20th Australasian Fluid Mechanics Conference
Perth, Australia
5-8 December 2016

Integration of Wind Tunnel Pressure Measurements with the Structural Model for a New Zealand Stadium Roof

A. McBride[1], R.G.J. Flay[2], T. Stuart[1], J.B. Davidson[1], D.J. Le Pelley[3], B.J. Davidson[1], Y.F. Li[2]

[1]Compusoft Engineering Limited, Auckland 1023, New Zealand

[2] Department of Mechanical Engineering, University of Auckland, Auckland 1142, New Zealand

[3]Doyle Sailmakers, Auckland 1746, New Zealand

Abstract

Wind tunnel tests were performed on a 1:200 scale model of the long span west stadium canopy proposed for QBE Stadium in Auckland. The tests measured the pressures on the proposed roofing elements. These were then processed into pressure coefficients based on the tunnel wind speed. Selection of critical load cases was carried out by integrating for key metrics for this structure, defined as maximum base shears and overturning moments for the whole structure. The wind tunnel pressure data results for these critical load cases were then integrated directly with the structural analysis model. This approach enabled 10 unique load cases to be determined. It was found that this was a very effective way of determining the wind loads and designing an efficient structure to resist them.

Introduction

The QBE stadium is located 17 km north of the Auckland CBD at Albany. In order to increase its utility, Stadiums Auckland is investigating roofing it completely. This comprises a new roof above the west stand to be built in phase 1, and an openable central roof section which completes the stadium enclosure to be built in phase 2. Wind loads are crucial to the design and thus wind tunnel tests were commissioned in order to obtain loads for various design configurations.

The wind tunnel investigation was carried out in the boundary layer wind tunnel at the University of Auckland's Newmarket Campus on a 1:200 scale model of the long span west stadium canopy proposed for QBE Stadium in accordance with the requirements of the Australasian Wind Engineering Society quality assurance manual [1]. The tests measured the pressures on both the existing and proposed roofing elements.
The paper relates to the results from the modelling of the new phase 1 canopy, as well as the analysis including

- A description of the proposed structure to be built in phase 1.
- Estimation of the design loads using conventional 'Code' approaches, i.e. AS/NZS1170.2 [2].
- Processing of the wind tunnel data for implementation into an analysis model.
- A description of the process adopted for the selection of appropriate design load cases.
- Analysis and design of the structure.

It is anticipated that additional work will be published that will look at the interpretation of the wind tunnel data on the existing roof, as well as dynamic sensitivity and interpretation of results for fatigue assessment of the structure.

Description of the structure

The west stand canopy comprises a 250 m span leading edge tri-chord steel truss that rises to 40 m above pitch level at mid-span. From this truss a total of 12 radial tri-chord trusses in turn span down to the ground level behind the west stand embankment. All structural steel truss elements are to be fabricated from circular hollow sections of varying sizes that have been optimised according to the loading demands derived from wind tunnel testing. The steel structure is then clad with a combination of tensioned PVC architectural membrane fabric and glass fibre, fluo-polymer reinforced membrane fabric. The total area of the tensioned membrane is 7,800 m^2.

The maximum width of the structure at the mid-line is approximately 55 m and is kept elevated above the ground and embankment levels to ensure air flow under the structure. Because of the extent of the structure a number of rear props have also been introduced to assist with the structural integrity. The general structure located on the site is illustrated in figure 1.

Underlying the stadium are layers of alluvium material varying in depth which in turn overlay the bedrock horizon and hence all foundations to the canopy are piled to bedrock. Understanding the interaction of the superstructure with its foundation system is an integral step in the process of understanding the whole structure's response to wind loading.

At the time of writing, construction is targeted to commence in the fourth quarter of 2016 or the first quarter of 2017.

Figure 1. Rendered view of west canopy (in the background) at the site.

