BOOTH ENGINEERS
& ASSOCIATES PTY LTD
CONSULTING ENGINEERS
ABN: 58 086 071 436

12 Old Cleveland Road STONES CORNER QLD 4120
PO BOX 1306 COORPAROO DC QLD 4151
Ph: 07 3397 7876
Fax: 07 3847 7739
Email: admin@boothengineers.com.au

BE:GB:07/6181A

12 December 2008

The Manager
Extreme Marquees
100 Pickering Street
ENOGGERA Q 4051

Dear Sir

RE: WIND ACTION ANALYSIS FOR EXTREME MARQUEES

Design Conditions:
AS 1170 Part 0 – General Principles
AS 1170 Part 2 – Wind Actions

Certification:
Booth Engineers & Associates Pty Ltd have undertaken an analysis of the Hold Down requirements for a range of marquee sizes by Extreme Marquees.

The tabulated values below, provide the gravity loads to be added to each leg of each frame for a marquee size of 5.0m x 5.0m and 6.0m x 6.0m.

Extreme Marquees Size	Hold Down Weight (Frame + Sand Bags in kg)	Maximum Wind Speed (km/hr)
5.0m x 5.0m	210	50
6.0m x 6.0m	220	45

Provided the marquees are installed in accordance with the above information, the Hold Down for the marquees are suitable for the appropriate wind speed.

Notations:
- It is assumed that the marquee frames are fabricated and installed to an acceptable standard of design and workmanship and hence its failure capacity subject to wind actions has not been considered in the analysis.

Yours faithfully

G N Booth

G N BOOTH
BEng MIEAust RPEQ 5581
For and on behalf of
Booth Engineers & Associates Pty Ltd

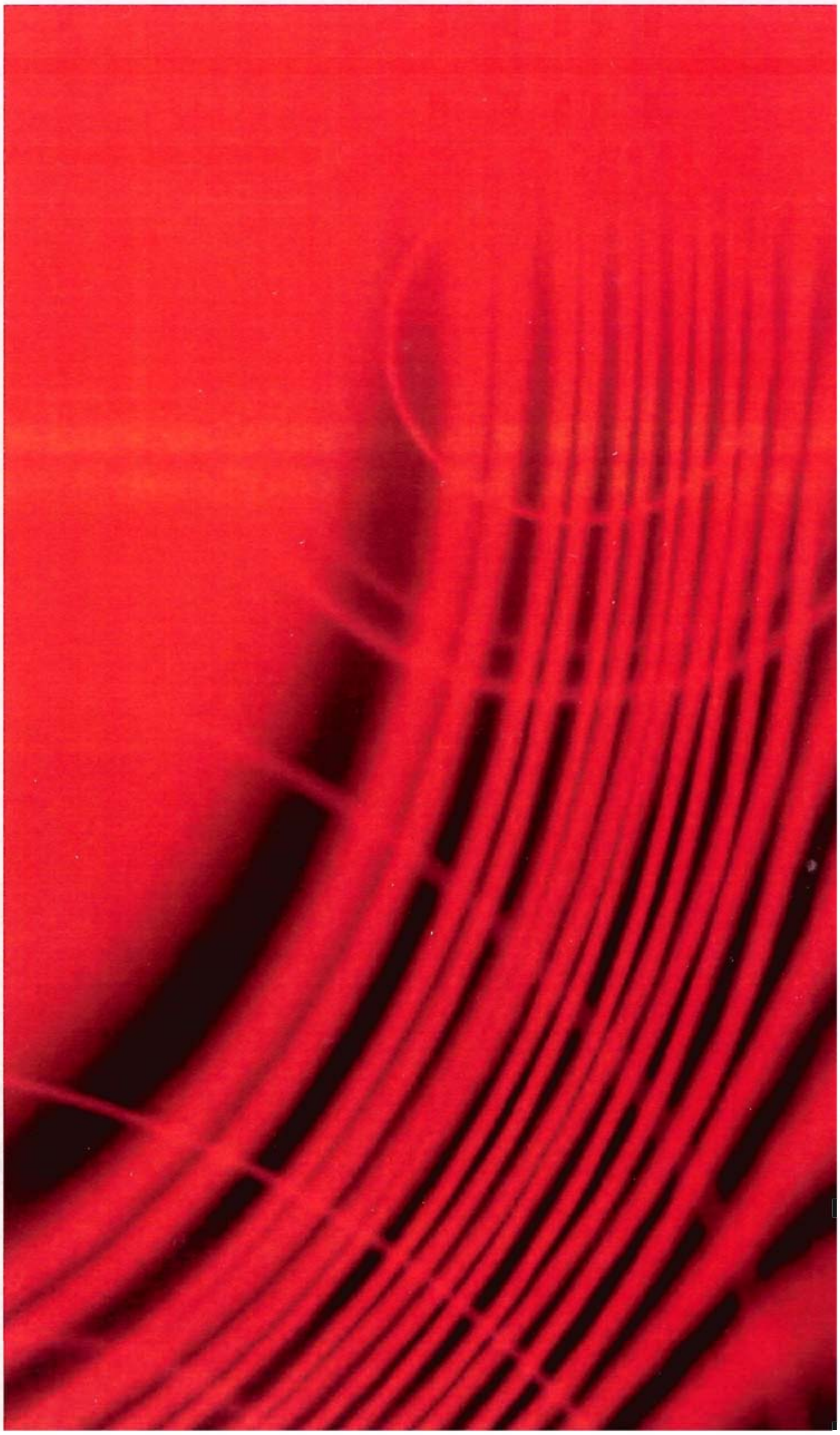
AS/NZS 1170.0:2002
(Incorporating Amendment Nos 1, 2, 3, 4 and 5)

STANDARDS
NEW ZEALAND
PAEREWA AOTEAROA

Australian/New Zealand Standard™

Structural design actions

Part 0: General principles



STANDARDS
Australia

This Joint Australian/New Zealand Standard was prepared by Joint Technical Committee BD-006, General design requirements and loading on structures. It was approved on behalf of the Council of Standards Australia on 29 March 2002 and on behalf of the Council of Standards New Zealand on 28 March 2002.
This Standard was published on 4 June 2002.

The following are represented on Committee BD-006:

Association of Consulting Engineers Australia
Australian Building Codes Board
Australian Steel Institute
Building Research Association of New Zealand
Cement and Concrete Association of Australia
Concrete Masonry Association of Australia
CSIRO, Building, Construction and Engineering
Cyclone Testing Station—James Cook University
Electricity Supply Association of Australia
Housing Industry Association
Institution of Engineers Australia
Institution of Professional Engineers New Zealand
Master Builders Australia
New Zealand Heavy Engineering Research Association
Steel Reinforcement Institute of Australia
University of Canterbury New Zealand
University of Melbourne
University of Newcastle

Additional Interests:

Monash University
Curtin University of Technology

Keeping Standards up-to-date

Standards are living documents which reflect progress in science, technology and systems. To maintain their currency, all Standards are periodically reviewed, and new editions are published. Between editions, amendments may be issued. Standards may also be withdrawn. It is important that readers assure themselves they are using a current Standard, which should include any amendments which may have been published since the Standard was purchased.

Detailed information about joint Australian/New Zealand Standards can be found by visiting the Standards Web Shop at www.saiglobal.com.au or Standards New Zealand web site at www.standards.co.nz and looking up the relevant Standard in the on-line catalogue.

For more frequent listings or notification of revisions, amendments and withdrawals, Standards Australia and Standards New Zealand offer a number of update options. For information about these services, users should contact their respective national Standards organization.

We also welcome suggestions for improvement in our Standards, and especially encourage readers to notify us immediately of any apparent inaccuracies or ambiguities. Please address your comments to the Chief Executive of either Standards Australia or Standards New Zealand at the address shown on the back cover.

This Standard was issued in draft form for comment as DR 00904.

AS/NZS 1170.0:2002
(Incorporating Amendment Nos 1, 2, 3, 4 and 5)

Australian/New Zealand Standard™

Structural design actions

Part 0: General principles

Originated in Australia as part of AS CA1—1933.
Originated in New Zealand as part of NZS 1900:1964.
Previous Australian editions AS 1170.1—1989 and AS 2867—1986.
Previous New Zealand edition NZS 4203:1992.
AS 1170.1—1989, AS 2867—1986, and NZS 4203:1992 jointly revised, amalgamated and redesignated in part as AS/NZS 1170.0:2002.
Reissued incorporating Amendment No. 1 (January 2003).
Reissued incorporating Amendment No. 2 (November 2003).
Reissued incorporating Amendment No. 4 (April 2005).
Reissued incorporating Amendment No. 3 (April 2011).
Reissued incorporating Amendment No. 5 (September 2011).

Jointly published by SAI Global Limited under licence from Standards Australia Limited, GPO Box 476, Sydney, NSW 2001 and by Standards New Zealand, Private Bag 2439, Wellington 6140

ISBN 0 7337 4469 9

PREFACE

This Standard was prepared by the Joint Standards Australia/Standards New Zealand Committee BD-006, General Design Requirements and Loading on Structures to supersede, in part, AS 1170.1—1989, *Minimum design loads on structures,* Part 1: *Dead and live loads*, and, in part, NZS 4203:1992, *Code of practice for general structural design and design loadings for buildings,* Volume 1: *Code of practice* and, in part, AS 2867—1986, *Farm structures—General requirements for structural design.*

This Standard incorporates Amendment No. 1 (January 2003), Amendment No. 2 (November 2003), Amendment No. 3 (April 2011), Amendment No. 4 (April 2005), and Amendment No. 5 (September 2011). The changes required by the Amendments are indicated in the text by a marginal bar and amendment number against the clause, note, table, figure or part thereof affected.

A4 This Standard is published as a joint Standard (as are also AS/NZS 1170.1 and AS/NZS 1170.2) and it is intended that it is suitable for use in New Zealand as well as Australia.

For Australia, this Standard will be referenced in the Building Code of Australia by way of BCA Amendment 11 to be published on 1 July 2002, thereby superseding in part the previous Edition, AS 1170.1—1989, which will be withdrawn 12 months from the date of publication of this edition. AS 1170.1—1989 may be used for structures not covered by the Building Code of Australia, until an Appendix is developed for inclusion in this Standard by amendment.

The objective of this Standard is to provide designers with general procedures and criteria for the structural design of structures. It outlines a design methodology that is applied in accordance with established engineering principles.

This Standard includes revised Clauses covering load combinations (referred to as combinations of actions) and general design and analysis clauses. It does not include values of actions (e.g. values of dead or live loads; referred to as permanent or imposed actions).

This Standard is Part 0 of the 1170 series, *Structural design actions*, which comprises the following parts, each of which has an accompanying Commentary published as a Supplement:

A4 AS/NZS 1170.0 General principles

AS/NZS 1170.1 Permanent, imposed and other actions

AS/NZS 1170.2 Wind actions

AS/NZS 1170.3 Snow and ice actions

A4 A5 AS 1170.4 Earthquake actions in Australia

NZS 1170.5 Earthquake actions – New Zealand

The Commentary to this Standard is AS/NZS 1170.0 Supp 1, *Structural design actions—General principles—Commentary* (*Supplement to* AS/NZS 1170.0:2002).

This Standard is based on the philosophy and principles set out in ISO 2394:1998, *General principles on reliability for structures.* ISO 2394 is written specifically as a guide for the preparation of national Standards covering the design of structures. It includes methods for establishing and calibrating reliability based limit states design Standards.

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.

A5 Statements expressed in mandatory terms in notes to tables are deemed to be requirements of this Standard. Notes to the text contain information and guidance and are not considered to be an integral part of the Standard.

CONTENTS

STANDARDS AUSTRALIA/STANDARDS NEW ZEALAND

Australian/New Zealand Standard
Structural design actions

Part 0: General principles

SECTION 1 SCOPE AND GENERAL

1.1 SCOPE

This Standard specifies general procedures and criteria for the structural design of a building or structure in limit states format. It covers limit states design, actions, combinations of actions, methods of analysis, robustness and confirmation of design.

The Standard is applicable to the structural design of whole buildings or structures and their elements.

This Standard covers the following actions:

(a) Permanent action (dead load).

(b) Imposed action (live load).

(c) Wind.

(d) Snow.

(e) Earthquake.

A3 (f) Static liquid pressure.

(g) Ground water.

(h) Rainwater ponding.

(i) Earth pressure.

NOTES:

1 Where this Standard does not give information required for design, special studies should be carried out. Guidance is given in Appendix A.

2 Where testing is used to determine data for design or to confirm a design, guidance on methods is given in Appendix B.

3 Normal design practice is that all likely actions be considered. Any actions considered in design that are not in the above list should be the subject of special studies, as they are not covered by this Standard.

4 Additional information on other actions such as movement effects, construction loads and accidental actions is given in the Commentary (see Preface).

5 Movement effects include actions on structures resulting from expansion or contraction of materials of construction (such as those due to creep, temperature or moisture content changes) and also those resulting from differential ground settlement. Serviceability may be particularly affected by such actions.

6 Guidance on criteria for serviceability is given in Appendix C, which have been found to be generally suitable for importance level 2 buildings. Structures of special importance or structures where more stringent criteria are appropriate may require the stated criteria to be tightened.

1.2 APPLICATION

This Standard may be used as a means for demonstrating compliance with the Requirements of Part B1 of the Building Code of Australia.

A3 This Standard is intended for citation by New Zealand's Department of Building and Housing as a document that contributes towards establishing compliance with Clause B1 'Structure' of the New Zealand Building Code (NZBC). Citation of the Standard means that compliance with the NZBC can be achieved by applying this Standard in conjunction with the appropriate material standards, provided that an engineer with relevant experience and skills in structural engineering is responsible for interpretation of the requirements.

1.3 REFERENCED DOCUMENTS

The following documents are referred to in this Standard:

AS
A3 1170 Minimum design loads on structures
1170.4 Part 4: Earthquake actions in Australia

AS/NZS
1170 Structural design actions
1170.1 Part 1: Permanent, imposed and other actions
1170.2 Part 2: Wind actions
A1 1170.3 Part 3: Snow and ice actions

Australian Building Codes Board
Building Code of Australia

A4 NZS
A5 1170 Structural design actions
1170.5 Part 5: Earthquake actions – New Zealand

1.4 DEFINITIONS

For the purpose of this Standard the definitions below apply.

1.4.1 Action

Set of concentrated or distributed forces acting on a structure (direct action), or deformation imposed on a structure or constrained within it (indirect action).

NOTE: The term load is also often used to describe direct actions.

1.4.2 Action effects (internal effects of actions, load effects)

Internal forces and bending moments due to actions (stress resultants).

1.4.3 Combination of actions

Set of design values used to confirm that the limit states are not exceeded under simultaneous influence of different actions.

1.4.4 Design action effect

The action effect computed from the design values of the actions or design loads.

1.4.5 Design capacity

The product of the capacity reduction factor and the nominal capacity.

1.4.6 Design situation

Set of conditions for which the design is required to demonstrate that relevant limit states are not exceeded.

1.4.7 Imposed action

A variable action resulting from the intended use or occupancy of the structure.

1.4.8 Limit states

States beyond which the structure no longer satisfies the design criteria.

NOTE: Limit states separate desired states (compliance) from undesired states (non-compliance).

1.4.9 Limit states, serviceability

States that correspond to conditions beyond which specified service criteria for a structure or structural element are no longer met.

NOTE: The criteria are based on the intended use and may include limits on deformation, vibratory response, degradation or other physical aspects.

1.4.10 Limit states, ultimate

States associated with collapse, or with other similar forms of structural failure.

NOTE: This generally corresponds to the maximum load-carrying resistance of a structure or structural element but, in some cases, to the maximum applicable strain or deformation.

1.4.11 Load

The value of a force appropriate to an action.

1.4.12 Permanent action

Action that is likely to act continuously and for which variations in magnitude with time are small compared with the mean value.

1.4.13 Proof testing

Application of test loads to a structure, sub-structure, member or connection, to ascertain the structural characteristics of that one item under test.

1.4.14 Prototype testing

Application of test loads to one or more samples of structures, sub-structures, members or connections to ascertain the structural characteristics of the population that the sample represents.

1.4.15 Reliability

Ability of a structure or structural element to fulfil the specified criteria, including the working life, for which it has been designed.

NOTE: Reliability covers structural safety and serviceability, and can be expressed in terms of probability.

1.4.16 Serviceability

Ability of a structure or structural element to perform adequately for normal use under all expected actions.

1.4.17 Shall

Indicates that a statement is mandatory.

1.4.18 Should

Indicates a recommendation (non-mandatory).

1.4.19 Structure

Organized combination of connected structural elements designed to provide some measure of resistance.

1.4.20 Structural element

Physically distinguishable part of a structure, for example, wall, column, beam, connection.

1.4.21 Structural robustness

Ability of a structure to withstand events like fire, explosion, impact or consequences of human errors, without being damaged to an extent disproportionate to the original cause.

1.4.22 Special study

A procedure for justifying departure from this Standard or for determining information not covered by this Standard.

NOTE: Special studies are outside the scope of this Standard.

A2

1.4.23 Design working life

Duration of the period during which a structure or a structural element, when designed, is assumed to perform for its intended purpose with expected maintenance but without major structural repair being necessary.

NOTE: In the context of this Standard, the design working life is a 'reference period' usually stated in years. It is a concept that can be used to select the probability of exceedance of different actions.

1.4.24 Environmental influences

Chemical, biological or physical influences on a structure, which may deteriorate the materials constituting the structure, and which in turn may affect its reliability in an unfavourable way.

1.5 NOTATION

Where non-dimensional ratios are involved, both the numerator and denominator are expressed in identical units.

The dimensional units for length and stress in all expressions or equations are to be taken as millimetres (mm) and megapascals (MPa) respectively, unless specifically noted otherwise.

Unless otherwise stated, the notation in this Standard has the following meanings:

E = action effect

E = earthquake action

E_s = serviceability earthquake action

E_u = ultimate earthquake action

E_d = design action effect

$E_{d,dst}$ = design action effect of destabilizing actions

$E_{d,stb}$ = design action effect of stabilizing actions

F_e = earth pressure action

$F_{e,u}$ = ultimate earth pressure action

F_{ice} = ice action

F_{gw} = ground water action

F_{lp} = liquid pressure action

F_{pnd} = rainwater ponding action

F_{sn} = snow action

G = permanent action (self-weight or 'dead' action)

k_p = probability factor

k_t = factor to allow for variability of structural units

N = design working life of a building or structure, in years

P = the annual probability of exceedance

P_{ref} = reference probability of exceedance for safety

Q = imposed action (due to occupancy and use, 'live' action)

R = nominal capacity (based on the fifth percentile strength)

R_d = design capacity (equal to ϕR)

S_u = ultimate value of various actions appropriate for particular combinations

V_{sc} = coefficient of variation of structural characteristics

W = wind action

W_s = serviceability wind action

W_u = ultimate wind action

δ = values of the serviceability parameter determined on the basis of the design actions

δ_ℓ = limiting value of the serviceability parameter (the subscript 'ℓ' stands for limiting value)

ϕ = capacity reduction factor

ψ_c = combination factor for imposed action

A3 ψ_E = combination factor for earthquake actions

ψ_s = factor for determining frequent values (short-term) of actions

ψ_ℓ = factor for determining quasi-permanent values (long-term) of actions

SECTION 2 STRUCTURAL DESIGN PROCEDURE

2.1 GENERAL

Structural design shall be carried out using the procedure given in Clause 2.2 for ultimate limit states and Clause 2.3 for serviceability limit states.

2.2 ULTIMATE LIMIT STATES

Design for ultimate limit states shall be carried out by the following procedure:

A2 (a) Adopt the importance level for the building or structure and the associated annual probability of exceedance (P) for wind, snow and earthquake as follows:

   (i) For Australia—

      (A) structures covered by the Building Code of Australia—as given in the Building Code of Australia.

      (B) structures not covered by the Building Code of Australia and for which no design events are specified by the applicable legislation or by other Standards—as given in Appendix F.

   (ii) For New Zealand—as given in Section 3.

(b) Determine the permanent (G) and imposed (Q) loads in accordance with AS/NZS 1170.1.

(c) Determine the ultimate loads for wind (W) in accordance with AS/NZS 1170.2.

A4 (d) Determine the ultimate loads for earthquake (E_u) for Australia, in accordance with AS 1170.4 as modified by Appendix D of this Standard including the probability factor (k_p) and the changes to earthquake design category. For New Zealand determine the ultimate loads for earthquake (E_u), in accordance with NZS 1170.5.

A1 (e) Determine the ultimate loads for snow (F_{sn}) and ice (F_{ice}) in accordance with AS/NZS 1170.3.

(f) Where such actions are relevant, determine the ultimate loads for liquid pressure (F_{lp}) ground water (F_{gw}) rainwater ponding (F_{pnd}) and earth pressure loads ($F_{e,u}$) in accordance with AS/NZS 1170.1.

(g) Determine combinations of actions in accordance with Section 4.

(h) Analyse the structure and its parts for the relevant combinations in accordance with Section 5.

(i) Design and detail the structure in accordance with—

   (i) Section 6 for robustness; and

A4 (ii) for Australia, AS 1170.4 for earthquake, or

   (iii) for New Zealand, NZS 1170.5 for earthquake.

(j) Determine the design resistance using the applicable Standard or other document. The Building Code of Australia specifies the documents to be used within its jurisdiction.

(k) Confirm that the design resistance exceeds the appropriate action effects in accordance with Section 7.

2.3 SERVICEABILITY LIMIT STATES

Design for serviceability limit states shall be carried out by the following procedure as appropriate:

(a) Determine for the whole structure and for individual elements, the type of design serviceability conditions to be considered.

(b) Determine the design situation including the serviceability load event and serviceability limits for the design serviceability condition being considered (see Section 3 for New Zealand).

A2 NOTE: Guidelines for serviceability events and associated limits are given in Appendix C for loads associated with an appropriate annual probability of exceedance (P).

(c) Determine the permanent loads (G) and serviceability imposed loads (Q) in accordance with AS/NZS 1170.1.

(d) Determine serviceability loads for wind (W) in accordance with AS/NZS 1170.2.

A1 (e) Determine serviceability loads for snow (F_{sn}) and ice (F_{ice}) in accordance with AS/NZS 1170.3.

(f) Where such actions are relevant, determine serviceability loads for liquid pressure (F_{lp}) ground water (F_{gw}) rainwater ponding (F_{pnd}) and earth pressure ($F_{e,u}$) in accordance with AS/NZS 1170.1.

(g) Determine the applicable combinations corresponding to the selected design serviceability conditions in accordance with Section 4.

(h) Model the serviceability response of the structure and its parts for the relevant combinations for each serviceability condition using methods of analysis appropriate for the serviceability limit state in accordance with Section 5.

(i) Determine the serviceability response using the applicable Standard or other document. The Building Code of Australia specifies the documents to be used within its jurisdiction.

(j) Confirm, in accordance with Section 7, that the modelled serviceability response does not exceed the appropriate limiting values for each of the serviceability conditions identified.

A3 (k) Serviceability limits applicable to earthquake loading in New Zealand are to conform with the requirements of NZS 1170.5.

A2
SECTION 3 ANNUAL PROBABILITY OF EXCEEDANCE (FOR STRUCTURES IN NEW ZEALAND ONLY)

3.1 GENERAL

This Section shall be used to determine the annual probability of exceedance of ultimate limit state loads for New Zealand. It does not form part of the Standard for use in Australia.

Structures of importance level 5 are outside the scope of this Standard and require the annual probability of load exceedance (design event) to be determined by a special study.

NOTE: For buildings within Australia, refer to the Building Code of Australia.

3.2 DESIGN REQUIREMENTS

A structure shall be designed and constructed in such a way that it will, during its design working life, with appropriate degrees of reliability sustain all actions and environmental influences likely to occur. In particular it shall be designed as follows:

(a) To withstand extreme or frequently repeated actions, or both, occurring during its construction and anticipated use (resistance, deformability and static equilibrium requirements; that is, for safety).

Specifically, for earthquake actions for ultimate limit states this shall mean—

(i) avoidance of collapse of the structural system;

(ii) avoidance of collapse or loss of support of parts of the structure representing a hazard to human life inside and outside the structure or parts required for life safety systems; and

(iii) avoidance of damage to non-structural systems necessary for the building evacuation procedures that renders them inoperative.

(b) So that it will not be damaged to an extent disproportionate to the original cause, by events like fire, explosion, impact or consequences of human error (robustness requirement).

(c) To perform adequately under all expected actions (serviceability requirement).

Structural design carried out using the procedures given in Clause 2.2 for ultimate limit states and Clause 2.3 for serviceability limit states is deemed to comply with this Clause.

A4 NOTE: The design should include consideration of appropriate maintenance and the effects of environmental influences.

3.3 IMPORTANCE LEVELS

The importance level of the structure shall be determined in accordance with its occupancy and use, as given in Tables 3.1 and 3.2. The Table describes, in general terms, five categories of structure and gives some examples of each. For those buildings not specifically mentioned, the designer will need to exercise judgement in assigning the appropriate level.

Structures that have multiple uses shall be assigned the highest importance level applicable for any of those uses. Where access to a structure is via another structure of a lower importance level, then the importance level of the access structure shall be designated the same as the structure itself.

3.4 ANNUAL PROBABILITY OF EXCEEDANCE

3.4.1 Ultimate limit states

For ultimate limit states for structures of importance levels 1 to 4, the annual probability of exceedance (P) for wind, snow and earthquake loads shall be as given in Table 3.3.

The design working life of structures that are erected for a number of short periods of use and dismantled between each, is equal to the total of the periods of use.

3.4.2 Serviceability limit states

Serviceability limit states shall include—

A4 (a) SLS1—the structure and the non-structural components do not require repair after the SLS1 earthquake, snow or wind event; and

(b) SLS2—the structure maintains operational continuity after the SLS2 earthquake.

For serviceability limit states for structures of importance levels 2 to 4, the annual probability of exceedance (P) for wind, snow and earthquake loads shall be determined as given in Table 3.3.

NOTE: Guidelines for limits associated with serviceability events are given in Appendix C.

TABLE 3.1

CONSEQUENCES OF FAILURE FOR IMPORTANCE LEVELS

Consequences of failure	**Description**	**Importance level**	**Comment**
Low	**Low** consequence for loss of human life, *or* **small or moderate** economic, social or environmental consequences	1	Minor structures (failure not likely to endanger human life)
Ordinary	**Medium** consequence for loss of human life, *or* **considerable** economic, social or environmental consequences	2	Normal structures and structures not falling into other levels
High	**High** consequence for loss of human life, *or* **very great** economic, social or environmental consequences	3	Major structures (affecting crowds)
		4	Post-disaster structures (post disaster functions or dangerous activities)
Exceptional	Circumstances where reliability must be set on a case by case basis	5	Exceptional structures

A2

TABLE 3.2

IMPORTANCE LEVELS FOR BUILDING TYPES—NEW ZEALAND STRUCTURES

Importance level	Comment	Examples
1	Structures presenting a low degree of hazard to life and other property	Structures with a total floor area of <30 m^2 Farm buildings, isolated structures, towers in rural situations Fences, masts, walls, in-ground swimming pools
2	Normal structures and structures not in other importance levels	Buildings not included in Importance Levels 1, 3 or 4 Single family dwellings Car parking buildings
3	Structures that as a whole may contain people in crowds or contents of high value to the community or pose risks to people in crowds	Buildings and facilities as follows: (a) Where more than 300 people can congregate in one area (b) Day care facilities with a capacity greater than 150 (c) Primary school or secondary school facilities with a capacity greater than 250 (d) Colleges or adult education facilities with a capacity greater than 500 (e) Health care facilities with a capacity of 50 or more resident patients but not having surgery or emergency treatment facilities (f) Airport terminals, principal railway stations with a capacity greater than 250 (g) Correctional institutions (h) Multi-occupancy residential, commercial (including shops), industrial, office and retailing buildings designed to accommodate more than 5000 people and with a gross area greater than 10 000 m^2 (i) Public assembly buildings, theatres and cinemas of greater than 1000 m^2 Emergency medical and other emergency facilities not designated as post-disaster Power-generating facilities, water treatment and waste water treatment facilities and other public utilities not designated as post-disaster Buildings and facilities not designated as post-disaster containing hazardous materials capable of causing hazardous conditions that do not extend beyond the property boundaries
4	Structures with special post-disaster functions	Buildings and facilities designated as essential facilities Buildings and facilities with special post-disaster function Medical emergency or surgical facilities Emergency service facilities such as fire, police stations and emergency vehicle garages Utilities or emergency supplies or installations required as backup for buildings and facilities of Importance Level 4 Designated emergency shelters, designated emergency centres and ancillary facilities Buildings and facilities containing hazardous materials capable of causing hazardous conditions that extend beyond the property boundaries
5	Special structures (outside the scope of this Standard—acceptable probability of failure to be determined by special study)	Structures that have special functions or whose failure poses catastrophic risk to a large area (e.g. 100 km^2) or a large number of people (e.g., 100 000) Major dams, extreme hazard facilities

A2
A4

TABLE 3.3

ANNUAL PROBABILITY OF EXCEEDANCE

Design working life	Importance level	Annual probability of exceedance for ultimate limit states			Annual probability of exceedance for serviceability limit states	
		Wind	**Snow**	**Earthquake**	**SLS1**	**SLS2 Importance level 4 only**
Construction equipment, e.g., props, scaffolding, braces and similar	2	1/100	1/50	1/100	1/25	
Less than 6 months	1	1/25	1/25	1/25	—	
	2	1/100	1/50	1/100	1/25	
	3	1/250	1/100	1/250	1/25	
	4	1/1000	1/250	1/1000	1/25	
5 years	1	1/25	1/25	1/25	—	—
	2	1/250	1/50	1/250	1/25	—
	3	1/500	1/100	1/500	1/25	—
	4	1/1000	1/250	1/1000	1/25	1/250
25 years	1	1/50	1/25	1/50	—	—
	2	1/250	1/50	1/250	1/25	—
	3	1/500	1/100	1/500	1/25	—
	4	1/1000	1/250	1/1000	1/25	1/250
50 years	1	1/100	1/50	1/100	—	—
	2	1/500	1/150	1/500	1/25	—
	3	1/1000	1/250	1/1000	1/25	—
	4	1/2500	1/500	1/2500	1/25	1/500
100 years or more	1	1/250	1/150	1/250	—	—
	2	1/1000	1/250	1/1000	1/25	—
	3	1/2500	1/500	1/2500	1/25	—
	4	*	*	*	1/25	*

* For importance level 4 structures with a design working life of 100 years or more, the design events are determined by a hazard analysis but need to have probabilities less than or equal to those for importance level 3.

Design events for importance level 5 structures should be determined on a case by case basis.

SECTION 4 COMBINATIONS OF ACTIONS

4.1 GENERAL

The combinations of actions for use in design of structures shall be as given in this Standard. Other combinations may be required.

4.2 COMBINATIONS OF ACTIONS FOR ULTIMATE LIMIT STATES

4.2.1 Stability

The basic combinations for the ultimate limit states used in checking stability (see Clause 7.2.1) shall be as follows where the long-term and combination factors are given in Table 4.1:

(a) For combinations that produce net stabilizing effects ($E_{d,stb}$):

$E_{d,stb} = [0.9G]$ permanent action only (does not apply to prestressing forces)

(b) For combinations that produce net destabilizing effects ($E_{d,dst}$):

(i) $E_{d,dst} = [1.35G]$ permanent action only (does not apply to prestressing forces)

(ii) $E_{d,dst} = [1.2G, 1.5Q]$ permanent and imposed action

(iv) $E_{d,dst} = [1.2G, W_u, \psi_c Q]$ permanent, wind and imposed action

A3 (v) $E_{d,dst} = [G, E_u, \psi_E Q]$ permanent, earthquake and imposed action

(vi) $E_{d,dst} = [1.2G, S_u, \psi_c Q]$ permanent action, actions given in Clause 4.2.3 and imposed action

NOTE: Combination factors for prestressing forces are given in the appropriate materials design Standard.

4.2.2 Strength

The basic combinations for the ultimate limit states used in checking strength (see Clause 7.2.2) shall be as follows, where the long-term and combination factors are given in Table 4.1:

(a) $E_d = [1.35G]$ permanent action only (does not apply to prestressing forces)

(b) $E_d = [1.2G, 1.5Q]$ permanent and imposed action

(c) $E_d = [1.2G, 1.5\psi_\ell Q]$ permanent and long-term imposed action

(d) $E_d = [1.2G, W_u, \psi_c Q]$ permanent, wind and imposed action

(e) $E_d = [0.9G, W_u]$ permanent and wind action reversal

A3 (f) $E_d = [G, E_u, \psi_E Q]$ permanent, earthquake and imposed action

(g) $E_d = [1.2G, S_u, \psi_c Q]$ permanent action, actions given in Clause 4.2.3 and imposed action

A3 NOTES:

1 Combination factors for prestressing forces are given in the appropriate materials design Standard.

2 Refer to AS/NZS 1170.1, Clause 3.3.

A3 Where impact is a design consideration, and no other Standard sets the manner of calculation, the effect shall be considered as part of imposed action, that is substitute (Q + Impact) for Q in the relevant combinations.

TABLE 4.1

SHORT-TERM, LONG-TERM AND COMBINATION FACTORS

A3

Character of imposed action	Short-term factor (ψ_s)	Long-term factor (ψ_ℓ)	Combination factor (ψ_c)	Earthquake combination factor (ψ_E)
Distributed imposed actions, Q				
Floors				
Residential and domestic	0.7	0.4	0.4	0.3
Offices	0.7	0.4	0.4	0.3
Parking	0.7	0.4	0.4	0.3
Retail	0.7	0.4	0.4	0.3
Storage	1.0	0.6	0.6	0.6
Other	1.0	0.6	0.6	0.6
Roofs				
Roofs used for floor type activities (see AS/NZS 1170.1)	0.7	0.4	0.4	0.3
All other roofs	0.7	0.0	0.0	0.0
Concentrated imposed actions (including balustrades), Q				
Floors	1.0	0.6	as for distributed floor actions	0.3
Floors of domestic housing	1.0	0.4		0.3
Roofs used for floor type activities	1.0	0.6		0.3
All other roofs	1.0	0.0	0.0	0.0
Balustrades	1.0	0.0	0.0	0.0
Long-term installed machinery, tare weight	1.0	1.0	1.2	1.0

4.2.3 Combinations for snow, liquid pressure, rainwater ponding, ground water and earth pressure

Where appropriate to the design situation, the basic combinations shall be modified for the action of liquid pressure, ground water and earth pressure by the use of the following factored values:

A1 (a) $S_u = F_{sn}$ for snow determined in accordance with AS/NZS 1170.3.

(b) Where the liquid type and density is well defined and design liquid height cannot be exceeded—

(i) $S_u = 1.2\ F_{lp}$ for static liquid pressure; and

(ii) for self-weight of stored liquid, use the factor for permanent action

(c) Where liquid type or density is not well defined or design liquid height is not limited—

(i) $S_u = 1.5\ F_{lp}$ for static liquid pressure; and

(ii) for self-weight of stored liquid, use the factor for imposed action.

(d) $S_u = 1.2\ F_{pnd}$ for rainwater ponding where the water level is as given in AS/NZS 1170.1.

(e) $S_u = 1.2\ F_{gw}$ for ground water where the ground water level is as given in AS/NZS 1170.1, otherwise $S_u = 1.5\ F_{gw}$.

(f) For earth pressures:

(i) $S_u = 1.0\ F_{e,u}$ when $F_{e,u}$ is determined using an ultimate limit states method.

(ii) $S_u = 1.5\ F_e$ when determined using other methods.

A1 (g) $S_u = 1.2F_{ice}$ for ice determined in accordance with AS/NZS 1170.3.

4.2.4 Combinations of actions for fire

The combination of factored actions used when confirming the ultimate limit state for fire shall be as follows:

[G, thermal actions arising from the fire, $\psi_\ell Q$]

NOTE: Where it is appropriate to consider the stability of remaining walls that may collapse outwards after a fire event, other ultimate limit states criteria are given in Section 6.

4.3 COMBINATIONS OF ACTIONS FOR SERVICEABILITY LIMIT STATES

Combinations of actions for the serviceability limit states shall be those appropriate for the serviceability condition being considered. Appropriate combinations may include one or a number of the following using the short-term and long-term values given in Table 4.1:

(a) G

(b) $\psi_s Q$

(c) $\psi_\ell Q$

(d) W_s

(e) E_s

(f) Serviceability values of other actions, as appropriate.

4.4 CYCLIC ACTIONS

When checking structures or elements of structures for fatigue performance under repeated in-service cyclic actions, the level of repeated loading to be used shall be the actual load level expected for the design situation.

SECTION 5 METHODS OF ANALYSIS

5.1 GENERAL

The structural analysis used to determine action effects from loads shall be in accordance with the principles of structural mechanics.

5.2 STRUCTURAL MODELS

The structural model shall reflect the behaviour of the structure for the appropriate limit state being considered.

Structural models, parameters and properties shall be as given in the relevant Australian or New Zealand Standards for design of material for the appropriate limit states.

A3 Modelling shall be based on the following:

(a) Static or dynamic response, or both.

