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WIND LOADS ON THE FRAMES OF LARGE BULK-STORAGE SHEDS

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INTRODUCTION

Long storage sheds, with high pitch roofs in the range of 15 to 40 degrees used to store large quantities of solid product, such as cement, bulk sugar and mineral ore, are not uncommon in Australia. These sheds may be up to 300 metres long, with spans exceeding 50 metres. Often they are located at coastal locations, near port facilities, and if they are in the tropics, may be subjected to tropical cyclones. Wind loads are a dominant design consideration for these buildings, whose geometry extends beyond the usual low-rise buildings for which wind loading standards such as AS1170.2 and NZS4203 were intended. Recently two buildings of this type were studied for the wind loads on the main structural frames, at Monash University. This paper describes some results of these studies, and highlights the inadequacies of AS1170.2 in dealing with buildings of this type.

THE BUILDINGS

The main geometrical dimensions of the two buildings are shown in Figure 1. Building 1 has a roof of 16 degrees pitch, with length : height : span in the ratios 5.2 : 0.34 : 1.0. Building 2 has a 36 degree roof pitch with dimensions in the ratio 5.0 : 0.55 : 1.0. The building structures consisted of large steel frames with pinned column bases; the frames of Building 1 were slightly haunched, and the frames of Building 2 were designed with a pin connection at the ridge.

WIND-TUNNEL MEASUREMENTS

Models of each building at a scale of 1:250 were made, and pressure tapped over the tributary areas of a number of frames along the length of each building. For the present paper, the loads on only two frames on each building are discussed however - a frame near one end and a frame at the centre. The pressure taps were grouped into fourteen panels for each frame - three on each of the front and back walls and eight on the roof for Building 1, and two on each wall and ten on the roof for Building 2. The pressure taps were connected into an averaging manifold for each panel, and each manifold was connected to a multi-channel electronic Scanivalve system, via a restricted tube system of high frequency response.

The building models were tested in a simulated open country turbulent boundary-layer flow with a full-scale roughness length of about 40 mm, and turbulence intensity at 20 metres height of 0.20. For the test of Building 1, some adjacent buildings were also modelled and included in the testing; however these were at the opposite end of the building to the end frame discussed in this paper, and would have had little effect for the wind directions generating the largest wind loads.

The fluctuating pressures sampled at 500-1000 Hertz for 20-40 seconds, by the sensors of the multi-channel pressure measurement system were digitally processed by custom software. The processing included the calculation of correlation coefficients for every pair of panels within a frame tributary area.

PROCESSING OF DATA

The wind-tunnel pressure data, together with structural influence coefficients for the building frames, were further processed by EXCEL spreadsheets. The methods used were those developed at Ruhr-University Germany, and CSIRO, for determining the effective static loads for the various frame load effects of interest, and described in detail elsewhere [1,2,3,4]. To simplify the processing, the eigenvalues and eigenvectors of the covariance matrices of the fluctuating pressures were first determined using MATLAB.

RESULTS

The results of the processing are available in two forms : as values of the predicted peak (maximum and minimum) load effects (column reactions, bending moments) themselves, or in the form of effective static pressure distributions, corresponding to the peak values of each load effects. An example of the first type of output is shown in Figure 2, which shows the variation with wind direction of the maximum and minimum bending moment at one 'knee' of the end frame for Building 1. The bending moment is plotted in Figure 2 in the form of a non-dimensional bending moment coefficient defined as follows :

$$C_M = \frac{M}{\frac{1}{2}\rho \bar{u}^2 b^2 w} \tag{1}$$

where b is the span of the building, w is the frame spacing (i.e. the width of the tributary area of the frame), and $\bar{u}$ is the mean wind speed at the top of the building.

The definitions of the coefficients of vertical reaction and horizontal reaction are as follows ;

$$C_V = \frac{V}{\frac{1}{2}\rho \bar{u}^2 b\, w} \tag{2}$$

$$C_H = \frac{H}{\frac{1}{2}\rho \bar{u}^2 h\, w} \tag{3}$$

where h is the overall height of the building to the ridge.

An example of a computed effective static wind load distribution in the form of pressure coefficients is shown in Figure 3 (a). Also shown in Figure 3(b), are the directly measured mean and maximum and minimum pressure coefficients for the same frame, and same wind direction as in Figure 3(a).

COMPARISON WITH CODE VALUES

In Tables I to IV are compared the maximum and minimum values of various moment and force coefficients for the end and centre frames of the two buildings. In column 2 of each

Table are shown the extreme values of the coefficients obtained from the wind tunnel and subsequent calculations, irrespective of the wind direction producing them. Column 3 shows the corresponding values produced by application of AS1170.2, although in this case, only wind directions orthogonal to the walls are considered. For both sets of values the internal pressure coefficient has been assumed to be constant and equal to zero, that is a fully sealed building has been implied. Thus, this is a comparison of the effects of the external pressures only.

