

Australian Standard[®]

**Design of steel lattice towers and
masts**

This Australian Standard was prepared by Committee BD/73, Design of Steel Lattice Towers and Masts. It was approved on behalf of the Council of Standards Australia on 20 April 1994 and published on 11 July 1994.

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Design of steel lattice towers and masts

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PREFACE

This Standard was prepared by the Standards Australia Committee on Design of Steel Lattice Towers and Masts to supersede AS 3995(Int)—1991, *Design of steel lattice towers and masts*.

In addition to those issues covered by the previous Interim Standard, this Standard now incorporates the following:

- (a) Design and analysis of guyed lattice towers and masts.
- (b) Design of cable tension members.
- (c) Footing design.
- (d) Criteria for analysis of existing structures.

Guidance relating to earthquake design, footing design, maintenance and access to steel lattice towers and masts is given in the Appendices.

Revisions have been made to the wind load specifications described in Section 2. These changes are the result of recent research on the effect of ancillaries on the wind load.

The design of cold-formed steel, other than those complying with AS 1163, *Structural steel hollow sections*, and AS 1664, *Rules for the use of aluminium in structures (known as the SAA Aluminium Structures Code)*, is not covered by this Standard.

The terms ‘normative’ and ‘informative’ have been used in this Standard to define the application of the appendix to which they apply. A ‘normative’ appendix is an integral part of a Standard, whereas an ‘informative’ appendix is only for information and guidance.

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STANDARDS AUSTRALIA

Australian Standard

Design of steel lattice towers and masts

SECTION 1 SCOPE AND GENERAL

1.1 SCOPE This Standard sets out the procedures for determination of design wind speeds and wind loads, and other appropriate standards to be used in the structural design of steel lattice towers and masts, with or without ancillaries such as antennas, for communication purposes. It also applies to other lattice towers and masts where the predominant load is wind load on the structure. It further sets out the basis for the strength assessment of members and connections of lattice towers and masts.

This Standard is not intended to apply to the structural design of transmission line structures. The design of cold-formed steel, other than those complying with AS 1163, and aluminium is not covered by this Standard.

For all other aspects of design not specifically mentioned herein, reference shall be made to the appropriate Australian Standards including AS 1170 Parts 1 and 2, AS 1554, AS 1559, AS 1650, AS 3569 and AS 4100.

NOTES:

- 1 A general framework for maintenance and inspection of existing structures is given in Appendix A.
- 2 Recommendations for access to steel lattice towers and masts are given in Appendix B. If required by the Health and Safety Authority, provision of safe access to such structures should be considered at the design stage.

1.2 REFERENCED DOCUMENTS The following documents are referred to in this Standard:

AS

1163	Structural steel hollow sections
1170	SAA Loading Code
1170.1	Part 1: Dead and live loads and load combinations
1170.2	Part 2: Wind loads
1170.3	Part 3: Snow loads
1170.4	Part 4: Earthquake loads
1538	Cold-formed Steel Structures Code
1554	SAA Structural Steel Welding Code
1559	Fasteners—Bolts, nuts and washers for tower construction
1650	Hot-dipped galvanized coatings on ferrous articles
1657	Fixed platforms, walkways, stairways and ladders—Design, construction and installation
1664	SAA Aluminium Structures Code
2759	Steel wire rope—Application guide
2772	Radiofrequency radiation
2841	Galvanized steel wire strand

AS

- 3569 Steel wire ropes
 3679 Structural steel
 3679.1 Part 1: Hot-rolled bars and sections
 4100 Steel structures

BS

- 8100 Lattice towers and masts
 8100.1 Part 1: Code of practice for loading
 8100.2 Part 2: Guide to the background and use of Part 1 'Code of practice for loading'

1.3 DEFINITIONS For the purpose of this Standard, the definitions below apply.

1.3.1 Bracing members—members other than legs carrying the horizontal forces due to the imposed loads on the structure.

1.3.2 Leg members—members forming the main load-bearing components of the structure.

1.3.3 Linear ancillaries—ancillaries to the structure that are very long in relation to their sectional dimensions, and for which sectional drag force coefficients are available.

1.3.4 Secondary bracing members—members used to reduce the effective length of other members.

1.4 NOTATION Symbols used in this Standard are given in Table 1.4.

Unless specified otherwise, expressions and equations in this Standard are such that any consistent set of dimensional units may be used.

NOTES:

- 1 In Section 2, the typical set of units to be used for length, force and pressure are metres (m), kilonewtons (kN) and kilopascals (kPa) respectively.
- 2 In Section 3, the typical set of units to be used for length, force and stress are millimetres (mm), newtons (N) and megapascals (MPa) respectively.
- 3 In Appendix D, the typical set of units to be used for length, mass and force are metres (m), kilograms (kg) and newtons (N) respectively.

