Wind Engineering of Sports Stadium Structures

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Abstract

World class stadium structures feature tall light-towers with significant head-frames, long-span cantilevered roof forms, retractable elements and tensile membranes. This paper describes the assessment of dynamic effects due to wind loads for stadia recently completed; Eden Park, Simonds Stadium, and Adelaide Oval Redevelopment. The contribution of dynamic loads to the along-wind response for the Simonds Stadium Light Towers is detailed, along with cross-wind fatigue. Dynamic effects of the Adelaide Oval Grandstand Roofs are assessed, with structural loads determined using the load-response correlation method, and the complexity of load distribution effects on membrane structures introduced. Wind and dynamic effects on ETFE cushions are considered with regard to load distribution.

Introduction

Wind induced dynamic loading can be significant for tall and long span structures such as sport stadium light towers, cantilevered and membrane grandstand roofs, and flexible membrane elements. Although Australian Standards may be used to approximate dynamic wind loads for simple structures, this approach leads to inaccurate and conservative loads when applied to unusual forms. This paper considers wind loads on the Simonds Stadium Light Towers in Geelong, Victoria, Adelaide Oval's Grandstand Roofs in South Australia, the ETFE skin of Eden Park's South grandstand in Auckland, New Zealand. These structures have natural frequencies below 1 Hz, and may be dynamically excited by turbulent energy in the wind.

As part of the extensive redevelopment of the Simonds stadium, new light towers have been constructed to improve the lighting standard for HDTV. The 70 m highlight towers are of cylindrical tapered pole design, with a large triangular head frame supporting up to 130 light fittings (Figure 1).

The world renowned Adelaide Oval has undergone redevelopment of firstly the Western grandstand and more recently the Eastern and Southern Grandstands (Figure 2). The Southern Grandstand Roof is a large span (150 m) cantilevered roof, with cladding attached to a curved diagrid structure, as shown in Figure 7.

Eden Park is the main sports ground in Auckland, New Zealand. It is used for both rugby union during winter and cricket in summer. To accommodate the final of the 2011 Rugby World Cup, a new South Stand was built, see Figure 3, with an ETFE skin wrapping around the rear of the stand. ETFE cushions are dynamic structures with specific wind loads to assess their performance.

Wind tunnel testing was used to determine the structural loads and dynamic effects on both structures, as well as surface pressures for cladding design. Computational methods were also used to assess concept design solutions such as roof inclination, natural ventilation and thermal comfort of spectators, wind driven rain, fume dispersion from exhausts and the like.

Figure 1 3D image of Simonds Stadium featuring new light towers

Figure 2 3D image of the completed Adelaide Oval Redevelopment

Figure 3 Eden Park new South Stand

Case Study – Eden Park

The proposed new grandstand comprises a cantilevered roof structure to the north covering three tiers of seating. A section through the stand is shown in Figure 2. The exterior wall of the grand-stand is a transparent, ETFE shell. Above the seating area there is a conventional, opaque steel cantilever structure. At the outer edge of the roof there is a smaller roof panel stepped up from the main roof to allow lighting to be placed beneath. There is a gap between the two roof panels for access to the stadium lights below. The leading edge was ventilated prior to the wind tunnel test to minimise vortex shedding and the resulting uplift load as per the work of Killen and Letchford (2001).

The ETFE skin was revised as the design developed to incorporate ferns. These were introduced just prior to the wind tunnel test and are shown on Figure 4. The ETFE skin wraps around the ends of the grandstand. Aurecon originally used vacuum forming on a scaled mould of the ETFE cladding (rather than 3D printing). Pressure taps were incorporated either side of the ETFE skin and cantilevered roof. The wind tunnel model (1:200) is shown in Figure 5 below.

Wind loads for the cantilevered roof structure were measured as peak net pressures at each point (tributary areas or load response correlation methods were not considered at the time). Pressure coefficients (measured surface pressures normalised against the dynamic pressure at the reference height) were processed to determine peak cladding pressures using the up-crossing technique (Melbourne 1977). Dynamic effects were included by applying a dynamic factor as per Standards Australia 2011. The loads determined using this method were about 10% less than those estimated using Standards Australia (2011).

