WIND LOADING OF A ROOF PURLIN
Part II. Comparisons with AS 1170.2

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INTRODUCTION

The application of local pressure factors (LPF's) given in the Australian Standard for Wind Loads, AS 1170.2 [1] to the design of roof purlins has not been fully explained in the Commentary on that code [2], or any other widely distributed publication. A variety of interpretations of Clause 3.4.5 of the Standard (the clause relevant to LPF's) are appearing in practice, not all of which meet the intent of the Standard.

This paper describes the application of the LPF's to the design of roof purlins so that the intent of Clause 3.4.5 in AS 1170.2 is met. It also compares the results of computed structural load effects from wind tunnel data for a simply supported and a continuous purlin from an accompanying paper (Part I) with those obtained using the strict interpretation of AS 1170.2

PURLIN LOAD EFFECTS USING AS 1170.2

The purlin under consideration in this study was the first purlin from the eaves purlin towards the ridge on a building with an eaves height of 2.7 metres, a width of 6.7 metres and a length of 15 metres. The building had a roof pitch of 10°. The building was assumed to be located in Region A Terrain Category 2 with a purlin span assumed to be 3 metres for two support conditions: namely simply supported between supporting frames and continuous over supporting frames.

Spatially average pressures for the 0° and 90° wind directions using AS 1170.2 were calculated. Load effects calculated from wind tunnel data in the accompanying paper (Part I) were from 0°, 45° and 90°. It is assumed that in most cases, pressures calculated for the 0° and 90° wind directions using AS 1170.2 allow for pressures for intermediate wind directions. LPF's from AS1170.2 were applied (in conjunction with the spatially average pressures) to the purlin to achieve the most severe load effects as shown in Figures 1 and 2. It was clear that the LPF K_l = 2.0 was not going to be significant due to the fact that such a small strip (0.103 metres) of the K_l = 2.0 area being inside the tributary area of the purlin. A LPF K_l = 1.5 was therefore used. Positions adopted for the calculation of load effects were as noted in Figure 3. The load effects were calculated using conventional structural analysis techniques rather than the influence coefficients used in the accompanying paper (Part I) which suited the panel discretisation adopted in that paper.

