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Ref: 22233 22nd May 2024

Jocelyn Moorfoot
JCDecaux Australia & New Zealand
83 Main St, Kangaroo Point QLD 4169

RE: Existing Supersite Signage Parramatta Rd Overpass, Auburn NSW 2144
Comparison Of Design Codes With Current Codes.

1.0 Introduction

This assessment has been conducted by Dennis Bunt Consulting Engineers Pty Ltd (DBCE) at the request of JCDecaux. .

The purpose of this assessment was to review the design codes for the supersite signage at Parramatta Rd Overpass, Auburn, NSW which was designed in 2008, with today's current codes.

The two structural codes used for the design of the signage structure were the Steel Structures code AS4100:1998 and Structural Design Actions Part 2: Wind Actions AS1170.2 2002. The current codes are Steel Structures code AS4100 2020 and Structural Design Actions Part 2: Wind Actions AS1170.2 2021.

Reference is also made to the following documents:

- Industry Insights Steel Australia Spring 2000 pgs 16 and 17
- Wind Loading – History of Changes Aspec Engineering Pty Ltd, Brisbane, Australia
- Key-Changes-to-AS-NZS-1170.2-2021 by Chris Hackney. (Chris is a committee member of AS1170.2)

2.0 Discussion

AS4100 Steel structures code.

Referring to the document “Industry Insights Steel Australia Spring 2020 pg 16 and 17”

1. The primary reason for revising AS 4100:1998 was to reference AS/NZS 5131 Structural steelwork – Fabrication and erection.
2. There were changes to the definition and description of Definition and description of ‘architecturally exposed structural steel’ (AESS)
3. The new code addressed the likelihood of lamellar tearing in particular welded connections.

Item 1 refers to the recent development of a fabrication and erection code (AS5131) for structural steel. It brings Australia into line with other developed countries. It does not affect the structural design and hence the member, plate, and bolts sizes but the quality control of the fabrication process.
Item 2 refers to architectural items ie not structural.
Item 3 refers to lamellar tearing. This is applicable to welding relatively thick plates together and is not relevant to the signage structure which consists of SHS members and SHS members welded to plates.

Structural Design Actions Part 2: Wind Actions AS1170.2

Referring to the document Wind Loading – History of Changes Aspec Engineering Pty Ltd, Brisbane, Australia

> The table near the base of the document shows that the calculation for the wind load on a structure for the 2002 code was the same as for the 2011 code. It was done for a particular region and design factors but as a comparison tool it shows both codes producing the same wind load.

Referring to the additional document “Key-Changes-to-AS-NZS-1170.2-2021”

> The document compares the 2021 wind code to the previous 2011 code and illustrates no changes relevant to the signage structure.

I have reviewed the relevant sections of the 2002 code and the 2021 code :

> Section 2: Calculation Of Wind Actions
> Section 3: Regional Wind speeds
> Section 4: Site Exposure Multipliers
> Appendix D: Free Standing Walls, Hoardings and canopies

for calculating wind on the signage structure and the equations and factors are the same.

3.0 Summary/Conclusion

For the supersite signage at Parramatta Rd Overpass, Auburn, NSW which was designed in 2008 :

1. The changes to AS1170.2 between 2002 and 2021 do not affect the determination of the wind load calculation on the signage structure.

2. The changes to AS4100 between 1998 and 2020 do not affect the structural sizing of the members or the connections design.

3. Structurally the signage structure is in accordance with current codes and the structural sections of the NCC.

If you have any questions, please do not hesitate to ring the undersigned on 0400 023 714.

Yours Faithfully,

John Linsell BE(Hons), MIEAust, CPEng, NER(Struct)
for Dennis Bunt Consulting Engineers Pty Ltd

AS/NZS 5131 & AS 4100 2020 UPDATE

ON 14 AUGUST 2020, STANDARDS AUSTRALIA PUBLISHED AN AMENDMENT TO AS/NZS 5131:2016 STRUCTURAL STEELWORK – FABRICATION AND ERECTION. FOLLOWING THIS, ON 21 AUGUST 2020, STANDARDS AUSTRALIA PUBLISHED A REVISION TO AS 4100 STEEL STRUCTURES. AS 4100 AND AS/NZS 5131 WORK TOGETHER TO ENSURE RISK-MINIMISED, FIT-FOR-PURPOSE DESIGN AND CONSTRUCTION OUTCOMES FOR STEEL STRUCTURES. ALL MEMBERS OF THE STEEL SUPPLY CHAIN SHOULD BE AWARE OF THE 2020 CHANGES TO THESE STANDARDS, THE IMPLICATIONS FOR THEIR BUSINESS AND BUSINESS RELATIONSHIPS, AND THEIR DUTY OF CARE UNDER BOTH WORKPLACE HEALTH AND SAFETY (WHS) AND NATIONAL CONSTRUCTION CODE (NCC) REGULATIONS.

HISTORICAL CONTEXT

Fabrication and erection of structural steel was previously addressed in two chapters of AS 4100. This was in sharp contrast to the situation in America, Canada, Europe and the UK. In each of these first-world countries, fabrication of structural steel is referenced to a self-standing separate Standard or specification, usually of a few hundred pages in length.

To ensure Australia maintained a baseline of internationally accepted 'good practice' and clearly defined quality standards, the Australian Steel Institute (ASI) developed a fabrication and erection Code of Practice, with agreement from Standards Australia that it would be submitted to become the first Standard for fabrication and erection of structural steel in Australia and New Zealand. The new Standard, AS/NZS 5131 *Structural steelwork - Fabrication and erection*, was published in 2016.

Following the publication of AS/NZS 5131, the next step was to revise AS 4100 to reference AS/NZS 5131 and remove the existing requirements for fabrication and erection from AS 4100. Significantly, as AS 4100 is a primary reference under the National Construction Code (NCC), referencing AS/NZS 5131 from AS 4100 will effectively make AS/NZS 5131 a secondary reference under the NCC.

AMENDMENTS TO AS/NZS 5131

The amendments of significance that have been made to AS/NZS 5131 are outlined below.

Traceability

Modifications to the definitions (Section 4) and application (Section 5) of traceability have been made to better align with international practice.

There are now three types of traceability, lot, piece-mark and piece. The type of traceability is applied over an extent of components on the project, where the extent is defined in relation to the Construction Category. In effect, the requirements for traceability have been 'unpacked' to allow more responsive application.

A baseline of lot traceability is required for Construction Categories CC2, CC3 and CC4, with no specified traceability for CC1. Optional piece-mark or piece traceability may be selected by the specifier if, and only if, required for CC3 and CC4. The baseline requirements are better aligned with international practice in this area.

Apart from 'unpacking' traceability to make it more flexible in its application, the significant difference with the 2020 amendment to AS/NZS 5131 is that lot traceability is the baseline for the CC2, CC3 and CC4 construction categories. The extent of application from main members to all members and components varies between the construction categories. For CC3 and CC4, the specifier may choose to require increased type traceability, either piece-mark or piece traceability. The previous version of AS/NZS 5131 required, in effect, piece traceability for all components for the higher construction categories.

The application of traceability has been one of the significant functional requirements for fabricators. ASI believes that this amendment makes traceability better aligned with industry expectations and international good practice.

AS/NZS ISO 3834

In AS/NZS 5131:2020 there is now normative reference to AS/NZS ISO 3834 *Quality requirements for fusion welding of metallic materials*, which was previously an informative reference. The good news is that the processes in AS/NZS 5131 were already aligned with the requirements of AS/NZS ISO 3834, a specific decision made by the Standards committee at the time of first publishing AS/NZS 5131. Therefore, the now normative reference to AS/NZS ISO 3834 does not actually introduce much by way of new requirements. ASI will shortly publish a new Technical Note discussing certification and comparing the scope of AS/NZS 5131:2016 and AS/NZS ISO 3834.

ABCB ALIGNMENT

With the publication of the revision to AS 4100, which now directly references AS/NZS 5131, there were necessary wording revisions throughout the document to reflect Australian Building Codes Board (ABCB) requirements for documents referenced under the National Construction Code (NCC). These changes are generally not 'mission critical'. In many cases 'shall' has been changed to 'should', which effectively makes the requirement non-mandatory. However, these changes were often to do with referencing, for example, normative referencing of manufacturers specifications or installation instructions. Normative referencing of third-party material is not supported by the ABCB, as it effectively makes manufacturers' specifications or installation instructions part of regulation, yet the ABCB has no control over this documentation.

RISK MATRIX

There has been a small but important change to the risk matrix used to assess the Construction Categories. For structures which are Importance Level 3 under the NCC, and with simple construction, the recommendation has changed from 'CC3' to 'CC2/CC3' with a note to allow engineers to specify CC2 where the construction is simple. This small but important change will have significant impact on increasing the range of structures that CC2 fabricators can rightly work with.

These changes have been in response to industry feedback and ASI engagement with Standards to ensure AS/NZS 5131 continues to be responsive to industry needs and supports our fabricators and related industries. It is pleasing that with the first major amendment to the new Standard AS/NZS 5131, the changes required were limited and responsive to industry concerns. It is an indication that the new Standard is working and has been accepted by industry.

REVISION OF AS 4100

The primary reason for revising AS 4100:1998 was to reference AS/NZS 5131 *Structural steelwork – Fabrication and erection*. The Standards committee BD-01 took the opportunity to also include a number of further updates consistent with updates to a number of other steel-related Standards that had occurred since the 2012 amendment to AS 4100.

The major changes to AS 4100:1998 include:

- Fabrication and erection: Sections 14 and 15 on fabrication and erection respectively have been reduced considerably in extent, retaining only the critical elements where engineering input is required and referencing AS/NZS 5131 for the majority of fabrication and erection requirements
- Construction Category: Selection of the Construction Category (previously only in AS/NZS 5131) has been made an engineering requirement, together with a new Appendix L providing guidance on selection of the Construction Category, including the same risk matrix
- Architecturally exposed structural steel: Definition and description of 'architecturally exposed structural steel' (AESS) allows engineers to ensure their specifications accurately convey architectural intent and that intent ties into the requirements for AESS in AS/NZS 5131
- Lamellar tearing: Definition and description of lamellar tearing. Addressing the likelihood of lamellar tearing in particular welded connections is a responsibility shared between engineers and fabricators
- High strength bolts: Introduction of a new 'alternative bolt assembly type' to EN 14399-3 Type HR for grade 8.8 bolts and an 'additional bolt assembly type' to EN 14399-3 Type HR for grade 10.9 bolts, together with specific design requirements for grade 10.9 bolts
- Geometric tolerances: New specification of geometrical tolerances for fabrication and erection aligned with AS/NZS 5131
- Construction specification: Reference to the 'construction specification' (as the document containing the particular design data and details to be provided) as one deliverable from the design process

The nett effect of these changes is to more effectively tie fabrication and erection into the project process and ensure engineers clearly articulate engineering intent for risk-minimised fit-for-purpose fabrication outcomes.