(b) Elastic or non-elastic (plastic) response, or both.

(c) Geometrically linear or geometrically non-linear response, or both.

(d) Time-independent or time-dependent behaviour, or both.

SECTION 6 STRUCTURAL ROBUSTNESS

6.1 GENERAL

General detailing of components of the structural force-resisting system and of other components shall be in accordance with this Section.

Structures shall be detailed such that all parts of the structure shall be tied together both in the horizontal and the vertical planes so that the structure can withstand an event without being damaged to an extent disproportionate to that event.

Clause 6.2 is deemed to satisfy this Clause.

6.2 LOAD PATHS

6.2.1 General

The design of the structure shall provide load paths to the foundations for forces generated by all types of actions from all parts of the structure, including structural and non-structural components. The minimum actions shall be as given in Clauses 6.2.2 to 6.2.5.

6.2.2 Minimum resistance

A3 The structure shall have a minimum lateral resistance equivalent to the following percentage of ($G + \psi_c Q$) for each level, applied simultaneously at each level for a given direction:

(a) For structures over 15 m tall .... 1%.

(b) For all other structures .... 1.5%.

The height shall be the height of the top of the structure above the level where the structure is coupled with the ground for lateral resistance.

The direction of application of the lateral load shall be that which will produce the most critical action effect in the element under consideration, except that the application of this load in more than one direction simultaneously need not be considered in the design of any element.

6.2.3 Minimum lateral resistance of connections and ties

All parts of the structure shall be interconnected. Connections shall be capable of transmitting 5 percent of the value of ($G + \psi_c Q$) for the connection under consideration.

6.2.4 Diaphragms

Floor and roof diaphragms shall be designed—

(a) to resist required horizontal forces; and

(b) to have ties or struts (where used) able to distribute the required wall anchorage forces.

6.2.5 Walls

A3 Walls shall be connected to the structure to provide horizontal resistance to face loads. The connection between the wall and the structure shall be capable of resisting the forces of 5% of G.

SECTION 7 CONFIRMATION METHODS

7.1 GENERAL

It shall be confirmed that all limit states are complied with by consideration of the relevant design situations and load cases, in accordance with Clauses 7.2 and 7.3.

NOTE: Appendix B gives guidance on testing as a means for confirmation.

7.2 ULTIMATE LIMIT STATES

7.2.1 Stability

When considering a limit state of static equilibrium or of gross displacements or deformations of the structure, it shall be confirmed that—

$$E_{\mathrm{d,stb}} + R_{\mathrm{d}} \geq E_{\mathrm{d,dst}} \qquad \ldots 7.1$$

where

$E_{\mathrm{d,stb}}$ = design action effect of stabilizing actions (see Clause 4.2)

R_{d} = design capacity (equal to ϕR)

$E_{\mathrm{d,dst}}$ = design action effect of destabilizing actions

7.2.2 Strength

When considering a limit state of collapse, rupture or excessive deformation of a structure, section, member or connection it shall be confirmed that—

$$R_{\mathrm{d}} \geq E_{\mathrm{d}} \qquad \ldots 7.2$$

where

R_{d} = design capacity (equal to ϕR)

E_{d} = design action effect (see Clause 4.2)

7.3 SERVICEABILITY LIMIT STATES

When considering a serviceability limit state, it shall be confirmed that—

$$\delta \leq \delta_{\ell} \qquad \ldots 7.3$$

where

δ = value of the serviceability parameter determined on the basis of the design actions (see Clause 4.3)

δ_{ℓ} = limiting value of the serviceability parameter.

NOTE: The limiting value of the serviceability parameter should be determined based on accepted information, unless specific limits are specified for the particular structure being designed. Guidance on acceptable serviceability limits for some typical situations are given in Appendix C.

APPENDIX A

SPECIAL STUDIES

(Informative)

Where changes are made to a part or all of the design processes detailed in Clauses 2.2 and 2.3 or new information or methods are introduced, they should be established by special studies.

NOTE: Generally, design situations to be considered are covered by the clauses in Sections 2 and 4. However, actions other than those specified in the Standard and design considerations specific to the structure being designed may require special studies to be carried out.

Special studies should be used for the following:

(a) To establish information or methods for design not given in this Standard, or to define more accurately the information or methods used, or where more accuracy is considered necessary.

NOTES:

1 For example, to determine a design parameter such as a wind pressure coefficient, to establish values for an action or to confirm a structure or population of structures.

2 Methods for performing tests and analysing test information are given in Appendix B.

(b) To evaluate loads for actions other than those specified in this Standard. Where they are considered a possibility, special studies should be used to determine values for the following actions:

(i) Foundation movements.

(ii) Dynamic effects.

(iii) Time-dependent movement of materials.

(iv) Differential axial shortening.

(v) Shrinkage and expansion of materials.

(vi) Temperature changes and gradients (including those caused by fire).

NOTE: Care is needed in determining material properties for use in these design-loading conditions.

Where a study is used to establish design values for an action, the factors for appropriate combinations should be determined as part of the study. The variability of the loads derived should be taken into account when determining the factors used in the combinations.

A special study should include appropriate documentation to show the source of all data. Any documentation should demonstrate that the study is appropriate in the context of the particular evaluation of structural performance and should include the following, where relevant:

(A) A complete report similar in scope to that set out in Appendix B.

(B) Reference to other national or international Standards.

(C) Comparison with other data.

(D) Analytical methods used.

APPENDIX B

USE OF TEST DATA FOR DESIGN

(Informative)

B1 GENERAL

B1.1 Scope

The use in design of data from observation and testing (experimental models) should be as given in this Appendix. Methods for testing and for evaluation of the results are given. More specific methods for each type of action can be found in the relevant Part of this series of Standards.

Testing may be carried out as part of a study, where—

(a) more accurate information is required for use in structural design;

(b) specific design parameters are not included in the relevant Standard; or

(c) the situation is sufficiently unusual to require that limit states be checked by methods other than calculation.

Specific methods for proof testing and prototype testing of structures or parts of structures are covered in Paragraphs B2 and B3. Checks on material properties or other control tests are not considered to be part of this Appendix.

Examples of information to be determined using this Appendix include—

(i) values for an action at a particular site (including reliability parameters);

(ii) design parameters (e.g., wind pressure factors);

(iii) structural response under loads; and

(iv) reliability of a structure or population of structures.

B1.2 Reliability

The use, in design, of data determined in this Appendix should be carried out in such a way that the structure, as designed or tested, has at least the same reliability with respect to all limit states, as structures for which the design is based on calculation only.

B1.3 Use of test data

The general design procedure should be in accordance with Section 2 of this Standard. Where test data is required for some part of the procedure, all variables relevant to that part of the procedure should be considered.

The unknown coefficients or quantities to be evaluated from the test data should be clearly indicated and the supporting test information provided.

The evaluation of the data should be based on statistical methods consistent with the aim of Paragraph B1.2. The statistical approach used in the evaluation of the test data should be described.

Separate account should be taken of those variables or conditions that are not covered by the test procedures.

B1.4 Modelling

The test arrangement should be modelled taking into account the circumstances affecting the real situation being modelled. The differences between reality and the testing conditions should be accounted for by a suitably determined modification factor.

Apparatus should be appropriately calibrated.

The testing procedure and any analysis methods to be used should be established and documented.

B1.5 Report

The test report should include the following:

(a) Scope of information required from the test data.

(b) Description of conditions that could influence the behaviour under consideration.

(c) Details of the testing arrangement and measurement methods.

(d) Details of the testing procedure (including the methods established for analysis).

(e) Environmental conditions of the test.

(f) Materials tested (including number of samples, all relevant properties of samples, e.g., nature and size of characteristics in timber).

(g) Measurements of relevant properties.

(h) Results (including modes of failure if relevant).

(i) Evaluation of the data and conclusions.

(j) Any unusual aspects of the testing.

(k) The name and location of the testing laboratory or testing organization.

(l) The number of this Australian/New Zealand Standard, i.e. AS/NZS 1170.0.

B2 PROOF TESTING

B2.1 General

This test method establishes the ability of the particular unit under test to satisfy the limit state that the test is designed to evaluate. The relevant parts of Paragraph B1 should be followed.

B2.2 Test load

The target test load should be equal to the design action effect for the relevant limit state.

NOTE: The design action effect may need to be factored to account for the effect of duration of load on the strength and serviceability of the structure.

B2.3 Criteria for acceptance

The criteria for acceptance should be as follows:

(a) *Strength limit state* The item should be deemed to comply with the strength limit state if it is able to sustain the target test load for that limit state for at least 15 min. It should then be inspected to determine the nature and extent of any damage incurred during the test. The effects of the damage should be considered and, if necessary, appropriate repairs to the damaged parts carried out.

NOTE: For materials with time-dependent properties, the load should be removed within a reasonably short period of time after completion of the test. For example, reduce the design load by 25 percent within 15 min, and 50 percent within the following hour. Further guidance should be given in the relevant materials Standards.

(b) *Serviceability limit state* The maximum deformations of the item should be within the specified serviceability limits when subjected to the target test load for that limit state. Where the residual deflection exceeds 30 percent of the deflection at the target test load, the item should either be reloaded to the target test load to ensure that it has not sustained serious permanent damage, or other measures should be taken to determine the level of damage.

B3 PROTOTYPE TESTING

B3.1 General

This test method establishes the ability of a population of items to satisfy the limit state that the test is designed to evaluate. The method is not applicable to the testing of structural models, nor to the establishment of general design criteria or data.

Sampling should be carried out so that samples are representative of the population they are drawn from.

The test load should be applied at as constant a rate as practicable. Load-deflection curves should be plotted during each test on each unit. Deflections should be measured appropriate to the material being tested and should include values before the commencement of the test, after the test load has been applied and after removal of the test load.

B3.2 Design capacity of specific products and assemblies

The design capacity of a specific product or a specific assembly may be established by prototype testing of that product or assembly. The design capacity should not exceed the minimum value of the test results divided by the appropriate value of k_t as given in Paragraph B3.4.

B3.3 Units for testing

The units used in testing should be manufactured using the materials and methods that will be used in production. Where the units are sampled from a defined population they should be a representative sample. If the materials or methods of production change then the results of the testing may not be applicable to the new production without further investigation.

B3.4 Test load

The target test load should be equal to the design action effect for the relevant limit state determined in accordance with this Standard, multiplied by the appropriate factor k_t, given in Table B1 to allow for variability of structural units.

Other appropriate factors should be applied depending on the materials from which the unit is manufactured, including factors covering the effect of time-dependent properties. See materials design Standards for appropriate values.

The distribution and duration of forces applied during the test should represent those forces to which the unit is deemed to be subjected. For a short-term test, the test load should be applied at a uniform rate such that the test duration is not less than 5 min.

The coefficient of variation of structural characteristics of the parent population of the production units (V_{sc}) should be established taking into account variation due to fabrication and material.

TABLE B1

VALUES OF k_t TO ALLOW FOR VARIABILITY OF STRUCTURAL UNITS

Number of units to be tested	Coefficient of variation of structural characteristics (V_{sc}), percent						
	5	10	15	20	25	30	40
1	1.20	1.46	1.79	2.21	2.75	3.45	5.2
2	1.17	1.38	1.64	1.96	2.36	2.86	3.9
3	1.15	1.33	1.56	1.83	2.16	2.56	3.3
4	1.15	1.30	1.50	1.74	2.03	2.37	2.9
5	1.13	1.28	1.46	1.67	1.93	2.23	2.7
10	1.10	1.21	1.34	1.49	1.66	1.85	2.1

NOTE: For values between those listed in the Table, interpolation may be used. Extrapolation is not permitted.

B3.5 Criteria for acceptance

The criteria for acceptance are as follows:

(a) *Strength limit state* The unit is deemed to comply with the strength limit state if it is able to resist the target test load for that limit state.

(b) *Serviceability limit state* When subjected to the target test load for the serviceability limit state, the maximum deformation of the unit (or other serviceability criteria) should be within the serviceability limits specified. After the completion of the test, the residual deflection or deformation of any part of the unit should not exceed 5 percent of the acceptable amount under short-term loading or such other limit as may be specified.

APPENDIX C

GUIDELINES FOR SERVICEABILITY LIMIT STATES

(Informative)

This Appendix gives guidelines for the serviceability limit states resulting from deformation of complete structures and members of structures under load.

Except where absolute limits are required, it is generally best to deal with deflection design in terms of the individual loads being applied (for example, it is usually preferable to deal with the effects of permanent loads separately from the deflection effects of transient or short-term loads). Unlikely combinations of actions need not be considered and total deflection usually only needs to be considered where absolute clearance limits must be maintained.

Guidance on limits for the design of members for serviceability is given in Table C1. This Table identifies deflection limits related to actions with an annual probability of exceedance A2 of 1/25 (i.e., $P = 0.04$) beyond which serviceability problems have been observed. Such boundaries for acceptance are imprecise and should be treated as a guide only. These limits are not applicable in all situations.

Table C1 is arranged into building elements that could be affected by the structure. For each element several possible control phenomena are prescribed, each of which detail—

(a) the serviceability parameter for which the control is intended;

(b) the action that is to be applied to the structure; and

(c) the acceptable response of the element to that action.

Different deflection limits may apply depending on the phenomenon controlled, and the most stringent appropriate limit should control the design. The environment of the observer influences the tolerance of people to sensory deflection. Where a lot of movement is occurring, the stated sensory limits can often be exceeded without complaint.

Further information is given in the commentary to this Standard.

A2 For farm structures of low human occupancy, serviceability criteria should be as follows:

(i) Deflection criteria may be relaxed provided that, taking note of appropriate use of the structure, deflections do not—

  (A) weaken or damage the structure, cladding or lining material and their fixings; or

  (B) produce unacceptable cracking.

(ii) Deflection criteria for flat or near-flat roofs should take into account the possibility of ponding of rainwater.

The design should make adequate provision for any hazards affecting the life of the farm structure that may arise from its use (e.g., typical movement of animals in farm structures designed for animals).

TABLE C1
SUGGESTED SERVICEABILITY LIMIT STATE CRITERIA

A3

Element	Phenomenon controlled	Serviceability parameter	Applied action	Element response (see Notes 1 and 2)
Roof cladding				
Metal roof cladding	Indentation	Residual deformation	Q = 1 kN	Span/600 but <0.5 mm
	De-coupling	Mid-span deflection	$[G, \psi_s Q]$	Span/120
Concrete or ceramic roof cladding	Cracking	Mid-span deflection	$[G, \psi_s Q]$	Span/400
Roof-supporting elements				
Roof members (trusses, rafters, etc.)	Sag	Mid-span deflection	$[G, \psi_l Q]$	Span/300
Roof elements supporting brittle claddings	Cracking	Mid-span deflection	$[G, \psi_s Q]$ or $[W_s]$	Span/400
Ceiling and ceiling supports				
Ceilings with matt or gloss paint finish	Ripple	Mid-span deflection	G	Span/500 (see Note 3)
Ceilings with textured finish	Ripple	Mid-span deflection	G	Span/300
Suspended ceilings	Ripple	Mid-span deflection	G	Span/360
Ceiling support framing	Sag	Mid-span deflection	G	Span/360
Ceilings with plaster finish	Cracking	Mid-span deflection	$[G, \psi_s Q]$ or $[W_s]$	Span/200
Wall elements				
Columns	Side sway	Deflection at top	W_s	Height/500
Portal frames (frame racking action)	Roof damage	Deflection at top	$[W_s]$ or $[E_s]$	Spacing/200 (Note 4)
Lintel beams (vertical sag)	Doors/windows jam	Mid-span deflection	W_s	Span/240 but <12 mm (see Note 5)
Walls—General (face loaded)	Discerned movement	Mid-height deflection	W_s	Height/150
	Impact: soft body (neighbours notice)	Mid-height deflection	Q = 0.7 kN	Height/200 but <12 mm (see Note 6)
	Supported elements rattle	Mid-height deflection	W_s	Height/1000
Walls—Specific claddings (see Note 7):				
Brittle cladding (ceramic) face loaded	Cracking	Mid-height deflection	W_s	Height/500
Masonry walls (in plane)	Noticeable cracking	Deflection at top	$[W_s]$ or $[E_s]$	Height/600
Masonry walls (face loading)	Noticeable cracking	Deflection at top	$[W_s]$ or $[E_s]$	Height/400
Plaster/gypsum walls (in plane)	Lining damage	Mid-height deflection	W_s	Height/300
Plaster/gypsum walls (face loading)	Lining damage	Mid-height deflection	$[W_s]$ or $[E_s]$	Height/200
Movable partitions (soft body impact)	System damage	Deflection at top	Q = 0.7 kN	Height/160
Glazing systems	Bowing	Mid-span deflection	W_s	Span/400
Windows, facades, curtain walls	Facade damage	Mid-span deflection	$[W_s]$ or $[E_s]$	Span/250
Fixed glazing systems	Glass damage	Deflection	$[W_s]$ or $[E_s]$	2 × glass clearance (see Note 3)
Floors and floor supports				
Beams where line-of-sight is along invert	Sag	Mid-span deflection	$[G, \psi_l Q]$	Span/500 (see Notes 8, 9)
Beams where line-of-sight is across soffit	Sag	Mid-span deflection	$[G, \psi_l Q]$	Span/250
Flooring	Ripple	Mid-span deflection	$[G, \psi_l Q]$	Span/300
Floor joists/beams	Sag	Mid-span deflection	$[G, \psi_l Q]$	Span/300
Floors	Vibration	Static midspan deflection	Q = 1.0 kN	less than 1 to 2 mm (see Note 10)
Normal floor systems	Noticeable sag	Mid-span deflection	$[G, \psi_l Q]$	Span/400
Specialist floor systems	Noticeable sag	Mid-span deflection	$[G, \psi_l Q]$	Span/600
Floors—Side-sway (acceleration)	Sway	Acceleration at floor	$W_s (P=5)$	<0.01g (see Note 11)
Floors—Supporting masonry walls	Wall cracking	Mid-span deflection	$[G, \psi_l Q]$	Span/500
Floors—Supporting plaster lined walls	Cracks in lining	Mid-span deflection	$[G, \psi_l Q]$	Span/300
Floors supporting existing masonry walls—Underpinning floors	Wall cracking	Mid-span deflection	$[G, \psi_l Q]$	Span/750
Floors—For access for working by operators and maintenance	Sag	Midspan deflection	Q = 1 kN	Span/250
Handrails—Post and rail system	Side sway	Mid-span system deflection	Q = 1.5 kN/m	Height/60 + Span/240

NOTES:

1 Long-term creep, when present, needs to be included in assessing the long-term deflection of members that are prone to creep. In such cases, the long-term factored occupancy load, $G + \psi_l Q$, should be considered for the creep component and the difference between the short-term and long-term factored occupancy load, $(\psi_s - \psi_l)Q$ added to account for the incremental short-term deflection.

2 The span or height ratios used in the deflection criteria are the clear spacing between points of support.

3 The deflection limits for ceilings or floors are strongly influenced by the surface finish. Glass is an extreme example where the reflective surfaces amplify apparent bowing as the reflected images move with the surface distortions. Observers are often disturbed by such movements. Ripple effects appear more pronounced when the surface is flat (and has a reflective gloss finish). Textured surfaces tend to disguise ripple effects. Surfaces that extend over a wide expanse reveal both ripple and sag effects when light is reflected from the surface. Where the texture of the surface is unknown, the more stringent criteria of highly reflective surfaces will be conservative.

4 The limiting deflection for portal frame knee deflections is related to the behaviour of the cladding between the 'free portal' and a more rigid plane (typically the end wall of a structure). The deflection limit of such portals is based on the bay spacing and ability of the cladding to withstand in-plane shear distortion.

5 Problems with visually sensed deflections are frequently dependent on the presence of a visual cue for the observer to gauge linearity. Deflection limits are therefore a function of the line of sight of the observer.

6 Walls and partitions require stiffness control to minimise disturbance of elements or people often on the reverse side of the wall or partition (e.g., neighbours beyond inter-tenancy walls). The response of the wall to soft-body impact is greatly influenced by the nature and characteristics of the impacting body. The deflection criteria stated (height/200 from a concentrated load of 0.7 kN at mid-height) has been simplified for ease of application by designers. It is based upon a running person falling against a wall. Internal partitions may be subjected to differential pressures, which result from wind. A net coefficient of 0.5 may be considered appropriate when used in conjunction with the serviceability wind pressures.

7 Often different wall claddings have different tolerances to movement. Some of these have been specifically listed.

8 Where members are pre-cambered, the pre-camber present can be deducted from assessments of sag. Where construction progresses in stages, the incremental permanent action only needs to be considered for sag.

9 Floor sag may result in furniture that rocks or is not firmly seated or drainage surfaces that do not function adequately. Specialist floors are those upon which trolleys may move, sensitive equipment may be installed or special activities (e.g. bowls, etc.) may be undertaken. More restrictive deflection limits may be appropriate is such cases.

10 Floor vibration problems are very complex. Problem floors usually have low levels of elastic damping present. The limiting criteria stated (between 1 and 2 mm under a 1 kN point load) should give a guide as to whether the floor may have vibration problems. When the criterion is not satisfied, a more detailed examination of the dynamic behaviour of the floor may be merited. Where a floor system may be used for group rhythmic activity, such as marching, dancing, concerts, jumping exercises or gymnastics, and has a fundamental frequency of vibration less than 8 Hz, then a specific study of the resonant response should be considered, to demonstrate that the building remains functional.

A3 11 The criteria of 0.01g relates to a frequency range of 0.05 to 1 Hz. It is a first test to determine if further investigation is required. The sensitivity of people to motion in tall buildings varies widely and further research is being conducted.

A3

APPENDIX D

'Text deleted'

A1

APPENDIX E

'Text deleted'

A2
A5

APPENDIX F

ANNUAL PROBABILITY OF EXCEEDANCE (FOR AUSTRALIAN USE ONLY—STRUCTURES FOR WHICH DESIGN EVENTS ARE NOT GIVEN ELSEWHERE)

(Normative)

F1 GENERAL

This Appendix specifies minimum design events for safety (ultimate limit states design), in terms of annual probability of exceedance for wind, snow and earthquake, for the design of structures in Australia that are not covered either by the Building Code of Australia or by other Standards (such as that for transmission line structures). This Appendix does not apply to structures in New Zealand; for those structures, Section 3 of this Standard applies.

NOTE: Structures covered by this Appendix may include industrial structures, mining and oil and gas structures, and communication structures.

F2 IMPORTANCE LEVELS

The importance level of a structure shall be determined in accordance with Table F1.

Structures that have multiple uses shall be assigned the highest importance level applicable for any of those uses. Where an adjacent structure provides access to another structure with a higher importance level, then the structure providing access shall be designated the same importance level as the structure to which it provides access.

NOTE: Structures that have very low frequency fundamental modes of vibration should be considered for special study of their earthquake design event and structural response (e.g. very long conveyors).

TABLE F1
STRUCTURE TYPES FOR IMPORTANCE LEVELS

Consequences of failure	Description	Importance level	Comment
Low	**Low** consequence for loss of human life, *or* **small or moderate** economic, social *or* environmental consequences	1	Minor structures (failure not likely to endanger human life)
Ordinary	**Medium** consequence for loss of human life, *or* **considerable** economic, social *or* environmental consequences	2	Normal structures and structures not falling into other levels
High	**High** consequence for loss of human life, *or* **very great** economic, social or environmental consequences	3	Major structures (affecting crowds)
		4	Post-disaster structures (post-disaster functions or dangerous activities)
Exceptional	Circumstances where reliability must be set on a case by case basis	5	Exceptional structures

F3 DESIGN EVENTS

A2
A5
Design events (in terms of annual probability of exceedance) shall be as given in Table F2 for use in determining the actions affecting the structure. Structures whose failure might result in loss of human life shall not be designed for less than a 25 year life.

Importance Level 4 structures shall not be designed for less than a 25 year life. For importance Level 4 structures with design working life of 100 years or more, the design events shall be determined by a risk analysis but shall have probabilities less than or equal to those for importance Level 3.

The design working life of structures that are erected for a number of short periods of use, and dismantled between each, is equal to the total of the periods of use.

NOTE: The design life for normal structures is generally taken as 50 years. For further guidance on the use of Table F2 see AS/NZS 1170.0 Supp 1, *Structural design actions—General principles—Commentary (Supplement to AS/NZS 1170.0:2002.)*

A2
A4
A5

TABLE F2

ANNUAL PROBABILITY OF EXCEEDANCE OF THE DESIGN EVENTS FOR ULTIMATE LIMIT STATES

Design working life	Importance level	Design events for safety in terms of annual probability of exceedance		
		Wind	**Snow**	**Earthquake (see Note 1)**
Construction equipment (e.g. props, scaffolding, braces and similar)	2	1/100	1/50	Not required (see Note 3)
5 years or less (only for structures whose failure presents no risk to human life, see Note 2)	1	1/25	1/25	Not required (see Note 3)
	2	1/50	1/50	
	3	1/100	1/100	
25 years	1	1/100	1/25	Not required (see Note 3)
	2	1/200	1/50	1/250
	3	1/500	1/100	1/500
	4	1/1000	1/250	1/1000
50 years	1	1/100 (non-cyclonic)	1/100	1/250
	1	1/200 (cyclonic)		
	2	1/500	1/150	1/500
	3	1/1000	1/200	1/1000
	4	1/2500	1/500	1/2500
100 years or more	1	1/500	1/200	1/250
	2	1/1000	1/250	1/1000
	3	1/2500	1/500	1/2500
	4	(see Paragraph F3)	(see Paragraph F3)	(see Paragraph F3)

NOTES:

1 Design for earthquake is not required for structures for primary produce with low human occupancy.

2 For a design working life (L) between 5 and 100 years that is not listed in Table F2, the annual probability of exceedence ($1/R$) for wind and earthquake events is calculated as r/L, where the lifetime risk (r) is given in the following table:

Importance level	Risk of exceedance of design load (r)
1	0.20 to 0.25
2	0.10 to 0.125
3	0.04 to 0.05
4	0.020 to 0.025

3 Earthquake loads for these annual probabilities are low and design for robustness or other actions will provide sufficient resistance.

4 Structures in wind Regions C and D (i.e. cyclonic regions, as defined in AS/NZS 1170.2) that are erected and remain erected, only during the period of May to October, may be designed for regional wind speeds given in AS/NZS 1170.2, for Region A, or alternatively from a specific analysis of non-cyclonic wind events for the site. A structure not designed for cyclonic wind speeds shall not remain erected during the months of November to April inclusive.

AMENDMENT CONTROL SHEET

AS/NZS 1170.0:2002

Amendment No. 1 (2003)

REVISED TEXT

SUMMARY: This Amendment applies to the Clauses 1.3, 2.2, 2.3 and 4.2.3, and Appendix E.

Published on 8 January 2003.

Amendment No. 2 (2003)

REVISED TEXT

SUMMARY: This Amendment applies to the Clauses 1.4, 2.2 and 2.3, Section 3, and Appendices C and F (new).

Published on 28 November 2003.

Amendment No. 3 (2011)

REVISED TEXT

SUMMARY: This Amendment applies to Clauses 1.1, 1.2, 1.3, 1.5, 2.3, 4.2.1, 4.2.2, 5.2, 6.2.2 and 6.2.5, Table 4.1 and Appendices C and D.

Published on 11 April 2011.

Amendment No. 4 (2005)

CORRECTION

SUMMARY: This Amendment applies to the Preface and Clauses 1.3, 2.2, 3.2, 3.4.2, Table 3.3 and Table F2.

Published on 28 April 2005.

Amendment No. 5 (2011)

REVISED TEXT

SUMMARY: This Amendment applies to the Preface, Clause 1.3 and Appendix F.

Published on 22 September 2011.

NOTES

Standards Australia

Standards Australia is an independent company, limited by guarantee, which prepares and publishes most of the voluntary technical and commercial standards used in Australia. These standards are developed through an open process of consultation and consensus, in which all interested parties are invited to participate. Through a Memorandum of Understanding with the Commonwealth government, Standards Australia is recognized as Australia's peak national standards body.

Standards New Zealand

The first national Standards organization was created in New Zealand in 1932. The Standards Council of New Zealand is the national authority responsible for the production of Standards. Standards New Zealand is the trading arm of the Standards Council established under the Standards Act 1988.

Australian/New Zealand Standards

Under a Memorandum of Understanding between Standards Australia and Standards New Zealand, Australian/New Zealand Standards are prepared by committees of experts from industry, governments, consumers and other sectors. The requirements or recommendations contained in published Standards are a consensus of the views of representative interests and also take account of comments received from other sources. They reflect the latest scientific and industry experience. Australian/New Zealand Standards are kept under continuous review after publication and are updated regularly to take account of changing technology.

International Involvement

Standards Australia and Standards New Zealand are responsible for ensuring that the Australian and New Zealand viewpoints are considered in the formulation of international Standards and that the latest international experience is incorporated in national and Joint Standards. This role is vital in assisting local industry to compete in international markets. Both organizations are the national members of ISO (the International Organization for Standardization) and IEC (the International Electrotechnical Commission).

Visit our web sites

www.standards.org.au

www.standards.com.au

www.standards.co.nz

STANDARDS

NEW ZEALAND

PAEREWA AOTEAROA

AS/NZS 1170.2:2011

STANDARDS
NEW ZEALAND
PAEREWA AOTEAROA

Structural Design Actions

Part 2 – Wind actions

AS/NZS 1170.2:2011

STANDARDS
Australia

Australian/New Zealand Standard™

Structural design actions

Part 2: Wind actions

Originated in Australia as part of AS CA1—1933.
Originated in New Zealand as part of NZS 1900:1964.
Previous Australian edition AS 1170.2—1989.
Previous New Zealand edition NZS 4203:1992.
AS 1170.2—1989 and NZS 4203:1992 jointly revised, amalgamated and redesignated in part as AS/NZS 1170.2:2002.
Second edition 2011.

Jointly published by SAI Global Limited under licence from Standards Australia Limited, GPO Box 476, Sydney, NSW 2001 and by Standards New Zealand, Private Bag 2439, Wellington 6140.

ISBN 978 0 7337 9805 4

PREFACE

This Standard was prepared by the Joint Standards Australia/Standards New Zealand Committee, BD-006, General Design Requirements and Loading on Structures, to supersede AS/NZS 1170.2:2002.

The objective of this Standard is to provide wind actions for use in the design of structures subject to wind action. It provides a detailed procedure for the determination of wind actions on structures, varying from those less sensitive to wind action to those for which dynamic response must be taken into consideration.

The objectives of this revision are to remove ambiguities, to incorporate recent research and experiences from recent severe wind events in Australia and New Zealand.

This Standard is Part 2 of the AS/NZS 1170 series *Structural design actions*, which comprises the following parts:

AS/NZS 1170, Structural design actions
Part 0: General principles
Part 1: Permanent, imposed and other actions
Part 2: Wind actions
Part 3: Snow and ice actions

AS 1170, Structural design actions
Part 4: Earthquake actions in Australia

NZS 1170, Structural design actions
Part 5: Earthquake actions—New Zealand

The wind speeds provided are based on analysis of existing data. No account has been taken of any possible future trend in wind speeds due to climatic change.

This edition differs from the previous edition as follows:

(a) A torsional loading requirement in the form of an eccentricity of loading is prescribed for tall buildings greater than 70 m in height (see Clause 2.5.4).

(b) Addition of windborne debris impact loading criteria (Clause 2.5.7).

(c) Regional wind speeds V_1, V_{250}, V_{2500}, V_{5000} and V_{10000} have been added for serviceability design requirements, and for compatibility with AS/NZS 1170.0 (see Clause 3.2).

(d) Nominally closed doors, such as roller doors, to be treated as potential dominant openings unless it is shown that the doors and their supports and fixings are capable of resisting the applied wind loads and the impact of debris (see Clause 5.3.2).

(e) Addition of a new clause requiring consideration of wind loads on internal walls and ceilings (see Clause 5.3.4).

(f) Adjustment of internal pressure coefficients in Table 5.1(B) for dominant openings on leeward walls, side walls and roof, to more correctly reflect the relationship between internal and external pressures when multiple opening occur.

(g) Clause 5.4.3 on the combination factor (K_c) has been changed to remove some ambiguities and confusion in the previous edition. An expanded Table 5.5 gives more examples of the use of this factor.

(h) Several changes to Table 5.6 on local pressure factors have been made, including the following:

    (i) A factor of 1.5 for small areas on windward walls.

(ii) A factor of 3.0 for small areas near the corners of roofs.

(iii) Case SA5 ($K_\ell = 3.0$) will, in future, not be required to be applied to those buildings greater than 25 m in height with low aspect ratios.

(i) Values of maximum structural damping ratios for structures with dynamic response to wind have been made informative rather than normative.

NOTE: Users should seek other sources for advice on possible values of damping as a function of height of building and amplitude of vibration.

(j) A note to Table C3, Appendix C, for shape factors for curved roofs has been added to cover the case of building height to rise greater than 2.

(k) The load distribution specified in Paragraph D5, Appendix D, for cantilevered roofs has been revised to reflect recent research.

(l) Drag coefficients for pentagonal sections have been added to Table E4, Appendix E.

(m) Drag coefficients for sections of UHF television antennas Types 1 and 3 in Table E7, Appendix E, have been revised. The value of drag force coefficients for the Type 2 antenna have been removed from the Standard, since this type has not been used in Australia or New Zealand for many years.

The Joint Committee has considered exhaustive research and testing information from Australian, New Zealand and overseas sources in the preparation of this Standard. The design wind actions prescribed in this Standard are the minimum for the general cases described.

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.

Statements expressed in mandatory terms in notes to tables and figures are deemed to be an integral part of this Standard.

Notes to the text contain information and guidance and are not considered to be an integral part of the Standard.

The Joint Committee is currently considering possible amendments following recent severe wind events, including tropical cyclone Yasi in Australia.

CONTENTS

Page

STANDARDS AUSTRALIA/STANDARDS NEW ZEALAND

Australian/New Zealand Standard
Structural design actions

Part 2: Wind actions

SECTION 1 GENERAL

1.1 SCOPE

This Standard sets out procedures for determining wind speeds and resulting wind actions to be used in the structural design of structures subjected to wind actions other than those caused by tornadoes.

The Standard covers structures within the following criteria:

(a) Buildings less than or equal to 200 m high.

(b) Structures with roof spans less than 100 m.

(c) Structures other than offshore structures, bridges and transmission towers.

NOTES:

1 This Standard is a stand-alone document for structures within the above criteria. It may be used, in general, for all structures but other information may be necessary.

2 Where structures have natural frequencies less than 1 Hz, Section 6 requires dynamic analysis to be carried out (see Section 6).

3 In this document, the words 'this Standard' indicate AS/NZS 1170.2, which is regarded as Part 2 of the AS/NZS 1170 series of Standards (see Preface).

4 Further advice should be sought for geometries not described in this Standard, such as the roofs of podiums below tall buildings.

1.2 APPLICATION

This Standard shall be read in conjunction with AS/NZS 1170.0.

This Standard may be used as a means for demonstrating compliance with the requirements of Part B1 of the Building Code of Australia.

NOTE: Use of methods or information not given in this Standard should be justified by a special study (see AS/NZS 1170.0).

1.3 NORMATIVE REFERENCES

The following are the normative documents referenced in this Standard:

AS
4040 Methods of testing sheet roof and wall cladding
4040.3 Part 3: Resistance to wind pressures for cyclone regions

AS/NZS
1170 Structural design actions
1170.0 Part 0: General principles

Australian Building Codes Board
BCA Building Code of Australia

1.4 DETERMINATION OF WIND ACTIONS

Values of wind actions (W) for use in design shall be established. The values shall be appropriate for the type of structure or structural element, its intended use, design working life and exposure to wind action.

The following wind actions, determined in accordance with this Standard (using the procedures detailed in Section 2 and the values given in the remaining Sections), shall be deemed to comply with the requirements of this Clause:

(a) W_u determined using a regional wind speed appropriate to the annual probability of exceedence (P) specified for ultimate limit states as given in AS/NZS 1170.0, or the Building Code of Australia.

(b) W_s determined using a regional wind speed appropriate to the annual probability of exceedence for the serviceability limit states (see Note 3).

NOTES:

1 Information on serviceability conditions and criteria can be found in AS/NZS 1170.0 (see Preface).

2 Some design processes require the determination of wind pressure (ultimate or serviceability wind pressure). Such pressures should be calculated for the wind speed associated with the annual probability of exceedence (P) appropriate to the limit state being considered.

3 For guidance on Item (b), see AS/NZS 1170.0.

1.5 UNITS

Except where specifically noted, this Standard uses the SI units of kilograms, metres, seconds, pascals, newtons, degrees and hertz (kg, m, s, Pa, N, Hz).

1.6 DEFINITIONS

Definitions of the terms used in this Standard are given in Appendix A.

1.7 NOTATION

The notations used in this Standard are given in Appendix B.

SECTION 2 CALCULATION OF WIND ACTIONS

2.1 GENERAL

The procedure for determining wind actions (W) on structures and elements of structures or buildings shall be as follows:

(a) Determine site wind speeds (see Clause 2.2).

(b) Determine design wind speed from the site wind speeds (see Clause 2.3).