TABLE 1.4
NOTATION

Quantity symbol	Term	Text reference
A	area of cross-section	Clause 3.3.1
A_a	reference area of any ancillaries attached to a tower section	Clause 2.2.8.3
A_e	effective compression section area	Clauses 3.3.2, 3.3.3
A_n	net area of a cross-section	Clause 3.4.2.2
A_o	nominal plain shank area of a bolt	Clause 3.5.4.2
A_z	projected area of tower members in one face of a tower section, without ancillaries (except for Case (a) in Clause 2.2.8.3)	Clauses 2.2.6, 2.2.8.2, 2.2.8.3
a	constant in expression for K_{in} for cylindrical ancillary inside a square tower	Clause 2.2.8.4

(continued)

TABLE 1.4 (continued)

Quantity symbol	Term	Text reference
B_s	background factor	Clauses 2.3.8, Paragraph E7
b	average diameter or breadth of a section of an ancillary or tower member	Clauses 2.2.8.2, 2.2.8.4, Appendix F
C	distance between stitch bolts	Clause 3.3.5
C_d	drag force coefficient for the tower section without ancillaries	Clauses 2.2.6, 2.2.8.1, 2.2.8.2
C_{da}	drag force coefficient of the isolated ancillary	Clauses 2.2.8.2, 2.2.8.3, Paragraph E5, Appendix F
C_{de}	effective drag force coefficient for the tower section with ancillaries	Clauses 2.2.6, 2.2.8.3, 2.3.5.2
c	constant in expression for K_{in} for cylindrical ancillary inside a triangular tower	Clause 2.2.8.4
D_c	diameter of the cable	Clause 2.2.6
d_f	nominal diameter of a bolt	Clause 3.5.4.5
E	gust energy factor; or Young's modulus of elasticity (200×10^3)	Clause 2.3.8 Clause 3.3.2
F_d	drag force acting parallel to the wind stream	Clause 2.2.6
f_u	tensile strength used in design	Clause 3.4.2.2
f_{vf}	minimum shear strength of a bolt	Clause 3.5.4.2
f_y	yield stress of steel; or specified yield stress of a ply	Clauses 3.3.1, 3.5.4.5
G	dead load	Clauses 1.6.1, 1.6.5
G_s	gust response factor	Clauses 2.3.5.2, 2.3.8
g_B	peak factor for the background response	Clause 2.3.8
g_R	peak factor for resonant response	Clause 2.3.8
H	height of a hill, ridge or escarpment; or height factor	Clauses 2.2.4, 2.3.4, 2.3.8, Paragraph E7
h	height of a structure above ground	Clause 2.3.8, Appendix D
I	ice load	Clauses 1.6.3, 1.6.5
K_{ar}	aspect ratio correction factor	Clauses 2.2.8.2, 2.2.8.3, Paragraph F5
K_{in}	correction factor for interference	Clauses 2.2.8.3, 2.2.8.4
K_r	reduction factor to account for the length of a bolted lap connection	Clause 3.5.4.2, Paragraph G5.1
K_t	correction factor for the distribution of force in a tension member	Clauses 3.4.2.2, 3.4.2.3
k_f	form factor for members subject to axial compression	Clause 3.3.2
k_1, k_N	minimum stiffness for the first and last guy cluster systems on the mast for any wind direction	Appendix D

(continued)

TABLE 1.4 (continued)

Quantity symbol	Term	Text reference
L	length used to determine slenderness ratio of member	Clauses 3.2.3, 3.3.4.1, Paragraph H3
L_c	chord length of the cable	Clause 2.2.6
L_g	length to determine the topographic multiplier	Clauses 2.2.4, 2.3.4
L_h	measure of the effective turbulence length scale	Clause 2.3.8
L_j	length of a bolted lap connection	Clause 3.5.4.2
L_m	length used in the topographic multiplier	Clause 2.3.4
L_u	horizontal distance upwind from the crest of a hill, ridge or escarpment to a level half the height below the crest	Clauses 2.2.4, 2.3.4
l	length of the linear ancillary; or actual unsupported length of a member	Clauses 2.2.8.3, 3.3.4.1, Paragraph H3
l_1, l_2, l_3, l_4	length of member segment	Figure H3
M_{aj}	individual mass of ancillaries	Appendix D
M_d	wind direction multiplier	Clauses 2.2.2, 2.2.5, 2.3.2
M_s	design bending moment at height s	Clause 2.3.5.2
M_T	total mass of the mast section and half the total mass of all guy clusters attached to it	Appendix D
M_t	topographic multiplier for gust wind speeds	Clauses 2.2.2, 2.2.4
$M_{(z, cat)}$	gust wind speed multiplier for a terrain category at height z	Clauses 2.2.2, 2.2.3.2
$\bar{M}_s$	mean bending moment at height s	Clause 2.3.5.2
$\bar{M}_t$	topographic multiplier for mean wind speeds	Clause 2.3.2, 2.3.4, 2.3.8
$\bar{M}_{(z, cat)}$	mean wind speed multiplier for a terrain category at height z	Clauses 2.3.2, 2.3.3.2
M_1	generalized mass of the structure	Appendix D
M_2	total mass of the structure	Appendix D
m	added lumped mass	Appendix D
N	effective reduced frequency; or number of guy levels	Clause 2.3.8, Appendix D
N_A	number of ancillaries	Appendix D
N_c	nominal capacity of a member in compression	Clause 3.3.1
N_t	nominal capacity of a member in tension	Clause 3.4.2.2
N_c^*	design axial compression force	Clause 3.3.1, Paragraph H3
N_t^*	design axial tension force	Clause 3.4.2.1, Paragraph H3
n	first mode natural sway frequency of the structure without lumped mass	Clause 2.3.8, Appendix D
n_x	number of shear planes without threads intercepting the shear plane	Clause 3.5.4.2

(continued)

TABLE 1.4 (continued)

