WIND DESIGN OF A CRUCIFORM SHAPED MAST

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Introduction

The Telecom Building is a 47 storey building, approximately 196m above ground level and is situated on the North-East corner of Exhibition and Lonsdale Streets, Melbourne. The cruciform shaped mast over the Telecom Corporate Building rises to a height of approximately 25m above the roof level. The structural design of the building and the mast was carried out by Connell Wagner (Vic) Ltd.

The architectural and construction requirements led to the selection of a relatively light weight cruciform shaped structure. It consists of four 305 x 305 x 12 square hollow sections welded together (Fig. 2). The mast was connected to the main tower at 2 levels as shown in the elevation (Fig. 1). The dynamic analysis gave a fundamental natural frequency of 0.9 Hz for the mast.

As the members are subjected to fluctuating loads under wind, the design was mainly governed by wind-induced fatigue. A detailed fatigue analysis was carried out according to the requirements of AS4100 (Australian Steel Structures Code). The response for complete range of wind speeds was required for this analysis.

In the preliminary design, the along-wind response was evaluated by the formulae given in AS1170 Part II (1989). But the drag coefficient had to be estimated as this shape is not covered in the code. The method suggested in Ref. (1) was used to determine the cross-wind response. This method had been developed for square and rectangular shapes. The only reliable means to predict the dynamic response of this wind sensitive structure was to test in a wind tunnel. The wind tunnel testing was carried out to check the possibility of "galloping instability" and to obtain better estimates of cross-wind and along-wind loading.

To increase the damping of the structure a hanging chain damper was provided at the top. Also 4.5 m length of the mid section of the mast was filled with cementitious grout to increase the damping and to improve the stability of the top connection to the building.

This paper sets out the results of the wind tunnel testing and the comparison between these results and the values derived in the preliminary analysis. A comparison is also made between the wind tunnel results and the code coefficients for cross-wind moments.

Wind Tunnel Testing

The wind tunnel tests were carried out in the 450 kW boundary layer wind tunnel at the Department of Mechanical Engineering, Monash University. An aeroelastic model of the mast was built to a length scale of 1/57.5 and a density ratio of 1.0. The wind moment envelope derived from wind tunnel results for different wind directions is shown in Fig. 3. As seen from the graphs the wind design was governed by the along-wind response rather than the cross-wind response. Also it was shown that there was no evidence of galloping instability at 1% damping.

Comparison of wind tunnel results with results from preliminary analysis

1. Along-wind Moments

In the preliminary design gust factor approach was used. The gust factor was calculated assuming TC 3 conditions and using the Section 4 of AS1170 Part II. The gust factor was found to be approximately 2.03. As this shape is not covered in the code an estimated drag coefficient was used in the preliminary analysis. The average value for sharp-edged and smooth-edged square shapes given in Table B2 and Table B3 of the code was selected (equal to 1.75) as the drag coefficient. As seen from the wind envelope the maximum along-wind moment is approximately 1010 kNm. This moment gave an equivalent drag coefficient of 1.3.

2. Cross-wind moments

As mentioned before the cross-wind response for complete range of wind speeds was required for fatigue analysis. The response was estimated by using a random excitation model and a sinusoidal lock-in excitation model. This method is summarised below. A complete description of this method is given in Ref (1).

For low wind speeds ($\bar{V}(h)<15m/s$) sinosoidal lock-in excitation model was used. The cross-wind displacement response at the top of the structure is given by Eqn. (1). The C_{SL} values were interpolated from graphs given in Ref (1). These graphs have been derived from wind tunnel tests on square and rectangular shaped structures (Kwok and Melbourne, 1981). For higher wind speeds ($\bar{V}(h) >15m/s$) random excitation model was used to predict the standard deviation cross-wind displacement response. The S_{FS} values were obtained by Graphs given in Ref (1) for square shapes. They are based on experimental work done by Saunders and Melbourne (1975). These displacements were converted to peak base moments at the top connection level by assuming a cantilever model.

