AS 3995—1994

Australian Standard®

Design of steel lattice towers and masts

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Bureau of Steel Manufacturers of Australia

CSIRO, Division of Building, Construction and Engineering

Electricity Supply Association of Australia

Electricity Trust of South Australia

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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.

CONTENTS

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^*	design bearing force	Clause 3.5.4.5
V_f^*	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; *or* 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; *or* 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.—

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

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

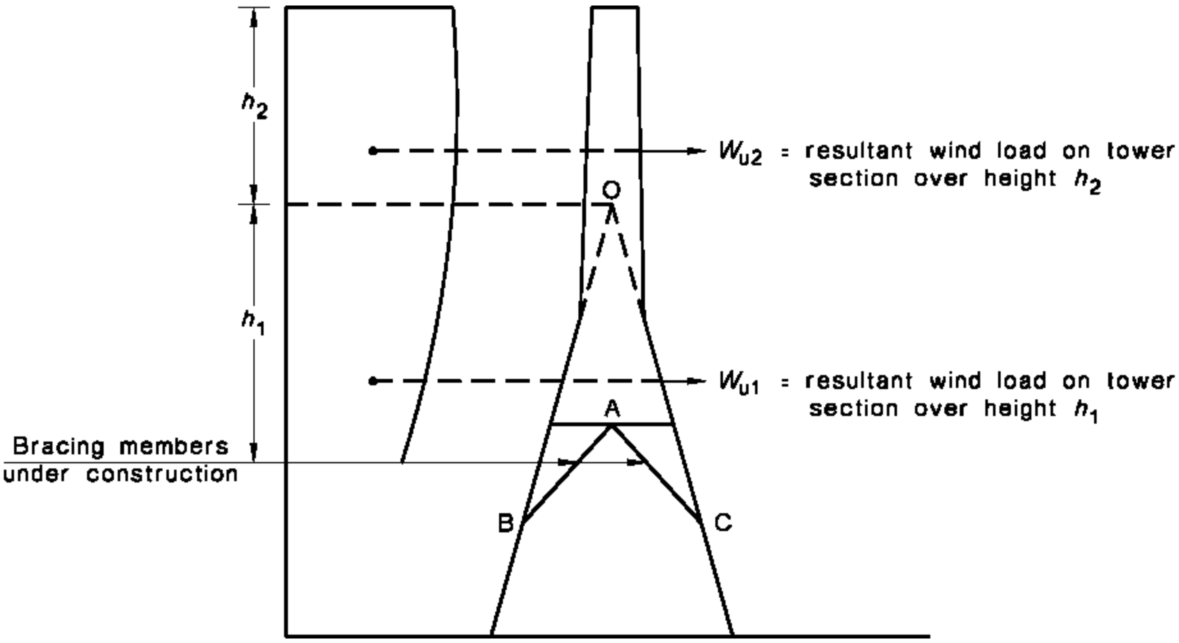
2.1 DESIGN PROCEDURES For the determination of wind loads on lattice towers and masts, detailed procedures are given in this Section. These towers and masts vary from those less sensitive to wind action, to those in which dynamic response shall be taken into consideration. *The dynamic analysis shall 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) $W_{u1} + W_{u2}$

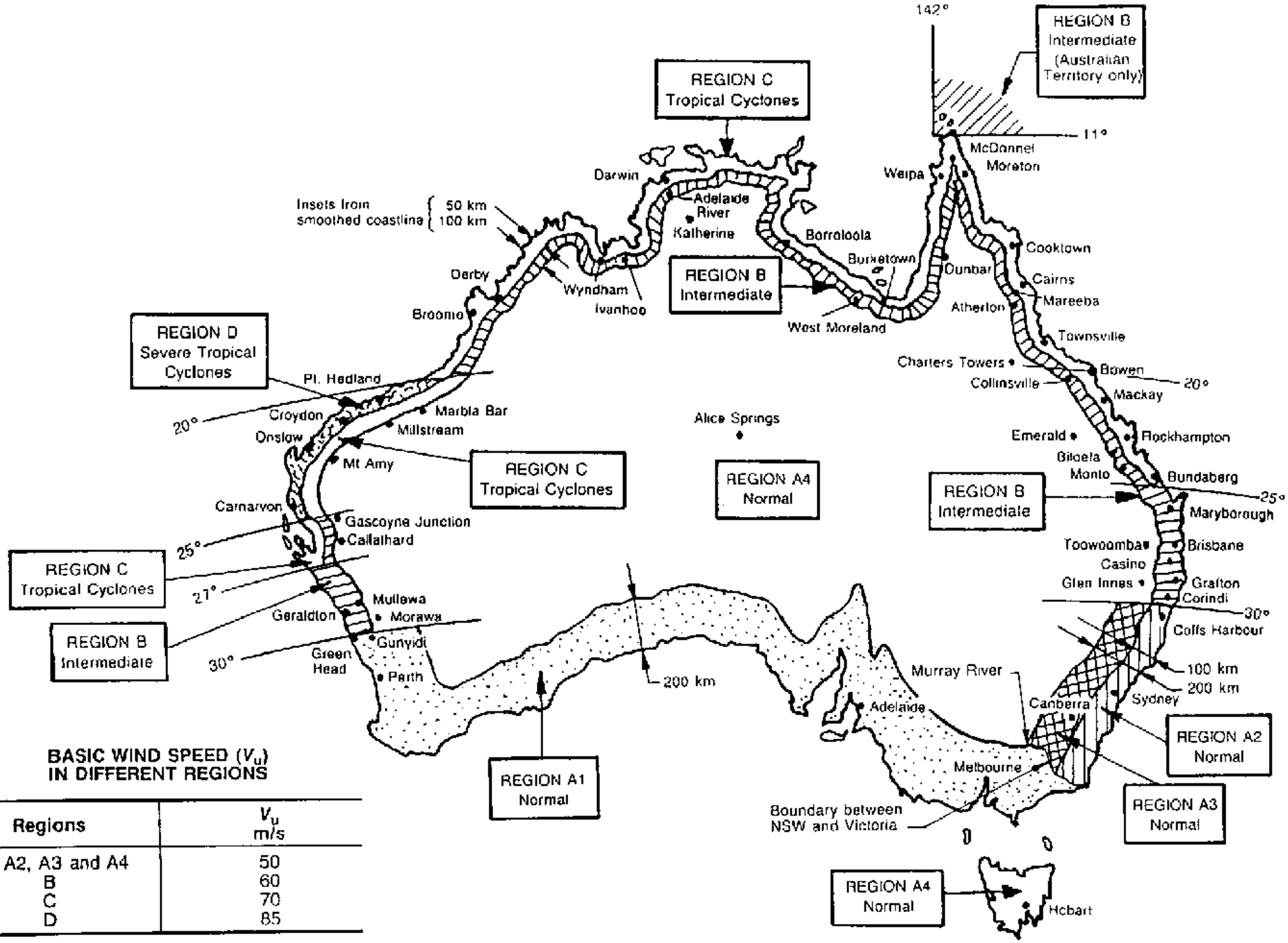
(b) $0.5W_{u1} + W_{u2}$

(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.



BASIC WIND SPEED (V_u) IN DIFFERENT REGIONS

Regions	V_u m/s
A1, A2, A3 and A4	50
B	60
C	70
D	85

FIGURE 2.2 BOUNDARIES OF REGIONS A1, A2, A3, A4, B, C AND D

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:

$$V_z = VM_{(z,\,cat)} M_t M_d \qquad \text{. . . 2.2.2}$$

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:

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

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_t, 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.

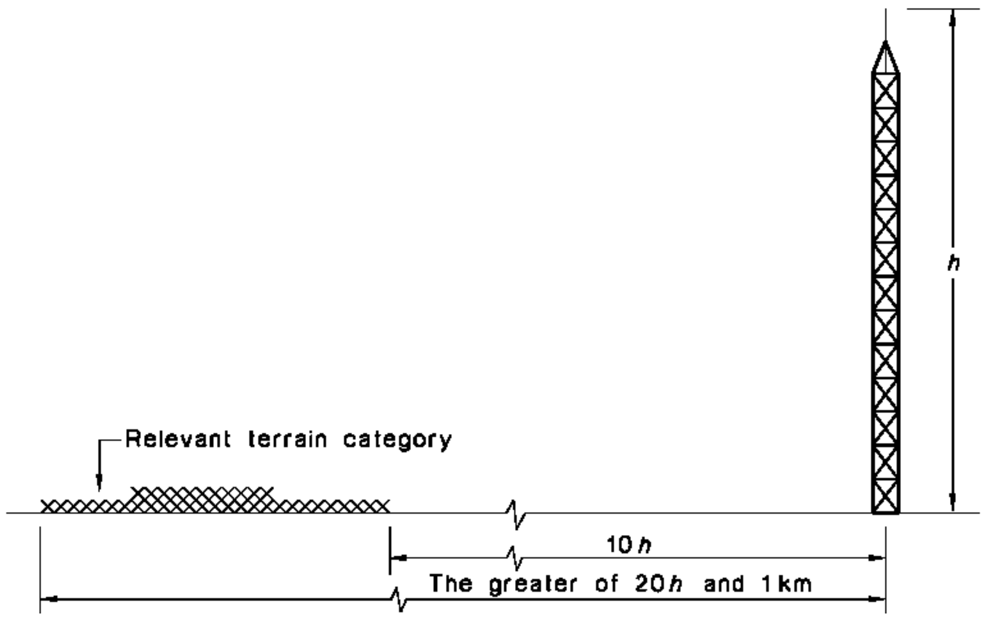


FIGURE 2.2.3.1 TERRAIN CATEGORY SELECTION

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

Height (z) m	Terrain height multiplier ($M_{(z, cat)}$)	
	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.

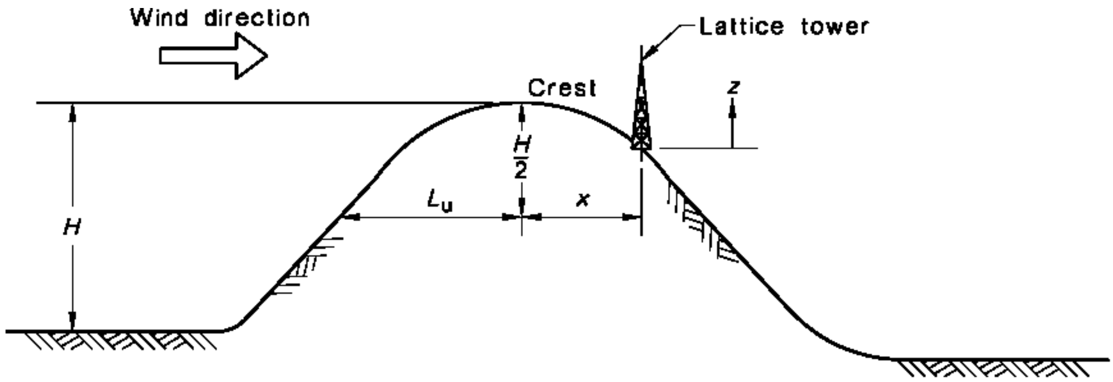


FIGURE 2.2.4(1) HILLS AND RIDGES

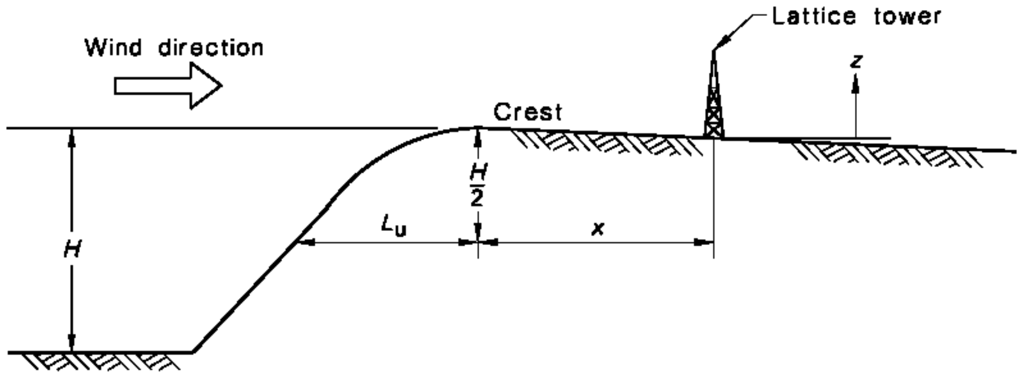


FIGURE 2.2.4(2) ESCARPMENTS

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:

$$F_d = C_d A_z q_z \qquad \ldots\ 2.2.6(1)$$

(b) For a tower section with ancillaries:

$$F_d = C_{de} A_z q_z \qquad \ldots\ 2.2.6(2)$$

(c) For guy cables:

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

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:

$$q_z = 0.6 \times 10^{-3} V_z^2 \qquad \ldots\ 2.2.7$$

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—

(a) the coefficients given in this Clause 2.2.8; or

(b) 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.

