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Asymmetric wind loading on large roof structures

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Abstract

The wind loading on large span roofs for structural design is not well covered in design standards such as AS/NZS1170.2. A series of simultaneous pressure measurement tests have been conducted on large span roofs mounted on the ground, but with large open sections allowing internal flow. The simultaneous pressure results have been combined with structural characteristics to determine the overall pressure distribution across the roof causing peak structural responses. These responses have been compared with those predicted using the Standards Australia (2011).

This paper will present the findings of the study illustrating the importance of asymmetric loading on the roof from a structural engineering perspective and the potential non-conservative nature of the design Standards.

Keywords: Structural loading, roof structures, wind engineering

Introduction

The design of modern structures requires an accurate assessment of wind loads on cladding and structural elements to ensure adequate strength at a reasonable cost. This need arises from the occurrence of peak design pressures being different to those specified in building design standards, and of asymmetric loading during the peak event. Each peak member response would likely occur at a different time and be caused by a different pressure distribution over the structure.

Cladding loads obtained from building design standards do not account for the effects of building shape significantly different from rectangular, which is particularly true for large roof structures. An accurate determination of cladding wind loads permits an economical façade design. Analytical methods such as computational fluid dynamics (CFD) are not capable, except in very simple geometries, of estimating peak wind pressures or distribution of frame loads causing peak responses.

Standards Australia (2011) describes structural pressure coefficients on roofs. Although not stated in the Standard, the limited nature of the data means it is only suitable for determining loads for fundamental responses such as peak uplift. It does not provide the necessary pressure distributions to determine specific member responses, such as bending at particular locations in the roof.

This paper will present a wind tunnel study, conducted by Cermak Peterka Petersen Pty. Ltd. for Lynar Consulting Structural Engineers, on the wind-induced loading on a stockpile cover and a large curved roof, examining the variations between the study roof load cases and the roof load estimates presented in Standards Australia (2011).

Test methodology overview

Wind tunnel tests were performed to investigate the wind induced loading on large roof structures.

Given the number of influential design variables, e.g. span, curvature, end closure etc., a limited number of configurations were tested.

To determine the pressure distribution over the roof, a simultaneous pressure test was conducted. This technique acquired pressure measurements at discrete points over a scale model of the roof surface. A layout of pressure measurement points was designed to determine the structural pressure distribution over a portal frame design.

Symmetry was utilised, both to reduce the number of tests required, and to optimise the location of the pressure measurement points. Cases in which the roof has a geometrically similar orientation to the wind will produce the same pressure field; therefore, it was not necessary to test all wind directions.

Data observed at model scale in the wind tunnel was scaled and filtered to present equivalent full scale observations. Time histories were processed to represent the peak conditions occurring during a 1-hour event.

The simultaneous pressure measurements were analysed to determine the peak structural responses through the implementation of an influence coefficient analysis. The influence coefficients, provided by the structural engineer, indicate the contribution of loading over a discrete section of the roof to a specific structural response, and can be combined to assess total response using linear superposition. Through this approach, examples of instantaneous nett pressures occurring on the roof that produce peak structural response can be identified.

Wind tunnel test

Modelling of the aerodynamic loading on a structure requires special consideration of flow conditions to obtain similitude between the model and the prototype. A detailed discussion of the similarity requirements and their wind tunnel implementation can be found in Cermak (1971, 1975, 1976). All testing was conducted in accordance with AWES (2001).

The wind tunnel test was performed in the boundary layer wind tunnel, Figure 1. The wind tunnel test section is 3 m wide by 2.4 m high with a porous slatted roof for passive blockage correction. This wind tunnel has a 20 m long test section, the floor of which is covered with roughness elements, preceded by a vorticity generating fence and spires.

The spires, barrier, and roughness elements were designed to provide a modelled atmospheric boundary layer approximately 1.2 m thick with a mean velocity and turbulence intensity profile similar to that expected to occur in the region approaching the modelled area.

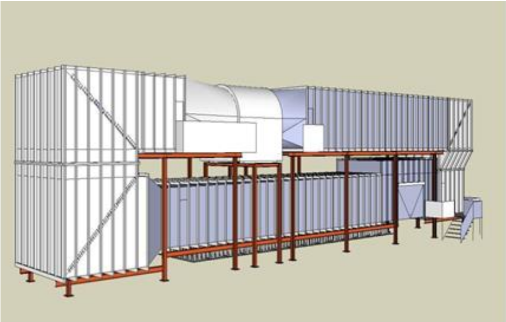

Figure 1. Schematic of the CPP closed circuit wind tunnel.