Quantity symbol	Term	Text reference
n_1	first mode natural sway frequency for structures with a lumped mass near the top	Appendix D
Q_s	design peak shear force at height s	Clause 2.3.5.2
$\bar{Q}_s$	mean shear force at height s	Clause 2.3.5.2
q_z	free-stream gust dynamic wind pressure at height z	Clauses 2.2.6, 2.2.7
$\bar{q}_z$	free-stream mean dynamic wind pressure at height z	Clause 2.3.5.2
R_u	nominal capacity	Clause 1.8
r	roughness factor, equal to twice the longitudinal turbulence intensity at height h ; or radius of gyration	Clauses 2.3.8, 3.2.3
$r_{\min.}$	minimum radius of gyration of a section	Clause 3.3.5
S	size factor for resonant response	Clause 2.3.8
S^*	design load effect	Clauses 1.8, 3.5.1
s	height of the design peak shear force and design bending moment (M_s) above ground level; or height of the mean shear force ($\bar{Q}_s$) and mean bending moment ($\bar{M}_s$) above ground level	Clause 2.3.8
T	averaging time	Clause 2.3.8
t	thickness of angle leg	Clause 3.3.3
t_p	thickness of a ply	Clause 3.5.4.5
V	basic wind speed	Clauses 2.2.2, 2.3.2
V_b	nominal bearing capacity of a ply	Clause 3.5.4.5
V_f	nominal shear capacity of a bolt	Clause 3.5.4.2
V_s	basic wind speed for serviceability limit state	Clauses 1.6.2, 2.2.2, 2.2.3.1, 2.3.2
V_u	basic wind speed for ultimate limit state	Clauses 1.6.2, 2.2.2, 2.2.3.1, 2.3.2
V_z	design gust wind speed at height z	Clauses 2.2.1, 2.2.2, 2.2.7
$\bar{V}_h$	mean wind speed calculated for z equal to h	Clause 2.3.8
$\bar{V}_z$	design mean wind speed at height z	Clauses 2.3.1, 2.3.2, Appendix D
$V_b^\bullet$	design bearing force	Clause 3.5.4.5
$V_f^\bullet$	design shear force	Clause 3.5.4.2
W_u	wind load for strength and stability limit state	Clauses 1.6.2, 1.6.5
W_{u1}	resultant wind load on tower section over height h_1	Figure 2.1
W_{u2}	resultant wind load on tower section over height h_2	Figure 2.1

(continued)

TABLE 1.4 (continued)

Quantity symbol	Term	Text reference
W_s	wind load for serviceability limit state	Clauses 1.6.2, 1.9
w	average width of a section of a structure over the length of the cylindrical ancillary; <i>or</i> flat width from root of fillet	Clauses 2.2.8.4, 3.3.3
w_a	average width of the structure	Appendix D
w_b	width at the base of the structure	Appendix D
w_o	average width of the structure between $h/2$ and h	Clause 2.3.8
w_s	average width of the structure between h and s	Clause 2.3.8
x	horizontal distance upwind or downwind from the structure to the crest of a hill or a ridge	Clauses 2.2.4, 2.3.4
y_j	heights of ancillaries above ground	Appendix D
z	height of the centroid of the tower section above ground level	Clauses 2.2.2, 2.2.3.2, 2.2.4, 2.2.6, 2.3.4
α_c	member slenderness reduction factor	Clauses 3.3.1 3.3.2
γ_i	ultimate ice load factor	Clause 1.6.5
γ_w	importance factor	Clause 1.6.5
ΔC_d	additional drag force coefficient due to an ancillary	Clause 2.2.8.3
δ	solidity ratio	Clauses 2.2.8.2, 2.2.8.4, 2.3.6
θ_a	angle of deviation of the wind stream from the normal of the ancillary	Clause 2.2.8.4, Paragraph F6
θ_g	angle of wind stream relative to the axis of the guy cable	Clause 2.2.6
$\bar{\psi}$	mean deflection or rotation at the same probability level	Clause 2.3.7
λ_c	factor for the determination of α_c	Clause 3.3.2
λ_e	effective slenderness ratio	Clauses 3.2.3, 3.3.2, 3.3.4.1, 3.3.4.2, 3.3.5, Paragraph G4
λ_1	slenderness ratio of the compound member assuming full composite action	Clause 3.3.5
λ_2	slenderness ratio of one component angle	Clause 3.3.5
λ_3	slenderness ratio of a segment of the mast between support levels	Clause 3.3.6
λ_4	slenderness ratio of the individual leg members	Clause 3.3.6
ζ	critical damping ratio	Clause 2.3.8
$\sigma_v/\bar{V}$	turbulence intensity	Clause 2.3.8
ϕ	upwind slope; <i>or</i> capacity factor	Clauses 1.8, 2.2.4, 2.3.4, 3.3.1, 3.4.2.1, 3.5.4.2, 3.5.4.5
ϕR_u	design capacity	Clause 1.8

1.5 TYPE OF STRUCTURE A structure shall be classified as Type I, II or III taking into account variations in the risks to life, and in the social or economic losses which may ensue if failure occurs.

1.5.1 Type I A structure shall be classified as Type I where—

- (a) the structure is designed to provide vital post-disaster communications services; or
- (b) the collapse of the structure and loss of the services provided causes unacceptable danger to life or extensive economic loss.