The Tables show that the code seriously underestimates all the load effects for both buildings. The underestimation is least for the vertical reactions - probably because the code values for roof pressures have been set up with uplift in mind. The biggest underestimation is for the horizontal reactions. More significantly for structural design, there are large underestimations for the worst frame bending moments, especially for the end frame of Building 2 (Table III). The wind-tunnel based estimations show that the biggest bending moment in this frame occurs for an oblique wind direction, which is not considered by the Standards.

Although these are unusual buildings, many of them are being designed without the benefit of wind-tunnel tests, and the Standards need to be modified to cater for the long lengths and high roof pitches characteristic of these large storage sheds.

REFERENCES

1. M.Kasperski and H-J. Niemann. The L.R.C. Method - A general method of estimating unfavourable wind load distributions for linear and non-linear structural behaviour. *Journal of Wind Engineering & Industrial Aerodynamics*, Vol. 43, pp 1753-1763, 1992.

2. J.D. Holmes. Optimised peak load distributions. *Journal of Wind Engineering & Industrial Aerodynamics*, Vol. 41, pp 267-276, 1992.

3. J.D. Holmes and M.J. Syme. Wind loads on steel-framed low-rise buildings. *Steel Construction (Journal of the Australian Institute of Steel Construction),* Vol. 28, pp2-12, 1994.

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TABLE I. Force and moment coefficients - Building 1 (16 degree pitch) - End frame

Load Effect	**Peak moment/force coefft. wind tunnel**	**Moment/force coefft. AS1170.2**	**Magnitude error (%) in AS1170.2**
Knee bending moment	-0.033 / +0.128	-0.038 / 0.097	-24 %
Vertical reaction	-1.09 / +0.32	-0.96 / +0.32	-12 %
Horizontal reaction	-2.49 / 0.53	-0.61 / +0.26	-75 %

TABLE II. Force and moment coefficients - Building 1 (16 degree pitch) - Central frame

Load Effect	Peak moment/force coefft. wind tunnel	Moment/force coefft. AS1170.2	Magnitude error (%) in AS1170.2
Knee bending moment	-0.035 / +0.099	-0.038 / 0.079	-20 %
Vertical reaction	-0.76 / +0.33	-0.50 / +0.32	-5 %
Horizontal reaction	-1.86 / 0.47	-0.61 / +0.26	-67 %

TABLE III. Force and moment coefficients - Building 2 (36 degree pitch) - End frame

Load Effect	Peak moment/force coefft. wind tunnel	Moment/force coefft. AS1170.2	Magnitude error (%) in AS1170.2
Centre-rafter bending moment	-0.176 / +0.140	-0.072 / +0.066	-59 %
Vertical reaction	-1.11 / +0.60	-0.93 / +0.31	-16 %
Horizontal reaction	-0.60 / 1.30	-0.12 / +0.46	-64 %

TABLE IV. Force and moment coefficients - Building 2 (36 degree pitch) - Centre frame

Load Effect	Peak moment/force coefft. wind tunnel	Moment/force coefft. AS1170.2	Magnitude error (%) in AS1170.2
Centre-rafter bending moment	-0.092 / +0.094	-0.072 / +0.066	-23 %
Vertical reaction	-0.49 / +0.30	-0.21 / +0.31	-36 %
Horizontal reaction	-0.53 / 0.64	-0.12 / +0.13	-80 %