Wind tunnel model and instrumentation

A model of the twelve sided stockpile cover, shown in Figure 2(T), was constructed at a length scale of 1:200, which is consistent with the modelled atmospheric flow, permitted a reasonable test model size with sufficient space for routing of pressure tubing, and was within wind tunnel blockage limitations. The model was fabricated using stereo-lithography and tested with various configurations of size of stockpile. Similarly, a cladding pressure model of the curved roof was fabricated from plywood at a geometric scale of 1:250, shown in Figure 2(B).

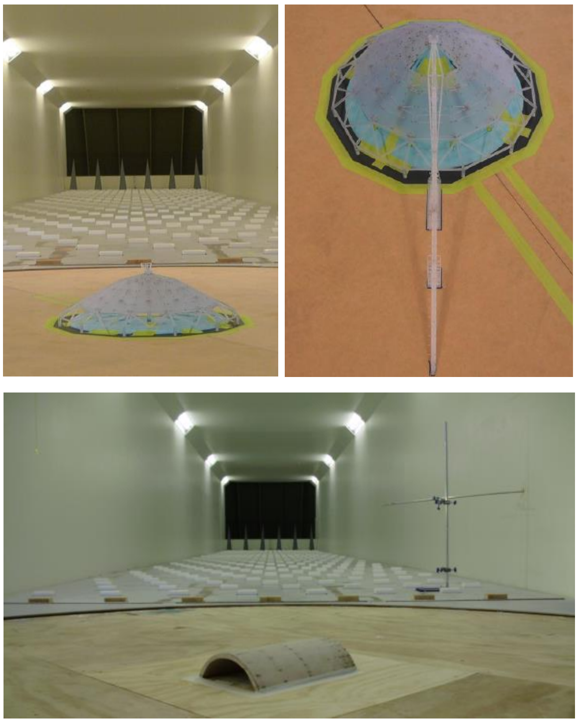

Figure 2. Model of the roof structures in the CPP wind tunnel. Stockpile cover (T), curved roof (B).

Significant variations in the structure surface were formed into the model. Pressure taps were placed into the inner and outer roof surface at a number of locations with a spatial distribution selected to adequately define structural loads on the roof. A minimum of three pressure tappings were used per structural averaging area.

Data acquisition and analysis

Fluctuating pressures were measured at each of the pressure taps. Pressure data was obtained at 10° intervals. A Pitot-static tube mounted in the wind tunnel was used to determine the velocity. A scaled equivalent of five 1-hour blocks of data for each configuration and wind direction was acquired.

The volume of data collected during the wind tunnel study means that it is not practical to check the entire structure for all instantaneous pressure cases observed. In order to identify instantaneous loading cases that maximised specific structural responses, an influence surface analysis was performed in which the measured pressure time histories were used to investigate instances of peak structural response.

Simultaneous pressure coefficient time histories at individual taps were averaged over larger roof areas in consultation with the structural engineer. The area averaged pressures were converted to full-scale pressure time histories and applied to sets of influence coefficients provided by the structural engineer. The structural response at a given time is calculated as:

$$R(t) = \sum_{i=1}^{n} P_i(t) \cdot C_i \tag{1}$$

where $R(t)$ is the response in a member at time t, $P_i(t)$ is the external pressure on panel i at time t, n is the number of areas, and C_i is the influence coefficient which indicates the contribution to response due to loading on the panel.

Results

The output of the influence coefficient analysis provided the respective simultaneous static loading on areas which maximized the structural response in specific sections of the roof. The influence coefficients were provided by Lynar Consulting.

Stockpile cover

For the stockpile cover, eleven influence cases were examined in each of 12 bays, shown in Figure 3, for 19 wind directions. For each influence case, in each bay, the peak positive and negative response was determined, and the associated pressure distribution that produced the median of the peak response from the five runs.