TABLE 2.2.8.2(1)
DRAG FORCE COEFFICIENT (C_d) FOR SQUARE AND EQUILATERAL-TRIANGLE PLAN LATTICE TOWERS WITH FLAT-SIDED MEMBERS

Solidity of front face (δ)	Drag force coefficient (C_d)		
	Square towers		Equilateral-triangle towers
	Onto face	Onto corner	
≤0.1	3.5	3.9	3.1
0.2	2.8	3.2	2.7
0.3	2.5	2.9	2.3
0.4	2.1	2.6	2.1
≥0.5	1.8	2.3	1.9

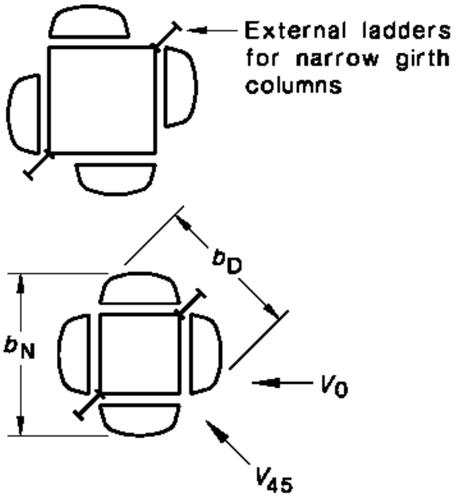
TABLE 2.2.8.2(2)
DRAG FORCE COEFFICIENT (C_d) FOR SQUARE PLAN LATTICE TOWERS WITH CIRCULAR MEMBERS

Solidity of front face (δ)	Drag force coefficient (C_d)			
	Parts of tower in sub-critical flow $bV < 3$ m²/s		Parts of tower in super-critical flow $bV \geq 6$ m²/s	
	Onto face	Onto corner	Onto face	Onto corner
≤0.05	2.2	2.5	1.4	1.2
0.1	2.0	2.3	1.4	1.3
0.2	1.8	2.1	1.4	1.6
0.3	1.6	1.9	1.4	1.6
0.4	1.5	1.9	1.4	1.6
≥0.5	1.4	1.9	1.4	1.6

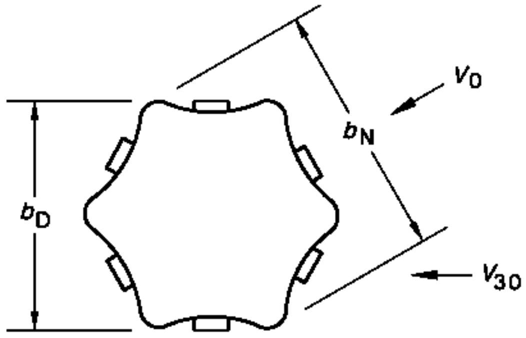
TABLE 2.2.8.2(3)
DRAG FORCE COEFFICIENT (C_d) FOR EQUILATERAL-TRIANGLE PLAN LATTICE TOWER WITH CIRCULAR MEMBERS

Solidity of front face (δ)	Drag force coefficient (C_d)	
	Parts of tower in sub-critical flow $bV < 3$ m²/s (all wind directions)	Parts of tower in super-critical flow $bV \geq 6$ m²/s (all wind directions)
≤0.05	1.8	1.1
0.1	1.7	1.1
0.2	1.6	1.1
0.3	1.5	1.1
0.4	1.5	1.1
≥0.5	1.4	1.2

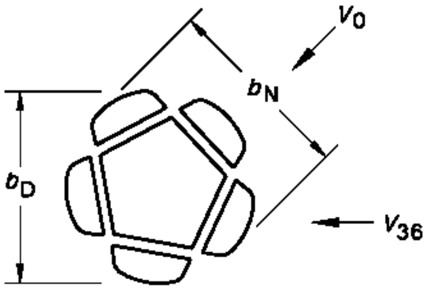
LEGEND TO TABLES 2.2.8.2(1), 2.2.8.2(2) AND 2.2.8.2(3):
δ = solidity ratio (solid area divided by the total enclosed area)
b = average diameter or breadth of a section of a tower member.
NOTE: For intermediate values of bV, interpolation is permitted.



(a) Antenna Type 1



(b) Antenna Type 2



(c) Antenna Type 3

(c) Antenna type 3

DRAG FORCE COEFFICIENT (C_d)
FOR UHF-ANTENNA SECTIONS

Antenna type	Wind direction	Drag force coefficient (C_d)
1	V_0, V_{45}	1.5
2	V_0, V_{30}	1.9
3	V_0, V_{36}	1.6

NOTES:

1 The values of drag force coefficient (C_d) given above are tentative but are believed to be conservative.

2 To calculate the area (A_z) in Equation 2.2.6(1), use breadth (b_D) or (b_N), as appropriate to the wind direction.

3 Reduction for aspect ratio may be carried out by multiplying by the correction factor (K_{ar}) given in Table F3, taking l equal to two times the height of the end-mounted antennas.

FIGURE 2.2.8.2 DRAG FORCE COEFFICIENTS (C_d) FOR SECTIONS OF UHF ANTENNAS

2.2.8.3 *Tower sections with ancillaries* The drag force coefficient for tower sections with ancillaries shall be calculated as follows:

(a) It shall be permissible to treat ancillaries attached symmetrically to all faces by adding their projected area to the projected area of the tower members (A_z).

(b) When the conditions in Item (a) are not applicable, the total effective drag force coefficient (C_{de}) for a tower section shall be taken as follows:

$$C_{de} = C_d + \Sigma \Delta C_d \qquad \text{. . . 2.2.8.3(1)}$$

The additional drag coefficient (ΔC_d) due to an ancillary attached to one face, or located inside the tower section, shall be calculated using the following equation:

$$\Delta C_d = C_{da} K_{ar} K_{in} (A_a / A_z) \qquad \text{. . . 2.2.8.3(2)}$$

where

C_{da} = drag force coefficient of the isolated ancillary which in the absence of wind tunnel data, may be obtained from Appendix F

K_{ar} = correction factor for aspect ratio

For linear ancillaries with aspect ratios less than 40, K_{ar} is given in Table F3. For all other cases, assume K_{ar} equals 1.0

K_{in} = correction factor for interference (see Clause 2.2.8.4)

A_a = reference area of any ancillaries attached to a tower section. For a linear ancillary, A_a shall be taken as lb, where l is the length of the linear ancillary and b is defined in Appendix F

NOTE: For background information on tower sections with ancillaries, see Paragraph E5 of Appendix E.

2.2.8.4 *Correction factor for interference* The correction factor for interference (K_{in}) shall be calculated as follows:

(a) For ancillaries attached to the face of the tower, K_{in} shall be determined from the following equations:

(i) To the face of a square tower (see Figure 2.2.8.4(a))—

$$K_{in} = [1.5 + 0.5\cos 2(\theta_a - 90°)]\exp[-1.2(C_d\delta)^2] \qquad \text{. . . 2.2.8.4(1)}$$

(ii) To the face of a triangular tower (see Figure 2.2.8.4(b))—

$$K_{in} = [1.5 + 0.5\cos 2(\theta_a - 90°)]\exp[-1.8(C_d\delta)^2] \qquad \text{. . . 2.2.8.4(2)}$$

(b) For lattice-like ancillaries inside the tower, K_{in} shall be taken either as 1.0 or shall be determined from the following equations:

(i) Inside a square tower (see Figure 2.2.8.4(c))—

$$K_{in} = \exp[-1.4(C_d\delta)^{1.5}] \qquad \text{. . . 2.2.8.4(3)}$$

(ii) Inside a triangular tower (see Figure 2.2.8.4(d))—

$$K_{in} = \exp[-1.8(C_d\delta)^{1.5}] \qquad \text{. . . 2.2.8.4(4)}$$

(c) For cylindrical ancillaries inside the tower, K_{in} shall be taken either as 1.0 or shall be determined from the following equations:

(i) Inside a square tower (see Figure 2.2.8.4(e))—

$$K_{in} = \exp[-a(C_d\delta)^{1.5}] \qquad \text{. . . 2.2.8.4(5)}$$

$$a = 2.7 - 1.3\exp[-3(b/w)^2] \qquad \text{. . . 2.2.8.4(6)}$$

(ii) Inside a triangular tower (see Figure 2.2.8.4(f))—

$$K_{in} = 1.5\exp[-c(C_d\delta)^{1.5}] \qquad \text{. . . 2.2.8.4(7)}$$

$$c = 6.8 - 5\exp[-40(b/w)^3] \qquad \text{. . . 2.2.8.4(8)}$$

where

θ_a = angle of deviation of the wind stream from the normal of the ancillary

δ = solidity ratio of the tower section specified in Clause 2.2.8.2

a, c = constant

b/w = ratio of the average diameter of the ancillary to the average width of the structure

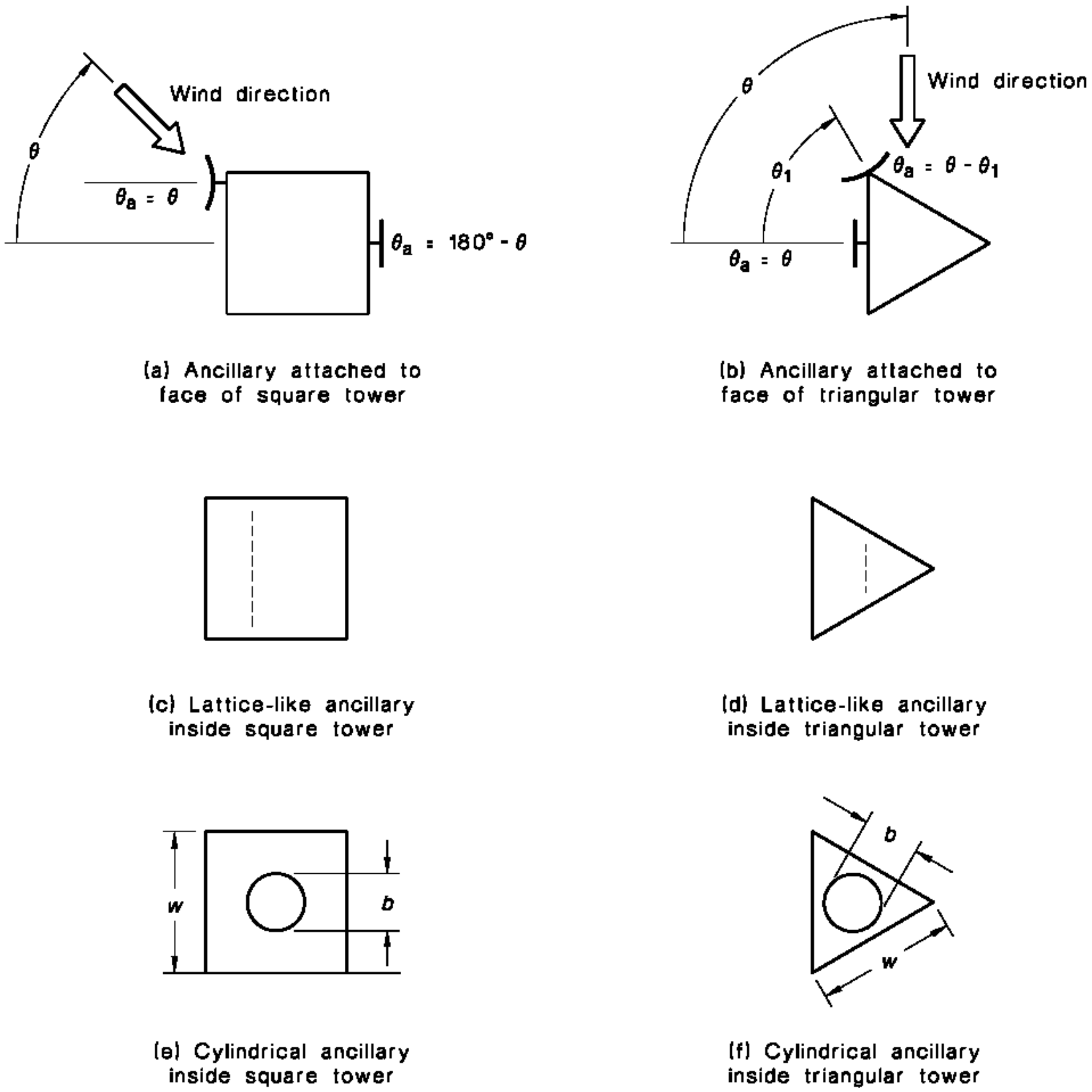


FIGURE 2.2.8.4 TOWER SECTIONS WITH ANCILLARIES

2.3 DYNAMIC ANALYSIS

2.3.1 General The design mean wind speed ($\bar{V}_z$) at height z shall be used to determine wind pressures and forces acting on a structure for the dynamic analysis procedure.

NOTE: Estimation of the first mode natural frequency is given in Appendix D.

2.3.2 Mean wind speed The design mean wind speed ($\bar{V}_z$) for calculation of dynamic response shall be determined from the basic wind speed (V) for the appropriate limit state given as follows:

$$\bar{V}_z = V\bar{M}_{(z,\ \mathrm{cat})}\bar{M}_t M_d \qquad \ldots 2.3.2$$

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

$\bar{M}_{(z,\ \mathrm{cat})}$ = mean wind speed multiplier for a terrain category at height z (see Clause 2.3.3.2)

$\bar{M}_t$ = topographic multiplier for mean wind speeds (see Clause 2.3.4)

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 2.3.2. If there are significant effects of shielding, see AS 1170.2.

2.3.3 Terrain and height multiplier ($\bar{M}_{(z,\ \mathrm{cat})}$)

2.3.3.1 *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 $M_{(z,\ \mathrm{cat})}$, see Paragraph E2 of Appendix E.

2.3.3.2 *Terrain multiplier* The variation of terrain multipliers with height (z) shall be taken from Tables 2.3.3.2(1) and 2.3.3.2(2), as appropriate.

2.3.4 Topographic multiplier The topographic multiplier for mean wind speeds ($\bar{M}_t$) at or near the crest of a hill, ridge or escarpment shall be calculated by the following equation:

$$\bar{M}_t = 1 + \frac{H\left(1 - \frac{|x|}{4L_m}\right)}{3.5(z + L_m)} \qquad \ldots 2.3.4$$

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_m = length used in the topographic multiplier

= $0.4H$ or $0.25L_u$, 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 $\bar{M}_t$, see Paragraph E3 of Appendix E.

A topographic multiplier of $\bar{M}_t = 1$ shall be used for all sites where $|x| > 4L_m$ or if the upwind slope (ϕ) is less than 0.05.

TABLE 2.3.3.2(1)

TERRAIN HEIGHT MULTIPLIER ($\bar{M}_{(z,\,cat)}$) FOR MEAN 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 ($\bar{M}_{(z,\,cat)}$)			
	Terrain Category 1	Terrain Category 2	Terrain Category 3	Terrain Category 4
≤3	0.61	0.48	0.38	0.35
5	0.65	0.53	0.38	0.35
10	0.71	0.60	0.44	0.35
15	0.74	0.64	0.49	0.35
20	0.77	0.66	0.52	0.35
30	0.80	0.70	0.57	0.38
40	0.83	0.74	0.60	0.40
50	0.85	0.76	0.63	0.42
75	0.89	0.81	0.68	0.51
100	0.92	0.84	0.72	0.55
150	0.97	0.89	0.78	0.62
200	1.00	0.93	0.82	0.67
250	1.03	0.96	0.86	0.71
300	1.06	0.99	0.89	0.74
400	1.10	1.04	0.94	0.81
500	1.14	1.08	0.99	0.86

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

2.3.5 Along-wind response The along-wind response for freestanding towers shall be calculated using the gust response factor method.