(c) Determine design wind pressures and distributed forces (see Clause 2.4).

(d) Calculate wind actions (see Clause 2.5).

2.2 SITE WIND SPEED

The site wind speeds ($V_{\mathrm{sit},\beta}$) defined for the 8 cardinal directions (β) at the reference height (z) above ground (see Figure 2.1) shall be as follows:

$$V_{\mathrm{sit},\beta} = V_{\mathrm{R}}\, M_{\mathrm{d}}\, (M_{z,\mathrm{cat}}\, M_{\mathrm{s}}\, M_{\mathrm{t}}) \qquad \ldots 2.2$$

where

V_{R} = regional 3 s gust wind speed, in metres per second, for annual probability of exceedence of 1/R, as given in Section 3

M_{d} = wind directional multipliers for the 8 cardinal directions (β) as given in Section 3

$M_{z,\mathrm{cat}}$ = terrain/height multiplier, as given in Section 4

M_{s} = shielding multiplier, as given in Section 4

M_{t} = topographic multiplier, as given in Section 4

Generally, the wind speed is determined at the average roof height (h). In some cases this varies, as given in the appropriate sections, according to the structure.

2.3 DESIGN WIND SPEED

The building orthogonal design wind speeds ($V_{\mathrm{des},\theta}$) shall be taken as the maximum cardinal direction site wind speed ($V_{\mathrm{sit},\beta}$) linearly interpolated between cardinal points within a sector ±45° to the orthogonal direction being considered (see Figures 2.2 and 2.3).

NOTE: That is, $V_{\mathrm{des},\theta}$ equals the maximum value of site wind speed ($V_{\mathrm{sit},\beta}$) in the range [$\beta = \theta \pm 45°$] where β is the cardinal direction clockwise from true North and θ is the angle to the building orthogonal axes.

In cases such as walls and hoardings and lattice towers, where an incident angle of 45° is considered, $V_{\mathrm{des},\theta}$ shall be the maximum value of $V_{\mathrm{sit},\beta}$ in a sector ±22.5° from the 45° direction being considered.

For ultimate state design, $V_{\mathrm{des},\theta}$ shall be not less than 30 m/s for permanent structures (design life greater than 5 years), or less than 25 m/s for temporary structures (design life less than or equal to 5 years).

NOTE: A conservative approach is to design the structure using the wind speed and multipliers for the worst direction. For example, for a building on an escarpment it may be easily checked whether the $V_{\mathrm{R}}\, M_{\mathrm{d}}\, (M_{z,\mathrm{cat}}\, M_{\mathrm{s}}\, M_{\mathrm{t}})$ on the exposed face (towards the escarpment) is the worst case. To simplify design, this value could then be used as the design wind speed for all directions on the building.

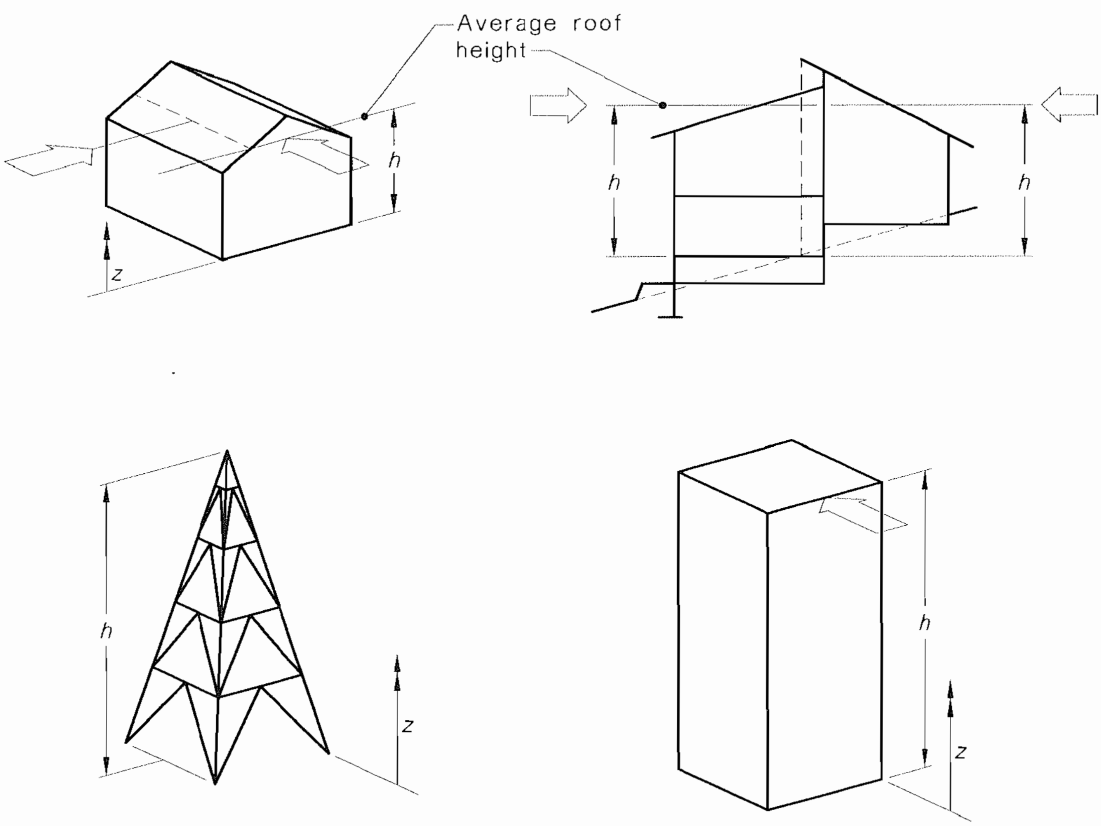


FIGURE 2.1 REFERENCE HEIGHT OF STRUCTURES

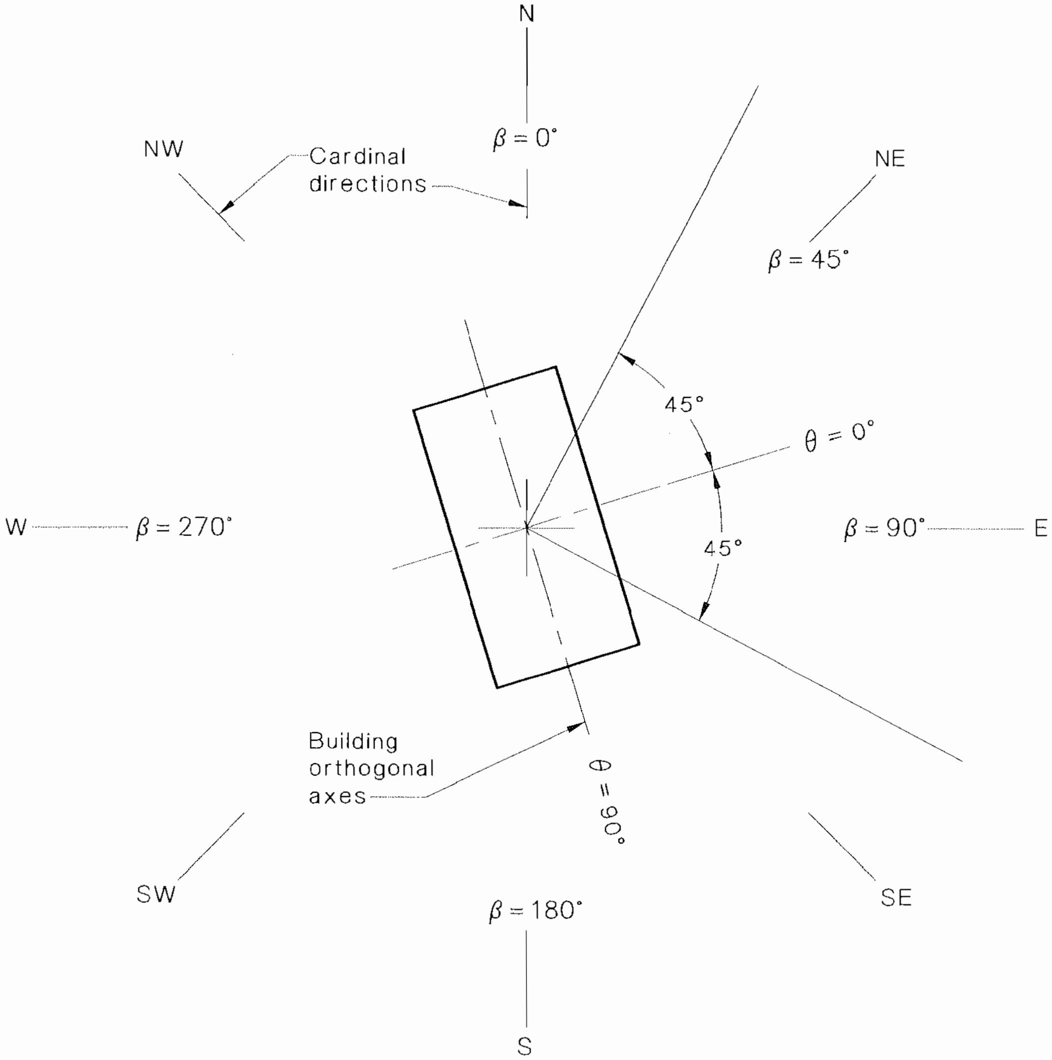
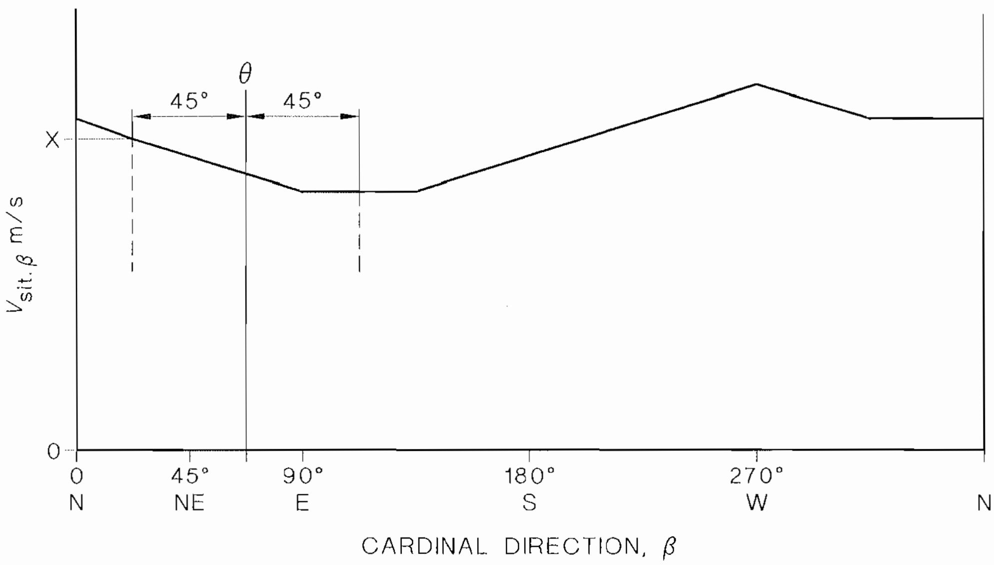


FIGURE 2.2 RELATIONSHIP OF WIND DIRECTIONS AND BUILDING ORTHOGONAL AXES



The value of $V_{des,\theta}$ is the maximum of $V_{sit,\beta}$ in the range θ±45°, which, in the case shown here, is the wind speed X.

FIGURE 2.3 EXAMPLE OF $V_{sit,\beta}$ CONVERSION TO $V_{des,\theta}$

2.4 DESIGN WIND PRESSURE AND DISTRIBUTED FORCES

2.4.1 Design wind pressures

The design wind pressures (p), in pascals, shall be determined for structures and parts of structures as follows:

$$p = (0.5\ \rho_{air})\ [V_{des,\theta}]^2\ C_{fig}\ C_{dyn} \qquad \ldots 2.4(1)$$

where

p = design wind pressure in pascals

= p_e, p_i or p_n where the sign is given by the C_p values used to evaluate C_{fig}

NOTE: Pressures are taken as positive, indicating pressures above ambient and negative, indicating pressures below ambient.

ρ_{air} = density of air, which shall be taken as 1.2 kg/m^3

$V_{des,\theta}$ = building orthogonal design wind speeds (usually, $\theta = 0°$, 90°, 180° and 270°), as given in Clause 2.3

NOTE: For some applications, $V_{des,\theta}$ may be a single value or may be expressed as a function of height (z), e.g. windward walls of tall buildings (>25m).

C_{fig} = aerodynamic shape factor, as given in Section 5

C_{dyn} = dynamic response factor, as given in Section 6 [the value is 1.0 except where the structure is dynamically wind sensitive (see Section 6)]

2.4.2 Design frictional drag force per unit area

The design wind frictional drag force per unit area (f), in pascals, shall be taken for structures and parts of structures as follows:

$$f = (0.5\ \rho_{air})\ [V_{des,\theta}]^2\ C_{fig}\ C_{dyn} \qquad \ldots 2.4(2)$$

2.5 WIND ACTIONS

2.5.1 General

Wind actions (W_u and W_s) for use in AS/NZS 1170.0 shall be determined as given in Clauses 2.5.2 to 2.5.5 and deflections and accelerations of dynamically wind-sensitive structures as given in Clause 2.5.6.

2.5.2 Directions to be considered

Wind actions shall be derived by considering wind from no fewer than four orthogonal directions aligned to the structure.

2.5.3 Forces on surfaces or structural elements

2.5.3.1 *Forces derived from wind pressure*

To determine wind actions, the forces (F) in newtons, on surfaces or structural elements, such as a wall or a roof, shall be the vector sum of the forces calculated from the pressures applicable to the assumed areas (A), as follows:

$$F = \sum(p_z A_z) \qquad \ldots 2.5(1)$$

where

p_z = design wind pressure in pascals (normal to the surface) at height z, calculated in Clause 2.4.1

NOTE: The sign convention for pressures leads to forces towards the surface for positive pressures and forces away from the surface for negative pressures.

A_z = a reference area, in square metres, at height z, upon which the pressure at that height (p_z) acts

For enclosed buildings, internal pressures shall be taken to act simultaneously with external pressures, including the effects of local pressure factors (K_ℓ).

NOTE: Generally, the most severe combinations of internal and external pressures shall be selected for design, but some reduction in the combined load may be applicable according to Clause 5.4.3.

Where it is required to divide the height of a tall structure into sectors to calculate wind actions {for example, windward walls of tall buildings [Table 5.2(A)] or for lattice towers (Clause E4.1)}, the sectors shall be of a size to represent reasonably the variation of wind speed with height, as given in Clause 4.2.

2.5.3.2 *Force derived from frictional drag*

To determine wind actions, the forces (F), in newtons, on a building element, such as a wall or a roof, shall be the vector sum of the forces calculated from distributed frictional drag stresses applicable to the assumed areas, as follows:

$$F = \sum(f_z A_z) \qquad \ldots 2.5(2)$$

where

f_z = design frictional drag per unit area parallel to the surface (calculated in Clause 2.4.2) at height z, in pascals

A_z = a reference area, in square metres, on which the distributed frictional drag stresses (f_z) act

2.5.3.3 *Forces derived from force coefficients*

Appendices E and F cover structures for which shape factors are given in the form of force coefficients rather than pressure coefficients. In these cases, to determine wind actions, the forces (F) in newtons, shall be determined as follows:

$$F = (0.5\ \rho_{air})\ [V_{des,\theta}]^2\ C_{fig}\ C_{dyn}\ A_z \qquad \ldots 2.5(3)$$

where

A_z = as defined in Paragraph E4, Appendix E, for lattice towers

= $l \times b$ for members and simple sections in Paragraph E3, Appendix E

= A_{ref} as defined in Appendix F for flags and circular shapes

2.5.4 Forces and moments on complete structures

To determine wind actions, the total resultant forces and overturning moments on complete structures shall be taken to be the summation of the effects of the external pressures on all surfaces of the building.

For rectangular enclosed buildings with h >70 m, torsion shall be applied, based on an eccentricity of $0.2b$ with respect to the centre of geometry of the building on the along-wind loading.

NOTE: For d/b >1.5, the torsional moments are primarily generated by crosswind forces and specialist advice should be sought.

For dynamic effects, the combination of along-wind and crosswind responses shall be calculated in accordance with Section 6.

2.5.5 Performance of fatigue-sensitive elements

In regions C and D, cladding, its connections and immediate supporting members and their fixings shall demonstrate performance under the pressure sequences defined in AS 4040.3 and the Building Code of Australia, based on the ultimate limit state wind pressure on external and internal surfaces, as determined in accordance with this Standard.

2.5.6 Deflections of dynamically wind-sensitive structures

Wind actions for dynamically wind-sensitive structures (as defined in Clause 6.1) which may include chimneys, masts and poles of circular cross-section, shall be calculated in accordance with Section 6.

NOTE: Information on peak acceleration of other wind-sensitive structures is given in Appendix G.

2.5.7 Impact loading from windborne debris

Where windborne debris loading is specified, the debris impact shall be equivalent to—

(a) timber member of 4 kg mass with a nominal cross-section of 100 mm × 50 mm impacting end on at 0.4 V_R for horizontal trajectories and 0.1 V_R for vertical trajectories; and

(b) spherical steel ball 8 mm diameter (approximately 2 grams mass) impacting at 0.4 V_R for horizontal trajectories and 0.3 V_R for vertical trajectories

where V_R is the regional wind speed given in Clause 3.2.

NOTES:

1 Examples of the use of this clause would be the application of Clause 5.3.2 or the building envelope enclosing a shelter room.

2 These impact loadings should be applied independently in time and location.

SECTION 3 REGIONAL WIND SPEEDS

3.1 GENERAL

This Section shall be used to calculate gust wind speeds appropriate to the region in which a structure is to be constructed, including wind direction effects.

3.2 REGIONAL WIND SPEEDS (V_R)

Regional wind speeds (V_R) for all directions based on 3-second gust wind data shall be as given in Table 3.1 for the regions shown in Figure 3.1(A) and Figure 3.1(B) where R (average recurrence interval) is the inverse of the annual probability of exceedence of the wind speed for ultimate or serviceability limit states.

The calculated value of V_R shall be rounded to the nearest 1 m/s.

TABLE 3.1

REGIONAL WIND SPEEDS

Regional wind speed (m/s)	Region				
	Non-cyclonic			Cyclonic	
	A (1 to 7)	W	B	C	D
V_1	30	34	26	$23\times F_C$	$23\times F_D$
V_5	32	39	28	$33\times F_C$	$35\times F_D$
V_{10}	34	41	33	$39\times F_C$	$43\times F_D$
V_{20}	37	43	38	$45\times F_C$	$51\times F_D$
V_{25}	37	43	39	$47\times F_C$	$53\times F_D$
V_{50}	39	45	44	$52\times F_C$	$60\times F_D$
V_{100}	41	47	48	$56\times F_C$	$66\times F_D$
V_{200}	43	49	52	$61\times F_C$	$72\times F_D$
V_{250}	43	49	53	$62\times F_C$	$74\times F_D$
V_{500}	45	51	57	$66\times F_C$	$80\times F_D$
V_{1000}	46	53	60	$70\times F_C$	$85\times F_D$
V_{2000}	48	54	63	$73\times F_C$	$90\times F_D$
V_{2500}	48	55	64	$74\times F_C$	$91\times F_D$
V_{5000}	50	56	67	$78\times F_C$	$95\times F_D$
V_{10000}	51	58	69	$81\times F_C$	$99\times F_D$
V_R ($R \geq 5$ years)	$67-41R^{-0.1}$	$104-70R^{-0.045}$	$106-92R^{-0.1}$	$F_C\,(122-104R^{-0.1})$	$F_D\,(156-142R^{-0.1})$

NOTES:

1 Values for V_1 have not been calculated by the formula for V_R.

2 For ultimate or serviceability limit states, refer to the Building Code of Australia or AS/NZS 1170.0 for information on values of annual probability of exceedence appropriate for the design of structures.

3.3 WIND DIRECTION MULTIPLIER (M_d)

3.3.1 Regions A and W

The wind direction multiplier (M_d) for regions A and W shall be as given in Table 3.2.

3.3.2 Regions B, C and D

The wind direction multiplier (M_d) for all directions in regions B, C and D shall be as follows:

(a) 0.95 for determining the resultant forces and overturning moments on complete buildings and wind actions on major structural elements.

(b) 1.0 for all other cases (including cladding and immediate supporting members).

TABLE 3.2

WIND DIRECTION MULTIPLIER (M_d)

Cardinal directions	Region A1	Region A2	Region A3	Region A4	Region A5	Region A6	Region A7	Region W
N	0.90	0.80	0.85	0.90	1.00	0.85	0.90	1.00
NE	0.80	0.80	0.80	0.85	0.85	0.95	0.90	0.95
E	0.80	0.80	0.80	0.90	0.80	1.00	0.80	0.80
SE	0.80	0.95	0.80	0.90	0.80	0.95	0.90	0.90
S	0.85	0.90	0.80	0.95	0.85	0.85	0.90	1.00
SW	0.95	0.95	0.85	0.95	0.90	0.95	0.90	1.00
W	1.00	1.00	0.90	0.95	1.00	1.00	1.00	0.90
NW	0.95	0.95	1.00	0.90	0.95	0.95	1.00	0.95
Any direction	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

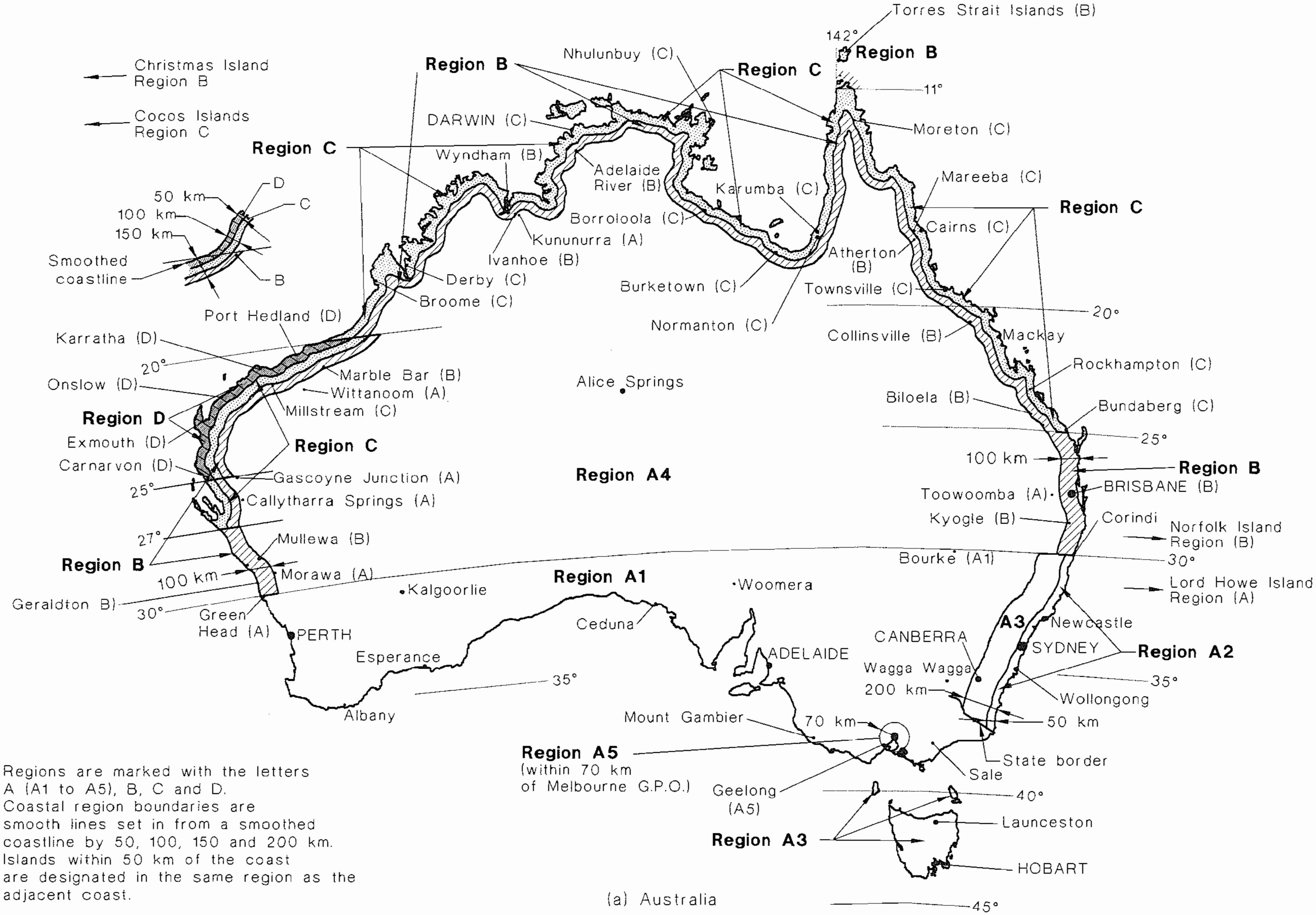
3.4 FACTORS FOR REGIONS C AND D (F_C, F_D)

The wind speeds given in Table 3.1 for regions C and D include additional factors (F_C and F_D) which shall be as follows:

(a) For $R \geq 50$ yrs, $F_C = 1.05$ and $F_D = 1.1$.

(b) For $R < 50$ yrs, $F_C = F_D = 1.0$.

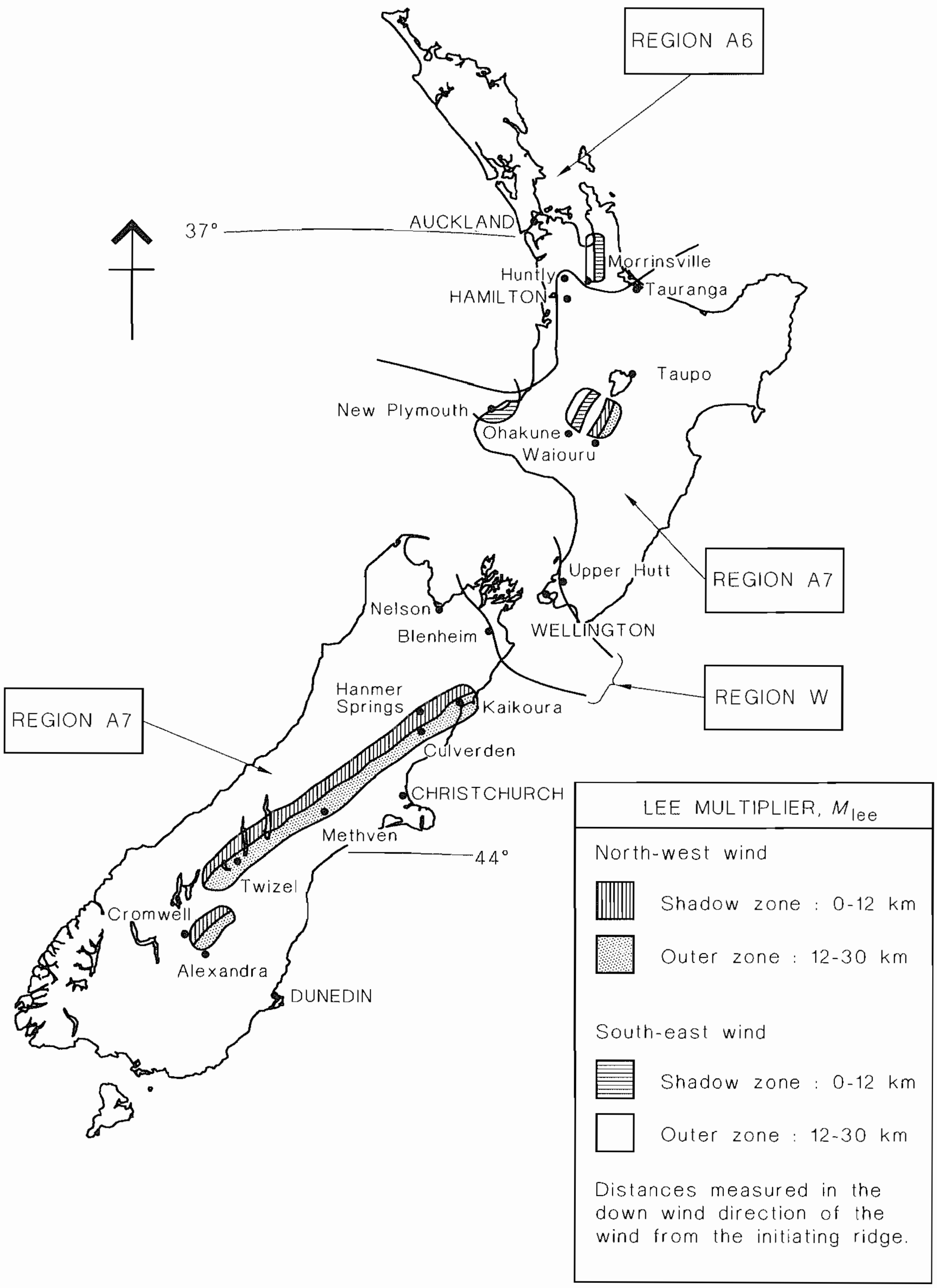
NOTE: The factors in this Clause have been introduced to allow for additional uncertainties in the prediction of design wind speeds in Regions C and D (tropical cyclone regions). The values of these factors may be revised in the future following simulations based on recorded cyclone tracks. Such an analysis would naturally include cyclone activity throughout the northern coast of Australia (i.e. in regions C and D).



Regions are marked with the letters A (A1 to A5), B, C and D.
Coastal region boundaries are smooth lines set in from a smoothed coastline by 50, 100, 150 and 200 km.
Islands within 50 km of the coast are designated in the same region as the adjacent coast.

(a) Australia

FIGURE 3.1(A) WIND REGIONS



(b) New Zealand

FIGURE 3.1(B) WIND REGIONS

SECTION 4 SITE EXPOSURE MULTIPLIERS

4.1 GENERAL

This Section shall be used to calculate the exposure multipliers for site conditions related to terrain/height ($M_{z,cat}$), shielding (M_s) and topography (M_t).

The design shall take account of known future changes to terrain roughness when assessing terrain category and to buildings providing shielding when assessing shielding.

4.2 TERRAIN/HEIGHT MULTIPLIER ($M_{z,cat}$)

4.2.1 Terrain category definitions

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.

(b) *Category 2*—Water surfaces, open terrain, grassland with few, well-scattered obstructions having heights generally from 1.5 m to 10 m.

(c) *Category 3*—Terrain with numerous closely spaced obstructions 3 m to 5 m high, such as areas of suburban housing.

(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.

Selection of terrain category shall be made with due regard to the permanence of the obstructions that constitute the surface roughness. In particular, vegetation in tropical cyclonic regions shall not be relied upon to maintain surface roughness during wind events.

4.2.2 Determination of terrain/height multiplier ($M_{z,cat}$)

The variation with height (z) of the effect of terrain roughness on wind speed (terrain and structure height multiplier, $M_{z,cat}$) shall be taken from the values for fully developed profiles given in Tables 4.1(A) and 4.1(B). For intermediate values of height and terrain category, use linear interpolation.

TABLE 4.1(A)

TERRAIN/HEIGHT MULTIPLIERS FOR GUST WIND SPEEDS IN FULLY DEVELOPED TERRAINS—SERVICEABILITY LIMIT STATE DESIGN—ALL REGIONS AND ULTIMATE LIMIT STATE—REGIONS A1 TO A7, W AND B

Height (z) m	Terrain/height multiplier ($M_{z,cat}$)			
	Terrain category 1	Terrain category 2	Terrain category 3	Terrain category 4
≤3	0.99	0.91	0.83	0.75
5	1.05	0.91	0.83	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

NOTE: For intermediate values of height z and terrain category, use linear interpolation.

TABLE 4.1(B)

TERRAIN/HEIGHT MULTIPLIERS FOR GUST WIND SPEEDS IN FULLY DEVELOPED TERRAINS—ULTIMATE LIMIT STATE DESIGN—REGIONS C AND D ONLY

Height (z) m	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, use linear interpolation.

4.2.3 Changes in terrain category

When considering a direction where the wind approaches across ground with changes in terrain category that lie within the averaging distances given in Table 4.2(A) for structure height, the terrain and structure height multiplier ($M_{z,cat}$) shall be taken as the weighted average value over the averaging distance upwind of the structure at height z above ground level [see Figure 4.1(a)].

The weighted average of $M_{z,cat}$ shall be weighted by the length of each terrain upwind of the structure allowing for the lag distance at each terrain category change. An example is given in Figure 4.1(b).

For evaluation at height (z), a change in terrain incorporates a lag distance (x_i) given as follows:

$$x_i = z_{0,r}\left[\frac{z}{0.3z_{0,r}}\right]^{1.25} \qquad \ldots 4.2$$

where

x_i = distance downwind from the start of a new terrain roughness to the position where the developed height of the inner layer equals z (lag distance)

$z_{0,r}$ = larger of the two roughness lengths at a boundary between roughnesses, as given in Table 4.2(B)

z = reference height on the structure above the average local ground level

TABLE 4.2(A)

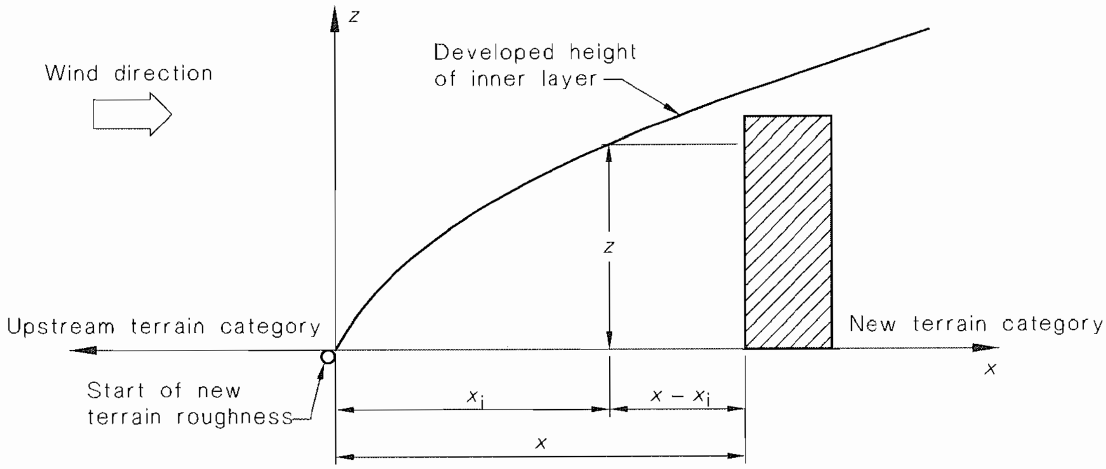
AVERAGING DISTANCE FOR STRUCTURE HEIGHT

Structure height (m)	Averaging distance upwind of structure (m)
h <50	1000
$50 \leq h < 100$	2000
$100 \leq h \leq 200$	3000

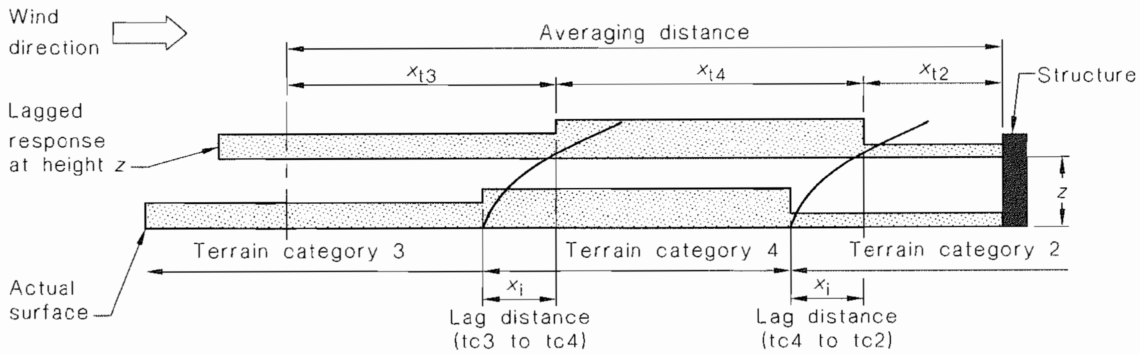
TABLE 4.2(B)

ROUGHNESS LENGTHS FOR TERRAIN CATEGORIES

Terrain category	Roughness length (m)
1	0.002
2	0.02
3	0.2
4	2.0



(a) Notation for changes in terrain category



$$M_{z,cat} = \frac{M_{z,2}\, x_{t2} + M_{z,4}\, x_{t4} + M_{z,3}\, x_{t3}}{\text{Averaging distance}} \quad \text{for the case illustrated}$$

(b) Examples of changes in terrain category

FIGURE 4.1 CHANGES IN TERRAIN CATEGORY

4.3 SHIELDING MULTIPLIER (M_s)

4.3.1 General

Shielding may be provided by upwind buildings or other structures. Shielding from trees or vegetation is not permitted in this Standard.

The shielding multiplier (M_s) that is appropriate to a particular direction shall be as given in Table 4.3. The shielding multiplier shall be 1.0 where the average upwind ground gradient is greater than 0.2 or where the effects of shielding are not applicable for a particular wind direction or are ignored.