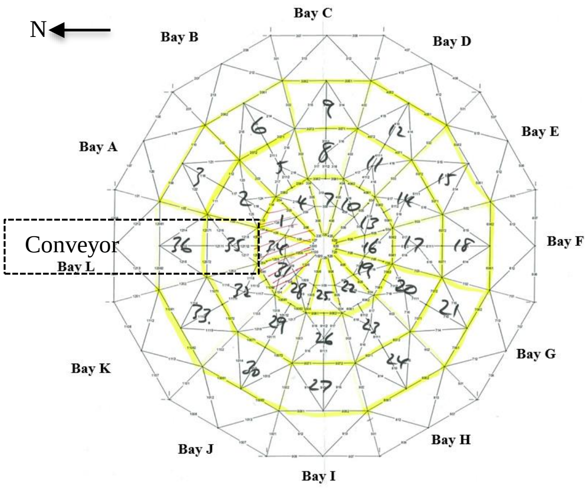

Figure 3. Stockpile cover averaging areas for structural loading investigation (Lynar Consulting, 2012).

For similarity with Standards Australia (2011), pressure coefficient distributions were referenced to a design gust speed at mid-roof height (22.5 m AGL). Table 1 shows a sample of the pressure coefficients experienced by the stockpile cover.

Bay	InfC	Dir /°	Structural Area								
			1	2	3	4	5	6	…	35	36
A	1	10	-0.56	-0.10	0.50	-0.61	-0.59	-0.22		0.06	0.62
	2	360	-0.33	0.01	0.38	-0.39	-0.25	-0.11		0.08	0.59
	3	20	-0.34	0.07	0.57	-0.47	-0.32	0.10		-0.11	0.43
	4	310	-0.49	-0.70	-0.53	-0.32	-0.17	-0.26		-0.35	-0.15
	5	300	-0.66	-0.80	-0.55	-0.54	-0.31	-0.38		-0.34	-0.13
	6	310	-0.78	-0.83	-0.44	-0.59	-0.37	-0.45		-0.35	0.15
	7	40	-0.27	0.10	0.75	-0.37	-0.08	0.52		-0.18	0.20
	8	0	-0.50	-0.30	-0.02	-0.57	-0.73	-0.45		0.07	0.50
	9	0	-0.50	-0.30	-0.02	-0.57	-0.73	-0.45		0.07	0.50
	10	360	-0.61	-0.24	0.26	-0.61	-0.45	-0.32		0.00	0.63
	11	60	-0.22	-0.01	0.41	-0.36	0.05	0.73		-0.43	-0.25

Table 1. Maximum response pressure coefficient distributions (C_p).

Due to the non-codified nature of the roof structure, it was impossible to compare the resulting pressure coefficients with the theoretical estimates from Standards Australia (2011). The pressure distribution across the roof producing each peak response varies significantly and therefore is not easily codified.

Curved roof

A large curved roof with a 50 m span, and 16.7 m rise, was tested with a mean wind speed of approximately 10 m/s measured at 20 m above ground level at full scale, from 0° to 90°. The roof was tested at three portal frame locations, shown schematically in Figure 4.

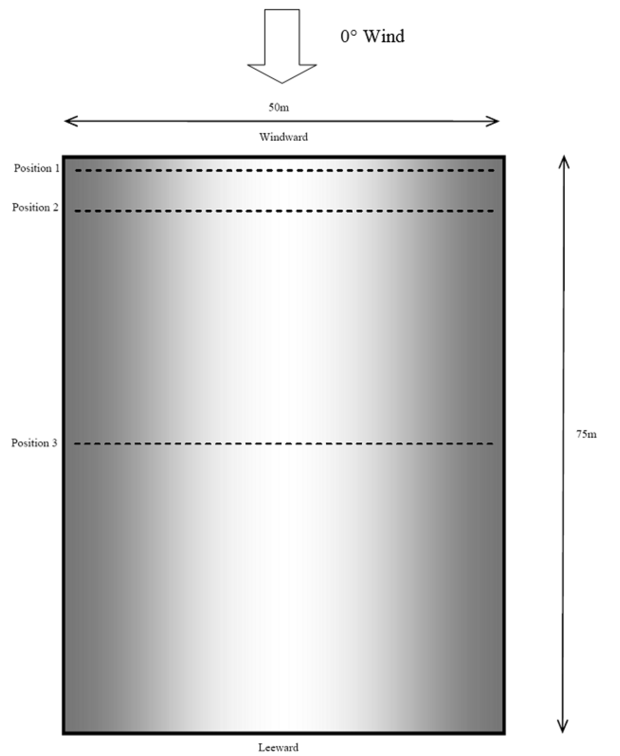

Figure 4. Approximate locations of trusses analyzed for the curved roof. Distances to the trusses from the windward edge are 1.5, 6.5, and 37.5 m.