It is important that engineers adequately convey design intent. The usual mechanism is via the construction specification prepared by the engineer for the project. ASI has created (and recently updated) the 'National Structural Steelwork Specification' (NSSS) and associated 'Standard Drawing Notes', specifically intended for engineers to utilise. These can be downloaded via the QR codes below.

A complete and correct specification helps to manage risk for the project and engineer's duty of care under the Workplace Health and Safety Act and the 'Safe Design of Structures' Code of Practice. ASI has prepared tools for achieving compliant steel and steelwork outcomes for many members of the supply chain under the 'Responsible steelwork procurement' initiative.

CLICK HERE TO ACCESS FURTHER INFORMATION ON ASI'S RESPONSIBLE STEELWORK PROCUREMENT INITIATIVE >>>

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Wind Loading – History of Changes

Claudia Cipressi[1]
[1] Aspec Engineering Pty Ltd, Brisbane, Australia

Wind loading of structures is a complex phenomenon and a major consideration when ensuring the safety and design of industrial assets. However, standards have changed over the years to incorporate the latest research findings, and the knowledge obtained from recent severe weather events. This article aims to investigate the historical changes to Australian wind loading standards, and the rationale behind them.

The first modern wind loading code published in Australia in 1971, was the CA 34.2, which replaced an earlier interim document, the SAA Int 350. Whilst printed in Imperial units, all subsequent standards have descended from it, with notable changes being made in each new edition. In 1973, the CA34.2 was converted to metric units in the AS 1170.2: 1973 edition, whilst the occurrence of Cyclone Tracy which hit Darwin in 1974, created many modifications relating to the findings following the severe weather event. The AS 1170.2: 1989 was a major revision of previous standards, with the conversion to limit state design which is still implemented in the following 2002 and 2011 editions.

To recognise the modifications made to Australian standards over the years, it is necessary to understand the underlying concepts wind interaction with structures, and how this is accounted for in the design process.

Using the current limit state design method, Australian Standards design for an ultimate wind speed, on a probability basis where the wind speed has a small chance of being exceeded in the life of the structure. Wind speed data is collected from anemometer stations around Australia, and was defined as a gust of 2-3 seconds duration, recorded at the meteorological height of 10m in flat open terrain. However, it has been since found that the averaging time of the peak gust was considerably less than 2-3 seconds, (Holmes, 2012), with the recent 2011 version redefining the peak gust as having a moving average time of approximately 0.2 seconds, (Standards Australia, 2017). CA 34.2-1971 contained a contour map of ‘regional basic wind speeds’ with a 50-year return period, and a table of wind velocities for 48 locations where data had been obtained. However, many of these values are identified as ‘short record’, as less than 15 years of records were available from the anemometer station, making 50 and 100-year return period estimates fairly tentative, (Australian Standards, 1971). This edition of the standard included a cyclone factor of 1.15, to be applied to the wind speed of locations within a ‘Tropical cyclone area’. The need for cyclone factors was eliminated in the 1989 edition of AS 1170.2, with the specification of high return-period design wind speeds, (i.e. 1000 years). Additionally, the need for importance multipliers in the 1989 edition was eliminated as variable annual probability of exceedance was adopted for wind speeds in the 2002 version.

Once regional wind speed has been decided, other factors must be considered such as the increase in wind speed which occurs with height. Taller structures have higher wind loads than low level structures, which is considered by applying height multipliers. It must be realised that standards prior to 1971 did not account for this effect and assume a single value over two central and coastal regions.

When strong winds interact with a structure, pressure and forces are generated, with the characteristics of these pressures being defined by the characteristics of the approaching wind, geometry and permeability of the structure. Due to the turbulent and gusty nature of wind, these pressures are not constant but highly fluctuating, and the interaction with the shape of structure itself, such as local eddies at the edges, causes the pressure distribution to vary over the surface of the structure.

The distribution of pressures is determined by aerodynamic shape factors, which have been developed and modified over the years reflecting new research findings. Additionally, for tall structures the dynamic response of the building must be considered. Information regarding the dynamic response has been expanded over the years, from an informative annex in 1971, to inclusion of a dynamic response factor in design pressure calculations in the current standard.

Considering all the above factors, in the table below it can be seen that the wind design pressure for a structure located in Central Queensland, varies with each standard. There is a particularly large increase in design pressure between a structure built in 1971, compared to a structure built in 1989, highlighting the changes in regional wind speeds post Cyclone Tracy. It can also be noted that the elimination of importance multipliers and instead the use of variable return periods in the 2002 edition, allows for a slightly decreased basic wind velocity.

Year	Australian Standard	Basic Wind Velocity	Height Multiplier	Formula for Design Pressure	Design Pressure (kPa)
Pre 1971	SAA Int 350	90 mph	None	$P = V^2/100$	0.97
1971	CA 34.2-1971	100 mph	1.08	$P = C_p q_z$ $q_z = V_z^{\,2}/400$	1.3
1973	AS 1170.2:1973 1st Edition	45 m/s	1.08	$P = C_p q_z$ $q_z = 0.6V_z^2 \times 10^{-3}$	1.42
1989	AS 1170.2:1989 3rd Edition	49 m/s	1.12	$q_z = 0.6V_z^2 \times 10^{-3}$	1.82
2002 & 2011	AS 1170.2:2002 2nd Edition AS 1170.2:2002 5th Edition	44 m/s	1.12	$P = (0.5\rho_{air})V_z^2 C_{fig} C_{dyn}$	1.46

Note:
The values assume the following:
- 50-year Return Period
- 10m structure located in Central Queensland, (Region B)
- Pressure coefficients $C_p, C_{fig}, C_{dyn} = 1.0$

Clearly, historical changes to the wind loading standards are important to understand when assessing existing structures as it can have a direct effect on the design specifications. Therefore, it is useful to know what standard for which the structure has been designed, to evaluate its susceptibility to wind based on the current wind standard.

1. References

Australian Standards. (1971). *CA34, Part II-1971.* Sydney: Standards Association of Australia.

Holmes, J. (2012). The gust wind speed duration in the AS/NZs 1170.2. *Australian Journal of Structural Engineering, 13*, 207-218. Retrieved from http://dx.doi.org/10.7158/S12-017.2012.13.3.\

Standards Australia. (1971). *SAA Interim 350 - 1952* (3rd ed.). Sydney: Standards Association of Australia.

Standards Australia. (1993). *AS 1170.2-1989* (3rd ed.). Sydney: Standards Association of Australia.

Standards Australia. (2005). *AS/NZS 1170.2:2002* (2nd ed.). Sydney: Standards Australia.

Standards Australia. (2017). *AS/NZS 1170.2:2011* (5th ed.). Sydney: Standards Association of Australia.

KEY CHANGES TO AS/NZS 1170.2-2021

Presented by **Chris Hackney**

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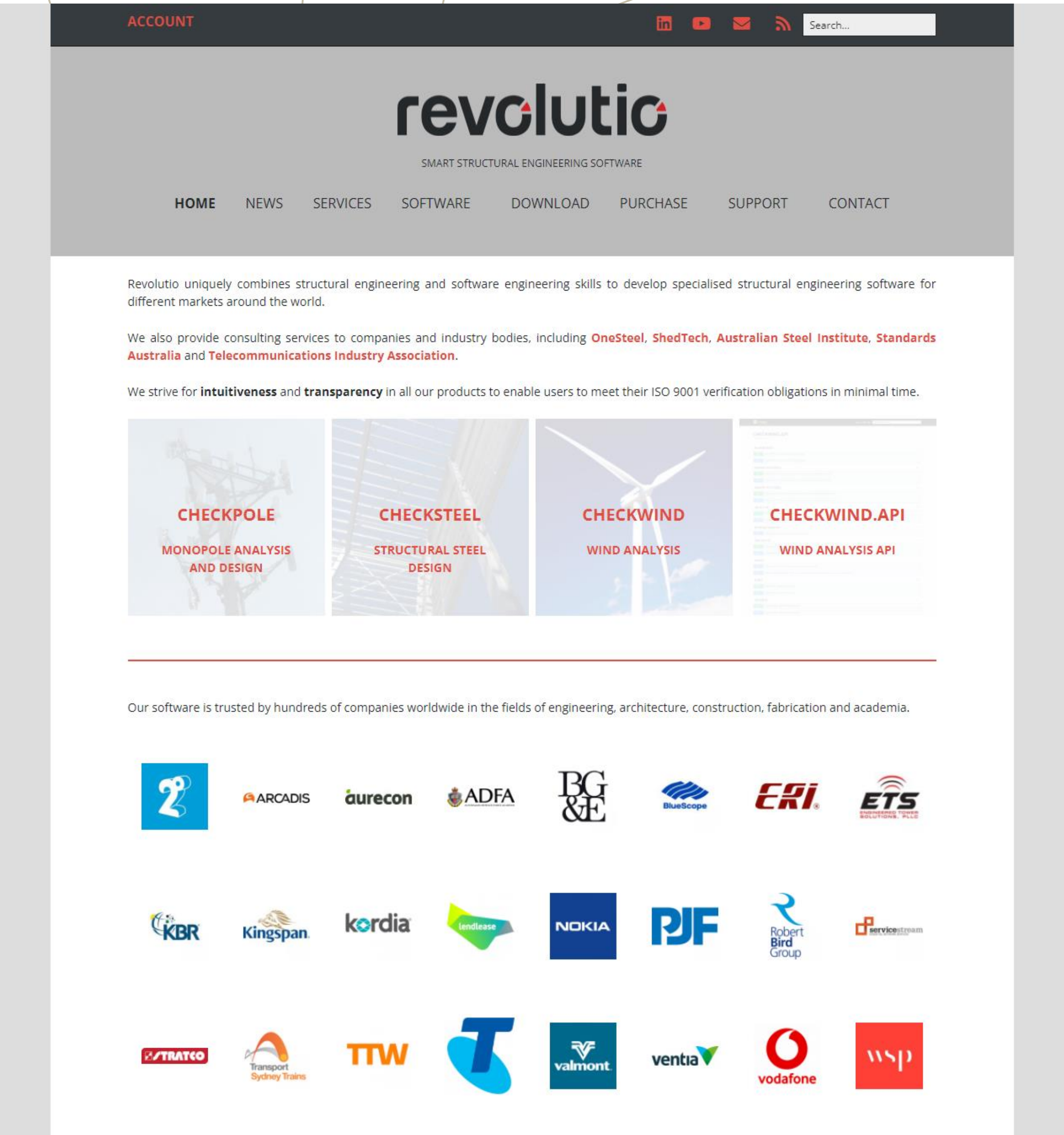
www.revolutio.com.au