Attention shall be given to possible combinations of tall buildings placed together, which lead to local and overall increases in wind actions.

TABLE 4.3

SHIELDING MULTIPLIER (M_s)

Shielding parameter (s)	Shielding multiplier (M_s)
≤1.5	0.7
3.0	0.8
6.0	0.9
≥12.0	1.0

NOTE: For intermediate values of s, use linear interpolation.

4.3.2 Buildings providing shielding

Only buildings within a 45° sector of radius $20h$ (symmetrically positioned about the directions being considered) and whose height is greater than or equal to z shall be deemed to provide shielding.

4.3.3 Shielding parameter (s)

The shielding parameter (s) in Table 4.3 shall be determined as follows:

$$s = \frac{l_s}{\sqrt{h_s b_s}} \qquad \ldots 4.3(1)$$

where

l_s = average spacing of shielding buildings, given by:

$$= h\left(\frac{10}{n_s} + 5\right) \qquad \ldots 4.3(2)$$

h_s = average roof height of shielding buildings

b_s = average breadth of shielding buildings, normal to the wind stream

h = average roof height, above ground, of the structure being shielded

n_s = number of upwind shielding buildings within a 45° sector of radius $20h$ and with $h_s \geq z$

4.4 TOPOGRAPHIC MULTIPLIER (M_t)

4.4.1 General

The topographic multiplier (M_t) shall be taken as follows:

(a) For sites in New Zealand and Tasmania over 500 m above sea level:

$$M_t = M_h M_{lee} (1 + 0.00015 E) \qquad \ldots 4.4(1)$$

where

M_h = hill shape multiplier

M_{lee} = lee (effect) multiplier (taken as 1.0, except in New Zealand lee zones, see Clause 4.4.3)

E = site elevation above mean sea level, in metres

(b) Elsewhere, the larger value of the following:

(i) $M_t = M_h$

(ii) $M_t = M_{lee}$

4.4.2 Hill-shape multiplier (M_h)

The hill shape multiplier (M_h) shall be assessed for each cardinal direction considered, taking into account the most adverse topographic cross-section that occurs within the range of directions within 22.5° on either side of the cardinal direction being considered. The value shall be as follows:

(a) For $H/(2L_u) < 0.05$, $M_h = 1.0$

(b) For $0.05 \le H/(2L_u) \le 0.45$ (see Figures 4.2 and 4.3):

$$M_h = 1 + \left(\frac{H}{3.5(z + L_1)}\right)\left(1 - \frac{|x|}{L_2}\right) \qquad \ldots 4.4(2)$$

(c) For $H/(2L_u) > 0.45$ (see Figure 4.4):

(i) Within the separation zone (see Figure 4.4)

$$M_h = 1 + 0.71\left[1 - \frac{|x|}{L_2}\right] \qquad \ldots 4.4(3)$$

(ii) Elsewhere within the local topographic zone (see Figures 4.2 and 4.3), M_h shall be as given in Equation 4.4(2)

where

H = height of the hill, ridge or escarpment

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

x = horizontal distance upwind or downwind of the structure to the crest of the hill, ridge or escarpment

L_1 = length scale, to determine the vertical variation of M_h, to be taken as the greater of 0.36 L_u or 0.4 H

L_2 = length scale, to determine the horizontal variation of M_h, to be taken as 4 L_1 upwind for all types, and downwind for hills and ridges, or 10 L_1 downwind for escarpments

z = reference height on the structure above the average local ground level

NOTE: Figures 4.2, 4.3 and 4.4 are cross-sections through the structure's site for a particular wind direction.

For the case where x and z are zero, the value of M_h is given in Table 4.4.

Irrespective of the provisions of this Clause, the influence of any peak may be ignored, provided it is distant from the site of the structure by more than 10 times its elevation above sea level.

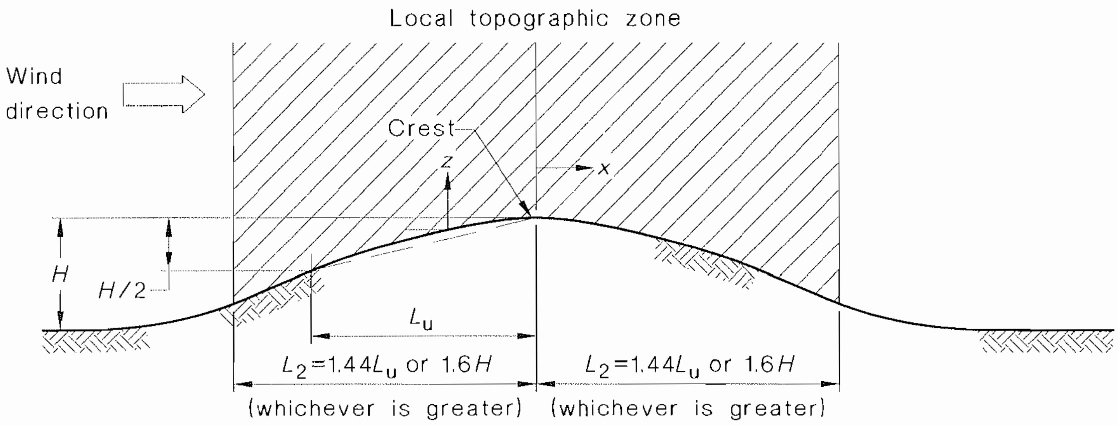
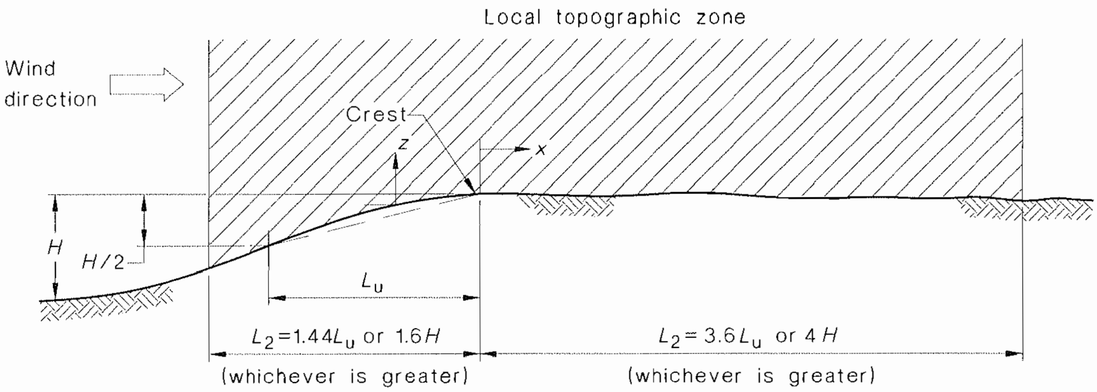


FIGURE 4.2 HILLS AND RIDGES



NOTE: For escarpments, the average downwind slope, measured from the crest to a distance of the greater of 3.6 L_u or 4 H shall not exceed 0.05.

FIGURE 4.3 ESCARPMENTS

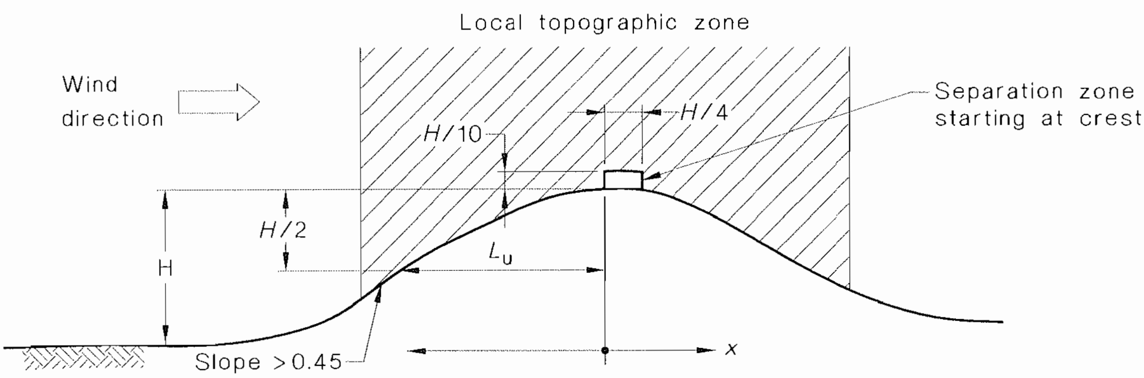


FIGURE 4.4 SEPARATION ZONE FOR HILLS AND ESCARPMENTS HAVING UPWIND SLOPES GREATER THAN 0.45

TABLE 4.4

HILL-SHAPE MULTIPLIER AT CREST $(|x| = 0)$, $z = 0$ (FOR GUST WIND SPEEDS)

Upwind slope ($H/2L_u$)	M_h
<0.05	1.0
0.05	1.08
0.10	1.16
0.20	1.32
0.30	1.48
≥0.45	1.71

4.4.3 Lee multiplier (M_{lee})

The lee (effect) multiplier (M_{lee}) shall be evaluated for New Zealand sites in the lee zones as shown in Figure 3.1(b). For all other sites, the lee multiplier shall be 1.0. Within the lee zones, the lee multiplier shall apply only to wind from the cardinal directions nominated in Figure 3.1(b).

Each lee zone shall be 30 km in width, measured from the leeward crest of the initiating range, downwind in the direction of the wind nominated. The lee zone comprises a 'shadow lee zone', which extends 12 km from the upwind boundary of the lee zone (crest of the initiating range), and an 'outer lee zone' over the remaining 18 km.

The lee multiplier shall be 1.35 for sites within the shadow lee zone (i.e., within 12 km of the crest of the range). Within the outer lee zone, the lee multiplier shall be determined by linear interpolation with horizontal distance, from the shadow/outer zone boundary (where $M_{lee} = 1.35$), to the downwind lee zone boundary (where $M_{lee} = 1.0$).

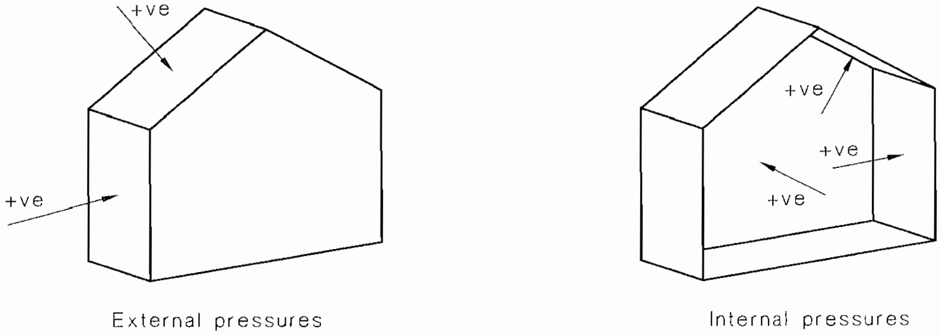
NOTE: No lee zones have been identified in Australia.

SECTION 5 AERODYNAMIC SHAPE FACTOR

5.1 GENERAL

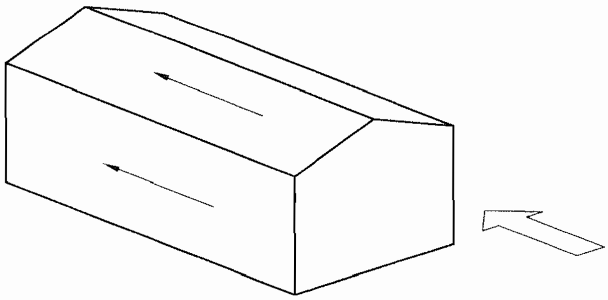
This Section shall be used to calculate the aerodynamic shape factor (C_{fig}) for structures or parts of structures. Values of C_{fig} shall be used in determining the pressures applied to each surface. For calculating pressures, the sign of C_{fig} indicates the direction of the pressure on the surface or element (see Figure 5.1), positive values indicating pressure acting towards the surface and negative values indicating pressure acting away from the surface (less than ambient pressure, i.e. suction). The wind action effects used for design shall be the sum of values determined for different pressure effects such as the combination of internal and external pressure on enclosed buildings.

Clauses 5.3, 5.4 and 5.5 provide values for enclosed rectangular buildings. For the purposes of this Standard, rectangular buildings include buildings generally made up of rectangular shapes in plan. Methods for other types of enclosed buildings, exposed members, lattice towers, free walls, free roofs and other structures are given in the appropriate Appendices, C to F.



NOTE: C_{fig} is used to give a pressure on one face of the surface under consideration. Positive value of C_{fig} indicates pressure acting towards the surface, negative acting away from the surface.

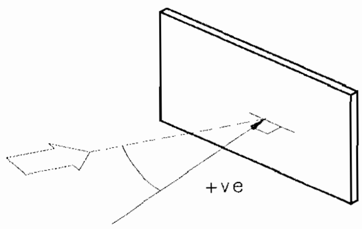
(a) Pressures normal to the surfaces of enclosed buildings



NOTE: C_{fig} is used to give a frictional drag on external surfaces of the structure only. Load per unit area acts parallel to the surface.

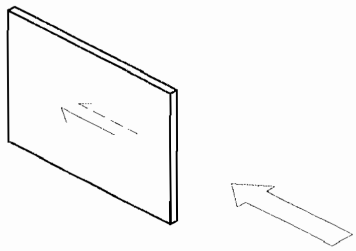
(b) Frictional drag on enclosed buildings

FIGURE 5.1 (in part) SIGN CONVENTIONS FOR C_{fig}



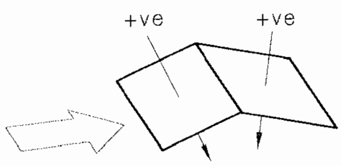
NOTE: C_{fig} is used to give a net pressure normal to the wall derived from face pressures on both upwind and downwind faces. The net pressure always acts normal to the longitudinal axis of the wall.

(c) Pressure normal to the surfaces of walls and hoardings



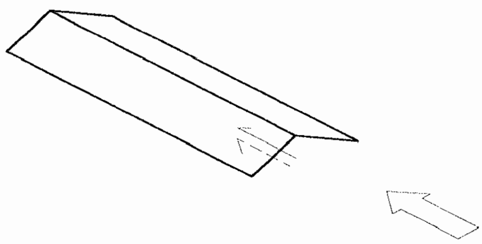
NOTE: C_{fig} is used to give frictional drag on both sides of the wall. Load per unit area acts parallel to both the surfaces of the wall.

(d) Frictional drag on walls and hoardings



NOTE: C_{fig} is used to give net pressure normal to the roof derived from face pressures on both upper and lower surfaces. The net pressure always acts normal to the surface and positive indicates downwards.

(e) Pressure normal to the surfaces freestanding roofs



NOTE: C_{fig} is used to give the total frictional drag forces derived from face frictional forces on both upper and lower surfaces. Load per unit area acts parallel to both the surfaces of the roof.

(f) Frictional drag on freestanding roofs

FIGURE 5.1 (in part) SIGN CONVENTIONS FOR C_{fig}

5.2 EVALUATION OF AERODYNAMIC SHAPE FACTOR

The aerodynamic shape factor (C_{fig}) shall be determined for specific surfaces or parts of surfaces as follows:

(a) *Enclosed buildings (see this Section 5 and Appendix C)*:

$C_{fig,i} = C_{p,i} K_{c,i}$, for internal pressures . . . 5.2(1)

$C_{fig,e} = C_{p,e} K_a K_{c,e} K_\ell K_p$, for external pressures . . . 5.2(2)

$C_{fig} = C_f K_a K_c$, for frictional drag forces . . . 5.2(3)

(b) *Circular bins, silos and tanks*—see Appendix C.

(c) *Freestanding walls, hoardings, canopies and roofs (see Appendix D)*:

$C_{fig} = C_{p,n} K_a K_\ell K_p$, for pressure normal to surface . . . 5.2(4)

$C_{fig} = C_f$, for frictional drag forces . . . 5.2(5)

(d) *Exposed structural members, frames and lattice towers*—see Appendix E.

(e) *Flags and circular shapes*—see Appendix F.

where

$C_{p,e}$ = external pressure coefficient

$C_{p,i}$ = internal pressure coefficient

C_f = frictional drag force coefficient

$C_{p,n}$ = net pressure coefficient acting normal to the surface for canopies, freestanding roofs, walls, and the like

K_a = area reduction factor

K_c = combination factor

$K_{c,e}$ = combination factor applied to external pressures

$K_{c,i}$ = combination factor applied to internal pressures

K_ℓ = local pressure factor

K_p = porous cladding reduction factor

5.3 INTERNAL PRESSURE FOR ENCLOSED RECTANGULAR BUILDINGS

5.3.1 General

Aerodynamic shape factors for internal pressure ($C_{p,i}$) shall be determined from Tables 5.1(A) and 5.1(B). Table 5.1(A) shall be used for the design case where potential openings are shut and the wall permeability dominates. Table 5.1(B) shall be used for the design case where openings are assumed to be open. In all cases, the height at which the wind speed is determined shall be the average roof height (h), as defined in Figure 2.1.

Internal pressure is a function of the relative permeability of the external surfaces of the building. The permeability of a surface shall be calculated by adding areas of opening to leakage on that surface of the building (e.g. vents, gaps in windows).

5.3.2 Openings

Combinations of openings shall be assumed to give internal pressures, which together with external pressures give the most adverse wind actions. Potential openings include doors, windows and vents. Closed doors (including roller doors) and windows shall be considered to be openings unless they are capable of resisting the applied wind pressures in all regions (and impact loading from wind-borne debris in Regions C and D). This structural assessment shall include elements such as supports, frames, jambs, roller door guides, windlocks and fixings where the resistance of roller doors relies on those. This assessment shall account for any catenary actions developed and relied upon in the structure.

In Regions C and D, internal pressure resulting from the dominant opening shall be applied, unless the building envelope (windows, doors and cladding at heights up to 25 m) can be shown to be capable of resisting impact loading from windborne debris determined in accordance with Clause 2.5.7.

NOTE: Garage doors designed for Regions C and D according to AS 4505—1998 can be taken as remaining closed and intact under wind forces, and hence need not be treated as a dominant opening; however garage doors in Regions A and B, should be able to resist wind loads according to AS/NZS1170.2, otherwise a dominant opening should be assumed.

5.3.3 Dominant openings

A surface is considered to contain dominant openings if the sum of the areas of all openings in that surface exceeds the sum of the areas of the openings in each of the other surfaces considered one at a time.

NOTE: A dominant opening does not need to be large and can occur as a result of a particular proposed scenario, such as an open air vent, while all other potential openings are shut.

5.3.4 Internal walls and ceilings

Internal walls that provide an effective seal between spaces within buildings shall be considered as being subjected to differential pressures derived from the internal pressure assessed for that space, determined in accordance with Clause 5.3.1 and Tables 5.1(A) and 5.1(B), with the worst combination pressure coefficient of ±0.2 applied to the other side.

The determination of pressures within a space shall account for known and likely openings derived in accordance with Clause 5.3.2. In Regions C and D, likely openings shall include failures of the building envelope unless specific debris impact resistance measures are employed in accordance with Clauses 2.5.7 and 5.3.2.

NOTES:

1 Ceilings may also be subjected to significant wind-induced pressures, depending on factors such as the roof permeability, proximity to rooms with potential dominant openings, and the location of manholes.

2 In those cases where internal walls and ceilings do not form a permanent seal, then differential pressures derived using a net pressure coefficient of ±0.3 may be appropriate.

3 Differential pressures on internal walls and ceilings may be relieved by provision of appropriate venting.

TABLE 5.1(A)

INTERNAL PRESSURE COEFFICIENTS ($C_{p,i}$) FOR BUILDINGS WITH OPEN INTERIOR PLAN—CASES FOR PERMEABLE WALLS WITHOUT DOMINANT OPENINGS

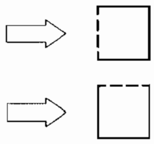
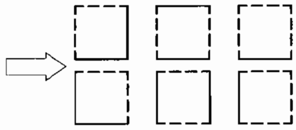
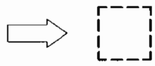
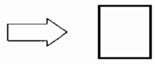
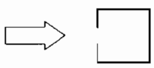
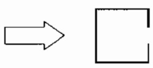
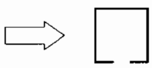
Condition	$C_{p,i}$	Examples showing openings, permeability and wind direction
One wall permeable, other walls impermeable: (a) Windward wall permeable (b) Windward wall impermeable	 0.6 −0.3	
Two or three walls equally permeable, other walls impermeable: (a) Windward wall permeable (b) Windward wall impermeable	 −0.1, 0.2 −0.3	
All walls equally permeable	−0.3 or 0.0, whichever is the more severe for combined forces	
A building effectively sealed and having non-opening windows	−0.2 or 0.0, whichever is the more severe for combined forces	

TABLE 5.1(B)

INTERNAL PRESSURE COEFFICIENTS ($C_{p,i}$) FOR BUILDINGS WITH OPEN INTERIOR PLAN—DOMINANT OPENINGS ON ONE SURFACE

Ratio of dominant opening to total open area (including permeability) of other wall and roof surfaces	Dominant opening on windward wall	Dominant opening on leeward wall	Dominant opening on side wall	Dominant opening on roof
0.5 or less	−0.3, 0.0	−0.3, 0.0	−0.3, 0.0	−0.3, 0.0
1	−0.1, 0.2	−0.3, 0.0	−0.3, 0.0	−0.3, 0.0
2	$0.7C_{p,e}$	$C_{p,e}$	$C_{p,e}$	$C_{p,e}$
3	$0.85C_{p,e}$	$C_{p,e}$	$C_{p,e}$	$C_{p,e}$
6 or more	$C_{p,e}$	$C_{p,e}$	$C_{p,e}$	$C_{p,e}$
				

NOTE: $C_{p,e}$ is the relevant external pressure coefficient at the location of the dominant opening.

5.4 EXTERNAL PRESSURES FOR ENCLOSED RECTANGULAR BUILDINGS

5.4.1 External pressure coefficients ($C_{p,e}$)

The external pressure coefficients ($C_{p,e}$) for surfaces of rectangular enclosed buildings shall be as given in Tables 5.2(A), 5.2(B) and 5.2(C) for walls and 5.3(A), 5.3(B) and 5.3(C) for roofs and for some special roofs Appendix C. The parameters (e.g. dimensions) referred to in these Tables are set out in Figure 5.2.

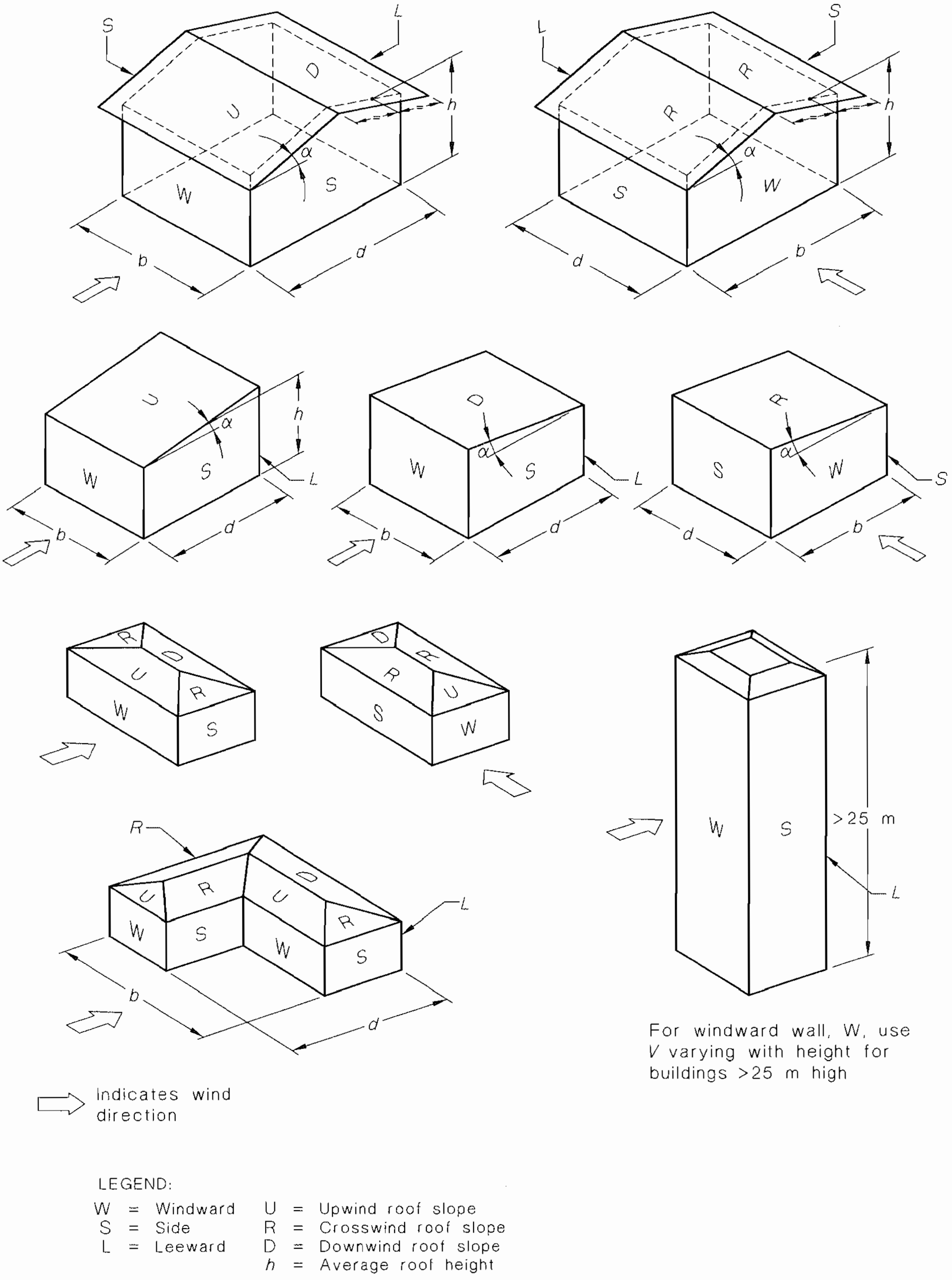


FIGURE 5.2 PARAMETERS FOR RECTANGULAR ENCLOSED BUILDINGS

For leeward walls, side walls and roofs, wind speed shall be taken as the value at $z = h$. The reference height (h) shall be taken as the average height of the roof.

Where two values of $C_{p,e}$ are listed, roofs shall be designed for both values. In these cases, roof surfaces may be subjected to either value due to turbulence. Alternative combinations of external and internal pressures (see also Clause 5.3) shall be considered, to obtain the most severe conditions for design.

For roofs, the following alternative load cases shall be considered:

(a) When using Table 5.3(A), for the appropriate roof type, slope and edge distance—

(i) apply the more negative value of $C_{p,e}$ to all pressure zones and surfaces; and

(ii) apply the less negative (or most positive) value of $C_{p,e}$ to all pressure zones and surfaces.

(b) When using both Tables 5.3(B) and 5.3(C), and for the appropriate parameters—

(i) apply the more negative value of $C_{p,e}$ from Table 5.3(B) to the upwind slope together with the value from Table 5.3(C) to the downwind slope; and

(ii) apply the less negative (or positive) value of $C_{p,e}$ from Table 5.3(B) to the upwind slope together with the value from Table 5.3(C) to the downwind slope.

(c) When using Table 5.3(C) only, for steeper crosswind slopes on hip roofs, apply the appropriate $C_{p,e}$ value to both slopes.

For the underside of elevated buildings, $C_{p,e}$ shall be taken as 0.8 and –0.6. For buildings with less elevation above ground than one-third of the height, use linear interpolation between these values and 0.0, according to the ratio of clear unwalled height underneath first floor level to the total building height. For the calculation of underside external pressures, wind speed shall be taken as the value at h for all z.

Under-eaves pressures shall be taken as equal to those applied to the adjacent wall surface below the surface under consideration.

TABLE 5.2(A)

WALLS—EXTERNAL PRESSURE COEFFICENTS ($C_{p,e}$) FOR RECTANGULAR ENCLOSED BUILDINGS—WINDWARD WALL (W)

h	External pressure coefficients ($C_{p,e}$)
>25.0 m	0.8 (wind speed varies with height)
≤25.0 m	For buildings on ground— 0.8, when wind speed varies with height; or 0.7, when wind speed is taken for $z = h$
	For elevated buildings— 0.8 (wind speed taken at h)

TABLE 5.2(B)

WALLS—EXTERNAL PRESSURE COEFFICIENTS ($C_{p,e}$) FOR RECTANGULAR ENCLOSED BUILDINGS—LEEWARD WALL (L)

Wind direction θ degrees (see Figure 2.2)	Roof shape	Roof pitch (α), degrees (see Note 1)	d/b (see Note 1)	External pressure coefficients ($C_{p,e}$)
0	Hip or gable	<10	≤1 2 ≤4	−0.5 −0.3 −0.2
0 0 0	Hip or gable Hip or gable Hip or gable	10 15 20	All values	−0.3 −0.3 −0.4
0	Hip or gable	≥25	≤0.1 ≥0.3	−0.75 −0.5
90	Gable (see Note 2)	All values	≤1 2 ≤4	−0.5 −0.3 −0.2

NOTES:

1 For intermediate values of d/b and α, linear interpolation shall be used.

2 For hip roofs use the same values as for $\theta = 0°$.

TABLE 5.2(C)

WALLS—EXTERNAL PRESSURE COEFFICENTS ($C_{p,e}$) FOR RECTANGULAR ENCLOSED BUILDINGS—SIDE WALLS (S)

Horizontal distance from windward edge	External pressure coefficients ($C_{p,e}$)
0 to 1h	−0.65
1h to 2h	−0.5
2h to 3h	−0.3
> 3h	−0.2

TABLE 5.3(A)

ROOFS—EXTERNAL PRESSURE COEFFICIENTS ($C_{p,e}$) FOR RECTANGULAR ENCLOSED BUILDINGS—FOR UPWIND SLOPE (U), AND DOWNWIND SLOPE (D) AND (R) FOR GABLE ROOFS, FOR α < 10°

Roof type and slope		Horizontal distance from windward edge of roof	External pressure coefficient ($C_{p,e}$)	
Crosswind slopes for gable roofs, (R)	Upwind slope, (U), Downwind slope, (D)		$h/d \leq 0.5$ (see Note 1)	$h/d \geq 1.0$ (see Note 1)
All α	α < 10°	0 to 0.5h 0.5 to 1h 1h to 2h 2h to 3h >3h	−0.9, −0.4 −0.9, −0.4 −0.5, 0 −0.3, 0.1 −0.2, 0.2	−1.3, −0.6 −0.7, −0.3 (−0.7), (−0.3) see Note 2

NOTES:

1 For intermediate values of roof slopes and h/d ratios, linear interpolation shall be used. Interpolation shall only be carried out on values of the same sign.

2 The values given in parentheses are provided for interpolation purposes.

TABLE 5.3(B)

ROOFS—EXTERNAL PRESSURE COEFFICIENTS ($C_{p,e}$) FOR RECTANGULAR ENCLOSED BUILDINGS—UPWIND SLOPE (U) $\alpha \geq 10°$

Upwind slope, (U)	Ratio h/d (see Note)	External pressure coefficients ($C_{p,e}$)						
		Roof pitch (α) degrees (see Note)						
		10	15	20	25	30	35	≥ 45
	≤ 0.25	−0.7, −0.3	−0.5, 0.0	−0.3, 0.2	−0.2, 0.3	−0.2, 0.4	0.0, 0.5	
$\alpha \geq 10°$	0.5	−0.9, −0.4	−0.7, −0.3	−0.4, 0.0	−0.3, 0.2	−0.2, 0.3	−0.2, 0.4	0, 0.8 sin α
	≥ 1.0	−1.3, −0.6	−1.0, −0.5	−0.7, −0.3	−0.5, 0.0	−0.3, 0.2	−0.2, 0.3	

NOTE: For intermediate values of roof slopes and h/d ratios, linear interpolation shall be used. Interpolation shall only be carried out on values of the same sign.

TABLE 5.3(C)

ROOFS—EXTERNAL PRESSURE COEFFICIENTS ($C_{p,e}$) FOR RECTANGULAR ENCLOSED BUILDINGS—DOWNWIND SLOPE (D), AND (R) FOR HIP ROOFS, FOR $\alpha \geq 10°$

Roof type and slope		Ratio h/d (see Note)	External pressure coefficients ($C_{p,e}$)			
Crosswind slope for hip roofs (R)	Downwind slope (D)		Roof pitch (α), degrees (see Note)			
			10	15	20	≥25
		≤ 0.25	−0.3	−0.5	−0.6	For $b/d < 3$; −0.6
$\alpha \geq 10°$	$\alpha \geq 10°$	0.5	−0.5	−0.5	−0.6	For $3 < b/d < 8$; $-0.06\,(7 + b/d)$
		≥ 1.0	−0.7	−0.6	−0.6	For $b/d > 8$; −0.9

NOTE: For intermediate values of roof slopes and h/d ratios, linear interpolation shall be used. Interpolation shall only be carried out on values of the same sign.

5.4.2 Area reduction factor (K_a) for roofs and side walls

For roofs and sidewalls, the area reduction factor (K_a) shall be as given in Table 5.4. For all other cases, K_a shall be taken as 1.0. Tributary area (A) is the area contributing to the force being considered.

TABLE 5.4

AREA REDUCTION FACTOR (K_a)

Tributary area (A), m^2 (see Note)	Area reduction factor (K_a)
≤ 10	1.0
25	0.9
≥ 100	0.8

NOTE: For intermediate values of A, linear interpolation shall be used.

5.4.3 Action combination factor (K_c)

Where wind pressures acting on a combination of surfaces of an enclosed building (e.g. windward wall, roof, side wall, leeward wall, internal surface) contribute simultaneously to a structural action effect (e.g. member axial force or bending moment) on a structural element, combination factors ($K_{c,e}$ and $K_{c,i}$), less than 1.0, may be applied to the external and internal surfaces when calculating the combined forces.

A surface shall be either a windward wall, a side wall, a leeward wall, a roof (the upwind and downwind roof shall be treated together as a single surface), or the internal surfaces of the building treated as a single surface. An internal surface shall not be treated as an effective surface if $|C_{pi}| < 0.2$.

Where pressures on two contributing surfaces act together in combination to produce a structural action effect, $K_{c,e}$ and $K_{c,i}$ may be taken as 0.9. Where three (or more) contributing surfaces act in combination, $K_{c,e}$ and $K_{c,i}$ may be taken as 0.8.

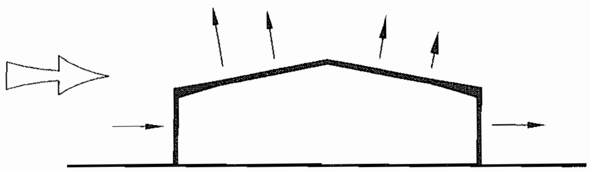
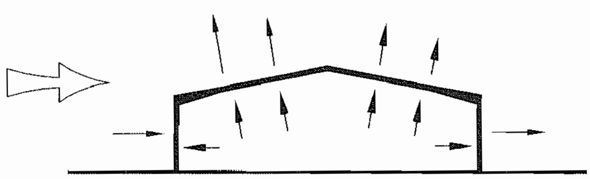
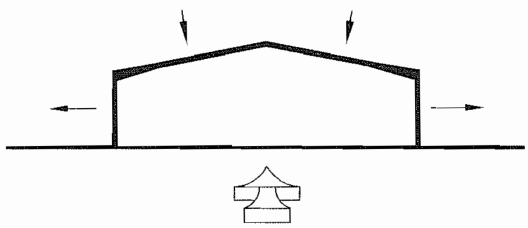
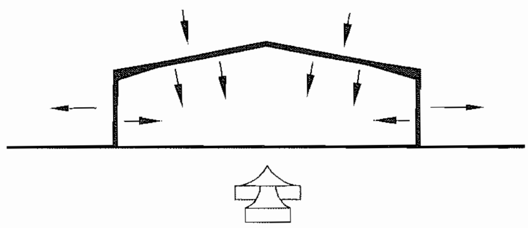
Examples of appropriate combination factors ($K_{c,e}$ and $K_{c,i}$) are given in Table 5.5.

For any roofs and side walls, the product K_a. $K_{c,e}$ shall not be less than 0.8.

NOTE: Action combination factors less than 1.0 account for the non-simultaneous action of peak pressures on effective surfaces.