1.5.2 Type II A structure may be classified as Type II where—

- (a) the danger to life in case of collapse may be negligible and adequate warning arrangements are incorporated to ensure the general public is not unduly endangered; and
- (b) the loss of the services provided is not critical, e.g. where alternative means of communication can be provided.

1.5.3 Type III A structure may be classified as Type III where all consequences of failure are more tolerable than those specified in Clause 1.5.2.

1.6 LOADING The following loads shall be considered in the design of lattice towers and masts, and their structural members and connections:

- (a) Dead loads (G).
- (b) Wind loads (W_u , W_s).
- (c) Ice loads (I).
- (d) Earthquake loads.
- (e) Incidental loads.

1.6.1 Dead loads (G) The dead loads of the structure shall consist of the following:

- (a) The weight of the structure.
- (b) The weight of all attachments, such as antennas.

1.6.2 Wind loads (W_u , W_s) The wind loads on the structure shall be assessed in accordance with Section 2.

The strength and stability limit state wind loads (W_u) shall be calculated using the basic wind speed for the ultimate limit state (V_u).

The serviceability limit state wind loads (W_s) shall be calculated using the basic wind speed for the serviceability limit state (V_s).

NOTE: Wind loads may be affected by ice.

1.6.3 Ice loads (I) The weight of ice loads, if applicable, shall be calculated by an appropriate method taking into account local conditions.

NOTE: For regions where ice loads may need to be considered, see AS 1170.3.

1.6.4 Earthquake loads Earthquake loads shall be considered if applicable.

NOTE: For general guidance on earthquake design, see Appendix C.

1.6.5 Load combinations Design loads for the strength and stability limit states shall be the combinations of factored loads which produce the most adverse effect on the structure, as determined from, but not limited to, the following:

- (a) $1.25G + \gamma_w W_u$
- (b) $1.25G + 0.5\gamma_w W_u + \gamma_i I$ (see Note 1)
- (c) $0.8G + \gamma_w W_u$

where

γ_w = importance factor given in Table 1.6.5

γ_i = ultimate ice load factor (see Note 2)

NOTES:

- 1 Wind load in Item (b) should include the shape effects of ice formation.
- 2 If I (see Clause 1.6.3) is the 50-year return period ice load, then γ_i should be taken as 1.5.

TABLE 1.6.5
IMPORTANCE FACTOR

Structure type	Importance factor (γ_w)
I	1.00
II	0.85
III	0.70

1.6.6 Incidental loads on members All members that form an angle of less than 30° to the horizontal shall be designed to support a concentrated vertical load of 1.7 kN at midspan. This load shall be applied independently of wind and ice loads.

1.7 STABILITY LIMIT STATE The structure as a whole and any part of it shall be designed to prevent instability due to overturning, uplift or sliding under the loads determined in accordance with Clause 1.6.

1.8 STRENGTH LIMIT STATE The structure and its component members and connections shall be designed for the strength limit state as follows:

- (a) The strength limit state design loads shall be determined in accordance with Clause 1.6.
- (b) The design load effects (S^*) resulting from the strength limit state design loads shall be determined in accordance with Clause 3.1.
- (c) The design capacity (ϕR_u) shall be determined from the nominal capacity (R_u) determined from Section 3, where the capacity factor (ϕ) shall not exceed the appropriate value given in Table 1.8.
- (d) All members and connections shall be proportioned so that the design capacity (ϕR_u) is not less than the design load effect (S^*), i.e.—

$$S^* \leq \phi R_u \quad \dots 1.8$$

1.9 SERVICEABILITY LIMIT STATE The structure and its components shall be designed for serviceability limit state by limiting deflection and rotations under the action of serviceability limit state wind loads (W_s) appropriate for the limit states under consideration.

Serviceability limits shall be assessed for each structure, dependent on the type of service provided from the structure.

