AUSTRALIAN STEEL INSTITUTE
STEEL SHED GROUP

Design Guide

Portal Frames

Steel Sheds & Garages

This Guide is applicable to steel framed and predominantly steel clad sheds & garages manufactured from materials certified or tested for compliance with Australian Standards

March 2009

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FOREWORD

The Steel Shed Group has produced this guide to promote excellence in the design of steel sheds and garages, based on building regulations and Australian and New Zealand standards and to encourage uniformity across all shed designers and manufacturers. Classification and BCA importance levels, design actions, analysis, design, testing, as well as other considerations such as good detailing, durability, and corrosion are all covered.

The Cyclone Testing Station has a strong interest in encouraging a consistent and knowledgeable standard of design and construction of sheds to resist wind loads as part of an on-going commitment to improve the resilience of low-rise buildings to severe winds. Therefore the use of this guide is a perfect fit with the Station's mission to reduce and mitigate the risk and costs to communities from wind damage.

As Manager of the Cyclone Testing Station, my congratulations to the Steel Shed Group for producing this Guide. I also commend the use of this Guide to all those involved in the shed and garage industry, as doing so will reduce the risk of wind damage to these buildings and their contents and also lead to safer and more resilient communities.

Mr. Cam Leitch
Manager Cyclone Testing Station
James Cook University, Queensland

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CHAPTER 1: SHED BASICS

1.1 INTRODUCTION

PURPOSE

This Guide outlines the principles for the design of *freestanding steel sheds, garages and similar buildings* for construction in Australia. It explains how structural designers should apply existing design criteria and concepts to the design of steel sheds falling within a defined scope. It applies to buildings with structural frames made predominantly from cold-formed steel and clad predominantly with steel wall and roof sheeting. It promotes consistent interpretation of critical requirements for the structural performance of steel sheds. It does not replace the Building Code of Australia (BCA), its referenced standards and other engineering texts but should be read in conjunction with them.

CONTRAINDICATIONS

Pharmaceutical manufacturers use the term “contraindications” to explain when you should *not* use their product. This Guide has contraindications too. It is not appropriate for the design of:

- Habitable buildings of any kind, and any structures attached to them
- Silos and similar produce stores where stored contents applies vertical or lateral wall loads
- Buildings larger or smaller than the dimensions described in the Scope

Please read the Scope carefully and *do not extrapolate the design criteria and procedures to a wider range of buildings*. For habitable buildings, the NASH Standard – Residential and Low-rise Steel Framing, Part 1: Design Criteria and related publications should be consulted. For low-rise commercial buildings, refer to either the NASH Standard or other relevant standards and publications.

ABOUT THE STEEL SHED GROUP

The Steel Shed Group promotes compliance for engineering standards for the steel shed industry via technical publications, education and creating awareness.

Membership of Steel Shed Group is open to all companies and individuals involved in the design, certification, manufacture and supply of Australian steel sheds and the materials from which they are manufactured.

Applicants are required to meet the membership criteria of Steel Shed Group.

1.2 WHAT IS A SHED?

“Shed” is a very common term in the community. Buildings fitting the general description of “shed” may be used for a wide range of purposes. According to the Macquarie Dictionary, a shed is:

1. A slight or rough structure built for shelter, storage, etc.
2. A large, strongly built structure, often open at the sides or end.

Whilst *private garage* is a defined term in the BCA, shed, carport, workshop and farm building are not. Structural designers cannot rely solely on a proposed building’s description. They must consider whether the building will be accessible to the public, used as a factory or workplace, as an assembly point or even as an emergency refuge.

For the purposes of this Design Guide:

- ***A shed is any freestanding non-habitable general purpose building used for domestic, commercial, industrial or agricultural purposes***. A *residential* shed is one constructed on a residential allotment and used predominantly for private, domestic purposes.
- ***A garage is a special-purpose freestanding building designed to shelter vehicles and with at least one vehicle-sized door***. Garages may be residential or non-residential. All other vehicle shelters, including those attached to buildings, are carports and are not covered by this Guide.

Buildings supplied by Australian shed manufacturers are frequently used as BCA Class 10a buildings. However, many may be used or adapted as Class 6, 7, 8 or 9b buildings, provided they are designed or modified accordingly. *The actual use of a building – not its physical appearance or commercial description - determines its classification.*

Whilst the majority of “sheds” will be easy to classify based on intended actual use, importance level is an even more significant consideration. Importance level is a function of the potential human hazard and public impact of building failure. Most “sheds” will have Importance Level (IL) 1 or 2, but two specific examples illustrate common exceptions:

- An open or partially open shed used as a shade shelter in a large school: IL = 3.
- A garage used for a bush fire service vehicle: IL = 4.

The classification and importance level of a specific building are regulatory matters for the relevant Building Authority. Depending on the building classification and importance level, the designer will make design decisions taking into account the performance requirements or building solutions of the BCA.

- *All sheds should be designed, supplied and constructed in accordance with the BCA and any specific local regulations.*
- ***Regardless of their importance level or classification, buildings should not fail when subjected to the ultimate loading events for which they are certified to be designed.***
- *Each building and its location are unique. The designer must ascertain the appropriate classification and importance level to determine the design actions on the structure.*
- *“Generic” designs should take into account, and clearly disclose in documentation and literature, the most adverse use for which a building may be sold or recommended, or is reasonably likely to be used.*

The next two sections discuss building classifications and importance levels in more detail.

1.3 BCA CLASSIFICATION OF SHED USES

The actual use of a building – not its physical appearance or commercial description - determines its *classification*. Only when a building's *intended use and physical location* are known can its *importance level* be assessed by the designer.

See Appendix 1 for a summary of Building Classifications.

PHYSICAL DESCRIPTION	USES & CLASSIFICATIONS
 **Rural Shed**	• Where used only for storage purposes, farm sheds are usually Class 10a. • Where selling to the public takes place, Class 6 would apply and for wholesaling, Class 7. If used for manufacturing, Class 8.
 **Domestic Garage**	• Domestic garages are Class 10a, even if used for hobbies and other domestic purposes.
 **Domestic Shed**	• Same as farm shed: where used only for storage purposes, Class 10a applies. • Where a home industry is involved, Class 6 or 7 would apply. If used for manufacturing, Class 8.
 **Commercial Shed**	• Where selling to the public takes place, Class 6 would apply and for wholesaling, Class 7. If used for manufacturing, Class 8

 **School Shelter**	• Generally people shelters are Class 9b.

1.4 IMPORTANCE LEVELS

Building authorities, on behalf of the community, regulate how strongly buildings are constructed to resist the loads they are expected to experience, and what risk of structural failure is acceptable for various types and uses of building. The national regulator, the Australian Building Codes Board, expresses this community interest via the Importance Level in the BCA.

The selection of design actions based on Importance Level is a regulatory obligation, not a discretionary engineering choice. The community expects, and designers and suppliers should strive for, *uniform risk of failure for buildings of equal importance*. BCA 2008 defines four importance levels:

TABLE 1 BCA IMPORTANCE LEVEL DEFINITIONS

Level 1	Buildings or structures presenting a low degree of hazard to life and other property in the case of failure
Level 2	Buildings or structures not included in importance levels 1, 3 or 4
Level 3	Buildings or structures that are designed to contain a large number of people
Level 4	Buildings or structures that are essential to post-disaster recovery or associated with hazardous facilities

Source: BCA 2008, Part B1.2

The BCA explains that importance levels:

- Apply to *structural safety only*, not to serviceability or functionality;
- Are a function of both *hazard to human life* and *public impact* of building failure, and
- Must be assigned on a case by case basis.

This last point is particularly important in the shed industry, where supply of standard or generic buildings is common. Terms such as *shed*, *garage* and *workshop* mean different things in different contexts, but it is the *specific use* to which the building will be put and its *physical location* in relation to other development that determine the community consequences of building failure. *Once assigned, the importance level determines the magnitude of the design actions that must be applied in assessing structural resistance in the strength limit state.*

In addition to these basic principles, the BCA offers further guidance to building authorities and designers in assigning importance levels. There is no prescriptive list of all possible building uses, just general guidance on applying the principles. The following table appears in the Guide to BCA **2008:**

TABLE 2 IMPORTANCE LEVEL MATRIX

Building Importance Level					
		Impact On The Public			
		Low	Moderate	Substantial	Extreme
Hazard To Human Life	Low	**1**	**2**	**2**	**3**
	Moderate	**2**	**2**	**3**	**3**
	Substantial	**2**	**3**	**3**	**4**
	Extreme	**3**	**3**	**4**	**4**

Source: Guide to the BCA 2008 Part B1.2

The most obvious conclusion from the table is that only buildings involving *both* low human hazard *and* low public impact may be assigned the least severe importance level 1. The Guide gives examples of importance level 1 as farm buildings, isolated minor storage facilities and minor temporary facilities. ***All forms of low-rise residential construction, including both dwellings and outbuildings such as garages and sheds, should be assigned importance level 2.***

One complication in assigning importance levels is worth a particular mention: what happens when a building use changes, or when through subdivision a previously isolated building is brought closer to adjoining allotments or to other buildings? This is fundamentally a planning matter for building authorities, rather than a commercial or technical matter for shed designers and manufacturers. Planning schemes may identify rural lands which will be subject to development in the near future and on which it may be a requirement to apply importance level 2 (or higher) from the outset. It reinforces that the assignment of importance levels by the designer must always be specific to the use and location of the building, and should not rely solely on generic descriptions or classification.

TABLE 3 IMPORTANCE LEVEL EXAMPLES

BUILDING DESCRIPTION	BCA CLASS	FAILURE CONSEQUENCES		IMPORTANCE LEVEL
		HUMAN HAZARD	PUBLIC IMPACT	
Farm building	10a	Low	Low	1
Residential shed/garage	10a	Mod	Mod	2
Small school shade structure	9b	Mod	Mod	2
Produce sales building	6	Mod	Mod	2
Shearing shed	8	Mod	Low	2
Large commercial storage warehouse	7	Mod	Sub	3
Large (250+) school assembly shelter	9b	Sub	Sub	3
Shed housing hospital emergency generator	10a	Sub	Ext	4
Emergency vehicle garage	10a	Sub	Ext	4

See BCA 2008 Table B1.2a and Guide to the BCA 2008 Part B1.2 for more detailed guidance on importance levels.

Importance Level 2 is the default level. It applies unless a lower level is justified, or a higher level is required, according to BCA risk assessment guidelines.

1.5 SCOPE OF THIS GUIDE

General

This Design Guide applies to steel portal framed and predominantly steel clad sheds and garages made from materials certified or tested for compliance with Australian Standards.

Primary Materials

- Cold formed steel sections of thickness and grade falling within AS/NZS 4600
- Claddings designed and installed in accordance with AS 1562.1

Other Members & Components

- RHS sections of thickness and grade falling within the scope of AS 1163
- Secondary hot rolled steel components falling within the scope of AS 4100

Situations

- Regions A, B, C & D
- BCA Class 6, 7, 8, 9b and 10a buildings
- Terrain Category 1, 2 and 3 (with coefficients for TC 2.5 included for convenience)
- Importance Level 1, 2, 3 & 4

Dimensions & Configuration

- Rectangular, L-shaped or T-shaped buildings
- Single portal frame spans 3 to 36 metres
- Minimum area 10 square metres
- Maximum height 10 metres
- Multi-level roofs

Specific Exclusions

- Corrosive environments (see also Durability section)
- Bulk storage of solids, grains or liquids (where stored material applies live lateral load to cladding or structure)
- Buildings with pre-tensioned elements
- Buildings with crane loads
- Buildings with brittle walls (masonry)
- Buildings with mezzanine floors and/or partitions
- Buildings where dynamic wind response is a consideration
- Ice actions

- Liquid pressure actions

1.6 MATERIALS & PROCESSES

The BCA requires that “every part of a building must be constructed in an appropriate manner to achieve the requirements of the BCA, using materials that are fit for the purpose for which they are intended” [BCA 2008 Clause A2.1].

The two most common steels used for steel shed structures are cold rolled metallic coated steel strip to AS 1397 and steel hollow sections to AS 1163. Other steels may be used provided they meet the requirements of AS 4100 or AS/NZS 4600.

Both AS 4100 and AS/NZS 4600 significantly down rate the design values of unidentified steels and place limitations on their use. ***<u>These limitations should be considered where the source or quality of steel is unknown.</u>***

1.7 STANDARDS AND OTHER REFERENCES

1. AS/NZS 4600:2005
   Cold-formed steel structures
2. AS/NZS 1170.0:2002
   Structural design actions - General principles
3. AS/NZS 1170.1:2002
   Structural design actions - Permanent, imposed and other actions
4. AS/NZS 1170.2:2002
   Structural design actions - Wind actions
5. AS/NZS 1170.3:2003
   Structural design actions - Snow and ice actions
6. AS 1170.4-1993
   Minimum design loads on structures (known as the SAA Loading Code) - Earthquake loads
7. AS/NZS 1170.0 Supp 1:2002
   Structural design actions - General principles -Commentary (Supplement to AS/NZS 1170.0:2002)
8. AS/NZS 1170.1 Supp 1:2002
   Structural design actions - Permanent, imposed and other actions - Commentary (Supplement to AS/NZS 1170.1:2002)
9. AS/NZS 1170.2 Supp 1:2002
   Structural design actions - Wind actions - Commentary (Supplement to AS/NZS 1170.2:2002)
10. AS/NZS 1170.3 Supp 1:2003
    Structural design actions - Snow and ice actions - Commentary (Supplement to AS/NZS 1170.3:2003)
11. AS 1170.4 Supp 1-1993
    Minimum design loads on structures (known as the SAA Loading Code) - Earthquake loads - Commentary (Supplement to AS 1170.4-1993)
12. AS 2870 – 1996 Residential Slabs and Footing Code
13. AS 4100
14. AS 1163
15. AS 1111.1-2002
16. AS 1252-1996
17. AS 3600
18. AS 1562.1
19. Manufacturers specifications and design capacity tables
20. G. J. Hancock: Design of Cold-formed Steel Structures
21. ASI: Connection Capacity Tables
22. NASH Handbook – Residential and Low-rise Steel Framing

1.8 DEFINITIONS

Action – the cause of stress, dimensional change, or displacement in a structure or a component of a structure

Action effect or load effect – the internal force, moment, deformation, crack, or like effect caused by one or more actions

Bend – portion adjacent to flat elements and having maximum inside radius-to-thickness ratio (r_i/t) of 8

Braced member – one for which the transverse displacement of one end of the member relative to the other is effectively prevented

Capacity (Strength reduction) factor – a factor used to multiply the nominal capacity to obtain the design capacity

Design action effect or design load effect – the action effect or load effect calculated from the design actions or design loads

Design action or design loads – the combination of the nominal actions or loads and the load factors, as specified in the relevant loading Standard

Design capacity – the product of the nominal capacity and the capacity (strength reduction) factor

Distortional buckling – a mode of buckling involving change in cross-sectional shape, excluding local buckling

Generic – a building or range of buildings designed to withstand specific actions but without reference to a specific site

High – Tensile Steel - steel with a yield stress of 450 MPa or higher

Limit state – any limiting condition beyond which the structure ceases to fulfil its intended function

May – indicates the existence of an option

Nominal capacity – the capacity of a member or connection calculated using the parameters specified in this Standard

Serviceability limit state – a limited state of an acceptable in service condition

Strength limit state – a limit state of collapse or loss of structural integrity

Tensile strength – the minimum ultimate strength in tension specified for the grade of steel in the appropriate Standard

Thickness – the base steel thickness (t), exclusive of coatings

Yield stress - the minimum yield stress in tension specified for the grade of steel in the appropriate Standard

CHAPTER 2: ACTIONS

2.1 WIND ACTIONS

GENERAL

Ignoring the special cases of earthquakes and crane loads, most actions on structures are due to gravity and wind. Estimating wind actions is all about probability, not certainty. The speed of the strongest wind that can *ever* blow is not known, but the longer the time interval, the higher the chance of a stronger wind. By taking measurements over many decades in many places, there is now reasonable scientific agreement as to the probability of a particular wind speed occurring in defined geographic regions.

REGULATORY MATTERS

The BCA requires that regional wind speeds of specific probability be used for building design. The more important the building, the less the allowable risk that the design wind speed will be exceeded in any one year and therefore the higher the wind speed required in design. **Regardless of their importance level or classification, buildings should not fail when subjected to the wind event for which they are certified to be designed.**

DESIGN STANDARDS FOR WIND ACTIONS

Steel Shed Group does not encourage the use of AS 4055 *Wind loads for housing* to determine wind actions on sheds and garages, or to specify requirements or suitability. Although generally based on an Importance Level of 2, which is conservative for many rural sheds, AS 4055 permits the use of net pressure coefficients which are valid only for the configuration of openings and overall permeability typical of houses.