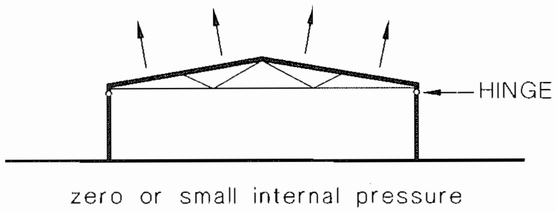
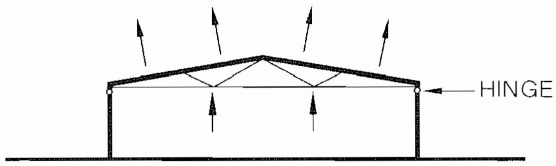
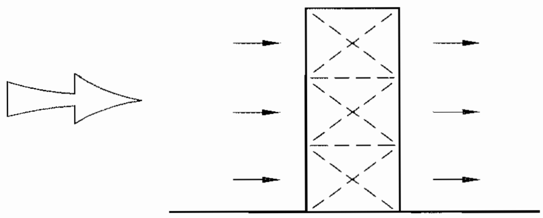
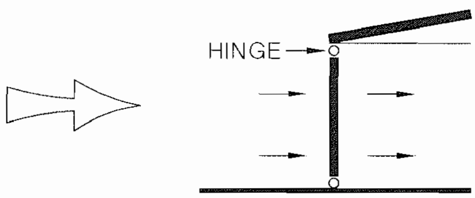
TABLE 5.5

EXAMPLES OF ACTION COMBINATION FACTORS $K_{c,e}$ AND $K_{c,i}$ FOR ACTION EFFECTS ON STRUCTURAL ELEMENTS FROM WIND PRESSURE ON EFFECTIVE SURFACES

Design case	Example diagram	External $K_{c,e}$	Internal $K_{c,i}$
(a) *3 effective surfaces* Pressures from windward and leeward walls in combination with roof pressures	 zero or small internal pressure	0.8	1.0 (not an effective surface)
(b) *4 effective surfaces* Pressures from windward and leeward walls in combination with roof pressures and internal pressures		0.8	0.8
(c) *3 effective surfaces* Pressures from side walls in combination with roof pressures	 zero or small internal pressure	0.8	1.0 (not an effective surface)
(d) *4 effective surfaces* Pressures from side walls in combination with roof pressures and internal pressures		0.8	0.8

(continued)

TABLE 5.5 (*continued*)

Design case	Example diagram	External $K_{c,e}$	Internal $K_{c,i}$
(e) *1 effective surface* Roof pressures acting alone		1.0	1.0 (not an effective surface)
(f) *2 effective surfaces* Roof pressures in combination with internal pressures		0.9	0.9
(g) *2 effective surfaces* Lateral pressure on windward and leeward walls		0.9	1.0 (not an effective surface)
(h) *2 effective surfaces* Lateral pressure on external and internal surfaces		0.9	0.9

5.4.4 Local pressure factor (K_ℓ) for cladding

The local pressure factor (K_ℓ) shall be taken as 1.0 in all cases except when determining the wind forces applied to cladding, their fixings, the members that directly support the cladding, and the immediate fixings of these members. In these cases K_ℓ shall be taken either as 1.0 or the value from Table 5.6 for the area and locations indicated, whichever gives the most adverse effect when combined with the external and internal pressures. Where more than one case applies, the largest value of K_ℓ from Table 5.6 shall be used.

Where the cladding or the supporting member extends beyond the zone a given in Table 5.6, a value of $K_\ell = 1.0$ shall apply to wind force contributions imposed from beyond that zone.

The value of dimension a is the minimum of $0.2b$ or $0.2d$ or the height (h) as shown in Figure 5.3.

Where interaction is possible, external pressures shall be taken to act simultaneously with internal pressures given in Clause 5.3 and with the under-eaves pressures given in Clause 5.4.1, and the resultant forces shall be added. Design cases for negative pressures in Table 5.6 are alternative cases and shall not be applied simultaneously.

For rectangular buildings, the negative limit on the product $K_\ell \, C_{p,e}$ shall be −3.0 in all cases. The RC1 case only applies to flat or near-flat roofs (slope less than 10°).

For flat or near-flat roofs (slope less than 10°) with parapets, values of K_ℓ for areas RA1 and RA2 in the lee of the parapet may be modified by multiplying the values from Table 5.6 by the parapet reduction factor (K_r), given in Table 5.7.

TABLE 5.6
LOCAL PRESSURE FACTOR (K_ℓ)

Design case	Figure 5.3 reference number	Building aspect ratio (r)	Area (A) m^2	Proximity to edge	K_ℓ
Positive pressures					
Windward wall	WA1	All	$A \leq 0.25a^2$	Anywhere	1.5
All other areas	—	All	—	—	1.0
Negative pressures					
Upwind corners of roofs with pitch <10°	RC1	All	$A \leq 0.25a^2$	$<a$ from two edges	3.0
Upwind roof edges	RA1 RA2	All All	$A \leq a^2$ $A \leq 0.25a^2$	$< a$ $< 0.5a$	1.5 2.0
Downwind side of hips and ridges of roofs with pitch ≥10°	RA3 RA4	All All	$A \leq a^2$ $A \leq 0.25a^2$	$< a$ $< 0.5a$	1.5 2.0
Side walls near windward wall edges	SA1 SA2	≤ 1	$A \leq a^2$ $A \leq 0.25a^2$	$< a$ $< 0.5a$	1.5 2.0
	SA3 SA4 SA5	>1	$A \leq 0.25a^2$ $A \leq a^2$ $A \leq 0.25a^2$	$>a$ $<a$ $\leq 0.5a$	1.5 2.0 3.0
All other areas	—	All	—	—	1.0

NOTES:

1 Figure reference numbers and dimension a are defined in Figure 5.3.

2 If an area of cladding is covered by more than one case in Table 5.6, use the largest value of K_ℓ obtained for any case.

3 The building aspect ratio (r) is defined as the average roof height (h) divided by the smaller of b or d.

TABLE 5.7
REDUCTION FACTOR (K_r) DUE TO PARAPETS

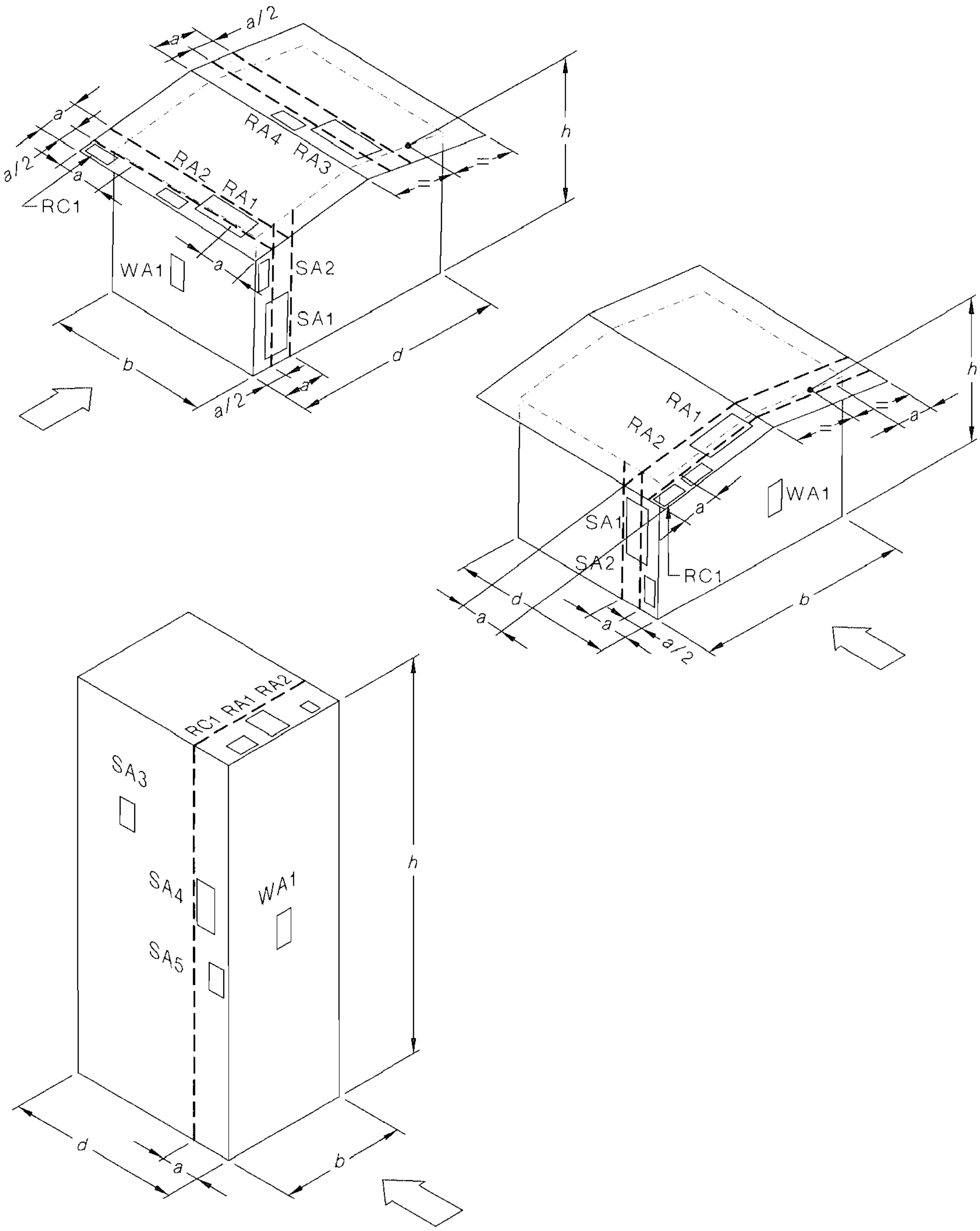
h	h_p (see Note)	K_r
≤ 25 m	$\leq 0.07\,h$ $0.1\,h$ $\geq 0.2\,h$	1.0 0.8 0.5
> 25 m	$\leq 0.02\,w$ $0.03\,w$ $\geq 0.05\,w$	1.0 0.8 0.5

LEGEND:

h_p = height of parapet above average roof level.

w = shortest horizontal dimension of the building.

NOTE: For intermediate values, linear interpolation shall be used.



NOTES:

1 The value of dimension a is the minimum of $0.2b$, $0.2d$ and h.

2 The side ratio of any local pressure factor area on the roof shall not exceed 4.

FIGURE 5.3 LOCAL PRESSURE FACTORS (K_l)

5.4.5 Permeable cladding reduction factor (K_p) for roofs and side walls

The permeable cladding reduction factor (K_p) shall be taken as 1.0 except that where an external surface consists of permeable cladding and the solidity ratio is less than 0.999 and exceeds 0.99, the values given in Table 5.8 may be used for negative pressure. The solidity ratio of the surface is the ratio of solid area to total area of the surface. Figure 5.4 shows dimension d_a.

TABLE 5.8

PERMEABLE CLADDING REDUCTION FACTOR (K_p)

Horizontal distance from windward edge (see Note)	K_p
0 to $0.2d_a$	0.9
$0.2d_a$ to $0.4d_a$	0.8
$0.4d_a$ to $0.8d_a$	0.7
$0.8d_a$ to $1.0d_a$	0.8

NOTE: d_a is the along-wind depth of the surface, in metres.

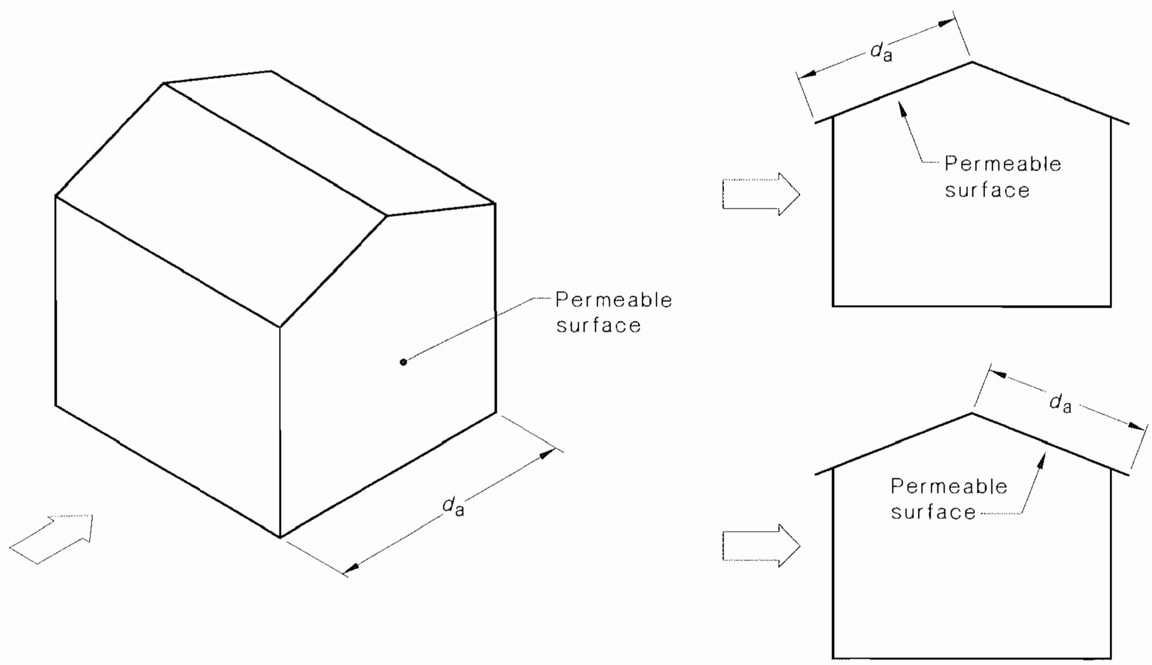


FIGURE 5.4 NOTATION FOR PERMEABLE SURFACES

5.5 FRICTIONAL DRAG FORCES FOR ENCLOSED BUILDINGS

The frictional drag (f) shall be calculated for roofs and side walls of enclosed buildings, in addition to pressures normal to the surface, only where the ratio d/h or d/b is greater than 4. The aerodynamic shape factor (C_{fig}) equals the frictional drag coefficient (C_f) in the direction of the wind as given in Table 5.9.

The effect shall be calculated on the basis of areas as follows:

(a) For $h \leq b$, area = $(b+2h)(d-4h)$.

(b) For $h > b$, area = $(b+2h)(d-4b)$.

TABLE 5.9

FRICTIONAL DRAG COEFFICIENT (C_f) FOR d/h>4 or d/b>4

Distance 'x' from windward edge	Surface description	C_f
$x \geq$ the lesser of $4h$ and $4b$	Surfaces with ribs across the wind direction	0.04
	Surfaces with corrugations across the wind direction	0.02
	Smooth surfaces without corrugations or ribs or with corrugations or ribs parallel to the wind direction	0.01
$x <$ the lesser of $4h$ and $4b$	All surfaces	0

SECTION 6 DYNAMIC RESPONSE FACTOR

6.1 EVALUATION OF DYNAMIC RESPONSE FACTOR

The dynamic response factor (C_{dyn}) shall be determined for structures or elements of structures with natural first-mode fundamental frequencies as follows:

(a) Greater than 1 Hz, $C_{dyn} = 1.0$.

(b) Less than 1 Hz—

  (i) for tall buildings and freestanding towers—

    (A) less than 0.2 Hz is not covered by this Standard;

    (B) between 1 Hz and 0.2 Hz, C_{dyn} shall be as defined in Clause 6.2 for along-wind response and Clause 6.3 for crosswind response;

    (C) where the frequencies of vibration for the two fundamental modes of sway are within 10% of each other and are both less than 0.4 Hz, this is not covered by this Standard;

  (ii) for cantilever roofs—

    (A) less than 0.5 Hz is not covered by this Standard;

    (B) between 1 Hz and 0.5 Hz, C_{dyn} shall be as defined in Paragraph D5, Appendix D.

NOTES:

1 Appendix G provides information on calculating accelerations for serviceability in tall wind-sensitive structures.

2 For natural frequencies less than 0.2 Hz, heights greater than 200 m, or whenever significant coupling is evident in the first three modes of vibration, wind-tunnel testing should be undertaken.

3 Dynamic response factors for roofs supported on two or more sides with natural frequencies less than 1 Hz are not provided in this Standard. Special studies such as wind-tunnel testing should be undertaken.

6.2 ALONG-WIND RESPONSE OF TALL BUILDINGS AND FREESTANDING TOWERS

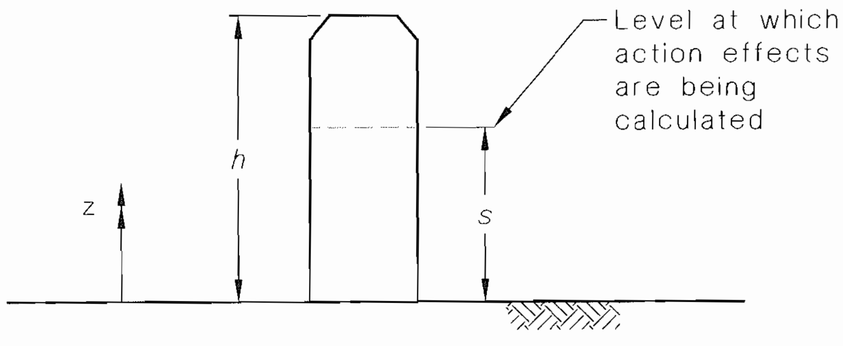
6.2.1 General

The dynamic response factor shall be as given in Clause 6.2.2.

NOTE: Information on peak along-wind acceleration for serviceability is given in Appendix G.

6.2.2 Dynamic response factor (C_{dyn})

For calculation of action effects (bending moments, shear forces, member forces) at a height s on the structure (see Figure 6.1), the wind pressures on the structure at a height z shall be multiplied by a dynamic response factor (C_{dyn}). This factor is dependent on both z and s and $s < z < h$. For the calculation of base bending moments, deflections and acceleration at the top of the structure, a single value of C_{dyn} shall be used with s taken as zero. For the calculation of C_{dyn}, the value of $V_{des,\theta}$ is calculated at the reference height (h).



$s < z < h$

FIGURE 6.1 NOTATION FOR HEIGHTS

The dynamic response factor (C_{dyn}) shall be calculated as follows:

$$C_{dyn} = \frac{1 + 2I_h \sqrt{g_v^2 B_S + \dfrac{H_s g_R^2 S E_t}{\zeta}}}{(1 + 2g_v I_h)} \quad \ldots 6.2(1)$$

where

s = height of the level at which action effects are calculated for a structure

h = average roof height of a structure above the ground, or height to the top of a tower

I_h = turbulence intensity, obtained from Table 6.1 by setting $z = h$

g_v = peak factor for the upwind velocity fluctuations, which shall be taken as 3.7

B_s = background factor, which is a measure of the slowly varying background component of the fluctuating response, caused by low-frequency wind speed variations, given as follows:

$$B_S = \frac{1}{1 + \dfrac{\sqrt{0.26(h-s)^2 + 0.46 b_{sh}^2}}{L_h}} \quad \ldots 6.2(2)$$

where b_{sh} is the average breadth of the structure between heights s and h

L_h = a measure of the integral turbulence length scale at height h in metres

= $85(h/10)^{0.25}$ ... 6.2(3)

H_s = height factor for the resonant response which equals $1 + (s/h)^2$

g_R = peak factor for resonant response (10 min period) given by:

= $\sqrt{2\log_e(600 n_a)}$ ... 6.2(4)

S = size reduction factor given as follows, where n_a is first mode natural frequency of vibration of a structure in the along-wind direction in hertz and b_{0h} is the average breadth of the structure between heights 0 and h:

$$= \frac{1}{\left[1 + \dfrac{3.5 n_a h (1 + g_v I_h)}{V_{des,\theta}}\right]\left[1 + \dfrac{4 n_a b_{0h}(1 + g_v I_h)}{V_{des,\theta}}\right]} \quad \ldots 6.2(5)$$

E_t = $(\pi/4)$ times the spectrum of turbulence in the approaching wind stream, given as follows:

$$= \frac{\pi N}{\left(1+70.8N^2\right)^{5/6}} \qquad \ldots 6.2(6)$$

where

N = reduced frequency (non dimensional)

= $n_a L_h [1 + (g_v I_h)] / V_{des,\theta}$

n_a = first mode natural frequency of vibration of a structure in the along-wind direction in hertz

$V_{des,\theta}$ = building design wind speed determined at the building height, h (see Clause 2.3)

ζ = ratio of structural damping to critical damping of a structure

NOTES:

1 For structural damping for *ultimate* limit states, recommended *maximum* values of ζ are as follows:

Steel structures: 0.02 of critical.

Reinforced-concrete structures: 0.03 of critical.

2 For structural damping for *serviceability* limit states, recommended *maximum* values of ζ are as follows:

Steel structures: 0.012 of critical for deflection calculations; 0.01 of critical for calculation of accelerations at the top of tall buildings and towers.

Reinforced-concrete structures: 0.015 of critical for deflection calculations; 0.01 of critical for calculation of accelerations at the top of tall buildings and towers.

3 Users should seek other sources for advice on possible values of structural damping as a function of the type of construction, building dimensions and amplitude of vibration.

TABLE 6.1
TURBULENCE INTENSITY (I_z)

Height (z) m	Limit state, region and terrain category			
	Serviceability limit states			
	Terrain category 1, all regions	Terrain category 2, all regions	Terrain category 3, all regions	Terrain category 4, all regions
	Ultimate limit states			
	Terrain category 1, Regions A, W and B	Terrain category 2, Regions A, W and B	Terrain category 3, Regions A, W and B Terrain categories 1, 2, and 3: Regions C and D	Terrain category 4, all regions
≤3	0.171	0.207	0.271	0.342
5	0.165	0.196	0.271	0.342
10	0.157	0.183	0.239	0.342
15	0.152	0.176	0.225	0.342
20	0.147	0.171	0.215	0.342
30	0.140	0.162	0.203	0.305
40	0.133	0.156	0.195	0.285
50	0.128	0.151	0.188	0.270
75	0.118	0.140	0.176	0.248
100	0.108	0.131	0.166	0.233
150	0.095	0.117	0.150	0.210
200	0.085	0.107	0.139	0.196

NOTE: For intermediate values of height (z) and terrain category, linear interpolation shall be used.

6.3 CROSSWIND RESPONSE

6.3.1 General

Clause 6.3.2 gives methods for determining equivalent static forces and base overturning moments and $C_{fig}\, C_{dyn}$ for tall enclosed buildings and towers of rectangular cross-section, and Clause 6.3.3 gives deflections and equivalent static forces for chimneys, masts and poles of circular cross-section. Calculation of crosswind response is not required for porous lattice towers.

NOTES:

1 Information on peak crosswind acceleration for serviceability is given in Appendix G.

2 UHF antennas of the cross-sections shown in Figure E3, Appendix E, may have significant potential for crosswind response.

6.3.2 Crosswind response of tall enclosed buildings and towers of rectangular cross-section

6.3.2.1 *Equivalent static crosswind force*

The equivalent static crosswind force per unit height (w_{eq}) as a function of z (evaluated using force equals mass times acceleration) in newtons per metre shall be as follows:

$$w_{eq}(z) = 0.5\rho_{air}\left[V_{des,\theta}\right]^2 dC_{fig}C_{dyn} \qquad \ldots 6.3(1)$$

where $V_{des,\theta}$ is evaluated at $z = h$, and d is the horizontal depth of the structure parallel to the wind stream and

$$\left(C_{\mathrm{fig}} C_{\mathrm{dyn}}\right)=1.5 g_{\mathrm{R}}\left(\frac{b}{d}\right) \frac{K_{\mathrm{m}}}{\left(1+g_{\mathrm{v}} I_{\mathrm{h}}\right)^{2}}\left(\frac{z}{h}\right)^{k} \sqrt{\frac{\pi C_{\mathrm{fs}}}{\zeta}} \quad \text{. . . 6.3(2)}$$

where

K_{m} = mode shape correction factor for crosswind acceleration, given by:

= $0.76 + 0.24k$

where

k = mode shape power exponent for the fundamental mode. Values of the exponent k should be taken as:

= 1.5 for a uniform cantilever

= 0.5 for a slender framed structure (moment resisting)

= 1.0 for a building with central core and moment-resisting facade

= 2.3 for a tower decreasing in stiffness with height, or with a large mass at the top

= value obtained from fitting $\phi_1(z) = (z/h)^k$ to the computed modal shape of the structure

$\phi_1(z)$ = first mode shape as a function of height z, normalized to unity at $z = h$

C_{fs} = crosswind force spectrum coefficient generalized for a linear mode shape given in Clause 6.3.2.3

6.3.2.2 *Crosswind base overturning moment*

The crosswind base overturning moment (M_{c}), (which can be derived by the integration from 0 to h of $w_{\mathrm{eq}}(z)\, z\, \mathrm{d}z$) shall be as follows:

$$M_{\mathrm{c}}=0.5 g_{\mathrm{R}} b\left[\frac{0.5 \rho_{\mathrm{air}}\left[V_{\mathrm{des}, \theta}\right]^{2}}{\left(1+g_{\mathrm{v}} I_{\mathrm{h}}\right)^{2}}\right] h^{2}\left(\frac{3}{k+2}\right) K_{\mathrm{m}} \sqrt{\frac{\pi C_{\mathrm{fs}}}{\zeta}} \quad \text{. . . 6.3(3)}$$

where the value $\left(\frac{3}{k+2}\right)K_{\mathrm{m}}$ is the mode shape correction factor for crosswind base overturning moment.

6.3.2.3 *Crosswind force spectrum coefficient (C_{fs})*

The reduced velocity (V_{n}) shall be calculated as follows using $V_{\mathrm{des},\theta}$ calculated at $z = h$, as follows:

$$V_{\mathrm{n}}=\frac{V_{\mathrm{des}, \theta}}{n_{\mathrm{c}} b\left(1+g_{\mathrm{v}} I_{\mathrm{h}}\right)} \quad \text{. . . 6.3(4)}$$

Values of the crosswind force spectrum coefficient generalized for a linear mode shape (C_{fs}) shall be calculated from the reduced velocity (V_{n}) as follows (see Figures 6.2 to 6.5):

(a) For a 3:1:1 square section (h:b:d), where V_{n} is in the range 2 to 16:

(i) For turbulence intensity of 0.12 at $2h/3$:

$$\log_{10} C_{\mathrm{fs}}=0.000353 V_{\mathrm{n}}^{4}-0.0134 V_{\mathrm{n}}^{3}+0.15 V_{\mathrm{n}}^{2}-0.345 V_{\mathrm{n}}-3.109 \quad \text{. . . 6.3(5)}$$

(ii) For turbulence intensity of 0.2 at $2h/3$:

$$\log_{10} C_{\mathrm{fs}}=0.00008 V_{\mathrm{n}}^{4}-0.0028 V_{\mathrm{n}}^{3}+0.0199 V_{\mathrm{n}}^{2}+0.13 V_{\mathrm{n}}-2.985 \quad \text{. . . 6.3(6)}$$

(b) For a 6:1:1 square section (h:b:d), where V_n is in the range 3 to 16:

(i) For turbulence intensity of 0.12 at $2h/3$:

$$\log_{10} C_{fs} = 0.000406 V_n^4 - 0.0165 V_n^3 + 0.201 V_n^2 - 0.603 V_n - 2.76 \quad \ldots 6.3(7)$$

(ii) For turbulence intensity of 0.2 at $2h/3$:

$$\log_{10} C_{fs} = 0.000334 V_n^4 - 0.0125 V_n^3 + 0.141 V_n^2 - 0.384 V_n - 2.36 \quad \ldots 6.3(8)$$

(c) For a 6:2:1 rectangular section (h:b:d), where V_n is in the range 2 to 18:

(i) For turbulence intensity of 0.12 at $2h/3$:

$$\log_{10} C_{fs} = \frac{-3.2 + 0.0683 V_n^2 - 0.000394 V_n^4}{1 - 0.02 V_n^2 + 0.000123 V_n^4} \quad \ldots 6.3(9)$$

(ii) For turbulence intensity of 0.2 at $2h/3$:

$$\log_{10} C_{fs} = \frac{-3 + 0.0637 V_n^2 - 0.00037 V_n^4}{1 - 0.02 V_n^2 + 0.000124 V_n^4} \quad \ldots 6.3(10)$$

(d) For a 6:1:2 rectangular section (h:b:d), where V_n is in the range 2 to 16:

(i) For turbulence intensity of 0.12 at $2h/3$:

$$\log_{10} C_{fs} = 0.000457 V_n^3 - 0.0226 V_n^2 + 0.396 V_n - 4.093 \quad \ldots 6.3(11)$$

(ii) For turbulence intensity of 0.2 at $2h/3$:

$$\log_{10} C_{fs} = 0.00038 V_n^3 - 0.0197 V_n^2 + 0.363 V_n - 3.82 \quad \ldots 6.3(12)$$

NOTE: For intermediate values of h:b, b:d, or turbulence intensity, linear interpolation of $\log_{10} C_{fs}$ shall be used.

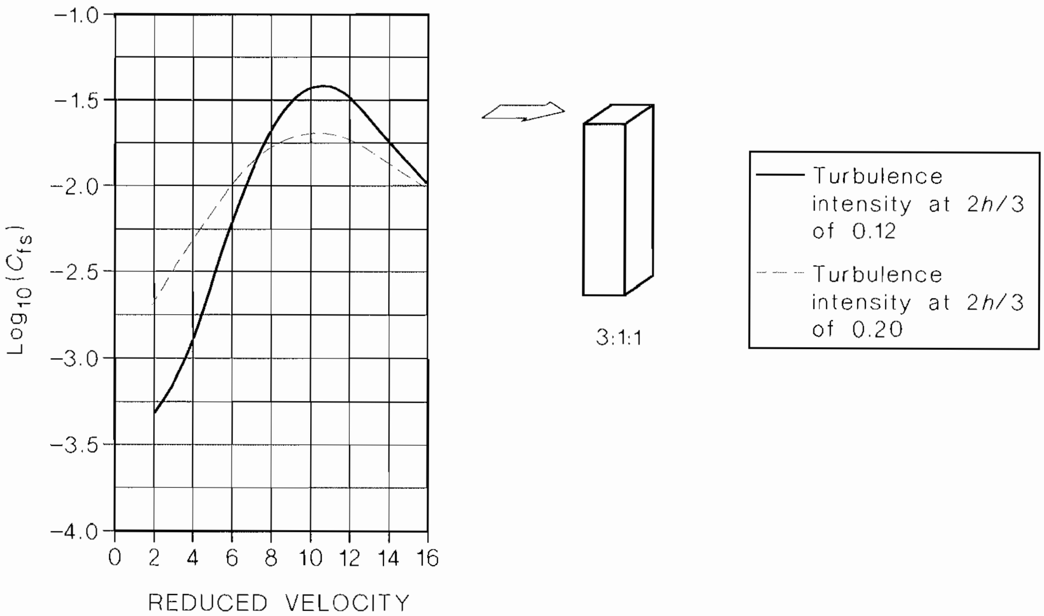


FIGURE 6.2 CROSSWIND FORCE SPECTRUM COEFFICIENT FOR A 3:1:1 SQUARE SECTION

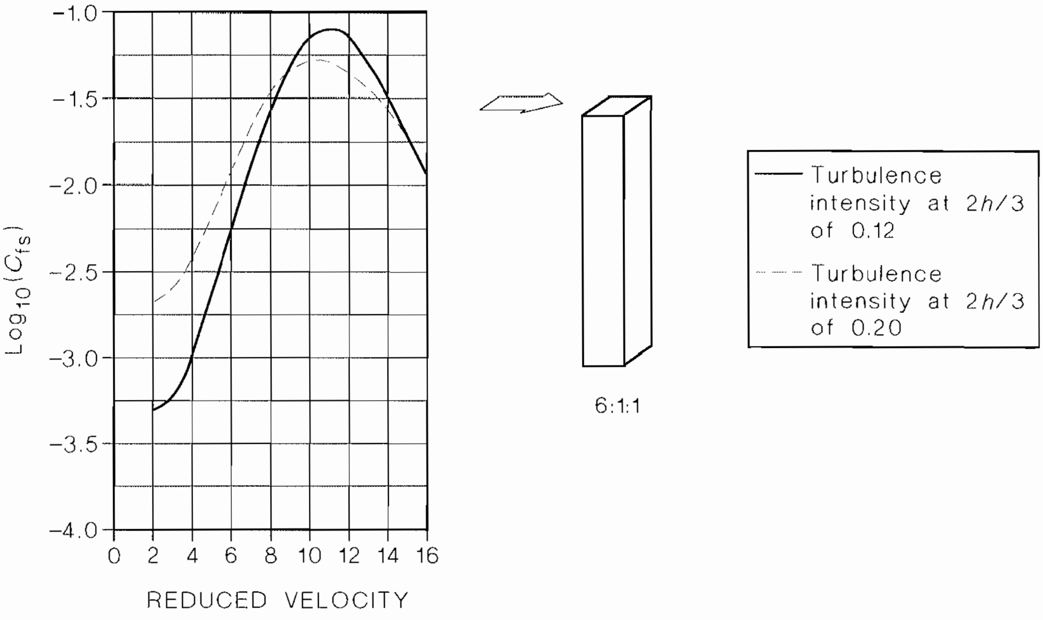


FIGURE 6.3 CROSSWIND FORCE SPECTRUM COEFFICIENT FOR A 6:1:1 SQUARE SECTION

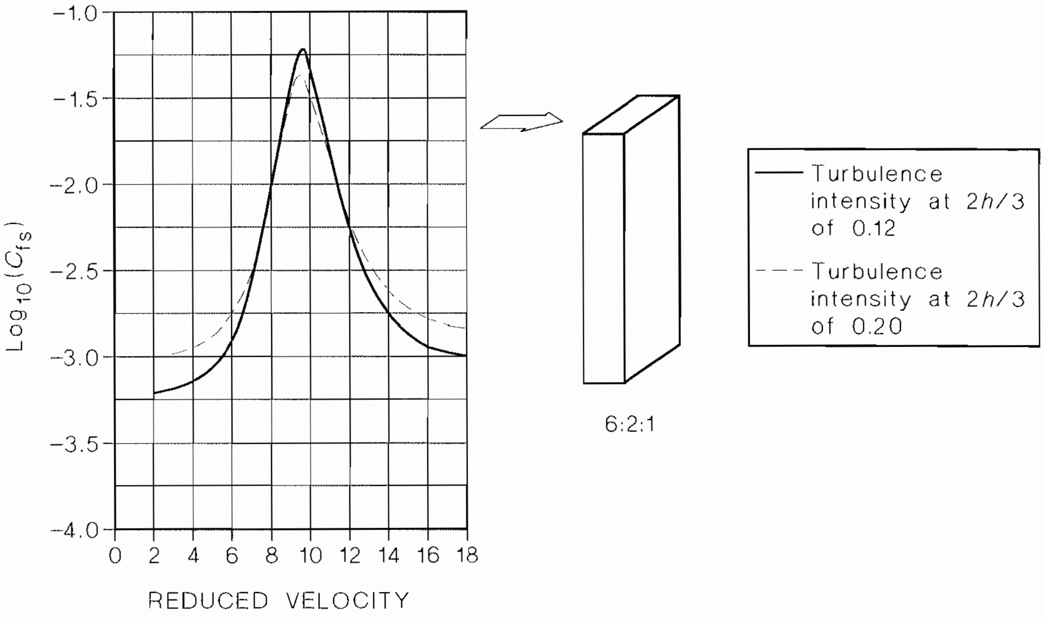


FIGURE 6.4 CROSSWIND FORCE SPECTRUM COEFFICIENT FOR A 6:2:1 RECTANGULAR SECTION

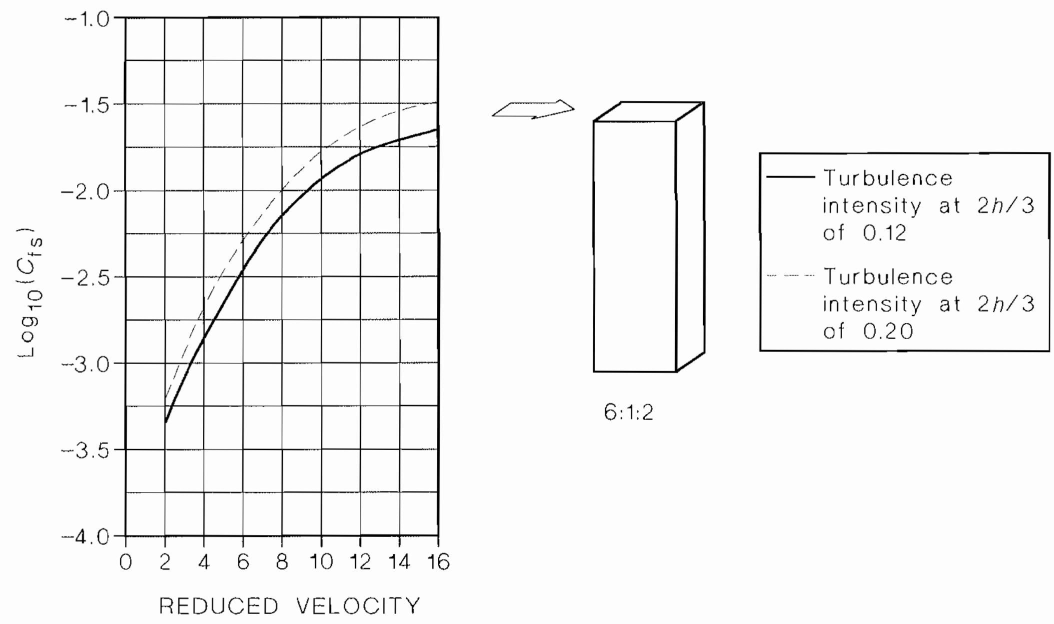


FIGURE 6.5 CROSSWIND FORCE SPECTRUM COEFFICIENT FOR A 6:1:2 RECTANGULAR SECTION

6.3.3 Crosswind response of chimneys, masts and poles of circular cross-section

6.3.3.1 *Crosswind tip deflection*

The maximum amplitude of tip deflection (y_{max}) in crosswind vibration at the critical wind speed due to vortex shedding for chimneys, masts or poles of circular cross-section (without ladders, strakes or other appendages near the top) shall be calculated as follows:

$$y_{max} = Kb_t/\mathrm{Sc} \qquad \ldots 6.3(13)$$

where

K = factor for maximum tip deflection, taken as 0.50 for circular cross-sections

b_t = average breadth of the top third of the structure

Sc = Scruton number given by:

= $4\pi m_t \zeta/(\rho_{air} b^2)$

m_t = average mass per unit height over the top third of the structure

ζ = ratio of structural damping to critical damping of a structure

6.3.3.2 *Equivalent static crosswind force*

The equivalent static wind force per unit height (w_{eq}) for chimneys, masts or poles of circular cross-section (without ladders, strakes or other appendages near the top), as a function of height z, $w_{eq}(z)$, shall be calculated as follows:

$$w_{eq}(z) = m(z)\,(2\pi n_1)^2\, y_{max}\, \phi_1(z) \qquad \ldots 6.3(14)$$

where

$m(z)$ = mass per unit height as a function of height (z)

n_1 = first mode natural frequency of vibration of a structure, in hertz

$\phi_1(z)$ = first mode shape as a function of height (z), normalized to unity at $z = h$, which shall be taken as $(z/h)^2$

NOTE: Equation 6.3(14) may be written as:

$$w_{eq}(z) = 0.5\, \rho_{air}\, [V_{crit}]^2\, b_t\, C_{fig}\, C_{dyn}$$

where

V_{crit} = critical wind speed for vortex shedding, which is approximately $5n_1 \times b_t$ for circular sections

$C_{fig} \times C_{dyn}$ = the product of effective aerodynamic shape factor and dynamic response factor

6.4 COMBINATION OF ALONG-WIND AND CROSSWIND RESPONSE

The total combined peak scalar dynamic action effect (ε_t), such as an axial load in a column, shall be as follows:

$$\varepsilon_t = \varepsilon_{a,m} + \left[\left(\varepsilon_{a,p} - \varepsilon_{a,m}\right)^2 + \varepsilon_{c,p}{}^2\right]^{0.5} \qquad \ldots 6.4(1)$$

where

$\varepsilon_{a,m}$ = action effect derived from the mean along-wind response, given as follows, where the values of g_v, I_h and C_{dyn} are defined in Clause 6.2.2:

= $\varepsilon_{a,p} / [C_{dyn} (1 + 2g_v I_h)]$

$\varepsilon_{a,p}$ = action effect derived from the peak along-wind response

$\varepsilon_{c,p}$ = action effect derived from the peak crosswind response

NOTE:

1 The factor $[C_{dyn} (1 + 2g_v I_h)]$ is a gust factor (G).

2 Maximum action effects derived from the crosswind response of chimneys, masts and poles of circular cross-section (Clause 6.3.3), which occur at the critical wind speed for vortex shedding, should not be combined with action effects for along-wind response calculated at a different wind speed.