Site specific wind speed estimation using AS/NZS1170.2:2011

The site is located in region "A6" as categorised by the Loadings Standard which gives rise to a regional wind speed, V_R, equal to 46 m/s for the prescribed annual probability of exceedance as per AS/NZS1170.2 [2] (= 1/1,000) for the Ultimate Limit State

(ULS). The site is best categorised as constituting Terrain Category 2.5 (TC2.5)

The derivation of the site wind speed for the ULS scenario has been determined in accordance with the Loadings Standard and is outlined in Equation 1 below:

$$V_{sit,\beta} = V_R M_d \left(M_{z,cat} M_s M_t\right) \quad (1)$$

Where,

- V_R is equal to a 46 m/s gust wind speed for an annual probability of exceedance equal to 1/1000 applicable for ULS strength design.
- M_d is the wind directional multiplier.
- $M_{z,cat}$ is the terrain / height multiplier determined as equal to 1.125 given a reference height for the wind tunnel investigation, $h = 50m$ and Terrain Category 2.5 (TC2.5).
- $M_s M_t$ the shielding and topographic multipliers determined as being equal to 1.0.

Because of the variation in the wind directional multiplier the design wind (gust) speed $V_{des,\theta}$ also varies accordingly. As for the wind directional multiplier, the design wind speeds for the structure have been developed for 36 incident angles (i.e. 10 ° increments around the compass) in accordance with the requirements of the Loadings Standard [2].

Wind tunnel test procedure and data acquisition

The tests measured the pressures on the roof of both the existing east stand and the proposed new west stand. The surroundings were modelled in detail to a radius of 360 m from the target site. The thin roof structures with internal pressure tubes were constructed by sandwiching thin plastic tubes between two skins of fibreglass moulded to the correct roof shape. The moulds for these skins were developed directly from the numerical shape files developed for the structural analysis model of the structure. A specific pressure tap system including brass tapping, 1m tubing and careful placement of tubing and pressure transducers was developed for the structure. This enabled the whole model to be rotated through the full wind direction range. The wind tunnel model is illustrated in figure 2. Each pressure tapping was referenced to a common static back-pressure which was obtained from the Pitot-static tube mounted in the roof of the wind tunnel.

Pressures were measured simultaneously at up to 512 locations around the model roofs, on both top and bottom surfaces, for all wind directions at 10 degree intervals at a sampling frequency of 400 Hz for a period of 80 s. These were then processed into time histories of pressure coefficients based on the mean wind speed at the reference height (equivalent to 50 m in full scale) within the wind tunnel. The analysis of the resulting pressure data was carried out by Compusoft Engineering Limited (CEL). The QBE stadium pressure data results were integrated directly with the structural analysis model.

Load case estimation from the wind tunnel data

The data obtained from the wind tunnel investigation have been normalised with reference to the mean dynamic wind pressure at a (full scale) reference height, $h = 50m$ and therefore incorporates several phenomena: net pressure coefficient; background loading fluctuation due to wind turbulence; and the variation in wind velocity throughout the boundary layer. For simplicity the "Aerodynamic Shape Factor", $C_{fig}(t)$ is used to reference the combined effects of all of these items, noting that they are not constant but vary with time.

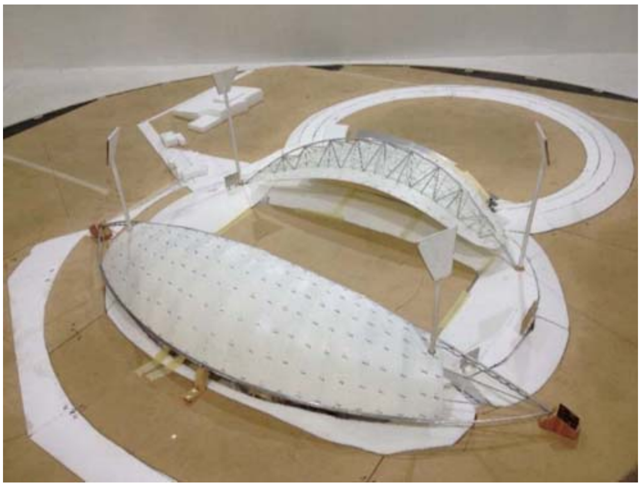

Figure 2. View of wind tunnel model with new canopy in foreground with the exterior pressure taps visible.

The derivation of the mean wind speed (denoted as $\bar{V}_{des,\theta}$ herein) has been determined by Equation 2.

$$\bar{V}_{des,\theta} = \frac{V_{des,\theta}}{1+g_v I_h} \quad (2)$$

where,

- $V_{des,\theta}$ is the directional gust wind speed.
- g_v is taken equal to 3.4 in accordance with AS/NZS1170.2:2011 [2].
- I_h is the turbulence intensity determined equal to 0.17 for $h = 50m$ in Terrain Category 2.5 (TC2.5).