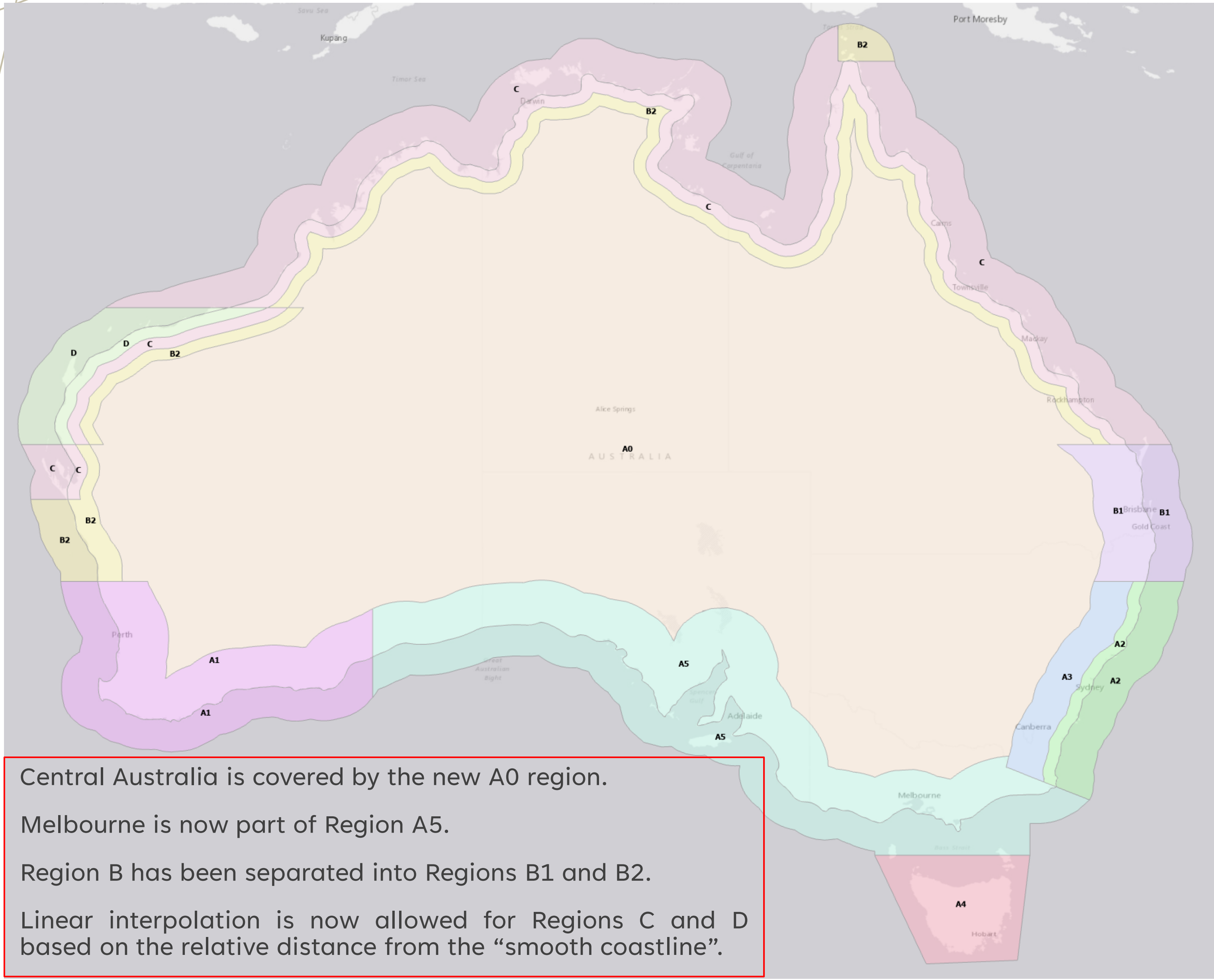
ABOUT US

Revolutio was founded in 2012 and specialises in developing desktop and web applications to automate engineering tasks.

Our software is trusted by over 2,800 users representing approximately 400 different companies in Australia, New Zealand, USA, Asia and the Middle East.

Committee Member:	AS/NZS 1170.2 (Australia/New Zealand)
Committee Member:	TR-14 / TIA-222-H (USA)
Consultant:	Australian Steel Institute
Contributing Author:	Design Capacity Tables for Structural Steel: Volume 1 – Open Sections (5th edn)
Education:	B.E. (Civil) (Structures) (Hons) University of Sydney (2009)
Previous Experience:	Taylor Thomson Whitting SCP Consulting Opus International Consultants

WIND REGION MAPS – AUSTRALIA



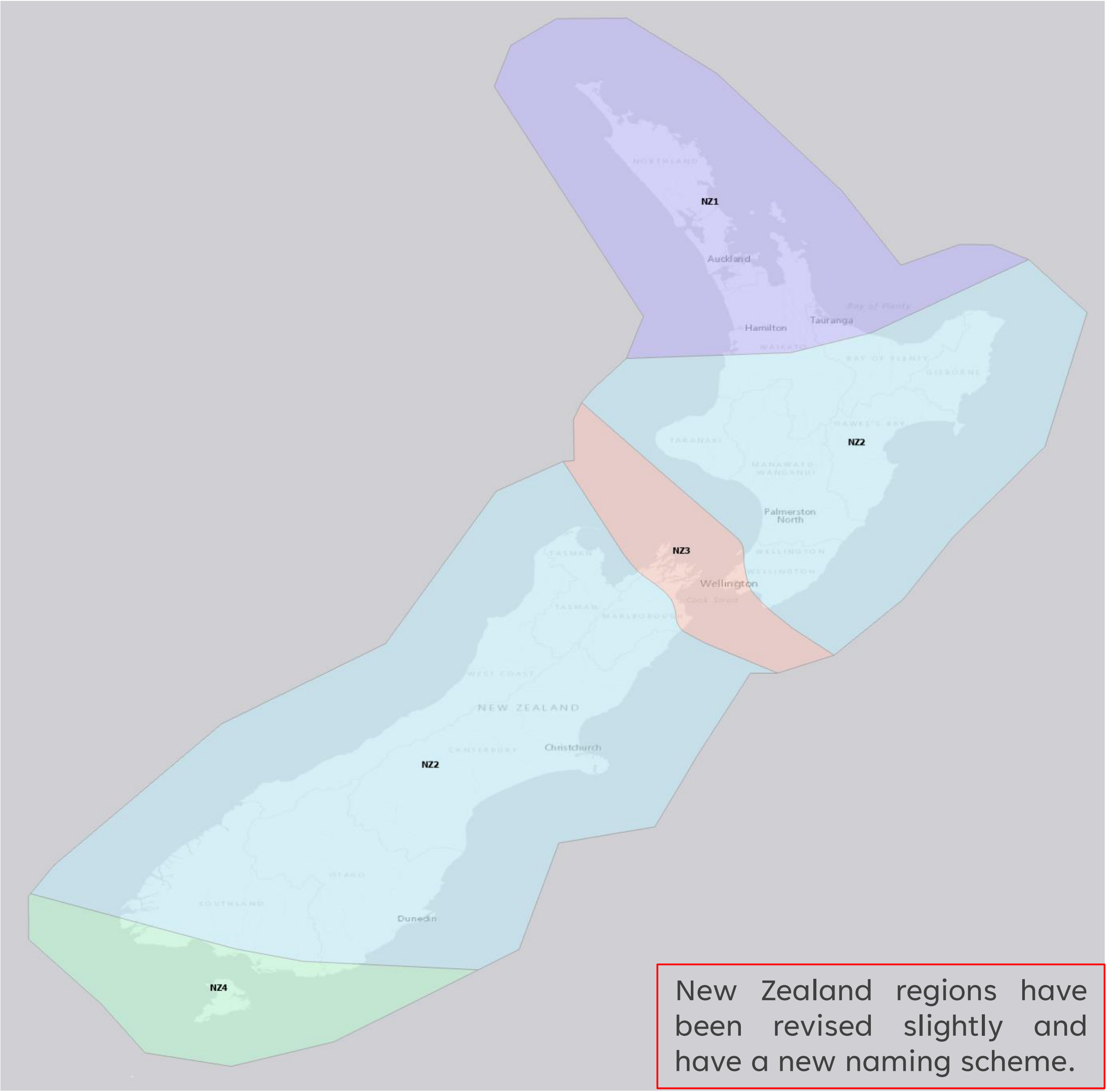
Central Australia is covered by the new A0 region.

Melbourne is now part of Region A5.

Region B has been separated into Regions B1 and B2.

Linear interpolation is now allowed for Regions C and D based on the relative distance from the "smooth coastline".

WIND REGION MAPS – NEW ZEALAND



New Zealand regions have been revised slightly and have a new naming scheme.

DIRECTIONAL MULTIPLIER (M_d)

3.3 Wind direction multiplier (M_d)

Except for the following cases, the wind direction multiplier (M_d) for all regions shall be as given in Table 3.2(A) or Table 3.2(B). For the following cases, M_d shall be taken as 1.0:

(a) structures such as chimneys, tanks and poles with circular or polygonal cross-sections; and

(b) cladding and immediate supporting structure (as defined in Clause 5.4.4) on buildings in Regions B2, C and D.

NOTE In regions where the prevailing wind directions vary with wind speed, wind direction multipliers have been calculated for the higher wind gusts (i.e. those associated with ultimate limit states design).

From John Holmes (edited for clarity):

"Item (a) in Clause 3.3, are only intended apply to stand-alone poles and masts. Transmission lines are very direction sensitive. Applying M_d from the Table 3.2(B) in AS/NZS 1170.2 should be fine, as called up by AS/NZS 7000."

Table 3.2(A) — Wind direction multiplier (M_d) — Australia

Cardinal directions	Region A0	Region A1	Region A2	Region A3	Region A4	Region A5	Region B1	Regions B2, C, D
N	0.90	0.90	0.85	0.90	0.85	0.95	0.75	0.90
NE	0.85	0.85	0.75	0.75	0.75	0.80	0.75	0.90
E	0.85	0.85	0.85	0.75	0.75	0.80	0.85	0.90
SE	0.90	0.80	0.95	0.90	0.80	0.80	0.90	0.90
S	0.90	0.80	0.95	0.90	0.80	0.80	0.95	0.90
SW	0.95	0.95	0.95	0.95	0.90	0.95	0.95	0.90
W	1.00	1.00	1.00	1.00	1.00	1.00	0.95	0.90
NW	0.95	0.95	0.95	0.95	1.00	0.95	0.90	0.90

NOTE In Region A0 non-synoptic winds are dominant. In Regions A1 and A4, extra-tropical synoptic winds are dominant. Extreme winds in Regions A2, A3, A5 and B1 are caused by a mixture of synoptic (extra-tropical large-scale pressure systems, or tropical cyclones in the case of B1) and non-synoptic (thunderstorm) events. In Regions B2, C, and D, extreme winds from tropical cyclones are dominant.