AS 4055 was developed for "houses as a group or large numbers of buildings". Steel sheds are highly wind sensitive structures, vulnerable to inappropriate design or siting based on simplified or invalid assumptions about critical design factors. Their structural design, and the suitability recommendations of their suppliers, should be based on actual expected service conditions for individual buildings rather than on generalised assumptions.

Steel Shed Group recommends that structural designers of sheds, garages and similar buildings should use AS/NZS 1170.2.2002 for all wind action computations, taking into account all relevant factors and applying, in an informed and fully transparent way, any simplifications or concessions appropriate to an individual building design.

OBSELETE STANDARDS

In some localities, the use of the "W" wind classification system has persisted. This system is defined in an early edition of AS 4055 and is based on the "permissible stress" design methodology, as described in AS 1170 – 1989. The "W" classification is obsolete and should not be used to describe the suitability of buildings for specific wind conditions.

With the implementation of the AS/NZS 1170 series of new standards in 2002, the previous wind load code AS 1170.2 - 1989 was superseded. Amendment No. 12 to the Building Code of Australia allowed the use of the old loading code series, clarifying that the old codes could be used but only in isolation. Similarly the new codes could be used, but only in isolation, thus preventing the use of old & new codes at the same time. The *farm structures code* AS 2867 was withdrawn from the BCA in 2007.

Steel Shed Group recommends that designs prepared and certified to AS 1170.2 - 1989 and/or AS 2867 should not be quoted or constructed unless they have been re-certified to current limit state design standards. New designs should be based only on current standards.

COMMON MISCONCEPTIONS REGARDING WIND ACTIONS

Sheds and garages are not automatically Importance Level 1.

The importance of a building should be correctly assessed in all cases according to the BCA and its guidelines. Domestic sheds, garages and outbuildings are always Importance Level 2, which is the default level.

Three-sided sheds are not enclosed buildings.

These sheds should always be designed for the appropriate dominant opening internal pressure.

A Topography Multiplier of 1.0 is not conservative and should not be the default value.

This could be a very unsafe assumption. Topography should be properly assessed in all cases and design assumptions clearly stated in documentation.

Trees and other vegetation do not provide shielding.

Only buildings provide shielding, and only when located in the upwind zone specified in AS/NZS 1170.2 on ground of less than 0.2 gradient. Except in fully developed suburban areas where a value of 0.85 may be used, the Shielding Multiplier should be 1.0 unless proven otherwise.

REGIONAL WIND SPEED

Selection of exceedance risk based on importance level

The annual probability of exceedance for wind action should be determined from Table 4 for the selected importance level and region type (non-cyclonic or cyclonic).

TABLE 4 ANNUAL PROBABILITY OF EXCEEDANCE FOR WIND

IMPORTANCE LEVEL	ANNUAL PROBABILITY OF EXCEEDANCE FOR WIND ACTION	
	NON-CYCLONIC	CYCLONIC
1	1:100	1:200
2	1:500	1:500
3	1:1000	1:1000
4	1:2000	1:2000

Source: BCA Table B1.2b

Selection of Regional Wind Speed V_R based on exceedance risk

The regional wind speed should be determined from Table 5 for the appropriate probability of exceedance and region.

TABLE 5 REGIONAL WIND SPEEDS

ANNUAL PROBABILITY OF EXCEEDANCE R	REGIONAL WIND SPEED V_R			
	Region A (2)	**Region B** (2)	**Region C**	**Region D**
1:20 (1) (2)	37	38	45	51
1:50 (2)	39	44	52	60
1:100	41	48	59 (3)	73 (4)
1:200	43	52	64 (3)	79 (4)
1:500	45	57	69 (3)	88 (4)
1:1000	46	60	74 (3)	94 (4)
1:2000	48	63	77 (3)	99 (4)

Source: AS/NZS 1170.2, Table 3.1 and Clause 3.4

Notes

1. V_{20} is used for serviceability limit state. Refer AS/NZS 1170.0 Appendix C.
2. $F_C = F_D = 1.0$ for the V_{20} and V_{50} and all Region A & B wind speeds.
3. $F_C = 1.05$ has been applied to Region C velocities $V_{100} – V_{2000}$
4. $F_D = 1.10$ has been applied to Region D velocities $V_{100} – V_{2000}$

LOCAL FACTORS

Selection of wind direction multiplier M_d

In specific situations, it is acceptable to take into account the directionality of maximum wind speed.

The wind direction multiplier (M_d) should be determined as follows:

- For buildings in Region A:
  - Where the final orientation of the building is *unknown*, M_d = 1.00.
  - Where the final orientation of the building is *disclosed in design documentation*, M_d should be determined from AS/NZS 1170.2 Table 3.2.
- For buildings in Regions B, C and D:
  - For forces on complete buildings and major structural elements resisting collapse, M_d = 0.95.
  - For wind actions on all other components, M_d = 1.0.

Selection of terrain/height multiplier $M_{z,cat}$

Close to ground level, the wind speed is reduced by its interaction with surface roughness. This effect reduces with height above the ground, and disappears almost completely above about 25 metres.

Because of this effect, wind speed measurements are standardised to a height of 10 metres in Terrain Category 2. The terrain/height multiplier ($M_{z,cat}$) is then applied to adjust the wind speed to alternative terrains and heights.

The multiplier varies with region, terrain category and building height, as defined in AS/NZS 1170.2 and shown in Table 6.

TABLE 6 TERRAIN/HEIGHT MULTIPLIERS

<table>
<tr><th rowspan="2">Building Height (m)</th><th colspan="4">Region A & B (Ultimate limit state)
All Regions (Serviceability limit state)</th><th colspan="3">Region C & D
(Ultimate limit state)</th></tr>
<tr><th>TC1</th><th>TC2</th><th>TC2.5</th><th>TC3</th><th>TC 1 & 2</th><th>TC2.5</th><th>TC 3</th></tr>
<tr><td><= 3</td><td>0.99</td><td rowspan="2">0.91</td><td rowspan="2">0.87</td><td rowspan="3">0.83</td><td>0.90</td><td>0.85</td><td rowspan="2">0.80</td></tr>
<tr><td>5</td><td>1.05</td><td>0.95</td><td>0.88</td></tr>
<tr><td>10</td><td>1.12</td><td>1.00</td><td>0.92</td><td>1.00</td><td>0.95</td><td>0.89</td></tr>
</table>

Source: AS/NZS 1170.2 Table 4.1(A). Linear interpolation may be used for intermediate values of height and terrain category. Building Height is defined in AS/NZS 1170.2 Fig 2.1.

TABLE 7 TERRAIN CATEGORIES

TC	Description
1	Exposed open terrain with few or no obstructions
2	Water surfaces, open terrain, grassland with few, well-scattered obstructions having heights generally from 1.5 m to 10 m.
2.5	Terrain with few trees, isolated obstructions such as agricultural land, cane fields or long grass, up to 600 mm high.
3	Terrain with numerous closely spaced obstructions 3 m to 5 m high such as areas of suburban housing.
4	Terrain with numerous large 10m to 30 m high closely spaced obstructions such as large city centres and well-developed industrial complexes.

Source: AS/NZS 1170.2 and AS 4055 (Cat 2.5)

***Note**: Generic shed documentation should include the above descriptions to clarify the conditions for which the design is suitable. Definition and interpolated values for TC 2.5 are included for convenience. Designers should evaluate the actual terrain conditions and select the appropriate multiplier in accordance with AS/NZS 1170.2.2002.*

Terrain Category 4 is the urban and heavy industrial category. In most cases of generic steel shed designs within the scope of this guide, there would be little advantage in designing specifically for TC 4.

SITE FACTORS

Selection of shielding multiplier M_s (simplified method)

The ***Shielding Multiplier*** is a *local development effect*. It *reduces* the design wind speed by taking into account the protection afforded by *upwind local buildings*. A Shielding Multiplier of 1.0 should be applied outside suburban areas unless a lower value is justifiable and supported by a competent site survey.

On *suburban sites* where all adjoining allotments are fully developed and the average upwind gradient is less than 0.2, a Shielding Multiplier of 0.85 may be applied as suggested in AS/NZS 1170.2 Supplement 1.

TABLE 8 SHIELDING MULTIPLIER

Shielding case	Description of windward obstructions	Shielding multiplier (M_s)
Suburban shielding	Garage or shed on suburban allotment with all adjoining allotments fully developed.	0.85
No shielding	All other cases including generic building designs	1.00

Source: AS/NZS 1170.2 and Supplement 1

Note: A more economical design for a specific building may be possible using AS/NZS 1170.2 Section 4.3.

Selection of topographic multiplier M_t

The ***Topographic Multiplier*** is a *local geographic effect*. It <u>increases</u> the design wind speed based on the gradient upwind of the site. The increase in wind pressure on an exposed building could be ***as much as 2.5 times*** the pressure it would otherwise experience. Any site on or near a hill, ridge or escarpment of any size must be properly evaluated to determine the Topographic Multiplier.

The only correct way to determine the Topographic Multiplier is to carry out an accurate topographic survey and apply the survey data to the formulae contained in AS/NZS 1170.2 Clause 4.4. Whatever value is calculated or selected by the designer, the value and its corresponding site description should be clearly stated on all documentation.

To determine whether the site of a proposed building is “at risk” of a topographic wind effect, the following guidelines may be applied:

1. Measure or estimate the height (H) of the hill, ridge or escarpment
2. Locate a point upwind from the crest which has a height of H/2, that is half the height of the crest
3. Measure or estimate the horizontal upwind distance (L) between the crest and the half-height point.
4. Calculate the value of H/(2L)

a. *If this value is **less than 0.05***, the hill, ridge or escarpment will have negligible effect on wind speed and may be ignored regardless of shed location.

b. *If this value is **greater than 0.05***, any building on the hill or ridge, on the upward slope of an escarpment or for some distance on the plateau beyond the escarpment will experience increased wind speed and will need to be designed accordingly.

WIND PRESSURES

Calculation of reference pressure q_u for strength limit state

$$q_u = 0.6(M_d \times V_R \times M_{z,cat} \times M_s \times M_t)^2/1000$$

Calculation of reference pressure q_s for serviceability limit state

$$q_s = 0.6(M_d \times V_{20} \times M_{z,cat} \times M_s \times M_t)^2/1000$$

PRESSURE COEFFICIENTS

Internal

The internal pressure coefficients selected by the designer depend mainly on the size, shape and orientation of the building and on the size and configuration of its openings. They also depend, in part, on the wind region in which the building is located.

AS/NZS 1170.2 Supplement 1 explains that in all regions "*in determining the most critical loading condition, the designer may use his discretion as to which opening can be relied upon to be closed, with closures capable of withstanding peak wind forces, at the critical loading conditions".* For example, a roller door with a design capacity of 1.0 kPa may only be assumed "closed" during peak wind events if the highest calculated design wind pressure on the door is no more than 1.0 kPa (including local pressure effects).

In some situations, it may be practical to improve the capacity of doors and windows with "wind locks" or other permanent strengthening devices. These locks may have secondary structural effects by changing the way wind force on the door curtain or window panel is transferred to the jambs. Designers should consider, in consultation with the client, whether the more economical solution would be to design the structure on the assumption that structurally critical doors and windows are fully or partially open during peak wind events.

In cyclonic regions C and D, a further requirement applies. Designers must consider the resistance of the entire building envelope – windows, doors, roof and wall cladding – to impact by flying debris. AS/NZS 1170.2 Clause 5.3.2 requires that "*internal pressure resulting from the dominant opening shall be applied, unless the building envelope (windows, doors and cladding) can be shown to be capable of resisting impact loading equivalent to a 4 kg piece of timber of 100 mm × 50 mm cross-section, projected at 15 m/s at any angle*". Evidence of successful testing, including any conditions or restrictions on product suitability, should be obtained from the manufacturer of each element of the building envelope. If satisfactory evidence cannot be obtained for all elements, the designer must select the most adverse internal pressure coefficients arising from failure of the impact susceptible elements. These coefficients are specified in AS/NZS 1170.2 Table 5.1(B).

Designers should be conscious that different parts of the building envelope may fail in different ways when subject to debris impact. For example, a window or skylight may shatter leading to a sudden increase in permeability, while a door or wall panel may suffer a small rupture leading to a negligible change in permeability. Any change in permeability through debris penetration must be considered in relation to the design permeability of the building in its normal configuration, taking into account gaps, vents, fixed openings etc.

External

Coefficients for walls vary with height (for side walls), depth-to-breadth ratio (for leeward walls), roof pitch (for leeward walls) and distance from the windward edge (for side walls, as a function of average building height).

Coefficients for roofs vary with the type, direction and pitch of each roof plane, distance from the windward edge and height-to-depth ratio.

A statistical combination factor may apply where wind pressures act simultaneously on two or more surfaces to produce action effects in a major structural member.

External pressure coefficients for all buildings in all situations should be selected from the appropriate tables in AS/NZS 1170.2 Clause 5.4.

PRESSURE FACTORS

Designers should take into account the following pressure factors where they apply to a particular design case.

Area reduction factor Ka

Concessional factor

Applies to calculation of actions due to tributary areas on roofs and side walls

Combination factor Kc

Concessional factor

Can be considered when wind pressures contribute simultaneously, on two or more surfaces, to the action effects in a major structural member.

Recognises the very low probability that wind pressures on separate surfaces will peak simultaneously.

Must not be applied to cladding, battens, purlins, girts or similar secondary elements.

Local pressure factor Kl

Amplification factor

Applies only to claddings, their directly supporting members and the immediate fixings of claddings and supporting members. This would include door and window framing when these members support cladding.

Permeable cladding factor Kp

Concessional factor

Applies to very specific cladding types not normally used in shed design

Frictional drag force F

An additional wind-related action effect to be considered

Applies to buildings of particular geometry

2.2 SNOW ACTIONS (F_{sn})

The majority of steel sheds and garages constructed in Australia will have their ultimate limit state governed by wind actions. However, in certain Alpine and Sub Alpine areas it is just as important to consider the effects of Snow Actions. To account for these actions in such areas is critical, especially considering that all of these areas are located in wind region A (resulting in the lowest wind pressures in Australia and hence the most lightweight and efficient designs if only wind actions are checked). ***Buildings should not fail when subjected to the snow event for which they are certified to be designed.***

As with estimating wind loads, the estimation of future snow actions is based on probability. The maximum snow action to *ever* be experienced in any specific area cannot be known, but through the use of data collected over time the probability of a certain snow loading event occurring or being exceeded, during a set period of time, can be calculated. Consequently, the magnitude of a snow loading event associated with a required probability of being exceeded can also be calculated.

The BCA requires that the annual probability of exceedance, used to determine the snow actions, be taken from the below table based on the importance level of the building.

TABLE 9 Selection of exceedance risk based on importance level for Snow Events

IMPORTANCE LEVEL	ANNUAL PROBABILITY OF EXCEEDANCE FOR SNOW ACTION
1	1:100
2	1:150
3	1:200
4	1:250

Source: BCA Table B1.2b

The BCA further requires that Snow Actions required to be resisted by a specific building be determined in accordance with AS/NZS 1170.3. AS/NZS 1170.3 defines Snow action (F_{sn}) as “the sum of the forces resulting from the accumulation of snow determined by applying the snow load(s) to appropriate areas of the structure.” The following sections of this design guide deal with the calculation of these snow loads.

Designers should be aware that compliance with this design guide does not ensure compliance with AS/NZS 1170.3. This guide contains no guidance on Ice Actions, which is included in AS/NZS 1170.3.

Snow Regions

Within Australia there are 4 regions where sub alpine or alpine conditions need to be considered for any Building Design. Within these regions, the altitude of the site is the dominant factor in assessing snow actions and must be known.

The four Alpine and Sub Alpine regions are defined in table 10.

TABLE 10 Alpine and Sub Alpine Regions of Australia (from AS1170.3 Clause 2.2)

Classification	Description	Alpine Altitudes	Sub Alpine Altitudes
AN	Northern Tablelands (Guyra Area)	NA	>600m
AC	Central Tablelands (Blue Mountains Area)	NA	>600m
AS	Southern Tablelands, NSW and Victoria (Snowy Mountains Area)	$h_0 \geq 1200$m	1200m$>h_0>$600m
AT	Tasmania (Central Highlands)	$h_0 \geq 900$m	900m$>h_0>$300m

Where h_0 = altitude above Australian Height Datum in metres.

Further clarification on the approximate locations of these snow regions can be found in AS1170.3 Figure 2.1. For exact location of snow region boundaries, altitude above the Australian Height Datum must be used.

Determination of Ground Snow Load (s_g)

The ground snow load must first be calculated before the determination of snow actions. Table 5.2 of AS1170.3 provides values for specific locations and Probability Factors, for other locations the ground snow load values can be calculated as shown below.
The first requirement is to determine the probability factor (k_p).

