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Lattice towers and masts —

Part 4: Code of practice for loading of guyed masts

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Committees responsible for this British Standard

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British Cement Association
British Constructional Steelwork Association Ltd.
British Masonry Society
Building Employer's Confederation
Department of the Environment (Building Research Establishment)
Department of the Environment (Construction Directorate)
Department of Transport
Federation of Civil Engineering Contractors
Institution of Civil Engineers
National Council of Building Material Producers
Royal Institute of British Architects
Timber Research and Development Association

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Foreword

This Part of BS 8100 has been prepared under the direction of Technical Committee B/525, Building and civil engineering structures. BS 8100 is a standard combining codes of practice with 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"*;

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

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

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.

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

This publication does not purport to include all the necessary provisions of a contract. Users are responsible for its correct application.

Compliance with a British Standard does not of itself confer immunity from legal obligations.

Summary of pages

This document comprises a front cover, an inside front cover, pages i to iv, pages 1 to 81 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 guyed masts of lattice construction.

The procedures given primarily apply to bolted, riveted or welded metallic structures composed of leg members and triangulated bracings. Section 5 may be inappropriate for masts of Vierendeel form, due to their stiffness characteristics. The response of such masts 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 United Kingdom 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 masts designed in accordance with this standard.

1.2 References

1.2.1 Normative references

This Part of BS 8100 incorporates, by dated or undated reference, provisions from other publications. These normative references are made at the appropriate places in the text and are listed on the inside back cover. For dated references, only the edition cited applies; any subsequent amendments to or revisions of the cited publication apply to this Part of BS 8100 only when incorporated in the reference by amendment or revision. For undated references, the latest edition of the standard applies, together with any amendments.

1.2.2 Informative references

This Part of BS 8100 refers to other publications that provide information or guidance. Editions of these publications current at the time of issue of this standard are listed on the inside back cover, but reference should be made to the latest editions.

1.3 Definitions

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

1.3.1
wind resistance
the resistance to the flow of wind offered by the elements of a mast and any ancillary items that it supports, given by the product of the drag (pressure) coefficient and a reference projected area

1.3.2
linear ancillary item
any non-structural component extending over several panels

NOTE Examples of linear ancillary items are waveguides, feeders, ladders and pipework.

1.3.3
discrete ancillary item
any non-structural component, concentrated within a few panels

NOTE Examples of discrete ancillary items are dish reflectors, aerials, lighting and insulators.

1.3.4
projected area
the shadow area of the element considered when projected on an elevation normal to the face of the mast under consideration

NOTE For wind blowing other than normal to the mast, a reference face is used for the projected area.

1.3.5
panel
a convenient division of a mast into vertical elements for the purposes of deriving projected areas and wind resistance

NOTE Panels are typically, but not necessarily, taken between intersections of legs and primary bracings.

1.3.6
section
a convenient division of a mast comprising several identical or nearly identical panels, for the purpose of deriving wind resistance

NOTE For the purposes of analysis it should be ensured that the mast column is subdivided into a sufficient number of sections between adjacent guy levels to model the mast loading and structural behaviour accurately. A minimum of four sections between adjacent guy levels should be used.

1.3.7
characteristic strength
the strength of a component having a prescribed probability of not being exceeded

NOTE For the purposes of this Part of BS 8100 the characteristic strength may be taken as the strength given in BS 8100-3 or BS 5950. In BS 5950, strengths are referred to as design strengths because the partial factor γ_m in BS 5950 is taken as 1.0.

1.3.8
design strength
the strength of a component obtained by dividing the characteristic strength (see **1.3.7**) by a partial factor γ_m

1.4 Symbols

1.4.1 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 **1.4.2**. Symbols are further clarified as appropriate in the text.

A		projected area
	or	cross-sectional area
b		width of mast
C		drag (pressure) coefficient
c		distance from leading edge of mast
D		diameter
	or	distance
E		elastic modulus
F		load effect in a member (e.g. force, shear or bending moment)
G		gust response factor
g		cost ratio
H		overall height
	or	height above mast base
h		height
I		inertia
i		service life
i_0		turbulence intensity
j		patch load pattern
K		factor
k		coefficient
L		projected length
l		length of guy
M		bending moment
m		mass per unit height
N		number
n		frequency
P		wind load

Q	parameter
R	wind resistance
R_e	effective Reynolds number
r	radial ice thickness
or	response
S	Strouhal number
or	wind factor
T	torque
or	wind factor
t	time
V	wind speed
$\bar{V}$	hourly mean wind speed
v	coefficient of variation
X	horizontal distance from crest of hill
or	spacing to upwind sheltered building
z	height
z_0	terrain roughness length
α	slope of guy to horizontal
β	parameter
γ_{DL}	partial safety factor on dead load
γ_m	partial safety factor on strength
γ_v	partial safety factor on wind speed and ice thickness
Δ	deflection
or	altitude
δ	logarithmic decrement of damping
η	shielding factor
θ	angle of wind incidence to the normal in plane
λ	factor
ν	kinematic viscosity of air
ρ	density
φ	solidity ratio
Ψ	angle of wind incidence to the longitudinal axis
or	tangent of upwind slope of topographic feature
ω	spacing ratio

1.4.2 Subscripts

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

A	ancillary item
a	aerodynamic
or	air
or	altitude
av	average
b	basic
C	cantilever
c	circular-section members
or	fetch

cr		critical
d		direction
e		effective
	or	excitation
F		face
f		flat-sided members
G		guy
g		gradient wind
H		mast height
h		topography
i		with ice
k		characteristic
L		length
l		level
M		bare mast or mast only
m		mast
N		overall
n		single frame
o		in the absence of wind
	or	obstruction
PL		patch load
R		resonant
r		reference
S		site
	or	serviceability
s		seasonal
	or	structure
	or	span
t		town
TL		turbulent length
w		with wind
W		in the direction of the wind
X		in the crosswind direction
Z		in the vertical direction
z		height z above ground level
θ		angle of wind incidence
—		due to hourly mean wind effects, e.g. $\bar{V}$, $\bar{P}$, $\bar{F}$
′		due to fluctuating wind effects, e.g. V', P', F'

Section 2. Performance

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 Part of BS 8100, consideration should be given to the reliability required of a mast 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 should also depend on the quality of materials and workmanship specified and achieved in construction.

Appropriate values of the safety factors should be determined in accordance with **2.2**, **2.3** and **2.4**.

For many masts, simple static analysis of the structure under factored and patch loads is sufficient to determine the loading effects. For masts which are likely to exhibit dynamic sensitivity, the spectral analytical method should be adopted. The criteria for when such methods are required are set out in **5.1.2**.

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

2.1.2 Service life

For the purposes of this Part of BS 8100, the service life i, in years, is taken as the intended period of service of the mast. A mast designed in accordance with this Part of BS 8100 may, if adequately maintained, provide consistent reliability beyond the period of its service life, that period being used only as a basis for the economic assessment of the appropriate annual risk of failure. However, the fatigue life may limit the period of reliable service.

During the construction of masts, the loading effects can differ greatly from those in service, and should be assessed. For partly erected masts and temporary guys a lower partial safety factor γ_v than for the in-service condition may be justified, depending on the consequences of failure of the partly completed structure.

2.2 Classification of required reliability

2.2.1 General

The safety factors to be used, appropriate to the reliability required of a mast, should be selected on the basis of the most onerous of the following:

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

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

2.2.2 Environmental conditions

The environmental category of the mast should be selected with due consideration of the potential risk to life in the event of the mast'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 1 indicates categories which should be used to select safety factors within the range appropriate to the environment of the mast.

2.2.3 Economic consequences or usage

2.2.3.1 ***General***

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

Reduced wind speeds can be used for temporary structures or during the erection of masts if the period of exposure is planned to be within summer months (see **3.1.3**). Safety factors for masts may also be reduced during their erection if it can be determined that the consequences of failure are lower than for the in-service condition.

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 mast 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 mast should then be evaluated.

Figure 1 indicates categories of the potential economic consequence of failure, represented as the logarithm of gi, where i is the service life.

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 mast as indicated in Figure 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 Classifications of quality

2.3.1 General

The reliability of a mast 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 Part of BS 8100 are given in **2.3.2**, **2.3.3** and **2.3.4**. The method of assessment given in Annex A may be used as a guide to determine the appropriate quality classification.

2.3.2 Class A masts

Masts should only be considered to be of quality class A if they conform to all of the following conditions.

a) The structural design should be in accordance with this Part of BS 8100 and BS 8100-3.[1)]

b) The design and detailing should be subject to a satisfactory comprehensive independent appraisal or type test.

c) To ensure that the required standards are achieved and maintained, quality control procedures, agreed with the designer, should be adopted in all stages of the production, fabrication and erection of all structural components.

d) The mast, including all guys and anchorages, should be 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 structure in good condition. The interval between inspections should not exceed 2 years for class A and 5 years for class B. Guy tensions and mast column verticality should be checked at regular intervals not exceeding 5 years. More frequent guy tension checks may be specified during the first 10 years of the life of the structure. Guy tensions should also be checked after major storms or significant loading changes.

NOTE Guidance is provided in Annex A.

2.3.3 Class B masts

Masts should be considered to be of quality class B if they only conform to conditions **2.3.2**a), **2.3.2**c) and **2.3.2** d) or meet the appropriate criteria of Annex A.

2.3.4 Class C masts

Masts should be considered to be of quality class C if they only conform to conditions **2.3.2**a) and **2.3.2**c) or meet the appropriate criteria of Annex A.

[1)] The strength of masts designed in accordance with other relevant British Standards, may be deemed to be in accordance with this Part of BS 8100, provided that the factors of safety given in **2.5** are used in a), and compatible standards are used in a), c) and d).

2.3.5 Appraisal of existing masts

Where an existing mast is to be appraised, and the designer is satisfied by detailed site inspection and from documentary evidence that conditions **2.3.2**a), **2.3.2**b), **2.3.2**c) and **2.3.2**d) are satisfied as appropriate, then class A or B may be considered in accordance with **2.3.2** and **2.3.3**. However, if no site inspection is undertaken or if the site inspection and documentary evidence show that conditions **2.3.2**b) and **2.3.2**d) are not satisfied then class C should be considered appropriate only if documentary evidence exists to show that conditions **2.3.2**a) and **2.3.2**c) were satisfied at the time of construction.

2.3.6 Reappraisal following significant loading changes

The design of the whole structure should be reappraised in the event of any significant increase or decrease in height or loading.

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 **3.2.1** and **3.5**, respectively. An appropriate value should be selected by reference to Figure 1 using the greater value obtained from the most onerous performance requirement, i.e. according to environment, economic consequences or usage.

When spectral analytical procedures are used in accordance with **5.5**, γ_v may be multiplied by 0.97.

2.4.2 Dead load and partial factors

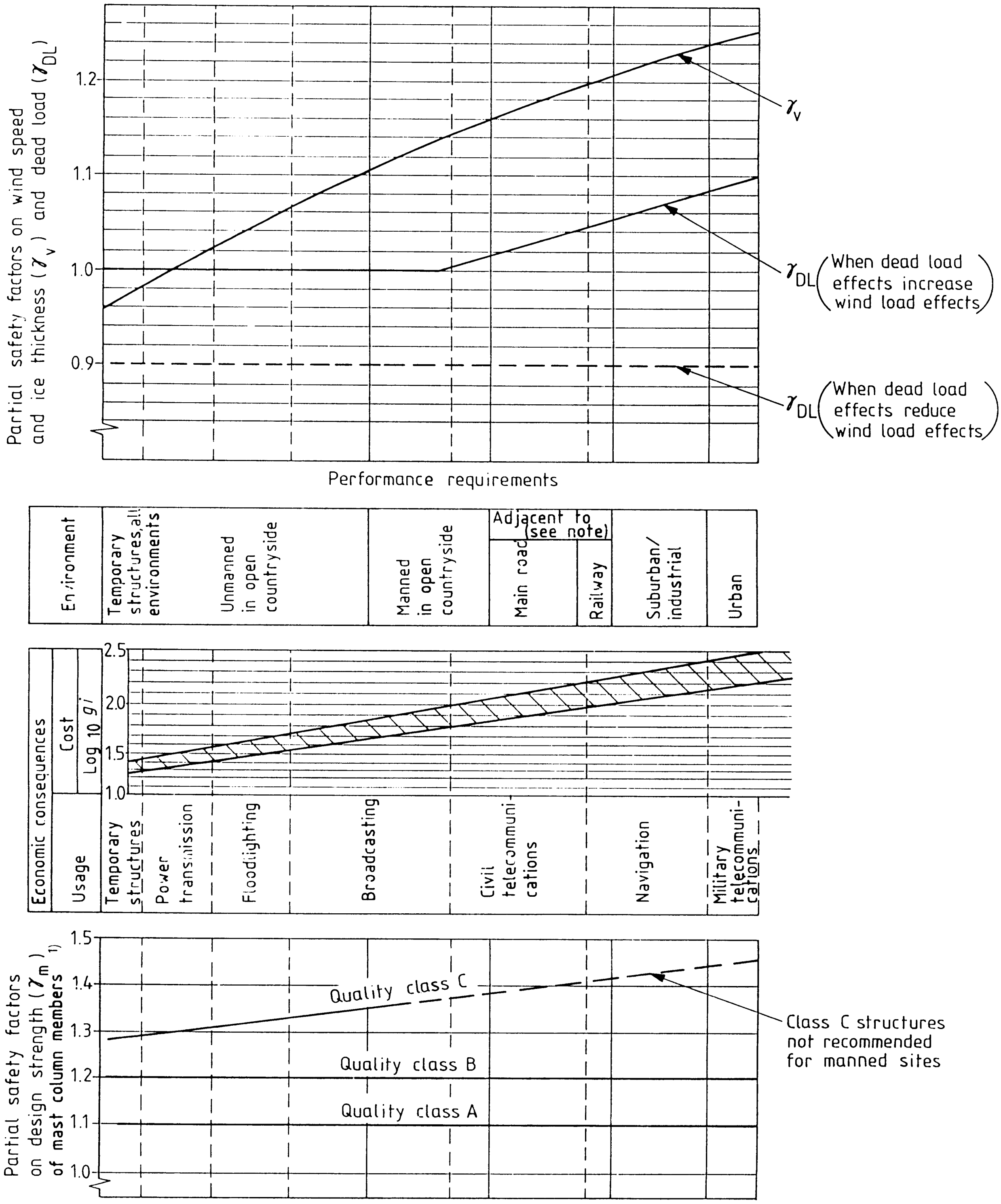
2.4.2.1 *Dead load factors*

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 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 effect, the factors are given by full lines. For conditions where the dead load effects reduce the wind load effects, the lower value given by the dashed line should be used.

2.4.2.2 *Calculation of dead loads*

The self-weight of the mast, including ladders, platforms and guys, should be based on nominal sizes of members, with an appropriate allowance for gussets, bolts, welds, insulators and linkages.

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



NOTE Adjacent to is defined as within a distance from the mast base of less than the height of the mast.

[1)] For partial safety factors on guys the above values of γ_m should be multiplied by 1.8.

Figure 1 — Partial safety factors on wind speed, ice thickness, dead load and design strength

2.5 Safety assessment

In assessing the safety of the design of a mast under extreme conditions, the mast loading derived from Section 3, Section 4 and Section 5 should be used to determine the forces in each component of the mast. 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 partial safety factors γ_m should be selected from Figure 1 appropriate to:

a) the quality class of the mast;

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

The factors of safety applied to mast guys and to certain of their associated components of linkage and tension assemblies are usually related to breaking load. They are normally different from the factors of safety applied to the components of the mast column. This allows for the wider scatter in strength tests, the greater difficulty of inspection and maintenance and the greater susceptibility to fatigue of guy components. The appropriate partial safety factors on characteristic strength for guys are given in Figure 1. Characteristic strengths of components should be determined using the method given in BS 8100-3.

NOTE 1 British Standard components incorporated into the guy assembly are frequently specified in terms of safe working loads for lifting. In such cases it is necessary to derive a characteristic strength which is consistent throughout the guy assembly.

NOTE 2 For guy components which are specified in terms of manufacturer's minimum breaking load, this value may be assumed to be equivalent to the characteristic strength of the component, subject to the approval of the designer.

NOTE 3 Alternative methods for deriving design strengths of components not covered by BS 8100-3 may only be used where the strengths can be shown to be characteristic strengths derived on the same basis as given in BS 8100-3.

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

2.6 Serviceability

When the performance of any system supported by a mast is dependent on the magnitude of movement or deformation of the structure, serviceability requirements should be related to the fraction of the annual operating period during which the performance is predicted to be unsatisfactory. The criteria to be assumed in calculating deflections are given in **5.1.5** and the period during which serviceability limits are exceeded due to excessive movement may then be estimated in accordance with Annex B.

2.7 Information to be provided by specifiers

Specifiers of masts should provide the information to enable the performance requirements to be determined by the designer. The specifier should select environmental conditions (see **2.2.2**), the economic consequences or usage (see **2.2.3**), the quality class (see **2.3**) and serviceability required for the system (see **2.6**).

NOTE Special consideration should be given by the specifier to whether the mast is required to survive under full or partial wind loading with any one guy broken. In such circumstances reduced partial safety factors may be appropriate.

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Section 3. Meteorological parameters

3.1 Wind speed data

3.1.1 General

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

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 structure 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**.

b) *Regions subject to hurricanes or typhoons*. Design wind speeds should be 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.

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 $\bar{V}_b$ is the hourly mean wind speed at 10 m above ground level in flat open country, reduced to mean sea level, with an estimated annual probability of being exceeded of 0.02, irrespective of direction. It should be obtained from Figure 2. The method used to derive the basic winds from the meteorological data is described in Annex C.

3.1.3 Site wind speed

The hourly mean site wind speed $\bar{V}_S$ for any particular direction should be calculated from

$$\bar{V}_S = \bar{V}_b S_a S_d S_s$$

where

$\bar{V}_b$ is the basic wind speed derived as in **3.1.2**;

S_a is an altitude factor given in **3.1.4**;

S_d is a direction factor given in **3.1.5**;

S_s is a seasonal factor given in **3.1.6**.

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 wind speed may be derived from the gradient wind speed $\bar{V}_g$ in accordance with Annex D. This procedure is approximate and should not be used for sites in the United Kingdom where wind map data are available.

3.1.4 Altitude factor

An altitude factor S_a should be used to adjust the basic wind speed $\bar{V}_b$ for the altitude of the site above sea level. For the United Kingdom it should be calculated from

$$S_a = 1 + 0.001\Delta$$

where Δ is the site altitude in metres above mean sea level of:

a) the ground level of the site when topography factor S_h is not used; or

b) the general level of the terrain when topography factor S_h is used.

The effects of local topographic features, such as hills and escarpments, are allowed for separately by the topography factor given in **3.2.4**.

3.1.5 Direction factor

A direction factor S_d may be used to adjust the basic wind speed $\bar{V}_b$ to produce wind speeds with the same risk of being exceeded in any wind direction. Values are given in Table 1 for the wind directions 0° to 330°. If the orientation of the structure is unknown or ignored, the value of the direction factor S_d should be taken as 1.0 for all directions. When the direction factor is used with other factors which have a directional variation, values from Table 1 which correspond to the specific direction under consideration should be used.

For conditions of wind combined with ice (see **3.5**) a value of S_d obtained from Table 1 but not greater than 0.85 may be used.

Table 1 — Values of S_d for the United Kingdom

Wind direction Degrees	S_d
0 N	0.78
30	0.73
60	0.73
90 E	0.74
120	0.73
150	0.80
180 S	0.85
210	0.93
240	1.0
270 W	0.99
300	0.91
330	0.82

NOTE The wind direction is defined in the conventional manner; an east wind is a wind direction of 90° and blows from the east to the site.

3.1.6 Seasonal factor

A seasonal factor S_s may be used to reduce the basic wind speed for structures which are expected to be exposed to the wind for specific sub-annual periods. Values of S_s which give wind speeds with a risk of being exceeded of 0.02 in the stated period are given in Annex E.

3.1.7 Terrain categories

3.1.7.1 *General*

The site wind speed $\bar{V}_S$ derived in **3.1.3** refers to a standard open country exposure at a height of 10 m above ground. To obtain the design wind speed the effects of varying ground roughnesses and the height and density of obstructions upwind of the site and the effects of topography should be taken into account.

3.1.7.2 *Ground roughness*

The following three categories of terrain are considered in this Part of BS 8100.

a) *Sea*. The sea, and inland water extending more than 1 km in the wind direction when closer than 1 km upwind of the site.

b) *Country*. All terrain which is not defined as sea or town.

c) *Town*. Built-up areas with a general level of roof tops at least 5 m above ground level. (Permanent forest and dense woodland may be treated as this category.)

Terrain categories are explained in more detail in Annex F.

3.1.7.3 *Effective height*

For structures in country terrain the effective height z_e should be taken as the actual height z of the section under consideration.

For structures in town terrain some shelter is afforded by the general level of the height H_o of the roof tops of the buildings or of the height of other permanent obstructions upwind of the site, and the upwind spacing X from the mast.

To account for this effect in towns the terrain factors given in **3.2.3** are determined with respect to an effective height z_e for the section of the structure under consideration.

The effective height in towns should be calculated as follows.

a) Where the spacing to the upwind sheltering building or permanent obstruction is less than $X = 2H_o$:

$$z_e = z - 0.8H_o$$

or

$$z_e = 0.4z$$

whichever is the greater.

b) Where the spacing is greater than $X = 6H_o$:

$$z_e = z$$

c) For intermediate values in the range $2H_o < X < 6H_o$:

$$z_e = z - 1.2H_o + 0.2X$$

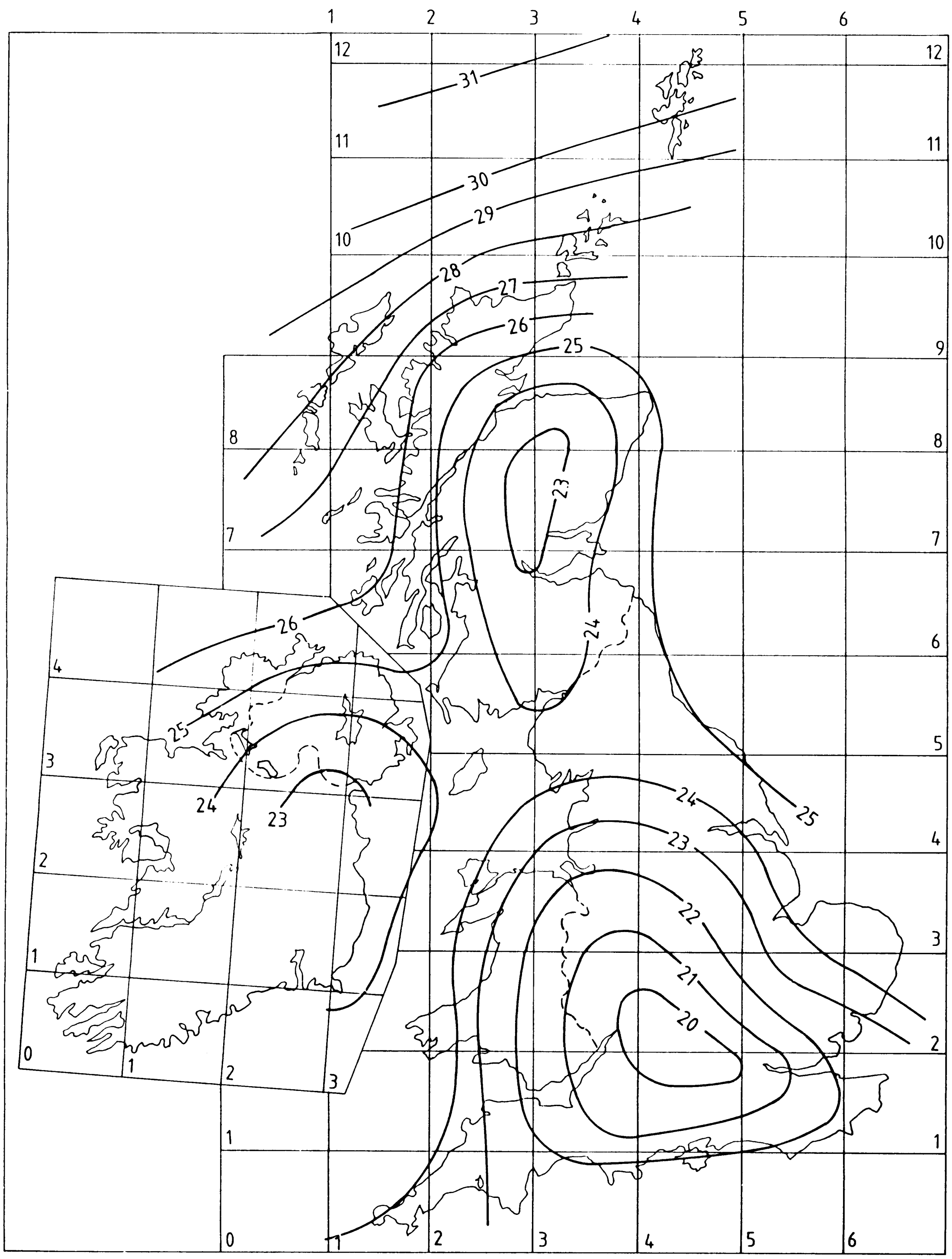
or

$$z_e = 0.4z$$

whichever is the greater.