Table 3.2(B) — Wind direction multiplier (M_d) — New Zealand

Cardinal directions	Region NZ1	Region NZ2	Region NZ3	Region NZ4
N	0.90	0.95	1.00	0.95
NE	0.95	0.90	0.75	0.75
E	0.95	0.80	0.75	0.75
SE	0.95	0.90	0.85	0.75
S	0.90	0.95	0.95	0.85
SW	1.00	1.00	0.95	0.95
W	1.00	1.00	0.90	1.00
NW	0.95	1.00	1.00	1.00

NOTE In all New Zealand regions, extra-tropical synoptic winds are dominant.

CLIMATE CHANGE MULTIPLIER (M_c) – AUSTRALIA ONLY

3.4 Climate change multiplier (M_C)

The climate change multiplier (M_C) shall be as given in Table 3.3.

Table 3.3 — Climate change multiplier (M_C)

Region	M_C
A (0 to 5)	1.0
B1	1.0
B2	1.05
C	1.05
D	1.05
NZ (1 to 4)	1.0
NOTE The climate change multiplier allows for possible changes in climate affecting extreme winds during the life of structures designed by this Standard. Values of M_C may be adjusted in future amendments, depending on observed or predicted trends.	

Effectively replaces F_C and F_D factors from AS/NZS 1170.2-2011, but now relevant to the new Region B2.

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

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) *Terrain Category 1 (TC1)* — Very exposed open terrain with very few or no obstructions, and all water surfaces (e.g. flat, treeless, poorly grassed plains; open ocean, rivers, canals, bays and lakes).

(b) *Terrain Category 2 (TC2)* — Open terrain, including grassland, with well-scattered obstructions having heights generally from 1.5 m to 5 m, with no more than two obstructions per hectare (e.g. farmland and cleared subdivisions with isolated trees and uncut grass).

(c) *Terrain Category 2.5 (TC2.5)* — Terrain with some trees or isolated obstructions, terrain in developing outer urban areas with scattered houses, or large acreage developments with more than two and less than 10 buildings per hectare.

(d) *Terrain Category 3 (TC3)* — Terrain with numerous closely spaced obstructions having heights generally from 3 m to 10 m. The minimum density of obstructions shall be at least the equivalent of 10 house-size obstructions per hectare (e.g. suburban housing, light industrial estates or dense forests).

(e) *Terrain Category 4 (TC4)* — Terrain with numerous large, high (10 m to 30 m tall) and closely-spaced constructions, such as large city centres and well-developed industrial complexes.

Selection of the terrain category shall be made with due regard to the permanence of the obstructions that constitute the surface roughness.

NOTE The aerodynamic roughness length, z_0, in metres, is related to the terrain category number by the following relation: $z_0 = 2\times10^{(TC\,\text{number}-4)}$

Terrain Category 1.5 ("open water surfaces subjected to shoaling waves") removed.

4.2.3 Averaging of terrain categories and terrain-height multipliers

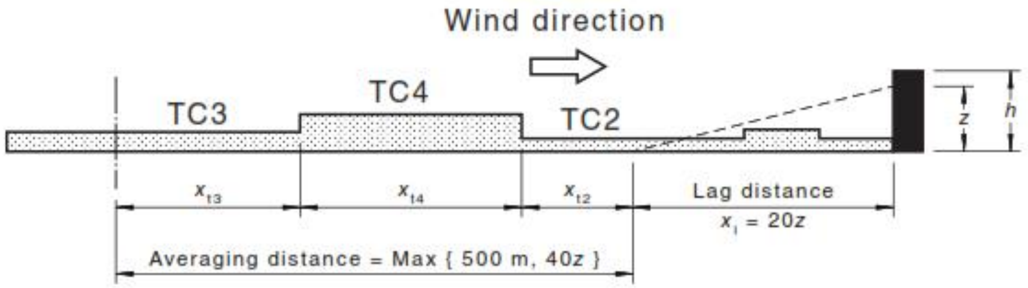
When the upwind terrain varies for any wind direction, an averaging of terrain-height multipliers shall be adopted. The terrain-height multiplier, $M_{z,cat}$, shall be taken as a weighted average over an averaging distance, x_a, depending on the height, z.

NOTE z is equal to the average roof height, h, of a building, when it is less than, or equal to, 25 m.

The averaging distance, x_a, shall be the larger of 500 m or 40z.

Terrain shall be assessed after ignoring the terrain immediately upwind for a lag distance, x_i, where x_i is taken as 20z.

An example of this averaging procedure is given in Figure 4.1.



$$M_{z,cat} = \frac{M_{z,cat2}\,x_{t2} + M_{z,cat4}\,x_{t4} + M_{z,cat3}\,x_{t3}}{x_{t2} + x_{t4} + x_{t3}} \quad \text{for the case illustrated}$$

NOTE The terrain within the lag distance, x_i, is ignored when averaging terrain-height multipliers.

Figure 4.1 — Example of averaging of terrain-height multipliers

The averaging distance is now MAX(500, 40z) instead of MAX(500, 40h) from AS/NZS 1170.2-2011.

For ***buildings*** $\leq$ 25 m, use z = h. For all other structures and buildings > 25 m, use z.

SHIELDING MULTIPLIER (M_s)

4.3 Shielding multiplier (M_S)

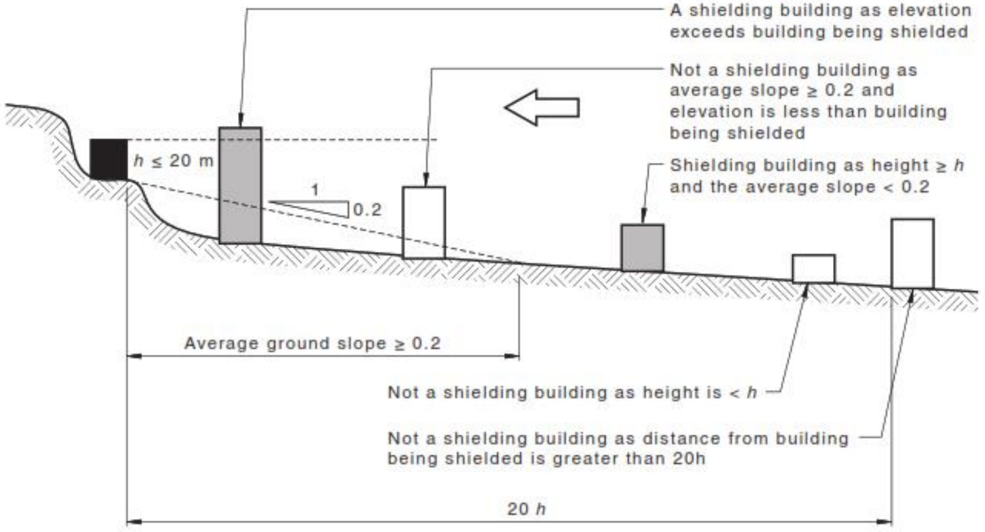
4.3.1 General

Shielding may be provided by upwind buildings or other structures. Shielding shall not be provided by trees or vegetation. An upwind building shall not be used to provide shielding on a slope with a gradient that is greater than 0.2, unless its overall height above a common datum, such as mean sea level, exceeds that of the subject building (see Figure 4.2).

The shielding multiplier (M_S) that is appropriate to a particular direction shall be as given in Table 4.2 for structures with $h \leq 25$ m in height (h is defined in Figure 2.1).

The shielding multiplier shall be 1.0 for structures with h greater than 25 m, where the effects of shielding are not applicable for a particular wind direction, or are ignored.

NOTE To accurately determine shielding and interference effects between buildings with h greater than 25 m, wind-tunnel testing is needed. Attention should be given to possible combinations of tall buildings placed together, which can lead to local and overall increases in wind actions.



This section has been expanded to remove the ambiguity present in AS/NZS 1170.2-2011 regarding maximum slope requirements. Structures exceeding 25 m in height must now adopt a value of M_s = 1.0 for all values of z unless wind tunnel testing is carried out to prove otherwise.

4.3.3 Shielding parameter (s)

The shielding parameter (s) in Table 4.2 shall be determined from Equation 4.3(1):

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

where

l_s = average spacing of shielding buildings, given by Equation 4.3(2):

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

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 $h_s \geq h$

The number of upwind shielding buildings (n_s) is now defined as the number of buildings with an average structure height (h_s) exceeding the height of the structure (h) under consideration.

TOPOGRAPHIC MULTIPLIER (M_t)

4.4 Topographic multiplier (M_t)

4.4.1 General

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

(a) For sites in Regions A4, NZ1, NZ2, NZ3 and NZ4 over 500 m above sea level, use Equation 4.4(1):

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

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) For sites in Region A0, use Equation 4.4(2):

$$M_t = 0.5+0.5M_h \quad 4.4(2)$$

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

(i) $M_t = M_h$

(ii) $M_t = M_{lee}$

Sites in Region A0 now have their own equation for M_t.

Topographic features < 10 m in height can be ignored.

Clarification on when upwind topography features can be ignored "*provided the crest is distant from the site of the structure by more than 10 times its **crest** elevation above sea level*".

4.4.2 Hill-shape multiplier (M_h)

The hill-shape multiplier shall be taken as 1.0 outside of the local topographic zones shown in Figures 4.3 to 4.5, and for H < 10 m. Within the local topographic zones, 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 values shall be as follows:

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

(b) For $0.05 \leq H/(2L_u) \leq 0.45$ (see Figures 4.3 and 4.4), use Equation 4.4(3):

$$M_h = 1+\left(\frac{H}{3.5(z+L_1)}\right)\left(1-\frac{|x|}{L_2}\right) \quad 4.4(3)$$

(c) For $H/(2L_u) > 0.45$ (see Figure 4.5):

(i) Within the rectangular peak zone (see Figure 4.5), use Equation 4.4(4):

$$M_h = 1+0.71\left[1-\frac{|x|}{L_2}\right] \quad 4.4(4)$$

(ii) Elsewhere within the local topographic zone (see Figures 4.3 and 4.4), M_h shall be as given in Equation 4.4(3).

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 4.3, 4.4 and 4.5 are cross-sections through the structure's site for a particular wind direction.

