Wind Engineering Applied to Heritage Structures

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Abstract

Re-use of existing building fabric has become an integral part of sustainable and economic design with a primary example being the South Australian Government's Tonsley Redevelopment. The adaptive re-use of the Main Assembly Building (MAB) prevented the loss of approximately 90,000 tonnes of carbon emissions embodied in its original construction and saved extensive costs and programme delays. Its rejuvenation has avoided carbon emissions of a scale equivalent to taking 25,000 average cars off the road for one year. The former Mitsubishi site was constructed before wind codes were developed in Australia, and the redevelopment required the use of extensive wind tunnel testing and analysis (using pressure time history correlation analysis) to avoid the use of costly strengthening of the existing steel saw-tooth roof structure otherwise required through the application of the current wind code (AS1170.2 [11]).

Introduction

Aurecon was engaged by the Land Management Corporation (LMC) to perform a structural assessment to current Building Code of Australia (BCA) compliance and design any necessary strengthening for the recently decommissioned Main Assembly Building at Tonsley Park, Adelaide, South Australia.

The majority of the structure was constructed in the early 1960's with staged construction thereafter. Figure 1 shows the extents of the building, which is approximately 570m long x 170m wide. The height of each bay to the bottom chord of the truss varies from 15m for high, 10m for medium and 6m for low bays respectively.

The Main Assembly Building consists of a series of triangular saw-tooth steel trusses spanning about 12m North-South (at about 4m centres), between a regular grid of steel columns, at about 12m centres (North-South) and 24m centres (East-West), and rectangular steel Pratt trusses 4.5m deep, spanning 24m (East-West).

The roof was originally clad in heavy corrugated asbestos sheeting, with vertical glazing to the south lights. The roof purlins consist of imperial sized unequal angles. The roof trusses typically consist of small angle members which have limited capacity to carry compression loads. The building relies on the columns to cantilever from the floor slab to transfer any lateral loads from wind or earthquake. Refer to Figure 2 showing a photo of the original steel structure and cladding.

As part of the redevelopment, openings in the structure were proposed, along with new light-weight steel cladding/insulation and well-sealed glazing. The existing asbestos sheeting was removed and replaced with light-weight cladding further exacerbating the upgrade issues structurally. Refer to Figure 15 for an image of the refurbishment.

Our work required the assessment of wind loads on the structure during temporary works (removal of the roof, then skylights), as well as at completion. Initially a code based approach to AS1170.2 [11] was used to determine strengthening requirements, however this resulted in an overly conservative outcome with high loads and extensive strengthening works. We were then engaged to carry out wind tunnel testing and analysis with the hope of reducing strengthening requirements necessary for the feasibility of the adaptive re-use proposed.

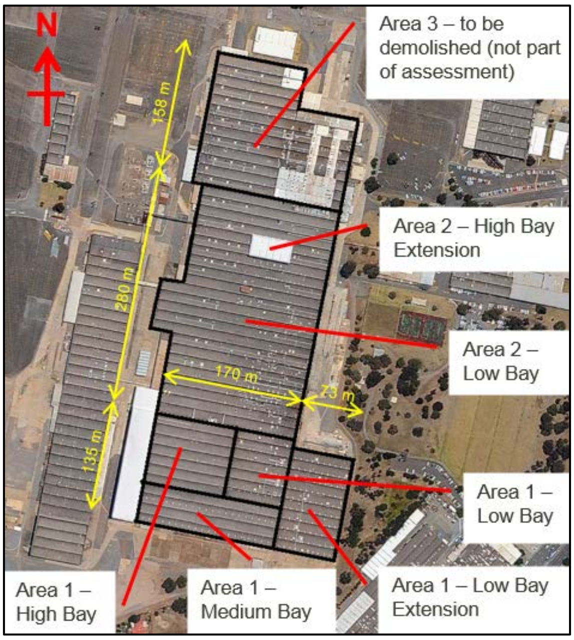


Figure 1 Plan view showing the extent of the building and representative heights of each bay

Figure 2 Photo of the original steel structure

Wind Codes

SAA Interim 350

The "Interim Code for Minimum Design Loads on Buildings", SAA Interim 350 (Standards Association of Australia, 1952) [10] was used to define wind loads for the design of the existing structure. This code superseded SAA Code No. CA.1 – 1939 for Structural Steel in Building which included a brief section dealing with wind loading. Imperial units were used in this code, with an "average wind velocity" designated as 75mph (33m/s) and "wind pressure on roofs" of "multiple-bay buildings" reduced from that for single-bay roofs such that:

- On the windward bay of the building – nil
- On the bay immediately leeward – 50%
- On the next bay to leeward – 75%
- On all the remaining bays – 87.5%

A clause to consider internal pressures and local pressure factors for cladding and fastenings were also included.