In the absence of more accurate information, the obstruction height H_o may be taken as 3 m × the typical number of storeys of upwind buildings.

The derivation of these values and further guidance is given in Annex G.



Wind speeds in metres per second.

NOTE 1 Values are at sea level in country terrain.

NOTE 2 $\bar{V}_b$ for the Channel Islands is 24 m/s.

Figure 2 — Basic hourly mean wind speeds $\bar{V}_b$ for the United Kingdom

3.2 Effective wind speed

3.2.1 General

The effective wind speed $\bar{V}_z$ for each wind direction for a structure on a particular site for the effective height z_e at a height z above the ground should be calculated from

$$\bar{V}_z = \bar{V}_S S_o \gamma_v$$

where

$\bar{V}_S$ is the site wind speed for each wind direction (see **3.1.3**);

S_o is the terrain factor appropriate to the wind direction being considered and should be obtained from **3.2.2** or **3.2.3**;

γ_v is the partial safety factor on wind speed determined from Figure 1 appropriate to the type of structure.

The effective wind speeds for each section of the structure should be calculated at the centre of the section being considered.

Values of turbulence intensity are given in **5.3.2.2**.

For serviceability requirements (see **3.3**) the characteristic wind speed $\bar{V}_k$ should be calculated from

$$\bar{V}_k = \bar{V}_b S_a S_o$$

3.2.2 Sites in country terrain: terrain factor S_o

The terrain factor S_o should be used to correct the site wind speed to take account of the effective height z_e of the section being considered, local topography and the terrain upwind of the site.

It should be calculated from

$$S_o = S_c (1 + S_h)$$

where

S_c is the fetch factor determined as described in this subclause;

S_h is the topography factor defined in **3.2.4** and evaluated in accordance with Annex H.

The fetch factor S_c allows for the fetch change from sea to country terrain. Its value should be obtained from Figure 3 appropriate to the effective height z_e of the section under consideration and the distance of the site from the sea in the relevant direction.

3.2.3 Sites in town terrain: terrain factor S_o

The terrain factor S_o should be used to correct the site wind speed to take account of the height of the section being considered above ground level, the local topography and the terrain upwind of the site. It should be calculated from

$$S_o = S_c T_c (1 + S_h)$$

where

S_c is the fetch factor determined as described in this subclause;

T_c is the fetch adjustment factor determined as described in this subclause;

S_h is the topography factor described in **3.2.4** and evaluated in accordance with Annex H.

The fetch factor S_c allows for the fetch change from sea to country terrain. Its value should be obtained from Figure 3 appropriate to the effective height z_e of the section under consideration and the distance of the site from the sea in the relevant direction.

Adjustment factor T_c allows for a fetch change from country to town terrain by modifying the fetch factor S_c. Its value should be obtained from Figure 4 appropriate to the effective height z_e of the section under consideration and the distance of the site from the edge of the town in the relevant direction.

3.2.4 Topography factor

The topography factor S_h should be used to correct the terrain factor to allow for local topographical features such as hills, valleys, cliffs, escarpments or ridges which can significantly affect the wind speed in their vicinity. Near the summits of hills, or the crests of cliffs, escarpments or ridges, the wind is accelerated. In valleys or near the foot of cliffs, steep escarpments or ridges, the wind may be decelerated.

Where the average slope of ground does not exceed 0.05 over a kilometre radius from the site the terrain may be taken as level and the topography factor S_h taken as zero.

Corrections for topography are based on wind speeds at the altitude of the general surroundings. Therefore to ensure that the height correction is not applied twice, where a topography factor is to be applied, the value of the site wind speed should be calculated from **3.1.3** taking the altitude factor to be appropriate to the general height of the terrain for a distance of 8 km upwind of the site. A method of determining the appropriate altitude Δ to be used in **3.1.4** for this purpose is given in Annex J.

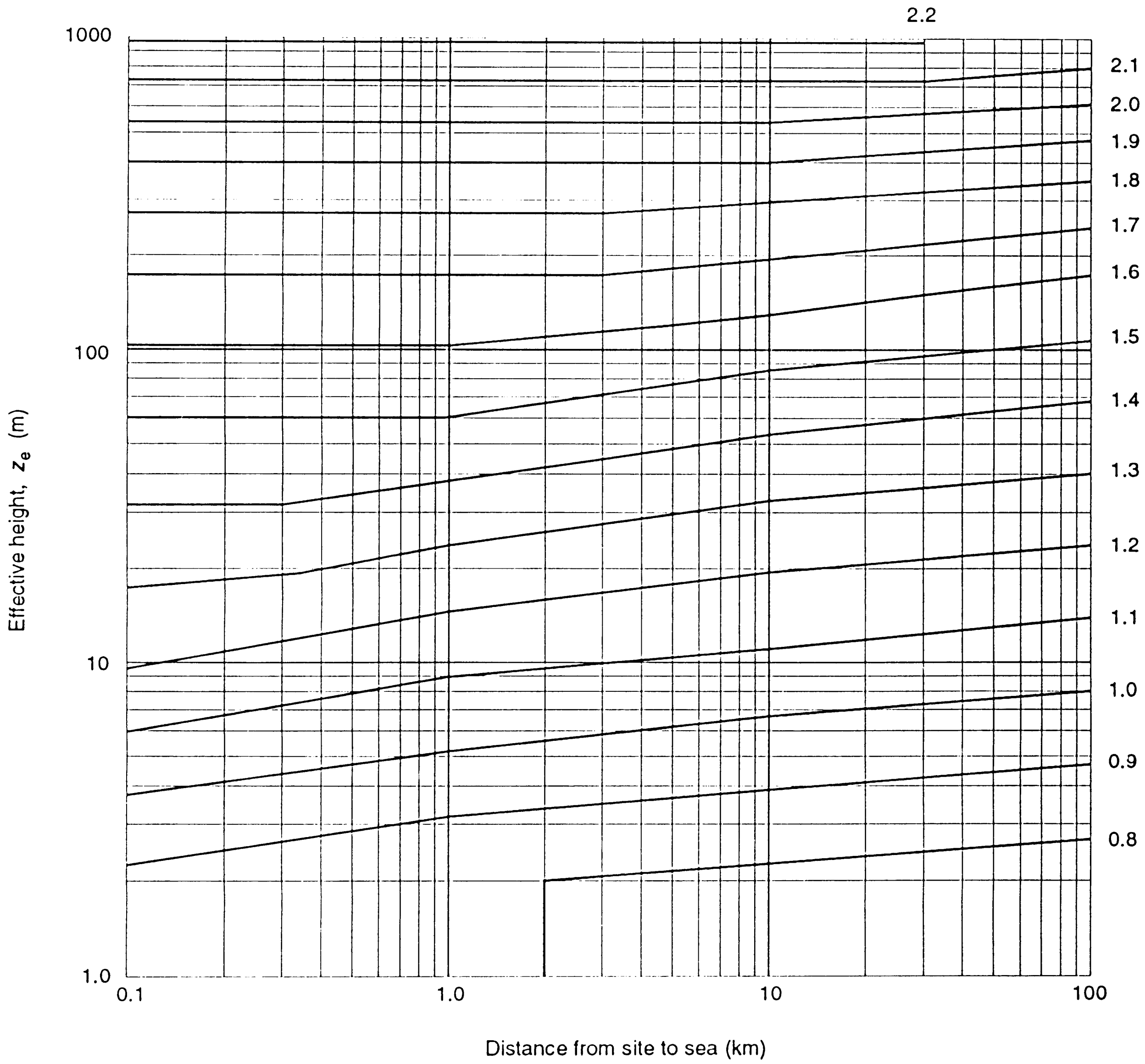
In the vicinity of local topographical features the factor S_h is a function of the upwind slope and the position of the site relative to the summit or crest and will be within the range of $0 < S_h < 0.6$. It should be noted that S_h will vary with the height above ground level from a maximum near to the ground reducing to zero at higher levels.

In certain steep sided valleys wind speeds may be less than in level terrain; however, before any reduction in wind speed is considered specialist advice should be sought.

A method of calculating S_h is given in Annex H.

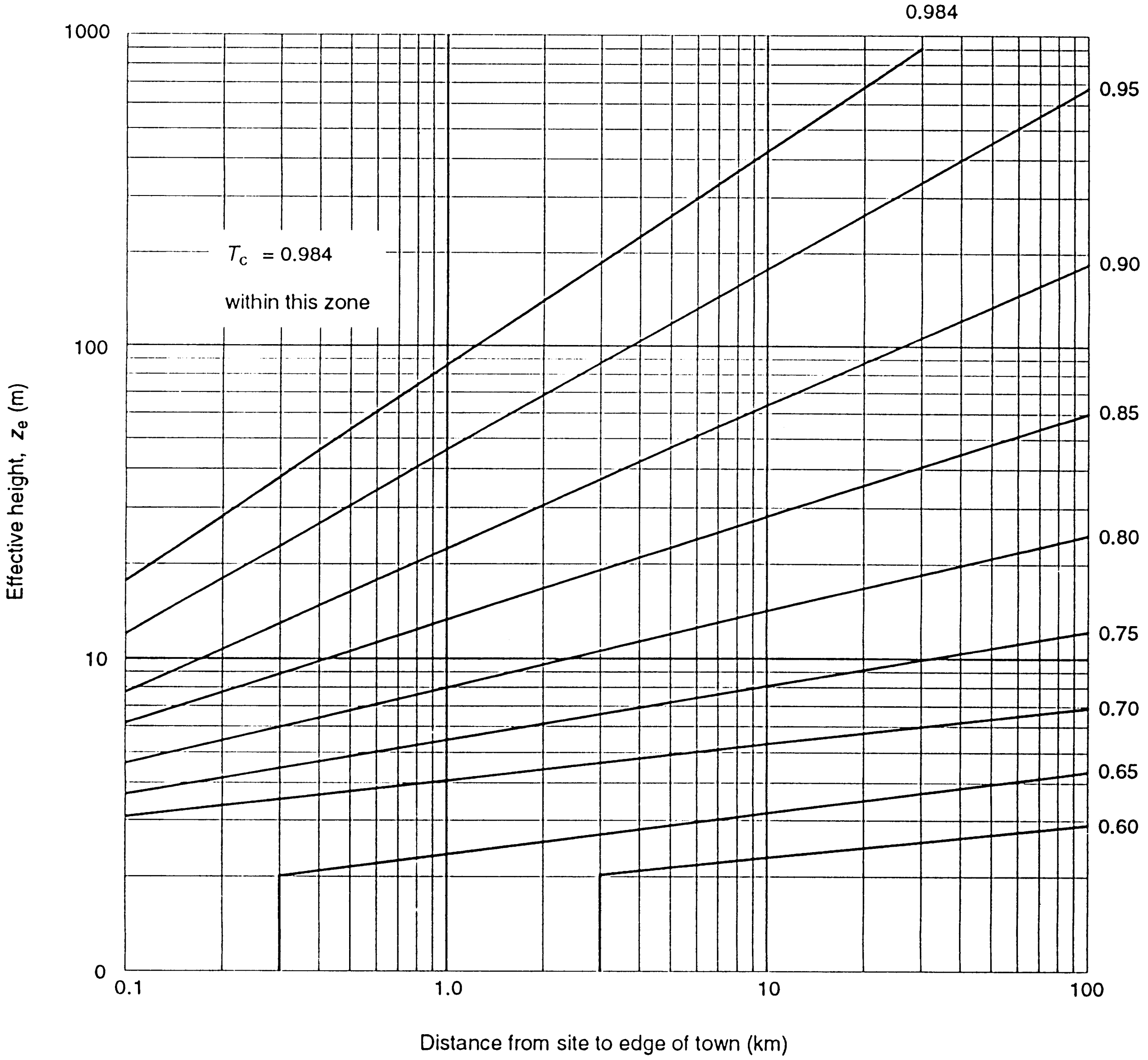
3.3 Serviceability

For the purpose of assessing the duration of unserviceability due to excessive deflection and for an estimation of fatigue life in accordance with **3.4** the procedure set out in Annex B should be adopted for sites in the United Kingdom.



NOTE Interpolation should be used to determine the appropriate value of S_c.

Figure 3 — Fetch factor S_c



NOTE 1 Interpolation should be used to determine the appropriate value of T_c.

NOTE 2 T_c should not exceed 0.984.

Figure 4 — Fetch adjustment factor T_c

3.4 Fatigue life assessment

3.4.1 In-line vibrations

For masts 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 **5.6**) may be assumed to exceed 50 years.

Caution should be exercised in the design of all steel masts containing welded details. For such masts, the fatigue life should be evaluated when either:

a) the service life is greater than 30 years; or

b) steel of yield stress greater than 355 N/mm^2 is used in welded areas.

For masts 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 from Annex B 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 one-quarter of the peak fluctuating stress 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 **5.6** for masts supporting cylindrical aerials or stacks. For each mode considered, the duration of such vibrations should be estimated as described in Annex B. The resulting duration of each mode of vibration, within the design life of the mast, should then be used to assess the accumulated fatigue damage appropriate to the class of detail, in accordance with BS 8100-3.

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 **5.6** and the fatigue assessment undertaken as for overall response.

3.5 Ice loading

3.5.1 General

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

a) *Extreme symmetrical icing in the absence of wind.* The weight of ice should be determined in accordance with **3.5.2**, **3.5.3** and **3.5.4**.

b) *Extreme asymmetrical icing in the absence of wind.* The weight of asymmetric ice on the guys should be determined by applying the appropriate ice thickness from **3.5.2**, **3.5.3** and **3.5.4** to all guys apart from:

1) one lane of the top guy level; and as a separate case

2) two lanes of the top guy level.

In conjunction with each of these, ice loading on the mast column should be considered to be applied to all faces of the mast, determined in accordance with **3.5.2**, **3.5.3** and **3.5.4**.

NOTE Asymmetric ice on the guys can cause critical stresses in the mast column. Clearly there are numerous combinations of iced and un-iced guys which could be considered in design. The conditions specified above have been found to be the most critical in many cases. However, if such conditions are found to dictate the design of any element of the mast, then further combinations of iced and un-iced guys (either in one guy level, or two lanes, or in any combination) should be considered.

c) *Combination of symmetrical icing with wind.* The weight of ice should be determined in accordance with **3.5.2**, **3.5.3** and **3.5.4**. The wind resistance (see **4.6**) should be based on projected areas increased to allow for a uniform coating of ice thickness determined in accordance with **3.5.2** and **3.5.3**.

The site effective wind speed $\bar{V}_i$ to be used should be taken as

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

where $\bar{V}_z$ is the site effective wind speed determined in accordance with **3.2.1** but with due allowance for the limiting value of 0.85 for the wind direction factor S_d in accordance with **3.1.5**, 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 $\bar{V}_z$.

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

d) *Combination of asymmetric icing with wind*. The combinations of ice on the mast column and guys should be taken as for item b) using the appropriate thickness and density of ice determined from **3.5.2**, **3.5.3** and **3.5.4** and wind speed in accordance with item c).

NOTE Local records should be studied to try to ascertain the extremes of asymmetric icing and wind speeds in combination, as the specified values are based on limited data.

3.5.2 Basic ice thickness

3.5.2.1 *General*

The formulae given in **3.5.2.2** and **3.5.2.3** are based on limited data and local records should be studied to ensure that the thicknesses quoted are not likely to be exceeded. Where such records are available statistical analyses should be undertaken as outlined in **3.5.2.2** and **3.5.2.3**.

3.5.2.2 *Ice thickness in the absence of wind*

The basic ice thickness r_b in the absence of wind, for the United Kingdom, should be taken as

$$r_b = K_i\left(r_o + \frac{\Delta - 200}{25}\right)$$

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, cables or guys; or

b) (⅔ + 4/D) but not more than 1.2 for round bars, circular tubes, cables or guys, where D is the diameter of the member (in mm);

Δ is the altitude of the mast top above sea level (in m);

r_o is the radial ice thickness in the absence of wind to be obtained from Figure 5 (in mm), appropriate to the position of the site. Alternatively, where data are available r_o may be derived from a statistical analysis assuming an extremal distribution based on records of the annual maximum thickness of ice formation on components of form and size similar to those to be used in the mast or its attachments at the latitude and altitude of the site and having an annual probability of occurrence of 0.02.

NOTE Ice formation is sensitive to the location of the mast within the topography of the site.

3.5.2.3 *Ice thickness in conjunction with wind*

The basic ice thickness r_b in conjunction with wind, for the United Kingdom, should be taken as

$$r_b = K_i\left(r_w + \frac{\Delta - 200}{25}\right)$$

but not less than $K_i r_w$

where

K_i and Δ are as defined in **3.5.2.1**;

r_w is the radial ice thickness in conjunction with wind to be obtained from Figure 5 (in mm), appropriate to the position of the site. Alternatively, r_w may be derived from records (see **3.5.2.1**) but should have 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

$$r_r = \gamma_v K_G r_b$$

where

γ_v is the partial safety factor to be obtained from Figure 1, appropriate to the type of mast (see **2.2**);

r_b is the basic radial ice thickness determined in accordance with **3.5.2**;

K_G is a guy or cable factor which should be taken as either:

a) 1.0 for all mast members, guys and ancillaries; or

b) $(N_G + 0.3)/1.3N_G$ for any multiple guys or cables supported by the mast in parallel formation;

N_G is the number of guys or cables.

3.5.4 Ice weight

The weight of ice deposited on the mast should be calculated assuming that all structural sections and ancillary parts are uniformly coated in ice of thickness r_r and that the following apply.

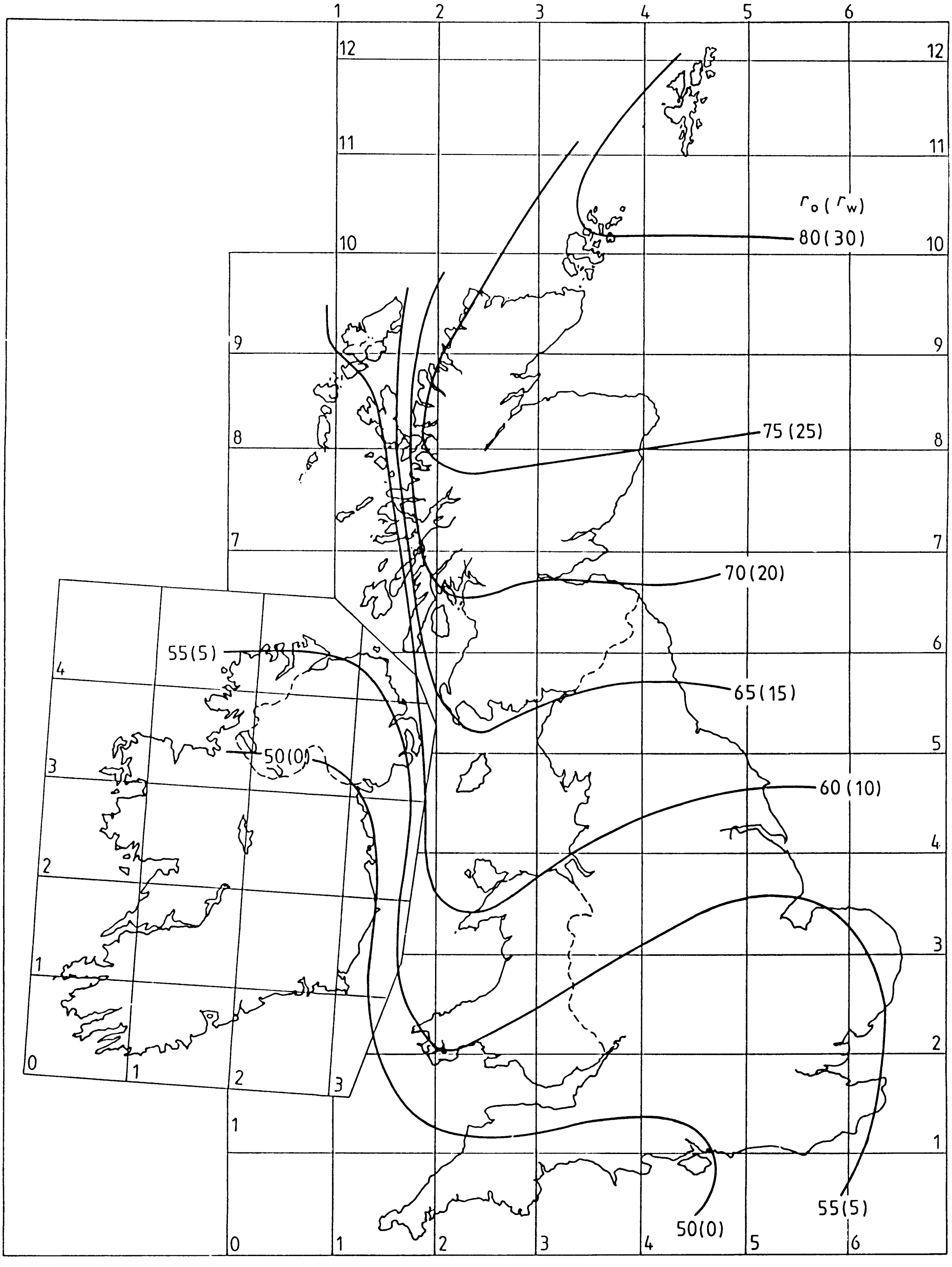
a) The unit weight of ice in the absence of wind should be taken as 5 kN/m^3.

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

1) 9 kN/m^3 for design against compression or where its application to guys increases their tension;

2) 5 kN/m^3 for design against tension.

c) The weight of ice deposited should include an allowance for gaps of less than 75 mm to be completely filled with ice in accordance with **4.6**.



All thicknesses are in millimetres.

Figure 5 — Ice thickness r_o and r_w for the United Kingdom

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 mast 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.

NOTE Requirements for wind tunnel tests are outlined in **C.6** of BS 8100-2:1986.

For the purposes of calculating wind resistance, a mast should be divided into a series of sections where a section comprises several identical or nearly identical panels (see Figure 6). Projections of bracing members from faces parallel to the wind direction, and plan bracing, should be ignored in determinations of the projected area of the structure. A mast should generally be divided into a sufficient number of sections to enable the wind loading to be adequately represented for analysis.

When the wind resistance under iced conditions is calculated, the projected areas of structural elements and ancillaries should take due account of the thickness of ice as appropriate.

4.1.2 Constraints on the method

The calculation of total wind resistance may be determined in accordance with **4.1.3** for square and triangular masts. For masts of other shapes the method given in Annex K should be used.

Some advantage may be obtained by using the general method given in Annex K for panels in masts containing ancillary items which do not conform to the following constraints:

a) the total projected area of those ancillary parts adjacent to the face under consideration is less than the projected area of the structural members in that face (see Figure 6);

b) the total projected area normal to any face on the mast 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 face width of the mast at that level.

NOTE Where the projected areas of ancillaries on each face 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 (pressure) coefficients may be calculated in accordance with **4.2**.

4.1.3 Calculation of total wind resistance

The total wind resistance ΣR_w in the direction of the wind over a section of a mast should be taken as

$$\Sigma R_w = R_M + R_{AW}$$

where

R_M is the resistance of the bare mast section, 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 **4.3** and **4.4**, as appropriate.

4.2 Wind resistance of mast structure

4.2.1 General

For a lattice mast of square or equilateral triangular cross section, with equal areas for each face, the total wind resistance R_M in the direction of the wind over a section height of the structural components may be taken as

$$R_M = K_\theta C_N A_s$$

where

C_N is the overall normal 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 in the face within one section height of the mast at the level concerned (see Figure 6) including icing when appropriate;

K_θ is the wind incident factor given in Figure 7 for commonly used values of θ. For other values of θ reference should be made to the formulae in **L.4.1**, using $A_F = A_s$;

θ is the angle of incidence of the wind to the normal face 1, in plan; face 1 should be taken as the windward face (see Figure 6).

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

The value of R_e is given by

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

where

$1.5\bar{V}_z$ is the assumed gust wind speed relevant to the effective height z_e to the centre of the member, with $\bar{V}_z$ 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 United Kingdom.

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

4.2.2 Overall normal drag (pressure) coefficients

Values of overall normal drag (pressure) coefficients C_N that are applicable to the structural framework of square or equilateral triangular section masts may be derived from Figure 8, in which the solidity ratio φ is as defined for Figure 7. For such masts 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 overall normal drag (pressure) coefficients for masts composed of flat-sided, subcritical circular-section and supercritical circular-section members respectively, to be obtained from Figure 8;

A_f is as defined on Figure 7;

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

$A_F = A_f + A_c + Ac'$

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

NOTE Circular-section members in supercritical regimes may conservatively be assumed to be in subcritical regimes for drag calculations.

4.3 Linear ancillaries

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 (pressure) coefficient appropriate to the item and its effective Reynolds number, values of which are given in Table 2 for common isolated individual members; C_N may be determined in accordance with Annex K for parts composed of single frames;

K_A is a reduction factor to take account of the shielding of the component by the mast itself; K_A is given in Table 3, except for circular sections in supercritical flow and for ancillaries not conforming to the constraints of **4.1.2**, in which case

$K_A = 1.0$

A_A is the area of the part visible when viewed in the wind direction including icing when appropriate; for cylinders with 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.

4.4 Discrete ancillaries

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 (pressure) coefficient for the item appropriate to the wind direction and wind speed and should be obtained from wind tunnel tests;

A_A is the reference area of the item as defined in the wind tunnel test and compatible with the value of C_A;

K_A is as defined in **4.3**.