The ULS design wind pressures which vary as a function of time (denoted as "$p_{des,\theta}(t)$" herein) have been determined as per Equation 3 below.

$$p_{des,\theta}(t) = 0.6\left[\bar{V}_{des,\theta}\right]^2 C_{fig}(t) C_{dyn} \quad (3)$$

where,

- $\bar{V}_{des,\theta}$ is the mean design wind speed for the ULS.
- $C_{fig}(t)$ refers to the aerodynamic shape factors
- C_{dyn} refers to the dynamic response factor

The pressure coefficients measured at all taps on the top of the roof structure were matched with pressure coefficients on the bottom surface to give differential pressures. These pressure coefficient differentials were mapped to the finite element geometry used to represent the fabric structure in SAP2000 [4], with the differential in closest proximity to the centroid of each finite element deemed to be representative of the conditions over that area.

Using the above formulae and the geometry given by the finite element model, the pressure profile at every point in time from all 36 measured wind directions could be calculated over the structure.

Load case estimation from the AS/NZS1170.2:2011 interpretation of pressure coefficients

While the scope of AS/NZS1170.2:2011 [2] precludes its use for a structure of this size, it is still useful to consider at a preliminary stage to gain an understanding of the possible load distribution that might be developed from wind tunnel testing.

The key parameters for developing the site specific wind velocity profile applicable for a structure of this size and importance level is still considered applicable though was developed considering actual structure height rather than wind tunnel velocity measurement height. Typical net pressure coefficients were interpreted from a variety of code examples. They were then integrated over the whole roof in the same manner as that for the wind tunnel derived data to enable direct comparison between the two.

The design process and selection of important load cases

The instantaneous wind pressure distribution over the extent of the proposed roof was recorded for 36 different directions of wind attack for 32,000 time steps (i.e. 80 seconds @ 400 Hz), which results in over one million potential wind load scenarios which could be considered. In order to reduce the data set to a manageable level, a set of metrics were identified which were anticipated would yield the most onerous conditions on the canopy as a whole.

The most onerous design scenario for the canopy structure as a system is likely to coincide with the peak base reactions. On the basis of this, a set of 12 different metrics were selected for use in the design which correspond to the minimum and maximum base reactions for forces and moments about axes parallel and perpendicular to the main axes of the canopy. The wind loads are applied to a semicontinuous flexible membrane, and this tends to ‘average out’ the local pressure peaks. As a result, the global measure of base shear is judged to be a good metric for peak structural load.

A number of other “metrics” were initially considered for evaluation. However, it was found that integration of overall base shear forces and moments provided adequate enveloping of load cases for design. Of the twelve possible load cases, ten unique cases were determined. In two cases, the instance of greatest force was coincident with the greatest moment about associated axes.

Each of the critical design cases was determined programmatically. For each time step, for all wind directions, the total reactions in the three major directions and the moments about the three major axes of the structure were integrated over the whole roof surface. A summary of these design cases is shown in table 1.

Case	Metric	Min/ Max	Wind Dir. (Degrees)	Metric Result
01	Force: X Dir.	Min	240	-594 kN
02		Max	130	669 kN
03	Force: Y Dirn	Min	90	-3,023 kN
04		Max	290	2,607 kN
05	Force: Z Dirn	Min	90	-8,111 kN
06		Max	310	1.919 kN
07	Moment: about X axis	Min	90	-236,981 kNm
08		Max	240	28,138 kNm
09	Moment: about Y axis	Min	130	-137,627 kNm
10		Max	90	116,546 kNm
11	Moment: about Z axis	Min	320	-96,386 kNm
12		Max	240	92,412 kNm

Table 1. Summary of critical load cases. Axes orientation is with reference to the structure – i.e. X axis is oriented along the structure length

The time step and wind direction for the maximum and minimum reactions and moments, could then be determined. An example of a selected “point in time” load case is presented in figure 3. This particular example (Case 03) represents maximum total applied horizontal shear orthogonal to the canopy front face on structure axis Y, for a wind angle due East (090°) selected from the complete time history for all directions. Each critical time step selected for further structural analysis was further assessed to ensure that it was not simply a numerical anomaly. For example, the Case03 metric presented in figure 3 is identified as having an increasing peak response to a gust travelling down the wind tunnel over a period of time (in this case a number of seconds).