TABLE 2.3.3.2(2)

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

Height (z) m	Terrain height multiplier ($\bar{M}_{(z,\,cat)}$) Terrain Categories 1 and 2	Terrain Categories 3 and 4
≤3	0.50	0.40
5	0.54	0.40
10	0.60	0.47
15	0.64	0.54
20	0.68	0.59
30	0.75	0.67
40	0.80	0.74
50	0.84	0.80
75	0.93	0.91
≥100	1.00	1.00

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

2.3.5.1 *Limitations* This Clause 2.3.5 gives a method for calculation of the along-wind response of freestanding lattice towers. It is not applicable to guyed masts, for which a simplified approach is given in DAVENPORT and SPARLING (Ref. 1) (see Appendix J), otherwise specialist advice should be sought.

2.3.5.2 *Design load effects* The design peak shear force (Q_s) and design bending moment (M_s) at height s above ground level shall be calculated using Equations 2.3.5.2(1) and 2.3.5.2(2), respectively.

$$Q_s = G_s \bar{Q}_s \qquad \text{. . . 2.3.5.2(1)}$$

$$M_s = G_s \bar{M}_s \qquad \text{. . . 2.3.5.2(2)}$$

where

G_s = gust response factor calculated in accordance with Clause 2.3.8

$\bar{Q}_s$ = mean shear force at height s, calculated from the following equation:

$$\bar{Q}_s = \Sigma C_{de} A_z \bar{q}_z \quad \text{for } z > s \qquad \text{. . . 2.3.5.2(3)}$$

$\bar{q}_z$ = free-stream mean dynamic wind pressure at height z, in kilopascals, calculated from the following equation:

$$\bar{q}_z = 0.6 \times 10^{-3} \bar{V}_z^2 \qquad \text{. . . 2.3.5.2(4)}$$

$\bar{V}_z$ = design mean wind speed at height z, in metres per second

$\bar{M}_s$ = mean bending moment at height s, calculated from the following equation:

$$\bar{M}_s = \Sigma C_{de} A_z (z - s) \bar{q}_z \quad \text{for } z > s \qquad \text{. . . 2.3.5.2(5)}$$

C_{de} = effective drag force coefficient for a tower section with ancillaries; C_{de} is equal to C_d for tower sections without ancillaries

2.3.6 Cross-wind response The cross-wind response for lattice towers and masts shall be considered where there are substantial enclosed parts of the structure near the top. However, the cross-wind response may be neglected if the solidity ratio (δ) is less than 0.5.

When the cross-wind response is considered, structural effects due to a combination of along-wind and cross-wind responses shall be derived in accordance with AS 1170.2.

NOTE: For background information on cross-wind response, see Paragraph E6 of Appendix E.

2.3.7 Serviceability design If it is required that serviceability criteria for deflection or rotation be checked, the deflection or rotation at a specified probability level can be calculated using a detailed probabilistic analysis, or for along-wind response by use of the following equation:

$$\psi_1 = 1.1\bar{\psi} \qquad \ldots 2.3.7$$

where $\bar{\psi}$ is the mean deflection or rotation at the same probability level.

NOTE: For background information on serviceability, see Paragraph E1 of Appendix E.

2.3.8 Gust response factor for freestanding lattice towers The gust response factor (G_s) for ultimate limit states design for freestanding lattice towers shall be calculated from one of the following methods:

(a) The simplified method specified in this Clause.

(b) Detailed approaches given in BS 8100: Part 1, *ESDU* (Ref. 2) and Holmes (Ref. 3), (see Appendix J).

When using the simplified method, G_s shall be calculated from the following equation:

$$G_s = 1 + rH\sqrt{\left[g_B^2 B_s + g_R^2\left(\frac{SE}{\zeta}\right)\right]} \qquad \ldots 2.3.8(1)$$

where

r = roughness factor, equal to twice the longitudinal turbulence intensity at height h

$$r = \frac{2}{\bar{M}_t}\left(\frac{\sigma_v}{\bar{V}}\right) \qquad \ldots 2.3.8(2)$$

$\frac{\sigma_v}{\bar{V}}$ = turbulence intensity given in AS 1170.2

$\bar{M}_t$ = topographic multiplier for mean wind speeds (see Clause 2.3.4)

H = height factor

$$H = 1 + 0.2\left(\frac{s}{h}\right)^2 \qquad \ldots 2.3.8(3)$$

s = height of the design peak shear force (Q_s) 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

h = height of the structure above ground

g_B = peak factor for the background response

$$g_B = 3.7[1 + (0.925r\sqrt{B_s})] \qquad \ldots 2.3.8(4)$$

B_s = background factor

$$B_s = \frac{1}{1 + \dfrac{\sqrt{[36(h-s)^2 + 64w_s^2]}}{L_h}} \qquad \ldots 2.3.8(5)$$

w_s = average width of the structure between h and s

L_h = measure of the effective turbulence length scale

$$L_h = 1000\left(\frac{h}{10}\right)^{0.25} \qquad \ldots 2.3.8(6)$$

g_R = peak factor for resonant response

$$g_R = \sqrt{[2\log_e(Tn)]} \qquad \ldots 2.3.8(7)$$

T = averaging time (for example, T = 600 seconds for a 10 minute averaging time)

n = first mode natural sway frequency

S = size factor for resonant response

$$S = \frac{1}{\left[1 + \left(\dfrac{3.5nh}{\bar{V}_h}\right)\right]\left[1 + \left(\dfrac{4nw_o}{\bar{V}_h}\right)\right]} \qquad \ldots 2.3.8(8)$$

w_o = average width of the structure between $h/2$ and h

$\bar{V}_h$ = mean wind speed calculated for z equal to h, calculated from Equation 2.3.2

E = gust energy factor

$$E = \frac{0.47N}{(2 + N^2)^{5/6}} \qquad \ldots 2.3.8(9)$$

N = effective reduced frequency

$$N = \frac{nL_h}{\bar{V}_h} \qquad \ldots 2.3.8(10)$$

ζ = critical damping ratio, which, in the absence of more accurate values, is to be taken as 0.05 for bolted steel and 0.02 for welded steel

NOTE: For background information on gust response factor for freestanding lattice towers, see Paragraph E7 of Appendix E.

SECTION 3 STRUCTURAL ANALYSIS AND DESIGN

3.1 STRUCTURAL ANALYSIS

3.1.1 General Freestanding lattice towers shall be analysed using a first order linear elastic method. For the analysis of lattice mast structures which are supported by guys, the non-linear properties of the guys and other second order effects shall be taken into account.

For the application of the design provisions of this Section, the axial forces in triangulated lattice towers shall be determined by assuming all members are pin connected.

A rational design may be used in lieu of the design procedures provided in this Standard. In such case, it shall be ensured that the adopted procedure will lead to a level of safety and performance equivalent to that envisaged by, or implicit in, this Standard.

Where guys provide supplementary stability to the inherent stiffness of a tower, the analysis of guyed lattice towers shall be in accordance with this Standard, taking into account the non-linearity of the guy and tower interaction.

Adequate allowance for torsional effects due to asymmetric positioning of antennas shall be considered in the structural design. For guyed masts with torsional outriggers, secondary torsional effect due to the outriggers' reaction shall be considered. Members of the torsional outriggers shall be designed to allow for the worst combination of guy tensions.

NOTE: For background information on freestanding lattice towers, see Paragraph G1 of Appendix G.

3.1.2 Guyed masts The stability of guyed masts shall be ensured by considering guy displacements under critical loading conditions. In addition, serviceability performance under lesser loading conditions shall be considered.

As a minimum, a quasi-static analysis shall be considered, taking into account—

(a) the full non-linear, second order effects caused by changes in the structure geometry;

(b) all loads on guys; and

(c) maximum shear forces and moments resulting from a patch loading analysis for guyed masts taller than 150 m.

NOTES:

1 The dynamic analysis of guyed masts is beyond the scope of this Standard.

2 For background information on guyed masts, see Paragraph G2 of Appendix G.

3.2 GENERAL DESIGN REQUIREMENTS

3.2.1 General The provisions of this Section are applicable to the design of hot-rolled angle members conforming to AS 3679.1 and guy cables conforming to AS 2841 and AS 3569. For all other sections, AS 4100 is applicable.

3.2.2 Minimum sizes Minimum thicknesses for members shall be 3 mm and for connection plates 5 mm. Allowance shall be made for steel exposed to corrosion at the ground line.

3.2.3 Limiting slenderness ratios The limiting slenderness ratio for members shall be as follows:

(a) For members subject to design compressive stresses:

(i) Leg members: $\lambda_e \leq 150$

(ii) Bracing members: $\lambda_e \leq 200$

(iii) Secondary bracing members: $\lambda_e \leq 250$

where λ_e is the effective slenderness ratio determined in accordance with Clause 3.3.4.

(b) For members subject to tension, excluding guy cables:

$$\frac{l}{r} \leq 400$$

where

l = actual unsupported length of member for buckling about the relevant axis

r = radius of gyration

3.3 COMPRESSION MEMBERS

NOTES:

1 The rules for compression members of this Clause 3.3 are based on *ASCE*, Manuals and Reports on Engineering Practice, No. 52, Guide for Design of Steel Transmission Towers, 2nd edition, 1988.

2 For background information on single bolted compression members, see Paragraph G3 of Appendix G.

3.3.1 Member compression capacity A member subject to a design axial compression force (N_c^*) shall satisfy the following:

$$N_c^* \leq \phi N_c \qquad \text{. . . 3.3.1(1)}$$

where

ϕ = capacity factor (see Table 1.8)

N_c = nominal capacity of the member in compression, calculated from the following equation:

$$N_c = \alpha_c A f_y \qquad \text{. . . 3.3.1(2)}$$

α_c = member slenderness reduction factor

A = area of cross-section

f_y = yield stress of steel

3.3.2 Member slenderness reduction factor The member slenderness reduction factor (α_c) shall be determined from the effective slenderness ratio (λ_e) as follows:

(a) $$\alpha_c = k_f\left[1 - 0.5\left(\frac{\lambda_e}{\lambda_c}\right)^2\right] \quad \text{for } \lambda_e \leq \lambda_c \qquad \text{. . . 3.3.2(1)}$$

(b) $$\alpha_c = \frac{\pi^2 E}{f_y(\lambda_e)^2} \quad \text{for } \lambda_e > \lambda_c \qquad \text{. . . 3.3.2(2)}$$

where

k_f = form factor for members subject to axial compression

$$k_f = \frac{A_e}{A} \qquad \text{. . . 3.3.2(3)}$$

A_e = effective compression section area (see Clause 3.3.3)

λ_c = factor for the determination of α_c

$$\lambda_c = \pi\sqrt{\frac{2E}{k_f f_y}} \qquad \text{. . . 3.3.2(4)}$$

E = Young's modulus of elasticity
= 200×10^3 MPa

3.3.3 Effective compression area The effective compression section area (A_e) shall be calculated as follows:

(a) $$A_e = A \quad \text{for } \left(\frac{w}{t}\right) \leq \left(\frac{w}{t}\right)_{lim} \qquad \text{. . . 3.3.3(1)}$$

(b) $$A_e = A\left[1.677 - \left(0.677\frac{\left(\frac{w}{t}\right)}{\left(\frac{w}{t}\right)_{lim}}\right)\right] \quad \text{for } \left(\frac{w}{t}\right)_{lim} < \left(\frac{w}{t}\right) < \frac{378}{\sqrt{f_y}} \qquad \text{. . . 3.3.3(2)}$$

(c) $$A_e = A\left(\frac{65\,500}{f_y\left(\frac{w}{t}\right)^2}\right) \quad \text{for } \left(\frac{w}{t}\right) \geq \frac{378}{\sqrt{f_y}} \qquad \text{. . . 3.3.3(3)}$$

where

w = flat width from root of fillet (larger width for unequal angles (see Figure 3.3.3))

t = thickness of angle leg

$\left(\frac{w}{t}\right)_{lim}$ = limiting (w/t) ratio calculated from the following equation:

$$\left(\frac{w}{t}\right)_{lim} = \frac{210}{\sqrt{f_y}} \qquad \text{. . . 3.3.3(4)}$$

f_y = yield stress of steel, in megapascals

The (w/t) ratio shall not exceed 25.

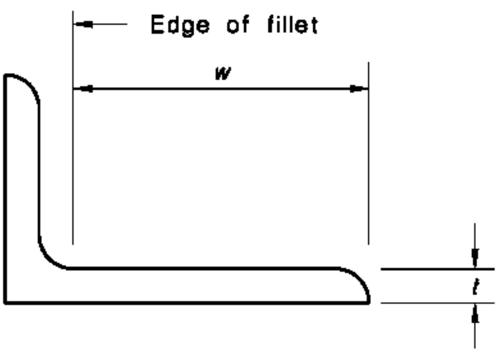


FIGURE 3.3.3 DETERMINATION OF (w/t) RATIO

3.3.4 Effective slenderness ratios

3.3.4.1 *General* The effective slenderness ratio (λ_e) shall be calculated in accordance with Clauses 3.3.4.2, 3.3.4.3 and 3.3.4.4, using the member slenderness ratio (L/r).

NOTE: For background information on effective slenderness ratios, see Paragraph G4 of Appendix G.

The slenderness ratio for leg members with staggered bracing and for members in tension-compression bracing systems shall be determined in accordance with Appendix H. For other members, the length (L) used to determine the slenderness ratio shall be the actual unsupported length (l) of the member for buckling about the relevant axis.