APPENDIX A
DEFINITIONS
(Normative)

For the purposes of this Standard, the definitions given herein apply.

A1 Aerodynamic shape factor

Factor to account for the effects of the geometry of the structure on surface pressure due to wind.

A2 Annual probability of exceedence of the action

The probability that a value will be exceeded in any one year.

NOTE: This is the inverse of the so-called 'return period' better described as the average recurrence interval.

A3 Aspect ratio

Ratio of the average roof height of a building to the smallest horizontal dimension, or the ratio of the largest dimension of a structural member to its crosswind breadth.

A4 Awning

Roof-like structure, usually of limited extent, projecting from a wall of a building.

A5 Canopy

Roof adjacent to or attached to a building, generally not enclosed by walls.

A6 Cladding

Material that forms the external surface over the framing of a building or structure.

A7 Design wind speed

Wind speed for use in design, adjusted for annual probability of exceedence, wind direction, geographic position, surrounding environment and height.

A8 Dominant opening

Opening in the external surface of an enclosed building, which directly influences the average internal pressure in response to external pressures at that particular opening.

NOTE: Dominant openings need not be large.

A9 Downdraft

Vertical air motion originating in a thunderstorm, resulting in severe horizontal winds at ground level.

A10 Drag

Force acting in the direction of the wind stream; *see also* lift.

A11 Dynamic response factor

Factor to account for the effects of fluctuating forces and resonant response on wind-sensitive structures.

A12 Eccentricity

The distance from the centroid of a surface, to the point of application of the resultant force derived from the net wind pressure.

A13 Effective surface

A wall, roof or internal surface of a building that contributes significantly to load effects on major structural elements.

A14 Elevated building

Building with a clear, unwalled space underneath the first floor level with a height from ground to underside of the first floor of one-third or more of the total height of the building.

A15 Enclosed building

Building that has a roof and full perimeter walls (nominally sealed) from floor to roof level.

A16 Escarpment

Two-dimensional, steeply sloping, face between nominally level lower and upper plains where the plains have average slopes of not greater than 5%.

A17 First mode shape

Shape of a structure at its maximum amplitude under first mode natural vibration.

A18 First mode natural frequency

Frequency of free oscillation corresponding to the lowest harmonic of vibration of a structure.

A19 Force coefficient

Coefficient that, when multiplied by the incident wind pressure and a reference area, gives the force in a specific direction.

A20 Free roof

Roof (of any type) with no enclosing walls underneath (e.g. freestanding carport).

A21 Freestanding walls

Walls that are exposed to the wind on both sides, with no roof attached (e.g. fences).

A22 Frictional drag

Wind force per unit area acting in a direction parallel to the surface in question.

A23 Gable roof

Ridged roof with two sloping surfaces and vertical triangular end walls.

A24 Hill

Isolated three-dimensional topographic feature standing above the surrounding plains having slopes <5%.

A25 Hip roof

A roof with four sloping (pitched) surfaces, pyramidal in shape, and with level eaves all round. A hip roof on a rectangular plan has two triangular sloping roofs at the short sides (hip ends) and two trapezoidal sloping roofs at the long sides.

A26 Hoardings

Freestanding (rectangular) signboards, and the like, supported clear of the ground.

A27 Immediate supports (cladding)

Those supporting members to which cladding is directly fixed (e.g. battens, purlins, girts, studs).

A28 Lag distance

Horizontal distance downwind, required for the effects of a change in terrain roughness on wind speed to reach the height being investigated.

A29 Lattice towers

Three-dimensional frameworks comprising three or more linear boundary members interconnected by linear bracing members joined at common points (nodes), enclosing an open area through which the wind may pass.

A30 Lift

Force acting at 90° to the wind stream; see also drag.

A31 Mansard roof

A roof with two slopes on all four sides, the lower slope steeper than the upper slope.

NOTE: A mansard roof with the upper slopes less than 10° may be assumed to be flat topped.

A32 Monoslope roof

Planar roof with a constant slope and without a ridge.

A33 Obstructions

Natural or man-made objects that generate turbulent wind flow, ranging from single trees to forests and from isolated small structures to closely spaced multi-storey buildings.

A34 Permeable

Surface with an aggregation of small openings, cracks, and the like, which allows air to pass through under the action of a pressure differential.

A35 Pitched roof

Bi-fold, bi-planar roof (two sloping surfaces) meeting at a ridge.

A36 Pressure

Air pressure referenced to ambient air pressure.

NOTE: In this Standard, negative values are less than ambient (suction), positive values exceed ambient. Net pressures act normal to a surface in the direction specified.

A37 Pressure coefficient

Ratio of the pressure acting at the point on a surface, to the free-stream dynamic pressure of the incident wind.

A38 Rectangular building

For the purposes of Section 5 of this Standard, rectangular buildings include buildings generally made up of rectangular shapes in plan.

A39 Reynolds number

The ratio of the inertial forces to the viscous forces in the airflow.

A40 Ridge (topographic feature)

Two-dimensional crest or chain of hills with sloping faces on either side of the crest.

A41 Roughness length

Theoretical quantification of the turbulence-inducing nature of a particular type of terrain on airflow (wind).

A42 Scruton number

A mass-damping parameter.

A43 Shelter room

Any space designated to provide shelter to one or more persons.

A44 Solidity (of cladding)

Ratio of the solid area to the total area of the surface.

A45 Structural elements, major

Structural elements with tributary areas are greater than 10 m2.

A46 Structural elements, minor

Structural elements with tributary areas are less than or equal to 10 m2.

A47 Terrain

Surface roughness condition when considering the size and arrangement of obstructions to the wind.

A48 Topography

Major land surface features, comprising hills, valleys and plains, that strongly influence wind flow patterns.

A49 Tornado

Violently rotating column of air, that is suspended, observable as a funnel cloud attached to the cloud base of a convective cloud.

A50 Tributary area

Area of building surface contributing to the force being considered.

A51 Tropical cyclone

An intense low-pressure centre accompanied by heavy rain and gale-force winds or greater. It forms over warm tropical oceans and decays rapidly over land. Such systems affect a large area and, in the southern hemisphere, winds spiral clockwise into the centre.

A52 Troughed roof

Bi-fold, bi-planar roof with a valley at its lowest point.

A53 Turbulence intensity

The ratio of the standard deviation of the fluctuating component of wind speed to the mean (time averaged) wind speed.

APPENDIX B
NOTATION
(Normative)

Unless stated otherwise, the notation used in this Standard shall have the following meanings with respect to a structure, member or condition to which a clause is applied.

NOTE: See Clause 1.5 for units.

A = surface area of the element or the tributary area that transmits wind forces to the element, being—

= area upon which the pressure acts, which may not always be normal to the wind stream when used in conjunction with the pressure coefficient (C_p);

= projected area normal to the wind stream when used in conjunction with a drag force coefficient (C_d); or

= areas as defined in applicable clauses (see Appendix E) when used in conjunction with a force coefficient ($C_{F,x}$) or ($C_{F,y}$)

A_a = reference area of ancillaries on a tower

A_{ref} = reference area of flag

$A_{z,s}$ = total projected area of the tower section at height z

A_z = a reference area, at height (z), upon which the pressure (p_z) at that height acts

a = constant for ease of calculation (Paragraph E4.2.3, Appendix E)

or

dimension used in defining the extent of application of local pressure factors

B_s = background factor, which is a measure of the slowly varying background component of the fluctuating response, caused by low-frequency wind speed variations

b = breadth of a structure or element, usually normal to the wind stream (see Figures 5.2, C5, C7 of Appendix C, D1 of Appendix D, E1, E2, E4 and Tables E3, E4 and E5 of Appendix E)

or

average diameter of a circular section

b_D = diagonal breadth of UHF antennas

b_i = average diameter or breadth of a section of a tower member

b_N = normal breadth of UHF antennas

b_{0h} = average breadth of the structure between heights 0 and h

b_s = average breadth of shielding buildings, normal to the wind stream

b_{sh} = average breadth of the structure between heights s and h

b_t = average breadth of the top third of the structure

b_z = average breadth of the structure at the section at height (z)

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

C_d = drag force coefficient for a structure or member in the direction of the wind stream

C_{da} = value of drag force coefficient (C_d) on an isolated ancillary on a tower

C_{de} = effective drag force coefficient for a tower section with ancillaries

C_{dyn} = dynamic response factor

$C_{F,x}$ = force coefficient for a structure or member, in the direction of the x-axis

$C_{F,y}$ = force coefficient for a structure or member, in the direction of the y-axis

C_f = frictional drag force coefficient

C_{fig} = aerodynamic shape factor

$C_{fig,1}$ = aerodynamic shape factor for the first frame in the upwind direction

C_{fs} = crosswind force spectrum coefficient generalized for a linear mode shape

$C_{p,b}$ = external pressure coefficient for sides of bins, silos and tanks

$C_{p,e}$ = external pressure coefficient

$C_{p,i}$ = internal pressure coefficient

$C_{p,l}$ = net pressure coefficient for the leeward half of a free roof

$C_{p,n}$ = net pressure coefficient acting normal to the surface for canopies, freestanding roofs, walls and the like

$C_{p,w}$ = net pressure coefficient for the windward half of a free roof

$C_{p1}(\theta_b)$ = external pressure coefficient on walls of bins, silos or tanks of unit aspect ratio ($c/b = 1$) as a function of θ_b

c = constant for ease of calculation (Paragraph E4.2.3)

or

= net height of a hoarding, flag, bin, silo or tank (not including roof or lid height)

or

= height between the highest and lowest points on a hyperbolic paraboloid roof

D = downwind roof slope

d = depth or distance parallel to the wind stream to which the plan or cross-section of a structure or shape extends (e.g. the outside diameter)

or

= length of span of curved roof

d_a = along-wind depth of a porous wall or roof surface

d_s = length of span of the first pitched roof in a multi-span building

E = site elevation above mean sea level

E_t = spectrum of turbulence in the approaching wind stream

e = the base of Napierian logarithms (≈ 2.71828)

e = horizontal eccentricity of net pressure

F = force on a building element, in newtons

F_C = factor for region C to account for lack of recent analysis of cyclone activity

F_D = factor for region D to account for lack of recent analysis of cyclone activity

f = frictional force per unit area parallel to a surface, in newtons per square metre

f_z = the design frictional-distributed force parallel to the surface, calculated in Clause 2.4.2 at height z, in newtons per square metre

g_R = peak factor for resonant response (10 min period)

g_v = peak factor for the upwind velocity fluctuations

H = height of the hill, ridge or escarpment

H_s = height factor for the resonant response

h = average roof height of structure above ground

h_c = height from ground to the attached canopy, freestanding roof, wall or the like

h_p = height of parapet above average roof level

h_r = average height of surface roughness

h_s = average roof height of shielding buildings

I_h = turbulence intensity, obtained from Table 6.1 by setting z equal to h

I_z = turbulence intensity at height z given for various terrain categories in Table 6.1

K = factor for maximum tip deflection

K_a = area reduction factor

K_{ar} = aspect ratio correction factor for individual member forces

K_c = combination factor

$K_{c,e}$ = combination factor for external pressures

$K_{c,i}$ = combination factor for internal pressures

K_i = factor to account for the angle of inclination of the axis of members to the wind direction

K_{in} = correction factor for interference

K_ℓ = local pressure factor

K_m = mode shape correction factor for crosswind acceleration

K_p = net porosity factor, used for free walls

or

= porous cladding reductive factor, used for cladding on buildings

K_r = parapet reduction factor

K_{sh} = shielding factor for shielded frames in multiple open-framed structures

k = mode shape power exponent

k_b = factor for a circular bin

L_h = measure of integral turbulence length scale at height h

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

L_1 = length scale, in metres, to determine the vertical variation of M_h, to be taken as the greater of 0.36 L_u or 0.4 H

L_2 = length scale, in metres, to determine the horizontal variation of M_h, to be taken as 4 L_1 upwind for all types, and downwind for hills and ridges, or 10 L_1 downwind for escarpments

L = leeward wall

l = length of member

l_f = flag length

l_s = average spacing of shielding buildings

M_c = crosswind base overturning moment

M_d = wind direction multiplier (see Clause 3.3)

M_s = shielding multiplier

M_t = topographic multiplier

M_h = hill shape multiplier

M_{lee} = lee (effect) multiplier (taken as 1.0, except in New Zealand lee zones, see Clause 4.4.3)

$M_{z,cat}$ = terrain/height multiplier

m_0 = average mass per unit height

m_f = mass per unit area of flag

m_t = average mass per unit height over the top third of the structure

$m(z)$ = mass per unit height as a function of height z

N = reduced frequency (non-dimensional)

n = number of spans of a multi-span roof

n_1 = first mode natural frequency of vibration of a structure, in hertz

n_a = first mode natural frequency of vibration of a structure in the along-wind direction, in hertz

n_c = first mode natural frequency of vibration of a structure in the crosswind direction, in hertz

n_s = number of upwind shielding buildings within a 45° sector of radius 20 h and with $h_s \geq h$

p = design wind pressure acting normal to a surface, in pascals

= p_e, p_i or p_n where the sign is given by the C_p values used to evaluate C_{fig}

NOTE: Pressures are taken as positive, indicating pressures above ambient and negative, indicating pressures below ambient.

p_e = external wind pressure

p_i = internal wind pressure

p_n = net wind pressure

p_z = design wind pressure, in pascals (normal to the surface), at height z, calculated in Clause 2.4.1

NOTE: The sign convention for pressures leads to forces towards the surface for positive pressures and forces away from the surface for negative pressures.

R = inverse of the annual probability of exceedence of the wind speed

R = crosswind roof slope

Re = Reynolds number

r	= rise of a curved roof;
	corner radius of a structural shape; or
	aspect ratio of a building (Clause 5.4.4)
S	= size reduction factor
S	= side wall
Sc	= Scruton number
s	= shielding parameter; *or*
	= height of the level at which action effects are calculated for a structure
T	= top roof section
U	= upwind roof slope
$V_{des,\theta}$	= building orthogonal design wind speeds (usually, $\theta = 0°, 90°, 180°$ and $270°$), as given in Clause 2.3 NOTE: $V_{des,\theta}$ may be expressed as a function of height z, for some applications, e.g. windward walls of tall buildings (>25m).
$V_{des,\theta}(z)$	= building orthogonal design wind speeds as a function of height z
V_n	= reduced velocity (non dimensional)
$V_{sit,\beta}$	= wind speeds for a site, varying according to compass direction
V_R	= regional three-second gust wind speed, in metres per second, for annual probability of exceedence of $1/R$
W	= wind actions (see AS/NZS 1170.0)
W	= windward wall
W_s	= wind actions for serviceability limit states (determined using a regional wind speed appropriate to the annual probability of exceedence for serviceability limit states)
W_u	= wind actions for ultimate limit states (determined using a regional wind speed appropriate to the annual probability of exceedence specified for ultimate limit states)
$w_{eq}(z)$	= equivalent static wind force per unit height as a function of height z
w	= width of a tower; *or*
	= shortest horizontal dimension of the building
w_c	= width of canopy, awning carport, or similar, from the face of the building
x	= distance from the windward edge of a canopy or cantilevered roof; *or*
	= horizontal distance upwind or downwind of the structure to the crest of the hill, ridge or escarpment
x_i	= distance downwind from the start of a new terrain roughness to the position where the developed height of the inner layer equals z (lag distance)
$\ddot{x}_{max}$	= peak acceleration, at the top of a structure in the along-wind direction
$\ddot{y}_{max}$	= peak acceleration, at the top of a structure in the crosswind direction
y_{max}	= maximum amplitude of tip deflection in crosswind vibration at the critical wind speed

z	= reference height on the structure above the average local ground level
$z_{0,r}$	= larger of the two roughness lengths at a boundary between roughnesses
α	= angle of slope of a roof
β	= angle of compass wind direction, measured clockwise from North (0°), for determining site wind velocities
ΔC_d	= additional drag coefficient due to an ancillary attached to one face or located inside the tower section
Δz	= height of the section of the structure upon which the wind pressure acts
δ	= solidity ratio of the structure (surface or open frame) which is the ratio of solid area to total area of the structure
δ_e	= effective solidity ratio for an open frame
$\varepsilon_{a,m}$	= action effect derived from the mean along-wind response
$\varepsilon_{a,p}$	= action effect derived from the peak along-wind response
$\varepsilon_{c,p}$	= action effect derived from the peak crosswind response
ε_t	= combined peak scalar dynamic action effect
ζ	= ratio of structural damping to critical damping of a structure
θ	= angle of the upwind direction to the orthogonal axes of a structure, in degrees
θ_a	= angle of deviation of the wind stream from the line joining the centre of the tower cross-section to the centre of the ancillary, in degrees
θ_b	= angle from the wind direction to a point on the wall of a circular bin, silo or tank, in degrees
θ_m	= angle between the wind direction and the longitudinal axis of the member, in degrees
λ	= spacing ratio for parallel open frames, equal to the frame spacing (centre-to-centre) divided by the projected frame width normal to the wind direction
π	= the ratio of the circumference of any circle to its diameter (approx. 3.14159)
ρ_{air}	= density of air, which shall be taken as 1.2 kg/m^3 NOTE: This value is based on 20°C and typical ground level atmospheric pressure and variation may be necessary for very high altitudes or cold environments.
$\phi_1 (z)$	= first mode shape as a function of height z, normalized to unity at $z = h$

APPENDIX C

ADDITIONAL PRESSURE COEFFICIENTS FOR ENCLOSED BUILDINGS

(Normative)

C1 ADDITIONAL PRESSURE COEFFICIENTS

The external pressure coefficients ($C_{p,e}$) given in this Appendix shall be used to calculate the aerodynamic shape factor for pressures on appropriately shaped enclosed buildings in accordance with Clauses 5.2 and 5.4.

C2 MULTI-SPAN BUILDINGS (α <60°)

External pressure coefficients ($C_{p,e}$) for the multi-span buildings shown in Figures C1 and C2 for wind directions $\theta = 0°$ and $\theta = 180°$ shall be obtained from Table C1 or Table C2.

Where two values are listed for pressure coefficients in Tables C1 and C2, the roof shall be designed for both values.

All pressure coefficients shall be used with the value of wind speed applying at average roof height (h).

External pressure coefficients for wind directions of $\theta = 90°$ and $\theta = 270°$ shall be obtained from Table 5.3(A) but [$-0.05(n-1)$] shall be added to the roof pressure coefficients in the region 0 to 1h from the leading edge, where n is the total number of spans. For this calculation, take n = 4, if n is greater than 4.

TABLE C1

EXTERNAL PRESSURE COEFFICIENTS ($C_{p,e}$) FOR MULTI-SPAN BUILDINGS—PITCHED ROOFS

Surface reference (see Figure C1)				
A	B	C	M	Y
0.7	Use Table 5.3(a), 5.3(b) or 5.3(c) for same (h/d_s) and α, as appropriate		−0.3 and 0.2 for α <10° −0.5 and 0.3 for α ≥10°	−0.2

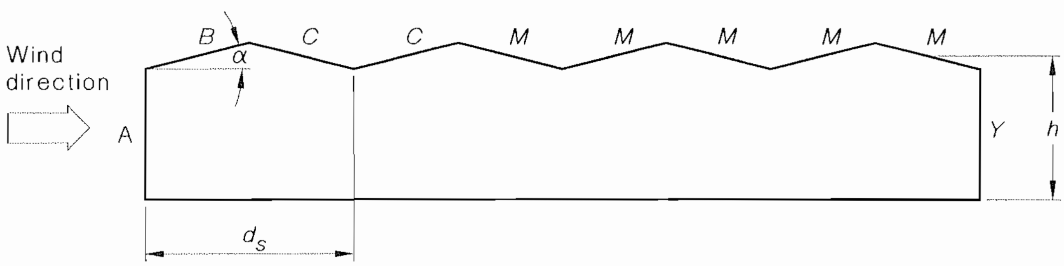


FIGURE C1 EXTERNAL PRESSURE COEFFICIENTS ($C_{p,e}$) FOR MULTI-SPAN BUILDINGS—PITCHED ROOFS

TABLE C2

EXTERNAL PRESSURE COEFFICIENTS ($C_{p,e}$) FOR MULTI-SPAN BUILDINGS—SAW-TOOTH ROOFS

Wind direction (θ) degrees	Surface reference (see Figure C2)								
	A	B	C	D	M	N	W	X	Y
0	0.7	−0.9	−0.9	−0.5, 0.2	−0.5, 0.5	−0.5, 0.3	−0.3, 0.5	−0.4	−0.2
180	−0.2	−0.2, 0.2	−0.3	−0.2, 0.2	−0.4	−0.4	−0.7	−0.3	0.7

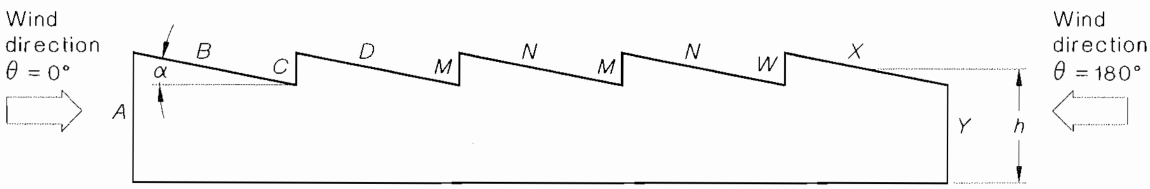


FIGURE C2 EXTERNAL PRESSURE COEFFICIENTS ($C_{p,e}$) FOR MULTI-SPAN BUILDINGS—SAW-TOOTH ROOFS

C3 BUILDINGS WITH CURVED ROOFS

For external pressure coefficients ($C_{p,e}$) of curved, arched or domed roofs with profiles approximating a circular arc, wind directions normal to the axis of the roof shall be obtained from Table C3.

When two values are listed, the roof shall be designed for both values. In these cases, roof surfaces may be subjected to either positive or negative values due to turbulence. Alternative combinations of external and internal pressures (see Clause 2.5) shall be considered, to obtain the most severe conditions for design.

All pressure coefficients shall be used with the value of wind speed applying at average roof height (h).

External pressure coefficients ($C_{p,e}$) for wind directions parallel to the axis (ridge) of arched roofs shall be obtained from Table 5.3(A).

The zero values provided for the windward quarter are alternative values for action effects, such as bending, which are sensitive to pressure distribution. (Turbulence and fluctuations in pressure will produce a range of values occurring at different times during a wind event.)

For arched roofs, the effect of breadth-to-span ratio shall be taken into account by multiplying all the coefficients in Table C3 by a factor of $(b/d)^{0.25}$, where b = breadth normal to the wind and d = span (see Figure C3). If $(b/d)^{0.25}$ is less than 1.0, it shall be taken as 1.0.

Table C3 provides external pressure coefficients for circular arc roofs with no substantial interference to the airflow over the roof. Where a ridge ventilator of a height at least 5% of the total height of the roof is present, the external pressure coefficient on the central half of the roof (T) shall be modified by adding +0:3; that is, the value of a negative coefficient (suction) is reduced by 0.3. Such reductions shall not be made for the wind direction along the axis of the roof, for which the ridge ventilator has little effect on the airflow and resulting external pressures.

All combinations of external pressure coefficients on U, T and D shall be checked.

TABLE C3

EXTERNAL PRESSURE COEFFICIENTS ($C_{p,e}$)—CURVED ROOFS

Rise-to-span ratio (r/d)	Windward quarter (U)	Centre half (T)	Leeward quarter (D)
0.18	$(0.3 - 0.4\ h/r)$ or 0.0	$-(0.55 + 0.2\ h/r)$	$-(0.25 + 0.2\ h/r)$ or 0.0
0.5	$(0.5 - 0.4\ h/r)$ or 0.0		$-(0.1 + 0.2\ h/r)$ or 0.0

NOTES:

1 h is the average roof height and r is the rise of the arch (see Figure C3).

2 For intermediate values of rise to span ratio, linear interpolation shall be used.

3 For h/r >2, Table C3 shall be applied with h/r = 2.

4 For r/d < 0.18, Table 5.3(A) shall be applied.

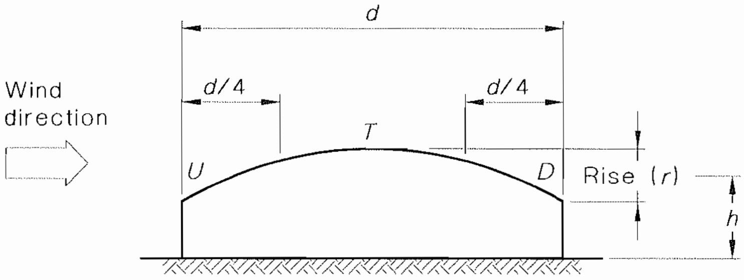


FIGURE C3 EXTERNAL PRESSURE COEFFICIENTS ($C_{p,e}$)—CURVED ROOFS

C4 MANSARD ROOFS

The external pressure coefficients ($C_{p,e}$) for a flat-topped mansard roof (see Figure C4) for the wind direction $\theta = 0°$ shall be determined as follows:

(a) *For upwind slope* (U)—using values for upwind slope given in Clause 5.4.1.

(b) *For downwind slope* (D)—using values for downwind slope given in Clause 5.4.1, using the same roof pitch α as for the upwind slope.

(c) *For flat top* (T)—using the same values as determined for downwind slope.

The external pressure coefficients ($C_{p,e}$) for the wind direction $\theta = 90°$ shall be determined from Clause 5.4.1 assuming R for gable roofs.

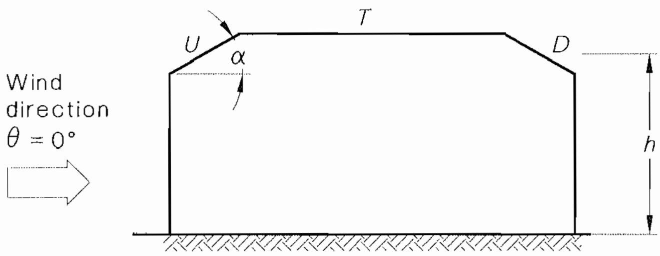


FIGURE C4 EXTERNAL PRESSURE COEFFICIENTS ($C_{p,e}$) FOR MANSARD ROOFS

C5 CIRCULAR BINS, SILOS AND TANKS

C5.1 General

Grouped circular bins, silos and tanks with spacing between walls greater than two diameters shall be treated as isolated silos. Closely spaced groups with spacing less than 0.1 diameters shall be treated as a single structure for wind actions and pressure determined using Tables 5.2 and 5.3. For intermediate spacings, linear interpolation shall be used.

C5.2 Isolated circular bins, silos and tanks

C5.2.1 *Walls*

The aerodynamic shape factor (C_{fig}) for calculating external pressures on the walls of bins, silos and tanks of circular cross-section shall be equal to the external pressure coefficients ($C_{p,b}$) as a function of the angle θ_b (see Figure C5), given as follows for shapes in the ranges indicated:

$$C_{p,b}\,(\theta_b) = k_b C_{p1}(\theta_b) \qquad \ldots \text{C5(1)}$$

where

the cylinder is standing on the ground or supported by columns of a height not greater than the height of the cylinder (c)

c/b is in the range 0.25 to 4.0 inclusive

θ_b = angle from the wind direction to a point on the wall of a circular bin, silo or tank, in degrees

k_b = factor (or function) for a circular bin, given as follows:

= 1.0 for $C_{p1} \geq -0.15$, or

= $1.0 - 0.55(C_{p1}(\theta_b) + 0.15)\log_{10}(c/b)$ for $C_{p1} < -0.15$ ... C5(2)

$$C_{p1}(\theta_b) = -0.5 + 0.4\cos\theta_b + 0.8\cos 2\,\theta_b + 0.3\cos 3\theta_b - 0.1\cos 4\theta_b - 0.05\cos 5\theta_b \qquad \ldots \text{C5(3)}$$

For calculating the overall drag force on the wall section of circular bins, silos and tanks (both elevated and on ground) C_{fig} shall be taken as 0.63 (based on an elevation area $b \times c$). This drag force coefficient arises from an integration of the along-wind component of the normal pressures given by Equations C5(2) and C5(3).

External pressure coefficients for the underside of elevated bins, silos and tanks shall be calculated as for elevated enclosed rectangular buildings (see Clause 5.4.1).

Figure C6 is a graphical presentation of the external pressure coefficient (C_{p1}) for circular bins, silos and tanks of unit aspect ratio (i.e. c/b = 1.0) at individual locations around the perimeter, and θ_b degrees from the incident wind direction as calculated from Equation C5(1).

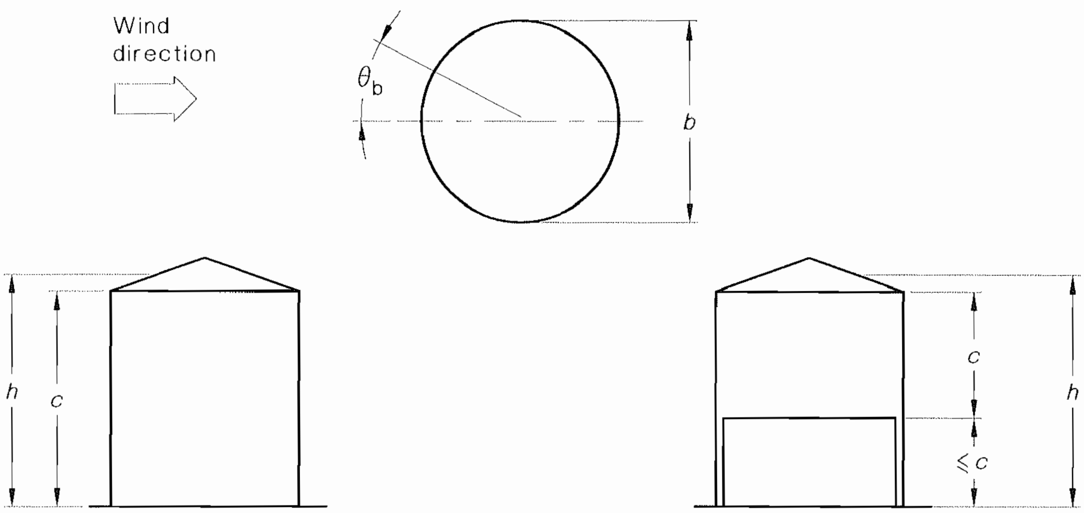


FIGURE C5 EXTERNAL PRESSURE COEFFICIENTS ($C_{p,b}$) ON WALLS OF CIRCULAR BINS, SILOS AND TANKS ($0.25 \leq c/b \leq 4.0$)

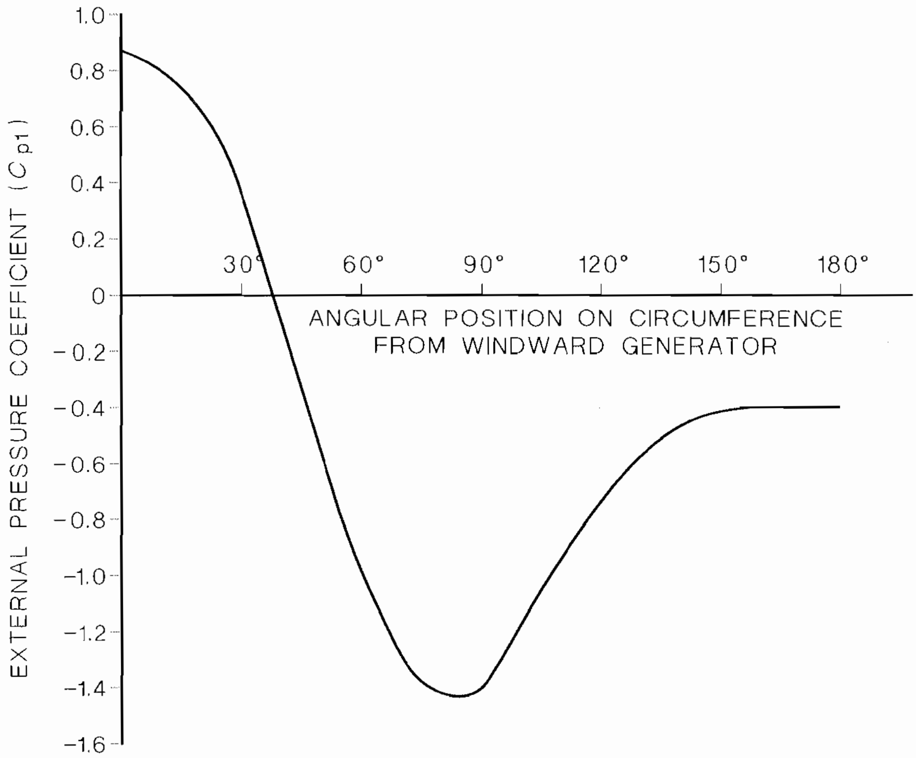


FIGURE C6 PLOT OF EXTERNAL PRESSURE COEFFICIENTS (C_{p1}) ON WALLS OF CIRCULAR BINS, SILOS AND TANKS (c/b = 1)

C5.2.2 *Roofs and lids*

The aerodynamic shape factor (C_{fig}) for calculating external pressures on the roofs or lids of bins, silos or tanks of circular cross-section, as shown in Figure C7, shall be as follows:

$$C_{fig} = C_{p,e} K_a K_\ell \qquad \text{. . . C5(4)}$$

where $C_{p,e}$ is given in Table C7 for zones A and B as shown in Figure C7. K_a is given in Clause 5.4.2 and K_ℓ is given in Clause 5.4.4.

The local pressure factor (K_ℓ) is applicable to the windward edges of roofs with slope less than or equal to 30°, and to the region near the cone apex for roofs with slope greater than 15°. The applicable areas are shown in Figure C7.