The corresponding crosswind resistance R_{AX} and lift resistance R_{AZ} should be calculated as for R_{AW} taking the reference direction as normal in plan to the mean wind direction, and C_A as the appropriate coefficient for crosswind and lift.

The corresponding torque resistance T_{AW} should be calculated using the appropriate coefficient, obtained from wind tunnel tests in association with the relevant moment arm for such torsion.

4.5 Guys

The wind resistance R_G normal to the guys in the plane containing the guy and the wind may be taken as

$$R_G = C_{NG} D_G l_G \sin^2 \Psi$$

where

C_{NG} is the overall normal drag (pressure) coefficient appropriate to the effective Reynolds number, the values of which are given in Table 2 for both ice-free and iced conditions;

l_G is the chord length of the guy;

D_G is the diameter of the guy with or without ice as appropriate;

Ψ is the angle of wind incidence to the chord.

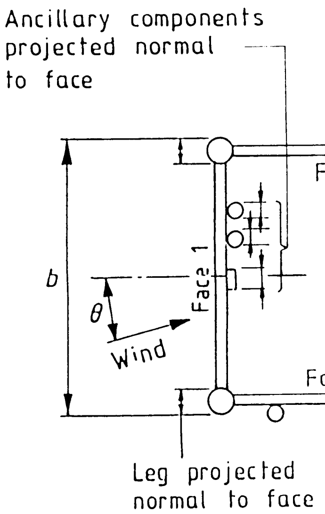
NOTE The wind resistance for any cables attached to the mast should be derived using this formula.

4.6 Icing

In calculations of the wind resistance of a mast and ancillaries under iced conditions, each element of the mast column, ancillary parts and guys should be considered to be coated on all sides, with a thickness of ice equal to that given in **3.5**.

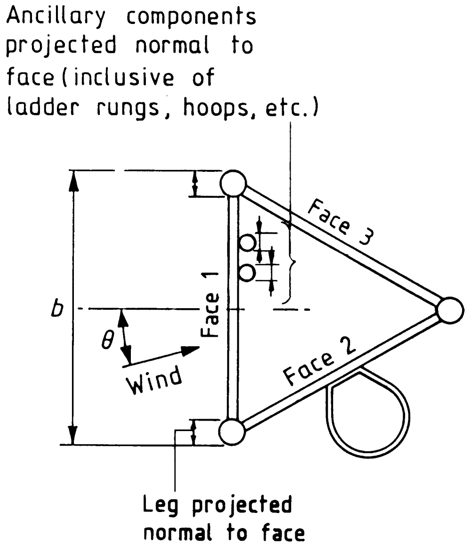
Where the gap between components when not iced is less than 75 mm, it should be considered to be completely filled by ice under icing conditions.

For drag (pressure) coefficients of individual members, see Table 2.



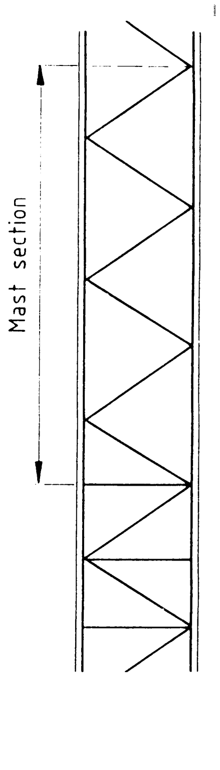
NOTE Face 1 should be taken as the windward face such that $-45° \leq \theta \leq 45°$

a) Plan on square structure

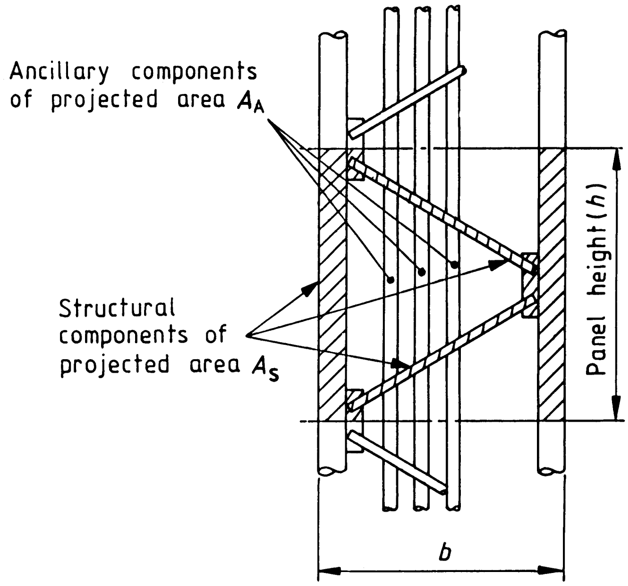


NOTE Face 1 should be taken as the windward face such that $-60° \leq \theta \leq 60°$

b) Plan on triangular structure



c) Mast section



For **4.2** and **4.3**: solidity ratio

$$\varphi = \frac{A_s}{hb}$$

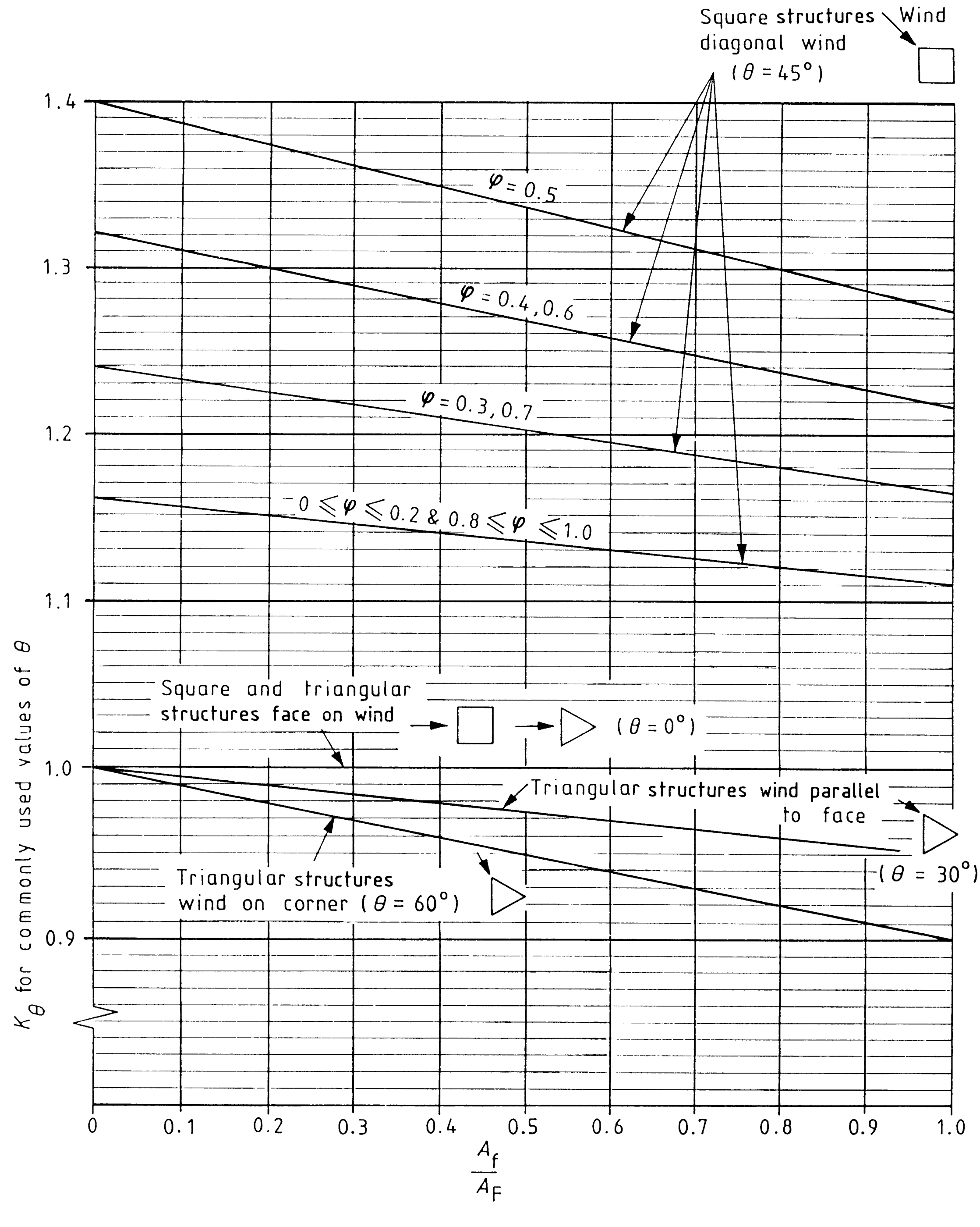
For the method in Annex P: solidity ratio

$$\varphi = \frac{A_s + A_A}{hb}$$

NOTE Structural components of the front face (shown hatched) have projected area A_s which is equal to A_F in **4.2**

d) Structure panel

Figure 6 — Projected panel area used to calculate the solidity ratio φ



Key

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

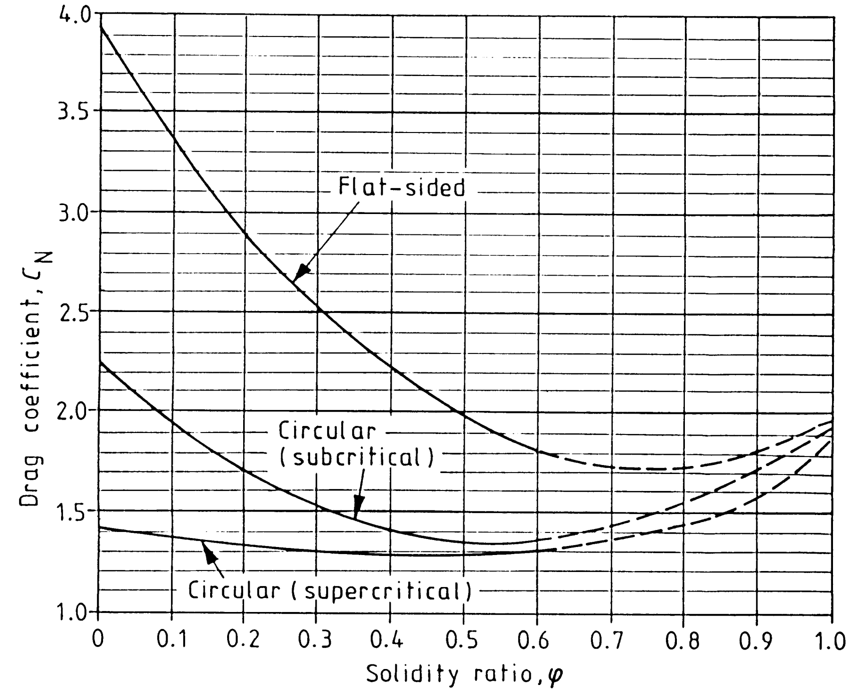
A_F is the total projected area normal to the face;

θ is the angle of incidence of the wind to the normal face 1, in plan;

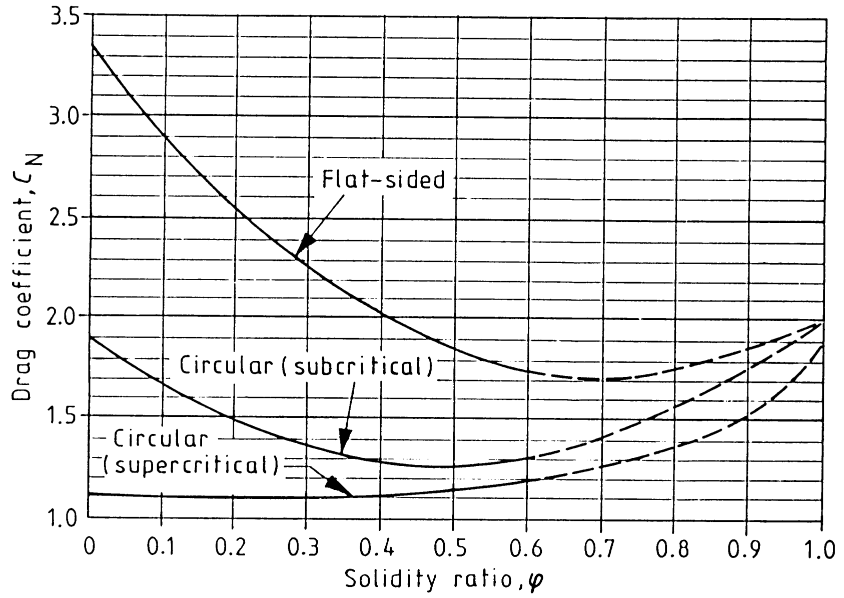
φ is the solidity ratio i.e. the ratio of the total projected area within a section height of the structural components in the windward face (A_S) visible when viewed normal to the face, to the area enclosed over the section height by the boundaries of the frame projected normal to the face, both at the level considered (see Figure 6).

NOTE For the basis of the curves, see Annex L.

Figure 7 — Wind incidence factor K_θ



a) Square structures



b) Triangular structures

NOTE 1 For the basis of the curves see Annex L.

NOTE 2 For structures with $\varphi > 0.6$ consideration should be given to the possibility of crosswind response due to vortex excitation (see **5.6**).

Figure 8 — Overall normal drag (pressure) coefficients C_N for square and triangular structures

Table 2 — Typical normal drag (pressure) coefficients for individual components

Member type	Effective Reynolds number $R_e = 1.5 \frac{\bar{V}_z D}{v}$	Normal drag (pressure) coefficient C_N	
		Ice free	**Iced**
a) Flat-sided sections and plates	All values	2.0	2.0
b) Circular sections and smooth wire	$\leq 2 \times 10^5$	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. steel core aluminium round conductor, locked coil ropes, spiral steel strand with more than seven wires	*Ice free:*		
	$\leq 6 \times 10^4$	1.2	
	$\geq 10^5$	0.9	
	Iced:		
	$\leq 1 \times 10^5$		1.25
	$\geq 2 \times 10^5$		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:*		
	$\leq 4 \times 10^4$	1.3	
	$> 4 \times 10^4$	1.1	
	Iced:		
	$\leq 1 \times 10^5$		1.25
	$\geq 2 \times 10^5$		1.0
e) Cylinders with helical strakes of height up to $0.12D$	All values	1.2	1.2

Key

D is the member diameter (in m);

$\bar{V}_z$ is the factored wind speed relevant to the effective height z_e to the centre of the member (see **3.2.1**) (in m/s);

v is the kinematic viscosity of air (see **4.2.1**) (in m²/s).

NOTE For intermediate values of R_e, C_N should be obtained by linear interpolation.

Table 3 — Reduction factor K_A for additional ancillary items

Position of additional ancillaries	Reduction factor K_A	
	Square and rectangular sections	**Triangular sections**
Internal to the section	0.6	0.5
External to the section	0.7	0.6

blank

Section 5. Structural response to wind

5.1 Procedure

5.1.1 General

The maximum forces to be used in the design of mast 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 hourly mean value, acting only in the wind direction, and fluctuating loading both downwind and, where relevant, crosswind due to gustiness.

Generally, static analysis procedures can be used to determine the maximum forces in the members of a mast. Only if the criteria set down in **5.1.2** are not satisfied is it necessary to undertake dynamic response methods. However, the design of major masts whose economic consequences of failure or potential hazards resulting from failure are high (see **2.2.1**) such that γ_v is greater than 1.22, should be checked by dynamic response procedures in any event.

Two static analytical methods are provided; the simple gust response factor method described in **5.2** can only be used for masts less than 150 m in height. The more complex patch loading method described in **5.3** can be used for all heights of masts. Both methods require the criteria of **5.1.2** to be satisfied.

5.1.2 Criteria for use of static methods

The following criteria should be satisfied for the static analytical procedures to be used.

a) The cantilever, if present, has a total length above the top guy level of less than half the spacing between the penultimate and top guys.

b) The parameter β is less than 1:

$$\beta = \frac{4E_m I_m / h_s^2}{(1/N_l) \sum_{i=1}^{N_l} K_{Gi} H_{Gi}}$$

where

N_l is the number of guy levels;

K_{Gi} $= 0.5 N_i A_{Gi} E_{Gi} \cos^2 \alpha_{Gi} / l_{Gi}$

A_{Gi} is the cross-sectional area of the guy at level i;

E_{Gi} is the elastic axial modulus for the guy at level i;

l_{Gi} is the length of guy at level i;

N_i is the number of guys attached at level i;

H_{Gi} is the height above the mast base of the ith guy level;

α_{Gi} is the slope of the guy chord at level i to the horizontal;

E_m is the elastic modulus for the mast;

I_m is the average mast bending inertia;

h_s is the average span between guy levels.

c) The parameter Q is less than 1:

$$Q = \frac{1}{30} \sqrt[3]{\frac{HV_H}{b_{av}}} \sqrt{\frac{m_{av}}{H \Sigma R_W}}$$

where

m_{av} is the average mass per unit height of the mast column including ancillaries (in kg/m);

b_{av} is the average face width of the mast (in m);

$\bar{V}_H$ is the hourly mean wind speed $\bar{V}_z$ at the top of the mast (in m/s);

ΣR_W is the average total wind resistance obtained from **4.1.3** (in m²/m);

H is the height of the mast, including cantilever if present (in m).

If any of these constraints are not satisfied, then the spectral analytical method (see **5.1.4**) should be followed.

5.1.3 Equivalent static methods

For masts of symmetrical structural cross section with triangulated bracing, either without ancillaries or with ancillaries symmetric in the wind direction being considered, and which conform to the constraints of **5.1.2**, the maximum forces should be derived in accordance with **5.2** for masts up to 150 m height or **5.3** for masts of any height.

For masts of any height containing ancillaries which are unsymmetric in the wind direction being considered, and which conform to the constraints of **5.1.2**, the maximum forces should be determined in accordance with **5.4**.

5.1.4 Spectral analytical method

For masts not conforming to the constraints of **5.1.2**, maximum member forces should be determined by spectral analytical methods, adopting the basis set out in **5.5** and the parameters given in Annex M.

5.1.5 Deflections

Deflections will normally only be required to satisfy serviceability requirements. The criteria for serviceability should be defined. The following two categories may be considered:

a) masts for which, during permitted periods of unserviceability, the specified limiting deflections may be only occasionally exceeded;

b) masts for which, during periods of unserviceability, the specified limiting deflections are governed by mean deflections.

5.1.6 Vortex-excited vibrations

When masts support large bluff bodies or may become heavily blocked by icing, consideration should be given to their susceptibility to vortex-excited vibrations, in accordance with **5.6**.

5.1.7 Guy vibrations

The mast guys can be subject to high frequency vortex-excited vibrations and to guy galloping, particularly when the guys are iced.

a) *Vortex excitation*. Guys may be subject to low amplitude resonant type vibrations at low wind speeds caused by vortex excitation at high frequency.

As excitation can occur in high modes general rules cannot be set down. However, as a guide, experience shows that such vibrations are likely to occur if the still-air tensions in the guys are in excess of 10 % of their breaking load.

b) *Galloping*. Guys may be subject to galloping excitation when coated with ice or thick grease. The accretion of ice or grease can form aerodynamic shapes which provide lift and drag instabilities. These result in low frequency, high amplitude vibrations. Similar vibrations of smooth plastics coated guys are also known to occur under conditions of rain.

Again general rules cannot be provided as the occurrence of galloping is critically dependent on the formation of ice, or profile of grease. It will generally only occur on large diameter guys and is relatively insensitive to initial guy tensions.

If guy vibrations are observed, dampers or spoilers may be required to limit the resulting stresses.

Fatigue checks of the anchorages should be made if such vibrations are known to have occurred and no remedial action has been taken. In such cases specialist advice should be sought.

5.2 Static gust response procedure for masts up to 150 m high

5.2.1 General

The mast should be analysed under the action of the hourly mean wind speed to determine the mean load effects in all the components of the structure, in accordance with **5.2.2**.

For these purposes the load effects should be taken as changes in axial loads, bending moments, shears and torsional moments in the mast column and tensions in the mast guys, from the still-air condition.

To account for the fluctuating forces due to the gustiness of the wind, the maximum load effects due to the hourly mean wind speed within the span under consideration are multiplied by a gust response factor G_M or G_G, appropriate to the element under consideration and its position in the structure, determined in accordance with **5.2.3**.

5.2.2 Mean load effects

The wind load $\bar{P}_{MW}$ in the direction of the wind on the mast column due to the hourly mean wind speed should be taken as

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

where

ρ_a is the density of the air at the reference temperature and pressure (ρ_a = 1.22 kg/m^3 for the United Kingdom when determining $\bar{P}_{MW}$ in newtons and with $\bar{V}_z$ in metres per second);

$\bar{V}_z$ is the hourly mean wind speed at the level of the centre of area of the section at a height z m 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 mast section concerned, determined in accordance with **4.1.3** or Annex K as appropriate.

The loads should be taken as acting at the level of the centre of areas of faces (including ancillaries if present) within the section height.

The wind load $\bar{P}_{GW}$ on the guys normal to the guys in the plane containing the guy and the wind, due to the hourly mean wind speed, should be taken as

$$\bar{P}_{GW} = \frac{\rho_a}{2} \bar{V}_z^2 R_G$$

where

R_G is the wind resistance of the guy under consideration determined in accordance with **4.5**;

$\bar{V}_z$ is the hourly mean wind speed at each level into which the guy is subdivided for analysis; if a uniform loading is used then $\bar{V}_z$ should be taken as the wind speed at a height of two-thirds of the height of the relevant guy attachment to the mast.

The load effects $\bar{F}_M$ and $\bar{F}_G$ due to the hourly mean wind should then be determined for each component of the mast by a non-linear static analysis under the hourly mean loading $\bar{P}_{MW}$ and $\bar{P}_{GW}$.

5.2.3 Fluctuating load effects

[A1) The fluctuating load effect F'_M in any component of the mast column should be determined from

$$F'_M = \bar{F}_{M\max} G_M \left(\frac{1}{1 + S_h}\right)$$ (A1]

where

$\bar{F}_{M\max}$ is the maximum hourly mean wind load effect in the mast column within the span between adjacent guy levels under consideration (or in the span from the ground to the first guy level or in the cantilever if present);

S_h is the topography factor, where relevant, determined in accordance with **3.2.4**;

G_M is a gust response factor determined in accordance with **5.2.4**.

Where guy eccentricities modify bending moments at guy levels, the value $\bar{M}_{max}$ for determination of the fluctuating moment applicable for any specific span should be taken as the greatest of the values $\bar{M}$ at the adjacent guy levels (or the midspan value if greater) (see Figure 9).

For spans from the ground to the first guy level which are pinned at ground level, the fluctuating bending moment should be taken to vary linearly from $\bar{M}_M$ at midspan to zero at the ground. Similarly, for top spans without cantilevers the fluctuating bending moment should be taken to vary linearly from $\overline{M}_M$ at midspan to the hourly mean bending moment immediately under the top guy level at the mast top (see Figure 9).

The fluctuating tension in any guy should be determined from

$$F'_G = \overline{F}_G \sqrt{G_G^2 + G_M^2} \left(\frac{1}{1 + S_h}\right)$$

where

- $\overline{F}_G$ is the mean tension in the guy determined in accordance with **5.2.2**;
- G_G is a gust response factor to be determined in accordance with **5.2.5**;
- G_M is a gust response factor determined in accordance with **5.2.4**.

5.2.4 Gust response factor for mast column

The gust response factor G_M used to determine the fluctuating load effects in components of the mast column may be obtained directly from Figure 10 as appropriate to the terrain of the mast site as used in **3.2.1**, and taking h_e as:

a) the length of the cantilever for components in cantilevers projecting above the top guy level;

b) the relevant distance between adjacent guy levels (or the distance from ground to the first guy level) for components within the mast column below the top guy level.

For components at guy levels the maximum fluctuating load effect from either the section immediately above or below the guy level under consideration should be used.

5.2.5 Gust response factors for mast guys

The gust response factor G_G used to determine the fluctuating axial tensions in each mast guy should be taken as

$$G_G = K_L K_z$$

where

- K_L is a length factor given in Figure 11;
- L is the total projected length of the guy (see Figure 12);
- K_z is a guy or cable height factor to be obtained from Figure 13 as appropriate to the terrain of the mast site as used in **3.2.1**, where z_G is the mean height of the guy above mast base level.

5.2.6 Total load effects

The total load effects ΣF_M for each component of the mast column should be determined from

$$\Sigma F_M = F_o + \overline{F}_M \pm F'_M$$

where

- F_o is the still-air load effect;
- $\overline{F}_M$ is the hourly mean load effect determined from **5.2.2**;
- F'_M is the fluctuating load effect determined from **5.2.3**, using the ± sign to produce the most severe effect.

The axial tension ΣF_G for each guy should be determined from

$$\Sigma F_G = F_o + \overline{F}_G \pm F'_G$$

where

- F_o is the still-air tension;
- $\overline{F}_G$ is the hourly mean tension determined from **5.2.2**;
- F'_G is the fluctuating tension determined from **5.2.3** using the ± sign to produce the most severe effect.

The procedure for determining each of the total load effects is shown in Table 4.