TABLE 11 Probability Factors (Adapted from AS1170.3 Section 5, Table 5.1)

Annual Probability of exceedance (P)	Probability Factor (k_p)
1:100	1.4
1:150	1.5
1:200	1.6
1:250	1.7

The calculation of Ground Snow Load differs in Alpine and Sub Alpine Regions.

TABLE 12 Ground Snow Load Calculations (Adapted from AS1170.3 Section 5)

Ground Snow Load Calculation			
Classification	**Alpine**	**Sub Alpine**	**k_l**
AN	NA	$s_g=k_pk_l[2.8h_0/1000-1.2]$	0.2
AC	NA	$s_g=k_pk_l[2.8h_0/1000-1.2]$	0.7
AS	$s_g=k_pk_t(h_0/1000)^{4.4}$	$s_g=k_pk_l[2.8h_0/1000-1.2]$	1.0
AT	$s_g=k_pk_t[(h_0+300)/1000]^{4.4}$	$s_g=k_pk_l[2.8(h_0+300)/1000-1.2]$	1.6
γ	4.3kN/m^3	2.9kN/m^3	NA

Where k_l = multiplying factor for latitude.
k_t = multiplying factor for terrain Classification.
= 0.7 for area in terrain classification 1 (net removal of ground snow depth).
= 1.0 for area in terrain classification 2 (no removal or increase of ground snow depth).
= 1.3 for areas in terrain classification 3 (net increase of ground snow depth).

Determination of Shape Coefficients (μ)

AS 1170.3 includes shape coefficients for a broad range of structural configurations. Only four of these shape coefficients relevant to the Steel Sheds and Garages industry are considered here. For further information refer to AS/NZS 1170.3.

None of the below include obstructed (either from external parapets, saw-tooth roofs etc) roof cases; if these are to be included in the design please refer to AS 1170.3.

TABLE 13 Shape Coefficients (Adapted from AS/NZS 1170.3 Sections 6 & 7)

Symmetric Duo Pitched Roofs	<u>LC1- Balanced Snow Load</u> Load applied equally on both sides of the roof. $\mu_1 = 0.7(60-\alpha)/50$, but in the range $0.7 \geq \mu_1 \geq 0$ <u>LC2- Unbalanced Load on Worst Case Side of Roof</u> Load applied to the worst case side of the roof only. $\mu_2 = 0.56(60-\alpha)/50$, but in the range $0.56 \geq \mu_2 \geq 0, C_e \geq 0.95$
Non Symmetric Duo Pitched Roofs	<u>LC1- Snow Load on both sides</u> Load applied to both sides of the roof in ratio to roof pitch. $\mu_{1a} = 0.7(60-\alpha_1)/50$, but in the range $0.7 \geq \mu_1 \geq 0$ $\mu_{1b} = 0.7(60-\alpha_1)/50$, but in the range $0.7 \geq \mu_1 \geq 0$ <u>LC2- Unbalanced Load on Worst Case Side 1</u> Load applied to side 1 of the roof only. $\mu_2 = 0.56(60-\alpha_1)/50$, but in the range $0.56 \geq \mu_2 \geq 0, C_e \geq 0.95$ <u>LC3- Unbalanced Load on Worst Case Side 2</u> Load applied to side 2 of the roof only. $\mu_2 = 0.56(60-\alpha_2)/50$, but in the range $0.56 \geq \mu_2 \geq 0, C_e \geq 0.95$
Mono Pitched Roofs	<u>LC1- Snow Load over entire roof</u> Load applied equally over entire roof. $\mu_1 = 0.7(60-\alpha)/50$, but in the range $0.7 \geq \mu_1 \geq 0$ <u>LC2- Unbalanced Load on Worst Case half of Roof</u> Load applied to the worst case half of the roof only. $\mu_2 = 0.5\ \mu_1$,
Lean to Arrangements	AS/NZS 1170.3 requires that Alpine and Sub Alpine cases be treated separately for this case.

where

μ_x	= Shape Coefficient for Load Case x.	
α	= Roof Pitch on Symmetric Duo Pitched Roof.	
α_1	= Roof Pitch on side 1 of a Non Symmetric Duo Pitched Roof.	
α_2	= Roof Pitch on side 2 of a Non Symmetric Duo Pitched Roof.	
C_e	= Exposure Reduction Coefficient	
	= 1.0	for sub-alpine regions
	= 1.0	for sheltered roofs
	= 0.75	for semi –sheltered roofs
	= 0.6	for windswept roofs

Design Snow Loads (s)
Design Snow Loads on Steel Sheds and Garages can be broken down into two of the subcategories covered by AS1170.3, roof snow load and snow overhanging the edge of a roof.
AS1170.3 states that the "loads shall be assumed to act vertically and refer to the horizontal projection of the area of the roof".

TABLE 14 Design Snow Loads (Adapted from AS1170.3 Section 4)

Roof Snow Load	Using the sections above for the respective load cases. $s = s_g C_e \mu_x$ Note that for all ground snow load less than 0.75kPa in sub alpine areas, one load case of a balanced distributed load of only 0.4 kPa needs to be applied.
Snow overhanging the edge of a roofs	This subcategory applies only to sections where the roof cantilevers out beyond the wall, such as an eave overhang system. Should be considered in addition to the roof snow load for that section of roof. $s_e = k(s_g C_e \mu_{x)})^2/\gamma$ where, k = 0.5, coefficient to take account of irregular shape of snow at roof edges.

2.3 PERMANENT and IMPOSED ACTIONS

PERMANENT ACTIONS (G)

Permanent actions shall be determined in accordance with AS/NZS 1170.1 including the self-weight of the structure, all attached materials, *permanent* equipment and fittings.

Some permanent types of equipment and fittings incorporated in a building may be removable, but should still be considered as permanent actions. Where such items are not an essential part of the structure, and their removal could create an adverse load case, this should be considered in the ultimate limit state design.

IMPOSED ACTIONS (Q)

Uniformly distributed and concentrated actions shall be considered separately, as required by AS/NZS 1170.1.

Floors

Mezzanine floors should be designed for an imposed uniformly distributed action Q_1 appropriate to the building classification and intended use or a concentrated action Q_2 in accordance with Table 3.1 of AS/NZS 1170.1. A reduction factor as noted in Clause 3.4.2 of AS/NZS 1170.1 may also be applied.

Imposed actions for earthen floors and slabs for farm structures should be determined in accordance with AS/NZS 1170.1 Appendix B.

Roofs

Most roofs of steel sheds will be R2 roofs as defined in AS/NZS 1170.1. R2 roofs are not accessible from adjoining structures, windows, awnings, balconies, etc.

As specified in AS/NZS 1170.1, an imposed uniformly distributed action Q_1 of (1.8/A + 0.12) kPa but not less than 0.25 kPa shall be applied vertically downwards to all R2 roof planes. The area “A” is the plan projection of the surface area supported by the member under analysis, in square metres.

Designers should note particularly the application of Q_1 where a structural element of an R2 roof supports more than 200 square metres of roof area.

As specified in AS/NZS 1170.1, roof cladding should be designed to resist a concentrated action Q_2 of 1.1 kN. A concentrated action shall also be applied to structural elements, including roof purlins or battens, of R2 roofs. The value of Q_2 selected by the designer for structural elements may vary between 0.5 kN and 1.4 kN depending on the accessibility of the particular member and on the building classification.

2.4 LIQUID PRESSURE ACTIONS

The BCA requires that resistance to liquid pressure actions be considered in the structural design of buildings. Such actions may arise from:

- Static or moving floodwater, or from water-borne debris or contaminants;
- Contained liquids, eg storage vessels or silos, or
- Hydraulic ground water pressure.

This guide does not include specific recommendations for the design of buildings that may be subject to liquid pressure actions. AS/NZS 1170.1 contains limited information on determining actions for static liquids and ground water.

It is recommended that generic designs should contain an appropriate general exclusion for the effects of these actions. Generic buildings should not be recommended or sold for construction on sites where liquid pressure may act on the building.

Where resistance to liquid pressure actions is a specific design requirement, or the Building Authority rejects the exclusion, designers should exercise judgement and seek appropriate specialist advice.

Some guidance on flood resistant design and construction can be found in ASCE 24-05 (2006).

2.5 ACTION COMBINATIONS

TABLE 15 BASIC COMBINATIONS FOR STABILITY

STRUCTURAL SITUATION	ACTION COMBINATIONS
Net stabilising effects	0.9G
Net destabilising effects	1.35G 1.2G + 1.5Q_1 1.2G + 1.5Q_2 1.2G + W_u 1.2 G + F_{sn}

TABLE 16 BASIC COMBINATIONS FOR STRENGTH

STRUCTURAL COMPONENT	ACTION COMBINATIONS
Cladding	1.2G + 1.5Q_1 1.2G + 1.5Q_2 0.9G + W_u 1.2G + W_u
Battens, purlins, girts	1.2G + 1.5Q_1 1.2G + 1.5Q_2 0.9G + W_u 1.2G + W_u
Rafters and columns	1.35G 1.2G + 1.5Q_1 1.2G + 1.5Q_2 0.9G + W_u 1.2G + W_u
Foundation anchors/footings	0.9G + W_u 1.2G + W reversal

Notes

G = permanent (gravity) actions

Q_1 = imposed actions, distributed

Q_2 = imposed actions, concentrated

W = wind actions

F_{sn} = snow actions

CHAPTER 3: ANALYSIS

3.1 3D ANALYSIS (OPTIONAL)

The analysis technique should be capable of performing 3D analysis of a whole building:

- Internal frames - Portal columns, rafters, knee and apex braces
- End frames – Columns, rafters, mullion columns, door framing
- Purlins and girts
- Cross bracing including compression struts
- Roof and wall panels (diaphragm action) converted to equivalent cross bracing in addition to true cross bracing
- Other major structural elements, when applicable – roof beams, mezzanine floors beams, columns and joists etc.

3.2 ANALYSIS WITH TENSION ONLY MEMBERS

This functionality is required when flexible elements such as rods or strap braces are used in cross bracing and when roof and wall cladding is designed as a stressed skin diaphragm. Tension only members, in particular cross bracing equivalents of a stressed skin diaphragm, may be designed as plastic fuse elements with limited maximum tension force. Analysis should perform redistribution of axial forces, shear forces and bending moments in cases where tension forces exceed the maximum tension capacity of plastic fuse elements. Alternatively, plastic fuse elements are not considered in the analysis whenever tension forces exceed the maximum tension capacity of plastic fuse elements.

3.3 PLASTIC ANALYSIS

Plastic analysis can be used with plastic hinges formed at different locations such as column bases and connections.
Plastic analysis can only be used only if research shows sufficient ductility and rotational capacity at locations of potential plastic hinges. In particular, plastic hinges at column bases may be formed due to various reasons:

- Limited capacity of footings
- Limited bolt slip capacity
- Limited base steel connections capacity

3.4 COLUMN BASE FIXITY

Fixed rotational supports at column bases should not be used except where it can be demonstrated that full fixity can be achieved such as in a case of cast in place columns. Partial rotational stiffness (spring supports) should be used instead with corresponding parameters based on research data.

3.5 TYPE OF ANALYSIS

1^{st} order analysis is considered sufficient for almost all buildings within the scope of this manual. 2^{nd} order analysis is recommended for unusually slender structures with significant P-delta effect resulting in more than 10% difference in bending moments as compared to 1^{st} order analysis. The engineer should use good engineering practice to determine if a 2^{nd} order analysis is required, taking into account the member configurations and restraint conditions.

Notes:

1. Secondary framing elements may not be necessary to consider in 3D analysis – bridging, window framing, local framing for roof ventilation units and similar, fly bracing.
2. Some structural elements such as bridging and fly braces may be pre-engineered and should be considered in the design of other elements. However it is not necessary to consider them in analysis.
3. Pre-tensioned elements, in particular cross bracing, should not be used within the scope of this Design Guide.

CHAPTER 4: DESIGN

4.1 DESIGN PRINCIPLES

- Steel sheds and garages should be designed by competent engineering practitioners to current Australian codes and standards using Limit States design principles.
- Actions and action combinations should be in accordance with AS/NZS 1170 series.
- Design of cold formed steel components should be in accordance with AS/NZS 4600.
- Design of other steel components should be in accordance with AS 4100 or AS 1163.
- Sheds and garages should always be fit for the stated purpose(s) for which they are designed or offered for sale and be constructed from materials that are fit for the purpose for which they are intended as required by the BCA.
- Design details should be documented to a level that can reasonably ensure satisfactory construction to meet structural design objectives.
- Design assumptions and limitations such as site conditions, soil types, drainage, flood datum level etc should be clearly explained in documentation.
- Any restrictions on future building use or alteration should be communicated in design documentation and reiterated in sales literature and training.
- The design process assumes the selection, installation and maintenance of appropriately durable materials for all buildings designed using this manual.

4.2 SECTION AND MEMBER DESIGN

STRUCTURAL MEMBER DESIGN

Section Properties

Member design relies heavily on the correct use of section properties and these values must be determined accurately before member design can be completed correctly. Steel building products suppliers can provide a detailed list of full section properties for their available sections. The designer needs to have good reason to vary from using these published section properties. The section properties from one manufacturer to another will vary for seemingly similar sections. These variances can be significant and designers should be careful to ensure equivalence when using section properties from one manufacturer when the end product is being purchased from another – especially when differences in section stiffeners are present.

Section properties should be calculated in accordance with AS/NZS 4600 Section 2 or AS 4100 as appropriate. The conventional method is to split the section into smaller simpler elements and sum the properties of each of these elements together in the appropriate manner. This method is known as the “linear” or “midline” method.

The most important consideration when using section properties in cold formed steel design is the appropriate use of either effective section properties or full section properties (gross section properties). Full section properties are those properties of the full unreduced section with no allowance made for localised buckling once the member is subjected to stresses due to design forces. The full section properties are greater than the effective section properties.

Full section properties are used for the determination of buckling moments or stresses. Effective section properties are used for the determination of section and member capacities, as specified in AS/NZS 4600. When determining the deflections in order to perform serviceability checks, it is recommended that a closer approximation to the actual effective section properties be used. This could be completed through using an iterative process. Alternatively, it is considered acceptable to use the average between the full and effective section properties to derive these deflections. This is recommended as the difference in the deflections when using the full and effective section properties can possibly be significant, whereas the difference in the bending moments and stresses is unlikely to be as this is more related to the relative stiffness of individual members.

Accurate effective section properties are required for a number of checks on member and section capacity. Effective section properties are the properties of a section under certain levels of stress. When a section is stressed certain elements in the section can suffer localized buckling and hence become ineffective in supporting the desired stresses. This does not constitute section failure but simply indicates that this part of the section is no longer effective in resisting loads and needs to be taken out of the full section properties creating the effective section properties. At a level of zero stress the effective section properties are equal to the full section properties.

It is common industry practice to use the effective section properties of the section at yield stress in the outer fibre of the section, but it is allowable to increase the effective section properties used if this is not the actual level of stress. These increased effective section properties would need to be calculated on an individual basis as interpolation between zero stress and yield stress in the outer fibre of the section is a very inaccurate method of determining effective section properties at varying stress levels. Calculation of effective section properties is complex and should be completed in accordance with AS/NZS 4600 Section 2 taking into account the effective widths of each element in the section. A good reference for this design procedure is the Design of Cold Formed Steel Structures by GJ Hancock 4th Edition, Australian Steel Institute, 2007.

There are a number of commercially available software packages that can be used to calculate section properties. However the user should ensure that they have adequate knowledge and understanding of the process used by the software, underlying assumptions and the results produced before using relying on such systems.

It is common industry practice to combine two C sections in a web to web (back to back) configuration in order to produce a single stronger I type section. In this case it is not acceptable to simply multiply the section properties of the single section by 2, but instead the section properties should be recalculated as this is now a doubly symmetric section. The method of fixing the sections together will affect the section properties, and it should also be noted that the l/t ratio of the web does not increase. Hence an increase in the effective section properties cannot be attained through this method because both webs can still independently suffer localised buckling. It is also a requirement of AS/NZS 4600 that such built up structural assemblies comply with Section 4. This includes cover sheets and stiffeners used to increase section capacity.

Effective Lengths of Members

The design of structural members, for various forms of buckling, requires the determination of accurate effective length values for the members in question – the design of cold formed steel sections used in steel shed design is no different.

The determination of effective lengths normally involves the use of an Effective Length Factor (k). This factor takes into account the influence of the end restraints of the member (or sections of the member) against rotation and translation. Information regarding the idealized theoretical k values can be found in section C3.4E (Table C3.4) of the commentary of AS/NZS 4600.

The designer must adequately assess each member to determine the appropriate value of the Effective Length Factor (k) and the resulting effective length of the member, for all three axes x, y and z (or the rotational/twisting axis). The designer should be able to justify the values used for each of these separate axes.