TABLE 1.8
CAPACITY FACTORS (ϕ) FOR STRENGTH LIMIT STATES

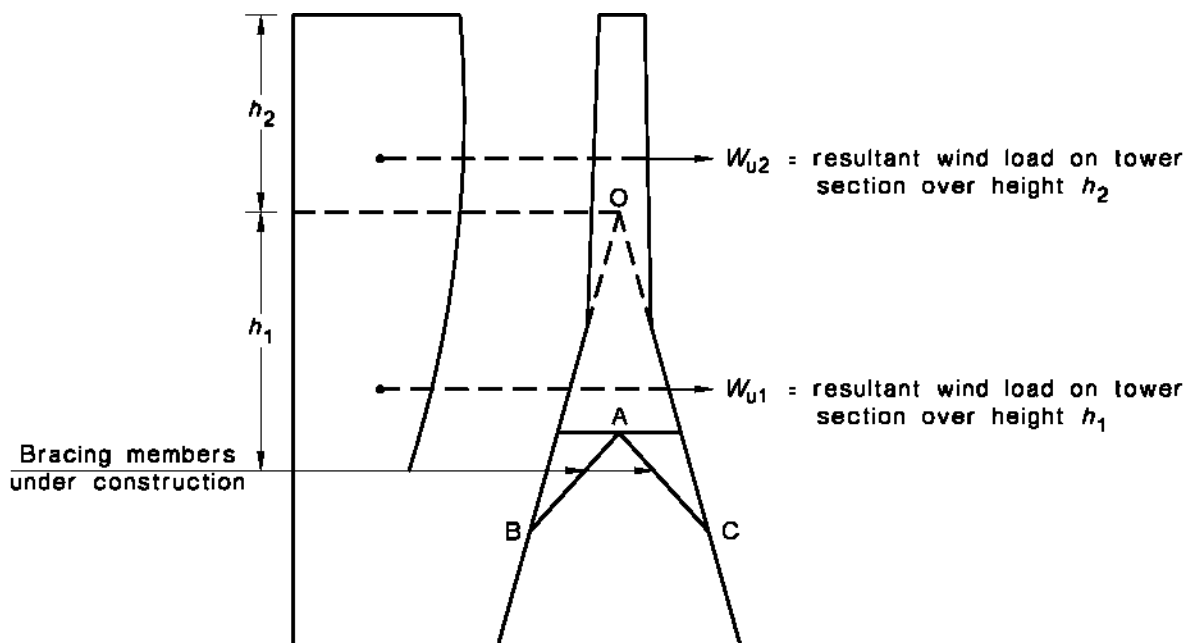
Design capacity for	Capacity factor (ϕ)
Member subject to bending	0.9
Member subject to axial compression	0.9
Member subject to axial tension	0.9
Member subject to combined actions	0.9
Cable member subject to axial tension	0.6
Connection component other than a bolt, pin or weld	0.9
Bolted connection:	
tower bolt in shear	0.9
other bolt in shear	0.8
bolt in tension	0.8
bolt subject to combined shear and tension	0.9
ply in bearing	0.9
Pin connection:	
pin in shear	0.8
pin in bearing	0.8
pin in bending	0.8
ply in bearing	0.9
Welded connection (SP Category):	
complete penetration butt weld	0.9
fillet weld and incomplete penetration butt weld	0.8
plug or slot weld	0.8
weld group	0.8

SECTION 2 WIND LOAD SPECIFICATIONS

2.1 DESIGN PROCEDURES For the determination of wind loads on lattice towers and masts, detailed procedures are given in this Section. These towers and masts vary from those less sensitive to wind action, to those in which dynamic response shall be taken into consideration. *The dynamic analysis shall be undertaken for those towers and masts having a first mode natural frequency less than 1 Hz. For structures with a first mode natural frequency greater than or equal to 1 Hz, static or dynamic analysis may be used.*

NOTES:

- 1 In this Section, reference to 'tower sections' should be taken to mean 'sections of towers and masts'.
- 2 For an eiffelated tower, the effect of temporal fluctuations in wind load should be considered (see Figure 2.1). For further guidance on bracing members in eiffelated towers with dynamic analysis, see BS 8100: Part 2.
- 3 Method of estimation of the first mode natural frequency of vibration is given in Appendix D.



NOTE: The forces in members AB and AC are sensitive to fluctuations in wind load when the total resultant wind load on tower sections over height $(h_1 + h_2)$ acts close to point O. Consequently, the following load cases should be considered:

- (a) $W_{u1} + W_{u2}$
- (b) $0.5W_{u1} + W_{u2}$
- (c) $W_{u1} + 0.5W_{u2}$

FIGURE 2.1 BRACING MEMBER FORCES IN EIFFELATED TOWERS

2.2 STATIC ANALYSIS

2.2.1 General The design gust wind speed (V_z) at height z shall be used to determine wind loads on a structure or part of a structure with the static analysis procedure set out in Clause 2.2.

