Survey of metal-clad, low-rise, low-pitch sheds in cyclonic regions

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INTRODUCTION

Low-rise sheds with spans of 12 to 30 m, lengths up to 50 m or more and a range of heights, roof shapes and pitch angles are used in industrial applications. The structural systems of these sheds generally consist of portal or pin-jointed frames, sometimes spaced evenly at the mid section and closer together at the gable-ends. Metal sheet cladding is attached to roof purlins and wall girts, which are fixed to these frames. Cross bracing between the end frames resist longitudinal (i.e. in direction of ridge-line) wind loads. Gable-ended low-pitch roofs (less than 10°) are most common, but moderate pitches to 20° and steeper pitches to 40° are also used. Wind loads for designing the envelope and structural system of these sheds are typically calculated using data given in wind load standards such as AS/NZS 1170.2 [1].

This paper describes wind load design parameters and components used in typical low-pitch roof metal-clad, metal framed sheds in cyclone region C of Australia, obtained from a survey carried out by Robertson [5]. Components at the windward edges of a low pitch roof generally experience the largest wind load, and are most susceptible to failure in a windstorm. These failures take place as a result of large net uplift wind loads caused by large external suction pressures combined with large positive internal pressures resulting from dominant windward wall openings. The design loads acting on the roof components are analyzed in detail, as the approach and data used by the shed designers is a significant factor of the shed's vulnerability to windstorms.

DESIGN APPROACH

Design of the structure and envelope is carried out by applying combinations of factored, permanent (dead), imposed (live) and wind actions (loads) obtained from the AS/NZS 1170 suite, and selecting components with the appropriate factored strength, obtained from manufacturer's data sheets [2], [3]. In cyclonic areas, wind loads are usually the critical design criterion, especially for cladding, purlins and girts and the selection of components is dependent on the data specified by designers.

Wind load effects for the design of cladding and primary structure on these sheds are usually calculated from pressures derived from nominal pressure coefficients, provided in AS/NZS 1170.2 [1]. The design pressures are calculated from Equation 1, where ρ is the density of air, V_h is the 3s-peak design gust wind speed at mid-roof height and C_{fig} is the aerodynamic shape factor. Quasi-steady internal pressure coefficients $C_{p,i}$ and external, pressure coefficients $C_{p,e}$ combined with factors for area-averaging K_a, loads on multiple surfaces K_c, and local-pressure effects, K_l are used to determine C_{fig} values for internal and external pressures. External and internal design pressures acting over the tributary area are combined to get the net design wind load, from which the wind load effect is calculated.

Edge regions are located within a distance a from the roof edges, where according to AS/NZS 1170.2 [1], 'a' is the minimum of ($0.2b$, $0.2d$, h). Areas of cladding, their fixings and purlins (and their fixings) less than $0.25a^2$ and a^2 respectively, supported by within $a/2$ and a from the windward roof edge are subjected to a local pressure factor K_l of 2.0 and 1.5 respectively. The external pressure coefficient in these regions is generally $C_{p,e} = -0.9$, and the internal pressure coefficient, $C_{p,i}$ for a nominally sealed building is 0.0 and for a building with a dominant windward wall opening is + 0.7.

$$p_{design} = 0.5\rho V_h^2 C_{fig} \qquad (1)$$

The nominal, 3s-peak gust wind speed at 10m elevation in terrain category 2 approach, V_N is modified by wind direction, terrain/height, shielding and topography multipliers M_d, $M_{z,cat}$, M_s and M_t respectively in Equation 2, to calculate V_h.

$$V_h = V_N M_d (M_{z,cat} M_s M_t) \qquad (2)$$

SHED SURVEY DATA

Low roof pitch ($\leq 10^{o}$), gable-ended metal-clad sheds of height (h), width (d), and length (b) shown in Figure 1, typically have a series of portal frames (or trusses) placed at regular intervals of between 4 m and 10 m along their length to which purlins are attached up to about 1.2 m apart. The roof cladding is screwed to the purlins by fasteners at a spacing of 150 to 200 mm. Often thicker gauge purlins are used in the end bays to account for the higher wind loads, and the purlins are usually lapped at the frames.