3.3.4.2 *Leg members* For leg members bolted in both faces at connections, the effective slenderness ratio (λ_e) shall be as follows:

$$\lambda_e = \frac{L}{r} \quad \text{for } 0 \le \frac{L}{r} \le 150 \qquad \text{. . . 3.3.4.2}$$

3.3.4.3 *Bracing members* Bracing members shall comply with the following:

(a) For members with concentric load at both ends of the unsupported length:

$$\lambda_e = \frac{L}{r} \quad \text{for } 0 \le \frac{L}{r} \le 120 \qquad \text{. . . 3.3.4.3(1)}$$

(b) For members with a concentric load at one end and normal framing eccentricity at the other end of the unsupported length:

$$\lambda_e = 30 + \left(0.75\frac{L}{r}\right) \quad \text{for } 0 \le \frac{L}{r} \le 120 \qquad \text{. . . 3.3.4.3(2)}$$

(c) For members with normal framing eccentricities at both ends of the unsupported length:

$$\lambda_e = 60 + \left(0.5\frac{L}{r}\right) \quad \text{for } 0 \le \frac{L}{r} \le 120 \qquad \text{. . . 3.3.4.3(3)}$$

(d) For members unrestrained against rotation at both ends of the unsupported length:

$$\lambda_e = \frac{L}{r} \quad \text{for } 120 \le \frac{L}{r} \le 200 \qquad \text{. . . 3.3.4.3(4)}$$

(e) For members partially restrained against rotation at one end of the unsupported length:

$$\lambda_e = 28.6 + \left(0.762\frac{L}{r}\right) \quad \text{for } 120 \le \frac{L}{r} \le 225 \qquad \text{. . . 3.3.4.3(5)}$$

(f) For members partially restrained against rotation at both ends of the unsupported length:

$$\lambda_e = 46.2 + \left(0.615\frac{L}{r}\right) \quad \text{for } 120 \le \frac{L}{r} \le 250 \qquad \text{. . . 3.3.4.3(6)}$$

3.3.4.4 *Secondary bracing members* Secondary bracing members shall comply with the following:

(a) For members with any end condition:

$$\lambda_e = \frac{L}{r} \quad \text{for } 0 \le \frac{L}{r} \le 120 \qquad \text{. . . 3.3.4.4(1)}$$

(b) For members unrestrained against rotation at both ends of the unsupported length:

$$\lambda_e = \frac{L}{r} \quad \text{for } 120 \leq \frac{L}{r} \leq 250 \qquad \ldots 3.3.4.4(2)$$

(c) For members partially restrained against rotation at one end of the unsupported length:

$$\lambda_e = 28.6 + \left(0.762\frac{L}{r}\right) \quad \text{for } 120 \leq \frac{L}{r} \leq 290 \qquad \ldots 3.3.4.4(3)$$

(d) For members partially restrained against rotation at both ends of the unsupported length:

$$\lambda_e = 46.2 + \left(0.615\frac{L}{r}\right) \quad \text{for } 120 \leq \frac{L}{r} \leq 330 \qquad \ldots 3.3.4.4(4)$$

3.3.4.5 *Member end restraints* A single bolt connection at either the end of a member or a point of intermediate support shall not be considered as furnishing restraint against rotation. A multiple bolt connection, detailed to minimize eccentricity, shall be considered to provide partial restraint if the connection is to a member capable of resisting rotation of the joint.

3.3.5 Compound leg members Compound members for legs built up with two or more angles in cruciform section welded continuously (see Figure 3.3.5(a)) or bolted together with stitch bolts at a distance less than $40r_{min.}$, shall be taken as fully composite and treated as single members.

Compound members for legs built up with two or more angles in cruciform section bolted together with stitch bolts in excess of $40r_{min.}$ (see Figure 3.3.5(b)), shall have their effective slenderness ratio (λ_e) modified to take account of possible additional shear deformations as given by the following equation:

$$\lambda_e = \sqrt{\left(\lambda_1^2 + \lambda_2^2\right)} \qquad \ldots 3.3.5$$

where

λ_1 = slenderness ratio of the compound member assuming full composite action

λ_2 = slenderness ratio of one component angle

= $C/r_{min.}$

C = distance between the stitch bolts

$r_{min.}$ = minimum radius of gyration of the section

3.3.6 Mast leg members For mast leg members, the effective slenderness ratio (λ_e) shall be calculated as follows:

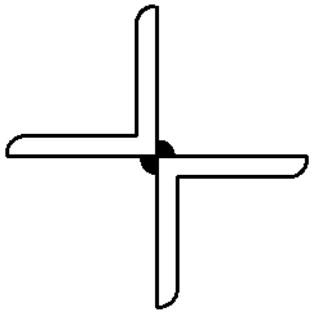
$$\lambda_e = \sqrt{\left(\lambda_3^2 + \lambda_4^2\right)} \qquad \ldots 3.3.6$$

where

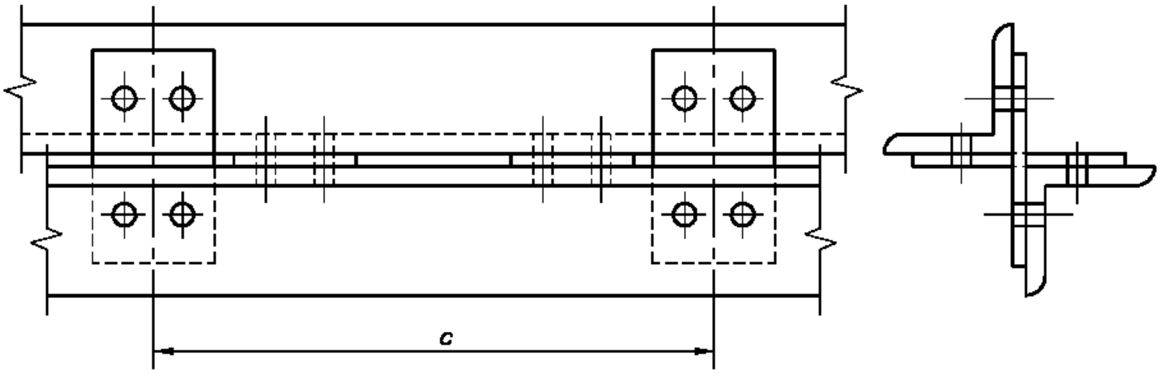
λ_3 = slenderness ratio of a segment of the mast between support levels

λ_4 = slenderness ratio of the individual leg members

3.3.7 Restraint forces in secondary bracing members Secondary bracing members meeting at a point and their connections restraining a compression member, shall together be able to support a load equal to 2.5% of the axial load of the supported member.



(a) Cruciform



(b) Typical compound compression member

FIGURE 3.3.5 COMPOUND ANGLE MEMBERS

3.4 TENSION MEMBERS

3.4.1 General Tension members, except for guys, shall be designed in accordance with AS 4100.

3.4.2 Guy tension members

3.4.2.1 *General* A guy tension member subject to a design axial tensile force (N_t^*) shall satisfy—

$$N_t^* \leq \phi N_t \qquad \text{. . . 3.4.2.1}$$

where

ϕ = capacity factor (see Table 1.8)

N_t = nominal section capacity in a guy tension member determined in accordance with Clause 3.4.2.2

NOTE: For background information on guy tension members, see Paragraph G5 of Appendix G.

3.4.2.2 *Nominal section capacity* The nominal section capacity of a guy tension member (N_t) shall be calculated as follows:

$$N_t = 0.85 K_t A_n f_u \qquad \text{. . . 3.4.2.2}$$

where

K_t = correction factor for the distribution of force in a tension member (see Clause 3.4.2.3)

A_n = net area of a cross-section

f_u = tensile strength used in design

3.4.2.3 *Distribution of forces* The correction factor (K_t) for the distribution of force into the guy shall be as given in Table 3.4.2.3. The Table is based on the efficiency of terminal steel guy attachments.

TABLE 3.4.2.3

EFFICIENCY OF TERMINAL ROPE ATTACHMENTS

Type of rope fitting or end attachment	Figure 3.4.2.3	Correction factor (K_t)	Resistance to vibration and impact
Closed metalled sockets (see Note 1)	(a)	1.0	Fair
Open metalled sockets (see Note 1)	(b)	1.0	Fair
Swaged sockets (see Note 1)	(c)	1.0	Good
Flemish eyes with swaged sleeve (see Note 2)	(d)	1.0	Good
Eyes with pressed aluminium alloy ferrule (see Note 2)	(e)	0.9	Good
Hand spliced eyes (see Note 2):	(f)		
5 mm rope		0.9	Excellent
15 mm rope		0.85	Excellent
25 mm rope		0.8	Excellent
40 mm rope (see Note 3)		0.75	Excellent
60 mm rope (see Note 3)		0.7	Excellent
Eyes with three fist grip type clips (see Note 2)	(g)	0.85	Good
Eyes with three wire rope grips fitted (see Note 2))	(h)	0.8	Good

NOTES:

1 Only termination types shown in Figure 3.4.2.3(a), (b) or (c) should be used for bridge strand guys.

Other types of terminations may be used if the manufacturer has demonstrated that the connection is appropriate for the installation and performance of the guy rope or strand.

2 Should be fitted with solid thimble.

3 Should be fitted in workshop.

3.5 CONNECTIONS

3.5.1 General The connections in a structure shall be proportioned so as to be consistent with the assumptions made in the analysis of the structure. Connections shall be capable of transmitting the design load effects (S^*) and shall be designed to minimize eccentricities between load carrying members.

3.5.2 Welded connections Welded connections shall be designed in accordance with AS 4100 using SP Category welds.

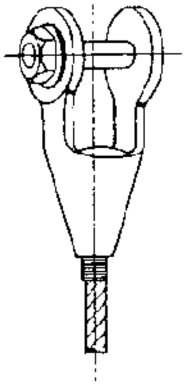
NOTE: For background information on testing of welds, see Paragraph G6 of Appendix G.

3.5.3 Bolted connections Bolted connections or pinned connections using bolts other than tower bolts shall be designed in accordance with AS 4100.

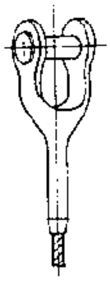
3.5.4 Bolted connections using tower bolts

3.5.4.1 *General* Bolted connections using tower bolts conforming to AS 1559 shall be designed in accordance with this Clause 3.5.4.

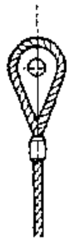
(a) Closed metalled sockets



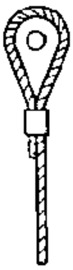
(b) Open metalled sockets



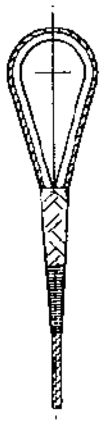
(c) Swaged sockets



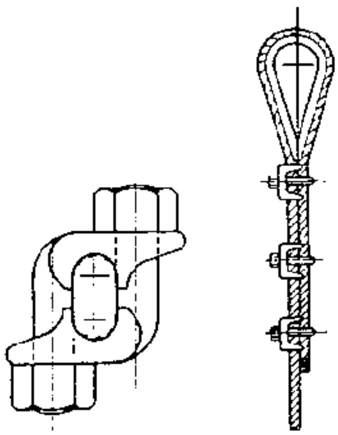
(d) Flemish eyes with swaged sleeve



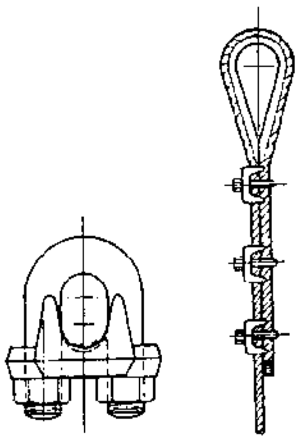
(e) Eyes with pressed aluminium alloy ferrule



(f) Hand spliced eyes



(g) Eyes with three fist grip type clips



(h) Eyes with three wire rope grips

FIGURE 3.4.2.3 TERMINAL ROPE ATTACHMENTS

3.5.4.2 *Bolts in shear* A bolt subject to a design shear force (V_f^*) shall satisfy the following:

$$V_f^* \leq \phi V_f \qquad \ldots 3.5.4.2(1)$$

where

ϕ = capacity factor (see Table 1.8)

V_f = nominal shear capacity of a bolt

The nominal shear capacity of a bolt (V_f) shall be calculated as follows:

$$V_f = f_{vf} K_r n_x A_o \qquad \ldots 3.5.4.2(2)$$

where

f_{vf} = minimum shear strength of a bolt as specified in AS 1559
= 320 MPa

K_r = reduction factor given in Table 3.5.4.2 to account for the length of a bolted lap connection (L_j); for all other connections K_r = 1.0

n_x = number of shear planes without threads intercepting the shear plane

A_o = nominal plain shank area of bolt

Threads shall be excluded from the shear plane.

TABLE 3.5.4.2
REDUCTION FACTOR FOR A BOLTED LAP CONNECTION (K_r)

Length mm	$L_j < 300$	$300 \leq L_j \leq 1300$	$L_j > 1300$
K_r	1.0	$1.075 - (L_j/4000)$	0.75

3.5.4.3 *Bolts in tension* Bolts in tension shall be designed in accordance with AS 4100.

3.5.4.4 *Bolts subject to combined shear and tension* Bolts subject to shear and tension shall be designed in accordance with AS 4100 except that V_f shall be in accordance with Clause 3.5.4.2.

