Vulnerability of metal-clad, low-rise, low-pitch sheds subjected to wind loads

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INTRODUCTION

Low-rise metal-clad, metal framed industrial sheds can be categorised by differences in size, shape, cladding type, roof shape and pitch, potential openings in envelope, structural system and age. The resilience of these sheds to wind loads is dependent on these features and the strength of their components and connections. Types of components and their strengths vary between the shed types and even within each type, because of variations in materials and construction. The vulnerability of these sheds to windstorms also depends on the approach terrain and topography, the wind speed and nature of the storm.

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. This paper describes the vulnerability of typical low-pitch roof metal-clad, metal-framed sheds in cyclone region C of Australia to wind-induced failures when subjected to a range of wind speeds. Probabilistic models of the wind load and component strength are applied using methods developed by Pham et al [1], Holmes [2], Pham [3] and Melchers [4], to determine the vulnerability of these sheds in windstorms. Statistical parameters are used to account for the uncertainty and variability associated with wind load and component strength.

DESIGN APPROACH

Criteria adopted in structural design standards used in Australia, are related to a specified limit state, such as the ultimate limit state of component or structural failure. AS/NZS 1170.0 [5] provides calibrated combinations of factored, permanent (dead), imposed (live) and wind actions (loads) to be applied on structural components and checked against their factored resistances. The basic framework for probability based, limit state design is provided by reliability theory. In this approach, the loads and resistances are random variables and the required statistical information is assumed to be available.

In this study, the critical design action is wind load, with component failure taking place when its strength, R is exceeded by the wind load, W. Data on both wind load and component strength are required in order to calculate the risk of component failure or reliability. The information required is the probability distributions of wind load and strength variables, and estimates of their mean and standard deviation or coefficient of variation (COV).

Wind load effects for the design of cladding and primary structure on these buildings are usually calculated from pressures derived from nominal pressure coefficients, provided in AS/NZS 1170.2 [6]. 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 nominal, net design wind load, W_N from which the wind load effect is calculated.

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

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.

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

METAL-CLAD SHEDS

Low roof pitch (<10°), 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 4m and 10m along its 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 frame supports.

The reliability of cladding fixings and purlins in regions near the roof edges of, low-pitch metal clad roof sheds, are estimated. Edge regions are located within a distance '*a*' from the roof edges, where according to AS/NZS 1170.2 [6], '*a*' is the minimum of (0.2*b*, 0.2*d*, *h*). External pressure coefficient $C_{p,e} = -0.9$, and local pressure factor K_l of 2.0 and 1.5 are generally applicable to cladding, fixings and purlin on tributary areas less than $0.25a^2$ and $0.25a^2$ to a^2 within distances of $a/2$ and a respectively from the windward edge. The area reduction factor is taken as $K_a = 1.0$. 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.

WIND LOADS – PROBABILISTIC MODEL

Wind load, *W* acting on roof components are given by the probabilistic model in Equation 3, where *V* is the maximum 3s gust velocity at 10m height in terrain category 2 in 50 yrs (lifetime) and the parameter *B* includes all the other components of the wind load. Pham et al [1] and Holmes [2] used a similar model to describe the wind load component in the limit state design approach used in AS/NZS 1170.2 [6].

$$W = B\,V^2 \qquad \text{where, } B = \lambda.\,A\,(C.\,E^2.\,D^2.\,G.\,\rho/2) \tag{3}$$

The variables within the bracket can be directly related to the nominal values given in AS/NZS 1170.2 [6], where, *C* is the quasi-steady pressure coefficient, *E* is a velocity height multiplier that accounts for the exposure and height, *D* is a factor for wind directionality effects, *G* is a factor that accounts for gusting effects and is related to K_a, K_c and K_l, and ρ is the density of air. The variable, λ is a factor to account for inaccuracies and uncertainties in analysis, and *A* is the tributary area.

The nominal values of these parameters are combined to give B_N which is used to deduce the nominal design wind load, W_N from Equation 4, where V_N is the ultimate limit state design wind speed typically with a mean return period of 500 to 1000yrs.

$$W_N = B_N\,V_N^2 \qquad \text{where, } B_N = \lambda_N\,A_N\,(C_N.\,E_N^2.\,D_N^2.G_N.\,\rho_N/2) \tag{4}$$

$$\text{Giving } [W/W_N] = [B/B_N]\,[V/V_N]^2 = ([\lambda/\lambda_N]\,[A/A_N]\,[C/C_N]\,[E/E_N]^2\,[D/D_N]^2[G/G_N]\,[\rho/\rho_N])\,[V/V_N]^2 \tag{5}$$

Each of the variables contained in *B* are assumed to have a log-normal probability distribution with assumed mean and coefficient of variation (COV), deduced from surveys and other studies [1, 2, 3]. Statistical data of these variables is used to estimate the mean and COV of the random variable *B*, which also has a log-normal probability distribution. In these assumptions, values in AS/NZS 1170.2 [6] are generally considered conservative, on average, especially when calculating design wind load effects on the primary structure. Pressure coefficients, related factors and multipliers prescribed in standards are mainly derived from wind tunnel model studies. Such model studies have shortcomings resulting from incorrect Reynolds No., insufficient details, and incorrect turbulence scaling. These deficiencies can in some cases underestimate pressures especially on small tributary areas near windward roof edges.