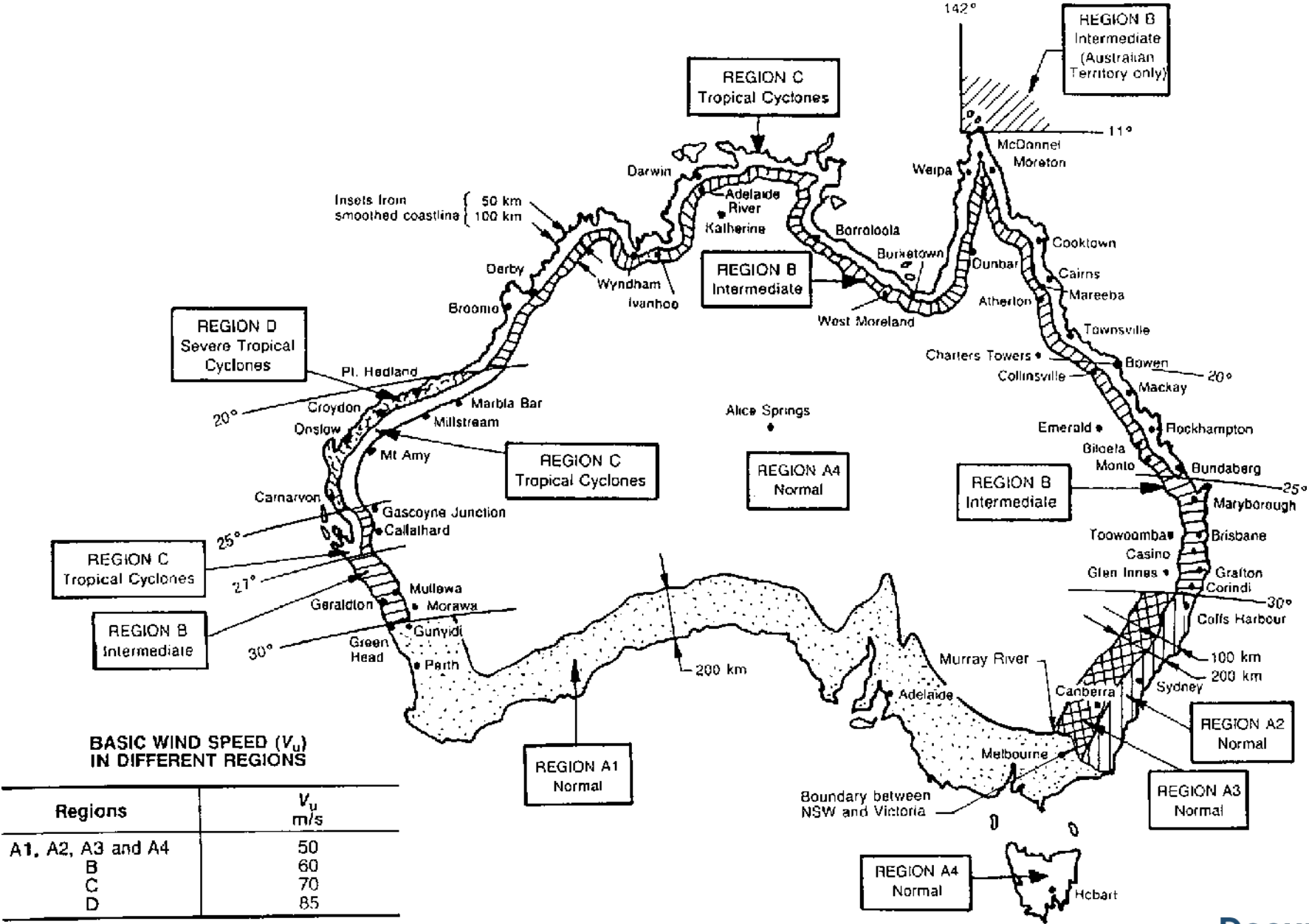


FIGURE 2.2 BOUNDARIES OF REGIONS A1, A2, A3, A4, B, C AND D
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2.2.2 Gust wind speed The design gust wind speed (V_z) shall be determined from the basic wind speed (V) for the appropriate limit state given as follows:

$$V_z = VM_{(z, cat)} M_t M_d \quad \dots 2.2.2$$

where

V = basic wind speed (V_u) for ultimate limit state (see Figure 2.2) and basic wind speed (V_s) for serviceability limit state

$M_{(z, cat)}$ = gust wind speed multiplier for a terrain category at height z (see Clause 2.2.3.2)

z = height of the centroid of the tower section above ground level

M_t = topographic multiplier for gust wind speeds (see Clause 2.2.4)

M_d = wind direction multiplier (see Clause 2.2.5)

NOTES:

- 1 For background information on V_u and V_s , see Paragraph E1 of Appendix E.
- 2 A shielding multiplier has not been included in Equation 2.2.2. If there are significant effects of shielding, see AS 1170.2.

2.2.3 Terrain and height multiplier ($M_{(z, cat)}$)

2.2.3.1 Terrain category Unless a detailed analysis of changes in terrain category as specified in AS 1170.2 is used, then the relevant terrain category shall be the lowest terrain category for the area within a distance of 10 times the height of the structure and extending to 20 times the height of the structure or 1 km whichever is greater, in the upwind direction (see Figure 2.2.3.1). Terrain, over which the approach wind flows towards a structure, shall be assessed on the basis of the following category descriptions:

- (a) *Category 1*—exposed open terrain with few or no obstructions and water surfaces at serviceability wind speeds (V_s) only.
- (b) *Category 2*—open terrain, grassland with few well-scattered obstructions having heights from 1.5 m to 10 m and water surfaces at wind speeds (V_u).
- (c) *Category 3*—terrain with numerous closely-spaced obstructions having the size of domestic houses (3 m to 5 m high).
- (d) *Category 4*—terrain with numerous large, high (10 m to 30 m high) and closely-spaced obstructions such as large city centres and well-developed industrial complexes.

NOTE: For background information on $M_{(z, cat)}$, see Paragraph E2 of Appendix E.

2.2.3.2 Terrain multiplier The variation of terrain multipliers with height (z) shall be taken from Tables 2.2.3.2(1) and 2.2.3.2(2), as appropriate.

2.2.4 Topographic multiplier The topographic multiplier (M_t) for gust wind speeds shall be obtained for sites at or near the crest of a hill, ridge or escarpment from the following equation:

$$M_t = 1 + \frac{H \left(1 - \frac{|x|}{4L_g} \right)}{3.5(z + L_g)} \quad \dots 2.2.4$$

where

H = height of the hill, ridge or escarpment

x = horizontal distance upwind or downwind from the structure to the crest of a hill or a ridge

L_g = length to determine the topographic multiplier
 $= 0.4H$ or $0.35L_u$, whichever is greater

L_u = horizontal distance upwind from the crest of a hill, ridge or escarpment to a level half the height below the crest

z = height of the centroid of the tower section above ground level

NOTES:

1 The above dimensions are calculated for a vertical section of the hill, ridge or escarpment in the wind direction under consideration (see Figures 2.2.4(1) and 2.2.4(2)).

2 For background information on M_p , see Paragraph E3 of Appendix E.

A topographic multiplier of $M_t = 1$ shall be used for all sites where $|x| > 4L_g$ or if the upwind slope (ϕ) is less than 0.05.