Sinosoidal lock-in Model

$$\sigma_y = \frac{C_{SL}\,\rho b^3 h}{2\,(1+\gamma)\,\eta m \left[4\pi \frac{n_c b}{\bar{V}(h)}\right]^2} \qquad (1)$$

σ_y^2	=	variance of the cross-wind response
n_c	-	fundamental Natural frequency
m	-	modal mass
η	-	The fraction of critical damping
$\bar{V}(h)$	-	The hourly mean speed at the top of the mast.
C_{SL}	-	Sinosoidal lock-in force coefficient
γ	-	Power law exponent of the mean longitudinal Velocity profile (assumed to be 0.25)
b	-	The breadth of the structure normal to wind
h	-	The height of the structure

Peak displacement = $g_f \cdot \sigma_y$

g_f = peak factor = $\sqrt{2 \log_e (3600 n_c)}$

Random Excitation Model

$$\sigma_y = \sqrt{\frac{\pi n_c \cdot S_{FS}}{(2\pi n_c)^4\, m^2\, 4\eta}} \qquad (2)$$

S_{FS} - Force spectrum coefficient

A damping coefficient of 2% was used in the final design. The bending moments at the top connection level for 2% damping are plotted in Fig. 4. The wind tunnel results are also plotted in Fig. 4. As expected wind tunnel results were lower than the estimated values.

Code Coefficients

The Section 4.4.3 of AS1170 Part II (1989) gives an equation to calculate the peak over-turning cross-wind moments for tall buildings and towers. This equation (Eqn. (3)) is derived from the random excitation model described before. The C_{fs} values calculated from wind tunnel test results for this structural shape are plotted in Fig. 5. It can be seen from Fig. 5 that for this section the peak coefficients are obtained at a lower $\frac{\bar{V}(h)}{n_c b}$ value.

$$\hat{M}_C = 0.5 g_f \cdot \bar{q}_h\, b h^2\, (1.06 - 0.06k) \sqrt{\frac{\pi C_{fs}}{\eta}} \qquad (3)$$

$$C_{fs} = \frac{n_c \cdot S_{FS}}{(\frac{1}{2}\rho \bar{V}(h)^2\, bh)^2}$$

$\hat{M}_C$ - Design peak base over-turning moment in cross-wind direction

$\bar{q}_h$ - hourly mean dynamic wind pressure at height h

Conclusions

1. The preliminary cross-wind analysis was based on the results obtained for square shapes. These methods over-estimated the actual cross-wind response of the cruciform shaped structure. However, the results show the correct trend and therefore this method is satisfactory for preliminary analysis.

2. The Along-wind moments obtained in the wind tunnel tests are significantly lower than the calculated moments. This is due to the over-estimation of the drag coefficient.

3. More wind-tunnel tests on cruciform shaped sections are required to derive generalised coefficients for the wind code.

References

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3. Saunders, J.W., and Melbourne, W.H., "Tall Rectangular Building Response to Cross-wind Excitation", Proceedings of the 4th International Conference on Wind Effects on Buildings & Structures, University Press, Sept 1975, pp. 369-379.