TABLE C7

EXTERNAL PRESSURE COEFFICIENTS ($C_{p,e}$) FOR ROOFS OF CIRCULAR BINS, SILOS AND TANKS

Zone A	Zone B
−0.8	−0.5

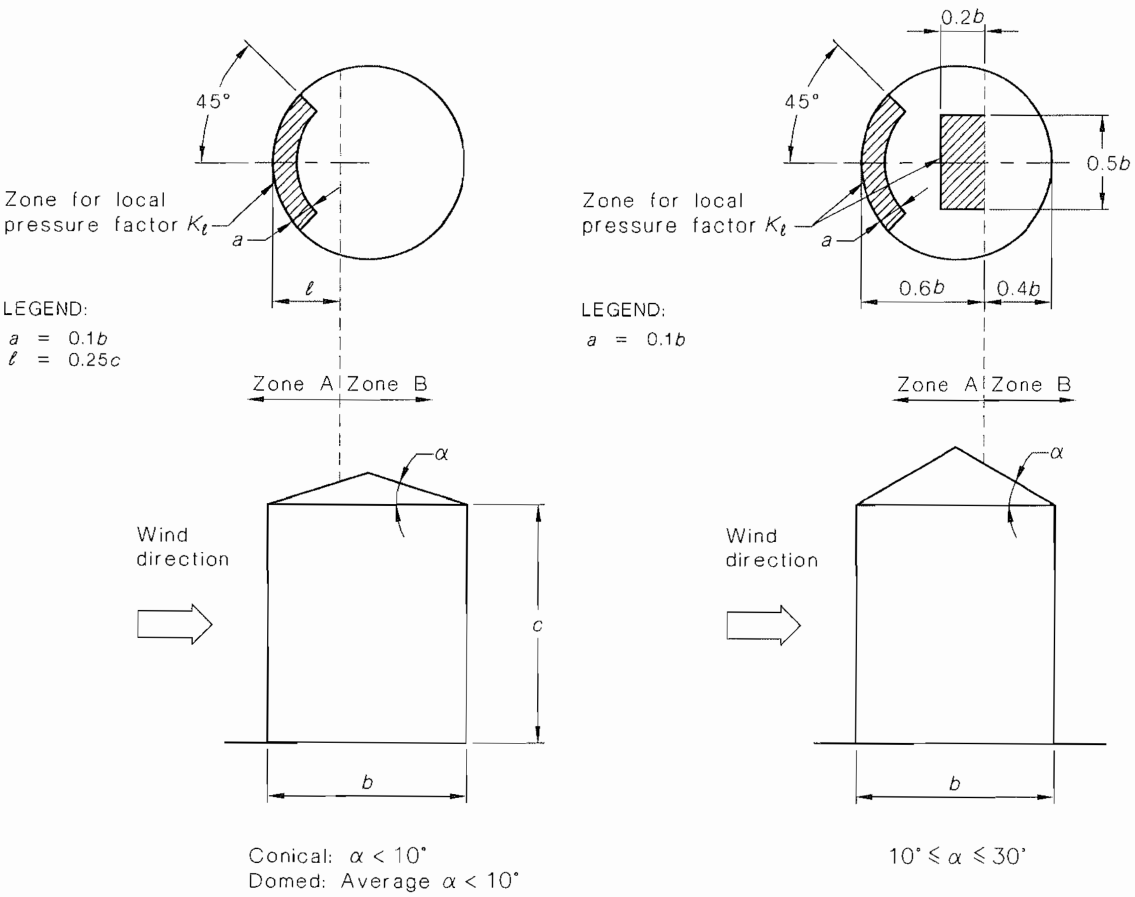


FIGURE C7 EXTERNAL PRESSURE COEFFICIENTS ($C_{p,e}$) FOR ROOFS OF CIRCULAR BINS, SILOS AND TANKS ($0.25 < c/b < 4.0$)

C5.2.3 *Internal pressures in bins, silos and tanks*

Internal pressures within bins, silos and tanks with vented roofs shall be determined as an area-weighted average of the external pressures at the position of the vents and openings, determined according to Paragraph C5.2.2.

For open-top bins, silos or tanks, the internal pressure shall be determined as follows:

$$C_{fig} = C_{p,i} \qquad \ldots C5(5)$$

$$= -0.9 - 0.35 \log_{10}(c/b)$$

APPENDIX D
FREESTANDING WALLS, HOARDINGS AND CANOPIES
(Normative)

D1 GENERAL

D1.1 Application

This Appendix shall be used to calculate aerodynamic shape factors (C_{fig}) for the following structures and structural elements:

(a) Free roofs, including hyperbolic paraboloid roofs.

(b) Canopies, awnings and carports (adjacent to enclosed buildings).

(c) Cantilevered roofs.

(d) Hoardings and freestanding walls.

To calculate forces on the structure, use the area of the structure (one side only) as the reference area for normal pressures, and use the area of all the affected sides as the reference area for frictional pressure.

D1.2 Area reduction factor (K_a)

For the design of freestanding roofs and canopies, the area reduction factor (K_a) shall be as defined in Clause 5.4.2. For all other cases in this Appendix, K_a = 1.0.

D1.3 Local net pressure factor (K_ℓ)

For the design of cladding elements and elements that offer immediate support to the cladding in free roofs and canopies, the values of local net pressure factor (K_ℓ) given in Table D1 shall be used. For other elements in free roofs and canopies and for all other cases in this Appendix, K_ℓ = 1.0.

TABLE D1

LOCAL NET PRESSURE FACTORS (K_ℓ) FOR OPEN STRUCTURES

Case	Description	Local net pressure factor (K_ℓ)
1	Pressures on an area between 0 and 1.0a^2 within a distance 1.0a from an upwind roof edge, or downwind of a ridge with a pitch of 10° or more	1.5
2	Pressures on an area of 0.25a^2 or less, within a distance 0.5a from an upwind roof edge, or downwind of a ridge with a pitch of 10° or more	2.0
3	Upward net pressures on an area of 0.25a^2 or less, within a distance 0.5a from an upwind corner of a free roof with a pitch of less than 10°	3.0

NOTES:

1 Where a is 20% of the shortest horizontal plan dimension of the free roof or canopy.

2 If an area of cladding is covered by more than one case in Table D1e largest value of K_ℓ shall be used.

3 The largest aspect ratio of any local pressure factor area on the roof shall not exceed 4.

D1.4 Net porosity factor (K_p)

For freestanding hoardings and walls, the net porosity factor (K_p) shall be as calculated in equation D1. For all other cases in this Appendix, K_p = 1.0.

$$K_p = 1 - (1 - \delta)^2 \qquad \text{. . . D1}$$

where

δ = solidity ratio of the structure (surface or open frame), which is the ratio of solid area to total area of the structure

D2 FREESTANDING HOARDINGS AND WALLS

D2.1 Aerodynamic shape factor for normal net pressure on freestanding hoardings and walls

The aerodynamic shape factor (C_{fig}) for calculating net pressure across freestanding rectangular hoardings or walls (see Figure D1) shall be as follows:

$$C_{fig} = C_{p,n} K_p \qquad \text{. . . D2}$$

where

$C_{p,n}$ = net pressure coefficient acting normal to the surface, obtained from Table D2 using the dimensions defined in Figure D1

K_p = net porosity factor, as given in Paragraph D1.4

NOTES:

1 The factors K_a and K_ℓ do not appear in this equation as they are taken as 1.0.

2 Height for calculation of $V_{des,\theta}$ is the top of the hoarding or wall,i.e. height (h) (see Figure D1).

Pressures derived from Equation D2 shall be applied to the total area (gross) of the hoarding or wall (for example,$b \times c$).

The resultant of the pressure shall be taken to act at half the height of the hoarding, ($h - c/2$), or wall, ($c/2$), with a horizontal eccentricity (e).

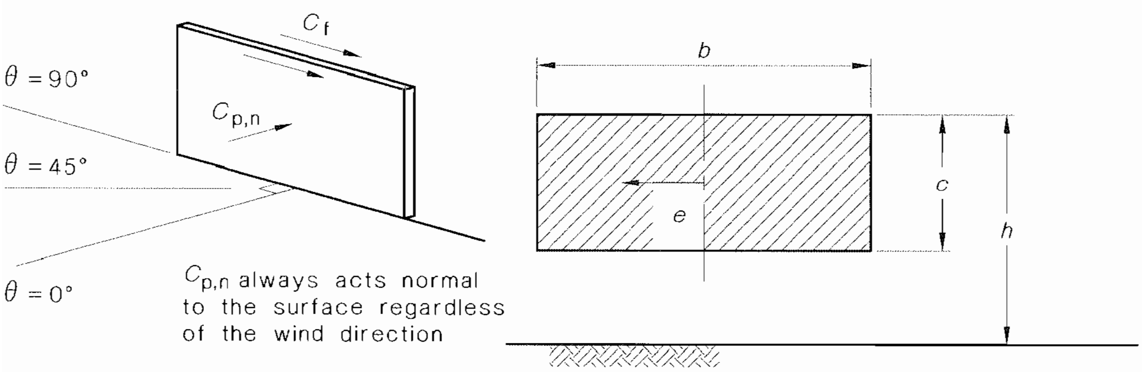


FIGURE D1 FREESTANDING HOARDINGS AND WALLS

TABLE D2(A)

NET PRESSURE COEFFICIENTS ($C_{p,n}$)—HOARDINGS AND FREESTANDING WALLS—WIND NORMAL TO HOARDING OR WALL, $\theta = 0°$

b/c	c/h	$C_{p,n}$	e
0.5 to 5	0.2 to 1	$1.3 + 0.5(0.3 + \log_{10}(b/c))(0.8 - c/h)$	0
>5		$1.7 - 0.5\ c/h$	0
all	<0.2	$1.4 + 0.3\log_{10}(b/c)$	0

TABLE D2(B)

NET PRESSURE COEFFICIENTS ($C_{p,n}$)—HOARDINGS AND FREESTANDING WALLS—WIND AT 45° TO HOARDING OR WALL, $\theta = 45°$

b/c	c/h	$C_{p,n}$	e
0.5 to 5 inclusive	0.2 to 1	$1.3 + 0.5(0.3 + \log_{10}(b/c))(0.8 - c/h)$	$0.2b$
	<0.2	$1.4 + 0.3\log_{10}(b/c)$	$0.2b$

TABLE D2(C)

NET PRESSURE COEFFICIENTS ($C_{p,n}$)—HOARDINGS AND FREESTANDING WALLS—WIND AT 45° TO HOARDING OR WALL, $\theta = 45°$

b/c	c/h	Distance from windward free end	$C_{p,n}$ (see Note)
>5	≤0.7	0 to $2c$	3.0
		$2c$ to $4c$	1.5
		>$4c$	0.75
	>0.7	0 to $2h$	2.4
		$2h$ to $4h$	1.2
		>$4h$	0.6

NOTE: Where a return wall or hoarding forms a corner extending more than $1c$, the $C_{p,n}$ on 0 to $2c$ for a hoarding shall be 2.2, and 0 to $2h$ for a wall $C_{p,n}$ shall be 1.8.

TABLE D2(D)

NET PRESSURE COEFFICIENTS ($C_{p,n}$)—HOARDINGS AND FREESTANDING WALLS—WIND PARALLEL TO HOARDING OR WALL, $\theta = 90°$

b/c	c/h	Distance from windward free end	$C_{p,n}$ (see Note)
All	≤0.7	0 to $2c$	±1.2
		$2c$ to $4c$	±0.6
		>$4c$	±0.3
	>0.7	0 to $2h$	±1.0
		$2h$ to $4h$	±0.25
		>$4h$	±0.25

NOTE: Take values of $C_{p,n}$ of the same sign.

D2.2 Aerodynamic shape factor for frictional drag

The aerodynamic shape factor (C_{fig}) for calculating frictional drag effects on freestanding hoardings and walls, where the wind is parallel to the hoarding or wall, shall be equal to C_f, which shall be determined as given in Table D3. The frictional drag on both surfaces shall be calculated and summed and added to the force on any exposed members calculated in accordance with Appendix E.

TABLE D3

FRICTIONAL DRAG COEFFICIENT (C_f)

Surface description	C_f
Surfaces with ribs across the wind direction	0.04
Surfaces with corrugations across the wind direction	0.02
Smooth surfaces without corrugations or ribs or with corrugations or ribs parallel to the wind direction	0.01

D3 FREE ROOFS AND CANOPIES

D3.1 Aerodynamic shape factor for net pressure on free roofs

The aerodynamic shape factor (C_{fig}) for calculating net pressures normal to free roofs of monoslope, pitched or troughed configuration shall be as follows:

$$C_{fig} = C_{p,n}\, K_a\, K_\ell \qquad \ldots \text{D3}$$

where

$C_{p,n}$ = net pressure coefficient acting normal to the surface, obtained for the windward half of a free roof ($C_{p,w}$) or net pressure coefficient for the leeward half of a free roof ($C_{p,l}$), as given in Tables D4 to D7 for roofs within the geometrical limits given (positive indicates net downward pressure)

K_a = area reduction factor, as given in Paragraph D1.2

K_ℓ = local pressure factor, as given in Paragraph D1.3

NOTE: The factor K_p does not appear in this equation as it is taken as 1.0.

For free roofs of low pitch with fascia panels, the fascia panel shall be treated as the wall of an elevated building, and the C_{fig} found from Clause 5.4.

In Tables D4, D5, D6 and D7, 'empty under' implies that any goods or materials stored under the roof, block less than 50% of the cross-section exposed to the wind. 'Blocked under' implies that goods or materials stored under the roof block more than 75% of the cross-section exposed to the wind.

To obtain intermediate values of blockage and roof slopes other than those shown, use linear interpolation. Interpolation shall be carried out only between values of the same sign. Where no value of the same sign is given, for interpolation purposes 0.0 shall be assumed.

Where alternative pressure coefficient values are listed in Tables D4(A), D4(B), D5 and D6, for the appropriate roof slope, blockage and wind direction, all combinations of values $C_{p,w}$ and $C_{p,l}$ shall be considered.

For $\theta = 90°$, with $0.25 \leq h/d \leq 1$ the roof pitch is effectively zero, and Table D4(A) with $\alpha = 0°$ shall be used to determine $C_{p,n}$.

TABLE D4(A)

NET PRESSURE COEFFICIENTS ($C_{p,n}$) FOR MONOSLOPE FREE ROOFS—0.25 ≤ h/d ≤ 1 (see Figure D2)

Roof pitch (α) degrees	θ = 0 degrees				θ = 180 degrees			
	$C_{p,w}$		$C_{p,\ell}$		$C_{p,w}$		$C_{p,\ell}$	
	Empty under	Blocked under	Empty under	Blocked under	Empty under	Blocked under	Empty under	Blocked under
0	−0.3, 0.4	−1.0, 0.4	−0.4, 0.0	−0.8, 0.4	−0.3, 0.4	−1.0, 0.4	−0.4, 0.0	−0.8, 0.4
15	−1.0	−1.5	−0.6, 0.0	−1.0, 0.2	0.8	0.8	0.4	−0.2
30	−2.2	−2.7	−1.1, −0.2	−1.3, 0.0	1.6	1.6	0.8	0.0

TABLE D4(B)

NET PRESSURE COEFFICIENTS ($C_{p,n}$) FOR MONOSLOPE FREE ROOFS—0.05 ≤ h/d < 0.25 (see Figure D2)

Conditions	h/d	Horizontal distance (x) from windward edge	Net pressure coefficients ($C_{p,n}$)
For $\alpha \leq 5°$, or For all α with θ = 90°	$0.05 \leq h/d < 0.25$	$x \leq 1h$	Values given for $C_{p,w}$ in Table D4(A), for α = 0°
		$1h < x \leq 2h$	Values given for $C_{p,\ell}$ in Table D4(A), for α = 0°
		$x > 2h$	−0.2, 0.2 for empty under −0.4, 0.2 for blocked under

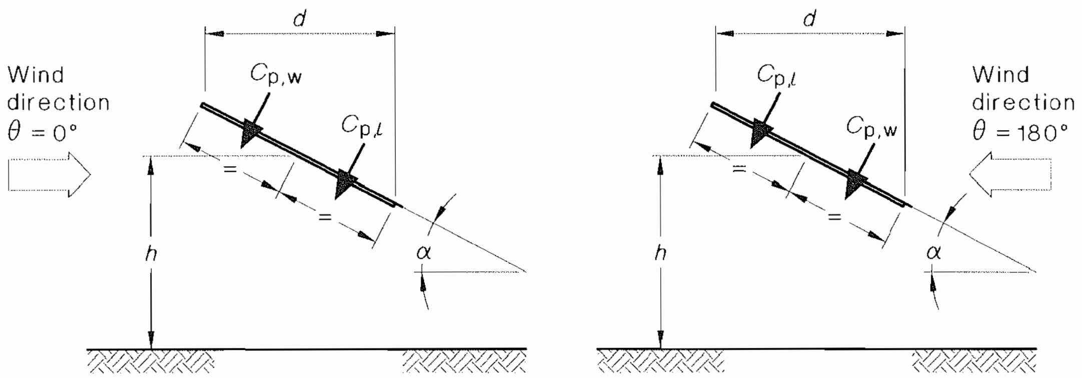


FIGURE D2 MONOSLOPE FREE ROOFS

TABLE D5

NET PRESSURE COEFFICIENTS ($C_{p,n}$) FOR PITCHED FREE ROOFS—0.25 ≤ h/d ≤ 1 (see Figure D3)

Roof pitch (α) degrees	$\theta = 0°$			
	$C_{p,w}$		$C_{p,\ell}$	
	Empty under	Blocked under	Empty under	Blocked under
≤15	−0.3, 0.4	−1.2	−0.4, 0.0	−0.9
22.5	−0.3, 0.6	−0.9	−0.6, 0.0	−1.1
30	−0.3, 0.8	−0.5	−0.7, 0.0	−1.3

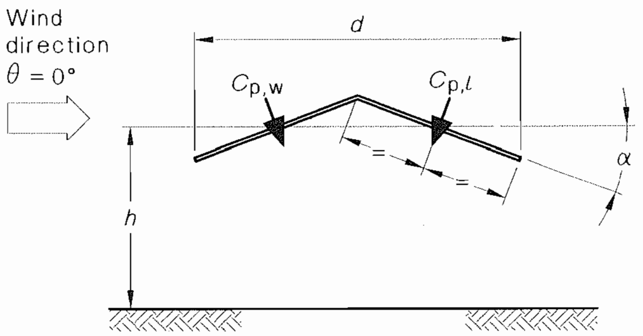


FIGURE D3 PITCHED FREE ROOFS

TABLE D6

NET PRESSURE COEFFICIENTS ($C_{p,n}$) FOR TROUGHED FREE ROOFS—0.25 ≤ h/d ≤ 1 (see Figure D4)

Roof pitch (α) degrees	$\theta = 0°$			
	$C_{p,w}$		$C_{p,\ell}$	
	Empty under	Blocked under	Empty under	Blocked under
7.5	−0.6, 0.4	−0.7	0.3	−0.3
15	−0.6, 0.4	−0.8	0.5	−0.2
22.5	−0.7, 0.3	−1.0	0.7	−0.2

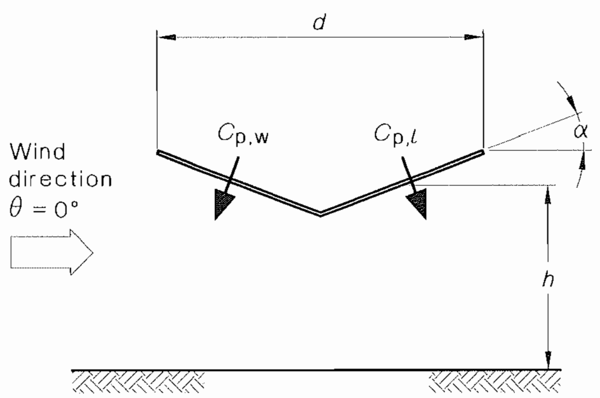


FIGURE D4 TROUGHED FREE ROOFS

TABLE D7

NET PRESSURE COEFFICIENTS ($C_{p,n}$) FOR HYPAR FREE ROOFS—EMPTY UNDER (see Figure D5)

Conditions	θ, degrees	$C_{p,w}$	$C_{p,\ell}$
Empty under,	0	+0.45	+0.25
$0.25 < h/d < 0.5$,		−0.45	−0.25
$0.1 < c/d < 0.3$, and	90	+0.45	+0.25
$0.75 < b/d < 1.25$		−0.45	−0.25

NOTE: $C_{p,n}$ is defined as positive downwards, and only combinations of values of the same sign need to be considered.

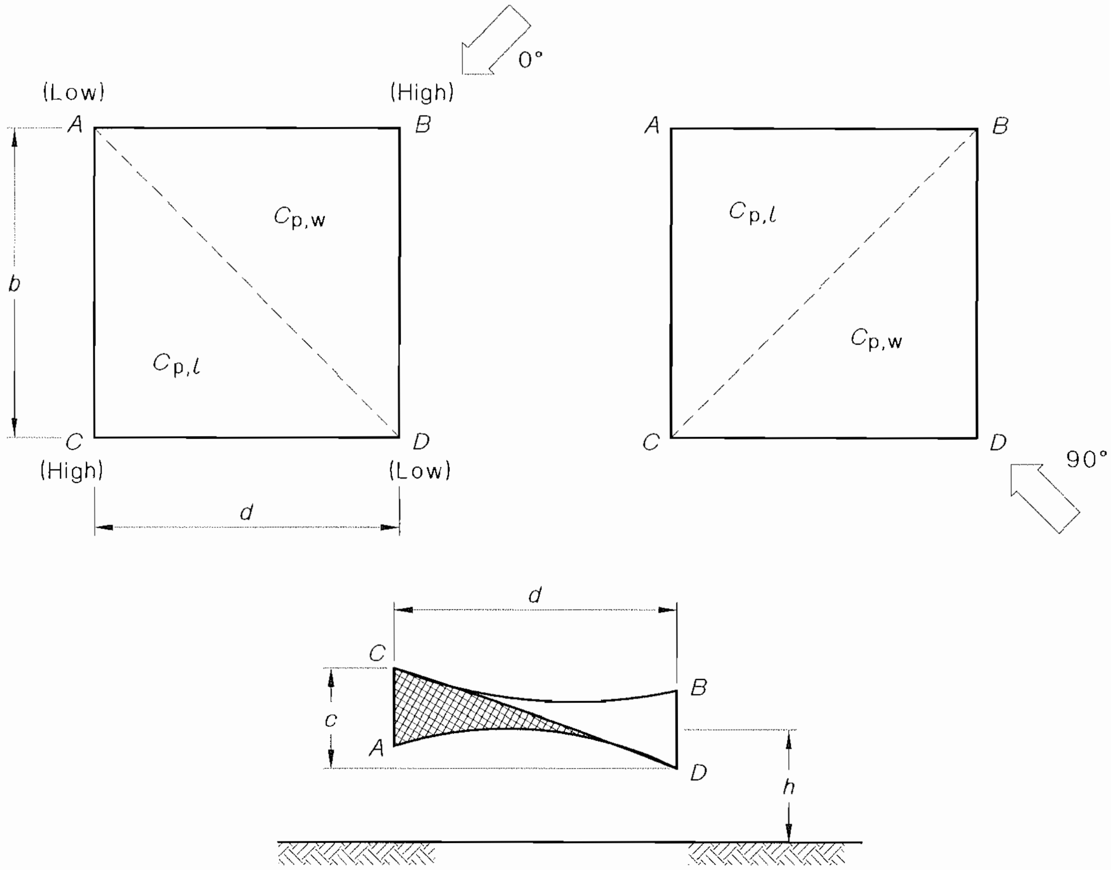


FIGURE D5 HYPERBOLIC PARABOLOID (HYPAR) ROOFS

D3.2 Aerodynamic shape factor for frictional drag and drag on exposed members for free roofs

The aerodynamic shape factor (C_{fig}) for calculating frictional drag on free roofs of monoslope, pitched or troughed configuration shall be equal to C_f calculated as given in Table D3. For free roofs, the frictional drag on both upper and lower surfaces shall be calculated and added to the drag on any exposed members calculated in accordance with Appendix E (see Clause 2.5).

Calculation of frictional drag pressure is not required for wind directions of 0° or 180°, as shown in Figures D2, D3 and D4, for free roofs with pitches of 10° or more.

D4 ATTACHED CANOPIES, AWNINGS AND CARPORTS (ROOFS)

D4.1 Aerodynamic shape factor for net pressure on attached canopies

The aerodynamic shape factor (C_{fig}) for calculating net pressures normal to the roof on canopies, awnings or carports adjacent to enclosed buildings and with a roof slope of 10° or less shall be calculated as follows:

$$C_{fig} = C_{p,n} K_a K_\ell \qquad \ldots \text{D4}$$

where

$C_{p,n}$ = net pressure coefficient acting normal to the surface, as given in Tables D8 and D9

K_a = area reduction factor, as given in Paragraph D1.2

K_ℓ = local pressure factor, as given in Paragraph D1.3

NOTES:

1 The values given for $C_{p,n}$ assume that any goods and materials stored under the canopy do not represent more than a 75% blockage.

2 The factor K_p does not appear in this equation as it is taken as 1.0.

Where indicated, attached canopies, awnings or carports shall be designed for both downward (positive) and upward (negative) net wind pressures.

For wind directions normal to the attached wall (θ = 0 degrees) for canopies and awnings, $C_{p,n}$ shall be taken from Tables D8 or D9 with reference to Figure D6. All pressure coefficients shall be used with the value of wind speed applying at average roof height (h) and h_c is the average height of the canopy above ground.

For wind directions parallel to the wall of the attached building (θ = 90° or 270°), the canopy or awning shall be considered as a free roof and the net pressure coefficients ($C_{p,n}$) shall be obtained in accordance with Table D4(A) or D4(B) or, where the canopy is partially enclosed, from Table D9.

TABLE D8

NET PRESSURE COEFFICIENTS ($C_{p,n}$) FOR CANOPIES AND AWNINGS ATTACHED TO BUILDINGS FOR $\theta = 0°$ (see Figure D6(a))

Design case	h_c/h (see Note 1)	Net pressure coefficients ($C_{p,n}$)
$h_c/h < 0.5$	0.1	1.2, −0.2
	0.2	0.7, −0.2
	0.5	0.4, −0.2
$h_c/h \geq 0.5$	0.5	0.5, −0.3
	0.75	0.4, [$-0.3 - 0.2(h_c/w_c)$] or −1.5 (see Note 2)
	1.0	0.2, [$-0.3 - 0.6(h_c/w_c)$] or −1.5 (see Note 2)

NOTES:

1 For intermediate values of h_c/h, linear interpolation shall be used.

2 Whichever is the lower magnitude.

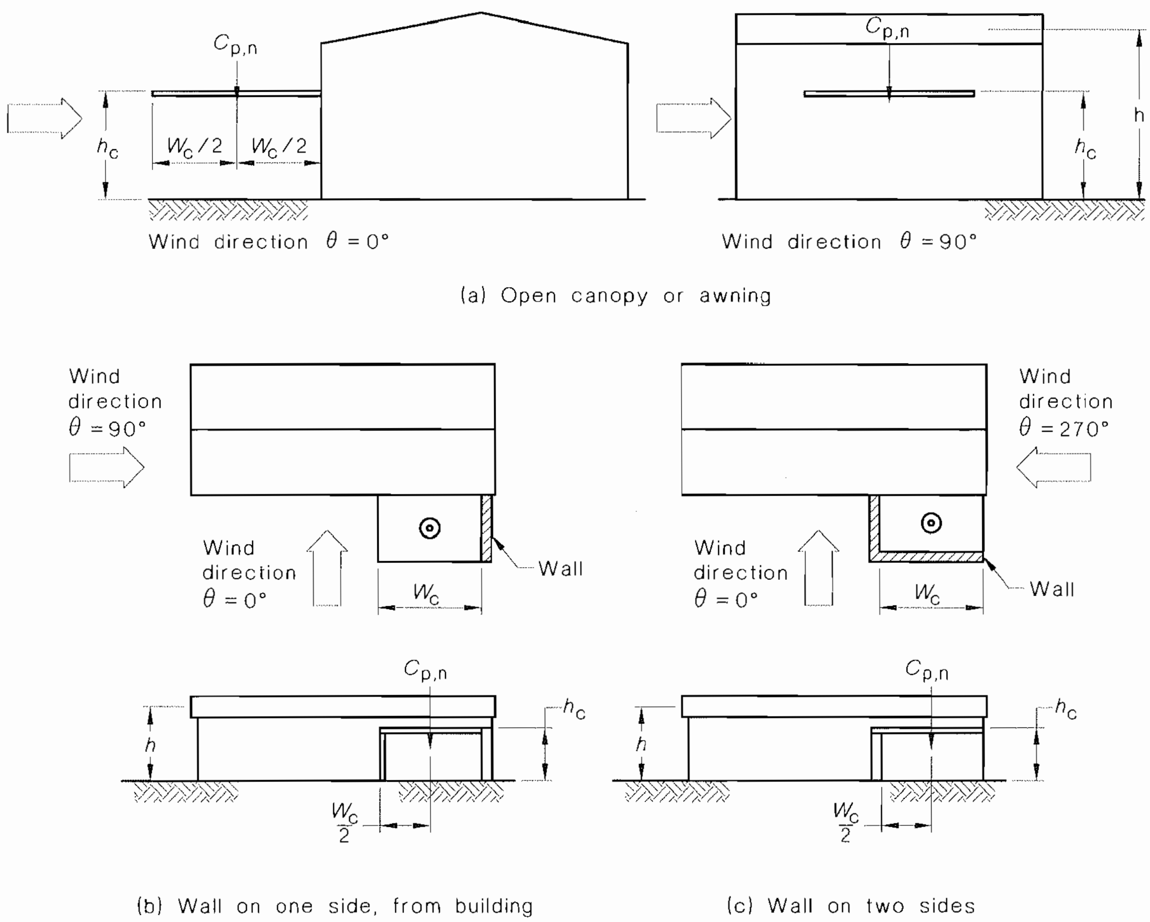


FIGURE D6 NET PRESSURE COEFFICIENTS ($C_{p,n}$) FOR CANOPIES, AWNINGS AND CARPORTS ATTACHED TO BUILDINGS

TABLE D9

NET PRESSURE COEFFICIENTS ($C_{p,n}$) FOR PARTIALLY ENCLOSED CARPORTS (see Figures D6(b) and D6(c))

Conditions	Partially enclosed	Wind direction (θ), degrees	Net pressure coefficients ($C_{p,n}$)
$h_c/w_c \leq 0.5$ and $h_c/h < 0.8$	Wall on one side attached to building, see Figure D6(b)	0 90	−0.7 −1.0
	Wall on two sides, see Figure D6(c)	0 270	−0.6 −1.2

D4.2 Aerodynamic shape factor for frictional drag and drag on exposed members of attached canopies

The aerodynamic shape factor (C_{fig}) for calculating frictional drag effects on attached canopies, awnings or carport roofs, where the wind is parallel to the attached wall, shall be equal to C_f as given in Table D3. For canopies, the frictional drag on both upper and lower surfaces shall be calculated and added to the drag on any exposed members calculated in accordance with Appendix E (see Clause 2.5).

D5 CANTILEVERED ROOFS

For an isolated cantilever roof with no interference from upstream structures within six roof heights, the aerodynamic shape factors ($C_{fig,1}$, $C_{fig,2}$) for structural loading of main supporting members is given in Table D10, with reference to Figure D7.

TABLE D10

AERODYNAMIC SHAPE FACTOR FOR ISOLATED CANTILEVER ROOFS WITH ROOF PITCH OF $-7° < \alpha < 7°$ AND WHERE $\theta = 0°$

		Height/span h/d ≤ 1.4		Height/span h/d > 1.4	
Load direction	Bay position	$C_{fig,1}$	$C_{fig,2}$	$C_{fig,1}$	$C_{fig,2}$
Upward loading (−)	Internal	−1.8	−1.1	−1.4	−1.4
	End	−1.3	−1.0	−1.9	−1.1
Downward loading (+)	Internal	0.25	0.15	0.20	−0.15
	End	0.55	0.65	0.20	0.0

Use Table D4(B) for $\theta = 90°$ for blocked under and $\alpha = 0°$.

Use Table D4(A) for $\theta = 180°$ for blocked under and $\alpha = 0°$.

NOTES:

1 For cladding loads on roofing elements, Paragraph D3 should be used, assuming blocked under.

2 Wind tunnel testing or similar studies should be carried out if there is a similar height grandstand roof within six roof heights of the cantilevered roof in question.

Dynamic response shall be taken into account by determining the dynamic response factor (C_{dyn}) as follows:

(a) For cases where cantilevered beams are greater than 15 m long, $\left(\frac{V_{des,\theta}}{1+g_v I_h}\right)\left(\frac{1}{n_1 d}\right) > 0.4$ and $n_1 < 1$ Hz;

$$C_{dyn} = \left(1.0 + 0.5\left[\left(\frac{V_{des,\theta}}{1+g_v I_h}\right)\left(\frac{1}{n_1 d}\right) - 0.4\right]\right) \qquad \text{... D5}$$

where

n_1 = first mode frequency of vibration of the cantilevered roof in the vertical bending mode

(b) For all other cases, C_{dyn} = 1.0.

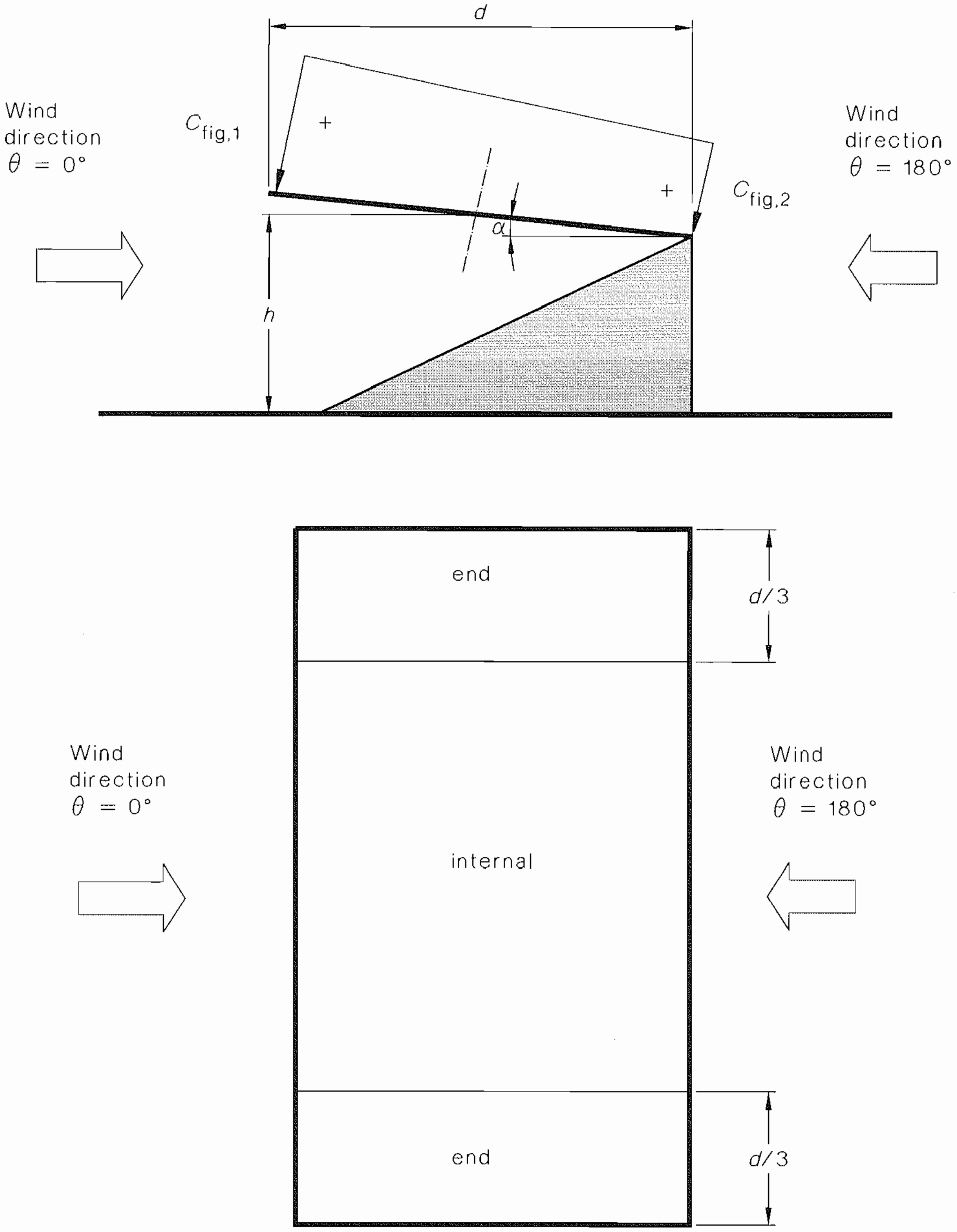


FIGURE D7 CANTILEVERED ROOF

APPENDIX E

AERODYNAMIC SHAPE FACTORS FOR EXPOSED STRUCTURAL MEMBERS, FRAMES AND LATTICE TOWERS

(Normative)

E1 GENERAL

This Appendix shall be used to calculate aerodynamic shape factors (C_{fig}) for structures and components consisting of exposed members, such as lattice frames, trusses and towers.