The determination of the effective lengths in a standard shed portal frame incorporating knee and apex braces can be quite complex and onerous. One method is the completion of an elastic buckling analysis.

The forces present within members also need to be considered when calculating the effective lengths of certain sections of the particular members. Hence effective lengths of members can vary depending on the member design check being performed and the load case applicable on the frame. It may not be appropriate to set the effective length values and use them for the entire member design for a range of applicable load cases. An example of this is the compressive load check performed on the column in a standard shed portal frame. Under uplift loading the compressive section of the column is generally the short length from the knee brace connection point to the haunch point, hence the effective length for the compression check is this length multiplied by an appropriate value for k. A different situation applies when the lower section of the column is in compression, when the base fixity is an important consideration in determining the value of k.

As a guide, appropriate values for effective lengths are shown in Table 17.

TABLE 17 - Effective Length Recommendations for Standard Shed Designs

Effective Length Recommendations for Standard Shed Designs			
Member		**Effective Length**	**Adjacent Bracing Points Definitions***
Column	**lex**	As per AS4100 recommendations	NA
	ley	Maximum length between adjacent bracing points.	1. Fly Bracing Point. 2. Girt Connection. 3. Column to Rafter Connection. 4. Column to Base Connection.
	lez	Maximum length between adjacent bracing points.	1. Fly Bracing Point. 2. Girt Connection. (If connected directly to the web of the column.) 3. Column to Rafter Connection. 4. Column to Base Connection.
Rafter	**lex**	As per AS 4100 recommendations	NA

	ley	Maximum length between adjacent bracing points.	1. Fly Bracing Point. 2. Purlin Connection. 3. Column to Rafter Connection. 4. Column to Base Connection.
Effective Length Recommendations for Standard Shed Designs			
Member		**Effective Length**	**Adjacent Bracing Points Definitions***
Rafter	**lez**	Maximum length between adjacent bracing points.	1. Fly Bracing Point. 2. Purlin Connection. (If connected directly to the web of the rafter.) 3. Column to Rafter Connection. 4. Rafter to Rafter Connection.
Apex Brace	**lex**	Length of Apex Brace.	NA
	ley	0.8 x Length of Apex Brace.	NA
	lez	0.8 x Length of Apex Brace.	NA
Knee Brace	**lex**	Length of Knee Brace.	NA
	ley	0.8 x Length of Knee Brace.	NA
	lez	0.8 x Length of Knee Brace.	NA
Gable End Wall Mullion	**lex**	Length of Gable End Wall Mullion.	NA
	ley	Maximum length between adjacent bracing points.	1. Fly Bracing Point. 2. Girt Connection. 3. Column to Rafter Connection. 4. Column to Base Connection.
	lez	Maximum length between adjacent bracing points.	1. Fly Bracing Point. 2. Girt Connection. (If connected directly to the web of the column.) 3. Column to Rafter Connection. 4. Column to Base Connection.
Purlins and Girts	**lex**	The length of purlin or girt between supports.	NA
	ley	2 x Distance Between adjacent Screw Fixings (Applies only to Piercing Type Cladding Fixings).	NA
	lez	Bridging Spacing.	NA

* NOTE: It must be shown that the bracing point is capable of acting as the restraint necessary to restrict the effective length. An example of this would be the necessity to ensure that fly brace is capable of resisting rotation of the section around the z axis. The

same applies to the purlin or girt connection as a translational support for buckling in the y axis.

It should also be noted that the knee and apex brace connections to the main frame are not considered to provide translational support in the y axis or rotational support in the z axis to the column or rafter, unless it can be proven by calculation or test that this resistance is being provided.

Member Capacity Checks

Member capacity checks cannot be completed without accurate effective and full section properties. The designer should ensure that bolt holes are taken into consideration when determining both full and effective section properties where this is appropriate.

The following member capacity checks must be made in accordance with AS/NZS 4600 Section 3. It should be noted that the use of AS/NZS 4600 Section 3 to determine member capacities is only acceptable for typical sections such as C & Z sections, singly-, doubly- and point-symmetric sections, closed box sections etc. If it is found that the code does not adequately cover the specific member design then another method will be required to determine the capacity – one option would be testing or use of manufacturers published data. If using manufacturers published data, care should be taken to ensure that the same assumptions and limitations are applicable. This is particularly relevant to sheeting and purlin and girt checks.

AS/NZS 4600 Section 3 requires the designer to perform the following limit state checks where appropriate to the member design. Each of these checks includes a capacity reduction factor available from AS/NZS 4600 Table 1.6.

List of required member checks:

a) **Axial Tension.** Section 3.2

   With this check care should be taken to apply the appropriate value for k_t (the correction factor for distribution of forces).

b) **Bending Moment.** Section 3.3

   Each of the following two design checks must be performed in their entirety.

   1. *Nominal Section Moment Capacity Section 3.3.2*

      This check deals with ensuring that the section of which the member constructed is capable of resisting the stresses induced in the section by the bending moment applied.

   2. *Nominal Member Moment Capacity Section 3.3.3*

      This involves checking that the member as a whole is capable of resisting buckling due to induced bending moments. This section requires the designer to check both lateral and distortional buckling, taking into account the correct member effective lengths.

c) **Shear Capacity.** Section 3.3.4

   The shear force at any cross section of the member should not exceed the member shear capacity.

d) **Bearing Capacity.** Section 3.3.6

   Attention to bearing capacity should be considered at the connection of main frame elements; particularly knee and apex brace connections.

e) **Stiffener Capacity.** Section 3.3.8

Checks are required for increasing the bearing capacity of members through the use of stiffeners to transfer loads more effectively to the web of the section.

f) **Compressive Capacity.** Section 3.4

This section outlines a number of checks for the section under compressive stress.

1. *Nominal Section Compressive Capacity Section 3.4.1*

   This check deals with ensuring that the section of which the member constructed is capable of resisting the stresses induced in the section by the compressive force applied.

2. *Nominal Member Compressive Capacity Section 3.4.1*

   This involves checking that the member as a whole is capable of resisting buckling due to induced compressive forces. This section requires the designer to check torsional, flexural torsional and distortional buckling, taking into account the correct member effective lengths.

   Important Note: The above compressive checks as set out in AS/NZS 4600 apply when the resultant compressive load is applied directly to the centroid of the effective section at the calculated critical stress. If this condition does not apply (as is generally the case) then the designer must ensure that any eccentricity in the loading condition is considered by allowing for secondary bending moments.

The above member checks apply for the situation where the member is only subject to a single resultant action at any one time. For example, the member is subject to only bending with the exclusion of shear, axial or torsional forces. However, this is very rarely the case in the design of Portal Framed buildings, and it is likely that the limiting design check will actually be one of the following combined action checks. Therefore, performing the following checks accurately is extremely important. Generally the member checks must be performed prior to completing the combined checks as certain inputs into the combined checks are created during the member check process.

List of required Combined Action Member Checks:

a) **Combined Bending and Shear.** Section 3.3.5

This is an important check on members at the haunch connection of buildings where knee braces are not provided as this can be a location of high combined bending and shear actions. This check can also be significant for the main frame at the connection of the knee braces and apex braces.

b) **Combined Bending and Bearing.** Section 3.3.7

This is an important check on the column or rafter at the knee brace and main column and rafter connection under different load combinations, as a high bearing load and bending moment reaction can occur at this point (dependent upon the connection type). If attaching the knee braces to flanges of the column and rafter this check may be critical, whereas if connecting the knee brace to the centroid of the column and rafter this check is not required. The axial force in the knee brace (when adjusted for the appropriate knee brace angle) is the bearing force that needs to be considered at this point.

c) **Combined Bending and Axial Compression or Tension.** Section 3.5

This check is most likely to be the limiting check on the structure's main frame. That said it should not be considered to the exclusion of the other checks.

Once again this is an important check on the column or rafter at the knee brace and main column and rafter connection under different load combinations, as a high axial load and bending moment reaction can occur at this point.

Also this check is extremely important at the base of building utilizing partially or fully fixed base connections as a high combination of bending and axial forces under different loadings can occur at this point.

NOTE: As per AS/NZS 4600 Section 7 it is also allowable to determine member capacities in accordance with the Direct Strength Method.

Steel shed design also quite often utilises SHS and CHS members that will need to be checked in accordance with AS 4100. Designers should be careful to ensure that they are applying the correct standard for the relevant member design.

It is common practice for the webs of door jambs and end columns adjacent to door openings to be oriented in the same direction as girts. Girts and roller doors support reactions due to wind pressure acting normal to walls will be resisted then by jams and columns bending in the weak axis. In these instance doors jams and end columns shall be proportioned to resist applied wind loads. Design of additional members to resist wind forces may be necessary.

4.3 DESIGN OF PURLIN AND GIRT SYSTEMS

As structural members within the building envelope, purlins and girts must be designed, as any other component in the building is required to be designed, in accordance with the appropriate member capacity checks. The designer should take the following points into account:

1. Design must incorporate local pressure factors.
2. Design must incorporate internal pressures.
3. Design must incorporate the connections at the purlin and girt support.
4. Design may be based on manufacturers' literature if the following points are complied with:
    a. All design assumptions in the manufacturers' literature are adhered to in the building design, for example the end and lap fixing assumptions, along with bridging.
    b. If any deviations from the design assumptions in the manufacturers' literature are made, the designer must adequately justify why the manufacturers' literature still applies to the design. This could be provided in the form of a letter from the manufacturer, testing or if possible through calculations.
5. Design can be carried out from first principles without reference to manufacturers' literature. Such designs must include:
    a. Calculation of effective lengths.
    b. Inclusion of buckling checks in accordance with AS/NZS 4600 Section 3.

   c. Continuous design of purlins and girts will need various segments of individual purlins and girts to be considered.
   d. Calculation of effective section properties.

6. If continuous purlin and/or girt design are being used then adequate moment transfer must be shown to be occurring over supports.
7. If continuous purlin and/or girt design are being used, the design documentation should warn that the addition of any opening that removes the continuity of the purlin and / or girt is not allowable.
8. The designer must ensure adequate care is taken if using manufacturer's data in the design process but the purlins and girts are being supplied by a different manufacturer. This is due to subtle differences in section geometry, material selection, tolerances and testing regimes that may adversely affect the performance of the system.
9. Bridging must be shown to adequately resist the translation and rotation of the compression flange of the purlin or girt in order to be used to reduce the ley and lez effective lengths.
10. Any additional loads added to the purlins or girt (i.e. door jamb or window connections) need to be considered and allowed for.
11. Proper detailing of all connections, laps and spacings must be shown on documentation (see Good Detailing Practice section).
12. It is common practice for purlins and girts to be used as compression struts as part of the longitudinal bracing system. They should be designed as compression struts if being used in this capacity.
13. Loads should not be applied to purlin lips unless the section has been designed to resist these loads. This includes attaching services to the purlin lips – including but not limited sprinkler systems and heating / cooling ducts.
14. It is not appropriate to rely on catenary action of purlins and girts to resist applied loading.

The design of purlins and girts can have an effect on other structural elements throughout the frame of the structure. For example, if a continuous purlin and girt system is being used then the effect on the first intermediate portal from the gable end wall needs to be considered. Generally the loads on this frame (and reaction loads) will be increased under certain load cases if the portal frame design is being completed as a 2D design, due to the increased first purlin support reaction in a continuous system. If an overall 3D design is completed incorporating purlins and girts then this effect will need to be included in the modelling.

4.4 BRACING SYSTEMS

BRACING PRINCIPLES

Typically steel sheds and garages have a rectangular floor plan and wind loads are effectively resisted in two directions: perpendicular to the ridge line of the building and parallel to it. A portal frame is primarily designed to resist the wall wind loads that are perpendicular to the ridge line along with the majority of roof loads. A bracing system is employed to resist the wall wind loads parallel to the ridge line. This bracing system is just as important as the main portal frame of the structure and must be given appropriate attention by the designer.

Several options are available to the designer when developing an appropriate bracing system, including but not limited to:

- Conventional tension only cross bracing;
- Moment frames;
- Compression elements;
- Combined compression and tension systems and
- Diaphragm action.

Due to this array of options, a number of general comments regarding bracing system design are outlined as follows. These comments should be considered by the designer during the development of the bracing system employed.

- Justification through calculations (if possible) or testing of the bracing system is required.
- The bracing elements are structural members and need to be checked in accordance with the relevant member checks previously outlined.
- During 3D modelling the designer should be cautious that bracing members do not become overstressed and thus provide greater stiffness to the model than can be justified by the preliminary design.
- If a bracing member is only capable of carrying tensile or compressive loads then it is critical that the modelling of the structure take this restriction on the bracing system's capabilities into account.
- If tensile-only bracing is employed then it is likely that compression struts are being used to carry the force to a connection where this load can then be resisted by tensile members. These struts must be designed to carry these compression loads in combination with any other loads that they are carrying. This is especially important for purlins and girts used in this way.
- If the main portal frame used to resist wall loads perpendicular to the ridge line is incorporated into the bracing system design, it is important that all combined checks be appropriately considered for the main portal frame.
- Adequate connections must be designed for all bracing systems – either through calculations (if possible) or through testing.
- Gable end wall frames are often not designed to resist in-plane wind forces through portal moment action and as such may require a bracing system to be employed. The basis of end wall structural design should be clearly indicated in design documentation.
- Standard additions such as mezzanine floors, lean-tos, open sided walls etc can all have an impact on the design of an appropriate bracing system. These building features need to be considered where appropriate.
- Similarly to main frame base reactions, bracing base reactions must be transferred appropriately to the foundations which must be designed to resist these loads.

CONVENTIONAL BRACING

Conventional tension-only cross bracing

This system refers specifically to bracing members that cannot support any compression. Examples include Strap Bracing, Cable and Rod bracing, as these types of members distort and buckle under very small compression loads. These members are installed in a "crossed" configuration where dependent upon the load case one member will become redundant and the other will provide load support in tension. Given these members tension only capacity, a compression strut is generally required in the design in order to provide a stable load path. These struts then become an integral part of the bracing system and as such need to be designed for the applied loads as any other member is designed.

Moment frames

Provide a bracing system that works in a similar way to how the main frame of columns and rafters supports loads acting perpendicular to the ridge line of the building - through utilising member moment capacity to provide load support. Moment frames are generally utilised to provide bracing support around an opening of some type. When utilising this form of bracing designers should be cautious if utilising the weak axis bending capacity of the columns in the main frame, and ensure that this extra load is taken into account when completing the appropriate combined action checks where required.

Combined compression and tension systems

This system is similar to the "conventional tension only cross bracing system". However the members are capable of providing resistance through compression and generally also through tension under different loading arrangements. These elements can be utilised by the designer in a "crossed" configuration similar to the *Conventional tension-only cross bracing* system or as single elements. Examples include angles and round hollow sections capable of resisting compression loads. As with any compression member, it is important that the designer considers buckling capacity and assumes the appropriate effective length of the member as these are likely to be the limiting factors.

DIAPHRAGM BRACING

General

Stressed skin roof and wall diaphragms may significantly stiffen steel sheds, reducing deflections and redistributing forces from internal frames to end frames. There are no particular limits to building geometry where stressed skin diaphragms can be utilized. However, larger buildings are affected less by stressed skin action and its influence can be negligible. Stressed skin diaphragms could be the only bracing system for relatively small sheds and could be used in combination with cross bracing such as rods or strap bracing for larger sheds. Where stressed skin diaphragms are used, they must be designed as any other structural member. An assumption that diaphragm action exists is not acceptable.

Necessary conditions

- Only pierced fixed cladding can form stressed skin diaphragms.

- The diaphragms have longitudinal edge members to carry flange forces arising from diaphragm action. In cladding with the corrugations oriented in the longitudinal direction of the roof the flange forces due to diaphragm action may be taken up by the cladding.
- The cladding is treated as a structural component that cannot be removed without proper consideration. The project specification, including the calculations and drawings, should draw attention to the fact that the building is designed to utilize stressed skin action.
- Suitable structural connections are used to transmit diaphragm forces to the main steel framework and to join the edge members acting as flanges.
- The diaphragm forces in the plane of a roof or floor are transmitted to the foundations by means of braced frames, further stressed-skin diaphragms or other methods of sway resistance.
- Stressed skin diaphragms may be used predominantly to resist wind loads, snow loads and other loads that are applied through the sheeting itself but may not be used to resist permanent external loads, such as those from equipment. (Refer to BS 5950 Part 9:1994).
- The design capacity cannot be based on assumptions about the diaphragm and its components' stiffness and capacity. All design parameters should be based on tests data or published literature. Data in published literature should not be used unless its components and configuration are directly equivalent to those used in the design, eg data for valley fixed fasteners should not be used for crest fixed fasteners.
- Small randomly arranged openings, up to 3% of the relevant area, may be present without special calculation, provided that the total number of fasteners is not reduced. Openings up to 15% of the relevant area (the area of the surface of the diaphragm taken into account for the calculations) may be introduced if justified by detailed calculations. Areas that contain larger openings should be split into smaller areas, each with full diaphragm action.
- All cladding that also forms part of a stressed-skin diaphragm should first be designed for its primary purpose in bending. To ensure that any deterioration of the cladding would be apparent in bending before the resistance to stressed skin is affected, it should then be verified that the shear stress due to diaphragm action does not exceed 25% of the maximum bending stress. Refer BS 5950: Part 9:1994 Section 4.2.1 b for further explanation.