AS1170.2-2011

As noted above, the current version AS1170.2:2011 "Structural Design Actions, Part 2 : Wind actions" [11] was first published as part of SAA Int. 350, before being revised and designated AS CA34.2 in 1971. This latter standard was subject to various revisions and reclassifications until the 1989 version first incorporated the extensive work of John Holmes [4][5] on wind loads on multi-bay sawtooth roofs, using gust as opposed to mean wind speeds as discussed further below.

Literature Review

The basis of AS1170.2 was derived from work carried out by the CSIRO, specifically John Holmes [4][5] as summarised in his paper "Wind loading of multi-span buildings " [6].

In Holmes' early work [4], a rural boundary layer (terrain category 2, TC2) was simulated at 1:200, with a 5 bay model (roof incline of 20°) shown below in Figure 3.

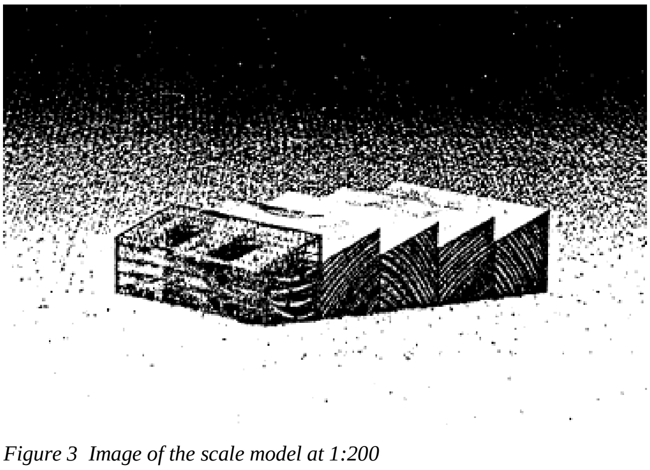


Figure 3 Image of the scale model at 1:200

One of the bays was fitted with pressure taps, and pressure coefficients (peak, mean, RMS) derived relative to a mean dynamic pressure at eaves height (10m full scale). As one bay was tested at a time in different locations along the multi-bay span, correlation across bays or lack there-of could not be considered for derivation of lateral loads (drag).

Quasi-steady pressure coefficients (relative to a dynamic pressure based on a gust wind speed) were derived for each bay (end and intermediate) with wind from each direction, as shown below in Figure 5 and Figure 4. The results were similar to those proposed for the original code, SAA Int. 350 outlined above.

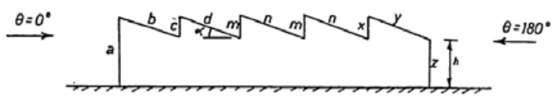


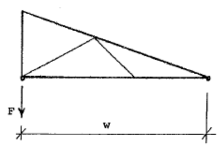
Figure 4 Naming convention for a multi-span building

Wind direction θ	External pressure coefficient C_p								
	First span		Second span		Other intermediate spans		End span		
	a	b	c	d	m	n	x	y	z
0°	+0.6	-0.9	-0.9	-0.5, +0.2	-0.3, +0.2	-0.3, +0.2	-0.3, +0.2	-0.4	-0.2
180°	-0.3	-0.2, +0.2	-0.3	-0.2, +0.2	-0.4	-0.4	-0.7	-0.3	+0.6

Figure 5 Proposed quasi-steady pressure coefficients for a multi-span building

In his subsequent work, Holmes [5] proposed amendments to the proposed quasi-steady pressure coefficients based on area averaged panel pressures (pneumatically), and a load response correlation (LRC) analysis to determine peak force coefficients (refer to Figure 6). This primarily affected the values for the intermediate spans for wind approaching at 0°, and appear to be based on an average between the edge and central elements of a bay. Holmes [5] work also included wind from oblique angles which were not adopted in the code. Pressure coefficients proposed for the code were justified based on having about the same peak force coefficient (using a gust factor at 10m for TC2 of 2.5) as that determined from the LRC method (using a peak factor of 4.0, however the "quasi-steady" force coefficient derived using a gust factor of 2.5 was considerably greater). The peak factor should in fact be 2.8 for TC2 at 10m height.

