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Wind Loads on extension of Light Towers at AAMI Stadium, Adelaide

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Abstract

This paper describes the calculation of effective static wind loads, incorporating dynamic along-wind response, for the upgraded lighting towers at AAMI Stadium, West Lakes, South Australia. Winds from both synoptic gales and downdrafts in the Adelaide area were considered. Drag coefficients were measured using a 1:50 scale model of the head-frame.

Introduction

The light towers at AAMI Stadium, shown in Figure 1, were to be extended and fitted with new head frames to make provision for lighting required for high definition television broadcasts.

Figure 1 Artist's impression of the proposed new head frames on the light towers at AAMI stadium.

Three options of the upgraded towers were under consideration (the new head frame is identical for all options, as shown in Figure 2):

a) *Augmented* Tower. This concept requires lifting of the existing tower, inserting a larger diameter and thicker plate section at the base to increase the height to 72.4 m, and installing a new head frame.

b) *Stiffened* Tower. The existing tower is stiffened by means of four stiffeners (300mm x 25mm PL) welded to the outside, is extended at the top, and a new head frame is installed.

c) *Extended* Tower. The existing tower is extended at the top, and a new head frame is installed.

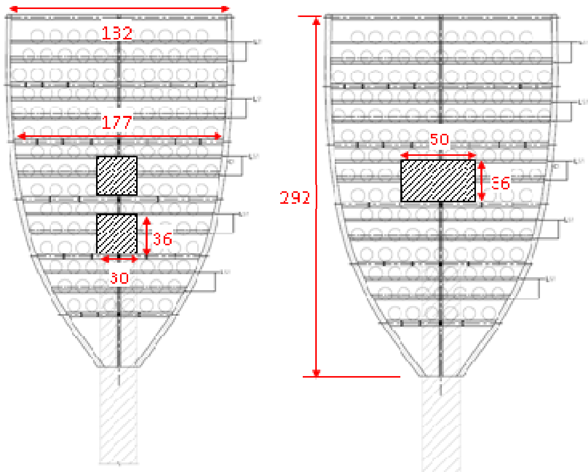

Figure 2 Proposed new head frames.

If additional strength is deemed necessary, the stiffened option will likely be required to the North-West tower that has been built within the existing northern grandstand. For the other 3 towers either the stiffened or augmented option will be considered primarily on the basis of cost and programme.

There are no surrounding structures of comparable height in the vicinity of the towers, which are situated in terrain categorised as Terrain Category 2.5. The existing grandstands have been included to define this terrain category (they could similarly be included as a shielding factor). The lighting towers are clearly 'major structures' potentially close to large crowds, and according to the Building Code of Australia (2004) and AS/NZS 1170.0 (2002) should be considered as Importance Level 3. For a lifetime of 50 years, the appropriate annual probability of exceedance of wind gusts is 1/1000 (i.e. 1000 year return period).

Head Frame Dynamic Characteristics

The first- and second sway mode natural frequency of the various tower options were estimated at :

- Augmented : 0.27 Hertz,
- Stiffened : 0.23 Hertz,
- Extended : 0.20 Hertz

The structural damping was taken as 0.01 (1% of critical). Aerodynamic damping has been included based on Holmes (1996a).

The mode shapes for the first two modes of vibration of the all tower options were also calculated, with the mode shapes for Modes 1 and 2 nearly identical to each other, and were fitted with a power law function of the following form:

$$y(z) \cong \left(\frac{z}{h}\right)^{1.94} \tag{1}$$

Design Wind Speeds

Daily maximum gusts at 10 metres height above 45 knots (23 m/s) from Adelaide Airport, Parafield Airport and Edinburgh RAAF Base have been recorded between 1956 and 1997 and were separated into those produced by local thunderstorm downdrafts, and those generated by larger synoptic systems, by inspection of the anemometer charts. The two different storm types were separately analyzed using the 'peaks over threshold' method in Holmes (2007).

It was found that although both types of events contribute to the combined probability of exceedance for return periods up to about 500 years, thunderstorm downdraft winds tend to be dominant for return periods greater than about 10 years. The predicted basic wind speed for synoptic winds for a 1000-year return period is 42.3 m/s; the predicted wind speed for the same return period due to downdraft winds alone is 46.4 m/s. For winds of any type, the gust wind speed for a 1000 year return period is 46.7 m/s (about the same value as given in AS/NZS 1170.2 (2002) for Region A, in which this development is located).