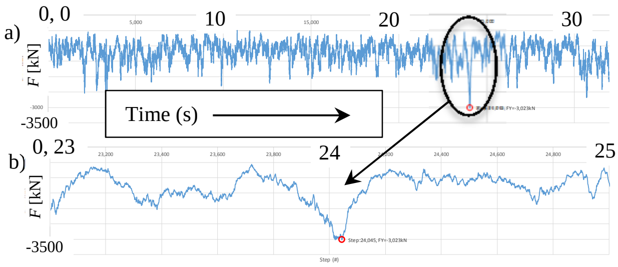

Figure 3. Selection of point in time load case for maximum total applied force (FY, kN) for Case 03; a) complete time history, b) 225 second window of record near peak.

Figure 4 displays a summary of the maximum horizontal base shears. For each time step and wind direction, the resultant x-y base shear was determined. A bounding line, showing the maximum base shear in every direction (quantised by 1 degree bins) is shown. The ten unique load cases that were selected for design analysis are also shown by the orange dots in figure 4. The critical load case presented in figure 3 (Case 03) can be identified as the plotted point at cardinal angle 120° (approximately 3,000 kN). Additionally, the base shears developed from estimation of possible pressure coefficients derived from AS/NZS1170.2:2011 [2] described above is also shown as yellow dots for comparison.

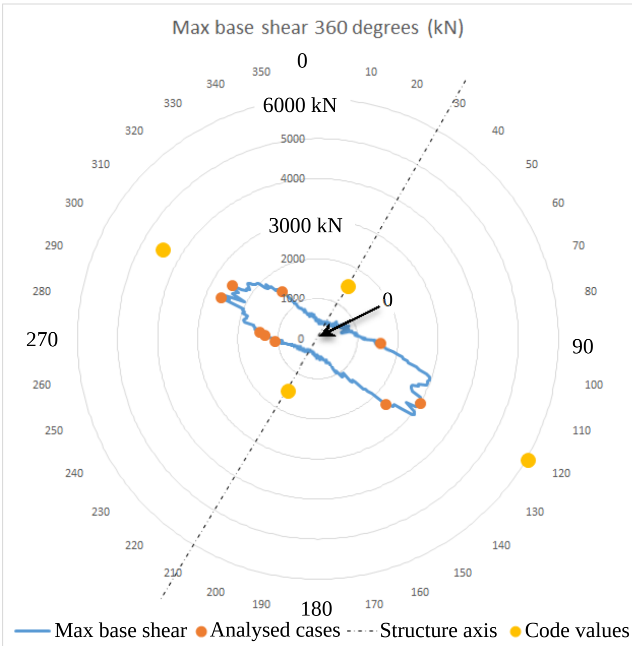

Figure 4. Peak base shear in any given direction developed from the wind tunnel for both the analysed cases and the preliminary code values.

Plotting graphical comparisons of wind tunnel metric tests on polar plots such as that shown in figure 4 proved to be a useful tool in further validating the selection process of critical load cases for structural design. For example there are additional load cases that are identified "either side" of the Case 3 metric that can be considered for further inclusion in the structural validation model that will further improve the certainty that the load cases chosen envelope the likely maximum actions throughout the structure.

Discussion of efficiencies in the design resulting from this approach

It is apparent from comparison between the wind tunnel derived base shears and the interpreted code derived base shears (although not strictly applicable) that the coarse derivation of surface pressures over a structure of this magnitude results in a significant increase in overall base reactions and hence internal member actions. In round numbers there is approximately a factor of 2 in magnitude difference between the two approaches which would seem appropriate on reflection.

When reviewing the original literature that forms the basis for each pressure coefficient derivation in the code (for example, Letchford and Killen [3], it is likely this is because the code values have necessarily been developed by enveloping overall maxima of pressure coefficients. Developing final load cases derived from actual project wind tunnel testing, particularly for membrane roofs of this magnitude, allows for more accurate determination of overall maximum load cases that account for correlated pressure variations.