Design approach

There are 2 different approaches to the design of diaphragms:

Approach A. The design is entirely based on parametric tests on full scale stressed skin diaphragms, applying the relevant test principles and procedures of AS/NZS 4600. The design parameters (strength and stiffness) are directly derived from test data as given in Section… of this Design Manual. These parameters would normally be converted to equivalent cross bracing with tension only members to be used in analysis. The advantage of this approach is that design calculations are very simple and parametric tests of diaphragm components are not necessary. The disadvantage is that test results should not be used for dissimilar diaphragms (different screws and their locations, cladding, geometry, supporting structure, other connectors), and no extrapolation is permitted. Where test-based diaphragm performance is relied on, the relevant supporting test data should be made available for compliance verification, on a confidential basis if required. (TG: The importance of geometry of the shear panel needs to be described further I believe.)

Approach B. The design is based on calculations, similar to examples as given in BS 5990: Part 9:1994 Annexes. The following steps would normally be necessary:

- Developing a design model (spreadsheet) with parameters of a diaphragm and its components: geometry, capacity, stiffness.
- Finding data for certain parameters available in published literature. Note that many parameters available in BS 5990: Part 9:1994 are not applicable to Australian design practice: valley fixed fasteners, mild steel, stiff frame supports and connections, different screws and washers.
- Parametric tests to obtain data for diaphragm components.
- Proof tests on full scale diaphragms to confirm design model.
- Revision of the design model if necessary.

This approach requires extensive parametric testing of components. However when testing is completed, accommodation of changes such as changing screw spacing could be easily done using the design model or require minimum of parametric testing of components without going to full scale test as would be necessary for Approach A. This approach is also more suitable for software applications.

4.5 SLABS AND FOOTINGS

GENERAL DESIGN PRINCIPLES

Footing design is a science that warrants an entire spectrum of publications. Any attempt to cover all of this information in this design manual would be futile. As such this design manual focuses only on the core requirements of a steel shed slab and footing system. These are outlined in the following points:

- All steel shed designs shall include or specifically exclude a slab and footing system design. If the design excludes a slab and footing design, this should be noted prominently on the design documentation.
- Reaction forces generated from the modelling and analysis of the structure shall be adequately resisted including an appropriate safety factor. This not only includes bearing forces, but also uplift and inward and outwards thrust forces from different loading combinations.
- Designs should include allowances for differing soil conditions and changes in soil conditions over time. This includes making allowances for differential settlement and the moisture cycle in reactive clay environments.
- Slab and footing designs stated to comply with AS 2870 on reactive clay material (‘H’ or ‘E’) sites should give special consideration to clause 3.1.5. This clause for a clad building such as a steel shed structure allows reducing the site classification by one class of reactivity and selecting the appropriately sized footing system for the structure from the deemed to comply design provided in AS 2870 for a clad frame type of construction. Slab stiffening beams are required for Class H and E.

- Designers should ensure good detailing practice when considering the following regarding slab and footing design:
  - Eccentricities
  - Crack mitigation
  - Moisture and vapour barriers
  - Reinforcing layout
- All slab and footing designs need to be adequately justified through the use of good engineering practice and documentation.

BASE FIXITY

Column base fixity is normally assumed to be fixed or pinned in the design practice. It is acceptable to assume pinned connection, but fixed connection should not be used in the analysis unless column bases are properly designed, detailed and tested. It is recommended to model partial fixity at column bases.

Partial fixity at column bases could be due to 2 different sets of reasons:

A. Formation of plastic hinges. Maximum bending moment capacity is limited by:
   - Bolt slip capacity (bolts connecting columns to base brackets through slotted holes)
   - Capacity of steel brackets in bending or tension
   - Anchors capacity in tension
   - Capacity of concrete footings

B. Reduced rotational stiffness (rotational spring supports). Reduced stiffness may be due to:
   - Base brackets working in bending
   - Concrete footing/soil interaction (rotation of foundations in soil).

All necessary parameters could be found by calculations or from test data.

Contraction and expansion joints may be necessary for slabs and footings in sheds used for non-residential purposes.

4.6 CLADDING

It is anticipated that sheds and garages designed in accordance with this guide will be clad with steel sheeting designed and installed in accordance with AS 1562.1. Steel sheeting may be of any thickness provided it meets all the requirements of AS 1562.1.

The performance of steel sheeting shall be supported by manufacturer's literature and test data. Design capacities of cladding based on tests results should not be applied to cladding made with steel from other suppliers, in particular imported steel, unless it is demonstrated such steel has higher yield stress and better ductility (as tested).

Roofs (including the cladding) required for floor type activities must be designed as floors using the relevant actions specified in AS/NZS 1170.1 and are not covered by this Manual.

Roofs may be designed as either type R1 or R2 as defined in AS/NZS 1170.1. Steel roof cladding must be capable of resisting the uniformly distributed and concentrated actions specified for each type of roof.

Where type R2 roofs are designed for access using ladders or boards, conspicuous notices should be installed at access points to the roof warning against walking directly on the roof sheeting.

Roof and wall cladding should be designed to resist wind actions calculated using the guidelines in this Manual and AS/NZS 1170.2. Due consideration should be given to local pressure areas near edges, corners and apexes.

In relevant localities, roof cladding should be designed to resist:

- Cyclic loading as required by Part B1.2 of the BCA, and/or
- Snow actions calculated in accordance with AS/NZS 1170.3.

Fixing may be crest, valley or concealed fixed as recommended by the sheeting manufacturer for the relevant design actions and performance requirements of the cladding. *In cyclonic regions, particular attention must be given by all levels in the supply channel to correct cladding fixing specifications.*

In cyclonic regions, the designer may need to consider the resistance of the roof and wall cladding, as part of the building envelope, to impact loading as described in AS/NZS 1170.2 Clause 5.3.2. Evidence of successful impact testing, including any conditions or restrictions on product suitability, should be obtained from the manufacturer of the cladding. If satisfactory evidence of impact resistance cannot be obtained, appropriate assumptions will need to be made regarding dominant openings and permeability in establishing the critical design cases.

All fasteners used in fixing should be physically, chemically and galvanically compatible with the sheeting and its supporting members. All trimming, flashing and other installation details should be carried out to minimise water entry to the building and the possibility of debris shedding in severe weather events.

Where the design uses diaphragm action as part of the bracing system, note the requirement to check bending and shear stress in the cladding. Refer to Diaphragm Bracing section above.

4.7 DOORS, WINDOWS & OPENINGS

The presence of doors, windows, skylights and other features in the building envelope must be carefully considered in the design. Openings in the structure alter the distribution of forces from actions applied to the building. Openings may interrupt the continuity of purlins and girts requiring their design as single spans. Additional loads may be transferred to purlins and girts via heads, sills and trimmers. Openings may also reduce the capacity of bracing systems or cause them to be installed in less effective configurations. All of these effects must be considered in the structural design.

Commercially available doors and windows incorporated in the design must have adequate capacity to resist the design actions (especially wind) to which they will be subjected. This capacity should be quoted as an ultimate limit state design capacity, with an appropriate capacity reduction factor already applied.

AS 2047 *Windows in buildings*, referenced in the BCA, allows a concession for window performance in Class 10 buildings. These windows do not need to pass the air infiltration and water penetration requirements of the standard. This concession applies only to Class 10 buildings.

It is common practice for roller type doors to be fitted with “wind locks” to prevent the withdrawal of the curtain from the guide track at high wind pressures. In this situation, the catenary action of the door curtain applies substantial lateral and torsional loads to the door jamb sections which should be taken into account in the design.

Doors and windows provide essential access, light and ventilation to buildings but, when closed, they form part of the building envelope to resist wind action. Some commercially available doors and windows may not have sufficient strength or stiffness to resist ultimate limit state design wind actions. Unless the designer is satisfied that the doors and windows to be fitted to the building have adequate capacity to resist design wind actions, they should be assumed to be openings and the building structure designed accordingly.

Note also that in cyclonic areas, the designer may need to consider the resistance of doors and windows, as part of the building envelope, to impact loading as described in AS/NZS 1170.2 Clause 5.3.2. Permanent screens or grilles may be designed to provide the required resistance for windows. Evidence of successful impact testing, including any conditions or restrictions on product suitability, should be obtained from the manufacturer of the door or window. If satisfactory evidence of impact resistance cannot be obtained, vulnerable features must be assumed to be open during peak wind events and the appropriate dominant opening pressure coefficients selected.

4.8 DESIGN PRINCIPLES FOR SERVICEABILITY

This Section gives guidelines for the serviceability limit states resulting from deformation of sheds and their elements. Table 18 identifies deflection limits related to actions with annual probability of exceedance of 1/20 (0.05) beyond which serviceability problems have been observed. These limits are imprecise and should be treated as guide only. These limits may not be applicable in all situations.

TABLE 18: Suggested Serviceability Limit State criteria

Element	Phenomenon controlled	Serviceability parameter	Applied action	Element response	
				Importance Level 1	Importance Level 2 or higher
Metal roof cladding	Indentation	Residual deformation	Q = 1 kN	N/A	Span/600 but < 0.5 mm
Metal roof cladding	De-coupling	Mid-span deflections	G and □sQ	Span/100	Span/120
Roof rafters	Sag	Mid-span deflection	G	Span/250	Span/300
Columns	Side sway	Deflection at top	Ws	Height/80	Height/100
Portal frames	Roof damage	Incremental deflections at top	Ws	N/A	Spacing (Bay)/200
Lintel beams (vertical sag)		Doors/windows jam	Ws	Span/240 but <12mm	Span/240 but <12 mm
Roof purlins	Sag	Mid-span	G	N/A	Span/300

		deflection			

Notes:
1. Masonry and other brittle materials are outside the scope of this Design Guide
1. Other deflection criteria may be specified in the contract documentation
2. Limited bolt-slip capacity at serviceability loads should be taken into account for deflection calculations
3. Buildings designed to be capable of future conversion to Class 1 (habitable) buildings may require tighter serviceability criteria.

CHAPTER 5: CONNECTIONS

5.1 GENERAL

Connection elements consist of members, connection components (cleats, gusset plates, brackets, connection plates) and connectors (welds, bolts, screws, rivets, clinches, nails, adhesives). The connections in a shed should be proportioned so as to be consistent with the assumptions made in the analysis of the structure and to comply with the recommendations in this Design Guide

It is expected designers will pay particular attention to the design of connections in steel sheds. This requires an understanding of how cold-formed steel members interact with each other and with the brackets and connectors that form the connection. Connection adequacy is vital to the structural integrity of the building. It is the designer's responsibility to prove that the connections used in the design are adequate.

5.2 DESIGN BASIS

The design of connections in cold formed steel sheds should be in accordance with AS/NZS 4600 Section 5. Alternatively the design of connections may be based on test data in accordance with Part F of this Guide. Where components are made from hot rolled steel or are thicker than the scope of AS/NZS 4600, designers should apply the relevant requirements of AS 4100.

It is particularly important to maintain the stability of connections involving asymmetrical cold-formed steel members. In an ideal situation, the components forming the connection should remain geometrically stable throughout the loading cycle. Ensuring that they remain as stable as possible is a design consideration.

Connectors (fasteners) used in connections should have a nominal failure capacity at least 125% of that of the connection (or 2.5 times the design capacity). This ensures that connector failure will not initiate connection failure, thus producing a more ductile connection.

5.3 TYPICAL PRIMARY CONNECTIONS

COLUMN BASE CONNECTION ELEMENTS WITH CAST-IN BRACKETS

These connections are structurally efficient and can develop significant fixity at bases. This is possible due to brackets being cast into concrete and resisting predominantly axial forces. Column connectors, typically tensioned bolts or screws, should be located as low as possible to minimize effects of horizontal support reactions. Brackets should be properly anchored into concrete to avoid pull out failure. U bars may be used as suitable anchorage.

COLUMN BASE CONNECTION ELEMENTS WITH HOLD DOWN BOLTS

These connections are typically used for larger sheds. They can resist large bending moments. However brackets are more flexible than cast-in brackets since they resist predominantly bending moments. As a result bending moments developed would be significantly lower than corresponding to full fixity. Thicker brackets or additional bracket stiffeners would be necessary to increase connection stiffness to the required level. Column connectors would typically be tensioned bolts.

OTHER COLUMN BASE CONNECTION ELEMENTS

Many other options may be used in the design of base connections including the following:

- Chemical anchor bolts and equivalents
- Bearing type bolts (hole sizes closely matching bolt diameter)
- T base brackets
- Simplified pin base connections

OTHER MAIN FRAMES CONNECTION ELEMENTS

Some connections are inherently more difficult to design efficiently, and testing may be the best approach. Connections in this category include knee, apex, knee brace to column and rafter and ridge ties. Testing should be carried out for every size member, as extrapolation of results may be invalid. Testing and rating whole portal frame can check the effect of connection on the portal frame stability.

5.4 BOLTED CONNECTIONS

BOLT SLIP

Bolt-slip capacity of tensioned bolts should be derived from test results in accordance to recommendations given in AS/NZS 4600 Suppl: 1998 Section C5.3 (a). It is known that bolt-slip capacity with zinc coated surfaces is significantly lower than for bare steel. Typical mean bolt-slip capacity of tensioned bolts will not exceed:

- M12 8.8/S bolts with mild washer 7 kN
- M12 8.8/S bolt with integrated washer 15 kN
- M16 8.8/S bolt with mild washer 15 kN
- M16 8.8/TB bolt with structural washer 17 kN
- M16 8.8/TB bolt with integrated washer 23 kN

Bolts with integrated washers are the most efficient connector from a bolt-slip perspective.

Bolt-slip capacity can be affected by:

- Size of holes
- Steel member and washer coating, paint
- Washer size, steel grade and surface quality
- Method of bolt tensioning

SLOTTED HOLES

Slotted holes 18 x 22 mm are suitable for both M12 and M16 bolts, while 22 mm round holes are also used for M16 bolts in main frame members. Whilst slotted holes do not comply with AS/NZS 4600 rules except for purlins, these clearance are generally acceptable provided bolt-slip capacity is derived from test results on specimens with purlin size holes and possible bolt slip is considered in deflection calculations.

PURLIN & GIRT CONNECTIONS

Slotted holes 18 x 22 mm typically punched in steel purlins and girts allow for +/- 3 mm nominal clearance transversely and +/- 5 mm longitudinally. These purlins form part of tested proprietary systems and cannot be designed using AS/NZS 4600 rules. When incorporated in steel shed designs, manufacturers' instructions regarding design and installation should be observed.

5.5 SCREWS

Screw manufacturers' capacities are sometimes based on average capacity. However, AS/NZS 4600 requires capacity to be determined from the minimum value from a sample of tests. The purpose of these tests is to reliably estimate the 5-percentile shear and tensile failure loads, as required by the BCA for structural materials and components. It is recommended that connector nominal capacities (especially screws and rivets) should be determined from a sample of not less than 10 connectors. Designers should ensure that fastener manufacturers provide nominal and design capacities determined in a way that is consistent with AS/NZS 4600 requirements. The NASH Handbook – Residential and Low-rise Steel Framing provides extensive data on screwed and riveted connections in cold-formed steels.

5.6 WELDING

Welding of structural components in steel sheds, whether carried out in a factory or on site, should comply with all relevant Australian Standards. Welded connections involving cold-formed steel must be correctly designed, taking into account a possible reduction in steel strength adjacent to the weld line (annealing).

Where both sections in a welded connection are thicker than 2.5 mm, welding should comply with AS/NZS 1554.1. Unless otherwise specified by the designer, when either section is less than 2.5 mm thick, the weld should comply with AS/NZS 1554.7. This Standard is applicable for the welding of materials up to 4.8 mm thick and therefore may be used for materials between 2.5 mm and 4.8 mm thick.

Any metallic coated areas damaged during welding should be touched up with a zinc rich paint to give similar durability to the original material.

The design of fillet welds when either of the joining components is less than 2.5 mm should be in accordance with AS/NZS 4600. For thin gauge steel the strength is governed by the tensile strength of the connecting plates rather than the strength of the weld. Where both of the joining components are greater then 2.5 mm the design should be in accordance with AS 4100.

The effect of welding on the mechanical properties of the members needs to be taken into account through testing. However, for all grades conforming to AS 1163 and grades G250, G300, G350 and G450 steel conforming to AS 1397, the design expressions given in AS/NZS 4600 allow for any change in properties. The capacity reduction factor has been reduced for longitudinal welds for G450.