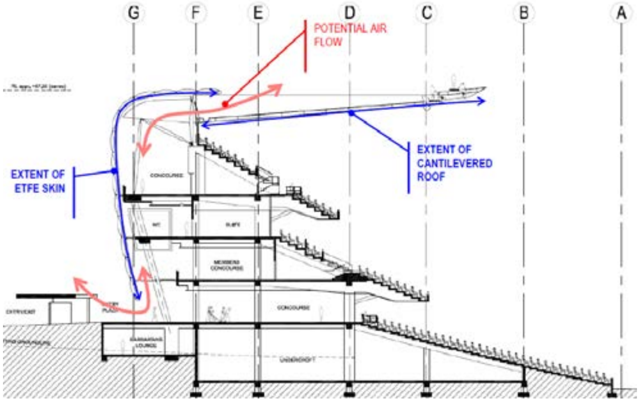

Figure 4 Eden Park South Stand

Figure 5 Eden Park Wind Tunnel Test

The ETFE skin consists of horizontal and vertical cushions as per Figure 6. Area averaged pressures were calculated by averaging for each tap on a cushion the net pressure, being the outer surface minus pressure on the inner surface, at each time step of the data time series (ie. not calculated from the average of the peak net pressures at each tap). Pressure time traces were determined for each cushion, as the tension on the membrane cushions are affected by the internal pressure within the cushion and the size of the cushion as per Figure 7.

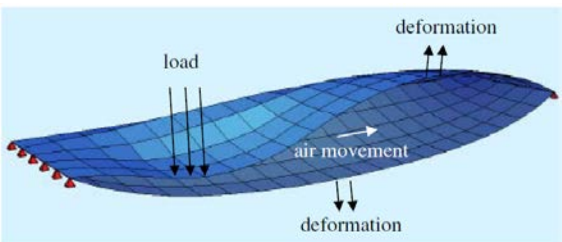

Figure 6 Load distribution on an ETFE cushion

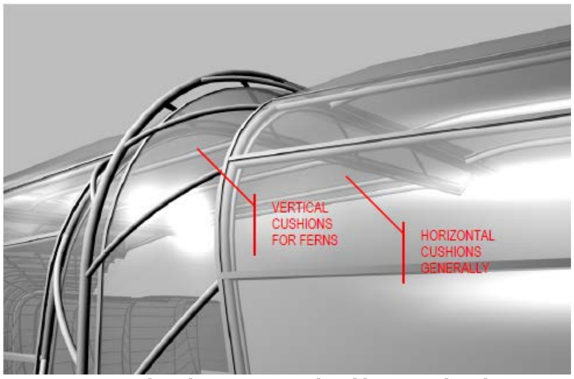

Figure 7 ETFE skin showing vertical and horizontal cushions

Case Study – Adelaide Oval

A 1:200 scale model of the complete Adelaide Oval Redevelopment (recently completed Western Grandstand plus proposed Southern and Eastern Grandstands) and surrounding structures was tested in the wind tunnel. The grandstand structures were constructed from architectural drawings using acrylic, with the complex curved roof shapes formed from a 3D CAD (Revit) model using stereo-lithography (Figure 9).

Figure 8 Diagrid roof structure

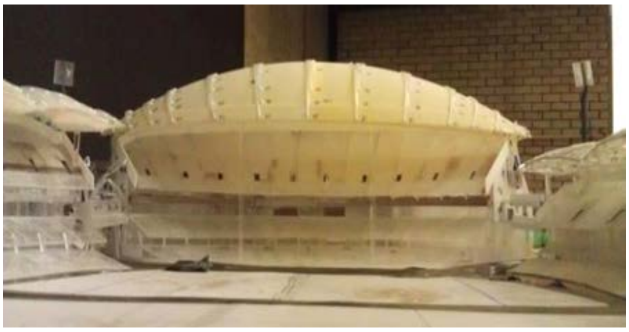

Figure 9 Wind tunnel model at 1:200 scale generated from 3D print

Pressure coefficients (measured surface pressures normalised against the dynamic pressure at the reference height) were processed to determine peak cladding pressures using the up-crossing technique (Melbourne 1977). The peak pressure coefficients were further processed to produce contours of peak positive and negative cladding pressures.