4. Standards Association of Australia, "SAA Loading Code Part II: Wind loads", AS 1170.2, 1989.

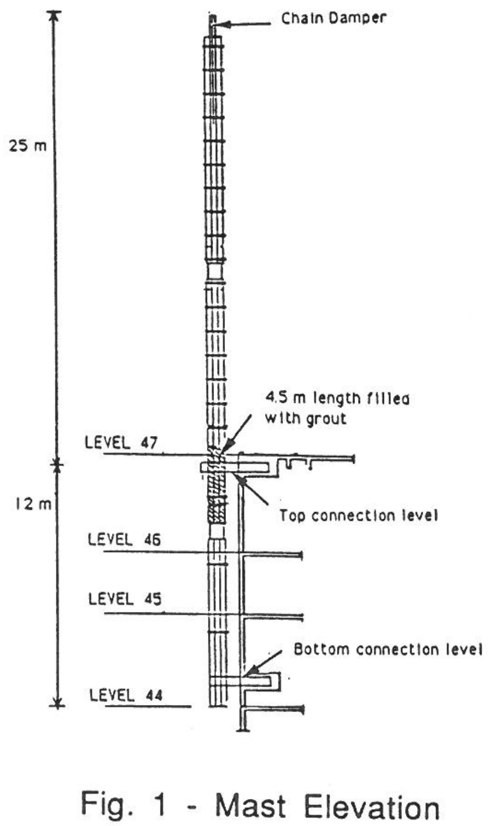

Fig. 1 - Mast Elevation

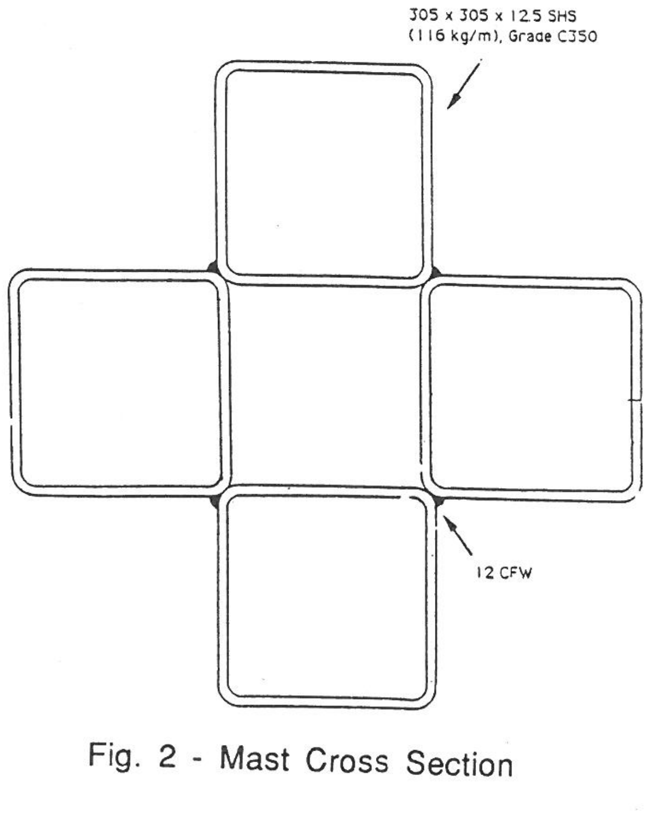

Fig. 2 - Mast Cross Section