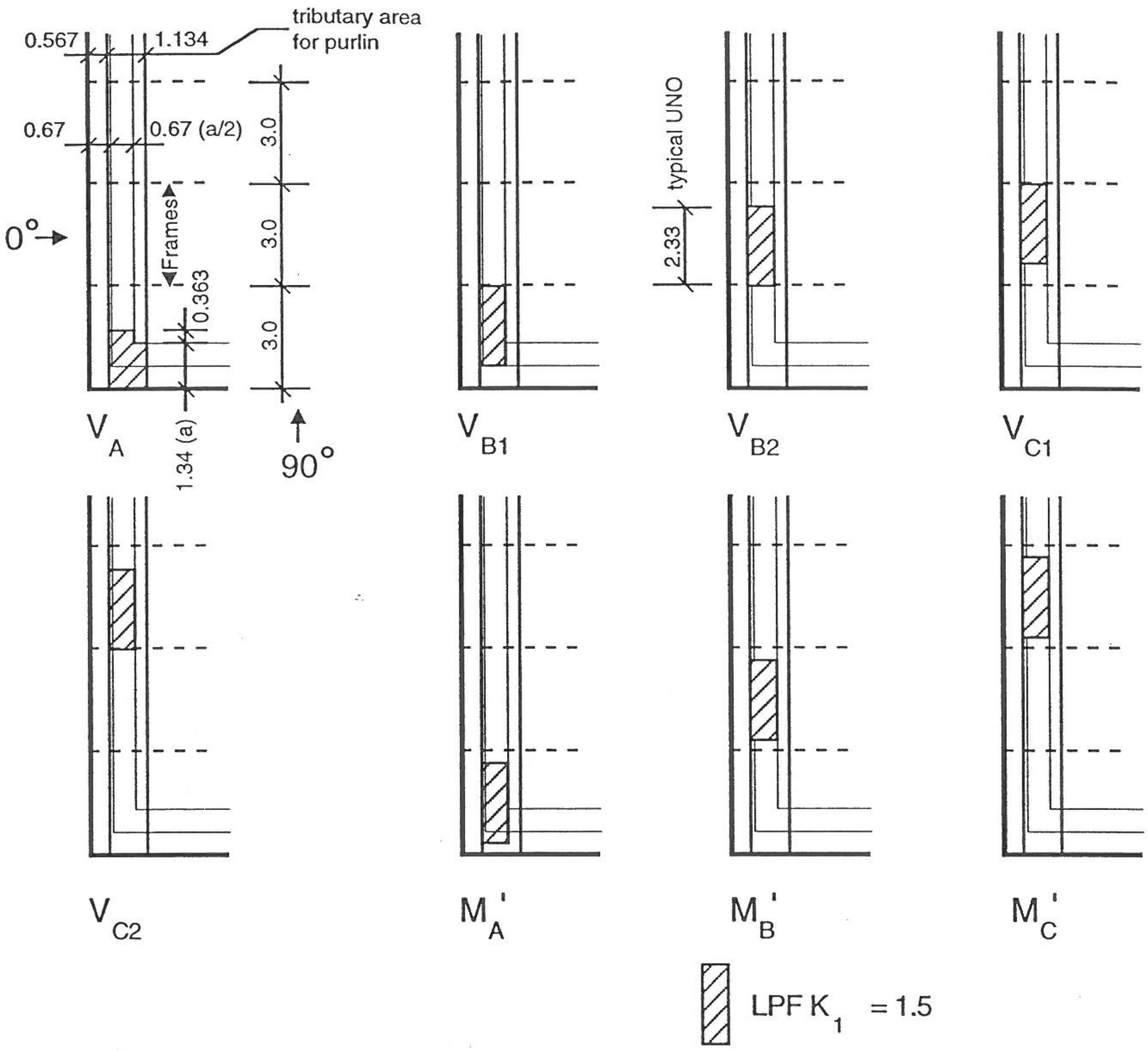

Figure 1. Location of local pressure areas to give most severe load effects
Simply supported case

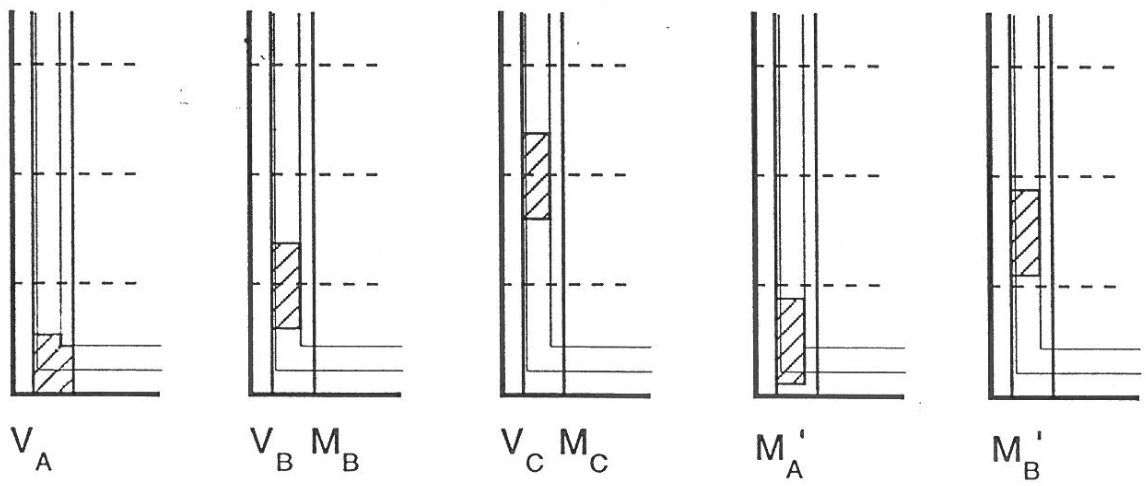

Figure 2. Location of local pressure areas to give most severe load effects
Continuous case

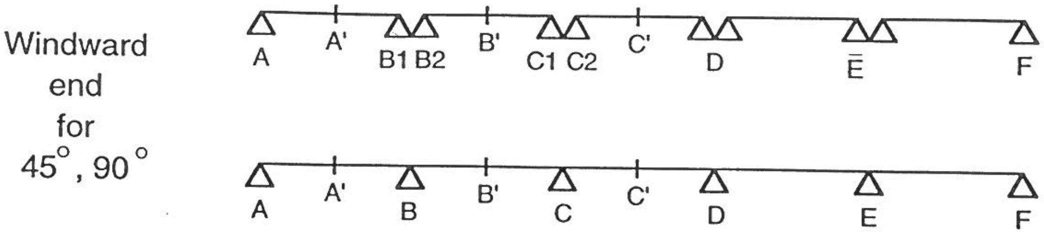

Figure 3. Positions adopted for the calculation of load effects
(a) Simply-supported (upper) (b) Continuous (lower)

COMPARISON OF AS1170.2 AND WIND TUNNEL RESULTS - SIMPLY-SUPPORTED PURLINS

Structural load effect coefficients (of greatest magnitude) using AS 1170.2 and those calculated from wind tunnel data are given in Table I for the simply supported purlins. The wind tunnel based values are based on the mean dynamic pressure at the eaves height of 2.7 metres for all wind directions. The AS 1170.2 values are based on the mean dynamic pressure at the eaves height of 2.7 metres for the 0° wind direction and the ridge height for the 90° wind direction, as required by the Standard. The reaction coefficients are normalised using the span (3m in full scale) times the purlin spacing. The bending moment coefficients are normalised using the span squared times the purlin spacing. The critical reaction and bending moment for each load effect, regardless of wind direction, have been shown in **bold** in Table I. These are the values of interest for design purposes. Critical reactions calculated using AS1170.2 are greater than the wind tunnel based values by between 14% and 59%. The lower magnitudes of pressure near the end of the building due to 'three dimensional flow' can be seen in the 0° wind tunnel results. AS 1170.2 does not reflect this effect. The values of bending moment calculated using AS 1170.2 are between 5% and 12% greater than the more realistic values based on wind tunnel data.