Peak pressures derived from measured pressure coefficients occur locally for small areas and should not be considered for the design of primary structural members (but were considered for the design of cladding and local support structure). Application of these peak loads to the structure simultaneously to perform analysis of structural members could produce an uneconomic design. This concept is shown conceptually in Figure 10. The load-response correlation (LRC) method derived by Kasperski and Nieman (1992) defines an effective pressure distribution, taking into account the correlation of the fluctuating pressure over the whole structure, and provides maximum or minimum load effects using influence coefficients:

A comparison of pressure distributions between peak negative cladding pressures and the maximum LRC load in the y (along span) direction is presented in Figure 12. It is apparent that the application of cladding pressures to the main structural members is incorrect for this load case, and may result in an inefficient structural design. A direct comparison between the upward (peak negative and maximum z LRC) pressure distributions on the roof (Figure 11 and Figure 13) indicates that the magnitude of the LRC pressures is approximately 15% less than the cladding pressures. This is a slightly smaller reduction than anticipated, and indicates that the correlation of pressures across the span of the roof is higher than assumed. It was also difficult to determine influence coefficients for other than principal axes as point of failure is understood to be at any point within the diagrid as opposed to a fixed point supporting the membrane.

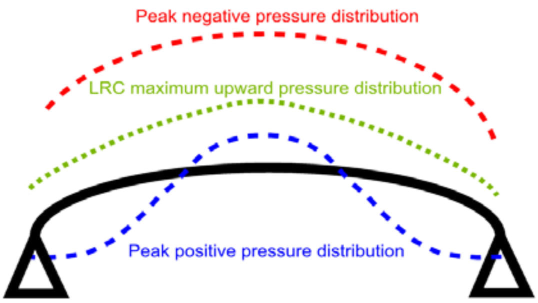

Figure 10 Typical Peak and LRC pressure distributions

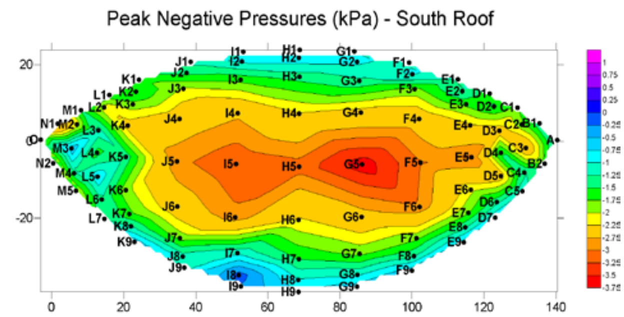

Figure 11 Peak negative cladding pressures

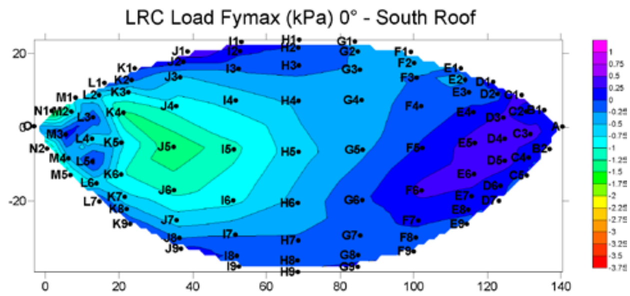

Figure 12 Maximum y (along span, left) LRC pressures

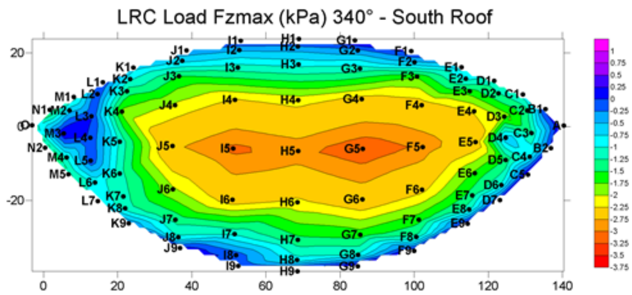

Figure 13 Maximum z (upward) LRC pressures

As opposed to Eden Park's cantilevered roof, the work by Holmes et al (1997) can be used to estimate the resonant component, with resonant loads included at each point by weighting the measured pressure coefficients by the two-dimensional mode shape (the generalised force, spectral density and mean of which were used to obtain the resonant response).

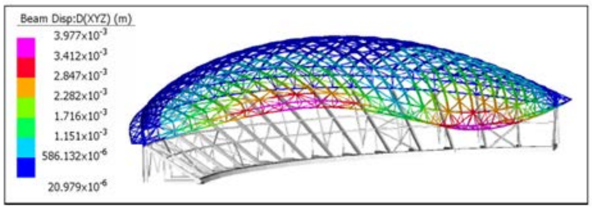

Figure 14 Modal analysis of membrane structure

Case Study – Simmonds & AAMI Stadium

The design extreme wind speed for the Geelong area was calculated using meteorological data with the 1000 year return period gust wind speed of 49.4 m/s about 7% higher than the 1000 year return period gust wind speed of 46 m/s provided in Standards Australia (2011). A gust wind speed of 49.4 m/s was therefore used as the design wind speed for design of the Simonds Stadium Light Towers.