5.2.7 Loading for calculating deflections

For category **5.1.5**a) the deflections should be derived from analysis under the hourly mean wind load and multiplying the resulting deflections by [A1⟩ $\left(1+\frac{G_M}{1+S_h}\right)$ ⟨A1] where G_M is the gust response factor appropriate to the total height of the mast (determined in accordance with **5.2.4**) for deriving the fluctuating component.

For category **5.1.5**b) the fluctuating components of wind loading may be ignored and deflections under mean wind loading only need be considered.

5.3 Static gust response procedure: general

5.3.1 General

To allow for the dynamic response of masts to wind loading, the mast should be analysed for a series of static patch loading patterns based on the hourly mean loading augmented by wind load patches. This procedure requires several static wind analyses for each wind direction considered, the results being combined to provide the maximum response.

5.3.2 Load cases to be considered

5.3.2.1 *Hourly mean loading*

Loading due to the hourly mean wind speed should be derived as described in **5.2.2**.

5.3.2.2 *Patch loads*

In addition to the hourly mean loading derived in accordance with **5.3.2.1**, successive patch loads should be applied as follows:

a) on each span of the mast column between adjacent guy levels (and on the span between the mast base and the first guy level);

b) over the cantilever if relevant;

c) from midpoint to midpoint of adjacent spans;

d) from the base to the mid-height of the first guy level;

e) from the mid-height of the span between the penultimate and top guy to the top guy if no cantilever is present, but including the cantilever if relevant.

These are shown in Figure 14. The patch load is given by

[A1⟩ $$P_{MW} = \rho_a i_o \bar{V}_z \bar{V}_{10} \Sigma R_w \left[\frac{1}{1+S_{h=10}}\right]$$ ⟨A1]

where

ρ_a, $\bar{V}_z$ and ΣR_w	are as defined in **5.2.2**;
[A1⟩ $S_{h=10}$	is the value of the topography factor, S_h, where relevant, determined from **3.2.4** for h = 10 m ⟨A1];
$\bar{V}_{10}$	is the hourly mean wind speed $\bar{V}_z$ at an effective height z_e = 10 m, determined in accordance with **3.2**;
i_0	is the turbulence intensity and is as given in Figure 15 according to the site terrain.

These patch loads should be applied to the mast in its equilibrium position, under hourly mean wind load determined in accordance with **5.3.2.1**.

5.3.2.3 ***Loading on guys***

For all patch loading cases the total wind load P_G normal to each guy in the plane containing the guy and the wind should be taken as

[A1] $$P_G = \frac{\rho_a}{2} \bar{V}_z^2 R_G \frac{\left[(1 + K_c K_T K_L)^{\frac{1}{2}} + S_h\right]^2}{(1 + S_h)^2}$$ [A1]

where

ρ_a, $\bar{V}_z$ and R_G are as defined in **5.2.2**;

K_c is as given in Figure 16, taking z_e as the effective height of the guy level under consideration;

[A1] K_L is a length factor given in Figure 11; [A1]

K_T is taken as 1.0 for sites in country terrain and is obtained from Figure 17 for sites in town terrain depending on the value of the effective height z_e.

[A1] NOTE 1 P_G represents the total wind load on the guy, including the mean and gust terms. As such this loading should be applied in conjunction with the mean loading on the mast and for each appropriate mean and patch load on the mast column. The fluctuating tension in the mast guys (see **5.3.2.4**) is determined as a result of the load effects in the guys arising from the patch loads on the mast column.

NOTE 2 The application of loading on guys is shown diagrammatically in Figure 18. [A1]

5.3.2.4 ***Derivation of load effect (response) under patch loads***

The load effect r_{PLj} in each element of the mast column and guys derived from each patch load applied successively should be calculated.

NOTE This will require calculation of the difference between the load effect from the patch load combined with the hourly mean load and the load effect of the hourly mean load alone.

These load effects should then be combined as the root sum of squares:

$$r_{PL} = \sqrt{\sum_{j=1}^{N} r_{PLj}^2}$$

where

r_{PLj} is the load effect (response) from the jth load pattern;

N is the total number of load patterns required;

r_{PL} is the total effective load effect of the patch loads.

The equivalent fluctuating load effect of components in the mast column for design purposes should then be determined from

$$F'_M = \lambda r_{PL}$$

where λ may be taken as follows:

— for each load effect below the top guy level, either λ may be taken as 3.78 or a more accurate value of λ for each load effect (except for cantilevers) may be calculated in accordance with Annex N;

— for each load effect in cantilevers projecting above the guy level, λ should be taken as 3.78.

The fluctuating tension in the mast guys should be determined from

$$F'_G = 3.78 r_{PL}$$

5.3.2.5 *Total load effects*

The total load effects ΣF_M for each component of the mast column should be determined from

$$\Sigma F_M = F_o + \bar{F}_M \pm F'_M$$

where

F_o is the still-air load effect;

$\bar{F}_M$ is the hourly mean load effect determined from **5.2.2**;

F'_M is the fluctuating load effect determined in accordance with **5.3.2.4** using the ± sign to produce the most severe effect.

The axial tension ΣF_G for each guy should be determined from

$$\Sigma F_G = F_o + \bar{F}_G \pm F'_G$$

where

F_o is the still-air tension;

$\bar{F}_G$ is the hourly mean tension determined in accordance with **5.2.2**;

F'_G is the fluctuating tension determined in accordance with **5.3.2.4** using the ± sign to produce the most severe effect.

5.3.3 Loading for calculating deflections

For category **5.1.5**a) the deflections should be derived from analysis under the sum of the mean hourly wind load and the envelope of the patch loading analyses for deriving the fluctuating component.

For category **5.1.5**b) the fluctuating components of wind loading may be ignored and deflections under mean wind loading only need be considered.

5.4 Wind loading for unsymmetrical masts or masts with complex attachments

For unsymmetrical masts or masts which contain unsymmetrically placed large ancillaries and/or cables imposing torsional and crosswind loads, 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 procedure set out in Annex P should be used.

5.5 Spectral analytical method

When spectral analytical methods are used to calculate response, they should be used for the resonance contribution to the response only. The background response may be determined using the general static procedure (see **5.3**) but using λ derived in accordance with Annex N with $\lambda_R = 1.0$.

The meteorological conditions to be assumed should be those described in Section 3, and the wind resistance taken as that given in Section 4. In addition, the parameters given in Annex M 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.

Further information on the analysis of the dynamic response of structures to the wind is given in references [1] to [5].

5.6 Crosswind response due to vortex excitation

5.6.1 Critical wind speed

A mast 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 V_{cr} at which such vibrations occur should be derived from

$$(V_{cr}) = \frac{n_r D_z}{S}$$

where

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

S is the Strouhal number, to be taken as 0.2 for circular cylinders and as 0.15 for sharp-edged bodies, in the absence of specific data from wind tunnel tests for the section under consideration;

n_r is the natural frequency in mode r of the mast in crosswind vibration.

If transverse oscillation is to be avoided at all times, the value of $(V_{cr})_1$ for the lowest mode should exceed $1.2\,\bar{V}_z$ at all levels z, where $\bar{V}_z$ is determined at an effective height z_e in accordance with **3.2**.

Alternatively, when occasional oscillations are permissible, the periods of unserviceability may be assessed by reference to **B.2**.

5.6.2 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

— 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:

$0.9\rho_a \bar{V}_z^2 k_e C_a D_z \sin(2\pi n_r t)$ 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; and

0.3 for circular cylinders;

t is the time within the cycle of vibration;

n_r is the frequency of vibration of mode r under consideration;

ρ_a is as defined in **5.2.2**;

$\bar{V}_z$ 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**;

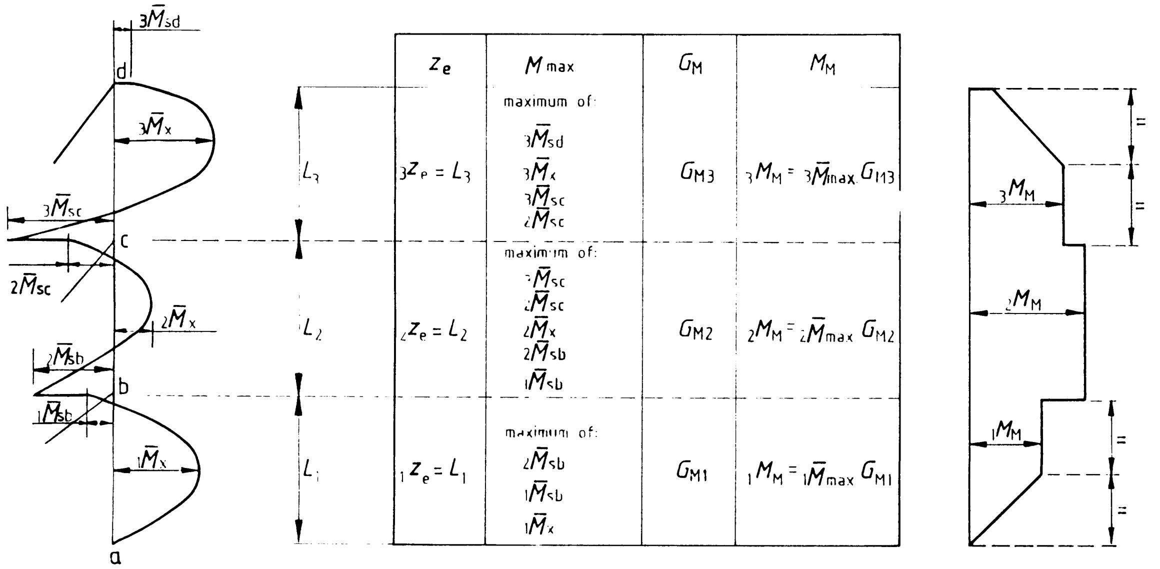
k_e is an excitation coefficient depending on the ratio $1.3\,\bar{V}_z/(V_{cr})r$ and may be determined from [A1) Figure 19 (A1] for circular or square cylinders;

$(V_{cr})r$ is the critical wind speed for mode r.

NOTE The application of this fluctuating load follows the modal displacement.

Damping decrements may be obtained as the structural decrements ignoring aerodynamic damping over the height of the bluff body and taking half the in-line aerodynamic damping over the lattice section (see Annex M).

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.

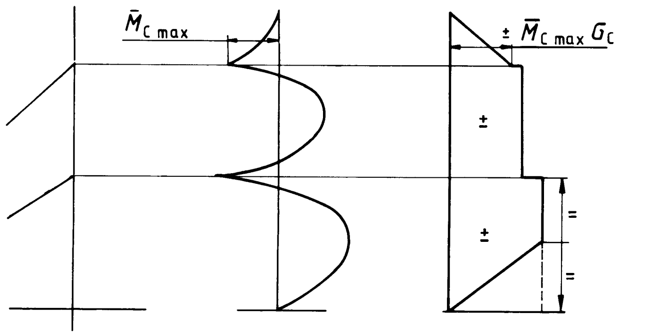


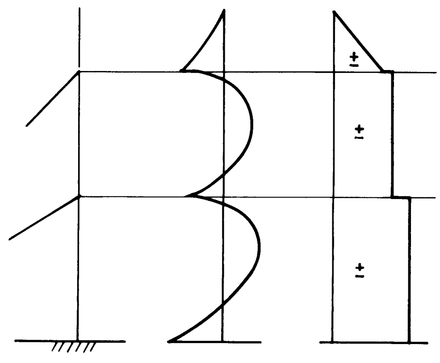
Hourly mean bending moment diagram

Fluctuating moment for each span

Fluctuating bending moment diagram

a) Pinned base mast with no cantilever





Hourly mean bending moment diagram

Fluctuating bending moment diagram

Hourly mean bending moment diagram

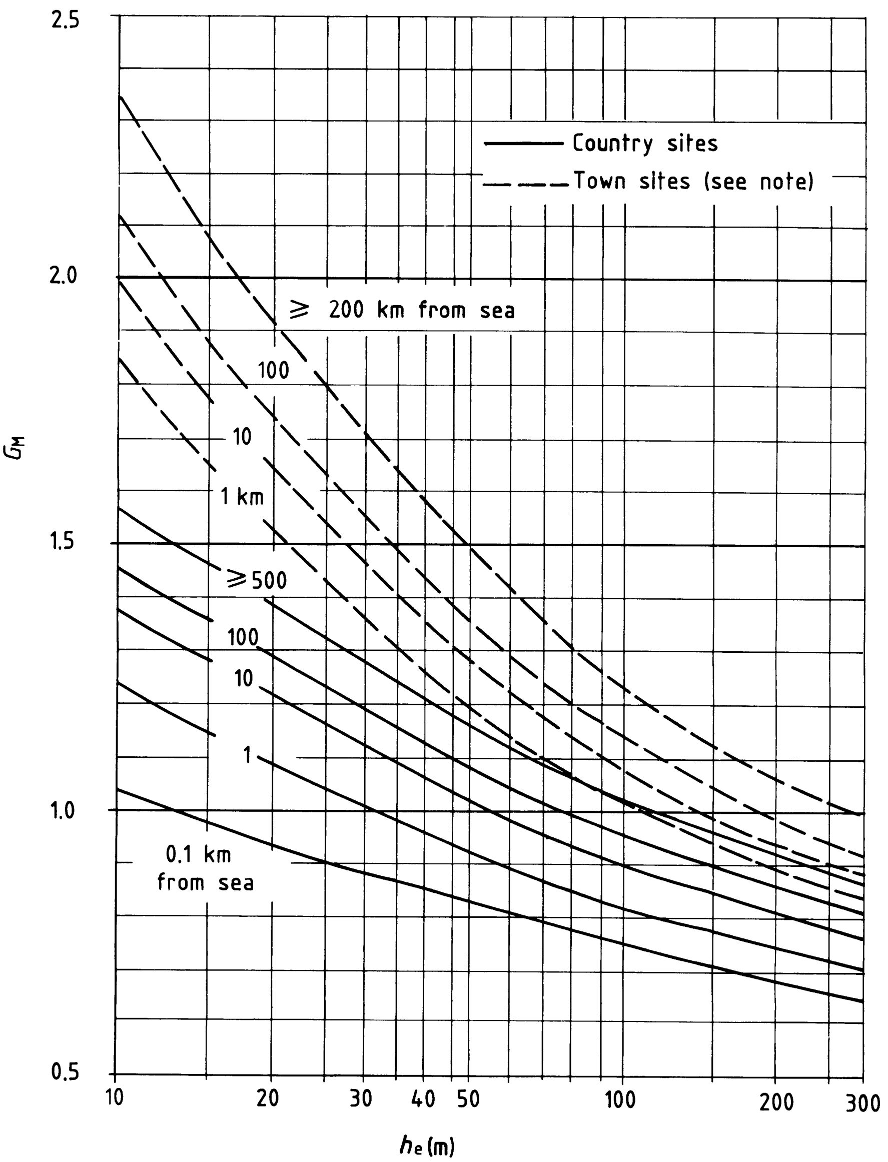
Fluctuating bending moment diagram

b) Pinned base mast with cantilever

c) Fixed base mast with cantilever

NOTE The fluctuating moment for a cantilever is taken to vary from 0 to $\pm\ \bar{M}_{C\ max}\ G_c$ at the base of the cantilever.

Figure 9 — Derivation of fluctuating bending moment diagrams



NOTE This graph has been plotted for sites 1 km into town. For greater distance, multiply the value obtained from the graph by f, where

$$f = \left(1 + \frac{D}{100}\right) \text{ but } \leq 1.15$$

D is the distance into town.

Figure 10 — Basic gust response factor

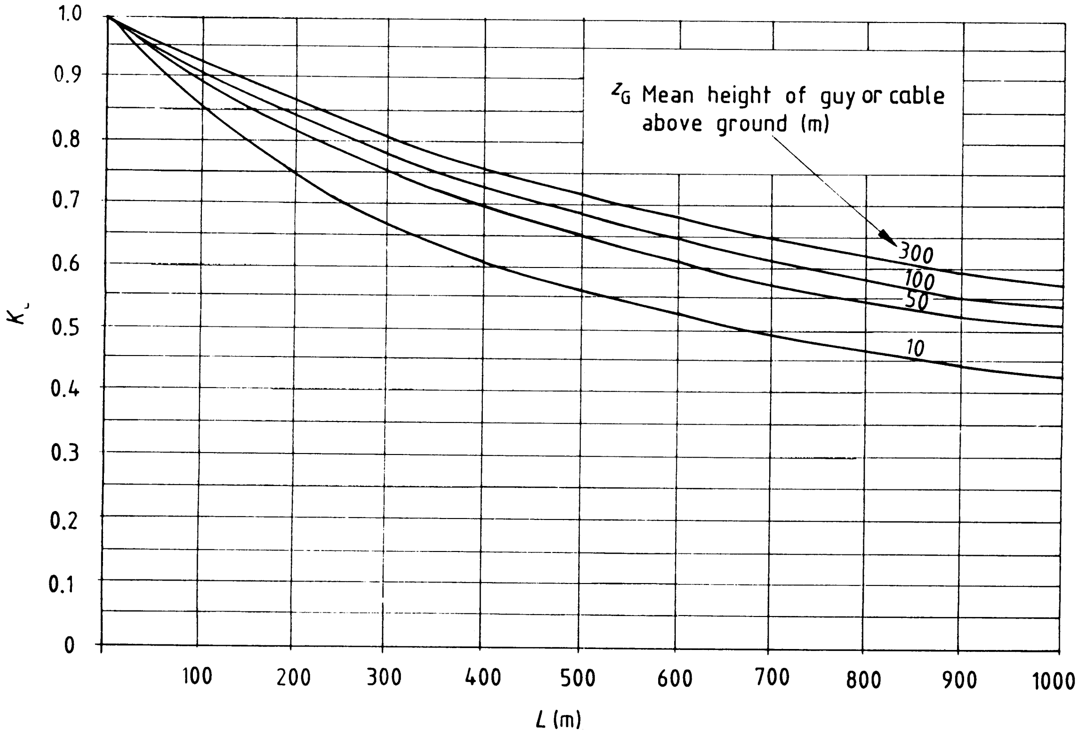


Figure 11 — Length factor K_L for guys or cables

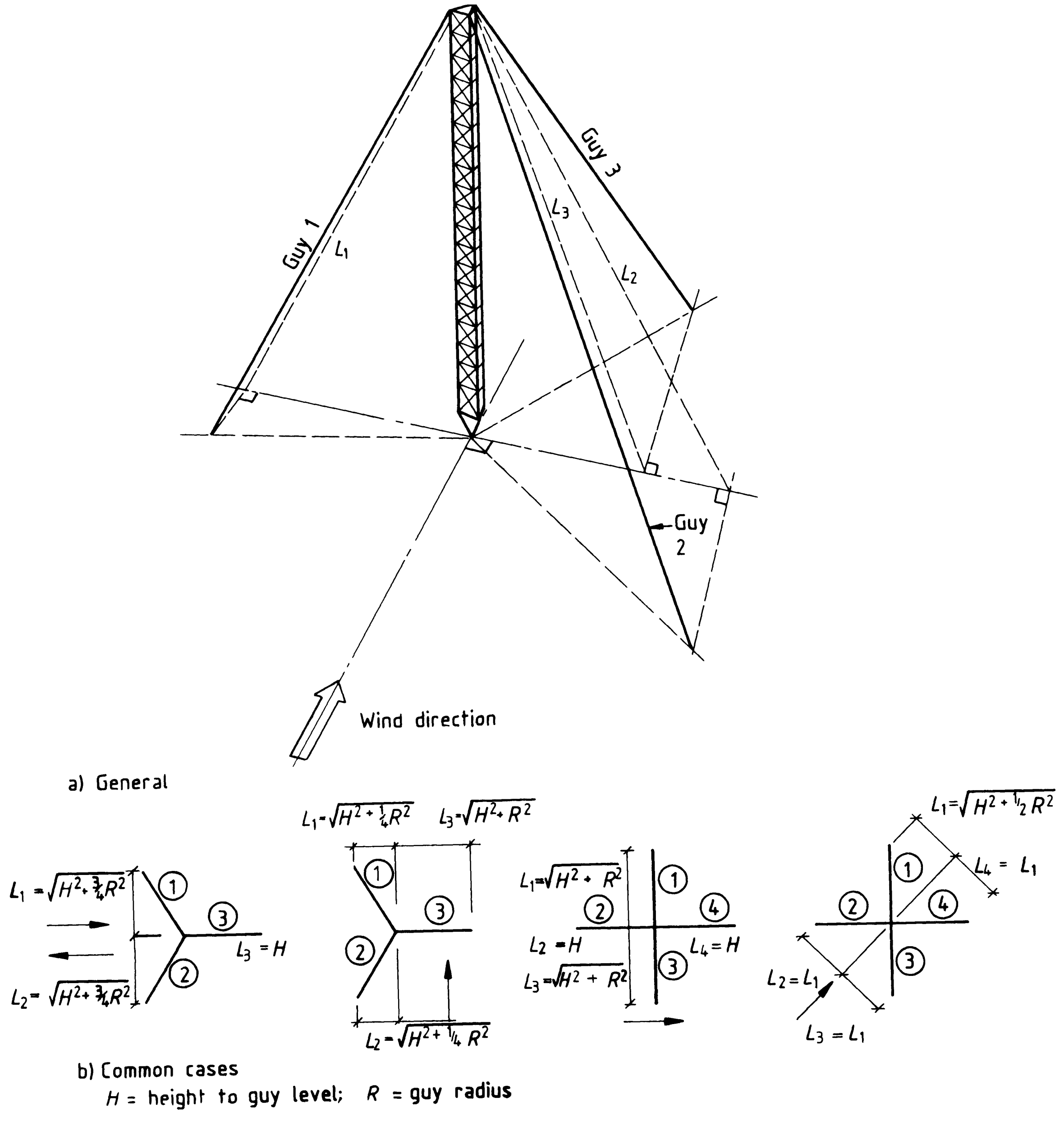
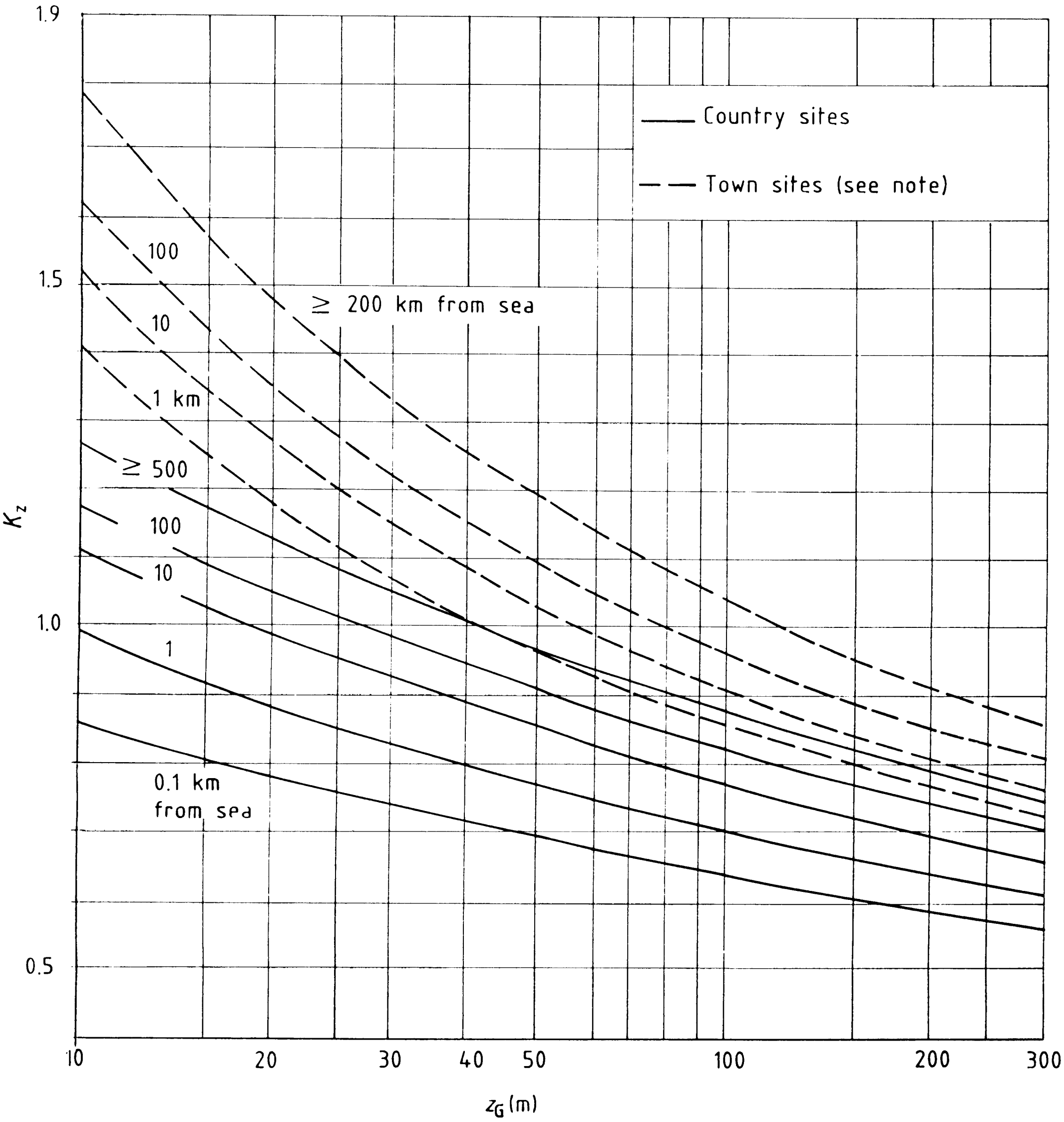