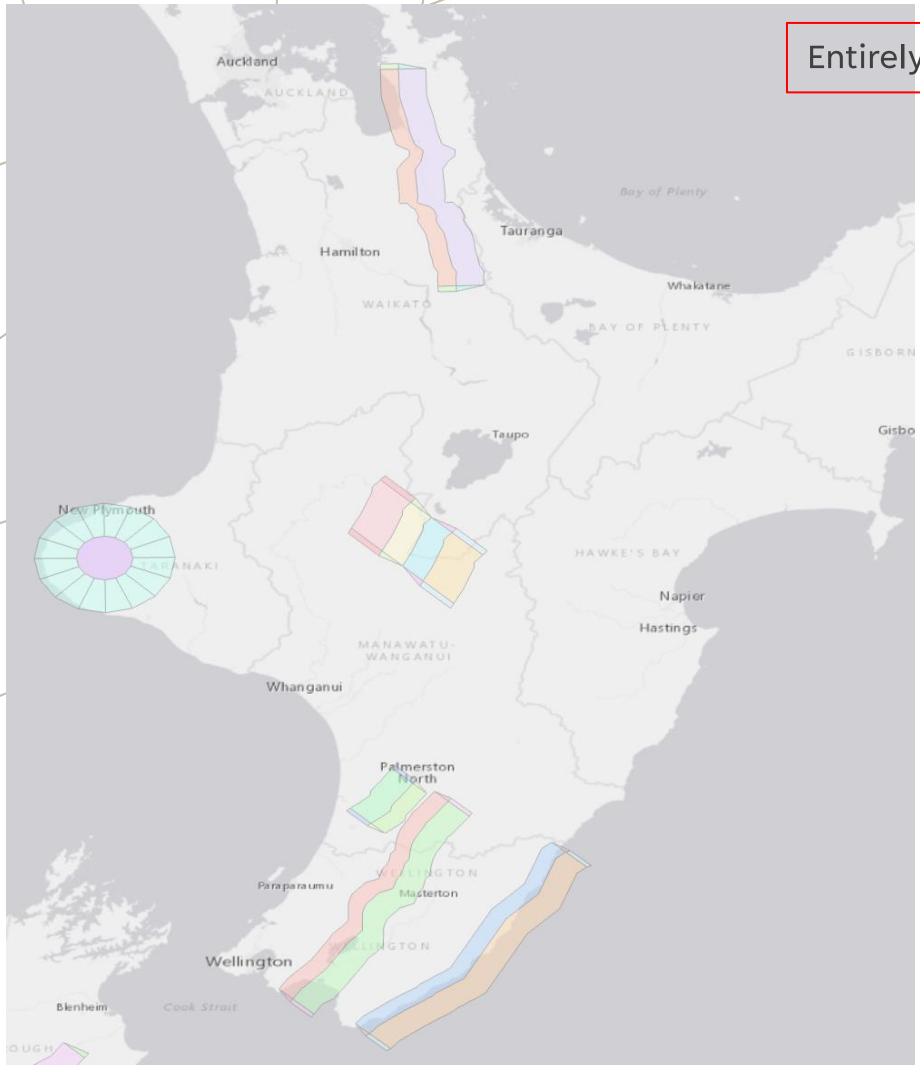
For the case where x and z are zero, the value of M_h is given in Table 4.3.

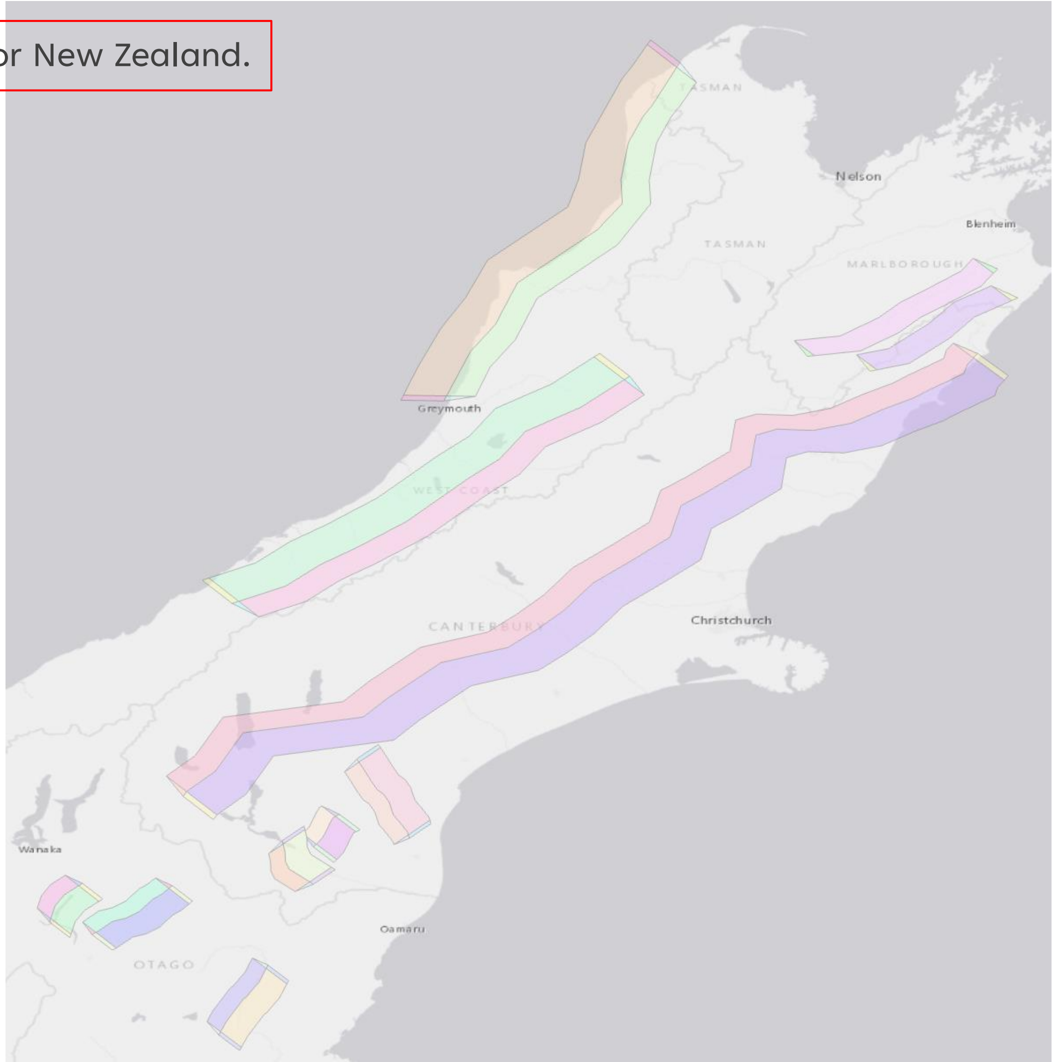
Irrespective of the provisions of this Clause, the influence of any peak may be ignored, provided the crest is distant from the site of the structure by more than 10 times its crest elevation above sea level, and any intervening valley is more than 10 times the distance of the valley floor below the crest.

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.

LEE ZONE MAPS – NEW ZEALAND

Entirely new lee zone maps for New Zealand.

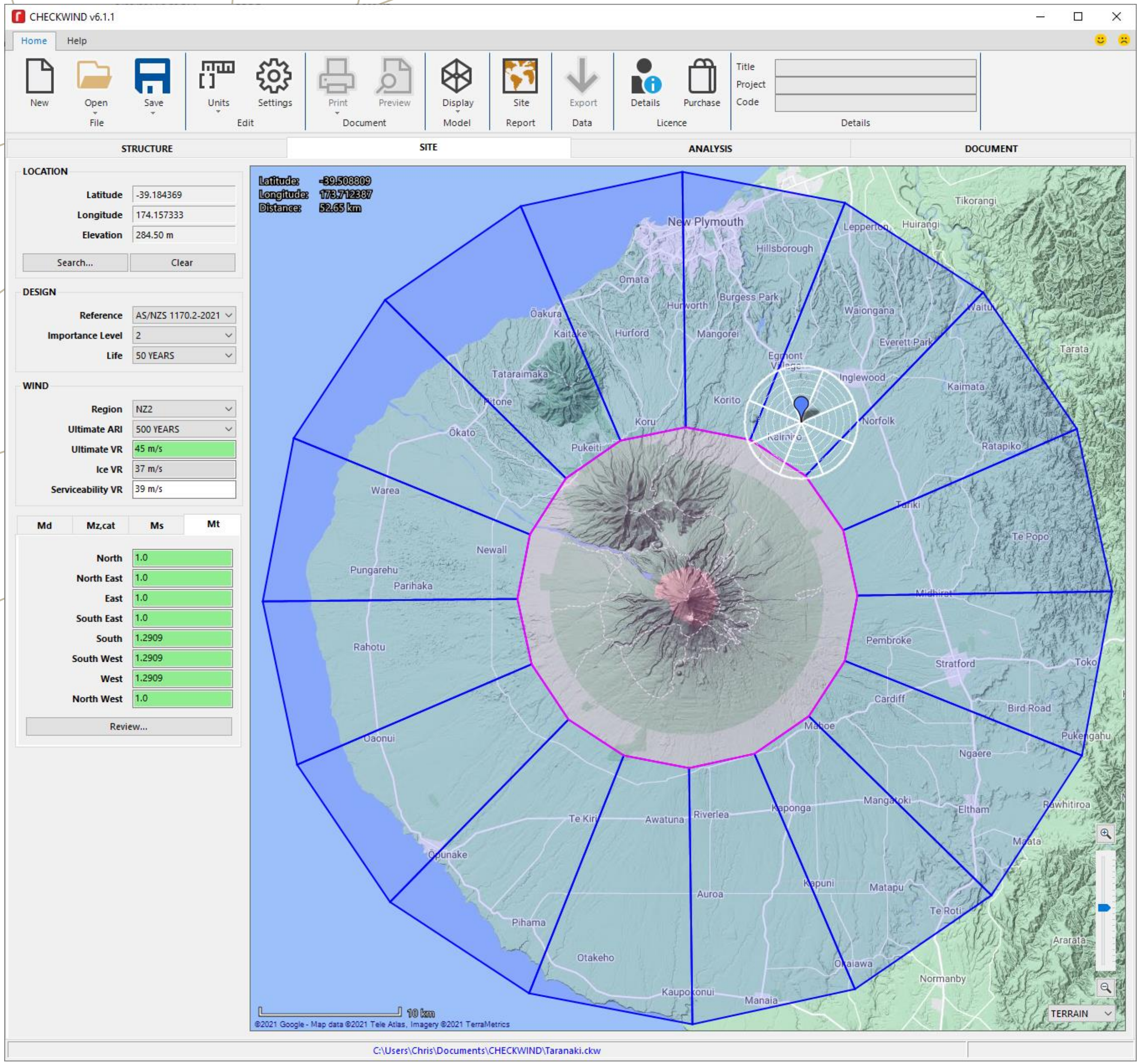




Web version available through our website:

https://www.revolutio.com.au/2021/09/28/explained-key-changes-introduced-in-as-nzs-1170-2-2021/