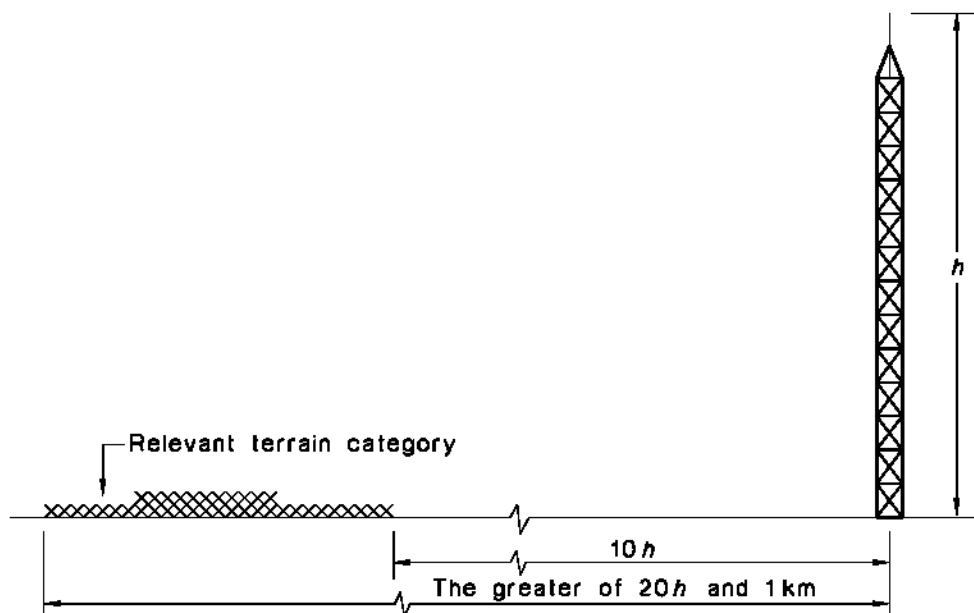


FIGURE 2.2.3.1 TERRAIN CATEGORY SELECTION

2.2.5 Wind direction multiplier The design gust wind speed may be adjusted with a wind direction multiplier (M_d) given in Table 2.2.5.

As an alternative, a detailed probability analysis to allow for the directional effects of wind is permitted.

The following wind directions providing the worst loading shall be considered:

- (a) Wind direction perpendicular to the tower face.
- (b) Wind direction making equal angles to adjacent tower faces.

NOTE: For background information on M_d , see Paragraph E4 of Appendix E.

TABLE 2.2.3.2(1)

TERRAIN HEIGHT MULTIPLIER ($M_{(z, \text{cat})}$) FOR GUST WIND SPEEDS IN FULLY-DEVELOPED TERRAIN, ULTIMATE LIMIT STATE DESIGN FOR REGIONS A1, A2, A3, A4 AND B ONLY, AND SERVICEABILITY LIMIT STATE DESIGN FOR ALL REGIONS

Height (z) m	Terrain height multiplier ($M_{(z, \text{cat})}$)			
	Terrain Category 1	Terrain Category 2	Terrain Category 3	Terrain Category 4
≤3	0.99	0.85	0.75	0.75
5	1.05	0.91	0.75	0.75
10	1.12	1.00	0.83	0.75
15	1.16	1.05	0.89	0.75
20	1.19	1.08	0.94	0.75
30	1.22	1.12	1.00	0.80
40	1.24	1.16	1.04	0.85
50	1.25	1.18	1.07	0.90
75	1.27	1.22	1.12	0.98
100	1.29	1.24	1.16	1.03
150	1.31	1.27	1.21	1.11
200	1.32	1.29	1.24	1.16
250	1.34	1.31	1.27	1.20
300	1.35	1.32	1.29	1.23
400	1.37	1.35	1.32	1.28
500	1.38	1.37	1.35	1.31

NOTE: For intermediate values of height (z) and terrain category, interpolation is permitted.

TABLE 2.2.3.2(2)

TERRAIN HEIGHT MULTIPLIER ($M_{(z, \text{cat})}$) FOR GUST WIND SPEEDS IN FULLY-DEVELOPED TERRAIN, ULTIMATE LIMIT STATE DESIGN FOR REGIONS C AND D ONLY

Height (z) m	Terrain height multiplier ($M_{(z, \text{cat})}$)	
	Terrain Categories 1 and 2	Terrain Categories 3 and 4
≤3	0.90	0.80
5	0.95	0.80
10	1.00	0.89
15	1.07	0.95
20	1.13	1.05
30	1.20	1.15
40	1.25	1.25
50	1.29	1.29
75	1.35	1.35
≥100	1.40	1.40

NOTE: For intermediate values of height (z) and terrain category, interpolation is permitted.