Table I. Coefficients of critical load effects for simply-supported purlins

	V_A	V_{B1}	V_{B2}	V_{C1}	V_{C2}	$M_{A'}$	$M_{B'}$	$M_{C'}$
0°								
AS 1170.2	-1.31	-1.29	**-1.29**	**-1.29**	**-1.29**	-0.270	**-0.270**	**-0.270**
wind tunnel	-0.681	-0.802	**-0.978**	**-1.013**	**-1.029**	-0.185	**-0.249**	**-0.256**
90° & 45°								
AS 1170.2	**-1.43**	**-1.37**	-0.84	-0.76	-0.56	**-0.295**	-0.166	-0.118
wind tunnel	**-1.253**	**-0.860**	-0.640*	-0.679*	-0.670*	**-0.263**	-0.167*	-0.166*

* denotes value from 45° wind tunnel case

COMPARISON OF AS1170.2 AND WIND TUNNEL RESULTS - CONTINUOUS PURLIN

Structural load effect coefficients (of greatest magnitude) for the continuous purlin using AS1170.2 and those calculated from wind tunnel data are given in Table II. The wind tunnel values are based on the mean dynamic pressure as described above for the simply-supported purlins. Again, the load effect coefficients are normalised as described above for the simply-supported purlins. Unlike the accompanying paper (Part I), estimates of pressure were made for the two purlin spans that were not pressure tapped. Load effects were subsequently recalculated. Although the change in load effects caused by this measure is not significant (for the reasons given in Part I), the accuracy of the results was improved. The critical reaction and bending moment for each load effect, regardless of wind direction, have been shown in **bold** in Table II. Critical reactions calculated using AS1170.2 are greater than the wind tunnel based values by between 9% and 34%. Bending moment increases of between 3% and 31% occurred when using AS 1170.2 in place of the more realistic wind tunnel based results. Being near the centre of the continuous span purlin the wind tunnel value for bending moment at M_C (that was 3% less than the AS 1170.2 value), was effected by a number of influence coefficients of relatively high magnitude with sign reversal in adjacent spans. This illustrates the point that for certain structural systems with influence lines that change sign, reductions in load effects may not be so great when using the techniques outlined in the accompanying paper (Part I). It is possible that a minimum (or maximum) pressure distribution envelope may not always give the most severe load effect on a real structure. The other factor influencing this result is the phenomenon, mentioned earlier, of the roof pressures near the ends of the building being less. An increased load effect would result from the lower pressure in span AB.

Table II. Coefficients of critical load effects for continuous purlin

	V_A	V_B	V_C	$M_{A'}$	M_B	$M_{B'}$	M_C
0°							
AS 1170.2	-1.082	**-2.602**	**-2.298**	-0.193	**0.230**	**-0.108**	**0.179**
wind tunnel	-0.500	**-1.944**	**-2.098**	-0.103	**0.175**	**-0.084**	**0.174**
90° & 45°							
AS 1170.2	**-1.214**	-2.432	-1.130	**-0.227**	0.216	-0.050	0.075
wind tunnel	**-1.113**	-1.429	-1.421*	**-0.189**	0.152	-0.066*	0.123*

* denotes value from 45° wind tunnel case

CONCLUSIONS

In this study it was clear that the LPF $K_l = 2.0$ was not going to be significant. A LPF $K_l = 1.5$ was therefore used. On another building where the LPF area may be greater than that shown in Figure 1, it may not be as easy to dismiss the LPF $K_l = 2.0$. It would therefore be necessary to calculate the load effects for both $K_l = 1.5$ and $K_l = 2.0$ to determine the more critical case. Inaccuracies relating the LPF's given in the Standard arise from the variety of eave details that are used in practice and other general aerodynamic effects. For example, flush box gutters, parapet walls, eaves gutters, h/d, roof pitch and effects near building ends. Given these uncertainties and the generally conservative results of load effects derived from the Standard (as opposed to computed results using wind tunnel data), it may be appropriate to reduce the LPF's $K_l = 1.5$ and 2.0 to a single LPF of 1.8 or 1.7 over a single area. Thus simplifying calculations relating to local pressure effects. For individual building projects, where significant cost implications may arise as a result of local pressure effects, or for buildings to be constructed in large quantities to the same design, it is possible that wind tunnel tests would be warranted to accurately assess local pressure effects. The importance of the structural system in relation to the 'real' load distribution giving the peak load effect has been shown.

REFERENCES

1. Australian Standard AS 1170.2-1989, SAA loading code: Part 2. Wind Loads (Standards Australia, Sydney, 1989).

2. J.D. Holmes, W.H. Melbourne and G.R. Walker. A commentary on the Australian Standard for wind loads (Australian Wind Engineering Society, 1990).