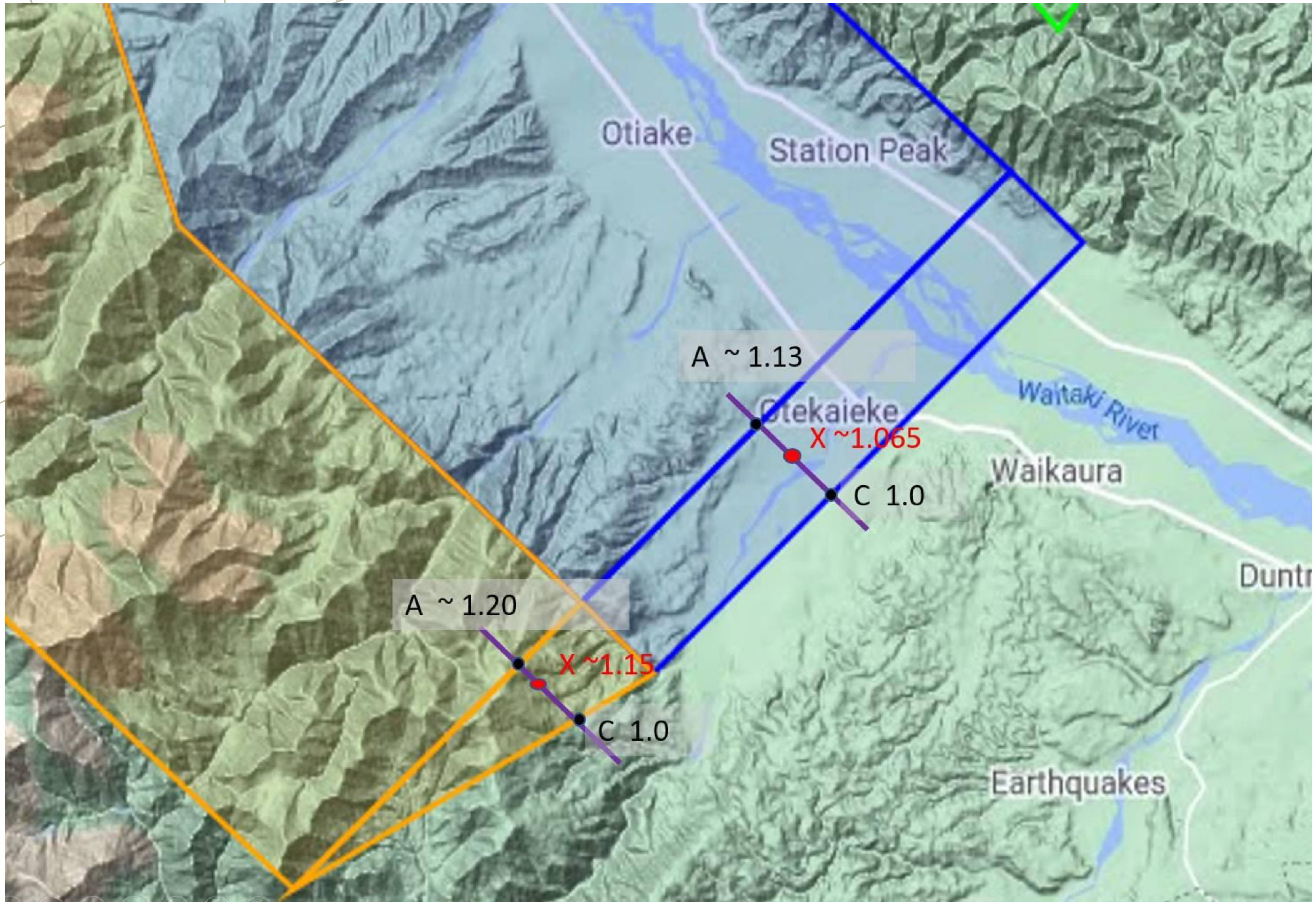
LEE ZONE MAPS – NEW ZEALAND



The Taranaki lee zone differs from the others in that the value is dependent on the relative bearing of the site from Mt Taranaki and the wind direction under consideration.

For example, if the site is to the **North East** of Taranaki, M_{lee} > 1.0 for wind directions within a 90° sector from the tip of Tarankai, so in this case for wind coming from the **South**, **South West**, and **West** directions.

LEE ZONE MAPS – NEW ZEALAND



The new "lateral transition zones" are intended to prevent the previous situation where M_{lee} could equal 1.35 in one location but 1.0 just 100 m away.

Linear interpolation of the value from the nearest point (A) on the side-edge of either the shadow or outer zone to the value (1.0) at the point (C) on the outer edge of the transition zone, where C is the intersection of the outer edge of the transition zone and the line that runs through A-X.

If for example it is a SW zone, then a line oriented perpendicular to the SW direction (i.e. NW to SE) should be parallel to A-X.

AERODYNAMIC SHAPE FACTOR (C_{shp})

5.2 Evaluation of aerodynamic shape factor

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

(a) Enclosed buildings — use Equations 5.2(1), 5.2(2) and 5.2(3):

$$\boxed{C_{shp} = C_{p,i} K_{c,i} K_v \text{, for internal pressures}} \quad 5.2(1)$$

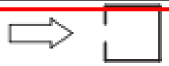
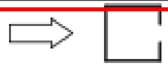
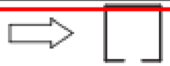
$$C_{shp} = C_{p,e} K_a K_{c,e} K_\ell K_p \text{, for external pressures} \quad 5.2(2)$$

$$C_{shp} = C_f K_a K_{c,e} \text{, for frictional drag forces} \quad 5.2(3)$$

C_{fig} renamed C_{shp} and introduction of **Volume Factor** (K_v) for internal pressures.

INTERNAL PRESSURE COEFFICIENTS ($C_{p,i}$)

Table 5.1(B) — Internal pressure coefficients ($C_{p,i}$) for buildings with openings greater than 0.5 % of the area of the corresponding wall or roof

Ratio of area of openings on one surface to the sum of the total open area (including permeability) of other wall and roof surfaces	Largest opening on windward wall	Largest opening on leeward wall	Largest opening on side wall	Largest 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	0.7 $K_a K_\ell C_{p,e}$	$K_a K_\ell C_{p,e}$	$K_a K_\ell C_{p,e}$	$K_a K_\ell C_{p,e}$
3	0.85 $K_a K_\ell C_{p,e}$	$K_a K_\ell C_{p,e}$	$K_a K_\ell C_{p,e}$	$K_a K_\ell C_{p,e}$
6 or more	$K_a K_\ell C_{p,e}$	$K_a K_\ell C_{p,e}$	$K_a K_\ell C_{p,e}$	$K_a K_\ell C_{p,e}$
				

NOTE 1 $C_{p,e}$ is the relevant external pressure coefficient at the location of the largest opening. For example, in Column 2, $C_{p,e}$ means the windward wall pressure coefficient obtained from Table 5.2(A); in Column 3, $C_{p,e}$ means the leeward wall pressure coefficient obtained from Table 5.2(B), in Column 5, $C_{p,e}$ means the roof pressure coefficient for that part of the roof containing the opening.

NOTE 2 K_a is the area reduction factor related to the total area of the opening(s), A, on the surface under consideration treating the "tributary area" as the area of the opening. See Clause 5.4.2.

NOTE 3 K_ℓ is the local pressure factor, based on the area and location of the opening on the surface under consideration, treating the "Area, A" as the area of the opening. See Clause 5.4.4.

NOTE 4 Surfaces with openings have a ratio of total open area, A, to the total area of that surface related to the internal volume (Vol) under consideration, greater than 0.5 %.

Internal Pressure Coefficients ($C_{p,i}$) are now determined using K_a and K_l factors for situations with "dominant" openings. Cases where K_l > 1.0 must be considered in the calculation of $C_{p,i}$ for **all structural elements** (both cladding and non-cladding).

VOLUME FACTOR (K_v)

5.3.4 Open area/volume factor, K_v

When the largest opening in a building is on a wall, and the open area is greater than the sum of the total open area on the roof and other wall surfaces by a factor of six or more, then the following Equation 5.3(1) applies:

$$K_v = 1.01 + 0.15\left[\log_{10}\left(100\frac{A^{3/2}}{Vol}\right)\right] \text{ for } 0.09 \leq \left(100\frac{A^{3/2}}{Vol}\right) \leq 3 \qquad 5.3(1)$$

$$K_v = 0.85, \text{ for } \left(100\frac{A^{3/2}}{Vol}\right) < 0.09$$

$$K_v = 1.085, \text{ for } \left(100\frac{A^{3/2}}{Vol}\right) > 3$$

where A is the open area on the wall and Vol is the internal volume.

For all other cases, K_v shall be taken as 1.0.

NOTE 1 Openings on side walls exposed to relatively small volumes (e.g. partially enclosed balconies on high-rise buildings) may generate significant cavity pressure oscillations.

NOTE 2 Internal volume means the volume of the enclosed space exposed to the opening.

A new factor designed to adjust internal pressure coefficients ($C_{p,i}$) for buildings with dominant wall openings based on the internal volume exposed to that opening.

AREA REDUCTION FACTOR (K_a)

5.4.2 Area reduction factor (K_a) for roofs and walls

For roofs and walls of enclosed buildings, 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. For intermediate values of A, linear interpolation shall be used.

Table 5.4 — Area reduction factor (K_a)

Tributary area (A), m^2	Roofs and side walls (K_a)	Windward walls (K_a) $h < 25$ m	Leeward walls (K_a) $h < 25$ m
≤ 10	1.0	1.0	1.0
25	0.9	0.95	1.0
≥ 100	0.8	0.9	0.95

Can now be applied to windward and leeward walls for buildings with $h < 25$ m.

ACTION COMBINATION FACTOR (K_c)

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, or cladding 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 actions.

A surface shall be either a windward wall, a side wall, a leeward wall, a roof (the upwind and downwind roof are 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 $|C_{pi}| < 0.4$

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.

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.

Internal surfaces are no longer considered effective if $|C_{p,i}| < 0.4$ (increased from $|C_{p,i}| < 0.2$ in AS/NZS 1170.2-2021).

LOCAL PRESSURE FACTOR FOR CLADDING (K_l)

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 actions 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. If any area of cladding is covered by more than one case in Table 5.6, the largest value of K_ℓ obtained for any case shall be used.