It is noted that the highest wind gust recorded in the Adelaide area, in the time period considered, was 82 knots (42.2 m/s) in a thunderstorm (downdraft) at Edinburgh RAAF base in 1983.

Drag Coefficient of the Head Frame

To determine the drag coefficient for the head-frame, a wind tunnel test was carried out. For the preliminary design, the head-frame was assumed to behave like a porous rectangular hoarding, hence AS/NZS 1170.2 (2002) methodology was used to provide a comparative assessment of the measured drag coefficient. The experimental results were also compared to guidelines set in Letchford (2001), which yields less conservative results when compared than the wind code.

A 1:50 scale model of the proposed AAMI Stadium Light tower was constructed and placed in the University of Sydney boundary layer wind tunnel for base load analysis. The model was placed on a JR3 Force-Moment Sensor and was orientated in the tunnel such that the positive X axis on the sensor was the same as the wind direction. The turntable was then turned clockwise and measurements of the base forces and moments were made every 10° using a data acquisition system that sampled at 400Hz (with the signals filtered at 200Hz).

The resultant of the X and Y components of the force recorded in the wind tunnel was used to determine the drag coefficient on the structure for the headframe and supporting pole attached. To determine the drag coefficient on the head frame alone, force components from a consistent wind incidence angle between the pole and the head frame were vector summed to determine the force components. The mean resultant force on the head frame was then computed once more and hence the resultant drag coefficient, by applying Equation (2). Further to this, it was found that the pole contributed to approximately 6% of the total drag on the structure. The maximum average drag coefficient measured on the head frame was found to be approximately 0.75.

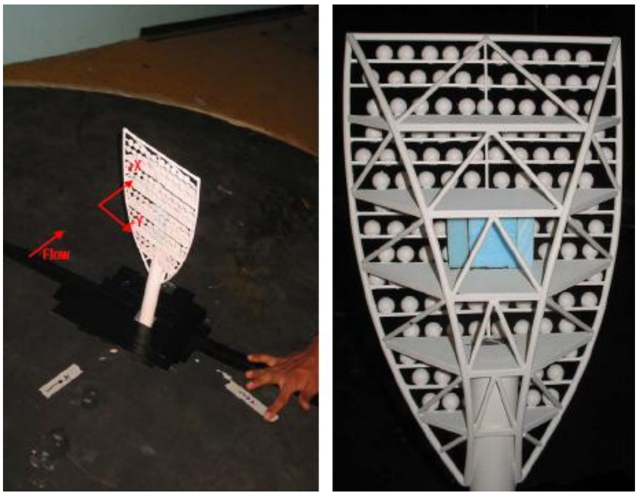

Figure 3 Model of the head frame in the wind tunnel.

The drag coefficients were verified using the Australian wind code (AS/NZS 1170.2, 2002) under the assumption of 50% solidity ratio of the structure. These drag coefficients were then compared to the aerodynamic shape factor, C_{fig} determined from the procedure outlined in AS/NZS 1170.2 (2002) for freestanding hoardings and walls. The maximum resultant drag coefficient is expected to be observed when the wind is normal to the structure at 0 or 180° wind incidence. The methodology to determine the aerodynamic shape factor for this wind incidence angle is outlined.

$$C_{fig} = C_{p,n} K_p \tag{2}$$

Where; C_{fig} = aerodynamic shape factor, $C_{p,n}$ = net pressure coefficient acting normal to the surface, and K_p = porous cladding reduction factor:

$$K_p = 1 - (1-\delta)^2 \tag{3}$$

Where; δ = solidity ratio of structure.

Given the geometric parameters of the hoarding, c/h = 0.2 and b/c = 0.65, the net pressure coefficient normal to the surface is given by the following.

$$C_{p,n} = 1.3 + 0.5(0.3 + \log_{10}(b/c))(0.8 - c/h) \tag{4}$$

Where; b = breadth, c = net height of hoarding, h = maximum height of hoarding above ground.

Therefore C_{fig} was found to be approximately 0.98. Comparing this to the maximum average resultant drag coefficient on the head frame of 0.75, a significant load reduction of more than 23% is observed. However it is worth noting that the Australian wind code tends to over predict the porous cladding reduction factor for porosities (Letchford, 2001). Letchford's (2001) results suggested a faster decay in the porous cladding reduction factor and suggested the following relationship:

$$K_p = 1 - (1-\delta)^{1.5}. \tag{2}$$

Using this result a predicted C_{fig} of 0.84 is obtained, which is only a 10% load reduction compared to the wind tunnel.