All pressure coefficients shall be used with the value of wind speed applying at the height of the component being considered.

E2 AERODYNAMIC SHAPE FACTORS FOR INDIVIDUAL MEMBERS AND FRAMES

E2.1 Simple shapes and individual members

The aerodynamic shape factor (C_{fig}) for individual exposed structural members, with an aspect ratio (l/b) greater than 8, shall be calculated as follows:

(a) For wind axes:

$$C_{fig} = K_{ar} K_i C_d \quad \text{. . . E2(1)}$$

(b) For body axes:

$$C_{fig} = K_{ar} K_i C_{F,x} \text{ along member's } x\text{-axis (major axis)} \quad \text{. . . E2(2)}$$

$$C_{fig} = K_{ar} K_i C_{F,y} \text{ along member's } y\text{-axis (minor axis)} \quad \text{. . . E2(3)}$$

where

l = length of member

b = breadth of element, normal to the wind stream

K_{ar} = aspect ratio correction factor for individual member forces, as given in Table E1

K_i = factor to account for the angle of inclination of the axis of members to the wind direction, determined as follows:

= 1.0, when the wind is normal to the member

= $\sin^2\theta_m$ for rounded cylindrical shapes

= $\sin\theta_m$ for sharp-edged prisms, (sharp-edged prisms are those with b/r greater than 16)

θ_m = angle between the wind direction and the longitudinal axis of the member, in degrees

r = corner radius of a structural shape

C_d = drag force coefficient for a structure or member in the direction of the wind stream, as given in Paragraph E3

$C_{F,x}$ and $C_{F,y}$ = drag force coefficients for a structure or member, in the direction of the x- and y-axes respectively, as given in Paragraph E3

TABLE E1

ASPECT RATIO CORRECTION FACTORS (K_{ar})

Aspect ratio, l/b (see Note)	Correction factor K_{ar}
≤ 8	0.7
14	0.8
30	0.9
40 or more	1.0

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

E2.2 Single open frame

The aerodynamic shape factor (C_{fig}) for a structure of open frame type, comprising a number of members where the members are sharp-edged rectangular or structural sections, lying in a single plane normal to the wind direction (see Figure E1), shall be taken as follows:

(a) For $0.2 < \delta_e < 0.8$ and $1/3 < (l/b) < 3$ (where l/b is the aspect ratio of the whole frame).

$$C_{fig} = 1.2 + 0.26\,(1 - \delta_e) \qquad \text{. . . E2(4)}$$

The reference area, A_{ref}, to be used in Equation E2(4) for an open frame shall be taken as the sum of the projected areas of all the members projected normal to the plane of the frame.

(b) For all other cases, wind action shall be the sum of the effects calculated on individual members and attachments determined in accordance with Clause 2.5.3.3 and Paragraph E2.1

where

δ_e = effective solidity ratio for an open frame, given as follows:

= δ for flat-sided members

= $1.2\delta^{1.75}$ for circular cross-section members

where

δ = solidity ratio of the structure (surface or open frame), which is the ratio of solid area to total area of the structure

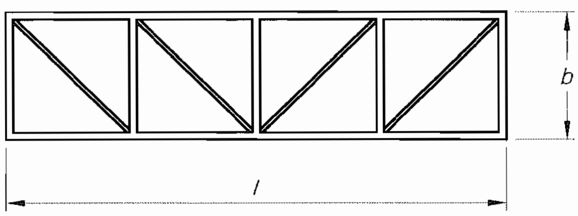


FIGURE E1 NOTATION FOR FRAME DIMENSIONS

E2.3 Multiple open frames

For structures comprising a series of similar open frames in parallel, the aerodynamic shape factors for the second and subsequent frames shall be taken as the aerodynamic shape factors on the windward frame calculated as in Paragraph E2.2, multiplied by the shielding factor (K_{sh}) obtained from Table E2. The aerodynamic shape factor (C_{fig}) for the structure shall be as follows:

$$C_{fig} = C_{fig,1} + \Sigma K_{sh} C_{fig,1} \quad \ldots \text{E2(5)}$$

where

$C_{fig,1}$ = aerodynamic shape factor for the first frame in the upwind direction, as given in Paragraph E2.2

K_{sh} = shielding factor for shielded frames in multiple open-framed structures, as given in Table E2

λ = spacing ratio for parallel open frames, equal to the frame spacing (centre-to-centre) divided by the smaller of l or b.

TABLE E2

SHIELDING FACTORS (K_{sh}) FOR MULTIPLE FRAMES

Angle of wind to frames (θ), degrees	Frame spacing ratio (λ)	Shielding factors (K_{sh})							
		Effective solidity (δ_e)							
		0	0.1	0.2	0.3	0.4	0.5	0.7	1.0
0 (wind normal to frames)	≤0.2	1.0	0.8	0.5	0.3	0.2	0.2	0.2	0.2
	0.5	1.0	1.0	0.8	0.6	0.4	0.2	0.2	0.2
	1.0	1.0	1.0	0.8	0.7	0.5	0.3	0.2	0.2
	2.0	1.0	1.0	0.9	0.7	0.6	0.4	0.2	0.2
	4.0	1.0	1.0	1.0	0.8	0.7	0.6	0.4	0.2
	≥8.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
45	≤0.5	1.0	0.9	0.8	0.7	0.6	0.5	0.3	0.3
	1.0	1.0	1.0	0.9	0.8	0.7	0.6	0.6	0.6
	2.0	1.0	1.0	1.0	1.0	1.0	0.9	0.8	0.6
	4.0	1.0	1.0	0.9	0.7	0.6	0.4	0.2	0.2
	≥8.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0

NOTE: For intermediate values of δ_e and λ, linear interpolation shall be used.

E3 DRAG FACTORS FOR STRUCTURAL MEMBERS AND SIMPLE SECTIONS

E3.1 Rounded cylindrical shapes, sharp-edged prisms and structural sections

Values of drag force coefficients (C_d, $C_{F,x}$ and $C_{F,y}$) for rounded cylindrical shapes, sharp-edged prisms and some structural sections shall be as given in Tables E3, E4 and E5 respectively.

Table E4 gives values for the most common polygonal sharp-edged cross-sections except for rectangular prisms that are covered separately in Paragraph E3.2.

NOTES:

1 Drag force coefficients of sharp-edged cross-sections are independent of the Reynolds number.

2 Note that in Table E5, the dimension b, used in the definition of the force coefficients, is not always normal to the flow direction, and d is not always parallel.

In the absence of experimental information for particular cables, C_d for helically wound, unwrapped cables shall be as follows:

(a) 1.2 for $bV_{des,\theta} < 0.5 m^2/s$.

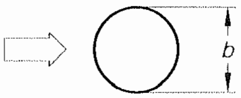
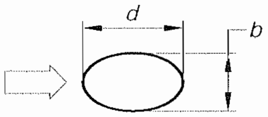
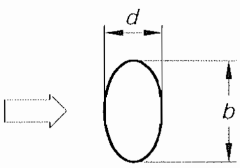
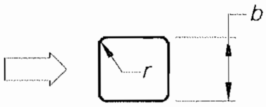
(b) 1.0 for $bV_{des,\theta} > 5.0 m^2/s$.

For values of $bV_{des,\theta}$ between 0.5 and 5.0, use interpolation.

Where icing of cables is considered, the increased cross-sectional area and changed shape shall be taken into account.

TABLE E3

DRAG FORCE COEFFICIENTS (C_d) FOR ROUNDED CYLINDRICAL SHAPES

Cross-sectional shape	Description	Drag force coefficient (C_d) (see Note 1)	
		$bV_{des,\theta} < 4 m^2/s$	$bV_{des,\theta} > 10 m^2/s$
	Cylindrical	1.2	(see Note 2)
 Ellipse $\frac{b}{d}=\frac{1}{2}$	Ellipse narrow side to wind	0.7	0.3
 Ellipse $\frac{b}{d}=2$	Ellipse broad side to wind	1.7	1.5
 $\frac{b}{d}=1$ $\frac{r}{b}=\frac{1}{3}$	Square with rounded corners	1.2	0.6

NOTES:

1 For intermediate values of $bV_{des,\theta}$, linear interpolation shall be used.

2 For smooth circular cross-sections and polygonal sections with more than 16 sides, where $bV_{des,\theta} > 10 m^2/s$, C_d shall be as follows:

$C_d = 0.5$ for $h_r/b \leq 0.00002$

$C_d = 1.6 + 0.105 \log_e (h_r/b)$ for $h_r/b > 0.00002$

where

h_r = average height of surface roughness

Some typical values for h_r in millimetres are as follows:

Glass, plastic: 0.0015

Steel: galvanized 0.15; light rust 2.5; heavy rust 15

Concrete, new smooth 0.06; new rough: 1.0

Metal, painted: 0.03

3 Attachments to circular cross-sections (e.g. ladders, pipes etc.) projecting more than 1% of the diameter of the cylinder will induce aerodynamic separation and in these cases C_d = 1.2.

4 Due consideration shall be taken of the projected area and drag of the attachments themselves.

TABLE E4

DRAG FORCE COEFFICIENT (C_d) FOR SHARP-EDGED PRISMS

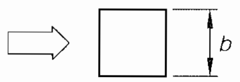
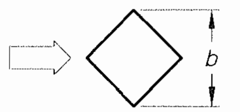
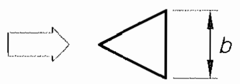
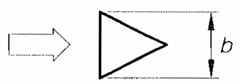
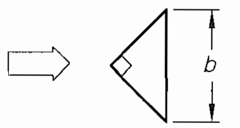
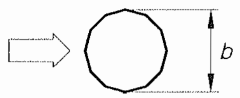
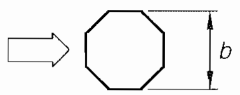
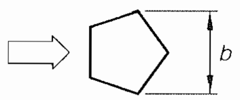
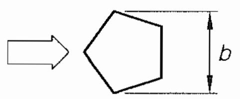
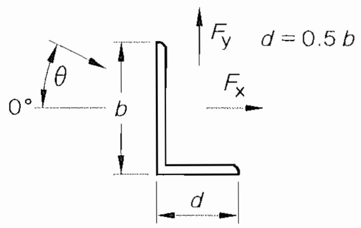
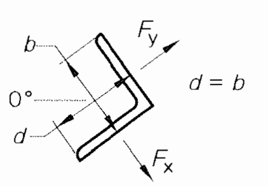
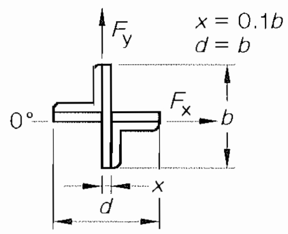
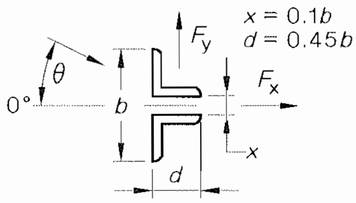
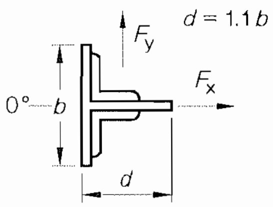
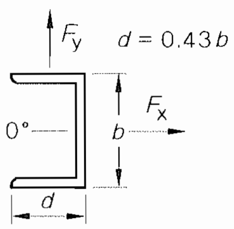
Cross-sectional shape		Drag force coefficient (C_d)
	Square with face to wind	2.2
	Square with corner to wind	1.5
	Equilateral triangle—apex to wind	1.2
	Equilateral triangle—face to wind	2.0
	Right-angled triangle	1.55
	12-sided polygon	1.3
	Octagon	1.4
	Pentagon with face to wind	1.1
	Pentagon with corner to wind	1.7

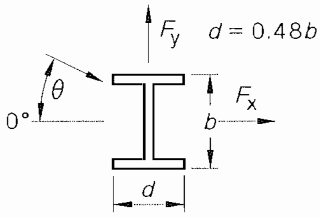
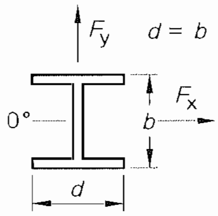
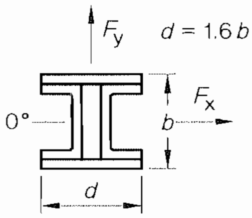
TABLE E5

FORCE COEFFICIENTS ($C_{F,x}$) AND ($C_{F,y}$) FOR STRUCTURAL SECTIONS

Section shape		Wind direction measured clockwise (θ)				
		0	45	90	135	180
	$C_{F,x}$	2.0	1.8	−2.0	−1.8	−1.9
	$C_{F,y}$	−0.1	0.1	−1.7	−0.8	−0.95
	$C_{F,x}$	1.8	1.8	−1.0	0.3	−1.4
	$C_{F,y}$	1.8	2.1	−1.9	−2.0	−1.4
	$C_{F,x}$	1.75	0.75	−0.1	−0.85	−1.75
	$C_{F,y}$	0.1	−0.75	−1.75	−0.85	−0.1
	$C_{F,x}$	1.6	1.5	−0.95	−0.5	−1.5
	$C_{F,y}$	0	0.1	−0.7	−1.05	0
	$C_{F,x}$	2.0	1.2	−1.6	−1.1	−1.7
	$C_{F,y}$	0	−0.9	−2.15	−2.4	±2.1
	$C_{F,x}$	2.05	1.85	0	−1.6	−1.8
	$C_{F,y}$	0	−0.6	−0.6	−0.4	0

(continued)

TABLE E5 *(continued)*

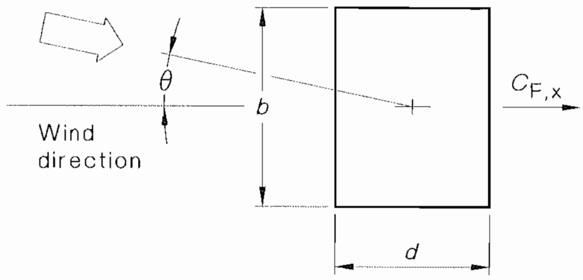
Section shape		Wind direction measured clockwise (θ)				
		0	45	90	135	180
	$C_{F,x}$	2.05	1.95	±0.5	—	—
	$C_{F,y}$	0	−0.6	−0.9	—	—
	$C_{F,x}$	1.6	1.5	0	—	—
	$C_{F,y}$	0	−1.5	−1.9	—	—
	$C_{F,x}$	1.4	1.2	0	—	—
	$C_{F,y}$	0	−1.6	−2.2	—	—

NOTE: Note that the direction of θ has been changed to clockwise and the values transposed accordingly, to align with the clockwise direction used elsewhere in the Standard. Also, dimension b, used in the definitions of the force coefficients, is not always normal to the wind direction.

E3.2 Rectangular prismatic sections

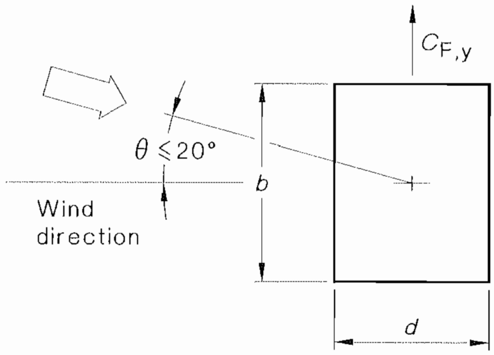
Values of force coefficients ($C_{F,x}$ and $C_{F,y}$) for rectangular prismatic cross-sections are given in Figures E2(A) and E2(B). This Paragraph does not cover the case where the wind direction angle θ is greater than 20°. For intermediate values of d/b, use linear interpolation.

NOTE: Figure E2(B) contains maximum values of $C_{F,y}$ for angles within 20° of the directions parallel to the faces of the rectangle. Fluctuations in wind direction of up to 20° may occur in turbulent flow nominally parallel to one face.



Aspect ratio **d/b**	Force coefficient **($C_{F,x}$)**	Multiplying factor for $\theta \leq 15°$
0.1 0.65 1	2.2 3.0 2.2	1.0
2 4 ≥10	1.6 1.3 1.1	[1+ $(d/b)\tan\theta$]

FIGURE E2(A) ALONG-WIND COEFFICIENTS FOR RECTANGULAR PRISMS



Aspect ratio **d/b**	Force coefficient **($C_{F,y}$)**
0.5 1.5 2.5	±1.2 ±0.8 ±0.6
4 ≥20	±0.8 ±1.0

FIGURE E2(B) CROSSWIND COEFFICIENTS FOR RECTANGULAR PRISMS

E4 LATTICE TOWERS

E4.1 General

Lattice towers shall be divided vertically into a series of sections (levels) and the aerodynamic shape factors (C_{fig}) shall be calculated for each section.

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

They shall be designed for winds in eight directions with $V_{des,\theta}$ being the value of $V_{sit,\beta}$ in a sector ±22.5° from the 45° direction being considered.

The aerodynamic shape factor (C_{fig}) shall be equal to the values calculated, as follows:

(a) C_d for a tower section without ancillaries, as given in Paragraph E4.2.1.

(b) C_{de}, for a tower section with ancillaries, as given in Paragraph E4.2.2.

(c) $1.2 \sin^2\theta_m$, for guy cables, using the wind speed calculated for 2/3 of the height of the cable

where

C_{de} = effective drag force coefficient for a tower section with ancillaries

θ_m = angle between the wind direction and the longitudinal axis of the member, in degrees

E4.2 Drag force coefficient

E4.2.1 *Tower sections without ancillaries*

The drag force coefficients (C_d) for complete lattice tower sections shall be taken from Tables E6(A) to E6(C).

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 given in Tables E3 and E4, and Figure E2 for the appropriate tower section shapes.

For UHF antenna sections, C_d shall be obtained from Table E7 and Figure E3. To calculate the area for the application of the pressure, breadth shall be taken as b_D or b_N, as appropriate to the wind direction.

Where used, the reduction for aspect ratio shall be carried out by multiplying by the correction factor (K_{ar}), given in Table E1, taking l as equal to two times the height of the end-mounted antennas.

TABLE E6(A)

DRAG FORCE COEFFICENTS (C_d) FOR LATTICE TOWERS—SQUARE AND EQUILATERAL TRIANGLE IN PLAN WITH FLAT-SIDED MEMBERS

Solidity of front face (δ)	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 E6(B)

DRAG FORCE COEFFICENTS (C_d) FOR LATTICE TOWERS— SQUARE PLAN WITH CIRCULAR MEMBERS

Solidity of front face (δ)	Parts of tower in sub-critical flow $b_i V_{des,\theta} < 3$ m²/s		Parts of tower in super-critical flow $b_i V_{des,\theta} \geq 6$ 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 E6(C)

DRAG FORCE COEFFICENTS (C_d) FOR LATTICE TOWERS— EQUILATERAL TRIANGLE PLAN WITH CIRCULAR MEMBERS

Solidity of front face (δ)	Parts of tower in sub-critical flow $b_i V_{des,\theta} < 3$ m²/s (all wind directions)	Parts of tower in super-critical flow $b_i V_{des,\theta} \geq 6$ 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

NOTES TO TABLES E6(A) to E6(C):

1 A_z = area of members in one face projected horizontally normal to the face (this area does not change with wind direction). This is the reference area for the drag coefficients in Tables E6(A), E6(B) and E6(C) in the application of Equation 2.5(3).

2 δ = solidity ratio of the structure (surface or open frame), that is the ratio of the area A_z as defined in Note 1, to the total projected area enclosed over the section height by the boundaries of the frame. For intermediate values of solidity, linear interpolation shall be used.

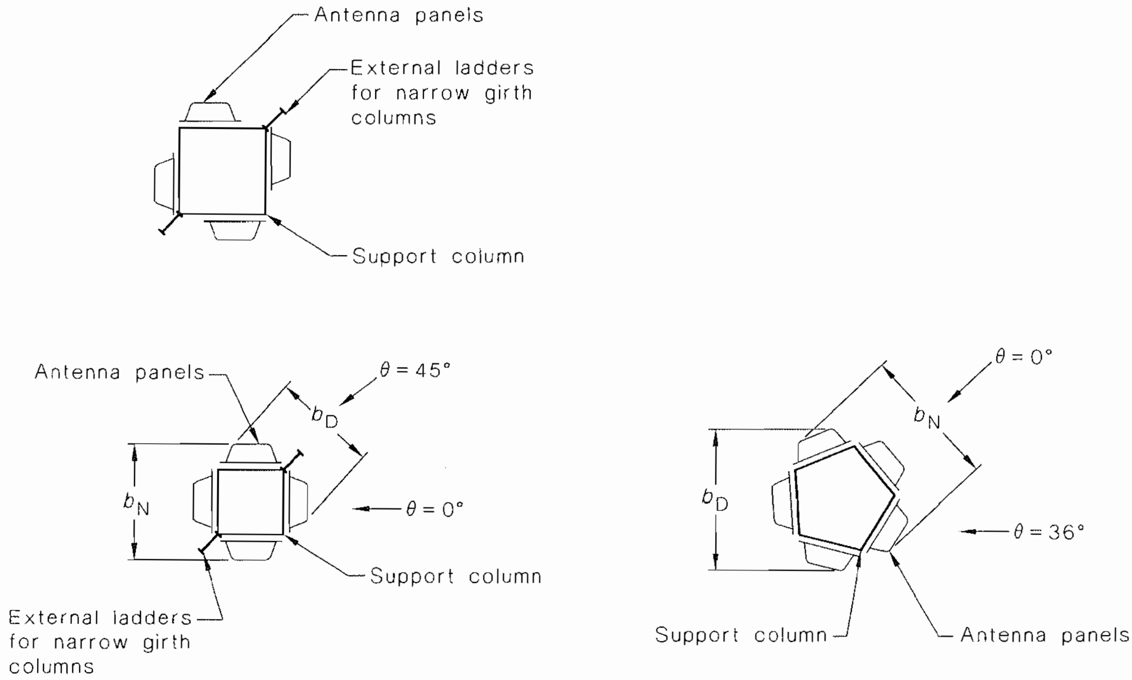
3 b_i = average diameter or breadth of a section of a tower member.

4 In Tables E6(B) and E6(C), linear interpolation shall be used for values of $b_i V_{des,\theta}$ between 3 and 6.

TABLE E7

DRAG FORCE COEFFICIENT (C_d) FOR UHF ANTENNA SECTIONS (see Figure E3)

Antenna type	Wind direction (θ) degrees	Drag force coefficient (C_d)
1 (4 sided)	0, 45	1.4
2 (5 sided)	0	1.5
2 (5 sided)	36	1.3



(a) Antenna Type 1
Square support column

(b) Antenna Type 2
Pentagonal support column

Plan view

NOTES:

1 To calculate the area (A_z) in Equation 2.5(3), breadth (b_D) or (b_N), shall be used, as appropriate to the wind direction.

2 Reduction for aspect ratio may be carried out by multiplying by the correction factor (K_{ar}) given in Table E1, taking l as equal to two times the height of the end-mounted antennas.

FIGURE E3 DRAG FORCE COEFFICIENTS (C_d) FOR SECTION OF UHF ANTENNAS

E4.2.2 *Tower sections with ancillaries*

The effective drag force coefficient (C_{de}) for a tower section with ancillaries shall be calculated as follows:

(a) Where ancillaries are attached symmetrically to all faces, their projected area shall be added to the projected area of the tower members (A_z).

(b) Where ancillaries are not symmetrically placed, the total effective drag force coefficient (C_{de}) for a tower section shall be taken as follows:

$$C_{de} = C_d + \Sigma \Delta C_d \qquad \ldots \text{E4(1)}$$

where

ΔC_d = additional drag coefficient due to an ancillary attached to one face or located inside the tower section:

$$= C_{da} K_{ar} K_{in} (A_a/A_z) \qquad \ldots \text{E4(2)}$$

where

C_{da} = value of drag force coefficient (C_d) on an isolated ancillary on a tower, as given in Tables E3 and E4 and Figure E2

K_{ar} = aspect ratio correction factor for individual member forces

= as given in Table E1, for linear ancillaries with aspect ratios less than 40

= 1.0, for all other cases

K_{in} = correction factor for interference, as given in Paragraph E4.2.3

A_a = reference area of ancillaries on a tower

= lb

where

l = the length of the linear ancillary and b is defined in Figure E4 and Tables E3 and E4

$A_{z,s}$ = total projected area of the tower section at height z

E4.2.3 *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:

(i) To the face of a square tower [see Figure E4(a)]:

$$K_{in} = [1.5 + 0.5\cos 2(\theta_a - 90°)] \exp [-1.2(C_d\delta)^2] \qquad \text{... E4(3)}$$

(ii) To the face of a triangular tower [see Figure E4(b)]:

$$K_{in} = [1.5 + 0.5\cos 2(\theta_a - 90°)] \exp [-1.8(C_d\delta)^2] \qquad \text{... E4(4)}$$

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

(i) Inside a square tower [see Figure E4(c)]:

$$K_{in} = \exp [-1.4 (C_d\delta)^{1.5}] \qquad \text{... E4(5)}$$

(ii) Inside a triangular tower [see Figure E4(d)]:

$$K_{in} = \exp [-1.8(C_d\delta)^{1.5}] \qquad \text{... E4(6)}$$

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

(i) Inside a square tower [see Figure E4(e)]:

$$K_{in} = \exp [-a(C_d\delta)^{1.5}] \qquad \text{... E4(7)}$$

$$a = 2.7 - 1.3 \exp [-3(b/w)^2] \qquad \text{... E4(8)}$$

(ii) Inside a triangular tower [see Figure E4(f)]:

$$K_{in} = \exp\left[-c(C_d\delta)^{1.5}\right] \qquad \text{... E4(9)}$$

$$c = 6.8 - 5 \exp [-40(b/w)^3] \qquad \text{... E4(10)}$$

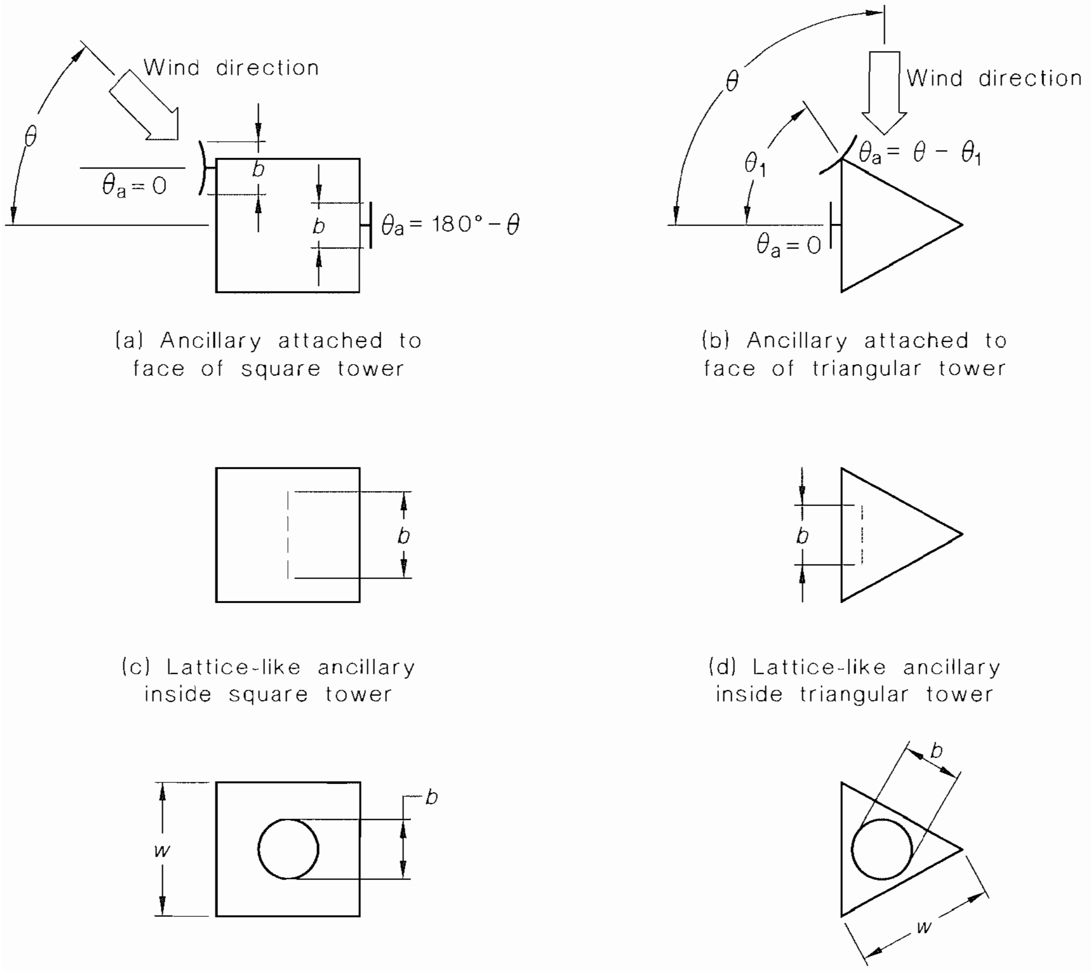
where

θ_a = angle of deviation of the wind stream from the line joining the centre of the tower cross-section to the centre of the ancillary, in degrees

δ = solidity ratio of the structure, as given in Paragraph E4.2.1

a, c = constants for ease of calculation

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



(a) Ancillary attached to face of square tower

(b) Ancillary attached to face of triangular tower

(c) Lattice-like ancillary inside square tower

(d) Lattice-like ancillary inside triangular tower

(e) Cylindrical ancillary inside square tower

(f) Cylindrical ancillary inside triangular tower

FIGURE E4 TOWER SECTIONS WITH ANCILLARIES

APPENDIX F

FLAGS AND CIRCULAR SHAPES

(Normative)

F1 GENERAL

This Appendix shall be used to calculate aerodynamic shape factors (C_{fig}) for drag forces on flags, discs and spherical shapes.

All pressure coefficients shall be used with the value of wind speed applying at the mid-height of the component being considered.

F2 FLAGS

The aerodynamic shape factor (C_{fig}) for flags is as follows:

(a) *Fixed flag*, shall be treated as elevated hoarding (see Appendix D).

(b) *Free flag (including dynamic effects from flutter)*;

$$C_{fig} = 0.05 + 0.7 \frac{m_f}{\rho_{air} c} \left(\frac{A_{ref}}{c^2} \right)^{-1.25} \text{, but not greater than 0.76} \qquad \text{. . . F1}$$

where

m_f = mass per unit area of flag, in kilograms per square metre

ρ_{air} = density of air which shall be taken as 1.2 kg/m^3

c = net height of flag (see Figure F1)

l_f = flag length (see Figure F1)

A_{ref} = reference area of flag, as given in Figure F1 (area of flag perpendicular to the wind direction)

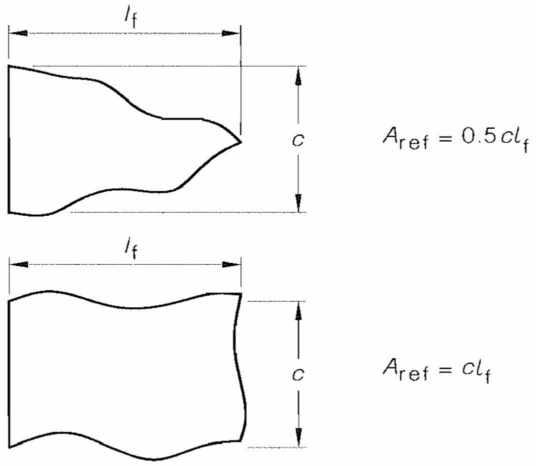
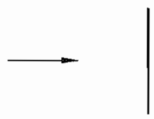
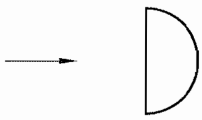
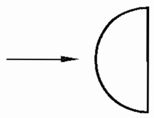
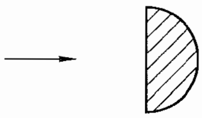
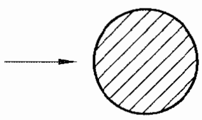


FIGURE F1 REFERENCE AREA FOR FLAGS

F3 CIRCULAR SHAPES

The aerodynamic shape factor (C_{fig}) for calculating drag forces on circular shapes shall be as given in Table F1.

TABLE F1
AERODYNAMIC SHAPE FACTOR FOR CIRCULAR SHAPES

Cross-sectional shape	Description of shape	Aerodynamic shape factor (C_{fig})
	Circular disc	1.3
	Hemispherical bowl (cup to wind)	1.4
	Hemispherical bowl	0.4
	Hemispherical solid (flat to wind)	1.2
	Spherical solid	0.5 for $bV_{des,\theta} < 7$ 0.2 for $bV_{des,\theta} \geq 7$

NOTE: The reference area A_{ref} for shapes in Table F1 shall be the projected area normal to the wind direction.

APPENDIX G

ACCELERATIONS FOR WIND-SENSITIVE STRUCTURES

(Informative)

G1 ACCELERATION FOR SERVICEABILITY

To provide some indication of motion serviceability it is noted that for wind-sensitive buildings, mostly exposed to free stream flow, acceptable crosswind acceleration levels may be exceeded if—

$$h^{1.3}/m_0 > 0.0016 \qquad \ldots \text{G1}$$

where

h = average roof height of a structure above the ground, in metres

m_0 = average mass per unit height

In conditions of high turbulence, due to interference from other buildings, a more conservative approach to the use of this indicator should be taken. Should the inequality indicate likely high acceleration levels then the designer should undertake more detailed analysis or wind tunnel model studies.

G2 PEAK ALONG-WIND ACCELERATION FOR SERVICEABILITY

The peak acceleration at the top of a structure in the along-wind direction ($\ddot{x}_{\max}$) in metres per second squared, is as follows:

$$\ddot{x}_{\max} = \frac{3}{m_0 h^2} \times \text{resonant component of peak base bending moment}$$

$$= \frac{3}{m_0 h^2} \frac{\rho_{\text{air}} g_{\text{R}} I_{\text{h}} \sqrt{\frac{S E_{\text{t}}}{\zeta}}}{\left(1+2 g_{\text{v}} I_{\text{h}}\right)} \left\{ C_{\text{fig, windward}} \sum_{z=0}^{h} \left[V_{\text{des},0}(z)\right]^2 b_z z \Delta z - C_{\text{fig, leeward}} \left[V_{\text{des},0}(h)\right]^2 \sum_{z=0}^{h} b_z z \Delta z \right\} \qquad \ldots \text{G2}$$

where

m_0 = average mass per unit height

ρ_{air} = density of air which, shall be taken as 1.2 kg/m^3

$V_{\text{des},0}(z)$ = building orthogonal design wind speeds as a function of height z

$V_{\text{des},0}(h)$ = building orthogonal design wind speeds evaluated at height h

b_z = average breadth of the structure at the section at height z

Δ_z = height of the section of the structure upon which the wind pressure acts

ζ = ratio of structural damping to critical damping of the structure

NOTE: Users should seek advice on possible values of damping as a function of height of the structure and amplitude of vibration.

G3 CROSSWIND ACCELERATION FOR SERVICEABILITY OF TALL BUILDINGS AND TOWERS OF RECTANGULAR CROSS-SECTION

G3.1 General

This Paragraph gives methods for determining peak accelerations at the top of tall enclosed buildings and towers of rectangular cross-section. Calculation of crosswind response is not required for porous lattice towers.

G3.2 Peak crosswind acceleration for serviceability

The peak acceleration in the crosswind direction $(\ddot{y}_{max})$ in metres/second squared, at the top of a structure with constant mass per unit height (m_0) should be determined as follows:

$$\ddot{y}_{max} = \frac{1.5 b g_R}{m_0}\left[\frac{0.5\rho_{air}\left[V_{des,\theta}\right]^2}{\left(1+g_v I_h\right)^2}\right] K_m \sqrt{\frac{\pi C_{fs}}{\zeta}} \quad \ldots \text{G3(1)}$$

where

b = breadth of a structure, normal to the wind stream

g_R = peak factor for resonant response (10 min period) given by:

$$= \sqrt{\left[2\log_e\left(600 n_c\right)\right]} \quad \ldots \text{G3(2)}$$

n_c = first mode natural frequency of vibration of a structure in the crosswind direction, in hertz

m_0 = average mass per unit height

g_v = peak factor for the upwind velocity fluctuations, which may be taken as 3.7

I_h = turbulence intensity, obtained from Table 6.1 by setting $z = h$

K_m = mode shape correction factor for crosswind acceleration, given by:

= $0.76 + 0.24k$

where

k = mode shape power exponent for the fundamental mode and values of the exponent k should be taken as:

= 1.5 for a uniform cantilever

= 0.5 for a slender framed structure (moment resisting)

= 1.0 for a building with central core and moment resisting façade

= 2.3 for a tower decreasing in stiffness with height, or with a large mass at the top

= the value obtained from fitting $\phi_1(z) = (z/h)^k$ to the computed modal shape of the structure

$\phi_1(z)$ = first mode shape as a function of height z, normalized to unity at $z = h$

C_{fs} = crosswind force spectrum coefficient generalized for a linear mode shape given in Clause 6.3.2.3

ζ = ratio of structural damping to critical damping of the structure.

NOTES