3.5.4.5 *Ply in bearing* A ply subject to a design bearing force (V_b^*) due to a bolt in shear shall satisfy the following:

$$V_b^* \leq \phi V_b \qquad \ldots 3.5.4.5(1)$$

where

ϕ = capacity factor (see Table 1.8)

V_b = nominal bearing capacity of a ply

The nominal bearing capacity of a ply (V_b) shall be calculated as follows:

$$V_b = 2.25 d_f t_p f_y \qquad \ldots 3.5.4.5(2)$$

where

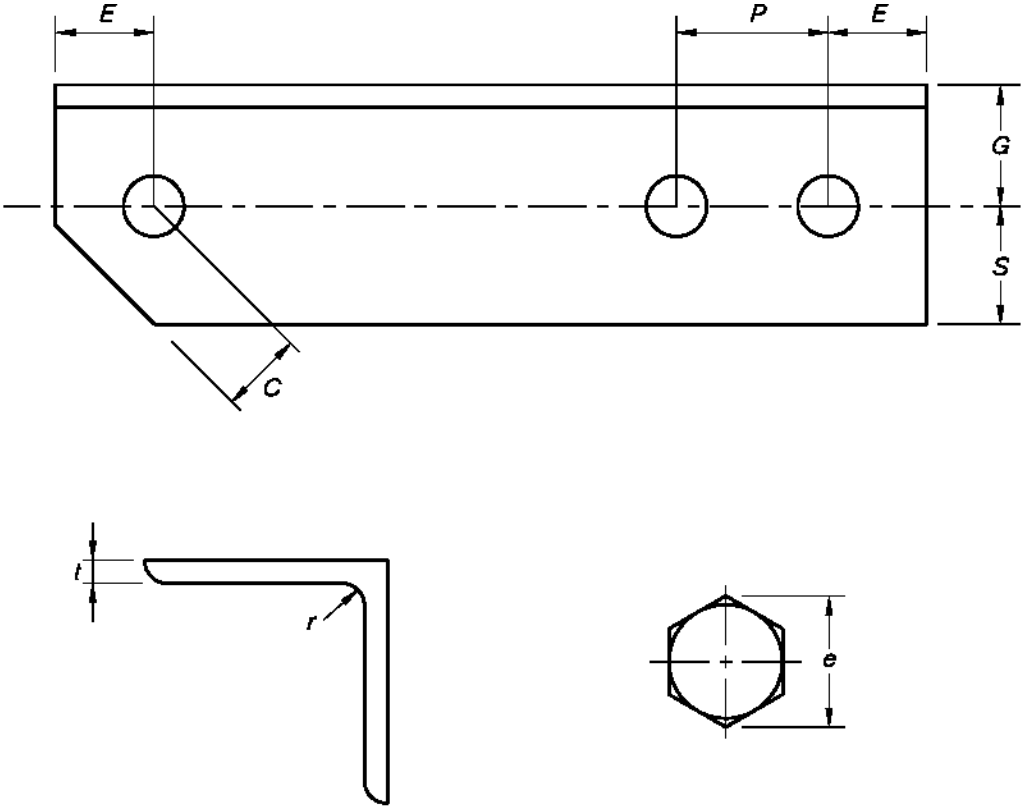
d_f = nominal diameter of the bolt

t_p = thickness of the ply

f_y = specified yield stress of the ply

3.5.4.6 *Detail requirements* The detail requirements shall be as shown in Figure 3.5.4.6 for hot-rolled sections and plates that are machine flame cut, sawn or sheared, or have planed edges. Gusset plates shall have a minimum edge distance equal to the end gauge (E) as shown in Figure 3.5.4.6.

For other sections or cutting methods, the minimum edge distances shall be in accordance with AS 4100.



millimetres

Nominal bolt diameter (d_f)	Hole diameter	Pitch (P) (min.)	End gauge (E) (min.)	Cut edge (C) (min.)	Back gauge (G) (min.)	Side gauge (S) (min.)
12	13.5	30	20	16		18
16	17.5	40	26	22		22
20	22	50	32	28	$t + \frac{2}{3}r + \frac{e}{2}$	27
24	26	60	38	33		32
30	32	75	47	42		40

FIGURE 3.5.4.6 DETAIL REQUIREMENTS

SECTION 4 FOOTING DESIGN

4.1 GENERAL Footings for freestanding lattice towers and guyed masts shall be designed to resist all load combinations applied to the structures in both vertical and horizontal directions combined with bending moments where appropriate, particularly at the ground line of freestanding lattice towers.

A rational analysis shall be used to determine the strength and deflection of the footing.

4.2 PERFORMANCE OF FOOTINGS The performance of the footing under short-term and long-term loadings shall be considered. The resulting strength and deflection shall be considered in the design of the lattice tower.

The effects of variation of water content of the soil shall be taken into account and shall include the allowance for reduction in the weight of materials due to buoyancy.

4.3 SOIL PROPERTIES A site investigation to determine the soil properties required for the design of footings shall be carried out.

NOTE: Guidance for footing design is given in Appendix I.

SECTION 5 CRITERIA FOR ASSESSMENT OF EXISTING STRUCTURES

5.1 STRUCTURAL ASSESSMENT Existing steel lattice towers and masts, and other supporting structures, shall be assessed when changes occur to—

(a) the antenna or ancillary conditions leading to a loading variation; and

(b) the operational requirements, e.g. twist, sway.

5.2 GENERAL DESIGN REQUIREMENTS Before an existing structure is modified or additional loads are applied to it, the following shall be considered:

(a) The physical condition and loading of the structure shall be verified by inspection.

(b) The structural adequacy of the structure shall be evaluated in accordance with this Standard.

The application of a reduced importance factor (see Clauses 1.5 and 1.6.5) may be considered when a reduction in design life or reliability of the structure is tolerable.

NOTE: The cost to modify an existing structure to comply with this Standard may be considered excessive in some instances, especially when the structure has survived quite well for some years. In these cases and where the reliability of the structure has been proven, it is considered appropriate to introduce a reduced importance factor concept into the evaluation of existing structures.

APPENDIX A
MAINTENANCE AND INSPECTION
(Informative)

A1 GENERAL The reliability of a structure depends on the quality of materials and construction, and the adequacy of maintenance provided after installation. This Appendix provides a general framework for maintenance and inspection.

A maintenance program will involve planned inspections and repairs during the life of the structure. Records should be kept of all inspections, modifications, facilities and equipment placement, and repairs carried out.

A2 SCOPE OF MAINTENANCE INSPECTION The scope of maintenance and inspection should cover all aspects that are relevant to the maintenance of structural integrity, and to the serviceability of the structure and its communications functions. The inspections should cover, as far as possible, the following:

(a) Loose or missing bolts.

(b) Fatigue cracking.

(c) Damage from structural overload.

(d) Vandalism (including rifle damage).

(e) Corrosion of galvanized steelwork.

(f) Degradation of paint systems.

(g) Vibration.

(h) Lightning damage.

(i) Foundation deterioration and cracking.

(j) Loose or damaged guy wires and fittings.

(k) Ground surface erosion.

(l) Evidence of soil creep or landslides.

(m) Settlement.

(n) Earthing integrity.

(o) Auxiliary antennas, mountings and feed systems.

(p) Maintenance of safety facilities.

(q) Site security.

(r) Guyed mast verticality and twist.

(s) Navigation lighting.

(t) Condition of insulators.

A3 MAINTENANCE INSPECTION FREQUENCY The inspection intervals need to be tuned to the operational environment and structural/service functional needs. Structures that have known vibrational problems, or are in a very corrosive environment, or are in a very windy or ice environment may need more frequent inspections.

The interval between maintenance inspections in particular will depend on factors such as—

(a) corrosion potential of the environment and the degree of protection required for maintenance of design reliability;

(b) importance of the structure to its service;

(c) severity of local conditions (i.e. wind, ice);

(d) sensitivity to structural response; and

(e) influence of ground conditions.

It is recommended that the interval between inspections should be between two and five years according to the relative importance of the above factors.

A4 SCHEDULED MAINTENANCE AND REPAIRS Maintenance and repair tasks should be undertaken by experienced crews with the appropriate equipment. The replacement of any structural members should be approached with caution and an engineering valuation may be necessary before work commences.

On guyed masts, variation in guy tensions may be critical to the performance of the facility. Where inelastic construction stretch is not removed from guys prior to installation, it may be necessary that retensioning be undertaken at the end of 12 to 18 months after construction. Guy tensions should be maintained to within ±5% of the design values.

A5 ELECTRICAL HAZARDS Maintenance crews should be aware of any electrical hazards, particularly radiation, while undertaking work on communication structures. Advice on these aspects should be sought from the site owner and referenced to AS 2772.

APPENDIX B

ACCESS TO STEEL LATTICE TOWERS AND MASTS

(Informative)

B1 GENERAL This Appendix gives recommendations for access to steel lattice towers and masts.

NOTE: For general requirements of platforms, walkways, ladders and guardrailing, see AS 1657.

B2 DESIGN The design of the structural work comprising the platform, walkways, stairways, ladders and guardrailing should satisfy the relevant requirements of the following Standards:

(a) For aluminium, see AS 1664.

(b) For steel, see AS 1538, AS 4100 or this Standard.

B3 DESIGN LOADING Design loading should follow the requirements of AS 1657.

B4 CLASSIFICATION OF STRUCTURE A structure should be classified as Class A, B or C taking into account the frequency of access as well as the experience and qualifications of the personnel requiring access to the structure.

B4.1 Class A A structure should be classified as Class A where inexperienced personnel may require frequent access to the structure, e.g. lookout towers.

B4.2 Class B A structure should be classified as Class B where experienced operating, inspection, maintenance and servicing personnel may require infrequent access to the structure, e.g. communication and television towers.

B4.3 Class C A structure should be classified as Class C where only experienced and qualified riggers require access to the structure, e.g. guyed masts. Personnel who are not riggers but have undertaken a training course accepted by a regulatory authority may be permitted access to these structures if accompanied by another suitably experienced and qualified person.

B5 ACCESS RECOMMENDATIONS

B5.1 Class A structures These structures should satisfy the requirements of AS 1657. For structures which are unable to meet the requirements of AS 1657, advice should be sought from the regulatory authority.

B5.2 Class B structures These structures should satisfy the requirements of AS 1657, with the following substituted recommendation:

(a) *Toe-boards* The top of the toe-boards should not be less than 75 mm above the top of the floor.

(b) *Angle of slope of rung ladders* The angle of slope of rung ladders should not be less than 70° to the horizontal and should not overhang the person climbing the ladder.

(c) *Distance between landings for rung ladders* Except where it is not practicable to provide an intermediate landing, the vertical distance between landings should not exceed 15.0 m.

(d) *Ladder cage* A ladder cage as shown in Figure B1 (as a minimum) should be provided where a person may fall more than 6.0 m. However, a regulatory authority accepted anti-fall device may be used on the ladder to take the place of the ladder cage. Where a protective device is fitted, a guard or anti-climbing device should be placed over the first 2.5 m of the ladder and should be locked to prevent use by unauthorised personnel. It should be provided with the notice: **'This ladder is to be used only by personnel using the anti-fall device.'**

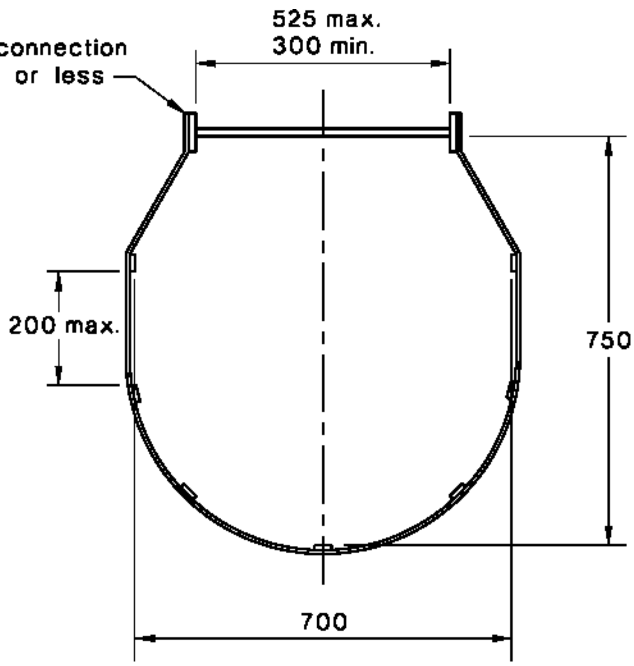
Where a cage is provided, the area between the cage and the top of the guardrailing need not be guarded.

(e) *Ladder width* The clearance between stiles should not be less than 300 mm but not more than 525 mm.

(f) *Rungs* Rungs should not be less than 16 mm diameter low carbon steel. Ladders with a stile width greater than 325 mm should have rungs not less than 20 mm diameter low carbon steel.

NOTE: Class B structures which have an internal rung type ladder and a tower width of less than 1.5 m need not have a ladder cage fitted.

B5.3 Class C structures These structures are not required to meet the requirements of AS 1657. A ladder should be provided with a minimum clearance between stiles of 260 mm. Alternatively, step bolts may be fitted to a corner leg member of the structure provided that the vertical climbing distance between rest points does not exceed 50 m and the height of the structure does not exceed 150 m. The step bolt should be in accordance with AS 1559.



DIMENSION IN MILLIMETRES

FIGURE B1 CLEARANCE DIAGRAM FOR LADDER CAGE (CLASS B STRUCTURES)

APPENDIX C

GUIDANCE FOR EARTHQUAKE DESIGN

(Informative)

Steel lattice towers and masts for communication purposes should be evaluated as Type III structures, as defined in AS 1170.4.

Steel lattice towers and masts are less sensitive to earthquake loads than most other types of structure. The following general guidance is provided:

(a) Freestanding lattice towers up to 100 m high and having insignificant mass concentrations less than 25% of their total mass need not be designed for earthquake.

(b) Freestanding lattice towers of high mass and over 100 m high or of lesser height with significant mass concentrations may experience base shears and base overturning moments approaching ultimate wind induced actions.

(c) Freestanding lattice towers and guyed steel masts that are in earthquake Design Categories C, D and E (see AS 1170.4) require vertical components of ground motion to be considered for design. For very tall guyed masts, some vertical ground motion differentials between mast base and guy anchorage points may be an important design consideration depending on local seismicity.

(d) Footing ties or other means may be provided to limit differential horizontal movement of footings for structures of Design Categories C, D and E on soft soil in accordance with AS 1170.4.

APPENDIX D

ESTIMATION OF THE FIRST MODE NATURAL FREQUENCY

(Informative)

An accurate method of estimating the natural frequencies of vibration of a lattice tower is to obtain a flexibility matrix (preferably with the use of a frame analysis program), then perform an eigenvalue calculation in conjunction with its mass matrix. This process gives not only the first mode frequency but also the frequencies of all other modes.