Wind angle θ	Span	Mean $\bar{C}_F$	R.m.s. C'_F	Peak value $\hat{C}_F$			
				Expected	Quasi-steady ˣ	Current AS 1170/2	Proposed wind code (Table VII)
0°	1	0.564	0.159	1.20*	1.41	0.83	1.12
45°	1	0.713	0.200	1.51*	1.78	-	1.60
90°	3	0.561	0.150	1.16†	1.40	0.90	1.10



* Derived from $\hat{C}_F = \bar{C}_F + 4C'_F$

† Derived from coincident peaks

x Derived from $\hat{C}_F = 2.5\ \bar{C}_F$

$$\hat{F} = \tfrac{1}{2}\ \bar{u}_h^2 . wb . \hat{C}_F$$

b = breadth of tributary area normal to truss

Figure 6 Calculation of roof support holding force

Saathoff and Stathopoulos [8][12] carried out independent wind tunnel testing (1:400 scale) and analysis for the American wind code [2] with quasi-steady pressure coefficients relative to a dynamic pressure based on a mean wind speed. Correcting for gust to mean dynamic pressure reference and applying a local pressure multiplying factor from AS1170, their local peak pressure coefficients were significantly greater (both positive and negative).

Recently Prevatt and Cui [8] provided a review of "Wind Tunnel Studies on Sawtooth and Monoslope Roofs", with independent testing (1:100 scale) carried out to attempt to resolve the discrepancy between the work of Holmes [4][5][6] and Saathoff and Stathopoulos [9][12] finding results consistent with the latter researchers.

Code Based Assessment

JDH Consulting (John Holmes) were originally engaged by LMC to carry out an assessment of wind loads according to AS1170.2. Significant strengthening of the structure was required, due to the removal of heavy asbestos cladding and the increase in uplift as a result of the current code evaluation.

The roof purlins needed to be strengthened around the edges of the building and where the building steps in height at the locations of localised peak wind pressures.

The roof trusses needed to be strengthened (diagonal and bottom chord as shown in Figure 8) typically around the perimeter of the building and where the building steps in height ie at the edges of the high bay locations. In most locations, the roof trusses have no bottom chord restraint. These needed to be introduced to prevent the bottom chord from translating sideways (bottom chord tie as shown in Figure 8).

The columns were significantly over-stressed and the footings had insufficient capacity to carry the lateral wind loads required by the current Australian Standards. As it would be a difficult and expensive exercise to modify the footings, the lateral stability system was strengthened by introducing a large beam to the underside of the roof trusses on the main column grids. This beam was to be rigidly connected to the existing columns to form portal frames reducing the loads in the columns and foundations.

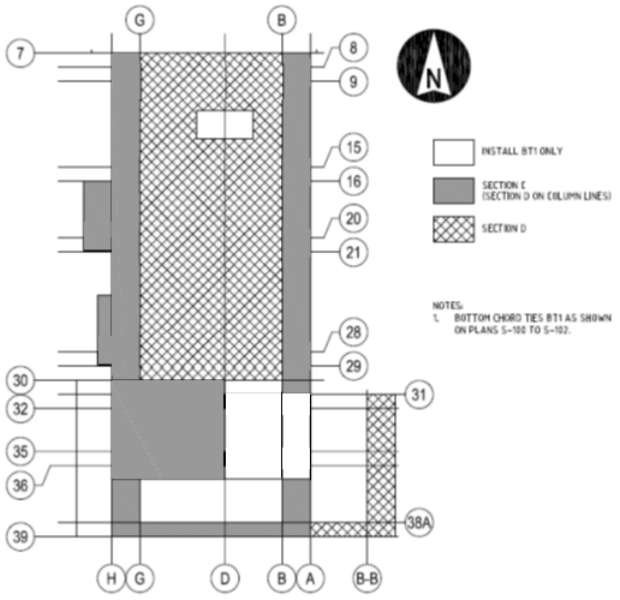


Figure 7 Secondary truss strengthening plan (Uplift only)

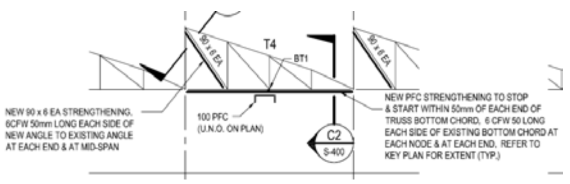


Figure 8 Strengthening requirements – Section C

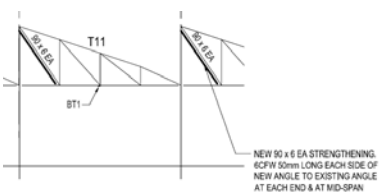


Figure 9 Strengthening requirements – Section D

The proposed extent of strengthening and associated cost led to consideration of a specific wind tunnel test given limitations of the wind code's application to the site and complexity of the existing building.