5.7 OTHER CONNECTION METHODS

Designers may choose alternative connection methods to those above, applying the appropriate principles and judgement. As previously stated it is up to the designer to prove the adequacy of every connection on which the integrity of the structure is dependent.

CHAPTER 6: TESTING

6.1 GENERAL

Tests and results evaluations should comply with general requirements of AS/NZS 1170.0 Appendix B. Tests should be conducted by NATA registered laboratories. Alternatively tests results should be verified by NATA registered laboratories.

Steel units designed by calculation in accordance with relevant Australian Standards are not required to be tested. Proof and prototype tests may be accepted as an alternative to calculations or may become necessary where:

- More accurate information is required for use in structural design
- Specific design parameters and methods are not included in relevant standards
- The situation is sufficiently unusual to require that limit states be checked by methods other than calculation
- There is a history of structural failures
- Necessary design data is not available from product manufacturers (connectors, cladding, etc)
- Designers are looking to support calculations or to provide more efficient designs.

The unit to be tested may be a structure, substructure, member, connection assembly or connection.

6.2 PROOF TESTING

Proof testing should comply with requirements of AS/NZS 1170.0 Appendix B2. This test method establishes the ability of the particular unit under test to satisfy the limit state that the test is designed to evaluate. Proof tests can be also used to evaluate structural models such as stressed skin diaphragms.

Additional requirements for time-dependent materials do not apply to steel structures used in sheds.

6.3 PROTOTYPE TESTING

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

Prototype testing should comply with requirements of AS/NZS 1170.0 Appendix B2 and AS/NZS 4600 Section 8.2.

Testing of sheet and wall cladding system shall be in accordance with AS/NZS 1562.1.

Stressed skin diaphragms could be tested using the test setup and test procedure as given in the following international Standards:

- BS 5950: Part 9:1994 Section 11
- EN 1993-1-3
- ASTM E455-04

Test results evaluation should be done in accordance with requirements of this Design Guide.

Test specimens should be arranged as equivalent to diaphragms used in design practice and particular attention should be paid to constraints on perimeter and supports. Flexibility of connectors and supporting structures should be correctly modelled.

Prototype testing would normally be required to design components of steel sheds such as:

- Column base connections, strength and stiffness
- Roof and wall diaphragms, strength and stiffness
- Knee and apex connection assemblies, strength and stiffness
- Bolt slip capacity for friction bolts, stiffness
- Non symmetric structural elements, strength
- Built up structural elements, strength and stiffness
- Cladding strength and stiffness if not supported by manufacturers' data.
- Multiple screw groups, to assess combined capacity

Tests should be conducted on every size of member used in the design. Full scale testing of a complete portal frame is useful for determining the effect of the connections on portal frame stability.

6.4 TESTS RESULTS EVALUATION

Test results evaluation should be done in accordance with AS/NZS 4600 Section 8.2 accompanied by test reports compiled in accordance to AS/NZS 1170.0 Appendix B1.5. Values for coefficients of variation of structural characteristics (Vsc) should be used as given in AS/NZS 4600 Supplement Section C6.2.2.3. Design capacities should be determined in accordance with AS/NZS 4600 Section 8.2. In particular minimum or average test capacity shall not be used as design capacity. The appropriate sampling factor must be applied to the minimum capacity from a sample of not less than 3 tests.

6.5 PRODUCT SUBSTITUTION

Design capacities based on tests results are not applicable to structural elements made with the steels or components from other suppliers, in particular imported steel, unless it is demonstrated such steel has higher yield stress and better ductility (as tested).

6.6 CONNECTORS AND CONNECTIONS

Testing of connection elements may have the same logic as testing of stressed skin diaphragms given in Part D of this Design Guide following design approaches A and B. There could be parametric or proof testing of connection elements and parametric testing of connectors.

Testing should comply with requirements of AS/NZS 1170.0 Appendix B2 and AS/NZS 4600 Section 8.2. Design models should be developed by the designer based on first principles and good design practice (refer to ASI Connections Handbook by Tim Hogan and Scott Munter).

For connections, design capacities determined by test may be based on samples of as little as 3 units. The minimum value from tests is divided by the appropriate sampling factor to give the design capacity of the connection. Note that the nominal capacity of the connector (screw, rivet or bolt) in any connection must be at least 25% greater than the maximum design force (in shear or tension) to which it will be subjected in the ultimate limit state.

For screws and rivets, manufacturers' capacities are sometimes based on average capacity. However, AS/NZS 4600 requires capacity to be determined from the minimum value from a sample of tests. The purpose of these tests is to reliably estimate the 5-percentile shear and tensile failure loads, as required by the BCA for structural materials and components. It is recommended that connector nominal capacities (especially screws and rivets) should be determined from a sample of not less than 10 connectors. Designers should ensure that manufacturers provide nominal and design capacities determined in a way that is consistent with AS/NZS 4600 requirements.

CHAPTER 7: OTHER CONSIDERATIONS

7.1 ANALYSIS SOFTWARE AND DESIGN AIDS

Where software is used to produce designs, particularly those that are not "signed off" by a qualified person, it is prudent to check the software for the following features to ensure that the designs will be in accordance with Australian regulatory requirements:

- Scope and limitation of application of software (particularly applications that are NOT appropriate for use);
- User qualifications - degree of training/competence required (if any)
- Name and edition of the BCA and its referenced documents that have been incorporated into the software;
- References for general installation instructions and specific recommended installation/transport procedures which are not part of the "general requirements" or "standard industry practice";
- Availability of a guide or training programme for users, and
- History of revision/upgrading.

7.2 GOOD DETAILING PRACTICE

Good building design includes attention to the details that will produce a structurally sound and serviceable building. Many of these details are in the form of standard drawing notations applied in the design office of the shed manufacturer. The following list is not exhaustive, and not all details will apply to all buildings.

PURLINS AND GIRTS

- Clearly distinguish between single and continuous spans, especially where known openings create single spans.
- Where structural continuity of purlins and girts is assumed in design, detail laps correctly with clear overlap dimensions.
- Include clear notation on drawings and design certifications that wall openings added later will require redesign of girt system.
- Ensure loads are not applied to C or Z purlin lips or top hat flanges, and include warnings to alert service installers.
- Where Cee and Zed purlins and girts are used, they should generally be installed with the top flange directed up the wall or roof slope.
- For Cee purlins, orientate web towards ridge for all pitches 10° and under, otherwise, purlin flanges to orientate to ridge line.

SECTION ORIENTATION

- Unless designed otherwise, end wall members should ideally be loaded in their major axis, ie with the plane of the web normal to the plane of the end wall. This requires careful consideration at corners where the end or side bay is an opening.
- Where end wall mullions connect to the end frame rafter, the connection detail must avoid rotational moment in the rafter due to eccentricity of loading.
- Correct door jamb orientation is important, especially where a door fills an entire bay at a corner.
- Transfer of roller door curtain loads to door jambs requires careful attention to resist rotational forces on the jamb section, especially where wind locks are installed.
- Detailing of door and window openings generally requires care to transfer wind and bracing loads to locations designed to take them. Supplier drawings and specifications should alert customers to precautions required where openings are made after construction.
- It is preferable for window or door openings to be located where wall bracing interference will not occur.
- Whenever possible, roof and side wall bracing to be located within same bays.
- If door mullions are to be orientated about the weaker axis, sufficient restraint must be considered to avoid section failure when under load.

SLAB AND HOLD-DOWN

- Drawings and specifications should clearly identify critical hold-down points where effective hold-down capacity must be provided.
- Where hold-down bolt edge distances are small, screwbolts or chemical anchors are preferable to expanding type anchors.
- Chemical anchors may not be suitable where fire resistance is a consideration.
- Beware of green concrete, and specify minimum curing times for setting and loading of anchors.
- All concrete specifications must include minimum curing period, method of curing, and characteristic strength at 28 days(f'c Mpa)
- Good construction practice dictates that masonry anchors should not be placed in concrete which has not reached 50% of its design strength (typically 7 days).
- When specifying cast-in hold-down bolts, ensure that adequate thread is available for the hold-down bracket and nut(s). For all cast in members, a distance from above concrete surface penetration must be specified to ensure all base plate and fasteners have sufficient anchor to fix to.
- Bituminous paint top 150-200 mm of cast-in HD bolts before casting slab to protect thread.
- Expanding type masonry anchors such as "Dynabolts" are susceptible to becoming loose due to frame movement and vibration. They should not be used on main frames due to limited restraint from wind uplift.

- In all cases, a cast in anchor is preferable to an expanding or chemical anchor.
- If framing members are to be cast into wet concrete, a surface coating, eg bitumen, must be considered to inhibit corrosion from member coating reacting with concrete.

CONNECTIONS & GENERAL

- Tight hole sizes minimise joint slip, while large hole sizes speed assembly. Maximum hole diameter should be specified and observed for each bolt size. Generally, cleat hole diameters should be 2 mm larger than bolt diameter.
- Ensure bolts are detailed to correct length to give adequate but not excessive stick through of threads. Minimum number of three full bolt threads to extruding outer locking nut.
- Hole patterns in brackets and members should ensure minimum edge distances for bolts. Similarly, where screws are used there must be adequate area for correct placement of the required number of screws.
- Minimum thicknesses should be adopted for specific components to minimise handling and construction damage, even if not required for strength or serviceability.
- Brackets should be designed to transfer loads between members with minimum eccentricity. Identify relevant load paths and detail connections to eliminate distortion.
- Sufficient edge bearing must be considered when locating holes in brackets or cleats. General rule is 20-25 mm for M12 bolts and 25-30 mm for M16 bolts.

BRACING

- Attachment of bracing should be detailed to ensure correct load paths into brackets and members. Where the component configuration requires some eccentricity, this should be adequately designed for.
- Connections to single unsymmetrical sections that may become unstable when loaded should be carefully detailed.
- Do not specify strap bracing for open bays (safety issue). All bracing to open bays should be robust and visible and free of hazardous edges.
- Slender bracing elements used horizontally in roofs should be prevented from sagging.
- If using strap bracing, consideration must be given as how to achieve correct tension in setting strap brace during installation.
- Whenever possible, it is desirable to locate side wall brace in same bay as roof bracing for load transfer.
- A roof purlin should be located at the extent of diagonal roof bracing for load transfer between roof framing members to vertical members.

CORROSION

- Detail design at the slab/sheeting junction must drain freely and avoid contact between sheeting and slab. Flashings should be inert, or separated from the slab by a suitable dampcourse material.

- It is particularly important to keep zinc-aluminium alloy coated steel away from concrete and mortar.
- Ensure adequate drainage from flashings, channels and trims.
- Where masonry anchors are specified, they must be galvanized and not zinc plated.
- Correct material grade and type should be specified for all componentry to be installed in highly corrosive environments, eg, within 500 m of ocean.
- Cast in anchors or hold down should be galvanised, stainless steel or bitumen treated

BUILDING ENVELOPE INTEGRITY AND ROBUSTNESS

- All products and components forming part of the structure of the building envelope should be clearly identified by reference to an appropriate Standard or product specification.
- Careful attention is required to connection details for cladding, flashings, trims, doors, windows and accessories to prevent debris shedding in severe wind loading events.
- Refer manufacturer's recommendations for correct installation and maintenance of the products they supply.
- Fixing of cladding in cyclonic areas to have necessary cyclonic cladding fixing screws- generally, roof fasteners have cyclonic washer.
- All windows and doors to be suitably wind rated for building compatibility.

7.3 DURABILITY & CORROSION

GENERAL PRINCIPLES

Durability is defined as the capability of a building or its parts to perform a function over a specified period of time. The BCA does not directly regulate the maintenance or durability of materials of construction. However, it requires that:

- A building or structure must resist reasonable actions including "time dependent effects" [BCA 2008 Clause BP1.1], and
- Every part of a building "must be constructed in an appropriate manner to achieve the requirements of the BCA, using materials that are fit for the purpose for which they are intended" [BCA 2008 Clause A2.1].

It is therefore up to the designer to specify, in consultation with the client, materials and construction details that have reasonable durability, are fit for purpose and meet the client's durability requirements and expectations.

Where the client's specific requirements are not known at the time of design, for example in the case of generic buildings, the BCA provides some guidance through its publication "Guideline on Durability in Buildings" [ABCB 2002]. This guideline encourages designers to classify components within buildings on the basis of the design life of the building and the accessibility and cost of the component for repair or replacement.

TABLE 19 Building and component design life

Design life of building (dl_b) (years)		Design life of components or sub-systems (dl_c) (years)		
		Category		
Category	**No of years**	Readily accessible and economical to replace or repair	Moderate ease of access but difficult or costly to replace or repair	Not accessible or not economical to replace or repair
Short	$1 < dl_b < 15$	5 or dl_b (if dl_b<5)	dl_b	dl_b
Normal	50	5	15	50
Long	100 or more	10	25	100

Source: ABCB Guideline on Durability in Buildings 2002

All building practitioners, regardless of their role in the process, should consider the impact of their work on durability. The ABCB Guideline on Durability in Buildings notes that:

> *"Durability is not an inherent property of a material or component. It is the outcome of complex interactions among service conditions, material characteristics, design and detailing, workmanship and maintenance. Consideration of all of these should be part of the design process."*

The design life or "life expectancy" of a building or any part of it may be defined by state or national regulation. It may also be implied by relevant standards, or stipulated by the owner. Once the design life is established, the designer should devise a design specification and associated procedures through which the design life may be met or exceeded. The selected materials, their stated method of assembly and construction and their nominated maintenance requirements all form part of the design.

Metallic coated steels and appropriately protected hot- and cold-rolled steels are highly durable, long life materials. Using commonsense and familiar tools, materials and methods, steel framing made from these steels can be adapted to meet to a wide variety of design challenges. In the majority of construction environments, steel can be expected to perform its structural function almost indefinitely provided it is protected from specific and well-understood hazards. Sources of potential damage are readily identifiable, and guidance is readily available on how to protect the integrity of the steel frame before, during and after construction.

DISSIMILAR METALS

Designers should identify potential locations in the overall building design where components made from incompatible materials may be fixed to steel framing members. Guidance on corrosion due to material incompatibility can be found in [REF].

The potential for dissimilar metals at fasteners and connections is high. Designers should specify fastening and connection systems with known durability and warn against any substitutions. The following table will assist in identifying high-risk interfaces in design and construction.

TABLE 20 Material compatibility guide

Material of Frame Member	Material of Component, Accessory or Fastener			
	Zinc or aluminium/zinc coated steel	Aluminium	Copper	Stainless steel
Zinc coated steel	Suitable	Suitable	Not suitable	Not suitable
Aluminium/zinc coated steel	Suitable	Suitable	Not suitable	Not suitable
Aluminium	Suitable	Suitable	Not suitable	Not suitable

Notes:

1. The above guidelines apply to direct contact between components.
2. Providing physical and electrical isolation between materials will extend the range of suitable applications.
3. The table assumes the individual components are suitable for the service environment.
4. Where possible, flashings should be made from the same material as the surface(s) being flashed.

PROXIMITY TO MARINE ENVIRONMENTS

Metallic coated steel

BCA Volume 2 (Housing Provisions) contains requirements and restrictions on the use of metallic coated steel products in marine environments. Whilst these requirements do not necessarily apply to sheds and garages, they may be useful for determining the suitability of metallic coated steels in these buildings.

Steel components with a Z275 or AZ150 coating designation may be used for steel framing *outside* the building envelope (the situation with most shed and garage framing):

- Beyond 1 km from calm salt water, such as a lake or estuary, or
- Beyond 10 km from a coastal area with breaking surf.

Where the framing material is *fully enclosed* within the building envelope, it may be used as close as 300 metres from breaking surf conditions.

Breaking surf normally occurs in areas exposed to the open sea, with regular breaking of waves about 4 days per week, but it does not include choppy, white-capped water.

Unwashed areas subject to condensation may be particularly vulnerable. The terms of any manufacturers' durability warranties should be considered when building in or near marine environments, and advice obtained from manufacturers in marginal situations. For further information, BlueScope Steel TB 35 bulletin on Salt Marine Environments provides useful information.

Uncoated steel

There are no explicit requirements for the protection of structural steelwork in BCA Volume 1. Volume 1 calls up AS 4100 of which Appendix C (informative) covers Corrosion Protection. This references AS 2312, which includes a classification system for atmospheric exposure of steelwork.

BCA Volume 2 contains a requirement for “protective coatings for steelwork” via multiple alternative solutions. Currently only 2 exposure classifications – moderate and severe - are listed. Moderate environments are more than 1 km from breaking surf or more than 100 m from calm salt water, while sever environments are within these distances.