The geometric parameters for the ten sheds analyzed for this study are listed in Table 1. The shed wind loading parameters, as specified in the original design documents submitted to the local government (council) records, are listed in Table 2. The $C_{p,i}$ values, specified in the design documents, are included as the third and fourth columns of this table and have values ranging from 0.0 (nominally sealed) to +0.7 (dominant windward wall opening). However, the drawings for Sheds #1 to #8 inclusive showed that these sheds have roller doors and windows on their walls and so the design should allow for these openings to be "blown in" as discussed in Clause 5.3 of AS/NZS 1170.2 [1]. Consequently, this paper analyzes these sheds as being subjected to full positive internal pressure. By contrast, Sheds #9 and #10 are fitted with ridge vents and permanent louvre openings to at least three walls and so the positive design $C_{p,i}$ value is reduced to +0.2.

DESIGN WIND LOADS ON ROOF COMPONENTS

The performance of these sheds in windstorms is analysed by calculating wind loads on selected components on the roof, for a series of design scenarios. Ultimate limit state (500yr return period) design wind loads are determined for cladding, its fixings and purlins in the end bays and within a distance a from the windward edge, for winds across (Cross Wind, $\theta = 0°$) and along (Longitudinal Wind, $\theta = 90°$) the ridge-line using AS/NZS 1170.2 [1].

The design loads are then compared to the design capacities. Components that have a design capacity significantly less than the design wind load (i.e. the Capacity to Load ratio is less than say 0.8) are considered to have a High Risk of Failure (HRF). The design capacities were taken from manufacturer's published literature. Any published design data using Permissible Stress Design values were multiplied by 1.5 to convert them to equivalent Strength Limit State values.

SAMPLE CALCULATIONS – SHED # 6 (Six bay pitched roof portal frame shed)

Terrain Category 2, 10m height ultimate limit state wind speed = 69 m/sec

Design wind speed at mid roof height ($h = 5.5$ m, $M_{z,cat} = 0.96$, $M_s = 1.0$, $M_t = 1.0$) = 66.5 m/sec

Free stream dynamic pressure (q_z) = 0.6 x $(66.5)^2$ = 2,660 Pa = 2.66 kPa

External roof pressure coefficient for $\theta = 0°$ ($h/d = 0.44$, $C_{p,e} = -0.85$)
Design external pressure on cladding/fixings for $\theta = 0°$ ($K_l = 2.0$, $K_a = 1.0$, $K_p = 1.0$, $K_c = 1.0$) = -4.51 kPa

External roof pressure coefficient for $\theta = 90°$ (0 to 1h, $C_{p,e} = -0.9$: 1h to 2h, $C_{p,e} = -0.5$)
Design external pressure on cladding/fixings for $\theta = 90°$ ($K_l = 2.0$, $K_a = 1.0$, $K_p = 1.0$, $K_c = 1.0$) = -4.78 kPa

Internal pressure coefficients $C_{p,i} = +0.7, -0.65$ (Use positive pressure for max uplift)
Design internal pressure on cladding/fixings (Kc = 1.0) = 1.86 kPa

Local Pressure Factor: Dimension a = Min [0.2 Min(b, d), h] = 2.48 m

Design nett pressures on cladding & fixings	Cross Wind, $\theta = 0°$	Longit Wind, $\theta = 90°$
Maximum ($K_l = 2.0$) total uplift pressure to roof cladding (kPa)	-6.37	-6.64

Check Strength of Roof Cladding in Local Pressure Zone ($K_l = 2.0$)

Roof cladding: 0.42 mm rib/pan profile. Fixings not specified, so assume fixed without cyclone washers

For Cross Wind on end spans of 1000 mm, Limit state design strength (Capacity) = 4.0 kPa
Applied Load = -6.37 kPa
Ratio of Capacity to Load = 4.0/6.37 = **0.63** (HRF)