Along-Wind Response

For the calculation of along-wind loading and response, the effective static loading approach described by Holmes and Kasperski (1996) was used. In this method, the total effective peak load distribution is formed by combining contributions from:

i) the mean load distribution
ii) the background load distribution (quasi-static fluctuating loading below the resonant frequency and,
iii) the resonant dynamic load contribution.

Contributions (i) and (iii) are independent of the analysis height. For low damping, the resonant component (iii) can be represented accurately by an inertial load distribution, proportional to mode shape and to the frequency squared.

The background contribution, (ii), depends on the analysis height but in the present case is insensitive to the load effect (i.e. shearing force, bending moment, member axial force) or its height, due to the dominance of the wind loading on the head frame compared with that on the tower itself. It is also independent of the dynamic free-vibration properties of the structure (natural frequency, mass distribution, damping).

In the present case, all three contributions to the loading are dominated by the headframe due to its large surface area and mass compared to the tower itself. For the background loading distribution, a "load-response" correlation is required (Holmes and Kasperski, 1996; Holmes, 1996). The approximate expression for the correlation coefficient between the load at any point on the tower (height z) and the base moment is :

$$R(z) = \exp(-|z-h| / {}^{z}L_u) \quad (3)$$

Where h is the reference height (top of the tower), and ${}^{z}L_u$ is a vertical length scale of longitudinal turbulence. That is, the above expression is closely equal to the correlation coefficient between the longitudinal velocity at height z and that at the height of the head frame.

Synoptic wind load calculations

Following the above procedure effective static loads were calculated for synoptic winds using a basic (regional) wind speed of 42.3 m/s (Terrain Category 2, at a height of 10 metres). For this case, the equivalent hourly mean wind speed at the top of the lighting tower is 31.0 m/s (for Terrain Category 2.5). The mean velocity profile for this case was calculated using parameters from AS/NZS1170.2 (2002) – i.e. the values of M_z, Cat 2.5 from Table 4.1(A) in AS/NZS1170.2 (2002), and turbulence intensity from Table 6.1 in AS/NZS1170.2 (2002).

Other parameters used in these calculations were :

- Drag coefficients for the tower shaft : 0.5 for augmented option, and 1.0 for stiffened option (due to the effect of the stiffeners)
- Turbulence intensity at top of tower : 0.16
- Vertical correlation length of turbulence : 50 m
- Exponent for mode shape : 1.94

Parameters gB, gR, S and E from AS/NZS1170.2 (2002) were also used in the calculations (refer to the reference for their description).

The peak factors used for the synoptic wind load calculations were calculated based on a time period of 1 hour (3600 seconds).

Convective downdraft wind load calculations

Calculations were also carried out for a convective downdraft loading case using the following parameters.

- Basic gust wind speed (10 metres height) of 46.7 m/s
- A uniform 'running' mean wind speed for all heights above 10 m
- Turbulence intensity at all heights : 0.100

The 'running mean' wind speed is similar to a mean wind speed for synoptic winds but is averaged over a period of only 30-120 seconds in the case of a downdraft. The uniform profile is based on recent full-scale measurements in Texas (Holmes et al, 2007) and Brazil, and is associated with the passage of a large vortex structure at the leading edge of the gust front resulting from the downdraft.

The peak factors used for the convective downdraft load calculations in the present case were calculated for a time period of 3 minutes (180 seconds).

Results

The values of base bending moments calculated are tabulated in Table 1 for all the cases considered.

Tower option	Synoptic wind	Downdraft
Augmented	8.38	7.68
Stiffened	9.53	8.90
Extended	8.18	7.46

Table 1 Calculated base bending moments (MN.m).

The base bending moments are 14 to 16% higher for the stiffened option compared with the augmented or extended options; this is because of the higher drag coefficient on the tower in the former case (due to the roughness provided by the stiffeners), and the lower natural frequency giving larger resonant response.

The bending moments for the synoptic loading are higher than those for the downdraft loading by 7-10%, despite the higher basic (10 m) wind speed of the latter. This is because of the steeper profile of the synoptic wind giving a higher (gust) wind speed at the top of the tower. The resonant dynamic components are also higher for the synoptic wind case.

The dynamic response factors, defined in this case as the base moment including resonance, to the base bending moment ignoring the effect of resonance, are tabulated in Table 2. These values are not much greater than 1.0 due to the very high aerodynamic damping of 6 to 11%, resulting primarily from the low mass and high projected area exposed to the wind provided by the head frame. The dynamic response factors for the downdraft case are lower due to the lower turbulence intensity and shorter storm duration.