Table 1 gives assumed mean and COVs of the normalized parameters contained in $[B/B_N]$ for determining wind loads on cladding and purlins. The estimation of these values is a difficult procedure requiring extensive data, especially at full-scale. Scarcity of such data means that a considerable amount of knowledge and experience is needed when estimating these values. Applying these assumed values, the variable $[B/B_N]$ which itself has a log-normal distribution with a mean value of 0.69 and COV of 0.30. The distribution of wind loads on the cladding and purlins in roof edge regions are then calculated for range of wind velocities, V, at the reference height of 10m in terrain category 2.

STRENGTH OF COMPONENTS - PROBABILISTIC MODEL

Robertson [7] surveyed sheds in cyclonic regions, to determine the structural form of roofs and cladding, and to ascertain load parameters and design capacities used by engineers. He found that minimal design internal pressures were applied in a significant proportion of the sheds. Design capacities (ΦR_N where Φ is the capacity reduction factor and R_N is the nominal strength) provided by cladding and purlin manufacturers are matched against the design wind loads calculated by engineers to select the cladding, purlin and fixing types and spacings. Determination of the external pressure coefficients on such sheds is straightforward, but the internal pressure coefficients are based on a judgement of the size and locations of openings in the envelope. For instance, if the shed is considered to be nominally sealed or if all its walls are equally permeable, then a $C_{p,i} = 0$ is appropriate. However, the internal pressure will be significantly increased ($C_{p,i} = +0.7$) if a dominant opening is created in a wall, say by the impact of flying debris or failure of a roller door. Table 2 presents assumed mean and COV of cladding and purlin strengths based on log-normal probability distributions and limited analysis.

RELIABILTY OF CLADDING FIXINGS AND PURLINS

The probability of failure of cladding fixings and purlins with varying wind speed V for sheds designed with $C_{p,i}$ of 0 and +0.7 are given in Figures 2a and 2b respectively. The analysis assumes progressively increasing damage responsible for an increasing percentage of dominant windward wall openings in the buildings causing large positive internal pressures. In the failure modes considered here, it is assumed that failure of a cladding fixing eliminates the possibility of purlin failure and vice-versa. Furthermore, the incremental failures in both modes with increasing wind speeds are calibrated and weighted proportionally to give the total combined failure. Figures 2a and 2b show that the cladding fixings are most vulnerable to wind damage, with about 30% and 7% combined failure at the ultimate limit state reference wind speed of 70 m/s, when the sheds are designed with $C_{p,i}$ values of 0 and +0.7 respectively. According to loading standards (i.e. AS/NZS 1170.0 [5]) the failure probability of typical primary structural components is calibrated to about 10^{-3}.

CONCLUSIONS

The wind-induced failure of cladding fixings and purlins in metal-clad, low-pitch roof, low-rise sheds in cyclonic regions of Australia, are estimated by applying probability concepts. Variations in cladding and purlin strengths and the wind loads on edge regions of the roof are considered in probabilistic terms. The statistical analysis carried out shows that cladding and purlins are susceptible to wind induced failures. The combined cladding fixing and purlin probability of failure is about 30% at an ultimate limit state reference wind speed of 70 m/s when the sheds are designed with $C_{p,i} = 0$.

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Table 1. Estimated mean and COVs of normalized parameters of B

Parameter	Cladding and Purlin	
	Mean	COV
λ/λ_N	0.95	0.05
A/A_N	1.00	0.05
C/C_N	0.90	0.15
E/E_N	0.90	0.10
D/D_N	0.95	0.05
G/G_N	1.10	0.10
ρ/ρ_N	1.00	0.02