For Longit Wind on internal spans of 1075 mm, Limit state design strength (Capacity) = 4.7 kPa
Applied Load = -6.64 kPa
Ratio of Capacity to Load = 4.7/6.64 = **0.71** (HRF)

Check Strength of End Bay Internal Purlin for Longitudinal Wind ($\theta = 90°$)

End Bay Purlins: Z20019, 1 row bridging (End bay span = 6m, Overhang = 0.3 m, Load Width =1.075 m)

Calculate Uniformly Distributed Loads (UDLs) to Internal End Bay Purlin (see Figure 2)
For $K_l = 1.5$ Zone (length = 2.18 m), UDL = 1.075(1.5 x -0.9 – 0.7) 2.66 = -5.86 kN/m
For $K_l = 1.0$ Zone (length = 3.02 m), UDL = 1.075(-0.9 – 0.7) 2.66 = -4.57 kN/m
For $C_{p,e} = -0.5$ (Past 1h, length = 0.8 m), UDL = 1.075(-0.5 – 0.7) 2.66 = -3.43 kN/m

Calculate equivalent average UDL to End Bay Internal Purlin (deduct 0.05 kPa cladding self weight)
Equiv. UDL = (-5.86 x 2.18 –4.57 x 3.02 – 3.43 x 0.8)/6 + 0.05 x 1.075 = -4.83 kN/m

For Longit Wind on End Bay Internal Purlin (Z20019, 1 row bridging, Four lapped spans = 6m)
Outwards strength capacity = 4.4 kN/m
Applied Load = -4.83 kN/m
Ratio of Capacity to Load = 4.4/4.83 = **0.92** (Marginal)

The cladding design specifications (at the edges of the sheds) for the purlin spacings and fixings are presented in Table 3. The adequacy of the design criteria is represented by the Capacity to Load ratios. These ratios should be not smaller than 1.0, but it is likely that values of down to about 0.9, although undesirable are unlikely to initiate failure during a design windstorm with full internal pressure. However, for both cross and longitudinal wind directions there are two sheds where the Capacity to Load ratio is less than 0.65 with a minimum value of 0.58. The edge cladding to these sheds should be considered a High Risk of Failure (HRF) during a design wind event if full positive internal pressure is applied.

The results from a design check on the end bay Internal Purlins subjected to Longitudinal Wind are presented in Table 4 for nine sheds where the roof was supported by purlins. Three of the nine sheds checked have a Capacity to Load ratio of less than 0.65 and so these purlins should also be considered a High Risk of Failure (HRF). Two of these same sheds also had a HRF cladding design. Three of the four sheds designated HRF had $C_{p,i}$ values of less than + 0.7 specified in the design documents.

The purlin analysis performed for this study calculated a straight average of the different roof pressure blocks applied to the purlin. However, Woolcock et al [6] suggests that a weighted average of the extra peak load block be added to the base uniformly distributed load. They propose a multiplier of 1.3 for the additional (peak) load applied at the end of the purlin (the case reported in Table 4 of this paper). If this approach were adopted here, the equivalent applied UDLs would be even larger and give an increased risk of failure.

The authors also checked the second internal purlins to the shed internal bays when subjected to non-square (i.e. rectangular) $K_l = 1.5$ local pressure zones with cross wind loading and found that five out of these nine sheds had Capacity to Load ratios of less than 0.75, but these results are not reported here. These results reflect the authors' understanding that many designers mistakenly believe that the shape of the local pressure zones are restricted to squares.

CONCLUSIONS

The results from this study suggest that some designers are not accounting for the possibility of high internal pressures, nor allowing for the local pressure effects of wind loading to the shed roof edges. A further study with a larger sample size is needed to confirm this.

The results from an expanded study could also be used in vulnerability studies to prepare estimates for damage curves by plotting design wind velocity against percentage of sheds at risk of roof failure. Ginger et al [4] discuss this approach in the companion paper.