Generally, internal structural steel in permanently dry locations requires little or no protection. However, typical sheds and garages that are neither well sealed against contaminants nor ventilated to reduce damp and condensation should be regarded as “moderate” environments.

OTHER CONSIDERATIONS AND PRECAUTIONS

Poor detailing and/or construction may reduce the durability of some building components even if they have been correctly designed for their service environment. In particular, the following items should be considered regardless of the environment in which the building is to be constructed:

- Cast–in bolts and brackets should have the top 200 mm coated in bituminous paint before casting the slab or footing.
- Keep Zincalume steel away from concrete – damp concrete is alkaline and a reaction will commence even with condensation. Where incidental contact is possible, provide an isolation membrane such as polythene or bituminous aluminium dampcourse material.
- Keep corrosive contents such as fertilizers away from steel products, and consider warning signs if such contact is possible.
- Use hot dipped galvanized bolts and brackets and Class 3 screws wherever possible.
- Where galvanized or paint-protected components are cut or welded on site, ensure the protective system is restored in the area of damage.

7.4 FIRE DESIGN

Fire safety requirements for all building classes are stipulated in the BCA. In specific situations, designers will need to consider the effects of fire engineering in the structural design. This could involve the effects of an internal fire, including relevant fire services, or the effects of an external building fire or bushfire.

Fire design involves complex and sometimes conflicting requirements. Small residential class 10a buildings should be straightforward, but appropriate expert advice should be obtained for other buildings. A predevelopment meeting with the local Council may clarify local requirements, especially in relation to bushfire.

APPENDICES

1. BUILDING CLASSIFICATIONS
2. IMPORTANCE LEVEL AND PRESSURE COEFFICIENT EXAMPLES
3. DESIGN PROCEDURE CHECKLIST
4. PRO FORMA CERTIFICATE
5. SHED SELECTOR POSTER
6. WORKED EXAMPLES – DESIGN WIND SPEED
7. WIND PRESSURE COMPARIONS

APPENDIX 1

BUILDING CLASSIFICATIONS

The actual use of a building – not its physical appearance or commercial description - determines its classification. The following information is based on the Guide to the BCA 2008 Part A3.

BCA Principle

The BCA states that "the classification of a building or part of a building is determined by the purpose for which it is designed, constructed or adapted to be used" [Ref BCA Clause A3.1].

TABLE 21 BCA BUILDING CLASSIFICATIONS

CLASS	TYPICAL BUILDINGS	DESIGN GUIDANCE
1a	Single dwelling-house	NASH Standard & Handbook
1b	Small guesthouse or boarding house	NASH Standard& Handbook
2	Residential flat and apartment	NASH Standard & Handbook (up to 2 storeys)
3	Motels & hostels (low care) (see also 9c)	NASH Standard & Handbook
4	Caretaker's residences	NASH Standard & Handbook
5	Offices	
6	Shops	
7a	Carparks	
7b	Warehouses or wholesalers	SSG Design Guide
8	Factories	
9a	Health-care buildings (including nursing homes)	
9b	Assembly buildings	SSG Design Guide
9c	Aged-care buildings (hostels, low care) (see also 3)	
10a	Non-habitable *buildings*: sheds & carports	SSG Design Guide
10b	Non-habitable *structures*: pools, fences, signs, towers, etc	

Difficult Classifications

Buildings from which goods are sold to the public are Class 6, while wholesale buildings are Class 7. If the general public has access to the building, it is Class 6.

Farm buildings may be Class 6, 7, 8 or 10a. They would only be classed as 10a if Class 6, 7 or 8 would be inappropriate.

Classifications are important for the orderly regulation of social and economic activities and for public fire safety, but do not have as great an effect on structural design as Importance Levels.

APPENDIX 2

IMPORTANCE LEVELS - Examples

Domestic Garage

Importance Level 2

Exceedance risk 1:500 in all wind regions

Farm Shed

Importance Level 1

Exceedance risk 1:100 in regions A & B

Exceedance risk 1:200 in regions C & D

PRESSURE COEFFICIENTS - Examples

Enclosed Shed or Garage

Fully enclosed with roller and access doors

Full internal pressure (dominant opening) in Regions C & D unless doors and cladding impact resistant

Open Rural Machinery Shed

Clad 3 sides

Full internal pressure (dominant opening) in all regions (typically +0.5 to 0.7 depending on opening dimensions and permeability)

APPENDIX 3

DESIGN PROCEDURE CHECK LIST

COLLECTING DATA

- Site specific data including topographic data, shielding, terrain, wind region, flood prone areas, snow ground load, proximity to marine environments
- Site class , geotechnical reports whenever relevant, in particular for Class P Sites
- Intended use of the building – residential, commercial, farm etc.
- Customer requirements related to serviceability – deflection limits, durability requirements (corrosion), R rating
- Specific customer requirements related to intended use of the building – ventilation, footings preferences, security, vermin insulation, doors and windows
- Possible extensions to the building or/and intended use of the building change in future
- Extra roof loads due to equipment, vents, solar panels, lining etc.
- Floor loads due to vehicle traffic, equipment
- Geometry of the building with openings

MODELLING

3D or set of 2D structural models of the building should be prepared using appropriate software (capable of performing analysis) including major and secondary structural elements:

- Major framing elements – portal columns and rafters, knee and apex braces, end and mullion columns
- Purlins and Girts (bridging modelling may not be necessary)
- Cross bracing (strap, rod, angle or similar)
- Equivalent diaphragm bracing where relevant
- Secondary framing elements – door framing, equipment support framing etc.

LOADING

- All necessary primary loads should be applied to appropriate elements of the building (members, cladding etc.) related to:
  - UDL wind load
  - Local pressure wind load – necessary for elements supporting cladding such as purlins/girts only
  - Self weight of structures
  - Superimposed Dead Load – equipment, architectural elements etc.
  - Snow load when applicable
  - Live Load
  - Flood pressure load when applicable

- All necessary load combinations with safety factors as necessary

ANALYSIS

- 1^{st} or 2^{nd} order analysis chosen
- Full section properties or effective section properties as necessary for serviceability checks for members
- Spring supports at column bases when applicable
- Tension only members (strap, rod bracing) when applicable
- Nodes and members releases relevant to connections design
- Materials (steel)

DESIGN OF MEMBERS

- Cold-formed steel members:
  - Section moment capacity
  - Member moment capacity – Lateral Buckling
  - Member moment capacity – Distortional Buckling
  - Shear capacity
  - Compression section capacity
  - Compression member capacity
  - Compression member capacity – distortional buckling
  - Combined compression & bending (a)
  - Combined compression & bending (b)
  - Combined compression & bending (c)
  - Tension section capacity
  - Combined bending & tension (a)
  - Combined bending & tension (b)
  - Combined bending & shear
  - Bearing capacity
  - Combined bending and bearing
- Hot-rolled steel members are not a subject of this Design Guide. An example for SHS sections only is presented:
  - Combined bending and compression of SHS sections
  - Combined bending and tension of SHS sections
  - Combined bending and shear of SHS sections
- Deflections

DESIGN OF CONNECTIONS

- Design of connections based on tests data or by calculations;
  - Bolted connections
  - Welded connections
  - Connections with screws, rivets
- Design of connection elements (assemblies) including members, connection components and connectors

DRAWINGS

- Structural drawings
- Architectural drawings
- Workshop steel detailing drawings

INSTALLATION GUIDES

It may necessary to prepare erection (assembly) guide subject to customer requirements which may include:

- Concrete formwork, concrete pour, finishing and curing requirements
- Sequence of erection
- Complex installation procedures (sliding doors, some connections etc.)
- Cranes requirements
- Construction methods including temporary bracing and restraints requirements
- Bolt tensioning requirements when applicable

CERTIFICATION

State specific certifications may be required to approve shed's design:

- Form 15 for Queensland
- Form 40 for NT
- Form 55 for Tasmania
- Form 1507 for Victoria

APPENDIX 4

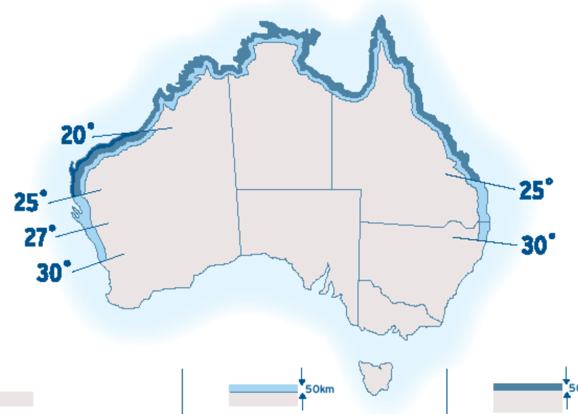
PRO-FORMA CERTIFICATE – Rectangular Buildings

DESIGN INFORMATION – Sheds & Garages			
Pro-forma request for design information by building certifier if design information supplied by shed supplier is inadequate			
LINE	**ITEM**	**DESIGN VALUE**	**NOTES**
Compliance Details			
1	Shed supplier		
2	Structural designer		
3	Certifying authority		
Building Details			
4	Building description		
5	Specification reference & date		
6	Owner's stated intended use		
7	BCA classification		
8	Length (m)		
9	Width (m)		
10	Height – maximum (m)		
11	Height to eave (m)		
12	Roof pitch (degrees)		
13	Internal pressure coefficient		
14	Average C_{pe} roof		
15	Average C_{pe} walls		
16	Local pressure effects applied?		
Site Details			
17	Site address		
18	Site plan reference & date		
19	Wind region		
20	Importance level		
21	Annual probability of exceedance for wind		
22	Cyclonic factor (F_C, F_D) (if applicable)		
23	Regional wind speed (V_R)		
24	Wind direction multiplier		
25	Terrain category		
26	Terrain-height multiplier		
27	Shielding multiplier		
28	Topographic multiplier		
29	Site wind speed (V_{sit})		
30	Design wind speed (V_{des})		

APPENDIX 5
Poster

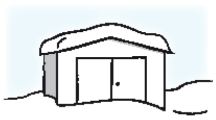
Choose The Right Shed
Shed Design Criteria Selector

Location: Wind Regions For Australia (AS/NZS 1170:2)

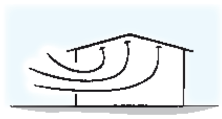


		50km	50km	50km
Wind Region	Region A1 - A5	Region B	Region C	Region D
Regional Wind Speed	45 m/s (metres per second)	57 m/s	69 m/s	88 m/s
Farm Sheds Wind Speed	41 m/s	48 m/s	64 m/s	79 m/s

Extreme Weather Conditions

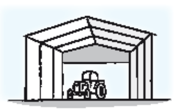
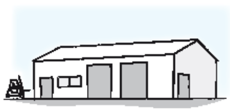


Snow loading may be required at higher altitudes



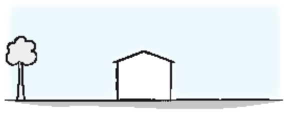
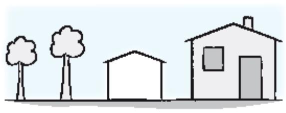
Cyclonic designs are required for regions C and D

Building Use

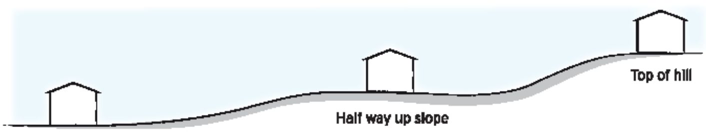

Building Use	Domestic	Farm Shed	Industrial/Commercial	Institutional
Design Importance Level	2	1	2	2 or 3 or 4

Terrain


Site Terrain	Open	Semi-open	Suburban
Terrain Category	TC 2	TC 2.5	TC 3

Topography



AUSTRALIAN STEEL INSTITUTE
STEEL SHED GROUP
It's an Australian Standard

Your Steel Shed Group member can assist with a shed design to comply to current Australian standards

The design criteria is required to be confirmed by your building surveyor or local council

APENDIX 6

WORKED EXAMPLES – DESIGN WIND SPEED

EXAMPLE 1: REGION A (NON-CYCLONIC)

BUILDING DESCRIPTION

- The project is a 6 m x 6 m x 3.0 m high double garage with twin roller doors, in the vicinity of a house on a 1000 m^2 allotment in outer suburban Melbourne, less than 70 km from Melbourne GPO. The precinct is fully developed with housing and associated buildings and structures. The building will be used for the garaging of private vehicles and other domestic activities such as workshop and storage. As a domestic building, it is a reasonable assumption that the main roller doors of the building will be closed during high winds provided this assumption is communicated to and accepted by the owner.
  - *The building is not a dwelling, but its use is associated with domestic purposes.*
  - *The BCA Classification of the building is 10a, which is appropriate for a non-habitable shed, garage or carport. There are no structural implications of this classification.*
  - *The building doors will be assumed closed during peak wind events. Internal pressure consistent with enclosed buildings may be used for structural design.*

SITE FACTORS

- Check region with Council.
  - *The Council has confirmed in writing that the allotment on which the proposed garage will be built is located in Region A5, as defined in AS/NZS 1170.2.*
- The consequences of structural failure are considered to be *moderate* in terms of human hazard (because the building is associated with domestic use) and *moderate* in terms of impact on the public (because the allotment is in a residential zone).
  - *Importance Level 2 is assigned – this is consistent with residential outbuildings generally. Importance Level 1 can only be justified if both hazard and impact of failure are low.*
  - *Importance Level 2 requires an annual probability of exceedance for wind events of 1:500.*
- The exact orientation of the building and roller door orientation may be design factors for an enclosed building in Region A5. However, in this case the design is to be based on “doors closed”.
  - *For Region A5, a wind direction multiplier of 1.00 is applied for all design cases. There is no dominant opening, so no structural design benefit can be gained by wind speed reduction in specific directions.*
- The general terrain of the property precinct is *suburban housing* in all directions. There is no reason to believe it would be redeveloped in any direction for non-housing purposes.
  - *The terrain is Category 3 with no change anticipated. A terrain/height multiplier of 0.83 is appropriate.*
- The proposed garage is well shielded by the house and other dwellings on adjoining blocks, with typically about 10 buildings in each direction. Effects of shielding should be considered.
  - *Evaluate shielding parameter (s) from AS/NZS 1170.2 Clause 4.3.3:*
    - *Average height of shielding buildings (h_s) is about 4 m.*
    - *Average breadth of shielding buildings (b_s) is about 9 m.*
    - *Roof height of garage being shielded (h) is 3 m.*
    - *Number of upwind shielding buildings (n_s) is about 10.*
    - *Shielding parameter s = (3 x (10/10 + 5))/(4 x 9)^$^{0.5}$ = 3.0*
    - *Look up shielding multiplier in Table 4.3, M_s = 0.8*

- The site and surrounding geography are essentially flat and level.
    - *There is no reason to apply a topographic factor higher than 1.0.*
- The steps in calculation of site wind speed are:
    - *Look up regional wind speed for region A5 and 1:500, $V_R = 45$*
    - *Wind directional multiplier for region A5, $M_d = 1.00$*
    - *Look up terrain/height multiplier $M_{z, cat} = 0.83$*
    - *Look up shielding multiplier $M_s = 0.8$*
    - *Look up topography multiplier $M_t = 1.0$*
    - *Calculate $V_{sit} = V_R \times M_d \times M_{z, cat} \times M_s \times M_t$*
    - *Value for this example, $V_{sit} = 45 \times 1.00 \times 0.83 \times 0.8 \times 1.0 = 30$ m/s*
    - *In this case, as the building orientation is irrelevant this is also the design wind speed V_{des}*
- The calculated design wind speed is then used to calculate the design wind pressures acting on various parts of the structure in accordance with AS/NZS 1170.2 Clause 2.4.