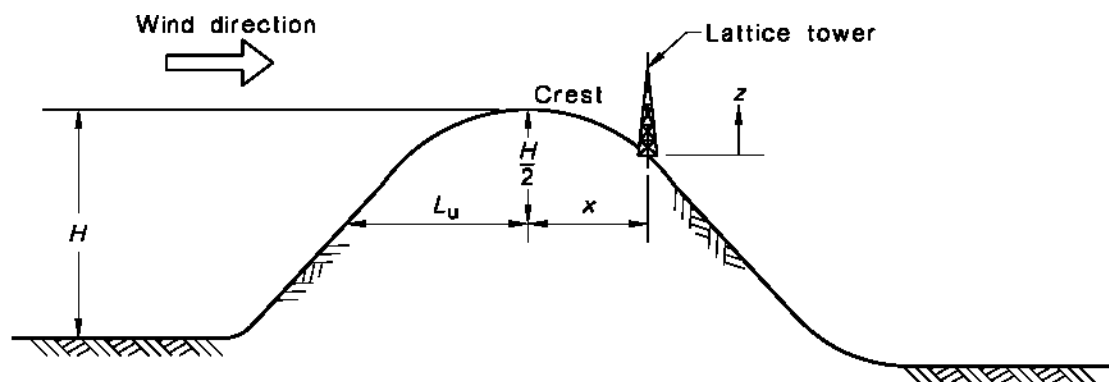


FIGURE 2.2.4(1) HILLS AND RIDGES

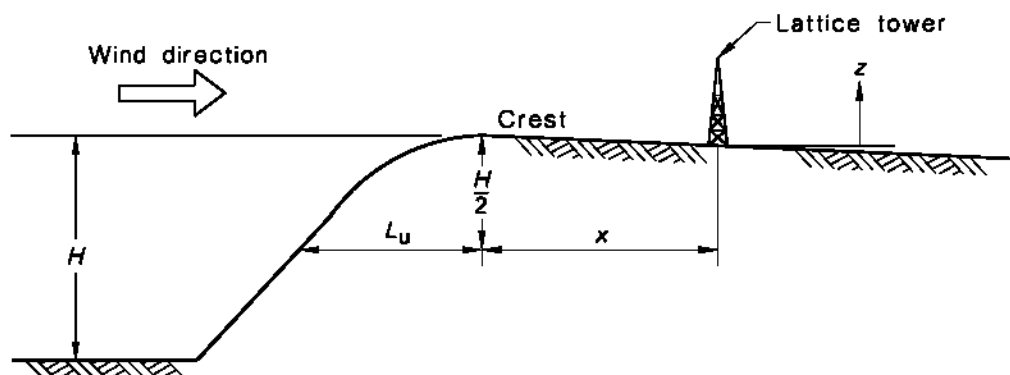


FIGURE 2.2.4(2) ESCARPMENTS

TABLE 2.2.5
WIND DIRECTION MULTIPLIER (M_d)

Wind direction	Wind direction multiplier (M_d)			
	Region A1	Region A2	Region A3	Regions A4, B, C and D
NE	0.80	0.80	0.80	0.95
E	0.80	0.80	0.80	0.95
SE	0.80	0.95	0.80	0.95
S	0.85	0.90	0.80	0.95
SW	0.95	0.95	0.85	0.95
W	1.00	1.00	0.90	0.95
NW	0.95	0.95	1.00	0.95
N	0.90	0.80	0.85	0.95

2.2.6 Calculation of drag forces For the purpose of calculating drag forces, a tower shall be divided into a series of sections.

NOTE: A minimum of 10 sections should be used where possible.

Drag forces (F_d) shall be calculated as follows:

- (a) For a tower section without ancillaries:

$$F_d = C_d A_z q_z \quad \dots 2.2.6(1)$$

- (b) For a tower section with ancillaries:

$$F_d = C_{de} A_z q_z \quad \dots 2.2.6(2)$$

- (c) For guy cables:

$$F_d = 1.2 q_z D_c L_c \sin^2 \theta_g \quad \dots 2.2.6(3)$$

q_z shall be taken at 2/3 of the height of the cable.

where

C_d = drag force coefficient for the tower section without ancillaries (see Clause 2.2.8.2)

C_{de} = effective drag force coefficient for the tower section with ancillaries (see Clause 2.2.8.3)

A_z = projected area of tower members in one face of a tower section, without ancillaries (except for Item (a) in Clause 2.2.8.3)

q_z = free-stream gust dynamic wind pressure at height z (see Clause 2.2.7)

z = height of the centroid of the tower section above ground level

D_c = diameter of the cable

L_c = chord length of the cable

θ_g = angle of wind stream relative to the axis of the guy cable

2.2.7 Dynamic wind pressure The free-stream gust dynamic wind pressure (q_z) at height z shall be calculated as follows:

$$q_z = 0.6 \times 10^{-3} V_z^2 \quad \dots 2.2.7$$

where

q_z = free-stream gust dynamic wind pressure at height z , in kilopascals

V_z = design gust wind speed at height z , in metres per second

2.2.8 Drag force coefficient

2.2.8.1 General Drag force coefficients (C_d) shall be derived from either—

- the coefficients given in this Clause 2.2.8; or
- tests of full-scale components or models in wind tunnels, either in smooth flow or in scaled turbulent flow.

2.2.8.2 Tower sections without ancillaries The drag force coefficients (C_d) for complete lattice tower sections shall be taken from Tables 2.2.8.2(1) to 2.2.8.2(3).

In these tables, the solidity (δ) of the front face of a tower section shall be taken as the ratio of the total projected area (A_z), to the projected area enclosed over the section height by the boundaries of the frame.

For equilateral-triangle lattice towers with flat-sided members, the drag force coefficient (C_d) shall be assumed to be constant for any inclination of the wind to a face.

For complete clad tower sections, C_d shall be taken as the value of C_{da} for the appropriate sections given in Tables F1 and F2, and Figure F1. For UHF antenna sections, C_d shall be obtained from Figure 2.2.8.2.