For preliminary estimates, the following approximations are suggested:

(a) For freestanding lattice towers, the first mode natural sway frequency (n) may be calculated from the following equation:

$$n = \frac{1500\, w_a}{h^2} \qquad \text{. . . D1}$$

where

w_a = average width of the structure, in metres

h = height of the structure above ground, in metres

For structures with a lumped mass (m) near the top, the first mode natural sway frequency (n_1) may be calculated from the following equation:

$$n_1 = n\sqrt{\frac{M_1}{M_1 + m}} \qquad \text{. . . D2}$$

where

n = first mode natural sway frequency of the structure without the lumped mass

M_1 = generalized mass of the structure calculated approximately from the following equation:

$$M_1 = \frac{M_2}{3}\left[\left(\frac{w_a}{w_b}\right)^2 + 0.15\right] \qquad \text{. . . D3}$$

M_2 = total mass of structure

w_a = average width of the structure

w_b = width at the base of the structure

m = added lumped mass

(b) For guyed masts, the first mode natural sway frequency in a direction parallel to that of the design mean wind speed ($\bar{V}_z$) at height z may be calculated approximately from the following equation:

$$n = 0.15\sqrt{\left(\frac{(k_1 + k_N)(N + 1)}{\left\{N\left[M_T + \sum_{j=1}^{N_A} 5M_{Aj}\left(\frac{y_j}{h}\right)^4\right]\right\}}\right)} \qquad \text{. . . D4}$$

where

k_1, k_N = minimum stiffness for the first and last guy cluster systems on the mast for any wind direction

N = number of guy levels

M_T = total mass of the mast section and half the total mass of all guy clusters attached to it

N_A = number of ancillaries, e.g. antennas

M_{Aj} = individual mass of ancillaries

y_j = heights of ancillaries above ground

h = height of mast above ground

APPENDIX E
GUIDANCE FOR DETERMINATION OF WIND LOADS
(Informative)

E1 GUST WIND SPEED The ultimate limit state basic wind speeds given in Figure 2.2 are identical to those specified in AS 1170.2. The ultimate limit state gust wind speed (V_u) has an estimated 5% probability of being exceeded in a 50-year period.

Serviceability criteria for communication and broadcasting towers are determined by—

(a) antenna specifications and service parameters;

(b) availability of standby facilities; and

(c) importance of the service.

Typically, ‘outages’ are required to be limited to 0.1% of the time for broadcasting (TV, radio) services (about 9 hours per year), and 0.001% for telecommunication services (about 5 minutes per year).

The value of the basic wind speed for serviceability (V_s) should be appropriate to the serviceability criterion under consideration. The variation of serviceability wind speeds across Australia is small, since the contribution from extreme storms such as tropical cyclones and severe thunderstorms is small. The value of gust wind speed exceeded for 0.1% of the time is approximately 20 m/s for all regions. Generally, a value of 27 m/s, which represents the value of basic gust wind speed exceeded for approximately 0.001% of the time, is used for telecommunication purposes. However, local wind speed information to determine the required probabilities of being exceeded should be utilized when available.

The regions given in Figure 2.2 are similar to those in AS 1170.2. Regions A1, A2, A3 and A4 differ only in their wind direction multipliers (see Clause 2.2.5).

Further background information on the derivation of the basic wind speeds is given in Appendix E of AS 1170.2, and in the Commentary to AS 1170.2 (HOLMES, MELBOURNE AND WALKER (Ref. 4)(see Appendix J)).

E2 TERRAIN AND HEIGHT MULTIPLIER The terrain and height multipliers allow for the increase in wind speed with height in the atmospheric boundary layer, for various terrain types. The terrain categories in Clauses 2.2.3.1 and 2.3.3.1 are the same as those specified in AS 1170.2. Most towers will be erected in homogeneous terrain for several kilometres upwind, but the most important terrain is that in a range of distances from 10 to 20 times the height of the tower upwind. The terrain in the range from 0 to 10 times the tower height only affects the wind loads on the bottom half of the tower. The terrain beyond 20 times the tower height upwind affects the loads on the tower, but the influence diminishes with increasing distance upwind of the tower.

E2.1 Terrain multiplier The multipliers in Table 2.2.3.2(1) for non-cyclonic winds are based on DEAVES and HARRIS model (Ref. 5)(see Appendix J), of the velocity profiles in the atmospheric boundary layer in strong winds (gales).

The following expressions are good approximations to the values in Table 2.2.3.2(1) for Terrain Categories 2 and 3.

Terrain Category 2: $M_{(z,2)} = 0.10 \ln z + 0.0001z + 0.77$ . . . E2.1(1)

Terrain Category 3: $M_{(z,3)} = 0.125 \ln z + 0.00025z + 0.55$ . . . E2.1(2)

The multipliers for cyclonic gust wind speeds in Table 2.2.3.2(2) are representative of those believed to occur in the zone of maximum winds in tropical cyclones. Approximate expressions for the multipliers in Table 2.2.3.2(2) are:

Terrain Categories 1 and 2: $M_{(z,\ cat)} = 0.15\ln z + 0.0003z + 0.67$ . . . E2.1(3)

Terrain Categories 3 and 4: $M_{(z,\ cat)} = 0.20\ln z + 0.0007z + 0.43$ . . . E2.1(4)

Equations E2.1(1) to E2.1(4) are applicable for values of between 10 and 100 m.

Further background information on the wind models used in this Standard and in AS 1170.2 is given in Chapter III of the Commentary to AS 1170.2 (HOLMES, MELBOURNE AND WALKER (Ref.4)(see Appendix J)).

E3 TOPOGRAPHIC MULTIPLIER In Clause 2.2.4, a method for the calculation of gust wind speeds, which differs from that specified in AS 1170.2, has been given. The new method is based on the numerical computation of boundary-layer flow over two-dimensional hills (escarpments, embankments and ridges) of various upwinds slopes (PATERSON AND HOLMES (Ref. 6)(see Appendix J)). The computations were validated against measurements in both full-scale and wind-tunnel for shallow upwind slopes. The accuracy may not be as good for steep upwind slopes greater than 0.3, where flow separation often occurs both upwind and downwind of the crest. The variation of the multiplier with height above the ground, given by Equation 2.2.4, is more realistic than the linear variation with height implied in AS 1170.2.

Where there is another crest of equal or greater height upwind of that of the hill of interest, Clause 2.2.4 can overestimate the topographic multiplier considerably. In some cases, values of M_t less than 1.0 can occur. Designers may take advantage of this by referring to appropriate wind-tunnel test measurements or by taking specialist advice.

To determine hill height where an undulating terrain approach exists (see Figure E1), the following procedure is recommended:

(a) Locate crest of hill.

(b) Determine average slope over a series of 500 m segments upwind of the crest.

(c) Locate the start of the hill (point P) at the downwind end of the first segment for which the average upwind slope (ϕ) is less than or equal to 0.05.

(d) Determine the vertical distance H between points P and C.

(e) Proceed with calculation in accordance with Clause 2.2.4.

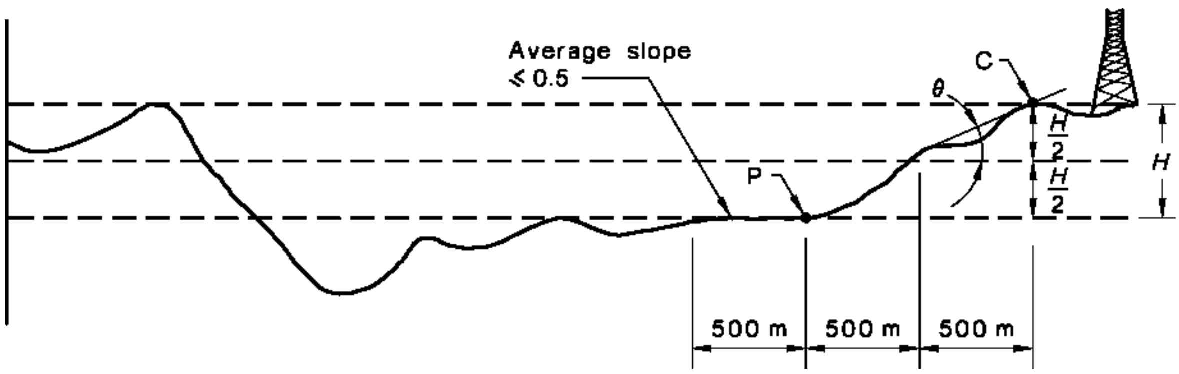


FIGURE E1 DERIVATION OF TOPOGRAPHIC MULTIPLIER

E4 WIND DIRECTION MULTIPLIER The multipliers given in Table 2.2.5 allow for the reduced probability of exceedence of extreme winds occurring from some directions for those regions where there is sufficient meteorological data.

For Regions A1, A2 and A3, the directional characteristics of extreme winds are known, and the values of the multipliers allow for the varying probability of winds exceeding the basic wind speed with wind direction. For Regions A4, B, C and D, the directional characteristics are not known, and the value of 0.95 is a statistical reduction factor for these regions, as used in AS 1170.2.

Designers should select those wind directions that produce the worst load effects on the structure.

E5 TOWER SECTIONS WITH ANCILLARIES The method specified in Item (b) of Clause 2.2.8.3 enables the total effective drag of a tower section with ancillaries to be calculated.

It is assumed that the drag coefficient for the isolated ancillary (C_{da}) is defined with respect to a reference area of the ancillary (A_a). In Equation 2.2.8.3(2), the drag coefficient of the ancillary is adjusted to the reference area for the tower section (A_z). Note that both A_a and A_z are independent of wind direction.

The expressions for K_{in} in Equations 2.2.8.4(1) to 2.2.8.4(8) are similar to those used by ENGINEERING SCIENCES DATA UNIT (Ref. 7)(see Appendix J). They allow a reduction in total effective drag when the tower shields the ancillary, or vice-versa. These expressions are probably more accurate when the ancillary is inside the tower section, as they depend on the drag coefficient and solidity for the tower section without ancillaries, but not on the drag or solidity of the ancillaries.

In some situations with ancillaries attached to the face of the tower, Equations 2.2.8.4(1) and 2.2.8.4(2) can provide interference factors that are greater than one. This is consistent with locally higher wind speeds around the ancillary producing increased loads on part of the tower (HOLMES, BANKS AND ROBERTS (Ref. 8)(see Appendix J)).

E6 CROSS-WIND RESPONSE The main source of cross-wind excitation is vortex shedding, which will only occur when a tower section has a high degree of solidity. However significant cross-wind motions, at relatively low wind speeds, have occurred for sections of UHF antennas at the top of broadcasting towers or masts. The ‘critical’ mean wind speed is the speed at which the frequency of vortex-shedding coincides with a natural mode frequency of the tower. Turbulence in the natural wind, in the form of wind direction changes, may also produce cross-wind excitation, even on porous lattice towers, but this is usually much less than the along-wind excitation.

Estimation of cross-wind response should normally be carried out by specialists in wind engineering.

E7 GUST RESPONSE FACTOR FOR FREESTANDING LATTICE TOWERS The Clause treats the along-wind dynamic response of free-standing lattice towers in a similar, but not identical, way to AS 1170.2. The approach is based on the stochastic response of a linear single-degree-of-freedom system (i.e. a tower assumed to respond in its first mode of vibration), excited by a random wind force of known spectral density. The gust response factor computed in the Clause is a multiplier on the actions of the mean wind (averaged over 10 minutes to 1 hour).

The equation for the gust response factor (Equation 2.3.8(1)) contains two dynamic terms, one for background effects and one for resonant effects. The response of the structure due to ‘background’ (i.e. sub-resonant) dynamic forces is the first term under the square root sign in Equation 2.3.8(1), and accounts for the quasi-static dynamic response below the natural frequency. The second term under the square root is an estimate of the amplified contribution response at the resonant frequency, and depends on the gust energy and aerodynamic admittance at the natural frequency, and on the damping ratio for the structure. The latter may include an allowance for aerodynamic damping but, because of the uncertainty in estimating structural damping, the aerodynamic damping is often neglected.

The resonant contribution is small for structures with natural frequencies greater than 1 Hz. The gust response factor for structures with small frontal area then approaches the square of the ratio of peak gust wind speed to the mean wind speed, i.e. the dynamic method will give similar loads to the static method. For structures with large frontal dimensions, the reduction produced by a low background factor (B_s) may result in the dynamic analysis giving lower loads than the static analysis.

The gust response factor increases with increasing height on the structure. This effect is incorporated into the height factor (H) and in the background factor (B_s). The effect of mode shape variations is small (HOLMES (Ref. 3)(see Appendix J)), and is not incorporated into the methods of the Clause.

APPENDIX F

DRAG FORCE COEFFICIENTS (C_{da}) FOR ANCILLARIES AND ASPECT RATIO CORRECTION FACTORS (K_{ar})

(Normative)

F1 INTRODUCTION This Appendix specifies drag force coefficients (C_{da}) and aspect ratio correction factors (K_{ar}) for ancillaries.

F2 ROUNDED CYLINDRICAL SHAPES Drag force coefficients (C_{da}) for rounded cylindrical sections shall be obtained from Table F1. For intermediate values of $V_z b$, linear interpolation is permitted.

NOTES:

1 Cables may also experience small cross-wind (lift) forces. For further information, see SACHS (Ref. 9)(see Appendix J).