Figure 12 — Projected length of guy



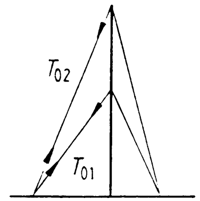
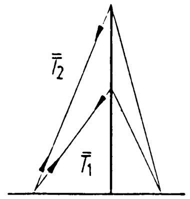
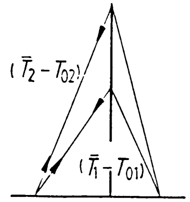
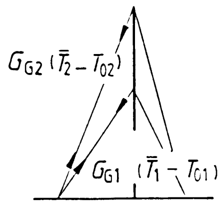
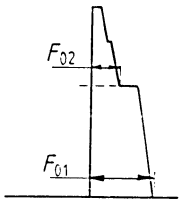
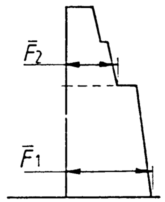
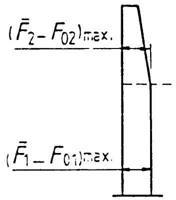
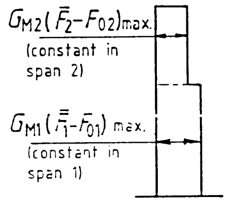
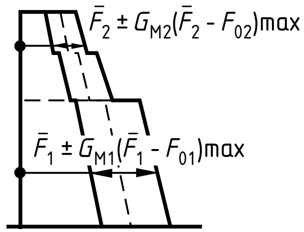
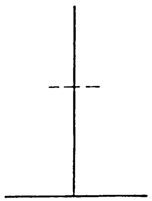
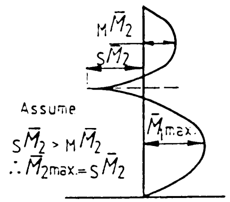
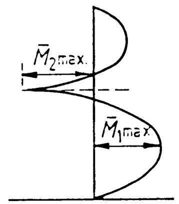
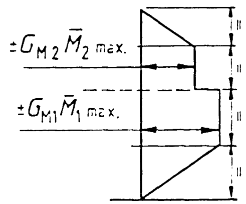
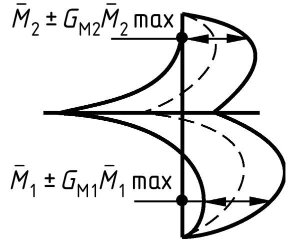
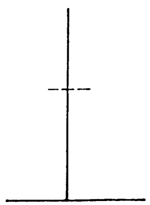
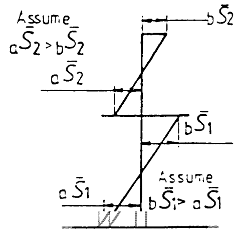
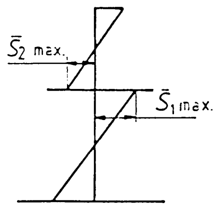
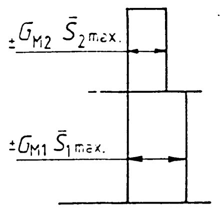
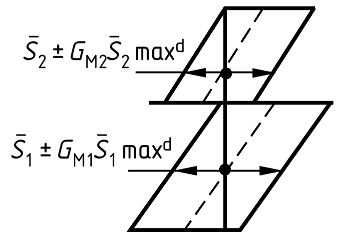
NOTE This graph has been plotted for sites 1 km into town. For greater distances, multiply the value obtained from the graph by f, where

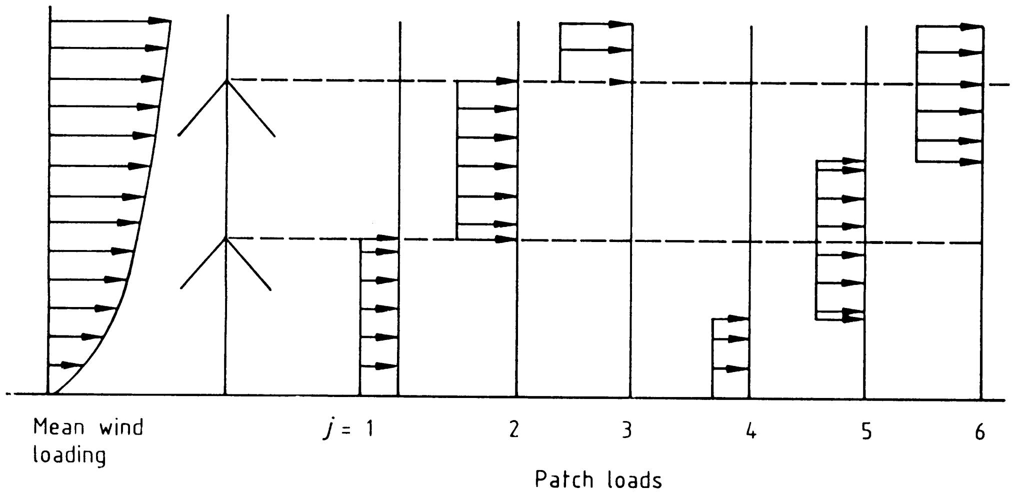
$$f = \left(1 + \frac{D}{100}\right) \text{ but } \leq 1.15$$

D is the distance into town.

Figure 13 — Guy or cable height factor K_z

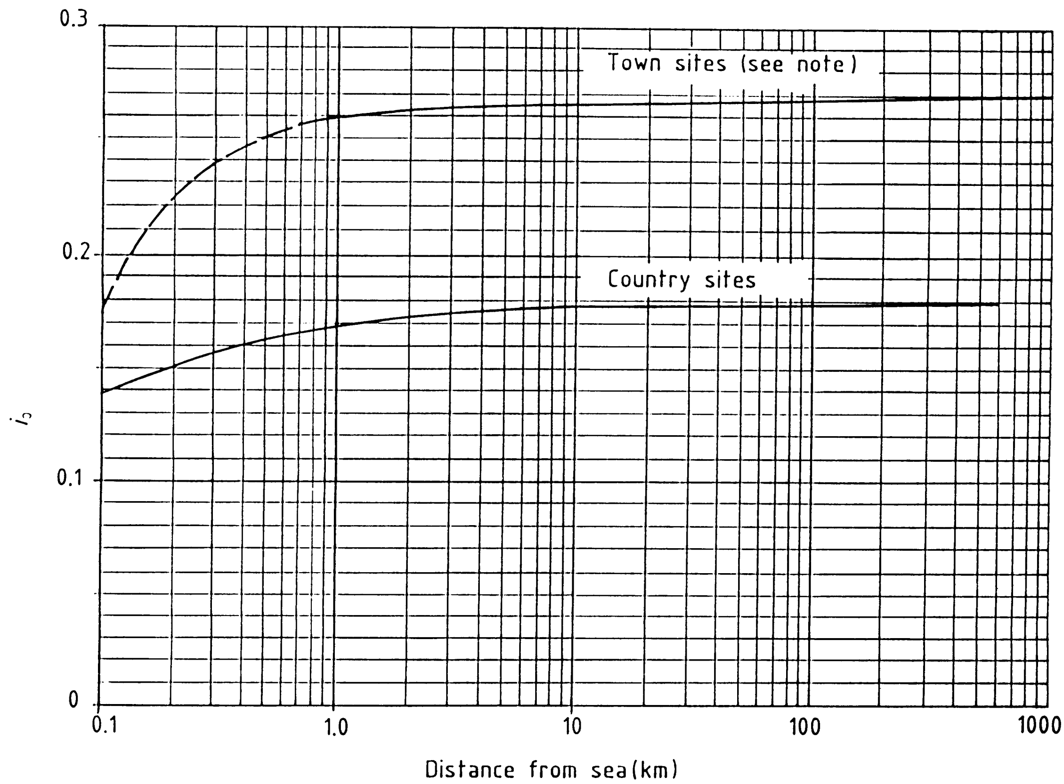
Table 4 — Derivation of total load effects for the procedure given in 5.2

Condition	Still air	Hourly mean (including still air)	Hourly mean — still air	Fluctuating	Total
Stay tensions					Stay 2: $\bar{T}_2 + G_{G2}\,(\bar{T}_2 - T_{02})$ Stay 1: $\bar{T}_1 + G_{G1}\,(\bar{T}_1 - T_{01})$
Column axial loads					
Column bending moments					
Column shear force					



NOTE Each value of j represents a patch load pattern.

Figure 14 — Application of patch loads



NOTE This graph has been plotted for sites 1 km into town. For greater distances, multiply the value obtained from the graph by f, where

$$f = \left(1 + \frac{D}{100}\right) \text{ but } \leq 1.15$$

D is the distance into town.

Figure 15 — Turbulence intensity i_0

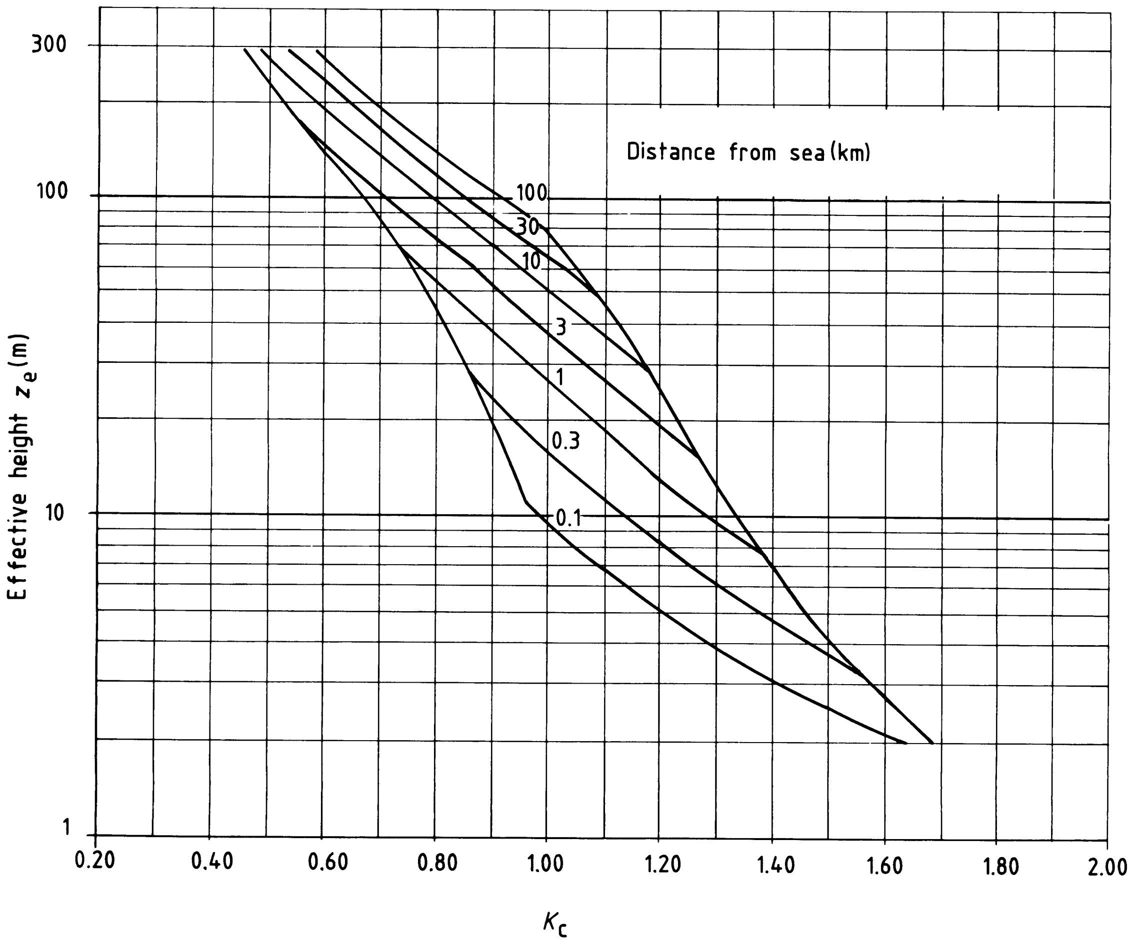


Figure 16 — Turbulence factor K_c

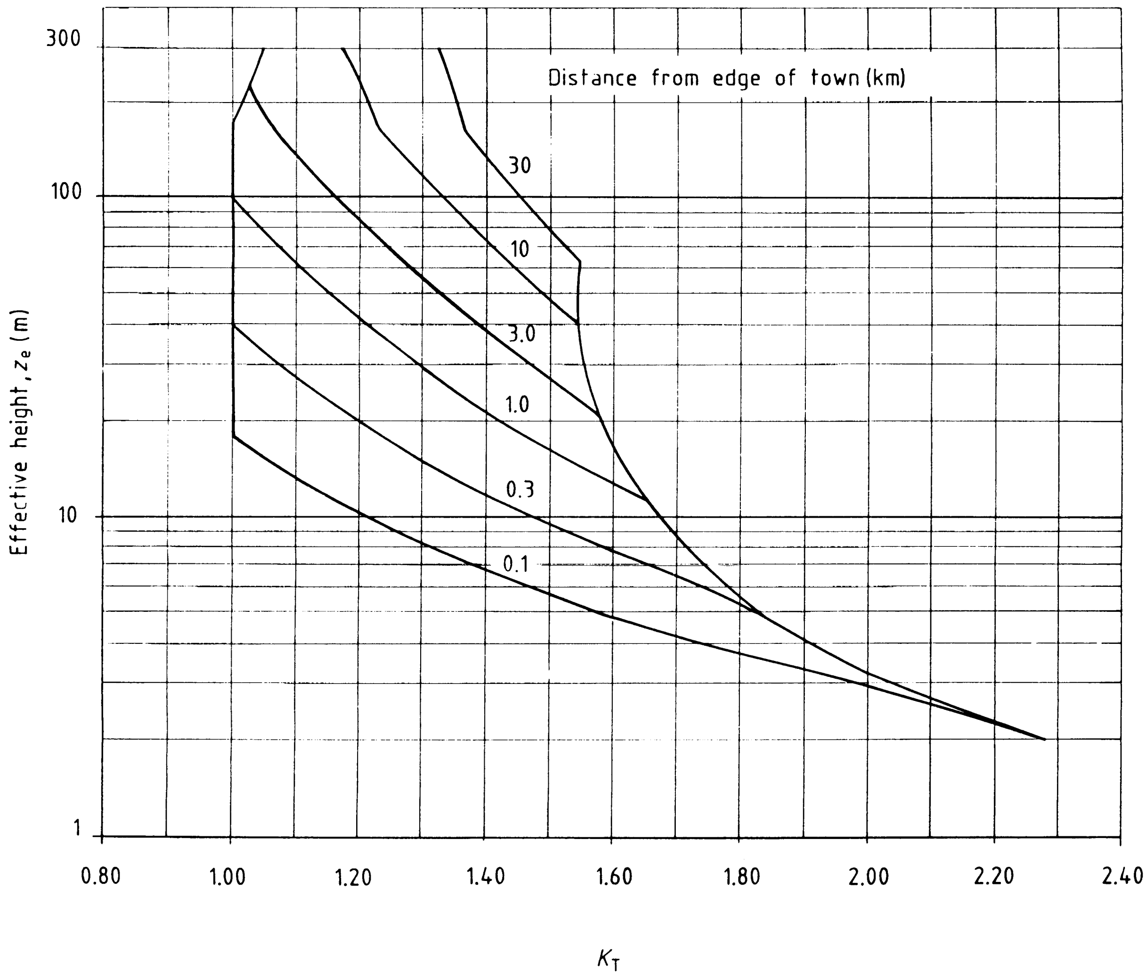
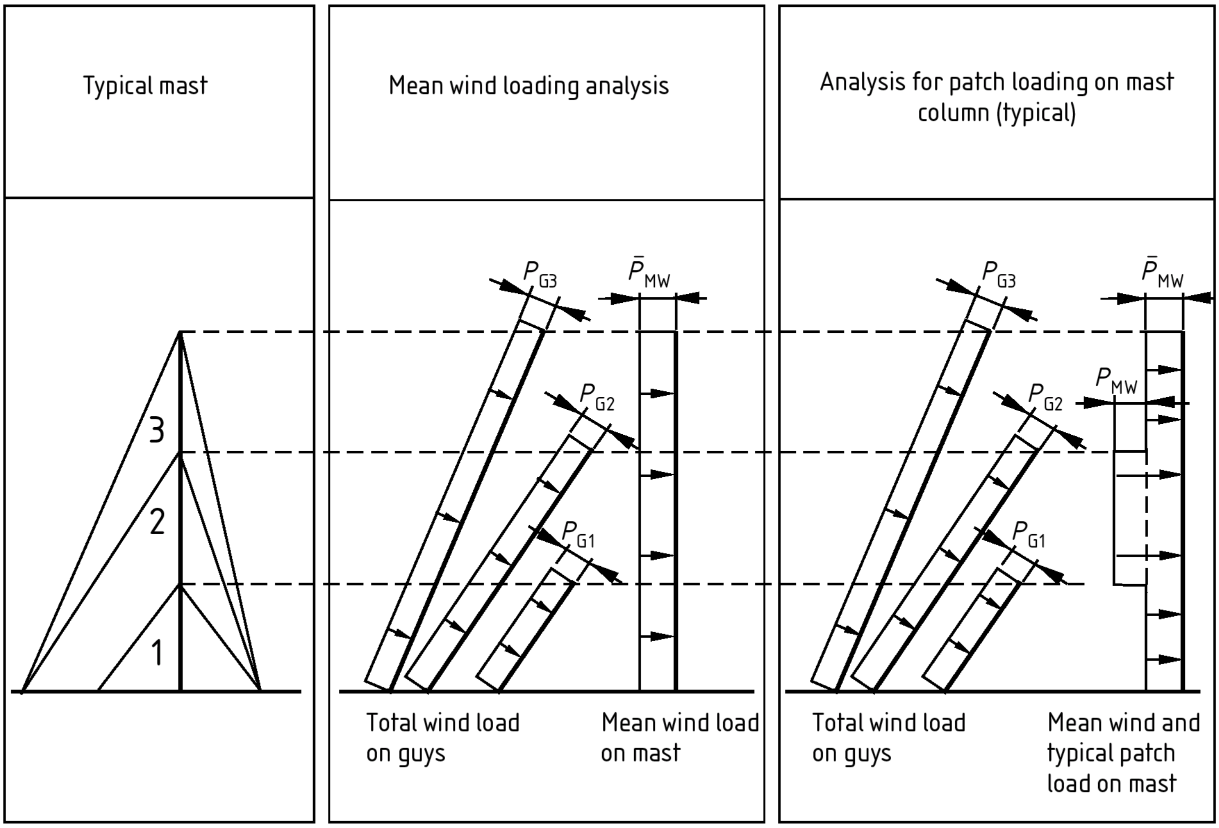


Figure 17 — Factor K_T



NOTE Loading shown for windward guys only.

Figure 18 — Application of patch loading

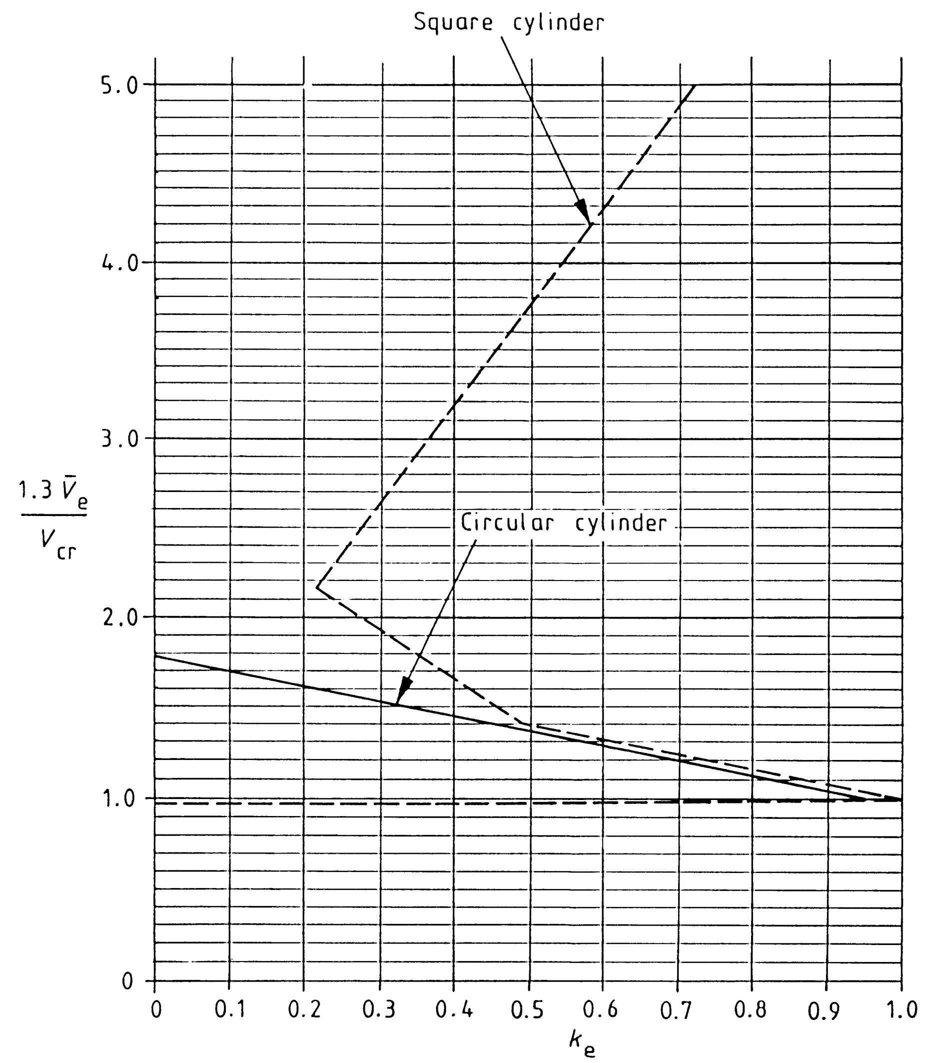


Figure 19 — Excitation coefficient k_e

Annex A (informative)
Method of assessment for classification of quality

A.1 Reliability of a mast

The reliability of a mast 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 Part of BS 8100 are set out in **2.3**, and the procedure detailed in **A.2** may be used as a guide to determine the appropriate quality classification.

A.2 Quality assessment

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

Item	Quality	Rating
1. Design	a) In accordance with all relevant standards 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 independent design checking	0
3. Material	a) In accordance with a British Standard or equivalent	1
	b) Not in accordance with a British Standard or equivalent	0
4. Workmanship and fabrications inspection	a) Very good; in accordance with a 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 inspection; no maintenance	0

Table A.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 A.1 and should be subject to quality control procedures for material, workmanship, fabrication and inspection.

Annex B (normative) Deflections and fatigue

B.1 Downwind deflections

When assessing the conformance of the design to any limitation on downwind deflection, the serviceability limit mean wind speed $\overline{V}_S$ expressed as a ratio of the characteristic wind speed $\overline{V}_k$ independent of direction should be derived as follows:

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

where

$\overline{V}_k$ is as defined in **3.2.1**;

γ_v is to be obtained from Figure 1 (see **2.1**);

Δ is the deflection, determined in accordance with **5.2.7**;

Δ_S is the specified serviceability limit deflection.

When the limiting deflection is applied independently of direction, the corresponding annual duration of unserviceability may be obtained from Figure B.1 as the number of hours that the serviceability limit speed is expected on average to be exceeded.

When the deflection limitation is confined to a certain direction the serviceability limit wind speed should be calculated within each 30° sector producing a deflection of Δ_S in that direction, and a corresponding annual duration of unserviceability for each sector found from Figure B.1. Each period should then be multiplied by the duration factor associated with this sector from Figure B.2; the summation of these factored periods gives the total annual duration of unserviceability in the specified direction.

B.2 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 **5.6**. If the predicted amplitudes of movement are in excess of the limits of satisfactory performance, the duration of serviceability should be estimated by reference to Figure B.3. 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 $\overline{V}_S$ to the characteristic wind speed $\overline{V}_k$ 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 B.3.