DESIGN INFORMATION – Sheds & Garages – Example 1

Pro-forma request for design information by building certifier if design information supplied by shed supplier is inadequate

LINE	ITEM	DESIGN VALUE	NOTES
Compliance Details			
1	Shed supplier		
2	Structural designer		
3	Certifying authority		
Building Details			
4	Building description	Supplied	Owner
5	Specification reference & date	Supplied	Owner
6	Owner's stated intended use	Machinery shed	Owner
7	BCA classification	10a	BCA
8	Length (m)	6.0 m	Owner's plans
9	Width (m)	6.0 m	Owner's plans
10	Height – maximum (m)	3.0 m	Owner's plans
11	Height to eave (m)	2.4 m	Owner's plans
12	Roof pitch (degrees)	20 deg	AS/NZS 1170.2, Tables 5.1A&B & 5.2A&B
13	Internal pressure coefficient	+ 0.2	AS/NZS 1170.2, Tables 5.3A, B & C
14	Average C_{pe} roof	- 0.7	AS/NZS 1170.2, Tables 5.2A, B & C
15	Average C_{pe} walls	+ 0.7, - 0.5	AS/NZS 1170.2, Table 5.6
16	Local pressure effects applied?	Yes	
Site Details			
17	Site address		
18	Site plan reference & date	Supplied	Owner
19	Wind region	A5	Council
20	Importance level	2	BCA Guide and Volume 1
21	Annual probability of exceedance for wind	1:500	BCA Volume 1 Table 1.2b
22	Cyclonic factor (F_C, F_D) (if applicable)	N/A	AS/NZS 1170.2 Clause 3.4
23	Regional wind speed (V_R)	45 m/s	AS/NZS 1170.2, Table 3.1
24	Wind direction multiplier	1.0	AS/NZS 1170.2, Table 3.2
25	Terrain category	3	AS/NZS 1170.2, Clause 4.2.1
26	Terrain-height multiplier	0.83	AS/NZS 1170.2, Table 4.1(A)
27	Shielding multiplier	0.8	AS/NZS 1170.2, Clause 4.3
28	Topographic multiplier	1.07	AS/NZS 1170.2, Clause 4.4
29	Site wind speed (V_{sit})	30 m/s	AS/NZS 1170.2, Clause 2.2
30	Design wind speed (V_{des})	30 m/s	AS/NZS 1170.2, Clause 2.2

EXAMPLE 2: REGION B (NON-CYCLONIC)

BUILDING DESCRIPTION

- The project is a 6 m x 6 m x 3.0 m high double garage with twin roller doors, in the vicinity of a house on a 1000 m^2 allotment in suburban outer Brisbane, less than 100 km from the coast. The precinct is fully developed with housing and associated buildings and structures. The building will be used for the garaging of private vehicles and other domestic activities such as workshop and storage. As a domestic building, it is likely that the main roller doors of the building will be closed during high winds.
  - *The building is not a dwelling, but its use is associated with domestic purposes.*
  - *The BCA Classification of the building is 10a, which is appropriate for a non-habitable shed, garage or carport. There are no structural implications of this classification.*
  - *The building doors may be assumed closed during peak wind events. Internal pressure consistent with dwellings may be used for structural design.*

SITE FACTORS

- Check region with Council.
  - *The Council has confirmed in writing that the allotment on which the proposed garage will be built is located in Region B, as defined in AS 1170.2.*
- The consequences of structural failure are considered to be *moderate* in terms of human hazard (because the building is associated with domestic use) and *moderate* in terms of impact on the public (because the allotment is in a residential zone).
  - *Importance Level 2 is assigned – this is consistent with residential outbuildings generally. Importance Level 1 can only be justified if both hazard and impact of failure are low.*
  - *Importance Level 2 requires an annual probability of exceedance for wind events of 1:500.*
- The exact orientation of the building and roller door orientation are not design factors for an enclosed building in Region B.
  - *For Region B, a wind direction multiplier of 0.95 is applied.*
- The general terrain of the property precinct is *suburban housing* in all directions. There is no reason to believe it would be redeveloped in any direction for non-housing purposes.
  - *The terrain is Category 3 with no change anticipated. A terrain/height multiplier of 0.83 is appropriate.*
- The proposed garage is well shielded by the house and other dwellings on adjoining blocks, with typically about 10 buildings in each direction. It appears to be worthwhile to consider the effects of shielding.
  - *Evaluate shielding parameter (s) from AS/NZS 1170.2 Clause 4.3.3:*
    - *Average height of shielding buildings (h_s) is about 4 m.*
    - *Average breadth of shielding buildings (b_s) is about 9 m.*
    - *Roof height of garage being shielded (h) is 3 m.*
    - *Number of upwind shielding buildings (n_s) is about 10.*
    - *Shielding parameter s = (3 x (10/10 + 5))/(4 x 9)^$^{0.5}$ = 3.0*
    - *Look up shielding multiplier in Table 4.3, M_s = 0.8*
- The site and surrounding geography are essentially flat and level.
  - *There is no reason to apply a topographic factor higher than 1.0.*
- The steps in calculation of site wind speed are:

  - *Look up regional wind speed for region B and 1:500,* $V_R = 57$
  - *Wind directional multiplier for region B,* $M_d = 0.95$
  - *Look up terrain/height multiplier* $M_{z, cat} = 0.83$
  - *Look up shielding multiplier* $M_s = 0.8$
  - *Look up topography multiplier* $M_t = 1.0$
  - *Calculate =* $V_R \times M_d \times M_{z, cat} \times M_s \times M_t$
  - *Value for this example,* $V_{sit} = 57 \times 0.95 \times 0.83 \times 0.8 \times 1.0 = 36$ *m/s*
  - *In this case, as the building orientation is irrelevant this is also the design wind speed* V_{des}
- The calculated design wind speed is then used to calculate the design wind pressures acting on various parts of the structure in accordance with AS/NZS 1170.2 Clause 2.4.

DESIGN INFORMATION – Sheds & Garages – Example 2			
Pro-forma request for design information by building certifier if design information supplied by shed supplier is inadequate			
LINE	**ITEM**	**DESIGN VALUE**	**NOTES**
Compliance Details			
1	Shed supplier		
2	Structural designer		
3	Certifying authority		
Building Details			
4	Building description	Supplied	Owner
5	Specification reference & date	Supplied	Owner
6	Owner's stated intended use	Machinery shed	Owner
7	BCA classification	10a	BCA
8	Length (m)	6.0 m	Owner's plans
9	Width (m)	6.0 m	Owner's plans
10	Height – maximum (m)	3.0 m	Owner's plans
11	Height to eave (m)	2.4 m	Owner's plans
12	Roof pitch (degrees)	20 deg	AS/NZS 1170.2, Tables 5.1A&B & 5.2A&B
13	Internal pressure coefficient	+ 0.2	AS/NZS 1170.2, Tables 5.3A, B & C
14	Average C_{pe} roof	- 0.7	AS/NZS 1170.2, Tables 5.2A, B & C
15	Average C_{pe} walls	+ 0.7, - 0.5	AS/NZS 1170.2, Table 5.6
16	Local pressure effects applied?	Yes	
Site Details			
17	Site address		
18	Site plan reference & date	Supplied	Owner
19	Wind region	B	Council
20	Importance level	2	BCA Guide and Volume 1
21	Annual probability of exceedance for wind	1:500	BCA Volume 1 Table 1.2b
22	Cyclonic factor (F_C, F_D) (if applicable)	N/A	AS/NZS 1170.2 Clause 3.4
23	Regional wind speed (V_R)	57 m/s	AS/NZS 1170.2, Table 3.1
24	Wind direction multiplier	0.95	AS/NZS 1170.2, Table 3.2
25	Terrain category	3	AS/NZS 1170.2, Clause 4.2.1
26	Terrain-height multiplier	0.83	AS/NZS 1170.2, Table 4.1(A)
27	Shielding multiplier	0.8	AS/NZS 1170.2, Clause 4.3
28	Topographic multiplier	1.07	AS/NZS 1170.2, Clause 4.4
29	Site wind speed (V_{sit})	36 m/s	AS/NZS 1170.2, Clause 2.2
30	Design wind speed (V_{des})	36 m/s	AS/NZS 1170.2, Clause 2.2

EXAMPLE 3: REGION C (CYCLONIC)

BUILDING DESCRIPTION

- The project is a 9 m long x 6 m span x 3.8 m high open-sided machinery shed in a specific location on an undulating produce farm near Rockhampton, approximately 25 km from the coast. The shed will be located in a paddock near the dwelling, but remote from any other dwellings or sheds. The building will be used for the garaging and servicing of farm machinery.
  - o *The building is not a dwelling, or associated with any domestic purpose.*
  - o *The BCA Classification of the building is 10a, which is appropriate for a non-habitable shed, garage or carport. There are no structural implications of this classification.*

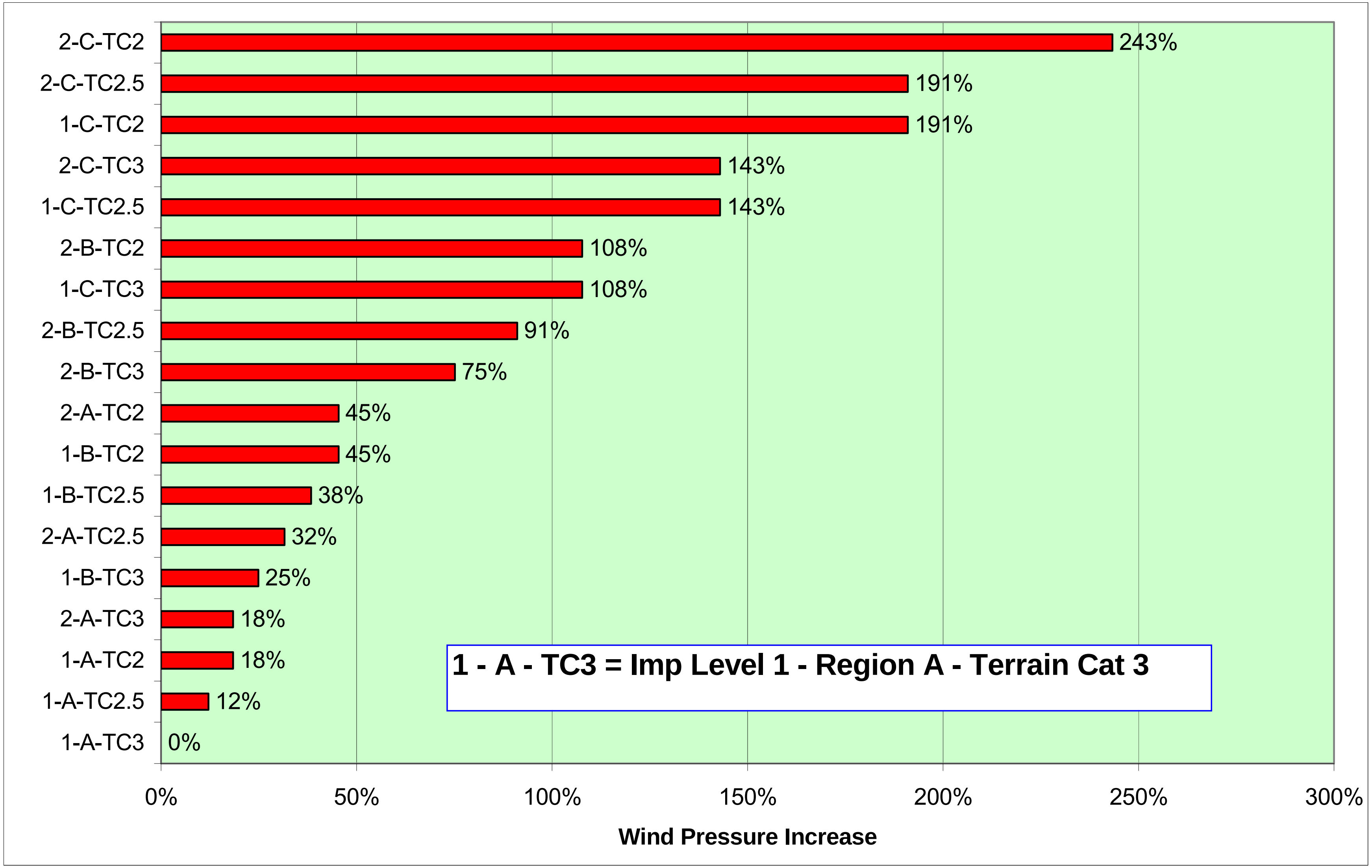
SITE FACTORS

- Check region with Council.
  - o *The Council has confirmed in writing that the allotment on which the proposed shed will be built is located in Region C, as defined in AS/NZS 1170.2. It is not exempt development under the Standard Building Regulations.*
- The consequences of structural failure are considered to be *low* in terms of human hazard (because the shed is not near or associated with a dwelling) and *low* in terms of impact on the public (because the shed is on a large allotment remote from unrelated dwellings and other buildings).
  - o *Importance Level 1 can only be justified if both human hazard and impact of failure are low. Importance Level 1 is justifiable in this case, consistent with remote rural buildings generally.*
  - o *Importance Level 1 allows an annual probability of exceedance for wind events of 1:200 in a cyclonic area.*
- The exact orientation of the building is not shown on the site layout drawing, but this is not relevant to the wind direction multiplier in region C:
  - o *For Region C, a wind direction multiplier of 0.95 is applied for the design of complete buildings and major structural elements. A multiplier of 1.00 is used for all other design cases.*
- The general terrain of the property is observed to be cleared open farming land with relatively few trees or buildings. There is no indication of future rezoning and redevelopment plans in available Council documentation.
  - o *The terrain is currently Category 2, and is considered to be "fully developed" in the absence of confirmed future rezoning proposals.*
  - o *The building height of 3.8 m allows linear interpolation of terrain/height multiplier between 0.90 and 0.95 giving $M_{z,cat} = 0.92$.*
- The shed is in the open, unshielded by other buildings. There are no plans to construct other buildings in its immediate vicinity.
  - o *The proposed building must be regarded as unshielded. No shielding concession on wind speed is justified.*
- The site and surrounding geography are undulating, with the shed located on an extensive flat area about 50 metres from a gentle downslope, beyond which is a flat plain. The topography should be checked to see if it falls within the limits for a topographic factor of 1.0:
  - o *Only the hill shape multiplier applies for Australian sites $M_t = M_h$.*
  - o *The slope is found to have an overall height (H) of about 12 metres. The height drops to 6 metres about 75 metres down from the crest (L_u). Therefore the value $H/(2L_u) = 0.08$.*
  - o *This value is more than 0.05 but less than 0.45. Therefore apply formula 4.4(2) of AS/NZS 1170.2 which gives $M_h = 1.07$.*
- The steps in calculation of site wind speed are:
  - o *Look up regional wind speed for region C and 1:200, V_R = 64 m/s*
  - o *Select wind directional multiplier $M_d = 0.95$ for region C*
  - o *Interpolate terrain/height multiplier $M_{z,\,cat} = 0.92$*
  - o *Look up shielding multiplier $M_s = 1.0$*
  - o *Calculate topography multiplier $M_t = 1.07$*
  - o *Calculate $V_{sit} = V_R \times M_d \times M_{z,\,cat} \times M_s \times M_t$*
  - o *Value for this example $V_{sit} = 64 \times 0.95 \times 0.92 \times 1.0 \times 1.07 = 60$ m/s*
  - o *In this case, as the building orientation is irrelevant, this is also the design wind speed V_{des}*
- The calculated design wind speed V_{des} is then used to calculate the design wind pressures acting on various parts of the structure in accordance with AS/NZS 1170.2 Clause 2.4.

DESIGN INFORMATION – Sheds & Garages – Example 3

Pro-forma request for design information by building certifier if design information supplied by shed supplier is inadequate

LINE	ITEM	DESIGN VALUE	NOTES
	Compliance Details		
1	Shed supplier		
2	Structural designer		
3	Certifying authority		
	Building Details		
4	Building description	Supplied	Owner
5	Specification reference & date	Supplied	Owner
6	Owner's stated intended use	Machinery shed	Owner
7	BCA classification	10a	BCA
8	Length (m)	9.0 m	Owner's plans
9	Width (m)	6.0 m	Owner's plans
10	Height – maximum (m)	3.8 m	Owner's plans
11	Height to eave (m)	3.0 m	Owner's plans
12	Roof pitch (degrees)	20 deg	AS/NZS 1170.2, Tables 5.1A&B & 5.2A&B
13	Internal pressure coefficient	+ 0.7	AS/NZS 1170.2, Tables 5.3A, B & C
14	Average C_{pe} roof	- 0.7	AS/NZS 1170.2, Tables 5.2A, B & C
15	Average C_{pe} walls	+ 0.7, - 0.5	AS/NZS 1170.2, Table 5.6
16	Local pressure effects applied?	Yes	
	Site Details		
17	Site address		
18	Site plan reference & date	Supplied	Owner
19	Wind region	C	Council
20	Importance level	1	BCA Guide and Volume 1
21	Annual probability of exceedance for wind	1:200	BCA Volume 1 Table 1.2b
22	Cyclonic factor (F_C, F_D) (if applicable)	1.05	AS/NZS 1170.2 Clause 3.4
23	Regional wind speed (V_R)	64 m/s	AS/NZS 1170.2, Table 3.1
24	Wind direction multiplier	0.95	AS/NZS 1170.2, Table 3.2
25	Terrain category	2	AS/NZS 1170.2, Clause 4.2.1
26	Terrain-height multiplier	0.92	AS/NZS 1170.2, Table 4.1(A)
27	Shielding multiplier	1.0	AS/NZS 1170.2, Clause 4.3
28	Topographic multiplier	1.07	AS/NZS 1170.2, Clause 4.4
29	Site wind speed (V_{sit})	60 m/s	AS/NZS 1170.2, Clause 2.2
30	Design wind speed (V_{des})	60 m/s	AS/NZS 1170.2, Clause 2.2



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0%
50%
100%
150%
200%
250%
300%
Wind Pressure Increase


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