Wind Tunnel Assessment

Measurements were carried out using the 1.4MW wind tunnel at Monash University. This tunnel is powered by 4x400kW electric motors running 2 x 5m diameter fans with an open-jet section (4 x 2.6 x 11m3), which housed an automated turn-table of diameter 4.0m and a development length of about 15m. The turbulent boundary layer was established using trip boards and roughness elements over the development length (or fetch).

A 1:200 scale model of the entire building was constructed with removable elements (parapets, cladding etc) to simulate the proposed refurbishment as shown below in Figure 10. The surrounds and terrain were included to 500m. Lines of pressure taps ran both North-South and East-West across the model, with internal/external pressures measured concurrently to enable a time history of net pressures to establish loads resulting from openable elements of cladding. Measurements were carried out every 10°, pressures sampled at 1kHz, with a low pass filter at 500Hz. Measurements were carried out in accordance with AWES [3] and ASCE [1] manuals.

Figure 10 Wind tunnel model with smoke visualisation

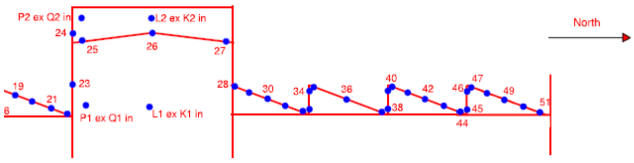


Figure 11 Pressure tap density included in the model

The load-response correlation (LRC) method derived by Kasperski and Nieman [7] was used to determine effective peak pressures on the truss for given load cases (responses). This method takes into account the correlation of the fluctuating pressure over the whole structure, and provides maximum or minimum load effects using influence coefficients:

$$\left(C_{p_i,eff}\right)_{\hat{F}} = C_{p_i,mean} \pm g\rho_{F,p_i}\sigma_{c_{p_i}} \qquad (1)$$

Where $C_{p_i,mean}$ is the mean pressure coefficient, g is the peak factor, and $\sigma_{c_{p_i}}$ is the standard deviation of the pressure coefficient. The correlation coefficient, ρ_{F,p_i}, between the pressure at a tap, i, and any force, F, is given by:

$$\rho_{F,p_i} = \frac{\sum_k \overline{C_{p_i}C_{p_k}} I_k}{\sigma_{c_{p_i}}\sigma_F} \qquad \sigma_F = \sum_i \sum_k \overline{C_{p_i}C_{p_k}} I_k I_i \qquad (2a,b)$$

Where I_i and I_k are the influence of the pressure at tap i and k on the load effect. These equations can be expressed conveniently in matrix notation to enable ease of application to structures with multiple pressure taps.

The responses considered were in the diagonal member connecting between the top edge of the truss and bottom chord (M1), and in the bottom chord of the truss (M2), as shown below in Figure 12. Pressure were area averaged where possible and included as (P3, P4, P5, P6). Lateral loads (Fc) were also considered using the same approach.

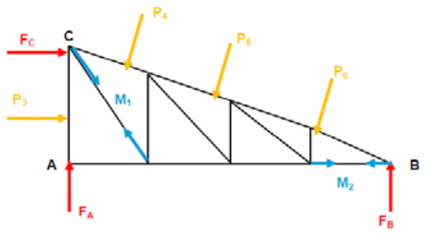


Figure 12 Loads and Responses for each truss

Using this approach, the resulting effective pressure distributions were provided to the structural engineers for their analysis, with the resulting reduction in strengthening requirements shown in Figure 13 and Figure 14.