2 The values in Table F1 are derived from wind-tunnel tests described by DELANEY and SORENSEN (Ref. 10)(see Appendix J).

TABLE F1

DRAG FORCE COEFFICIENTS (C_{da}) FOR ROUNDED CYLINDRICAL SHAPES

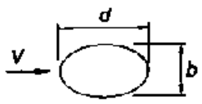
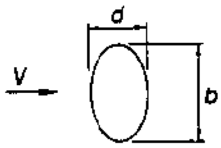
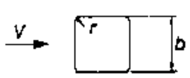
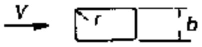
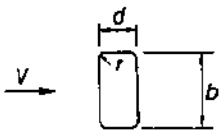
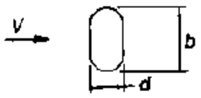
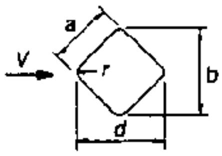
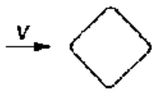
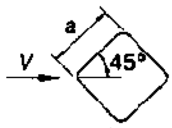
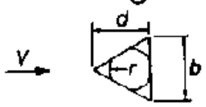
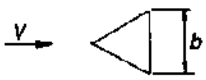
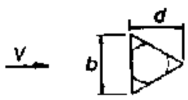
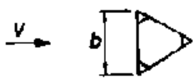
Cross-sectional shape		Drag force coefficient (C_{da})	
		$V_z b$ < 3 m²/s	$V_z b$ > 6 m²/s
	Rough or with projections	1.2	1.2
	Smooth	1.2	0.6
	Ellipse $\frac{b}{d} = \frac{1}{2}$	0.7	0.3
	Ellipse $\frac{b}{d} = 2$	1.7	1.5
	$\frac{b}{d} = 1$ $\frac{r}{b} = \frac{1}{3}$	1.2	0.6
	$\frac{b}{d} = 1$ $\frac{r}{b} = \frac{1}{16}$	1.3	0.7
	$\frac{b}{d} = \frac{1}{2}$ $\frac{r}{b} = \frac{1}{2}$	0.4	0.3

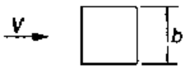
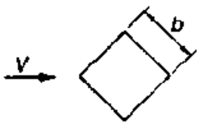
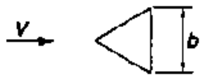
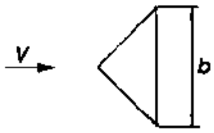
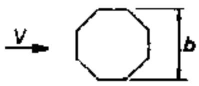
TABLE F1 (*continued*)

Cross-sectional shape		Drag force coefficient (C_{da})	
		$V_z b < 3$ m²/s	$V_z b > 6$ m²/s
(*continued*)			
	$\frac{b}{d} = \frac{1}{2}$ $\frac{r}{b} = \frac{1}{6}$	0.7	0.7
	$\frac{b}{d} = 2$ $\frac{r}{b} = \frac{1}{12}$	1.9	1.9
	$\frac{b}{d} = 2$ $\frac{r}{b} = \frac{1}{4}$	1.6	0.6
	$\frac{r}{a} = \frac{1}{3}$	1.2	0.5
	$\frac{r}{a} = \frac{1}{12}$	1.6	1.6
	$\frac{r}{a} = \frac{1}{48}$	1.6	1.6
	$\frac{r}{b} = \frac{1}{4}$	1.2	0.5
	$\frac{r}{b} = \frac{1}{12}$	1.4	1.4
	$\frac{r}{b} = \frac{1}{48}$	1.3	1.3
	$\frac{r}{b} = \frac{1}{4}$	1.3	0.5
	$\frac{1}{12} > \frac{r}{b} > \frac{1}{48}$	2.1	2.1

F3 SHARP-EDGED PRISMS Drag force coefficients (C_{da}) for sharp-edged sections, except for rectangular cylinders, shall be obtained from Table F2.

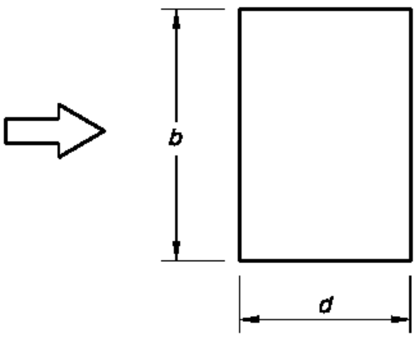
TABLE F2

DRAG FORCE COEFFICIENTS (C_{da}) FOR SHARP-EDGED PRISMS

Sectional shape		Drag force coefficient (C_{da})
	Square with face to wind	2.2
	Square with corner to wind	1.5
	Equilateral triangle—apex to wind	1.2
	Equilateral triangle—face to wind	2.0
	Right-angled triangle	1.55
	12-sided polygon	1.3
	Octagon	1.4

F4 RECTANGULAR PRISMATIC SECTIONS Drag force coefficients (C_{da}) for rectangular cross-sections shall be obtained from Figure F1.

NOTE: The data in Figure F1 are derived from tests by JANCAUSKAS (Ref. 11)(see Appendix J). The large value that occurs for sections with d/b equal to about 0.65 was first reported by NAKAGUCHI, HASHIMOTO and MUTO (Ref. 12)(see Appendix J).



Side ratio (d/b)	Drag force coefficient (C_{da})
0.1	2.2
0.65	3.0
1	2.2
2	1.6
4	1.3
10	1.1

NOTE: For intermediate values of d/b, linear interpolation is permitted.

FIGURE F1 ALONG-WIND FORCE COEFFICIENTS FOR RECTANGULAR PRISMS

F5 ASPECT RATIO CORRECTIONS The aspect ratio correction factors (K_{ar}) shall be obtained from Table F3.

NOTE: When the aspect ratio of a structure or structural member decreases, the air flow around the ends of the structure or structural member, is facilitated. This additional air path reduces the magnitude of the average force on a section.

TABLE F3

ASPECT RATIO CORRECTION FACTORS (K_{ar})

Aspect ratio (l/b)	Correction factor (K_{ar})
≤8	0.7
14	0.8
30	0.9
≥40	1.0

NOTE: For intermediate values of l/b, linear interpolation is permitted.

F6 DISH ANTENNAS The drag force coefficients (C_{da}) for dish antennas may be obtained from Table F4, in the absence of more accurate wind tunnel data.

TABLE F4

DRAG FORCE COEFFICIENTS (C_{da}) FOR SOLID UNSHROUDED DISH ANTENNAS (based on projected area normal to the axis)

Angle of attack (θ_a)	Drag force coefficient (C_{da})
0° to 40°	1.4
80° to 120°	0.4
160° to 180°	1.0

NOTES:

1 For intermediate wind directions, linear interpolation is permitted.

2 Drag force coefficient for other dish antennas can be obtained from manufacturers' data or special wind tunnel tests.

3 Cross-wind forces may be significant on dish antennas for some wind direction. Values of cross-wind force coefficients can be obtained from manufacturers' data or special wind tunnel tests.

APPENDIX G

GUIDANCE FOR STRUCTURAL ANALYSIS AND DESIGN

(Informative)

G1 FREESTANDING LATTICE TOWERS For conventional towers, a first order linear elastic analysis is usually an adequate method for load predictions. Highly flexible towers, where the displacement causes additional forces above those calculated by a first order linear elastic analysis, may require non-linear analysis. It is likely that a dynamic analysis of such structures may be necessary to accurately predict the structural response.

For square or rectangular tower structures over 60 m high, it is recommended horizontal plan bracings be installed at intervals not exceeding approximately 20 m.

Loading due to eccentric ancillaries should also be considered.

G2 GUYED MASTS The static or quasi-steady analysis approach is satisfactory for most conventional guyed lattice structures. The mast and guy modelling should allow for the following effects:

(a) Variation of guy stiffness with applied loads.

(b) Modelling true shape and wind loads on guys.

(c) Influence of axial forces on column bending.

(d) Torsion in column and guys due to eccentric ancillaries.

(e) Eccentric loading due to differential guy tensions at maximum wind loads and displacement.

(f) Any lack of concentricity in guy and mast connection detailing.

Initial guy tensions should be set such that the guys maintain column stability in both the longitudinal, i.e. in direction of the wind, and transverse directions. Cross-wind loads may become important when the leeward guys are slack.

For very small, lightly loaded structures, the effect of guy sag may be ignored in the guy analysis. It is considered that the likely variation in predicted load may be in the order of 30% under this condition.

Superposition of separate load effects cannot be undertaken and load combinations themselves have to be analysed individually.

It is recommended that the quasi-steady based design is checked for the dynamic wind load effects. Guyed masts fall into this category when—

(i) the first mode natural frequency is less than 1 Hz;

(ii) significant concentrations of mass combined with a relatively slender column occurs;

(iii) significant wind sensitive ancillary facilities are supported, e.g. bluff-bodied antennas); and

(iv) masts are in areas of large ice build up.

Special skills and considerable expertise are necessary to recognize the potential for, and to undertake a full investigation of the problems in these structures. Some guidance to the parameters to be considered is provided by the International Association for Shell and Spatial Structures (Ref. 13)(see Appendix J).

Guyed masts are essentially wind-sensitive structures and wind-induced vibrations may occur. The fatigue potential of susceptible elements should be considered, e.g. some guy links or anchorages.

G3 SINGLE BOLTED COMPRESSION MEMBERS In normal practice, single bolted compression members with low *l/r* values (less than 80) rarely occur. Furthermore, the normal tower bolt capacity governs the design for those low *l/r* members. If the designer chooses to use high strength bolts with single bolted low *l/r* members, the use of Clause 3.3 may lead to unconservative design. ELGAALY, DAGHER AND DAVID, (Ref. 14)(see Appendix J), highlights this effect.

G4 EFFECTIVE SLENDERNESS RATIOS FOR COMPRESSION MEMBERS The rules of ASCE, Manuals and Reports on Engineering Practice, No. 52, Guide for Design of Steel Transmission Towers are used in determining the effective slenderness ratio (λ_e) for a member in compression.

The effective slenderness ratio for leg members is taken as equal to the slenderness ratio (L/r), provided that there is concentric load in the member.

In determining the effective slenderness ratio for bracing members, the eccentricity of the connection is the dominant factor in the lower range of L/r, and the rotational restraint of the end of the member is the dominant factor in the higher range of L/r.

The following explanations are given for the various terms used in Clauses 3.3.4.3 and 3.3.4.4:

(a) *Concentric load* It implies either the member is continuous or bolted on both faces of the angle.

(b) *Normal framing eccentricity* It implies that the member is bolted on one face of the angle only and the centroid of the bolt pattern is located between the centroid of the angle and the centre-line of the connected leg. If the joint eccentricity exceeds this criterion, consideration should be given to additional stresses introduced in the member.

(c) *Unrestrained against rotation* (See Clause 3.3.4.4).

(d) *Partially restrained against rotation* The restrained member is to be connected to the restraining member with at least two bolts and furthermore, the restraining members are to have a stiffness about the appropriate axis greater than or equal to the sum of the stiffnesses of the restrained members that are connected to it.

G5 GUY TENSION MEMBERS There is little guidance in AS 4100 as to the design of guys. The guys are generally required to have a greater level of safety than other tension members. This is due to their sensitivity to damage from storage, handling, transportation and loading. Guys are also sensitive to wind-induced vibrational loading. All effort should be taken to avoid wind-induced oscillations as they may reduce the fatigue life of the guys and their attachments.

G5.1 Nominal section capacity The nominal section capacity of a tension member is significantly affected by the choice of termination fitting used. The correction factor (K_t) applies to the force distribution in a member as specified in AS 4100.

G5.2 Distribution of forces The values of K_t are derived from AS 2759. The lower bound efficiencies have been adopted for the connections thought to be appropriate for guyed steel lattice towers and masts. For more information on the types of connections available and their usage, see AS 2759 or consult the manufacturers of the fittings.

G6 WELDED CONNECTIONS Special consideration to the testing of welds should be considered in the light of—

(a) the type of loading to which welds will be subjected (static, dynamic or fluctuating loads);

(b) the quality of steel proposed to be used; and

(c) the importance of the joint in the structure (whether failure of the joint will cause the whole structure to collapse, e.g. guy attachment joint).

For minimum acceptable levels of welding quality, see AS 1554.

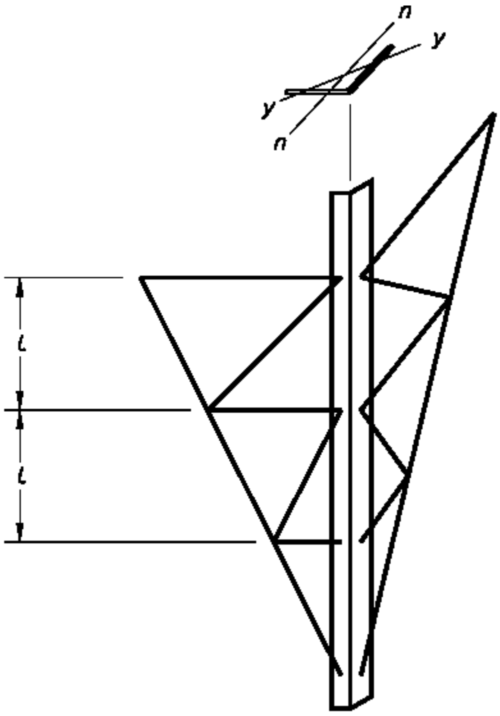
APPENDIX H
SLENDERNESS RATIO FOR COMPRESSION MEMBERS
(Normative)

H1 GENERAL This Appendix specifies slenderness ratios (L/r) for compression members.

H2 LEG MEMBERS The slenderness ratio for leg members shall be as shown in Figure H1.

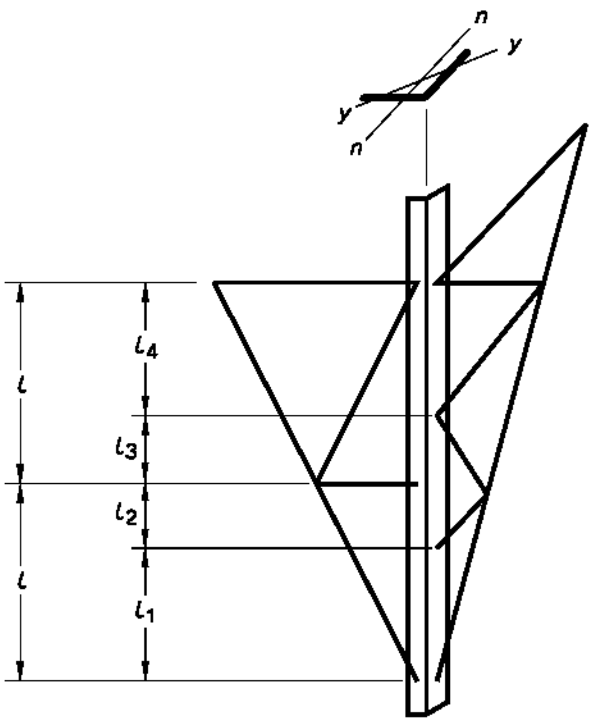
H3 BRACING MEMBERS For members in tension-compression cross bracing systems, the slenderness ratios shall be as shown in Figure H2. In this Figure, N_c^* is the design compression axial force in the brace under consideration and N_t^* is the design tension axial force in the other brace.