Structural analysis procedure of the structure

A three dimensional finite element analysis model was developed in SAP2000 v17.3.0 (CSI America, 2015, [4]) for use in determining the element forces and deformations resulting from the prescribed loading scenarios outlined above. The analysis model is presented graphically in figure 5.

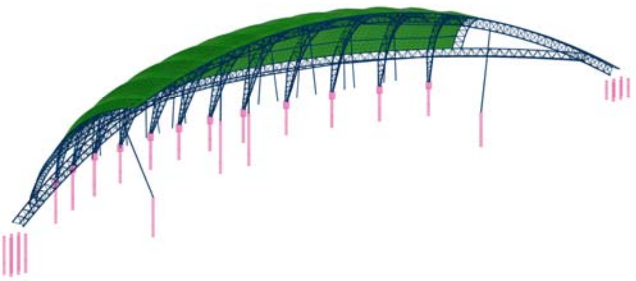

Figure 5. Overall view of the west stand canopy structural analysis model

The tensioned membrane surface ("fabric") is modelled to account for its geometric non-linear behaviour while the ground piles have also been modelled with representative non-linear "soil springs" to model the anticipated plasticity of the upper layers of soil. This ensures that variable foundation flexibility and the resulting load distribution is accounted for directly in the analysis. Additionally, structural second order effects (for example Euler buckling etc.) have also been accounted for directly in the analysis model.

Pressure distributions were applied to the membrane finite elements as described above for each of the ten representative load cases. The point in time wind pressure distribution presented previously is illustrated over the entire roof in figure 6.

In addition to the wind load pressures on the membrane surface, aerodynamic drag as a result of the exposed members which support the canopy have also been added to the analysis model in accordance with AS/NZS1170.2:2011.

Each of the ten identified critical load cases was run as an equivalent static load pattern applied in multiple non-linear load cases combined with other actions including dead loads, membrane prestress loads etc. Assessments of serviceability, fatigue etc. were carried out by first scaling the load patterns to the appropriate wind speed magnitude.

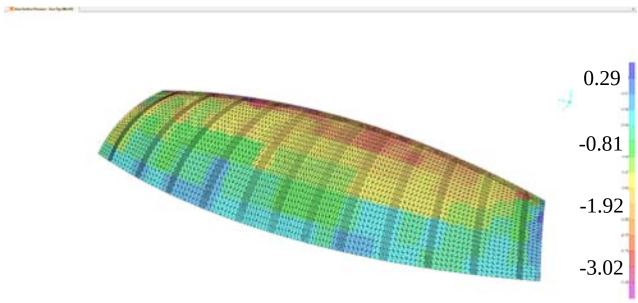

Figure 6. Example wind pressure distribution (Case03)

Design considerations including the optimisation and rationalisation of structural steel member sizes, has also taken place directly within the analytical model with automated design calculations also carried out external to the modelling software.

Conclusions

A technique for processing wind tunnel results on large roofs such that they can be directly integrated into the structural analytical model as equivalent static load cases is presented. Because of the extent of the roof and the structural behaviour of the membrane cladding this technique enabled the full suite of pressure tap time histories developed from wind tunnel testing to be effectively reduced to a series of critical load cases which are tested by identifying key metrics such as maximum base shears or moments. Comparisons of the results of these key metrics with possible interpretations of code pressure coefficients show that being able to integrate the spatial correlation of pressures over an entire roof surface can lead to a more efficient structural design to carry wind loads.

Acknowledgements

The authors wish to acknowledge the use of the rendered image in figure 2 which was developed by the Project Architect, Copeland Associates Architects. The authors also thank Simon Corkery and Ben Goodwin for building the wind tunnel models.

References

[1] Australasian Wind Engineering Society (AWES). Quality assurance manual on environment wind studies (2001). AWES-QAM-1-2001

[2] AS/NZS 1170.2. (2011). (Incorporating Amendment Nos 1, 2 and 3) Structural design actions, Part 2: Wind Actions. Standards New Zealand, Wellington, New Zealand, 104p.

[3] Letchford, C.W., and Killen, G.P. (2002) Equivalent static wind loads for cantilever grandstand roofs. Engineering Structures, 24:207-217

[4] CSI America. (2015) Computer program SAP2000 v17. Computers and Structures Inc., Walnut Creek, California.