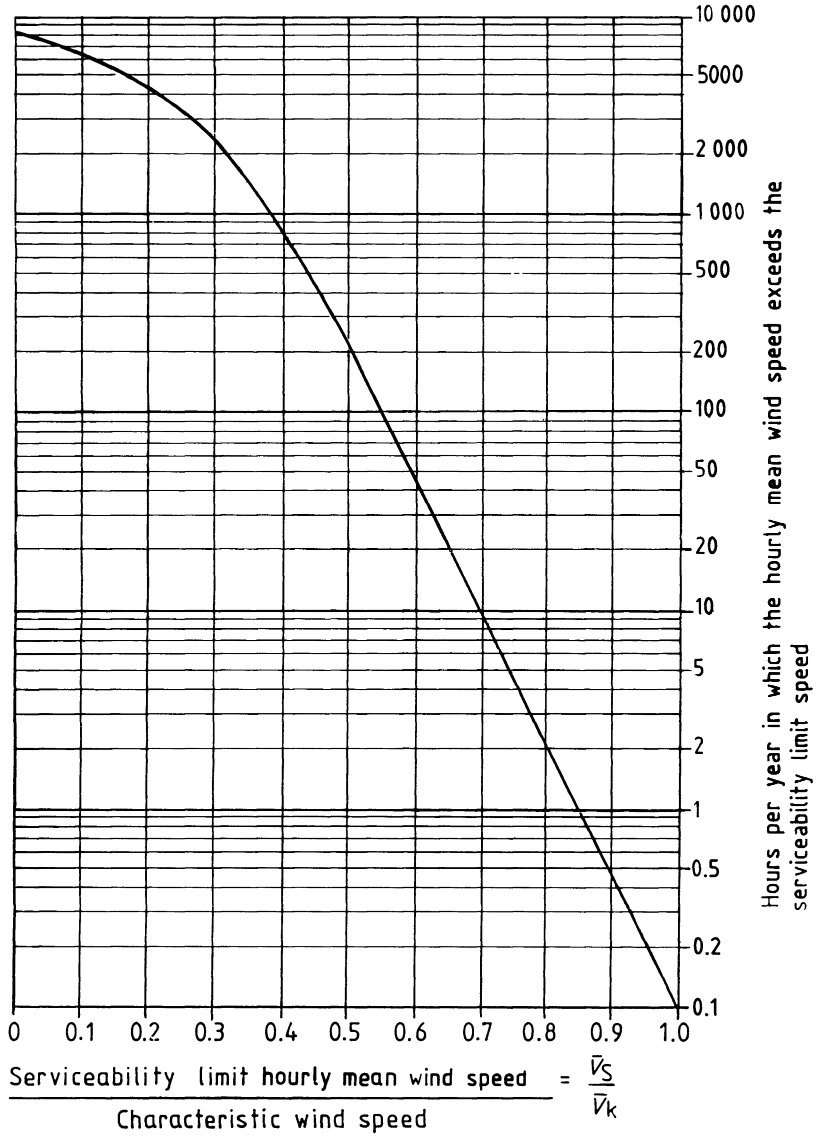


Figure B.1 — Hours per year in which the hourly mean wind speed exceeds the serviceability limit

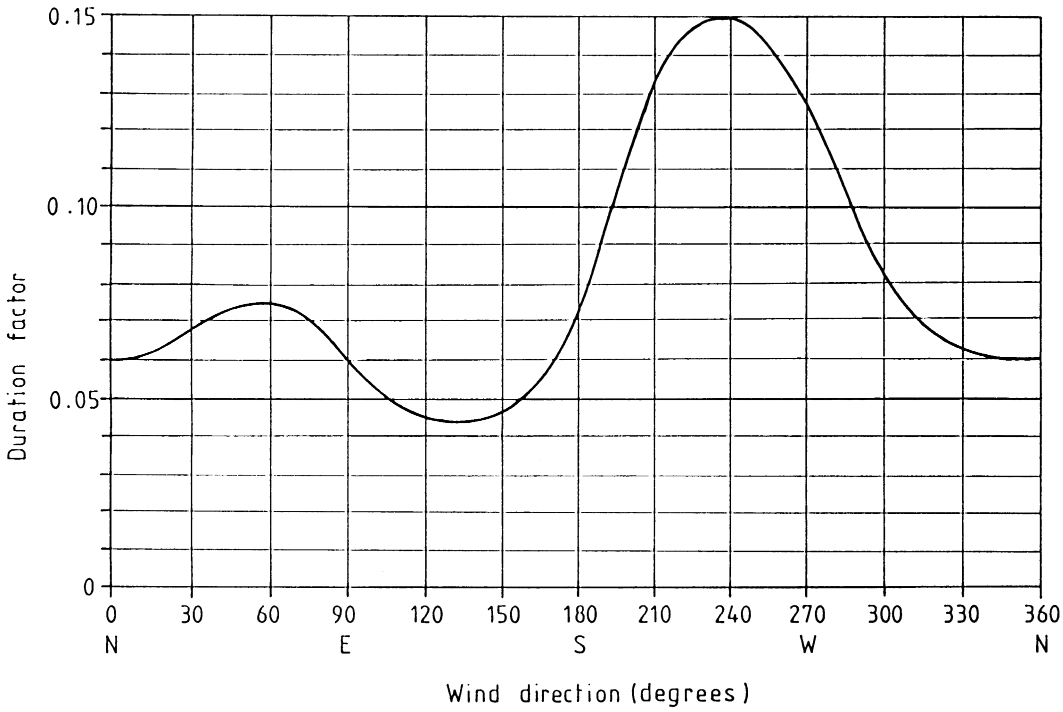
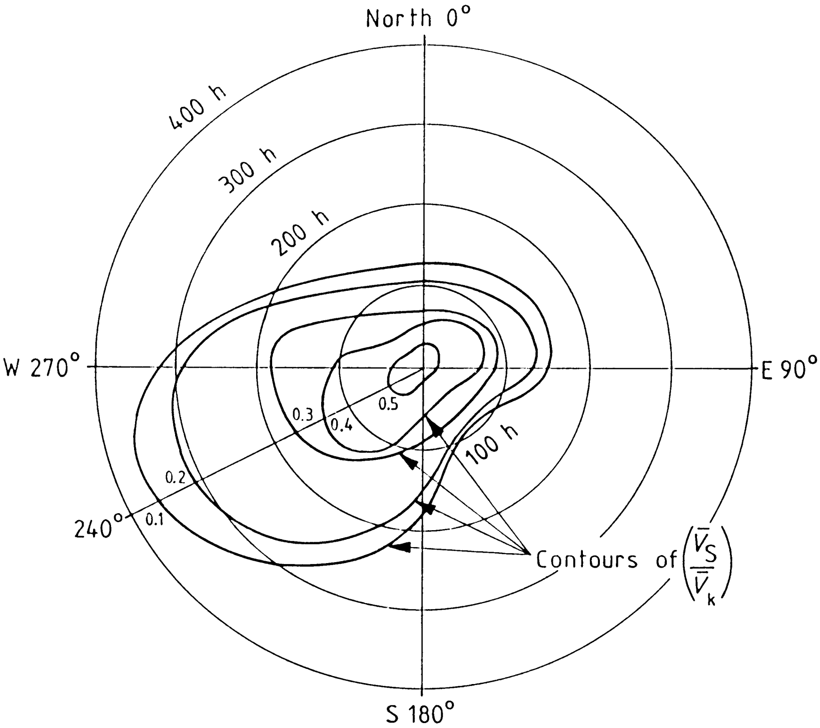


Figure B.2 — Wind speed duration factor



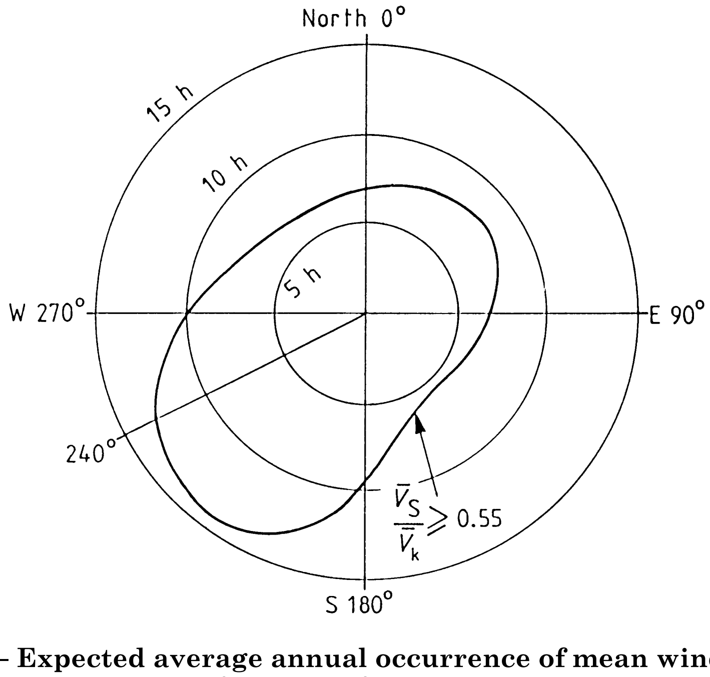


Figure B.3 — Expected average annual occurrence of mean wind speeds as a function of direction

Annex C (informative)
Derivation of extreme wind information: $\overline{V}_b$ and S_d

The wind data archived by the Meteorological Office are derived from continuously recording anemographs, normally exposed at a height of 10 m above ground in open, level terrain. Currently, the network numbers about 130 stations and the main archive comprises hourly mean wind speeds and wind direction, together with details of the maximum gust each hour. Many of these stations have past records spanning several decades, although the computer-held ones generally began in about 1970.

Conventionally, estimation of the extreme wind climate in temperate regions has involved the analysis of a series of annual maximum wind speeds, for example using the method proposed by Gumbel [6]. The main disadvantage of methods using annual maximum values is that many useful data within each year are discarded. For the preparation of the basic wind speed map given in Figure 2 a superior technique involving the maximum wind speed during every period of windy weather (or storm) was used. This approach greatly increased the amount of data available for analysis and enabled the directional and seasonal characteristics of the UK wind climate to be examined.

A storm was defined as an unbroken period of at least 10 h with an overall mean speed greater than 5 m/s. Such periods were identified for 50 anemograph stations, evenly distributed over the United Kingdom and mostly having standard exposures, using their records during the period 1970 to 1980. At the majority of these stations, the average number of storms each year was about 140. The maximum hourly mean wind speed blowing from each of twelve 30° wind direction sectors (centred on 0°, 30°, 60°, etc.) was calculated for each storm.

Three types of extreme wind information were needed: a map of basic wind speeds; wind direction factors; and seasonal factors.

The basic wind speed ($\overline{V}_b$) is estimated to have a probability of 0.02 of being exceeded only once in a year, and is sometimes called the value having a return period of 50 years, i.e. the value likely to be exceeded, on average, only once in 50 years. The map of basic wind speeds was obtained by analysing all the maximum wind speeds v_s in storms at each station, irrespective of direction. The maxima were sorted into ascending order of value and assigned a rank m, where $m = 1$ for the lowest value and $m = N$ for the highest value. The cumulative distribution function (CDF), P, representing the probability of a particular value not being exceeded, is given for storm maxima by $P(v_s) = m/(N + 1)$.

Maxima from different storms can be regarded as statistically independent, so the CDF of annual maxima $P(v)$ was found from $P(v) = \{P(v_s)\}^r$, where r is the average annual rate of storms. This CDF of annual maxima was fitted to a Fisher-Tippett Type 1 (FT1) distribution, defined by

$$P(v) = \exp\{-\exp(-y)\}$$

where

y $= a(v - U)$;

U is the mode; and

$1/a$ is the dispersion.

Hence

$$y = -\ln(-\ln[\{m/(N+1)\}^r])$$

and a plot of y versus v_s led to estimates of the annual mode and dispersion. The basic wind speed $\overline{V}_b$ was found from

$$\overline{V}_b = \left(U + \frac{1}{a}\ln R\right)$$

where R, the return period, is 50 years.

It was noted that the rate of convergence of storm maxima to the FT1 model was faster for dynamic pressures than for wind speeds, so a dynamic pressure model was used and the wind speeds corresponding to the 50 year return period dynamic pressures were determined. Corrections were made to the individual station estimates to ensure that all the basic wind speeds plotted represented a height of 10 m above ground in open, level terrain at mean sea level. Isotachs were then drawn to be a best fit to the speeds plotted.

The anemograph stations used in the storms analysis were all in the United Kingdom, so it was necessary to estimate the isotachs over Eire. This was done by comparing results from the storms analysis for Northern Ireland with a map, prepared by the Irish Meteorological Service, showing isotachs for gust speeds having a return period of 50 years.

The same analysis was performed on the series of maximum wind speeds from each 30° wind direction sector, to yield ratios of the 50 year return period sectorial extreme to the all-direction extreme for wind speed and dynamic pressure. After correction for site exposure, the directional characteristics of extreme winds showed no significant variation with location, with the strongest winds blowing from directions south-west to west. This enabled one set of direction factors to be proposed. The ratios calculated refer to a given risk in each sector. However, due to contributions from other sectors, the overall risk will be greater than the required value. The direction factors S_d have been derived by adjusting sectorial ratios to ensure an evenly distributed overall risk.

Further guidance on derivation of extreme wind information is given in references [6] to [11].

Annex D (informative) Gradient wind speeds

D.1 General

Where the 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 effective site wind speeds $\bar{V}_z$, as detailed in **3.2.1** in accordance with **D.2** and **D.3**.

D.2 Gradient wind speeds

The gradient wind speeds $\bar{V}_g$ should be obtained from the official Meteorological Office wind maps of the average hourly wind speeds, 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 being exceeded of 0.02. An approximate map for the United Kingdom is shown in Figure D.1, for indicative purposes only, based on an assumed gradient wind height of 900 m.

D.3 Effective wind speed

The effective wind speed $\bar{V}_z$ to be used for design should be based on a value of the basic wind speed $\bar{V}_b$ derived from

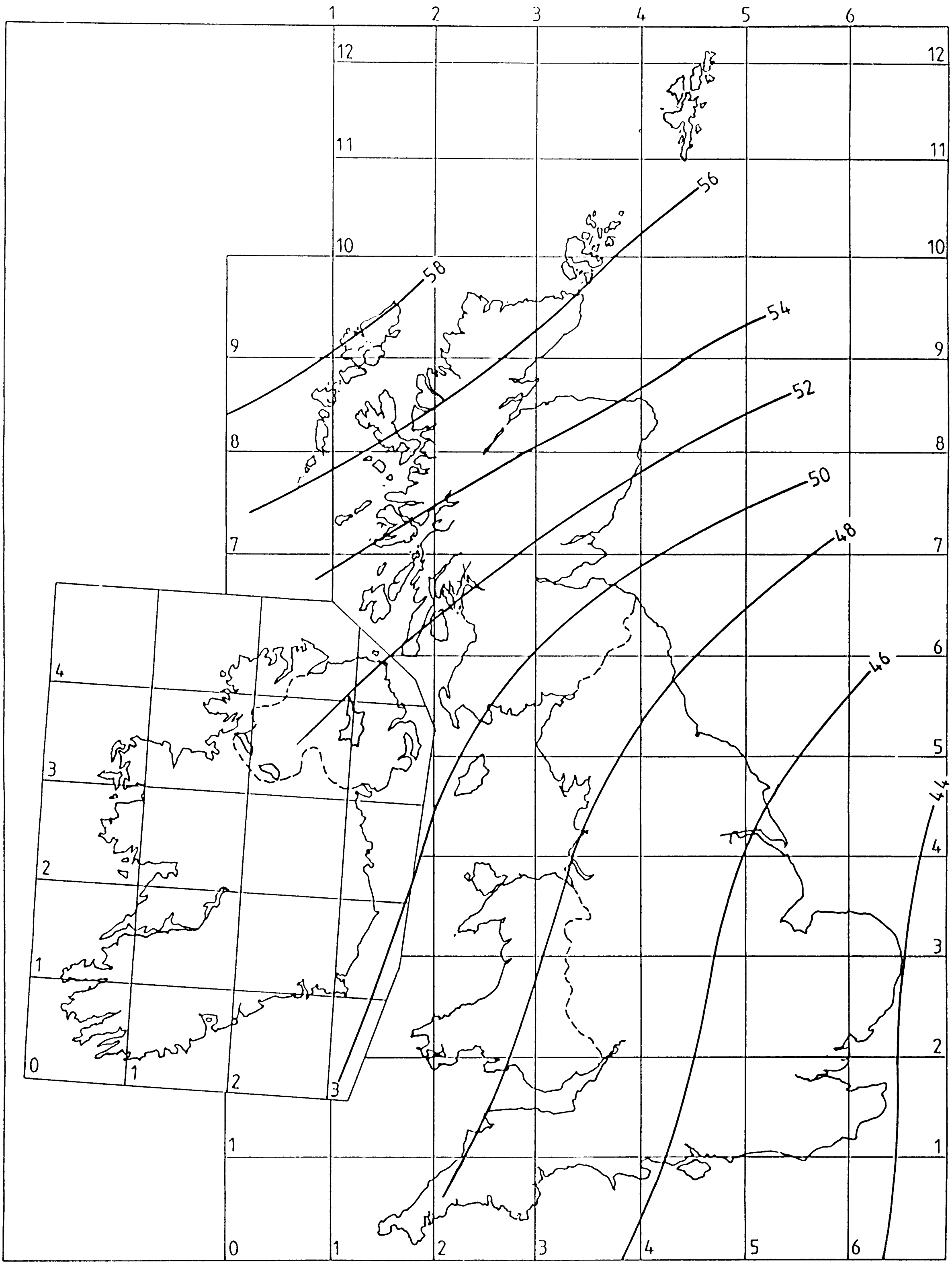
$$\bar{V}_b = K_g \bar{V}_g$$

giving

$$\bar{V}_z = \gamma_v S_a S_d S_o K_g \bar{V}_g$$

where

γ_v	is the partial safety factor on wind speed to be obtained from Figure 1, appropriate to the type of structure (see **2.4.1**);
S_a	is determined in accordance with **3.1.4**;
S_d	is determined in accordance with **3.1.5**;
S_o	is determined in accordance with **3.2.1**;
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 D.2;
$\bar{V}_g$	is the gradient wind speed, determined in accordance with **D.2**.



Wind speed in metres per second.

Figure D.1 — Hourly mean gradient wind speed $\overline{V}_g$ for the United Kingdom

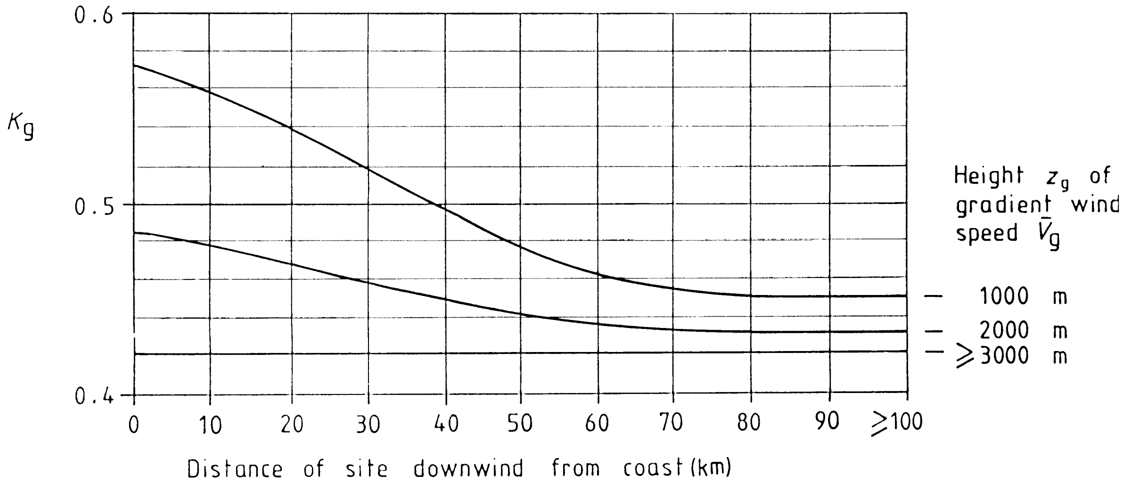


Figure D.2 — Gradient wind speed reduction factor K_g

Annex E (informative) Seasonal factor S_s

For structures which are expected to be exposed to the wind for specific sub-annual periods the factor S_s may be used to reduce the basic wind speeds which then have a risk of being exceeded of 0.02 in the stated period. Values may be obtained from Table E.1.

Table E.1 — Values of S_s for sub-annual periods

1 month	2 months	4 months
Jan 0.98	Jan to Feb 0.98	Jan to Apr 0.98
Feb 0.83	Feb to Mar 0.86	Feb to May 0.87
Mar 0.82	Mar to Apr 0.83	Mar to Jun 0.83
Apr 0.75	Apr to May 0.75	Apr to Jul 0.76
May 0.69	May to Jun 0.71	May to Aug 0.73
Jun 0.66	Jun to Jul 0.67	Jun to Sep 0.83
Jul 0.62	Jul to Aug 0.71	Jul to Oct 0.86
Aug 0.71	Aug to Sep 0.82	Aug to Nov 0.90
Sep 0.82	Sep to Oct 0.85	Sep to Dec 0.96
Oct 0.82	Oct to Nov 0.89	Oct to Jan 1.00
Nov 0.88	Nov to Dec 0.95	Nov to Feb 1.00
Dec 0.94	Dec to Jan 1.00	Dec to Mar 1.00

The factor for the six-month winter period October to March inclusive is 1.0, and the factor for the six-month summer period April to September inclusive is 0.84.

Annex F (informative) Terrain categories

F.1 General

The roughness of the ground surface controls both the mean wind speed and its turbulent characteristics. Over a smooth surface such as open country, the wind speed is higher near the ground than over a rougher surface such as a town. Three basic terrain categories have been defined from which wind speeds can be derived for any intermediate category or to account for the influence of upstream categories which differ from that of the site. The three basic categories defined are as follows.

a) *Sea*. This applies to the sea but also to inland lakes which are large enough to affect the wind speed at the site. Although this Part of BS 8100 does not cover offshore structures, it is necessary to define such a category so that the gradual deceleration of the wind speed from the coast inland can be quantified, and the wind speed for any land-based site can be determined. The aerodynamic roughness length for sea is taken as $z_0 = 0.003$ m.

b) *Country*. This covers a wide range of terrain, from the flat open level or nearly level country with no shelter, such as fens, airfields, moorland or farmland with no hedges or walls, to undulating countryside with obstructions such as occasional buildings and windbreaks of trees, hedges and walls. Examples are farmland and country estates and, in reality, all terrain not otherwise defined as sea or town. The aerodynamic roughness length for country is taken as 0.03 m.

c) *Town*. This terrain includes suburban regions in which the general level of roof tops is about 5 m above ground level (encompassing all two storey domestic housing) provided that the plan-area density of the site is at least as great as that for normal suburban developments for at least 100 m upwind of the site. Whilst it is not easy to quantify the plan-area density, it is expected that the plan area of the buildings is at least 8 % of the total area over 1 km and within a 30° sector of the site (see Figure F.1). The aerodynamic roughness length for town terrain is taken as 0.3 m.

NOTE The aerodynamic roughness of forests and mature woodland is similar to that of town terrain. It is not advisable to take advantage of the shelter provided by woodland unless it is permanent (not likely to be clear-felled).

F.2 Variation of fetch

Fetch refers to the terrain directly upwind of the site. The adjustment of wind speed characteristics as the wind flows from one terrain to another is not instantaneous. At a change from a smoother to a rougher surface the mean wind speed is gradually slowed down near the ground, and the turbulence in the wind increases.

The adjustment requires time to work up through the wind profile and at any site downwind of a change in terrain the wind speed is at some intermediate flow between that for the smooth terrain and that for the fully developed rough terrain.

This gradual deceleration of the mean speed and increase in turbulence has been accounted for in Table 1 by defining the site by its distance downwind from the coast, and in addition if it is in a town, by its distance from the edge of the town.

Shelter of a site from a town upwind of the site has not been allowed for, other than if the site is in a town itself. To do so would introduce too much complexity with only a marginal saving in the resulting wind loads. However, the computer program STRONGBLOW [5] or similar methods can be used to account for such effects.

It is important, if directional effects need to be considered, to take full account of the effects of terrain upwind of the site in conjunction with the direction factor. This becomes particularly significant if the effects of topography need to be considered, as the topography factor S_h will have a major influence on the direction of the appropriate wind speed affected by the topographic feature.

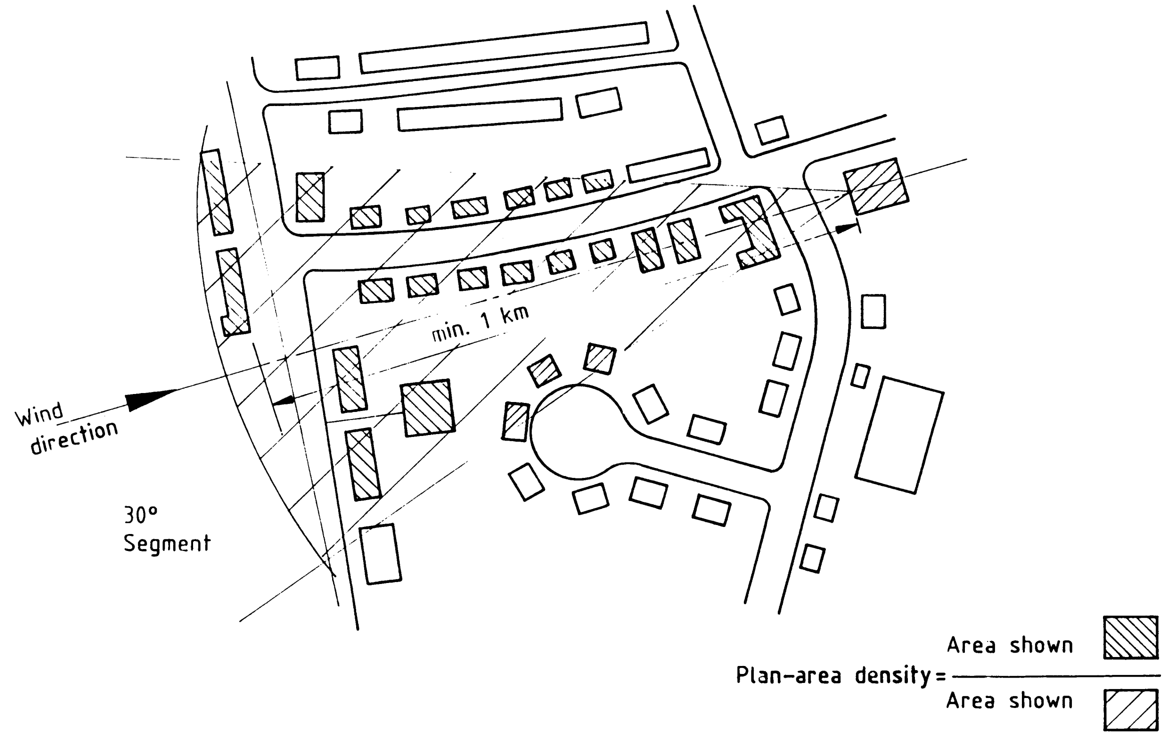


Figure F.1 — Typical town site

Annex G (informative) Effective height

In rough terrain such as towns and cities the wind tends to behave as if the ground level has been raised to a height just below the average roof height, leaving an indeterminate region below which is often sheltered. The displacement height H_d is a function of the plan-area density and general height H_o of the buildings or obstructions. The effective height H_e of any structure that is higher than its surroundings in such terrain is thus the reference height H_r less the displacement height H_d; thus

$$H_e = H_r - H_d$$

The displacement height has been determined by ESDU [1] from available references for urban and woodland terrain.