REFERENCES

1. AS/NZS 1170.2 Structural design actions – Part 2: Wind actions 2002.
2. Bluescope Lysaght (2003), "Cyclonic Area Design Manual – Steel Roofing and Walling"
3. Bluescope Lysaght (2003), "Lysaght Purlins & Girts - User's Manual"
4. Ginger J. D., Henderson D. J. & Leitch C. J (2006) "Vulnerability of metal-clad, low-rise, low-pitch sheds subjected to wind loads", 12th Australasian Wind Engineering Workshop.
5. Robertson W, (2005), "Vulnerability of sheds under cyclonic conditions", JCU BE Thesis
6. Woolcock S. T., Kitipornchai S. and Bradford M. A. (1999), "Design of Portal Frame Buildings", 3rd Edition, published by AISC.

Table 1 Shed Survey – Geometrical Data

Shed No.	Date on Shed Drawings	Shed Length l (m)	Shed Width d (m)	Average Roof Height h (m)	Roof Slope α (deg)	Local Pressure Zone Dimension a (mm)	Comments
1	Jan 98	36.6	17	9	5	3400	
2	Aug 04	24	15	5.7	10	3000	Kit shed
3	Oct 92	59	30.8	7.4	4.5	6160	
4	Mar 99	45.5	19.8	9.1	10	3960	
5	Feb 99	18.4	14.5	6.6	10	2900	Top hat battens as purlins
6	Oct 96	36.6	12.4	5.5	10	2480	
7	Dec 93	18.5	12.4	5.3	3	2480	Skillion Roof
8	Apr 98	40	31.6	10.8	5	6320	
9	Nov 00	112	27	9.6	5	5400	Ridge vent & louvres to 3 or 4 walls
10	Apr 97	29.2	24	7.5	3	4800	

Table 2 Shed Wind Loading Parameters (all located in Region C)

Shed No	Shed Wind Parameters ex Building Design Records (all with $M_t = 1.0$)								Pressure Coefficients Used		
	Terrain Category	Designer's $C_{p,i}$ + ve	Designer's $C_{p,i}$ - ve	Mid Roof Ht. (m)	$M_{z,cat}$	M_d	M_s	Site Wind Speed $V_{sit,\beta}$ (m/sec)	Cross Wind $C_{p,e}$	Longit Wind $C_{p,e}$	+ve Internal Pressure $C_{p,i}$
1	3	+0.5	-0.3	9.0	0.88	1.0	0.9	54.9	-0.90		
2	3	+0.7	-0.65	5.7	0.82	0.95	0.88	47.5	-0.80		
3	3	+0.7	-0.65	7.4	0.83	1.0	0.9	51.8	-0.90		
4	2	+0.3	-0.3	9.1	0.99	1.0	1.0	68.6	-0.87	-0.90	+0.7
5	2	+0.7	-0.65	6.6	0.96	1.0	0.95	63.2	-0.86		
6	2	0	-0.3	5.5	0.96	1.0	1.0	66.5	-0.85		
7	2	+0.7	-0.3	5.3	0.96	1.0	0.9	59.9	-0.90		
8	2	0	-0.3	10.8	1.0	1.0	0.97	67.2	-0.90		
9	2	+0.2	-0.3	9.6	0.98	0.95	1.0	64.5	-0.90	-0.90	+0.2
10	2	+0.2	-0.2	7.5	0.98	1.0	0.9	61.1	-0.90		

Table 3 Details, Design Loadings and Capacities of Roof Cladding in Local Pressure Zone (K_l = 2.0)
(All roof cladding is 0.42mm BMT rib/pan steel sheeting)