Tower option	Synoptic wind	Downdraft
Augmented	1.173	1.110
Stiffened	1.181	1.112
Extended	1.167	1.102

Table 2 Calculated dynamic response factors.

It can be shown that the resonant component is relatively small – this is due to the high total damping, including aerodynamic damping, as discussed previously.

Cross-Wind Response

This section examines the likely amplitudes of the cross-wind response due to excitation by shed vortices, at the critical wind speeds. This phenomenon is described in some detail in Holmes (2007) with calculation methods based on both sinusoidal excitation (deterministic), and random vibration, given. In the case of the AAMI lighting towers the vortex shedding excitation can be assumed to originate from the top third of the circular pole, with none from the headframe. The headframe will however contribute to the mass significantly.

Sway Modes

An approximate indication of the maximum cross-wind dynamic response due to vortex shedding, in the first and second sway modes, can be obtained by applying *Section 6.3.3.1*. of AS/NZS 1170.2 (2002). This method is based on the assumption of sinusoidal excitation (Holmes, 2007).

For the augmented option, the average mass per unit height over the top third of the structure, m_t = 420 Kg/m (including a contribution of 6700 Kg from the headframe). Average diameter over top third, b_t = 1.53 m, conservatively assume a critical damping ratio ζ = 0.005, and the Scruton number (Sc) and maximum deflection (y_{max}) become:

$$Sc = \frac{4\pi m_t \zeta}{\rho_{air} b_t^2} = \frac{4\pi \times 420 \times 0.005}{1.2 \times 1.53^2} = 9.4 \qquad (4)$$

$$y_{\max} = \frac{kb_t}{Sc} = \frac{0.50 \times 1.53}{9.4} = 0.081m \qquad (5)$$

If Bolton's (1994) formula is applied, the value of peak deflection is:

$$y_{\max} = \frac{14b_t^3}{m_t} = \frac{14 \times 1.53^2}{420} = 0.119m \qquad (6)$$

This is greater than the value predicted by AS/NZS 1170.2 (2002), however it is still small, and Bolton's (1994) method has some inbuilt conservative parameters.

Calculating the base moment using Equation 6.3(15) of AS/NZS 1170.2 (2002), using the peak deflection of 81 mm and numerical integration, gives the maximum base bending moment due to vortex shedding = 0.23 MN.m. This is about 1/30 of the design along-wind base bending moment, and is not critical. The corresponding amplitude of the bending stresses are likely to fall below the endurance limit for steel.

Higher Order Modes

The response to vortex shedding in the third mode can be determined in the same way as for the fundamental modes with appropriate adjustment of the factor K for the mode shapes. It can be shown that the factor K in Equation 6.3(14) of AS/NZS 1170.2 (2002) is given by Holmes (2007):

$$K = \frac{k.\hat{C}_\ell}{4\pi.St^2} \qquad (7)$$

The parameter, k, is a function of the mode shape and the height of the structure over which the vortex shedding forces act, with a value of 0.79 obtained for the third mode.

Following Figure 1 in Bolton (1994), $\hat{C}_\ell$ can be taken as 0.12. At that Reynolds Number the Strouhal Number is close to 0.4 (Schewe, 1983). Hence K is given by :

$$K = \frac{k.\hat{C}_\ell}{4\pi.St^2} = \frac{0.79 \times 0.12}{4\pi \times 0.4^2} = 0.047 \qquad (8)$$

Then, for response in the third mode,

$$y_{\max} = \frac{kb_t}{Sc} = \frac{0.047 \times 1.53}{9.4} = 0.0071m \qquad (9)$$

i.e. A maximum response of 8 mm in the third mode is predicted. This will clearly produce negligible base moments. A similar result for the fourth mode is obtained.

Conclusions

Peak base bending moments for both synoptic winds and convective downdrafts have been calculated. The use of circular members in the head frame and the tower itself, combined with high aerodynamic damping, result in relatively low total effective wind loading in both synoptic and convective downdraft events. Estimates of peak response for the augmented option due to vortex shedding in the first/second and higher order (third) modes, using the sinusoidal approach adopted by AS/NZS 1170.2 (2002) (and similar to that by Bolton, 1994), have been made. In all cases the maximum deflections are negligible, resulting in negligible base bending moments in comparison with the design base bending moments under along-wind loading.

Acknowledgments

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