Table 2. Assumed mean and COVs of component resistances

Component	Mean $R/\Phi R_N$	COV R
Cladding	1.2	0.20
Purlins	1.4	0.15

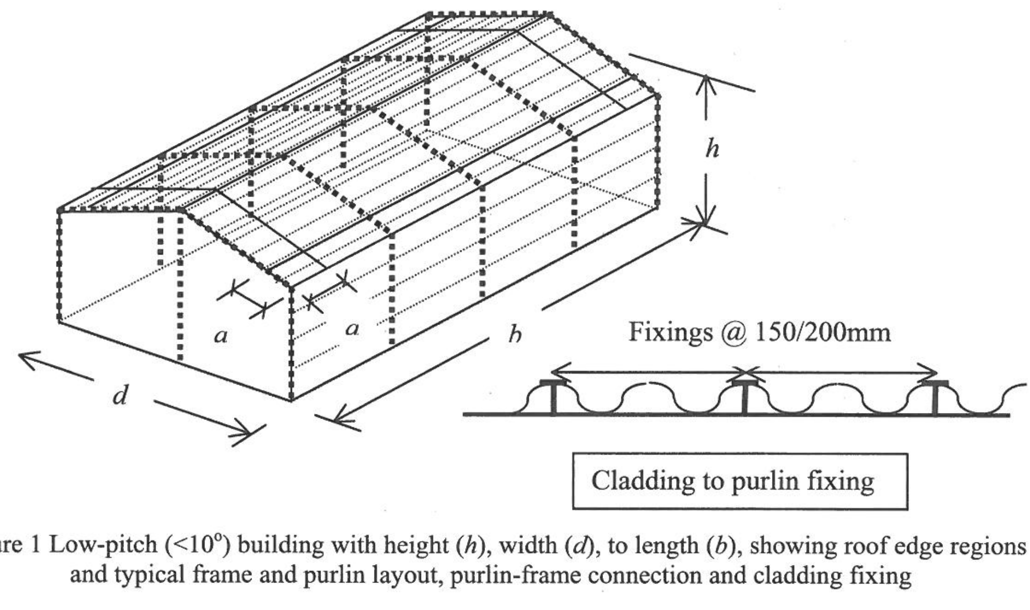

Figure 1 Low-pitch (<10°) building with height (h), width (d), to length (b), showing roof edge regions and typical frame and purlin layout, purlin-frame connection and cladding fixing

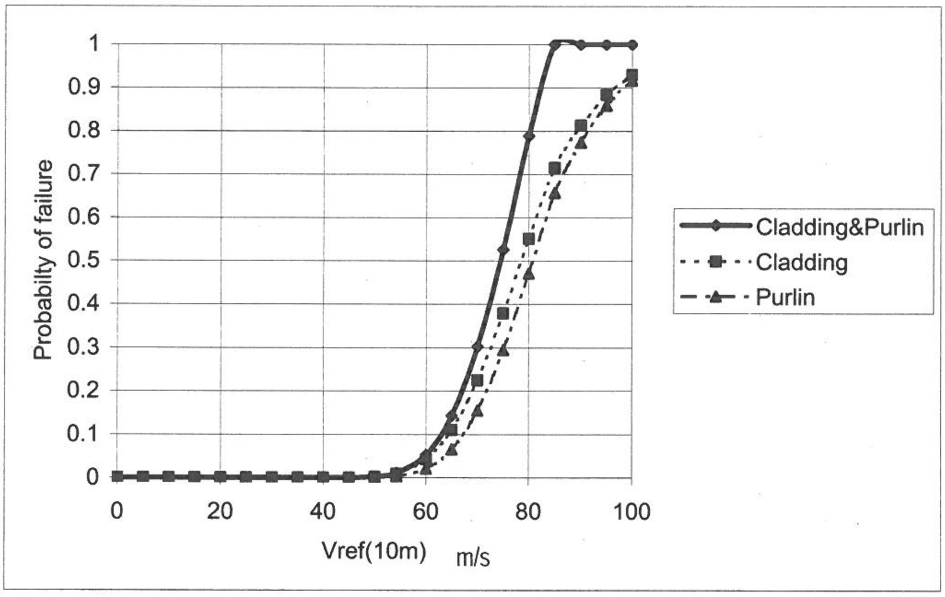

Figure 2a. Probability of failure vs wind speed – cladding and purlins. Design $C_{p,i} = 0$

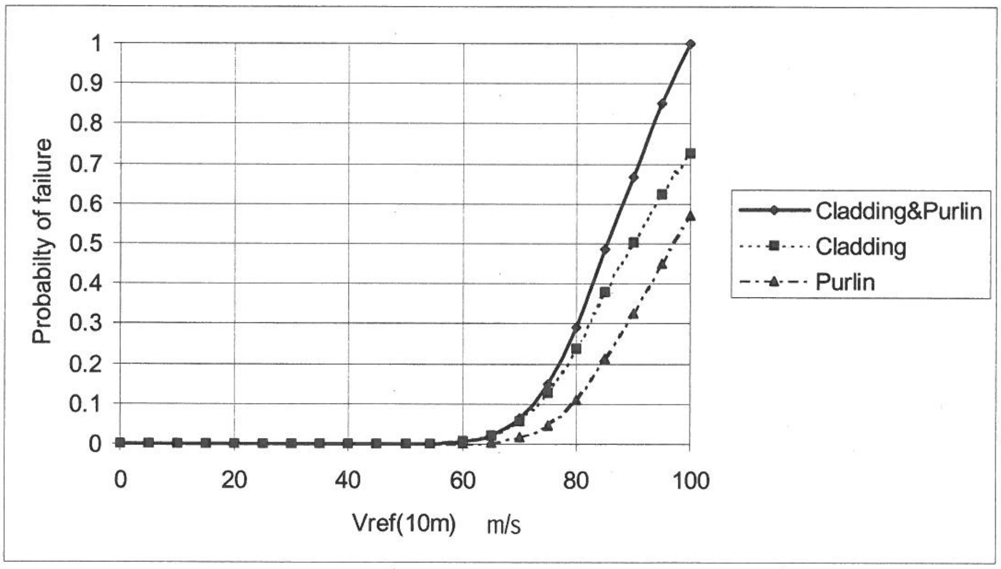

Figure 2b. Probability of failure vs wind speed – cladding and purlins. Design $C_{p,i} = +0.7$