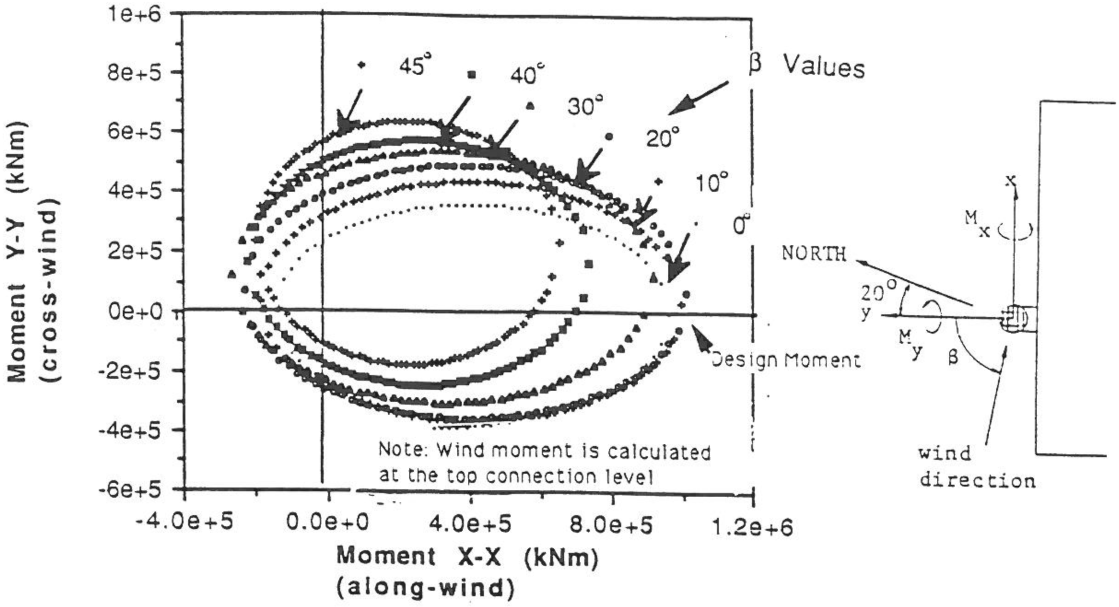

Fig. 3 - Moment Envelope

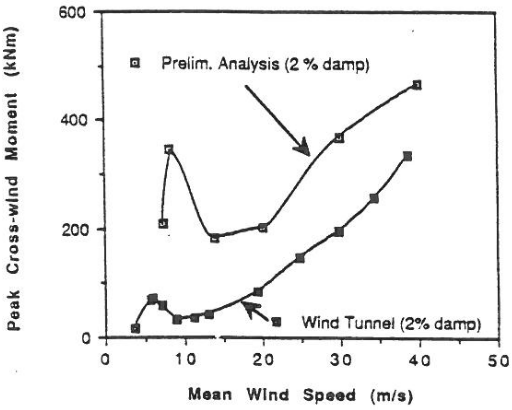

Fig. 4 -

Comparison of Cross-wind Moments

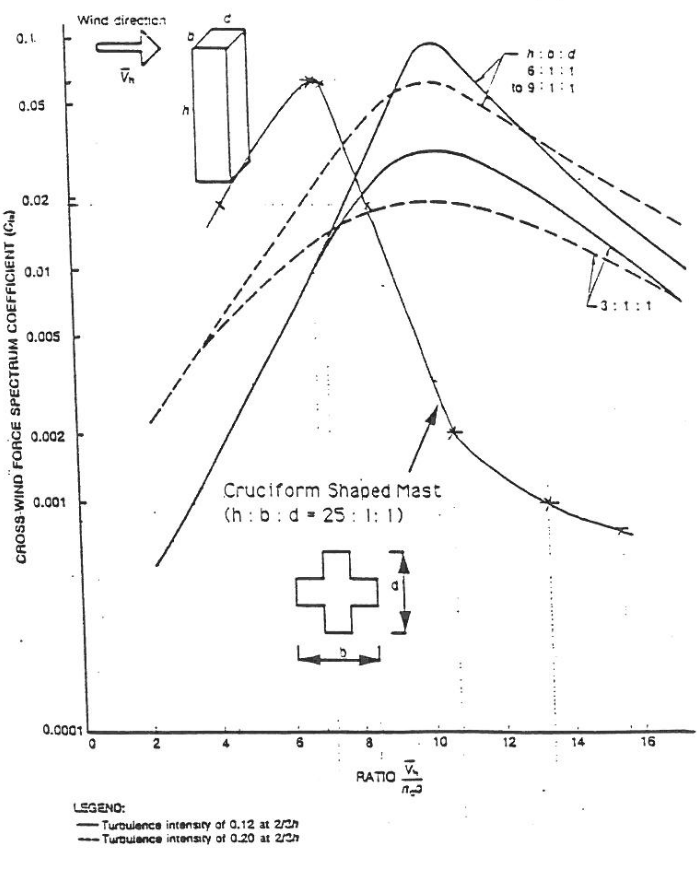

Fig. 5 - Comparison of Cross-wind Force Spectrum Coefficients