For other bracing members, the length (L) used to determine the slenderness ratio shall be the actual unsupported length (l) of the member for buckling about the relevant axis.



$$\frac{L}{r} = \frac{l}{r_{yy}}$$

(a) Symmetrical bracing



The following slenderness ratios shall be considered:

(i) $\frac{L}{r} = \frac{l_{max.}}{r_{yy}}$

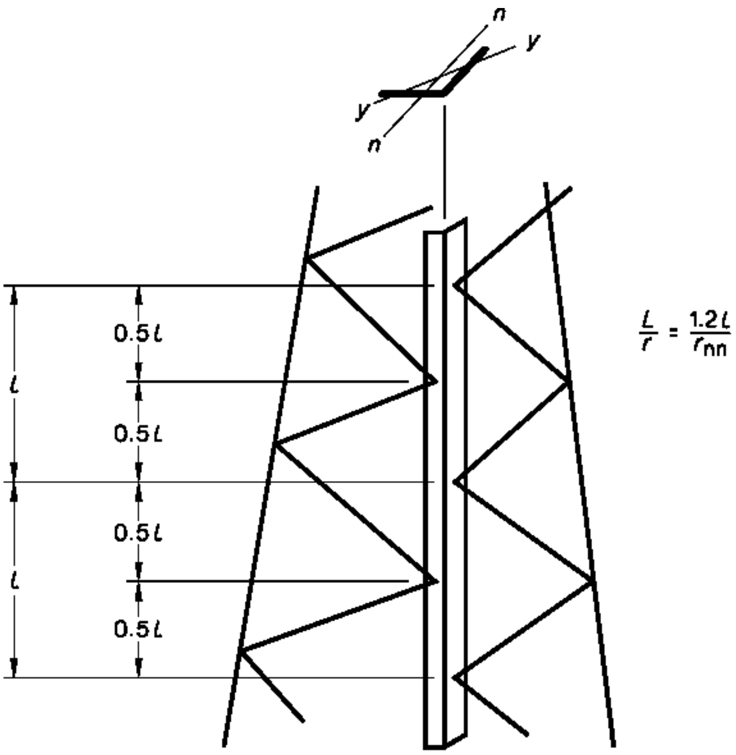
where $l_{max.}$ is the greater of l_1, l_2, l_3 and l_4.

(ii) $\frac{L}{r} = \frac{l}{r_{nn}}$ (see Note).

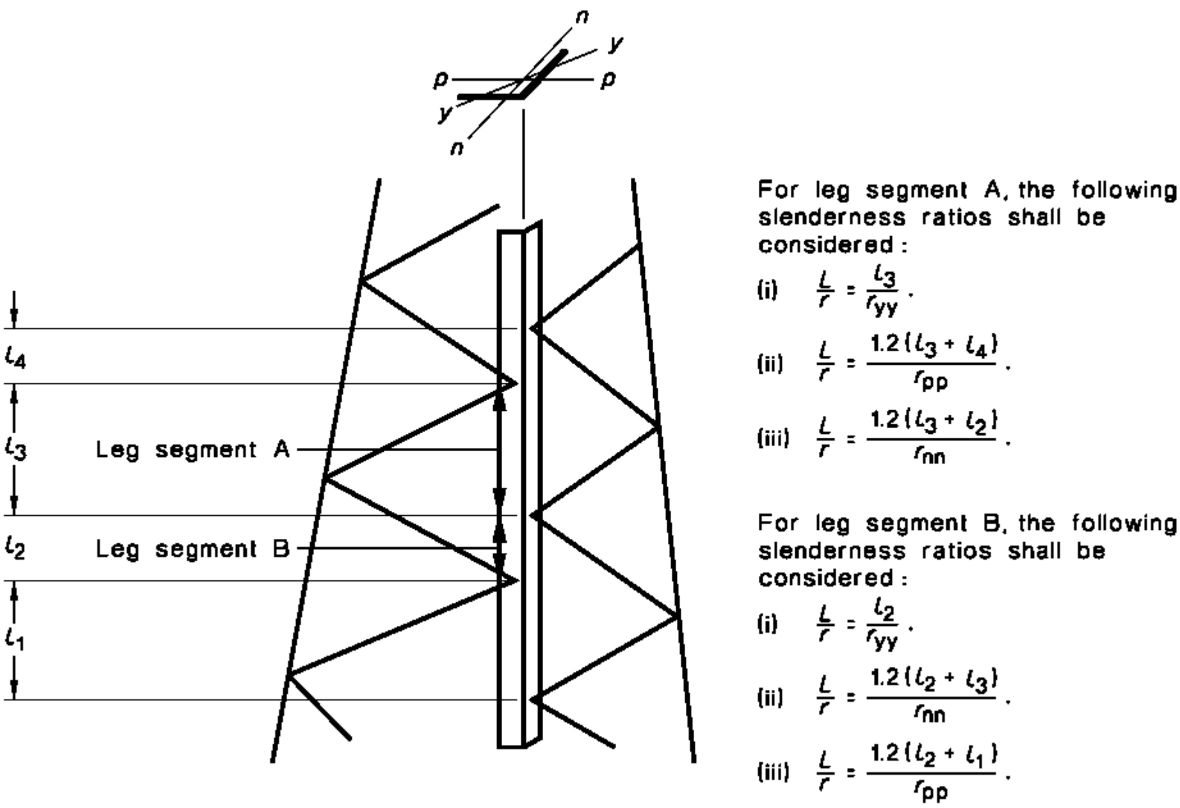
NOTE: This applies where there are no more than three points of staggered bracing between those points at which the leg member is supported by bracing in both faces at the same level.

(b) Partial staggered bracing

FIGURE H1 (in part) SLENDERNESS RATIOS OF LEG MEMBERS

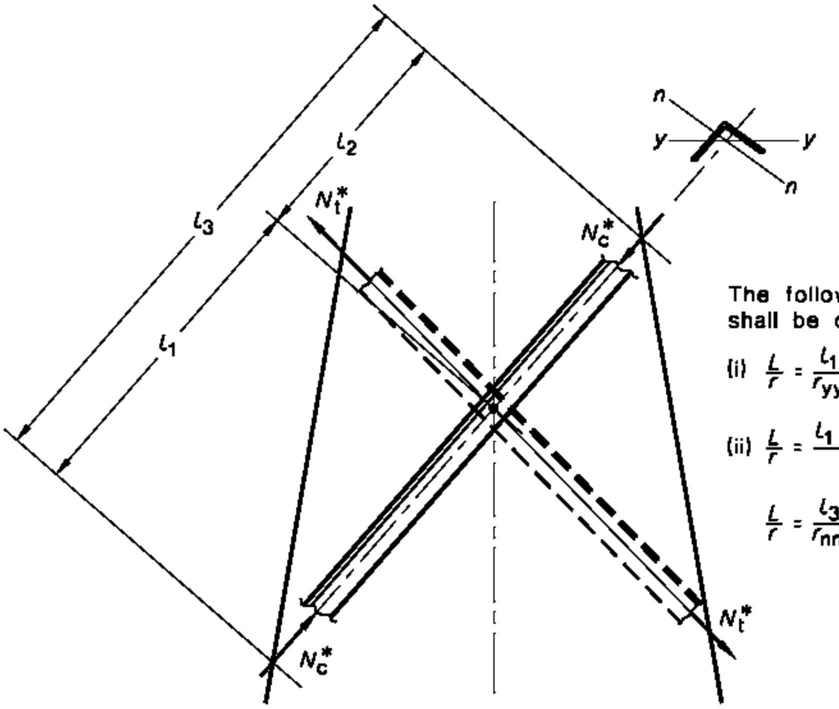


(c) Regular staggered bracing



(d) Irregular staggered bracing

FIGURE H1 (in part) SLENDERNESS RATIO OF BRACING MEMBERS



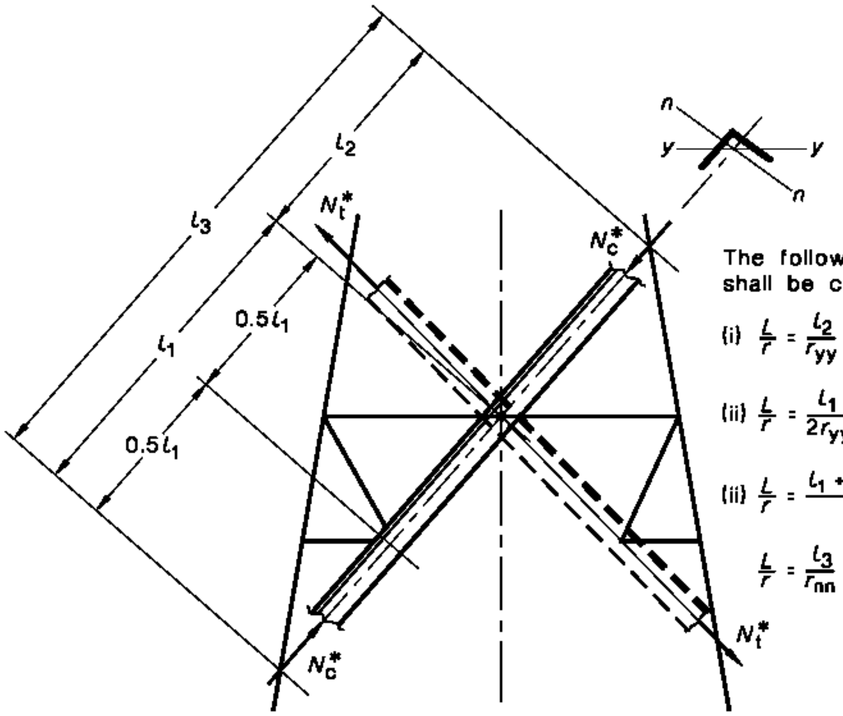
The following slenderness ratios shall be considered:

(i) $\frac{L}{r} = \frac{l_1}{r_{yy}}$.

(ii) $\frac{L}{r} = \frac{l_1 + 0.5 l_2}{r_{nn}}$ if $N_t^* \geqslant 0.6 N_c^*$; or

$\frac{L}{r} = \frac{l_3}{r_{nn}}$ if $N_t^* < 0.6 N_c^*$.

(a) Simple cross bracing



The following slenderness ratios shall be considered:

(i) $\frac{L}{r} = \frac{l_2}{r_{yy}}$.

(ii) $\frac{L}{r} = \frac{l_1}{2 r_{yy}}$.

(ii) $\frac{L}{r} = \frac{l_1 + 0.5 l_2}{r_{nn}}$ if $N_t^* \geqslant 0.6 N_c^*$; or

$\frac{L}{r} = \frac{l_3}{r_{nn}}$ if $N_t^* < 0.6 N_c^*$.

(b) Cross bracing with secondary braces

FIGURE H2 SLENDERNESS RATIO OF BRACING MEMBERS

APPENDIX I

GUIDANCE FOR FOOTING DESIGN

(Informative)

I1 GENERAL Theories used to determine the footing strength and deflection which are based on the characteristic soil properties are preferred.

The truncated cone of earth theory is widely used but is not always reliable, particularly in cohesionless soil or for deep foundations.

The following should also be taken into account when designing footings:

(a) Expansive clays.

(b) Changes in level of water table.

(c) Long-term settlement of fill materials.

(d) Loss of soil around footing.

(e) Build-up of soil around unprotected metal components.

(f) Allowable long-term movement.

(g) Maximum allowable movement at maximum load.

(h) Suitable placement of reinforcement in any concrete to ensure the load-carrying capacity and durability.

(i) Selection of footing type and details to minimize construction difficulties.

(j) Changes to loading on the tower during the life of the structure.

(k) Routing of earth cabling through the foundation and the effect of settlement.

(l) Protection of groundline components from accidental or malicious damage.

(m) Provision of secondary guy anchor points for use during maintenance.

(n) Sloping of all surfaces to avoid any collection of water and waterborne solids.

I2 PERFORMANCE OF FOOTINGS The sides of all footings and guy anchorages should be placed against undisturbed soil wherever possible. Backfill should be compacted in accordance with the assumptions used in the design.

The designer needs to ensure that likely deflections of the foundation (soil and footing) will not be detrimental to the tower.

Consideration needs to be given to—

(a) the variation in the load magnitude and duration;

(b) changes in soil properties, such as drained or undrained; and

(c) changes in the water table level.

There are many suitable references available for guidance.

In most cases, the worst load condition is for short-term maximum load. For most soils, deflection is not critical.

The designer will need to satisfy the requirement to provide a foundation with adequate security. To achieve this, it will often be necessary to take into account the following:

(i) The method used to obtain the soil properties.

(ii) The reliability of the theory used to predict the foundation capacity.

(iii) The degree of supervision during the construction.

(iv) The need to minimize damage to the footing should the tower be overloaded.

(v) Use of another structure, such as a building, to provide the foundation for the tower.

The loads applied to the footing by the tower are those determined from the analysis of the tower. The strength of the footing is assessed on the basis of the degree of certainty of each of the parameters used in the design and construction.

Experience has shown that a mass concrete gravity footing is most suitable for guy footing.

I3 SOIL PROPERTIES The extent of the site investigation will depend on the design methods adopted and the accuracy of information required.

Experience has shown that normal design and construction practices in the transmission industry produce adequate strength factors. The following information is provided as a general guide based on Australian experience:

(a) Loading type: short-term maximum load due to wind.

(b) Dry density: 16 kN/m^3.

(c) Cohesion:

(i) For soft clay: 40 kPa.

(ii) For medium clay: 60 kPa.

(iii) For stiff clay: 80 kPa.

(iv) For very hard clay: 150 kPa.

(d) Internal angle of friction equals 35° in cohesionless soil.

(e) Friction between *in situ* soil and compacted backfill:

(i) For soft cohesive soil (allows for saturated soil): 20 kPa.

(ii) For medium and hard cohesive soil: 40 kPa.

(f) Friction on driven or bored piles in soft and saturated soil: 20 kPa.

Local conditions may however need to be considered in any of the above application. The values given in Items (a) to (f) are ultimate strength values.

The soil parameters can be used in the determination of the strength of the footing by using theories provided in many references on soil and foundations.

APPENDIX J
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