Based on this work the normal practical range of displacement heights has been found to be $0.75H_o < H_d < 0.90H_o$. A value of $H_d = 0.8H_o$ has been adopted in **3.1.7.3**.

H_d can lie outside this range if the structure to be designed is of a similar height to or lower than its surroundings. A minimum effective height of $H_e = 0.4H_r$ should be adopted.

The displacement height reduces with the spacing X between the upwind buildings and the mast, particularly across open spaces within, or at the edge of, a built-up area. The rules for this effect given in **3.1.7.3** are illustrated in Figure G.1.

Accelerated wind speeds occur close to the base of buildings which are significantly taller than the displacement height. When considering low structures which are close to other tall buildings the rules for effective height will not necessarily lead to conservative values and specialist advice should be sought.

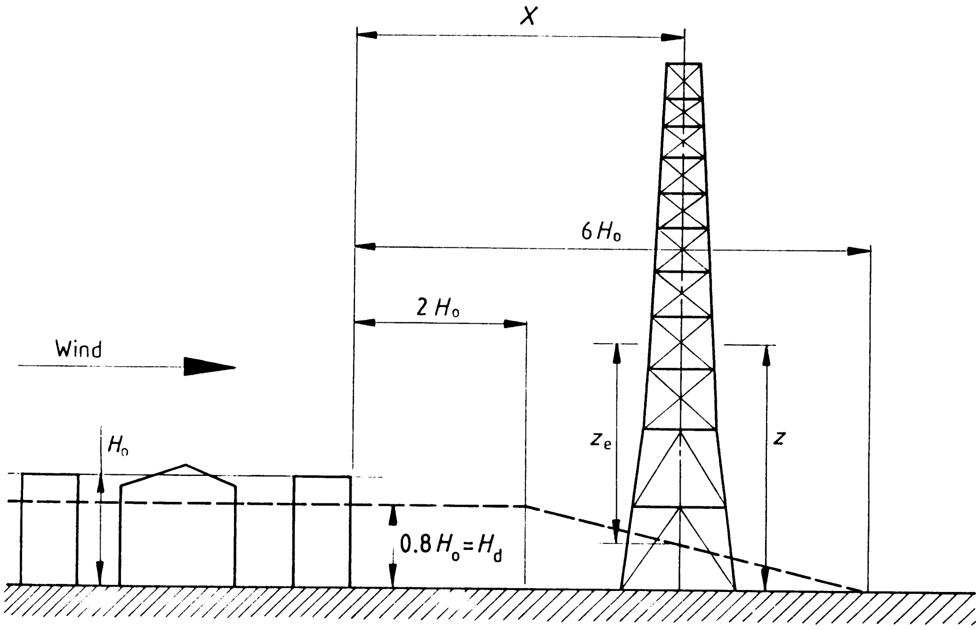


Figure G.1 — Effective heights in towns

Annex H (normative) Topography factors

H.1 General

[A1⟩ The topography factor S_h should be used to modify the terrain and building factor to allow for local topographical features such as hills, valleys, cliffs, escarpments or ridges which can significantly affect the wind speed in their vicinity. Values of S_h should be derived for each wind direction considered and used in conjunction with the corresponding direction factor S_d.

In the vicinity of local topographic features the topography factor S_h is a function of the upwind slope and the position of the site relative to the summit or crest. It should be noted that S_h will vary with height above ground level, from maximum near to the ground reducing to zero at higher levels, and with position from the crest, from maximum near the crest reducing to zero distant from the crest.

Values of the topography factor are confined to the range $0 < S_h < 0.6$ and apply only to the simple topographic features shown in Figure H.1. ⟨A1]

This procedure accounts for single topographic features only, and specialist advice should be sought where the topography is more complex [A1⟩ (see also Annex J) ⟨A1].

H.2 Topography factor

The topography factor S_h should be obtained from Table H.1 using the appropriate values for the slope ψ_U of the hill, the effective length L_e and the factor s which should be determined from:

a) [A1⟩ Figure H.2 for hills and ridges; or

b) Figure H.3 for cliffs and escarpments. ⟨A1]

NOTE 1 Where the downwind slope of a hill or ridge is greater than ψ_D = 0.3 there will be large regions of reduced acceleration or even shelter and it is not possible to give general design rules to cater for these circumstances. Values of s from Figure H.2 should be used as upper bound values.

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

Table H.1 — Values of L_e and S_h

Shallow slope ($0.05 < \psi_U < 0.3$)	**Steep slope** ($\psi_U > 0.3$)
$L_e = L_U$ $S_h = 2s\psi_U$	$L_e = h_e/0.3$ $S_h = 0.6s$
NOTE $\psi_U = h_e/L_U$.	

[A1] In cases transitional between hills and ridges in Figure H.1a), and cliffs and escarpments in Figure H.1b), i.e. when the downwind slope length L_D is much longer than the upwind slope length L_U, it may be difficult to decide which model is the more appropriate. In this case, a value of s may be derived from both Figure H.2 and Figure H.3 and the smaller value used.

At some distance from a topographic feature the effect of local topography is replaced by the general effect of altitude. In many cases, it will not be clear whether topography or altitude dominates. As each is assessed differently it is necessary to calculate the effective wind speed $\overline{V}_z$ twice, as follows, and to take the larger value of $\overline{V}_z$ obtained:

— with topography, using S_a for the terrain base altitude and the appropriate value of S_h; and

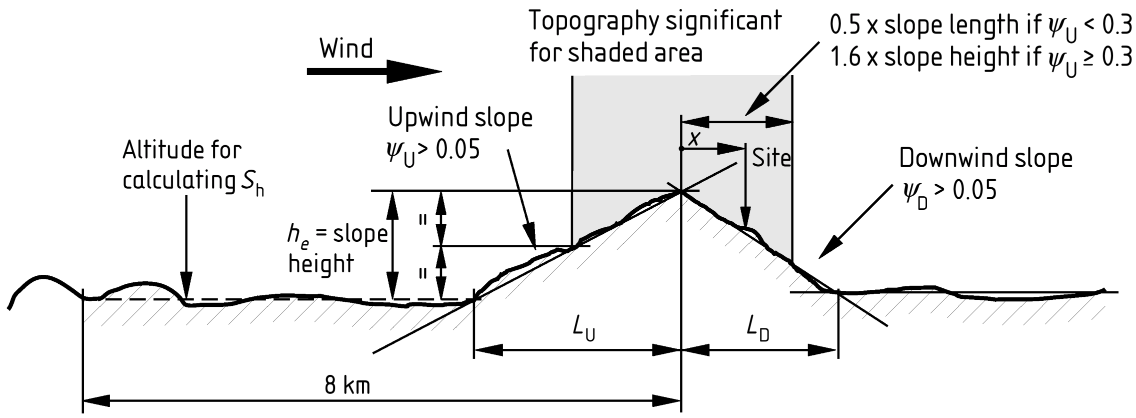
— without topography, using S_a for the site altitude and $S_h = 0$.

This procedure is recommended to determine the limit of topographic influence downwind of a cliff or escarpment.

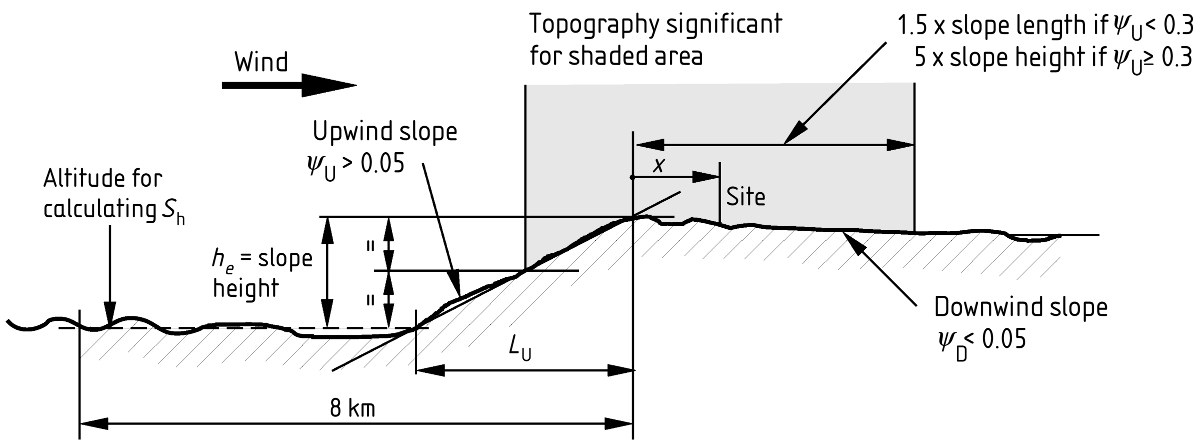
NOTE 1 Care should be exercised where the alternative values of $\overline{V}_z$ derived from the above two procedures are similar. The mast should be designed for both wind speed profiles to determine which gives rise to the greatest load effects.

NOTE 2 The altitude factor for high altitude sites (> 300 m) is subject to review and for such sites a special investigation should be undertaken. [A1]

A1



a) Hill and ridge (upwind slope > 0.05; downwind slope > 0.05)



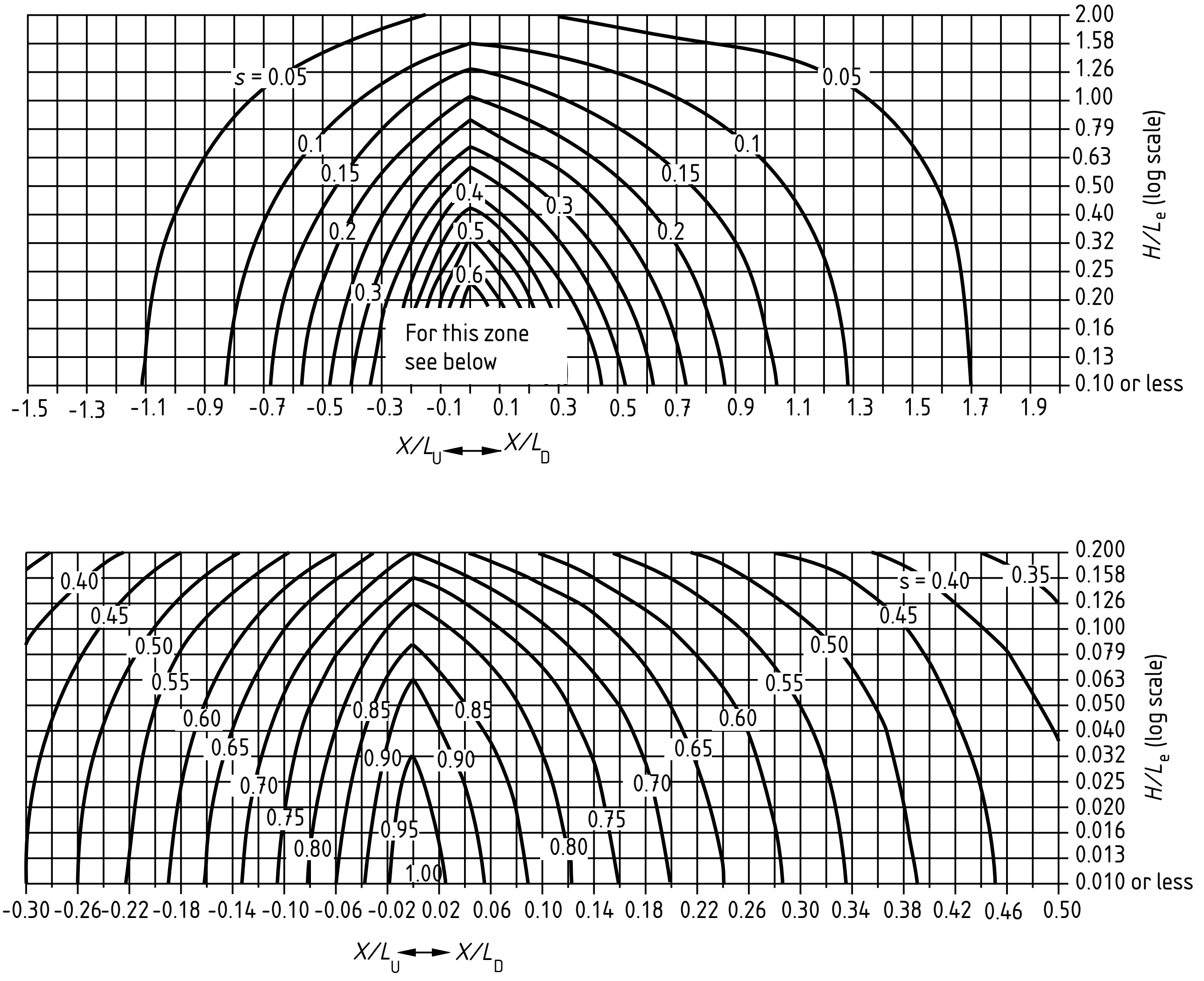
b) Escarpment (0.3 > upwind slope > 0,05; downwind slope > 0.05) and cliff (upwind slope > 0.03; downwind slope > 0.05)

Key

L_D	Length of the downwind slope in the wind direction
L_U	Length of the upwind slope in the wind direction
X	Horizontal distance of the site from the crest
h_e	Effective height of the feature
S_h	Topography factor
Ψ_U	Upwind slope h_e/L_U in the wind direction
Ψ_D	Downwind slope h_e/L_D in the wind direction

Figure H.1 — Definition of topographic dimensions

A1



NOTE The basis for the derivation of the values in Figure H.2 and Figure H.3 is given in Annex G of BS 6399-2.

Figure H.2 — Topographic location factor *s* for hills and ridges

A1

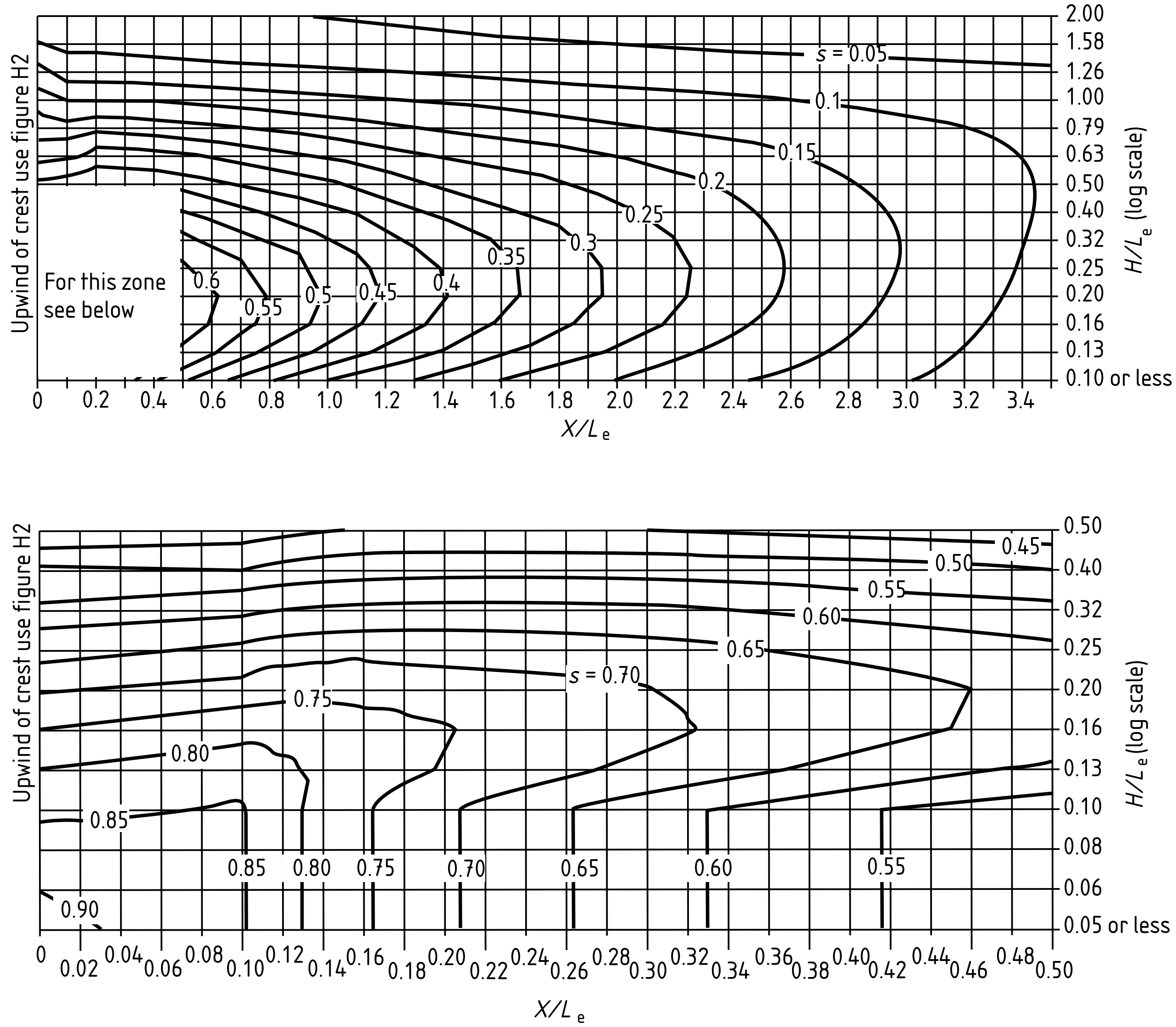


Figure H.3 — Topographic location factor *s* for cliffs and escarpments

A1

Annex J (informative)
Hill parameters in undulating terrain

The procedure for defining hill-site position, altitude of surrounding terrain and effective hill height h_e (see Annex H) is shown in Figure J.1.

1. Plot a profile for 8 km in front of the site in the windward direction

2. Draw a typical slope β_e from the hill on which the structure is situated (through C). If the site is behind the brow of the hill, draw an average slope in the windward direction

3. Draw a horizontal through the minimum height level h_{min} within the 8 km distance

4. Find the average height h_{av} over the distance between the 8 km position (O) and the intersection of the slope line with the minimum height line (D)

5. Draw a horizontal at height (h_{av} − h_{min}) above the average height level

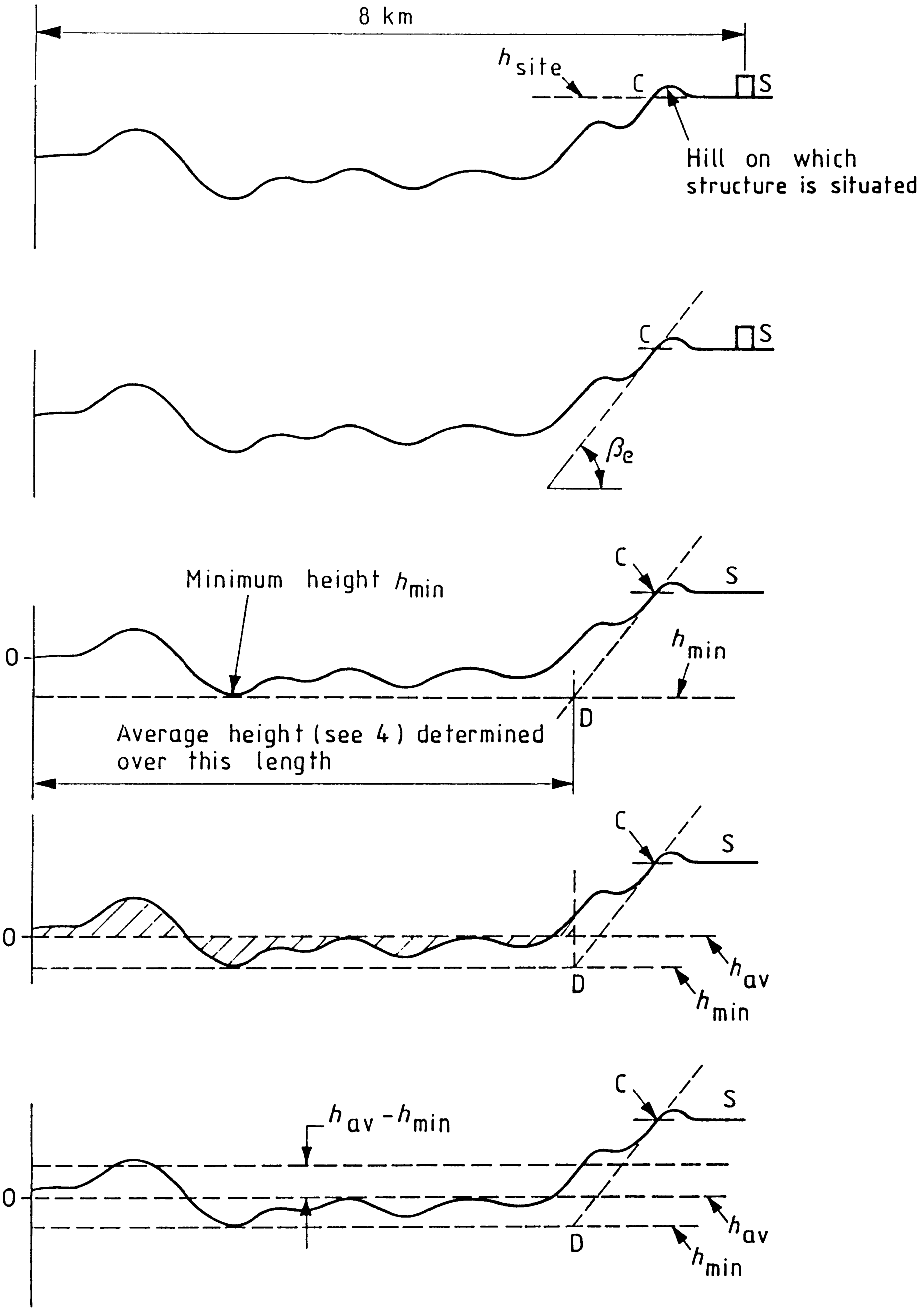
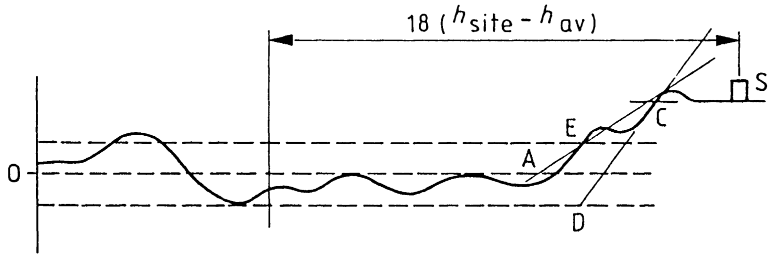
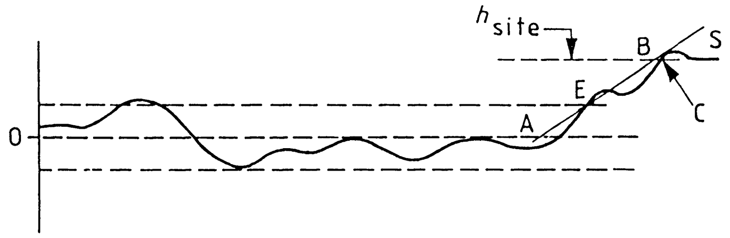


Figure J.1 — Hill parameters in undulating terrain *(continued)*

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



7. Draw a horizontal through the 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 **3.1.4** and annex H, taken as:

Δ = h_{av}
L = AF
h_e = h_{site} – h_{av}
ψ_U = h_e/L
X = BS

where

ψ_U is the mean hill slope;
X is the distance of the site downwind from the crest.