Where the cladding or the supporting member extends beyond the distance a given in Table 5.6, a value of K_ℓ = 1.0 shall apply to wind force contributions imposed beyond that distance.

Design cases for negative pressures in Table 5.6 are alternative cases and shall not be applied simultaneously.

For walls, the value of dimension a is the minimum of 0.2b or 0.2d or the height (h) as shown in Figure 5.3. For roofs, the value of a is the minimum of 0.2b or 0.2d, if (h/b) or (h/d) ≥ 0.2; or 2h if both (h/b) and (h/d) < 0.2. The side ratio of any local pressure factor area shall not exceed 4.

Value of *a* for roofs is now based on the building ratio *h/b*.

Table 5.6 — Local pressure factor (K_ℓ)

Design case	Figure 5.3 reference case	Building aspect ratio (r)	Area (A)	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° AND	RC1	All	$A \leq 0.25a^2$	< a from two edges	3.0
Downwind corners of roofs with pitch ≥ 10°	RC2			< a from both roof edge and ridge	3.0
Upwind roof edges	RA1	All	$A \leq a^2$	< a	1.5
	RA2	All	$A \leq 0.25a^2$	< 0.5a	2.0
Downwind side of hips and ridges of roofs with pitch ≥ 10°	RA3	All	$A \leq a^2$	< a	1.5
	RA4	All	$A \leq 0.25a^2$	< 0.5a	2.0
Side walls near windward wall edges	SA1	≤ 1	$A \leq a^2$	< a	1.5
	SA2		$A \leq 0.25a^2$	< 0.5a	2.0
	SA3	> 1	$A \leq 0.25a^2$	> a	1.5
	SA4		$A \leq a^2$	< a	2.0
	SA5		$A \leq 0.25a^2$	< 0.5a	3.0
All other areas	—	All	—	—	1.0

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

NOTE 2 The building aspect ratio (r) is defined as the average roof height (h) divided by the smaller of b or d.

RC2 case introduced for downwind corners of roofs with pitch ≥ 10°.

DYNAMIC RESPONSE FACTOR (C_{dyn})

Section 6 Dynamic response factor

6.1 Introduction

This Section covers those structures subject to dynamic excitation by wind, defined in Clauses 6.2 and 6.3.

This Standard does not provide dynamic response factors for the following:

(a) Structures with a first mode natural frequency less than 0.2 Hz, heights greater than 200 m, or whenever significant coupling is evident in the first three modes of vibration.

(b) Buildings and horizontal structures that have two fundamental modes of sway within 10 % of each other and are both less than 0.4 Hz.

(c) Buildings with a height to minimum overall width ratio of greater than 6.

(d) Linked or connected buildings where the connection extends above (h/5), and the first-mode natural frequency is less than 0.5 Hz.

(e) Cases that involve excitation resulting from aerodynamic interference from other structures.

(f) Roofs supported on two or more sides with natural frequencies less than 0.8 Hz.

(g) Cantilevered roofs with a first mode natural frequency of less than 0.5 Hz.

(h) Facade elements, such as sunshades.

6.2 Structures for which C_{dyn} = 1.0

A dynamic response factor (C_{dyn}) shall be taken to equal 1.0 for the following cases:

(a) Buildings and free-standing towers, where the natural frequency of the first mode of vibration is greater than 1 Hz.

(b) Poles and chimneys with height to average diameter aspect ratio less than 5.

(c) Ground-mounted solar panels with natural frequencies greater than 5 Hz.

Further clarification on the applicability of Section 6 to structures with "unusual" characteristics.

Introduction of basic geometric/dynamic parameters for which C_{dyn} = 1.0 can be assumed.

Table 6.1 — Turbulence intensity (I_z)

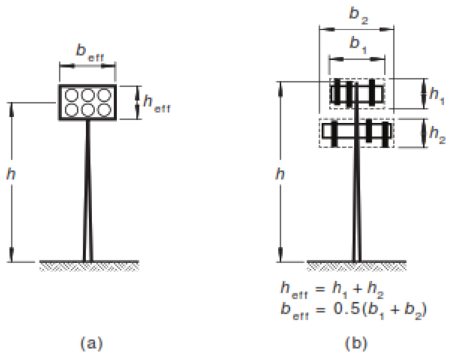
Height (z) m	Terrain Category 1	Terrain Category 2	Terrain Category 2.5	Terrain Category 3	Terrain Category 4
≤ 5	0.128	0.196	0.234	0.271	0.342
10	0.117	0.183	0.211	0.239	0.342
15	0.112	0.176	0.201	0.225	0.342
20	0.109	0.171	0.193	0.215	0.342
30	0.104	0.162	0.183	0.203	0.305
40	0.101	0.156	0.178	0.195	0.285
50	0.099	0.151	0.170	0.188	0.270
75	0.095	0.140	0.158	0.176	0.248
100	0.092	0.131	0.149	0.166	0.233
150	0.089	0.117	0.134	0.150	0.210
200	0.087	0.107	0.123	0.139	0.196

Updated turbulence intensity (I_z) values for Terrain Category 1.

DYNAMIC RESPONSE FACTOR FOR TOWERS, POLES AND MASTS WITH HEADFRAMES (C_{dyn})

6.4.2 Dynamic response factor for towers, poles and masts with head frames (C_{dyn})

This Clause is applicable when headframes have effective projected area that is greater than the total projected area of the supporting tower, pole or mast. For the supporting tower, pole or mast with a projected area greater than the headframe, see Clause 6.4.1.



Key

h_{eff} = the sum of the heights of the headframes/ancillaries

h = height of the pole/tower/mast supporting the headframe(s)

b_{eff} = the average width of headframes/ancillaries considered in h_{eff}

Figure 6.2 — Dimensions of a pole or mast with headframe

The dynamic response factor, C_{dyn}, for the along-wind response of towers, poles or masts with large headframes, such as lighting towers, shall be calculated from Equation 6.2(7):

$$C_{dyn} = \frac{1 + 2I_h \sqrt{g_v^2 + \frac{g_v^2 S' E_t}{\zeta}}}{1 + 2g_v I_h} \qquad 6.2(7)$$

where S' is the effective size reduction factor given by Equation 6.2(8):

$$S' = \frac{1}{\left[1 + \frac{3.5 n_a h_{eff}\left(1 + g_v I_h\right)}{V_{des,\theta}}\right]\left[1 + \frac{4 n_a b_{eff}\left(1 + g_v I_h\right)}{V_{des,\theta}}\right]} \qquad 6.2(8)$$

where h_{eff} and b_{eff} are the vertical and horizontal dimensions of the headframe (see Figure 6.2).

All other terms in Equations 6.2(7) and 6.2(8) are defined in Clause 6.4.1.

A revised equation for C_{dyn} for towers, poles and masts to better distinguish their behaviour from that of a tall building.

This section only applies to towers, poles and masts for which the projected area of headframes and attachments (e.g. lights, antennas) exceeds the projected area of the supporting structure, and will typically apply to lighting and telecommunications structures.

For other structures where this condition does not apply (e.g. chimneys and stacks) are instead required to meet the requirements of Clause 6.4.1 (i.e. the old way).

DYNAMIC RESPONSE FACTOR FOR TOWERS, POLES AND MASTS WITH HEADFRAMES (C_{dyn})

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

6.5.3.1 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 from Equations 6.3(10) and 6.3(11):

$$y_{max} = g_c \sigma_{y,max} \qquad 6.3(10)$$

$$\left(\frac{\sigma_{y,max}}{b_t}\right)^2 = c_1 + \sqrt{c_1^2 + c_2} \qquad 6.3(11)$$

where

b_t = average breadth of the top third of the structure

g_c = $\sqrt{2}\left(1+1.2\arctan(0.75(Sc/(4\pi K_a))^4)\right)$

c_1 = $\frac{a_L^2}{2}\left(1-\frac{Sc}{4\pi K_a}\right)$

c_2 = $\frac{a_L^2}{K_a}\frac{\rho_{air}b_t^2}{m_t}\frac{C_c^2}{St^4}\frac{b_t}{h}$

St = Strouhal number, taken as 0.20 for a circular cross-section

Sc = Scruton number given by:

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

where

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

NOTE For structural damping, values of ζ should be:

(a) for unlined welded steel poles and stacks: 0.002 of critical; and

(b) for reinforced concrete towers and chimneys: 0.005 of critical.

Equation 6.3(11) requires three aerodynamic constants (a_L, K_a and C_c) and for circular cross-sections these shall be calculated from Table 6.2.

Table 6.2 — Constants for determination of the response to vortex shedding

Constants	Re ≤ 10^5	Re = 5 × 10^5	Re ≥ 10^6
a_L	0.4	0.4	0.4
$K_{a,max}$	2	0.5	1
C_c	0.02	0.005	0.01

NOTE 1 For circular cylinders the constants C_c and $K_{a,max}$ are assumed to vary linearly with the logarithm of the Reynolds number for 1 × 10^5 < Re < 5 × 10^5 and for 5 × 10^5 < Re < 1 × 10^6.

NOTE 2 K_a decreases with increasing turbulence, using $K_a = K_{a,max}$ provides larger estimates of displacements or $K_a (I_h) = K_{a,max} \times \max(1 - 3 \times I_h, 0.25)$.

NOTE 3 $Re = \dfrac{bV_{des,\theta}}{15\times10^{-6}\left(1+g_v I_h\right)}$

Updated equations for crosswind tip deflection of chimneys, masts and poles.

APPENDIX A – MULTI-SPAN BUILDINGS

A.2 Multi-span buildings ($\alpha < 60°$)

External pressure coefficients ($C_{p,e}$) for the multi-span buildings shown in Figures A.1 and A.2 for wind directions $\theta = 0°$ and $\theta = 180°$ shall be obtained from Table A.1 or Table A.2.