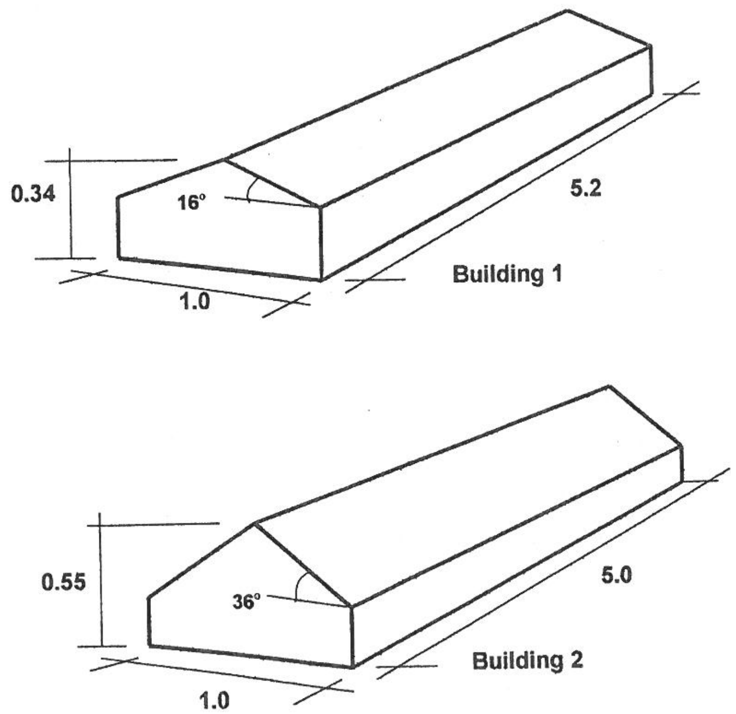


Figure 1. Geometry of the buildings

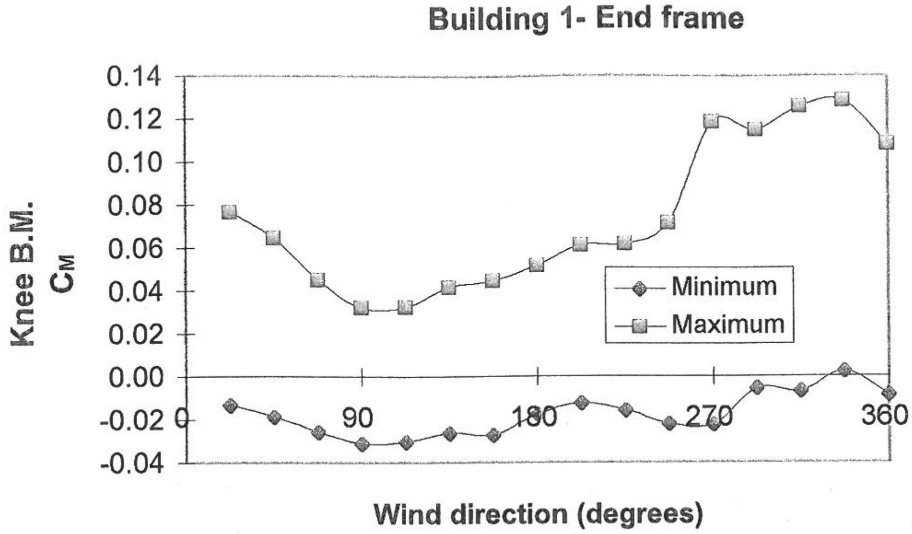
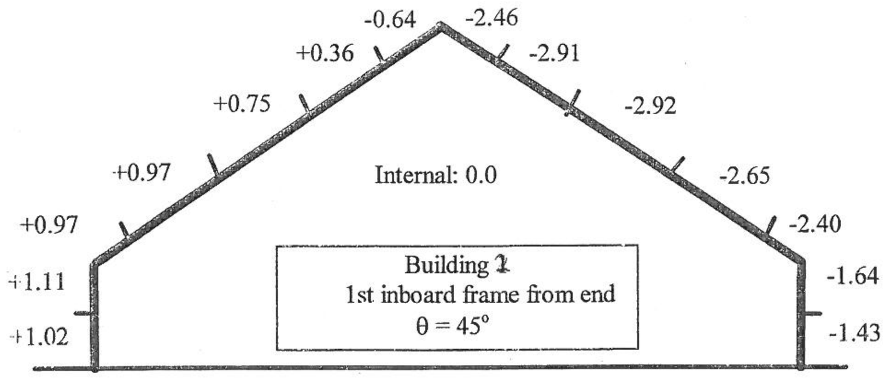
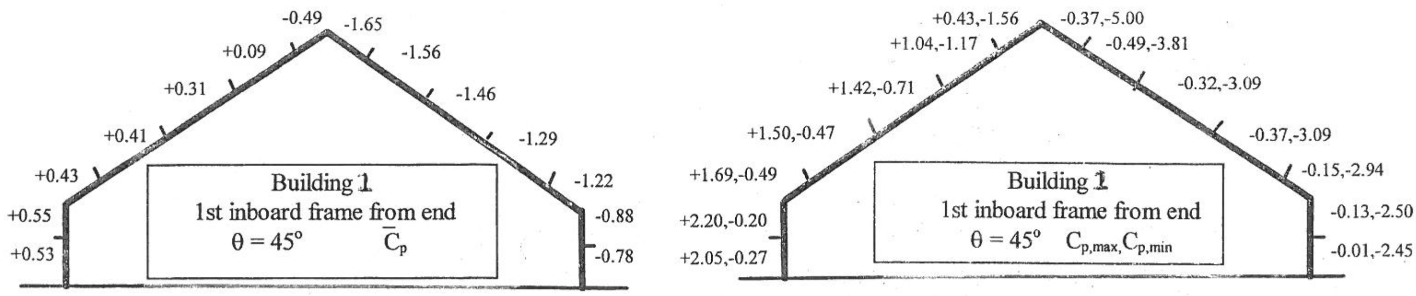


Figure 2. Variation of extreme knee bending moment coefficients for end frame of Building 1 with wind direction, as determined from wind-tunnel tests



(a) Effective static pressure coefficients for maximum centre-rafter bending moment (Building 2)



(b) Mean and peak panel pressure coefficients - Building 2 (45° wind direction)

Figure 3.