Pressure results were expressed in the form of non-dimensional pressure coefficients, which take the form:

$$p = \frac{1}{2} \cdot \rho \cdot \hat{v}^2 \cdot Cp \qquad (2)$$

where, p is the pressure [Pa], ρ is the density of the fluid taken as 1.205 kg·m^{-3}, $\hat{V}$ is the gust speed of approach flow at a height of 10 m AGL [m·s^{-1}], and Cp is the dimensionless pressure coefficient.

For each influence case, the peak positive and negative response was determined, as was the pressure distribution that produced the peak response. As the structural design of the roof is symmetric about the longitudinal axis, the most severe results of symmetric cases were extracted from the data as the design case.

The structural responses were then compared with theoretical estimates, calculated using the information in Figure 5, Table 3, and Table C3 in Standards Australia (2011). External pressure coefficients for the curved roof estimated from Standards Australia (2011) are shown in Table 2.

External Pressure Coefficients (Cp,e) - Curved Roofs			
Rise-to-span ratio (r/d)	Windward quarter (U)	Centre half (T)	Leeward quarter (D)
0.334	0.396 or 0.0	-0.55	-0.178 or 0.0

Table 2. External pressure coefficients combinations using Standards Australia (2011).

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Load Case	M_{11} kNm	N_{11} kN	V_1 kN	H_1 kN	V_2 kN	H_2 kN
1	-2.57	0.50	2.33	-3.49	0.22	-0.46
2	-7.37	1.48	2.51	-2.13	0.66	-1.34
3	-11.18	2.34	2.63	-0.80	1.08	-2.11
4	-13.44	3.03	2.67	0.43	1.47	-2.68
5	-13.74	3.51	2.63	1.48	1.81	-3.02
6	-11.79	3.73	2.53	2.29	2.11	-3.07
7	-7.49	3.67	2.35	2.83	2.35	-2.83
8	-0.92	3.34	2.11	3.07	2.53	-2.29
9	7.70	2.74	1.82	3.01	2.63	-1.48
10	17.96	1.91	1.47	2.68	2.67	-0.43
11	23.89	1.10	1.08	2.11	2.63	0.80
12	14.67	0.71	0.66	1.34	2.51	2.13
13	4.84	0.25	0.22	0.46	2.33	3.49

Table 3. Influence coefficient data (Lynar Consulting, 2010).

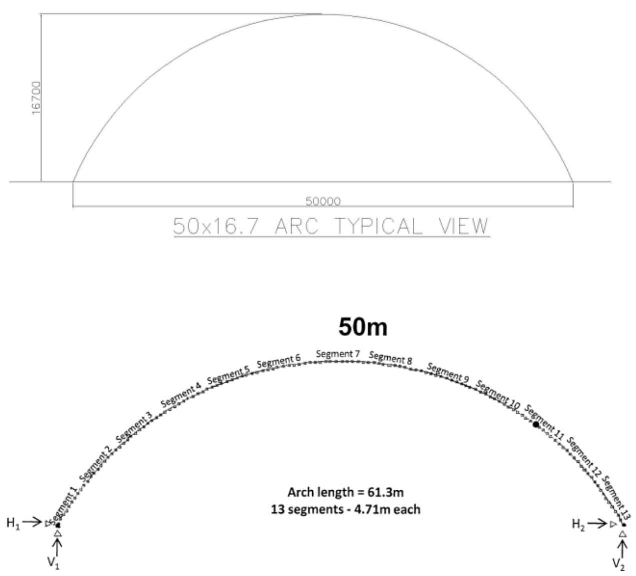

Figure 5. Sectional dimensions of the curved roof (T), and influence coefficient segment locations (B) (Lynar Consulting, 2010).

The roof was divided into three sections for direct comparison with Standards Australia (2011). The peak structural bending moment response for influence coefficient M11, and associated pressure distribution along the curved roof for various building opening configurations are shown in Table 4. Similarly, Table 5 shows the peak vertical uplift, V1, structural response and associated pressure distribution along the curved roof for the various opening configurations tested. It should be noted that the pressure coefficients for the wind tunnel test case have been calculated in accordance with the influence coefficient; therefore, if the influence coefficient changes sign across the structural panel areas, the required pressure for the simplified influence coefficient case will be amplified. So although the pressure distributions are not physically comparable, the peak responses can be directly compared.

For the closed and Standards Australia (2011) cases the internal pressure has not been included in the analysis, and for the partially open and open cases differential measurements were taken on either sides of the roof element and integrated to provide a net pressure on the roof material for the analysis.