Where two values are listed for pressure coefficients in Tables A.1 and A.2, 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). However, [−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 A.1 — External pressure coefficients ($C_{p,e}$) for multi-span buildings — Pitched roofs

Surface reference (see Figure A.1)				
A	***B***	***C***	***M***	***Y***
+0.7, +0.2	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°	+0.1, −0.2
			−0.5 and ± 0.3 for α ≥ 10°	

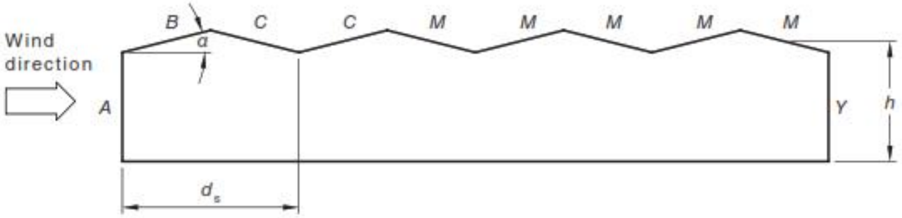


Figure A.1 — External pressure coefficients ($C_{p,e}$) for multi-span buildings — Pitched roofs

Table A.2 — External pressure coefficients ($C_{p,e}$) for multi-span buildings — Saw-tooth roofs

Wind direction (θ) degrees	Surface reference (see Figure A.2)								
	A	***B***	***C***	***D***	***M***	***N***	***W***	***X***	***Y***
0	+0.7, −0.1	−0.9, −0.4	−0.9, −0.4	−0.5, +0.2	−0.5, +0.5	−0.5, +0.3	−0.3, +0.5	−0.4, −0.2	−0.2, +0.1
180	±0.2	±0.2	−0.3, +0.2	±0.2	−0.4, 0.0	−0.4, 0.0	−0.7, −0.3	−0.3, +0.1	+0.7, −0.1

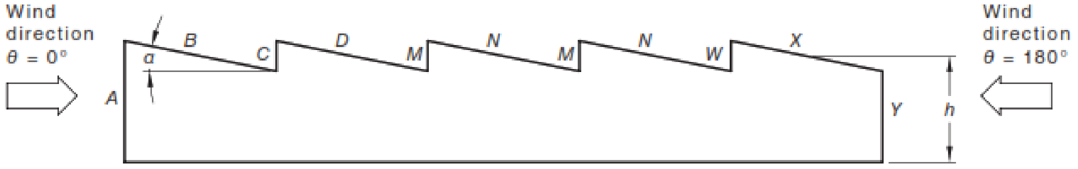


Figure A.2 — External pressure coefficients ($C_{p,e}$) for multi-span buildings — Saw-tooth roofs

Updated $C_{p,e}$ values for multi-span buildings.

APPENDIX B – GROUND MOUNTED SOLAR PANELS

B.6.2 Panels mounted on ground

The use of this Section shall be limited to the calculation of wind loads on solar panel arrays with the following restrictions as shown in Figure B.11:

(a) Panels attached to a ground mounted frame with aspect ratios $2 \le d/h \le 5$ and $b/d \ge 2$.

(b) Panels attached to the frame at an inclination to ground, $\alpha \le 30°$.

(c) Panel arrays with a spacing of $3.5 \le s/h \le 10$.

(d) Panels with a minimum gap between the underside of the panel and the ground surface $c/h \ge 0.2$.

NOTE Tracking, ground-mounted, solar panel arrays have occasionally experienced severe vibrations due to aeroelastic forces (flutter). These effects are not covered in this Standard; specialist advice should be sought.

The aerodynamic shape factor (C_{shp}) for net pressures normal for solar panels, satisfying the above conditions, shall be calculated from Equation B.6:

$$C_{shp} = C_{p,n} K_a K_\ell \qquad \text{B.6}$$

where

$C_{p,n}$ = net pressure coefficient acting normal to the surface, obtained for the windward half of a panel array ($C_{p,w}$) or net pressure coefficient for the leeward half of a panel array ($C_{p,\ell}$), as given in Tables B.13 and B.14 (positive indicates net downward pressure). For $\theta = 90°$, the array pitch is effectively zero, and Tables B.13 and B.14 with $\alpha = 0°$ shall be used to determine $C_{p,n}$.

K_a = area reduction factor, as given in Clause B.1.2

K_ℓ = local pressure factor, as given in Clause B.1.3

NOTE 1 The factor K_p does not appear in this Equation as it is taken as 1.0.

NOTE 2 Areas A1, A2, A3, B1, B2, B3 apply to the upwind row of panels. Areas C1, C2, C3, D1, D2, D3 apply to all shielded downwind panels.

New section for ground mounted solar panels.

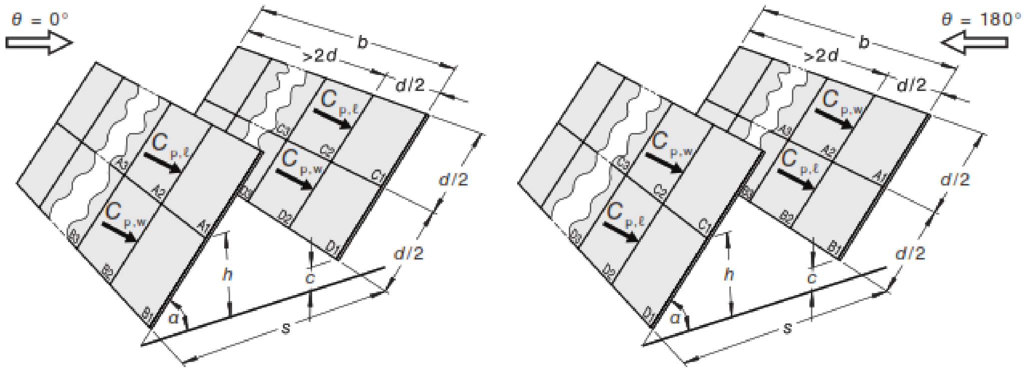


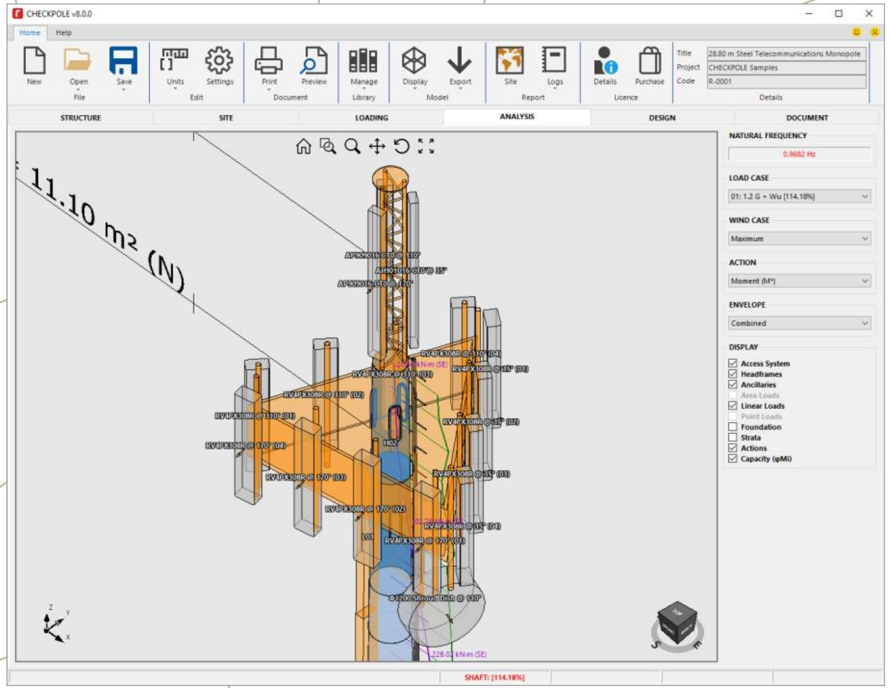
Figure B.11 — Solar panel arrays

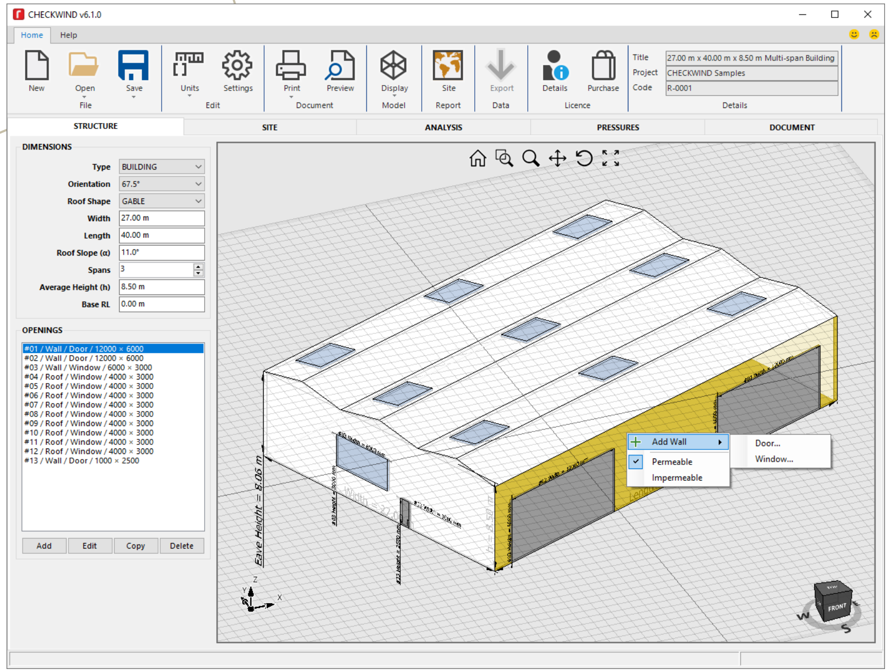
Table B.13 — Net pressure coefficients ($C_{p,n}$) for solar panel array — $\theta = 0°$ (see Figure B.11)

Panel pitch (α) degrees	θ = 0 degrees							
	$C_{p,w}$		$C_{p,l}$		$C_{p,w}$		$C_{p,l}$	
	B1	B2, B3	A1	A2, A3	D1	D2, D3	C1	C2, C3
0	0.45	0.45	0.25	0.10	0.40	0.25	0.25	0.10
15	1.20	1.20	0.80	0.45	1.40	0.80	0.90	0.40
20	1.30	1.20	0.80	0.45	1.50	0.75	0.90	0.45
25	1.45	1.35	0.95	0.60	1.60	0.85	1.00	0.55
30	1.50	1.25	0.95	0.70	1.70	0.85	1.10	0.65

Table B.14 — Net pressure coefficients ($C_{p,n}$) for solar panel array — $\theta = 180°$ (see Figure B.11)

Panel pitch (α) degrees	θ = 180 degrees							
	$C_{p,w}$		$C_{p,l}$		$C_{p,w}$		$C_{p,l}$	
	A1	A2, A3	B1	B2, B3	C1	C2, C3	D1	D2, D3
0	−0.50	−0.55	−0.35	−0.20	−0.50	−0.35	−0.35	−0.15
15	−1.20	−1.40	−0.60	−0.85	−1.40	−1.45	−0.70	−0.65
20	−1.40	−1.45	−0.75	−0.90	−1.40	−1.40	−0.70	−0.70
25	−1.50	−1.45	−0.75	−0.95	−1.50	−1.35	−0.75	−0.80
30	−1.60	−1.50	−0.80	−0.95	−1.55	−1.30	−0.90	−0.85





THANK YOU FOR YOUR TIME

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