Shed No.	Dimens a	$C_{p,i}$	Cyclone Washers Fitted?	Cross Wind on End Spans				Longit Wind on Internal Spans			
		Used for Anal.		End Span	Design Capacity	Applied Load	Capacity to Load	Internal Span	Design Capacity	Applied Load	Capacity to Load
	(mm)		(See Note)	(mm)	(kPa)	(kPa)	Ratio	(mm)	(kPa)	(kPa)	Ratio
1	3400	+0.7	Y	1080	6.3	-4.5	1.40	1200	7.5	-4.5	1.66
2	3000		NS (N)	1000	4.0	-3.1	1.29	1300	3.8	-3.4	1.11
3	6160		NS (N)	900	4.4	-4.0	1.09	1200	4.1	-4.0	1.03
4	3960		NS (N)	900	4.4	-6.9	**0.64**	1200	4.1	-7.1	**0.58**
5	2900		N	800	5.1	-5.8	**0.88**	800	6.4	-6.0	1.07
6	2480		NS (N)	1000	4.0	-6.4	**0.63**	1075	4.7	-6.6	**0.71**
7	2480		N	750	5.5	-5.4	1.02	940	5.3	-5.4	0.99
8	6320		N	650	6.2	-6.8	0.92	1190	4.2	-6.8	**0.62**
9	5400	+0.2	NS (N)	1000	4.0	-5.0	**0.81**	1100	4.6	-5.0	0.92
10	4800		NS (N)	900	4.4	-4.5	0.98	1100	4.6	-4.5	1.02

Note: Not Specified (NS) on shed drawings whether or not cyclone washers were fitted. Assume not fitted.

Table 4 Details, Design Loads and Capacities of Internal End Bay Purlins in Local Pressure Zone (K_l = 1.5)
(Local Pressure Zone acts over first length ("a" – Overhang) & General Pressure Zone over remainder)

Shed No.	Longitudinal Wind Loading Data				Purlin End Bay Details & Design Capacities					Design Loads	
	Dimens a	$C_{p,i}$ Used	Design Wind Uplift Pressures (kPa)		Size	No. of Rows	End Bay Span	Load Width	Design Capacity	Applied Load	Capacity to Load
	(mm)		General	K_l = 1.5		Bridging	(mm)	(mm)	(kN/m)	(kN/m)	Ratio
1	3400	+0.7	-2.89	-3.71	Z20019	1	6000	1200	-4.4	-3.9	1.13
2	3000		-2.17	-2.78	Z15012	1	6000	1300	-1.5	-3.1	**0.50**
3	6160		-2.57	-3.30	Z20020	2	5800	1200	-6.0	-3.9	1.54
4	3960		-4.52	-5.79	Z20019	2	6400	1200	-3.8	-6.2	**0.62**
5	2900		-3.83	-4.91	TH96-120	0	4500	800	No Data	-3.6	No Data
6	2480		-4.25	-5.44	Z20019	1	6000	1075	-4.4	-4.8	0.92
7	2480		-3.44	-4.41	Z15020	2	6000	940	-2.9	-3.4	0.87
8	6320		-4.34	-5.56	Z30024	2	10000	1190	-3.8	-5.9	**0.64**
9	5400	+0.2	-2.75	-3.87	Z20019	2	7000	1100	-3.1	-3.9	0.81
10	4800		-2.47	-3.47	Z20019	2	7150	1100	-3.0	-3.4	0.88

Note: Applied loads to the internal end bay purlins are calculated as the average of the local pressure zone load (K_l= 1.5) and the general pressure zones (with both $C_{p,e}$ = -0.9 and –0.5 as appropriate) and allow for the end bay purlin overhang (varies between 250 & 500 mm).

