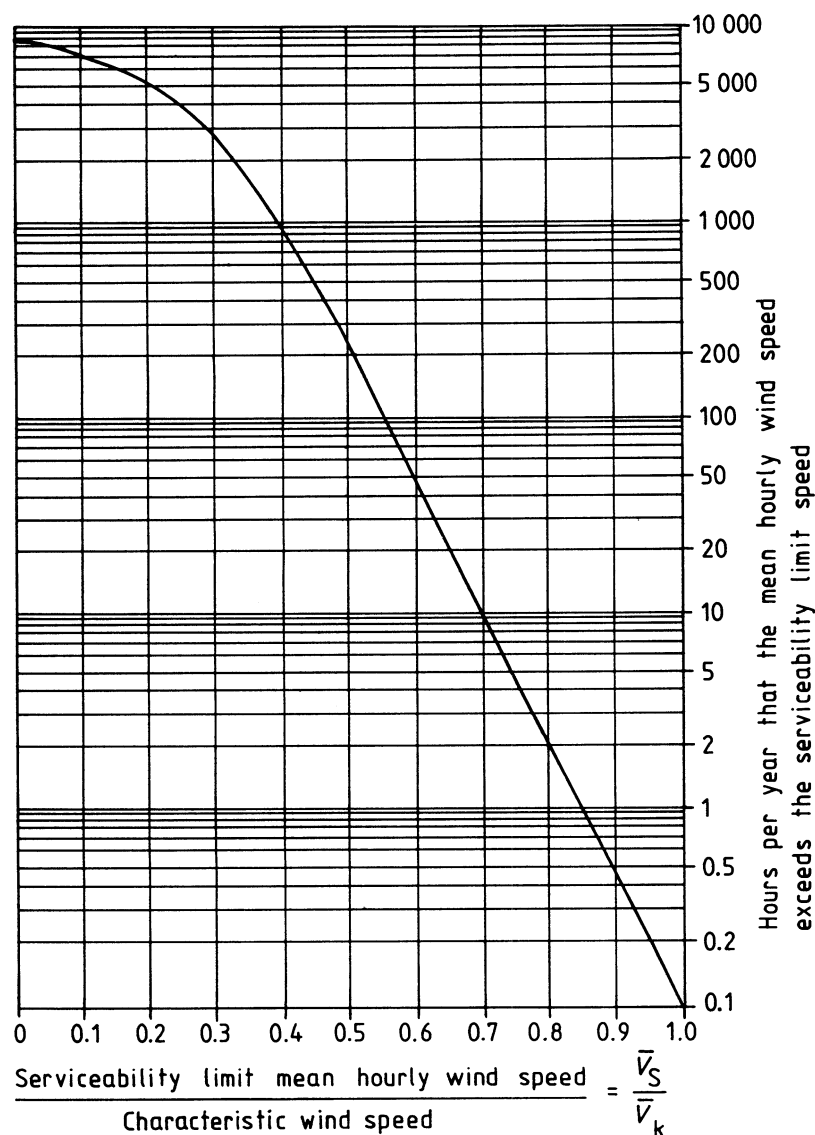


Figure 3.6 — Hours per year that the mean hourly wind speed exceeds the serviceability limit

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 $\pm 2.5\%$ 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.

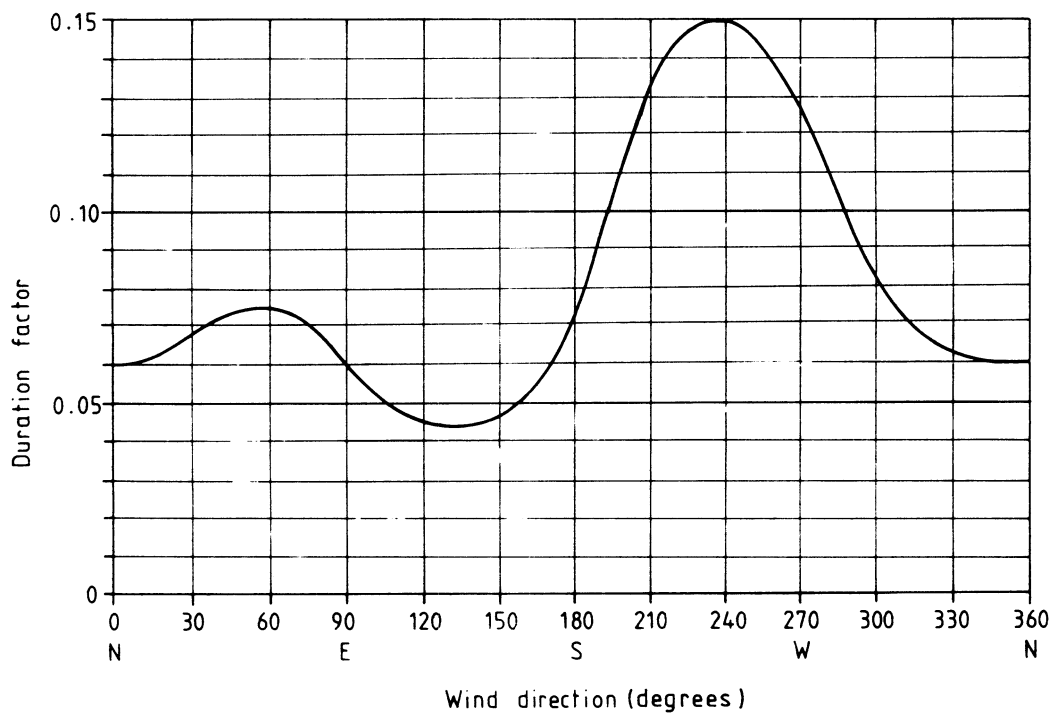


Figure 3.7 — Wind speed duration factor