Configuration	Dir / °	Maximum bending moment response (M11), at Position 3 (Cp)	Segment Distribution (Cp)		
			1 to 3	4 to 10	11 to 13
Open	70	62.52	-0.208	-1.817	0.430
Closed	50	54.41	-0.584	-1.970	-0.016
Windward closed	70	53.13	-0.087	-1.662	0.350
AS/NZS1170.2:2011		32.90	-0.178	-0.550	0.396

Configuration	Dir / °	Minimum bending moment response (M11), at Position 3 (Cp)	Segment Distribution (Cp)		
			1 to 3	4 to 10	11 to 13
Open	20	-23.31	0.074	0.464	-0.269
Closed	10	-24.50	-0.030	0.317	-0.421
Windward closed	0	-23.37	0.063	0.338	-0.339
AS/NZS1170.2:2011		-4.14	0.396	-0.550	-0.178

Table 4. Curved roof maximum (T) and minimum (B) bending moment response.

Configuration	Dir / °	Maximum vertical response (V1), at Position 3 (Cp)	Segment Distribution (Cp)		
			1 to 3	4 to 10	11 to 13
Open	10	2.61	0.066	0.145	-0.074
Closed	0	4.35	0.107	0.207	0.171
Windward closed	10	3.36	0.065	0.172	0.102
AS/NZS1170.2:2011		-5.61	0.396	-0.550	0.000

Configuration	Dir / °	Minimum vertical response (V1), at Position 3 (Cp)	Segment Distribution (Cp)		
			1 to 3	4 to 10	11 to 13
Open	70	-16.54	-0.310	-0.924	0.082
Closed	50	-21.66	-0.713	-1.030	-0.151
Windward closed	90	-16.44	-0.333	-0.920	0.194
AS/NZS1170.2:2011		-9.90	-0.178	-0.550	0.000

Table 5. Curved roof maximum (T) and minimum (B) vertical response.

It is evident from Table 4 and Table 5 that the code estimates are non-conservative in all cases compared with the closed-end configurations, which is similar to the standard codified structure for curved roofs. Responses at portal locations close to the building edge, Positions 1 and 2 in Figure 4, are greater respectively by about 60% and 30% compared with Position 3 in the middle of the roof. The wind loading from Standards Australia (2011) is constant along the entire length of the roof.

It is evident that the responses for the open configurations are similar in magnitude to the closed case, and do not provide any significant increase in loading.

It should be noted that any dynamic response of the curved roof was not included in the analysis. The curved roof was not expected to be susceptible to self-induced resonance, hence a quasi-steady analysis technique was adopted.

Discussion

The curved roof load cases identified, and their intended application in design, tend to differ from the loading information for curved roofs presented in Standards Australia (2011).

Loads presented in Standards Australia (2011) represent global loading cases, whereas the load cases identified in the curved roof study represent maximization of specific design actions due to loads over smaller sections of the structure.

Global loading on structures are typically reduced upon conducting wind tunnel studies, particularly on large structures in complex conditions. Localized cladding loadings and structural responses can be greater than those estimated using Standards Australia (2011), particularly on non-codified structures.

By virtue of the small areas investigated and increased specificity of the identification process, it is reasonable to expect that the peak responses identified in the wind tunnel study would be higher and the associated wind loading patterns different to those in the Standard. The significant change in design pressure coefficients has significant design implications for this type of structure as evidenced by recent failure of such roofs in non-extreme wind events. It is evident that reasonably well correlated vertical loads are experienced across the roof for winds coming from over the gable end of the structure.

Although it may be suitable to use Standards Australia (2011) to determine the reference pressure to apply to the pressure coefficients presented above, the pressure coefficient distributions resulting from the wind tunnel study are not intended to be used with factors such as K_a and K_l in the standard, as these are included in the analysis.

Conclusions

A stockpile cover and a curved roof were tested in a variety of configurations. The results of the studies apply only to the geometries tested, and variation in the geometric configuration of the structure will result in different loading conditions.

However, reasonable inferences can be made from the results of the study presented above about the wind loads on structures with similar geometries to those tested.

From a structural engineering perspective, the structural responses caused by asymmetric loading on large roof structures are found to be greater than those estimated using Standards Australia (2011). From the results presented, designing large roof structures based on the code would generally lead to non-conservative design loads, which can consequently lead to structural failure and undesired costs.

Acknowledgments

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