Figure 15 HFBB measurements of drag coefficient

The HFBB method could not be used to calculate base loads which could then be distributed as an ESLD as Reynolds Number effects could not be properly simulated at the geometric scale. Instead a 1:50 scale model of the headframe was constructed and the drag coefficient measured using a HFBB in smooth flow, with little variation of velocity with height and low turbulence. Drag coefficients of the tower were defined from AS1170.

A finite element model of the tower was constructed using ETABS software, with the first along-wind sway mode estimated to have a natural frequency of about 0.7 Hz, and a mode shape of the form $\varphi(z) = (z/h)^{\beta}$ with $\beta \cong 2.0$. The first sway mode was the critical mode as the natural frequency was below 1 Hz and the drag coefficient of the head frame was largest in the along-wind direction. Other modes had natural frequencies above 1 Hz and hence were unlikely to be excited by turbulent fluctuations. The mass distribution was well defined from section properties of the structure.

The mean load was calculated discretely (with Δz) using the mean velocity profile with height based on the terrain category. The background load was calculated using the correlation coefficient derived efficiently given the head-frame dominated the response. Hence correlation of loads not associated with the head-frame could be ignored.

The resonant load was calculated with the critical damping ratio for a steel structure taken as 0.5% (as measured and reported by Kwok et al (1985)), though this was modified by accounting for aerodynamic damping calculated according to Holmes (1996) which added a further 2.5%. As discussed previously, this significantly reduces the resonant response.

Importantly, it can be seen that the resonant load exceeds the mean load for the top third of the tower, with the mean and background load about equal. At the height of the head-frame, given the size of the head-frame, the mean exceeds the resonant load, while the background load exceeds the mean.

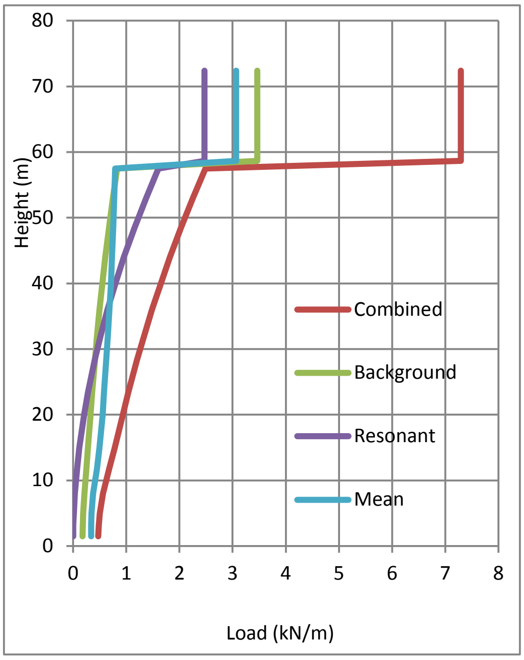

Figure 16 Load contributions to base shear force

The response of the tower was also considered from a serviceability aspect. While motion of the tower is not relevant for human comfort (given rare occupancy), it is relevant given the potential to cause light flicker effects due to specific HDTV broadcast requirements for lighting. Peak displacements of up to 100 mm or RMS of about 25 mm were predicted. Cross-wind effects were considered due to vortex shedding from the tower, with the maximum deflection about 80 mm, occurring at a mean wind speed of about 7.5 m/s, with the base bending moment considerably less than that generated by along-wind loads.

Finally, fatigue effects were also considered due to random (background) and sinusoidal (vortex shedding cross-wind and wind induced resonance along-wind) time variance of the load. Refer to separate paper by Mackenzie and Tanner (2016).

Conclusions

This paper has outlined methods used to analyse wind loads and dynamic effects from a code based dynamic load factor approach (Eden Park), to the difficulties of applying a load response correlation approach for a membrane structure while including dynamic effects based on mode shape (modal mass, stiffness and damping). Dynamic load effects across an ETFE cushion were introduced, with simultaneous pressure time trace applied to a finite element model of each cushion assess compliance with stress limits.

Acknowledgments

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