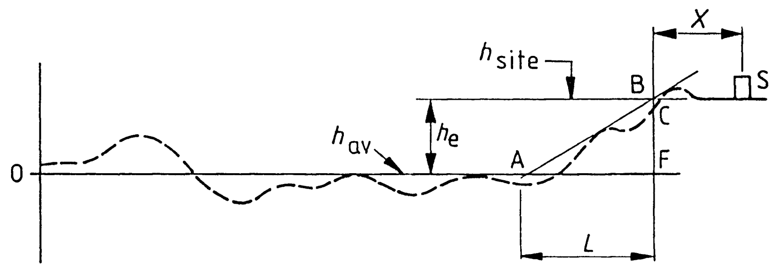


Figure J.1 — Hill parameters in undulating terrain *(concluded)*

Annex K (normative) General method for determination of wind resistance

K.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 structure containing parts or ancillaries outside the constraints of **4.1.2** or of a structure of rectangular unequal-sided cross section should be taken as follows.

— For square and rectangular structures:

$$\Sigma R_W = R_{1e}\cos^2 \theta_1 + R_{2e}\sin^2 \theta_1$$

— For triangular structures:

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

where

R_{1e} is an effective resistance given by the following.

— For square and rectangular structures:

$$R_{1e} = (R_1 + \eta_1 R_3)K_{\theta 1}$$

— For triangular structures:

$$R_{1e} = \left\{R_1 + \frac{\eta_1}{2}(R_2 + R_3\right\} K_{\theta 1}$$

R_{2e} is an effective resistance given by the following.

— For square and rectangular structures:

$$R_{2e} = (R_2 + \eta_2 R_3)K_{\theta 2}$$

— For triangular structures:

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

R_1 to R_4 are wind resistances given by

$$R_1 = A_{s1}C_{N1} + R_{AW1}$$
$$R_2 = A_{s2}C_{N2} + R_{AW2}$$
$$R_3 = A_{s3}C_{N3} + R_{AW3}$$
$$R_4 = A_{s4}C_{N4} + R_{AW4}$$

A_{s1} to A_{s4} are the areas projected normal to faces 1 to 4, respectively, of the components treated as structural members within the same panel height of faces 1 to 4 including icing, where appropriate (see Figure 6).

C_{N1} to C_{N4} are the normal drag (pressure) coefficients appropriate to faces 1 to 4, respectively, of the components treated as structural members which may be determined in accordance with **K.2**.

R_{AW1} to R_{AW4} are the wind resistances appropriate to faces 1 to 4, respectively, for the ancillary items not treated as structural members, determined in accordance with **4.3** or **4.4** as appropriate, but taking $K_A = 1.0$ in all cases.

η_1 and η_2 are the effective shielding factors for faces 1 and 2, respectively, including both structural and ancillary components.

For square and rectangular structures, η_1 and η_2 should be taken as

$$\eta' + 0.15(\omega - 1)\ (\varphi - 0.1)$$

but not greater than 1.0.

For triangular structures, η_1 and η_2 should be taken as

$$\tfrac{2}{3}\ \eta' + 0.15(\omega - 1)(\varphi - 0.1)$$

but not greater than 1.0.

η' [A1] $= \eta_f \dfrac{A_f + 0.83A_c + 2.1A_{c'} + A_A}{A_s + A_A}$ [A1]

but not greater than 1.0.

η_f is obtained from Figure K.1 applicable to face 1 or 2, as appropriate using φ as defined in this subclause.

A_f, A_c and $A_{c'}$ are as defined in **4.2.2** applicable to face 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 to 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 face 1 or 2, as defined on Figure 7, but including both structural and ancillary components (see Figure 6).

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 structures.

$K_{\theta 1}$ and $K_{\theta 2}$ are to be obtained from Figure 7, applicable to face 1 or 2, as appropriate, using A_F, A_f and φ as defined in this subclause.

θ is the plan angle of incidence of wind to the normal to face 1.

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

K.2 Normal drag (pressure) coefficients for single frames

Normal drag (pressure) coefficients C_N for single frames are given in Figure K.2.

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

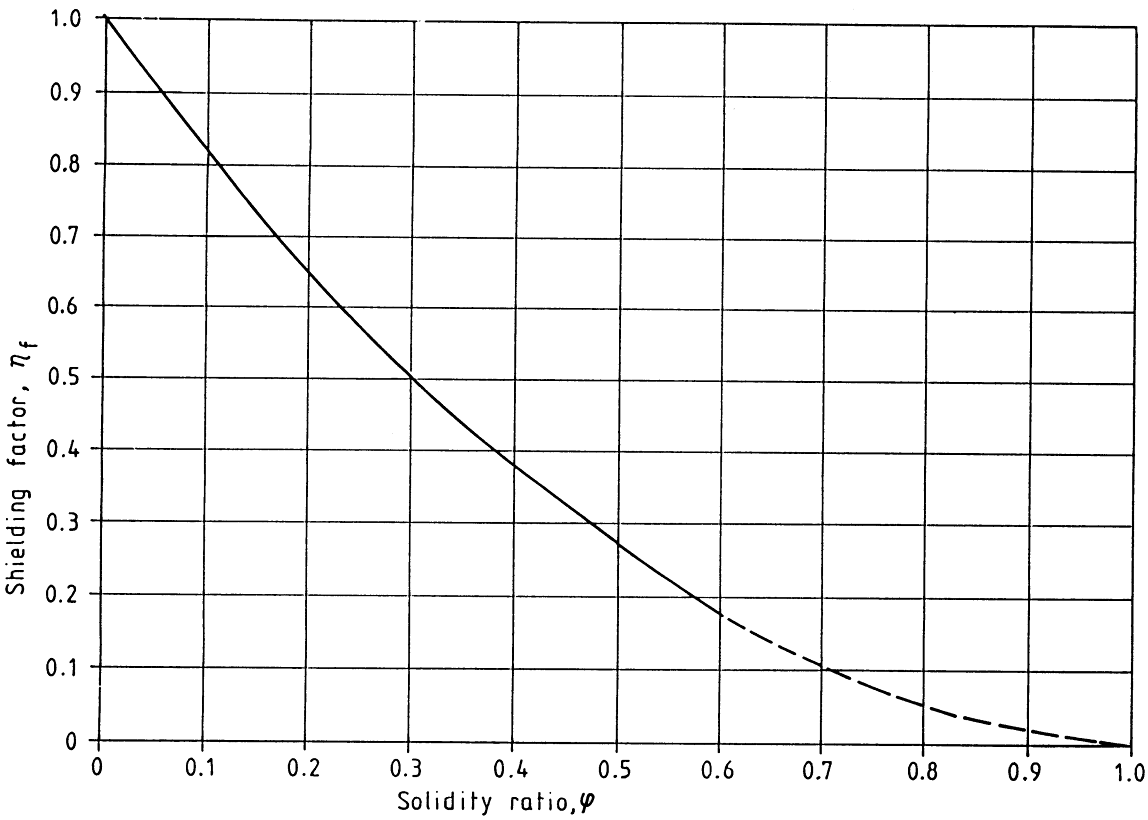
$$C_{Nf}\frac{A_f}{A_s} + C_{Nc}\frac{A_c}{A_s} + C_{Nc'}\frac{A_{c'}}{A_s}$$

where

C_{Nf}, C_{Nc} and $C_{Nc'}$ are the normal drag (pressure) coefficients for flat-sided, subcritical circular-section and supercritical circular-section members, respectively, to be obtained from Figure K.2, appropriate to the solidity ratio of the face considered, as defined in **K.1**;

A_s is as defined in **4.2.1**;

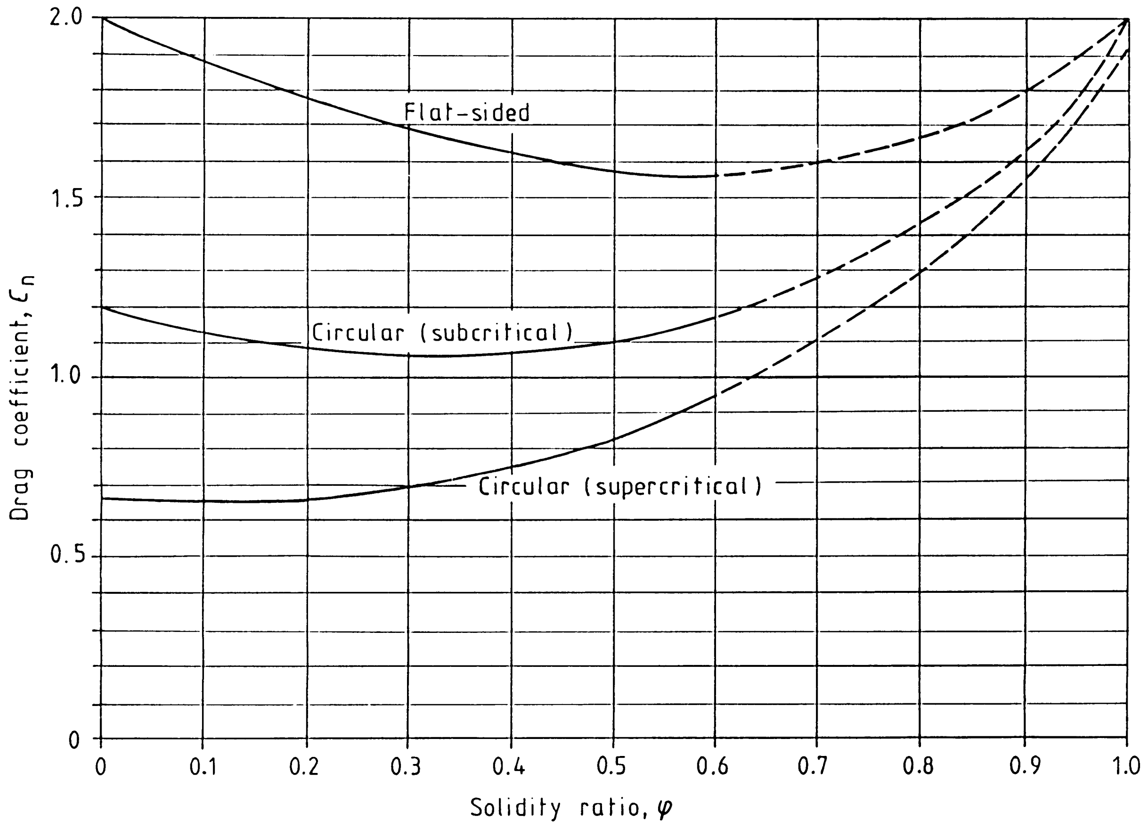
A_f, A_c and $A_{c'}$ are as defined in **4.2.2**.



NOTE 1 For the basis of the curves see Annex L.

NOTE 2 For structures with $\varphi > 0.6$, consideration should be given to the possibility of crosswind response due to vortex excitation (see **5.6**).

Figure K.1 — Shielding factor η_f for single frames composed of flat-sided members



NOTE 1 For the basis of the curves see Annex L.

NOTE 2 For structures with $\varphi > 0.6$ consideration should be given to the possibility of crosswind response due to vortex excitation (see **5.6**).

Figure K.2 — Normal drag (pressure) coefficient C_N for single frames

Annex L (normative) Equations used for the production of the curves in the figures

L.1 General

As an alternative to obtaining values from the graphs given in the figures in this Part of BS 8100, the equations given in **L.4**, **L.5** and **L.6** may be used for calculation of the required values; these equations should only be used within the bounds of the figures themselves. However, some figures have been produced from either best-fit curves or complex computer output. For these no direct equations can be provided.

L.2 Symbols

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

L.3 Meteorological parameters in Figure 3 and Figure 4: fetch factor S_c and fetch adjustment factor T_c

These curves have been derived from reference [6].

L.4 Wind resistance

L.4.1 *Figure 7: wind incidence factor,* K_θ

The wind incidence factor K_θ is given by the following.

— For square structures:

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

— For triangular structures:

$$K_\theta = \frac{A_c + A_{c'}}{A_F} + \frac{A_f}{A_F}(1 - 0.1\sin^2 1.5\,\theta)$$

where

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

K_2 = 0.2 for $0 \le \varphi \le 0.2$ and $0.8 \le \varphi \le 1.0$

= φ for $0.2 < \varphi < 0.5$

= $1 - \varphi$ for $0.5 \le \varphi < 0.8$;

A_f, A_F, θ and φ are as defined on Figure 7;

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

L.4.2 ***Figure 8: overall normal drag (pressure) coefficients*** C_N ***for square and triangular structures***

The normal drag (pressure) coefficients for structures composed of flat-sided members, C_{Nf}, subcritical circular-section members, C_{Nc}, and supercritical circular-section members, $C_{Nc'}$, are given by

$$C_{Nf} = 1.76C_1(1 - C_2\varphi + \varphi^2)$$
$$C_{Nc} = C_1(1 - C_2\varphi) + (C_1 + 0.875)\varphi^2$$
$$C_{Nc'} = 1.9 - \sqrt{(1 - \varphi)(2.8 - 1.14C_1 + \varphi)}$$

where

C_1 is equal to:

2.25 for square structures;

1.9 for triangular structures.

C_2 is equal to:

1.5 for square structures;

1.4 for triangular structures;

φ is as defined on Figure 7.

L.5 Structural response to wind

L.5.1 ***General***

Figure 10, Figure 11 and Figure 13 were derived from the meteorological data reference [6] together with the gust response factors derived from BS 8100-1. The curves so derived are empirical and no direct equations can be provided.

L.5.2 ***Figure 15***

Figure 15 was derived from reference [6].

L.6 General method for wind resistance (see Annex K)

L.6.1 ***Figure K.1: shielding factor,*** η_f

The shielding factor η_f is given by

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

where φ is as defined on Figure 7.

L.6.2 ***Figure K.2: normal drag (pressure) coefficient*** C_N ***for single frames***

The normal drag (pressure) coefficient C_N for single frames is given by

[A1] $$C_N = C_{Nf}\left(\frac{A_f + (0.6 + 0.4\varphi^2)A_c + (0.33 + 0.62\varphi^{5/3})A_{c'}}{A_s}\right)$$ [A1]

where

C_{Nf} is the normal drag (pressure) coefficient for single frames equal to:
$1.58 + 1.05(0.6 - \varphi)^{1.8}$ for $\varphi \leq 0.6$;
$1.58 + 2.625(\varphi - 0.6)^2$ for $\varphi > 0.6$;

[A1] A_f and φ are as defined on Figure 7;

A_c and $A_{c'}$ are as defined in **4.2.2**; and

A_s is as defined in **4.2.1** [A1].

Annex M (normative)
Parameters for spectral analytical methods

M.1 Spectral response

M.1.1 *General*

Where it is necessary or desirable to calculate the resonant response more reliably than using the static response procedures described in **5.2** and **5.3**, the procedures and parameters described in **M.1.2**, **M.1.3**, **M.1.4** and **M.1.5** may be used.

M.1.2 *Total response*

The total response to wind loading may be taken to be built up from three distinct components of response as follows:

$$f_{\mathrm{T}} = \tau \pm \sqrt{f_{\mathrm{G}}{}^{2} + \sum_{r} f_{\mathrm{R}r}{}^{2}}$$

where

f_{T} is the peak total response;

τ is the mean response (see **M.1.3**) (equivalent to $\overline{F}_{\mathrm{M}}$ in **5.2.6**);

f_{G} is the peak gust response (see **M.1.4**);

$f_{\mathrm{R}r}$ is the peak resonant response in mode r (see **M.1.5**).

M.1.3 *Mean response*

The mean response should be calculated in accordance with **5.2.2**.

M.1.4 *Peak gust response*

The peak gust response should be calculated in accordance with **5.3** using λ values from Annex N and setting $\lambda_{\mathrm{R}} = 1.0$.

NOTE Influence line methods may be used as an alternative using, for example, the ESDU Wind Engineering Data Items [1] to obtain the necessary extra wind data.

M.1.5 *Resonant response*

The resonant response should be calculated using modal methods and the wind parameters of **M.2**. Modes of vibration should be calculated using the geometric stiffness and deflected shape associated with the mean wind loads.

M.2 Wind parameters for resonant response calculation

The following parameters may be used for calculating the response at the natural frequencies of the mast in the absence of more accurate site information. Alternatively parameters obtained from the ESDU Wind Engineering Data Items [1] may be used.

a) *Spectral density of wind velocity*. The power spectrum of wind energy at higher frequencies may be taken as

$$V_z\sqrt{nF_{\mathrm{u}z}} = 0.020\,\frac{z_0{}^{-0.02}\,\overline{V}_{\mathrm{s}}{}^{2.2}\,(1 + S_{\mathrm{h}}\varphi)^{1.33}}{n^{1/3}}$$

where

$\bar{V}_z$ is the hourly mean wind speed at height z;

nF_{uz} is the alongwind power spectral density at frequency n;

z_0 is the terrain roughness length taken

0.003 m over sea;

0.03 m over open country;

0.3 m over a long fetch of town terrain;

$\bar{V}_s$ is the hourly mean site wind speed defined in **3.1.3**;

S_h is the topography factor (see **3.2.4** and Annex H);

ψ is the hill slope (see Annex H).

In a transition region between different categories of terrain, it is only slightly conservative to use the lower value of z_0 in the equation above.

b) *Correlation*. The co-spectral power density may be taken as

$$R_{nsz} = \exp(-8ns/\bar{V}_z)$$

where s is the separation between the two points considered in the z direction.

$$L_{nz} = 2\bar{V}_z/8n = \int_0^{\infty} \exp(-8ns/\bar{V}_z)\mathrm{d}s$$

M.3 Damping

M.3.1 *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 **M.3.2**;

δ_a is the logarithmic decrement of aerodynamic damping, determined in accordance with **M.3.3**.

M.3.2 *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

$$\delta_s = \delta_m$$

where δ_m is the logarithmic decrement for the superstructure appropriate to the type of connection used, to be obtained from Table M.1.

M.3.3 *Aerodynamic damping*

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

$$\delta_a = \frac{\int_0^H \rho_a \bar{V}_z (\Sigma R_w/H_p)_z \mu_z^2 \mathrm{d}z}{2n_r M}$$

where

δ_a is the logarithmic decrement of aerodynamic damping;

ρ_a is the density of air;

ΣR_w is the total wind resistance of a panel;

H_p is the total height of a panel;

μ_z is the mode shape function;

M is the modal mass

$$= \int_0^H m_z \mu_z{}^2 \mathrm{d}z$$

n_r is the natural frequency of mode r;

m_z is the mass per unit height = M_p/H_p;

M_p is the mass of a panel;

H is the total height of the mast column.

In the crosswind direction the aerodynamic damping should be calculated using only the drag from lattice sections of the mast. The crosswind aerodynamic damping should then be taken as half the value given by the formula for the alongwind damping.

M.4 Modal response

The root mean square (r.m.s.) modal response factor γ_{Rr} may be calculated from

$$\gamma_{Rr} = \frac{\sqrt{\int_0^H (\rho_a \bar{V}_z \sqrt{n F_{uz}}\ (\Sigma R_w/H_p)_z \mu_z)^2 \frac{\pi^2}{2\ \delta} 2 L_{nz} \mathrm{d}z}}{(2\pi n_i)^2 M}$$

The r.m.s. modal response should be multiplied by a peak factor g_{Rr} given by

$$g_{Rr} = \sqrt{2\ln(3600 n_r) + 0.15}$$

The resonant response f_{Rr} in mode r should be obtained by multiplying the response corresponding to unit modal displacement by the r.m.s. modal response factor and by the modal peak factor.

Table M.1 — Logarithmic decrements of damping δ_m for superstructure

Nature of structural connections	Surface finish at connections	Logarithmic decrement δ_m
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	0.06
	Grit blasted, metal sprayed	0.045
	Galvanized	0.03
Black bolted bracing: bolted flange plate connection to legs	Cleaned, unpainted	0.04
	Grit blasted, metal sprayed	0.03
	Galvanized	0.02
Black bolted bracing: black bolted gusset connection to legs	Cleaned, unpainted	0.08
	Grit blasted, metal sprayed	0.06
	Galvanized	0.04

Annex N (informative) Parameters λ

N.1 General

The factor λ is made up of three components as follows:

$$\lambda = 4\lambda_B\lambda_R\lambda_{TL}$$

These parameters λ_B, λ_R and λ_{TL} may be taken conservatively as

$$\lambda_B = 0.75$$

$$\lambda_R = 1.2$$

$$\lambda_{TL} = 1.05$$

leading to

$$\lambda = 3.78$$

More accurate values for the parameters λ_B, λ_R and λ_{TL} may be determined as described in **N.2**, **N.3** and **N.4**, depending on the load effect being considered:

N.2 Background scaling factor λ_B

The background scaling factor λ_B may be determined as follows:

— for midspan moments

$$\lambda_B = 0.69\beta^{0.07}$$

— for deflections

$$\lambda_B = 0.84\beta^{0.05}$$

— for shears and support moments

$$\lambda_B = 0.70$$

where β is as defined in **5.1.2**b).

N.3 Resonant magnification factor λ_R

The resonant magnification factor λ_R may be determined as follows:

— for moments

$$\lambda_R = 0.34Q + 0.91$$

— for shears

$$\lambda_R = 0.30Q + 0.92$$

— for deflections

$$\lambda_R = 0.16Q + 0.95$$

where Q is as defined in **5.1.2**c).

In all cases λ_R should be greater than or equal to 1.

N.4 Turbulent length scale factor λ_{TL}

The turbulent length scale factor λ_{TL} may be determined as follows:

— for midspan moments

$\lambda_{TL} = (h_s/L_U)^{0.135}$

— for support moments

$\lambda_{TL} = 1.0$

— for shears at midspan

$\lambda_{TL} = 1.0$

— for shears at supports

$\lambda_{TL} = (h_S/L_U)^{-0.066}$

— for deflection

$\lambda_{TL} = (h_S/L_U)^{-0.165}$

where

h_s	is the average span between adjacent guy levels (in m);
L_U	is the vertical length scale of alongwind turbulence (in m) and is given by $L_U = 3.1h_e^{0.55}(z_0)_e^{-0.1}$
h_e	is the height above ground level of the area under consideration (in m);
$(z_0)_e$	is an effective terrain roughness length and is approximately given by $(z_0)_e = 0.1\log(1.585 + x)\log(1.41 + y)$
x	is the distance of the site from the sea (in km);
y	is the distance of the site into town (in km), if relevant, and is taken as zero for sites in the country.

Annex P (normative)
Loading for unsymmetric masts or masts with complex attachments

P.1 General

Loads other than in the wind direction can occur on unsymmetric masts or on unsymmetric aerials or ancillaries mounted on the mast. Examples of such loads are as follows:

a) crosswind or twisting steady state loads from the hourly mean speed, for example due to the lift coefficient or twisting moment coefficient of large aerials;

b) crosswind fluctuating loads on masts constructed of an unsymmetric cross section with respect to the wind direction, for example a triangular or four-sided mast with at least two faces of unequal length;

c) crosswind fluctuating loads on large aerials or other attachments unsymmetrically mounted with respect to the wind direction.

P.2 Steady state loads

The maximum mean lateral loads ($\bar{P}_{AX}$), vertical loads ($\bar{P}_{AZ}$) or twisting moments ($\bar{T}_{AW}$) on large discrete items attached to a mast, such as a dish aerial, should be taken as

$$\bar{P}_{AX} = \frac{\rho_a}{2}\bar{V}_{ZA}{}^2 R_{AX}$$

$$\bar{P}_{AZ} = \frac{\rho_a}{2}\bar{V}_{ZA}{}^2 R_{AZ}$$

$$\bar{T}_{AW} = \frac{\rho_a}{2}\bar{V}_{ZA}{}^2 (R_{AT} l_T)$$

where

$\bar{V}_{ZA}$ is the hourly mean wind speed at the height of the item, determined as $\bar{V}_z$ in accordance with **3.2**;

R_{AX} and R_{AZ} are the wind resistances of the item in the crosswind and vertical directions, determined from the lift coefficient for the item appropriate to the wind direction and wind speed, and should be obtained from wind tunnel tests;

$(R_{AT}l_T)$ is the torque wind resistance of the item with a lever arm of length l_T as determined from the appropriate twisting moment coefficient for the item appropriate to the wind direction and wind speed, and should be obtained from wind tunnel tests.

P.3 Fluctuating loads

The maximum fluctuating loads in the crosswind direction due to gusts should be derived from the in-line fluctuating loads, determined in accordance with **5.2** or **5.3** as appropriate, by multiplying the in-line fluctuating loads by

$$\frac{R_{AX}}{R_{AW}}$$

where

R_{AX} is as defined in **P.2**;

R_{AW} is the wind resistance of the item in the in-line wind direction determined in accordance with **4.3** and **4.4**.

P.4 Calculations of wind forces in mast members

The total load effect ΣF in a mast member due to wind loading is given by

$$\Sigma F = F_o + \bar{F}_M + F'$$

where

F_o is the load effect in the member due to still-air tensions in the guys and due to dead loads;

$\bar{F}_M$ is the maximum load effect in the member due to the mean wind loading on the mast and ancillaries determined in accordance with **5.2.1**, and including lateral and twisting loads calculated in accordance with **P.2** as appropriate;

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

$$F' = F'_M + F'_X$$

F'_M is the fluctuating force due to gusts in the downwind direction calculated in accordance with **5.2** or **5.3** as appropriate;

F'_X is the fluctuating force due to gusts in the crosswind direction calculated in accordance with **P.3**.

blank

List of references (see 1.2)

Normative references

BSI publications

BRITISH STANDARDS INSTITUTION, London

[A1] BS 8100-3, *Code of practice for strength assessment of members of lattice towers and masts.* [/A1]

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BS 8100-1:1986, *Code of practice for loading.*

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Other references

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