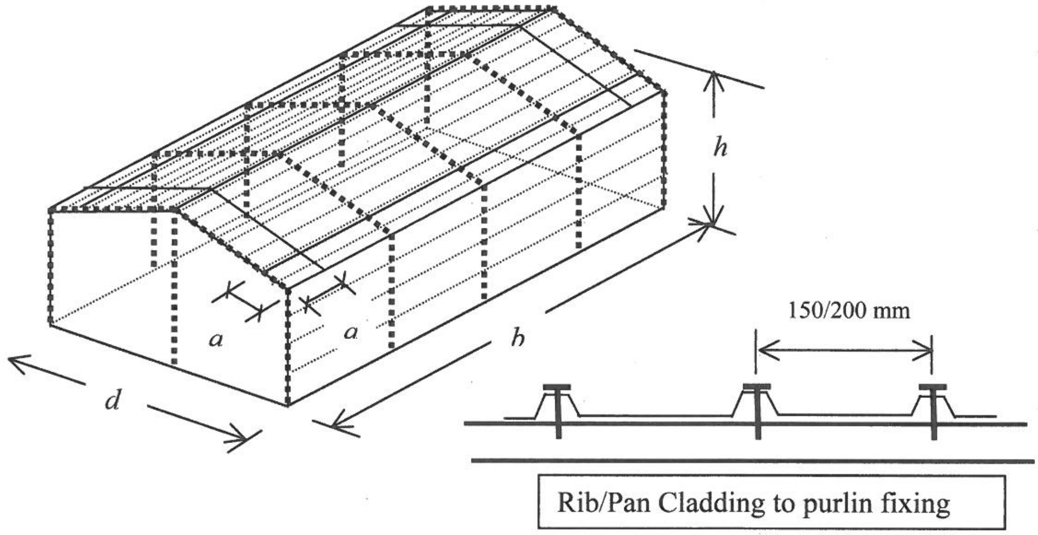

Figure 1 Low-pitch (<10°) building with height (h), span (d), to length (b), showing roof edge regions and typical frame and purlin layout, purlin-frame connection and cladding fixing

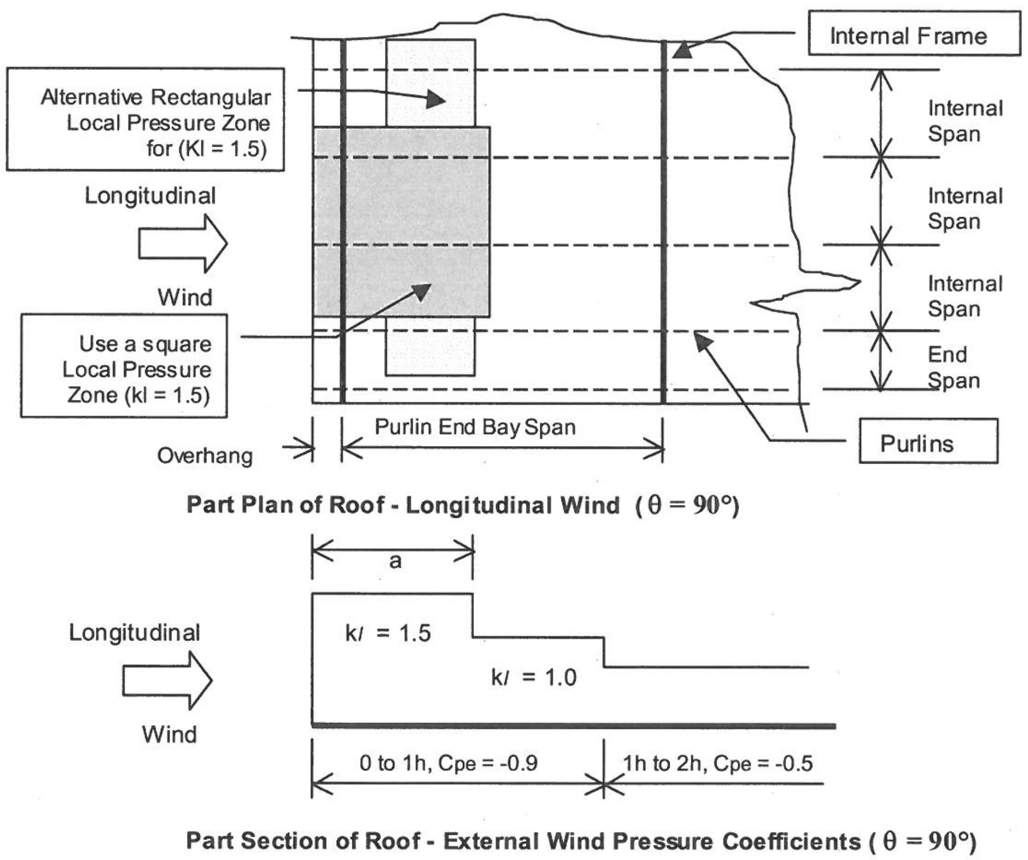

Figure 2 Roof Wind Pressures on End Bay of Shed – Longitudinal Wind