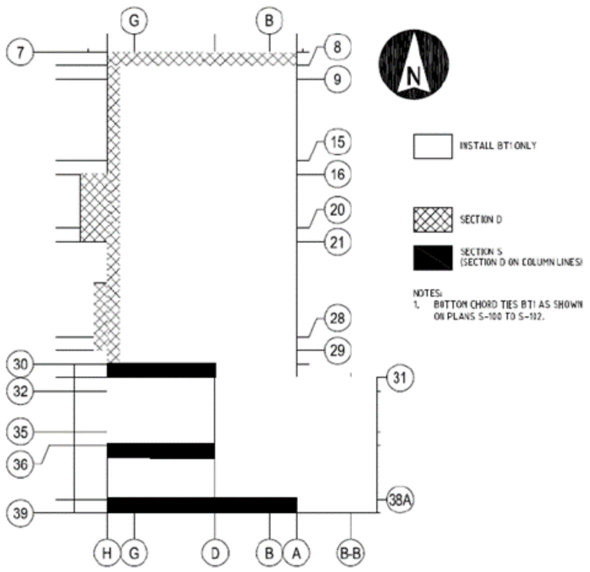


Figure 13 Strengthening required following wind tunnel testing and analysis

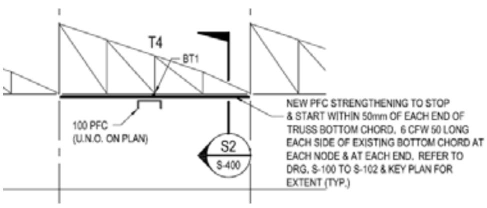


Figure 14 Strengthening required – Section S

The removal of the bottom chord and diagonal members, resulted in a saving of almost 100 tonnes of steel and associated labour.

Conclusions

The use of improved analysis and measurement techniques in wind engineering enabled a significant reduction in strengthening required to allow cost effective re-use of a significant building asset. The approach modelled the specific complex geometry and surrounding built form, while taking into account the local wind environment and terrain. The use of the load-response-correlation method allowed consideration of critical load cases rather than a generalised structural response. The literature review highlighted issues for further consideration in the wind code. The building as completed is shown below in Figure 15.

Figure 15 Completed building showing the extent of refurbishment

Acknowledgments

We acknowledge the Land Management Corporation (LMC) for allowing us to participate in the urban renewal of this significant site. Aurecon's structural engineering team comprising led by Garth Rowland (Project Leader), James Trezona (Project Director), Sam Johnsson (Project Engineer) and Mark Ellis (now with the University of SA). Thanks also to Jason Gaekwad who carried out the project work during his time with Aurecon, and the staff at Monash University's wind tunnel for allowing us to use their facility.

References

[1] ASCE (1999), "Wind Tunnel Studies of Buildings and Structures", ASCE Manuals and Reports on Engineering Practice No. 67

[2] ASCE/SEI. (2006). "Minimum design loads for buildings and other structures." ASCE 7-05, Reston, Va.

[3] Australian Wind Engineering Society (2001), "Wind Engineering Studies of Buildings", AWES-QAM-1-2001AWES QA Manual

[4] Holmes, J. D. (1983). "Wind loading of saw-tooth roof buildings—I. Point pressures." Internal Rep. No. 83/17, CSIRO, Div. of Building Research, Highett, Australia.

[5] Holmes, J. D. (1984). "Wind loading of saw-tooth roof buildings—II. Panel pressures." Internal Rep. No. 84/1, CSIRO, Div. of Building Research, Highett, Australia.

[6] Holmes, J. D. (1987). "Wind loading of multi-span buildings." First National Structural Engineering Conf., Institution of Engineers, Australia, Melbourne, Australia.

[7] Kasperski, M & Niemann, H-J (1992), The L.R.C. (load-response-correlation) – Method: A General Method of Estimating Unfavourable Wind Load Distributions for Linear and Non-Linear Structural Behaviour, J. Wind Eng. Ind. Aerodyn., Vol 43, pp 1753-1763.

[8] Prevatt, D.O. and Cui, B. (2010). "Wind Tunnel Studies on Sawtooth and Monosloped Roofs", J. Struct. Eng., 136(9), 1161-1171.

[9] Saathoff, P. J., and Stathopoulos, T. (1992). "Wind loads on buildings with sawtooth roofs." J. Struct. Eng., 118(2), 429–446.

[10] Standards Association of Australia (1952). "SAA Interim 350".

[11] Standards Australia (2011). "Structural Design Actions, Part 2: Wind Actions", AS/NZS 1170.2:2011.

[12] Stathopoulos, T., and Saathoff, P. (1992). "Codification of wind pressure coefficients for sawtooth roofs." J. Wind. Eng. Ind. Aerodyn., 43(1–3), 1727–1738.