Wind Actions on Unusual Structures

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1. Introduction

The increasing complexity of modern day structures presents constant challenges in the determination of wind loads on increasingly twisted, contorted and complex building forms. Very often these complex shaped buildings tend to have equally complex dynamic properties, which require special treatment from a wind engineering perspective.

A number of very significant advances have been made over the past 20years and particularly over the past decade in enabling more accurate estimations of the dynamic responses of these complex building forms to extreme wind actions. Equally important, these new techniques enable more accurate determinations of the critical load cases and critical load distributions as well as other information not previously available to the structural engineer. The outcome of all this is that we are now able to achieve more efficient and at the same time more robust designs.

This paper presents an overview of some of the latest techniques available in dealing with various types of complexities in the building forms.

A number of case studies are presented to illustrate the techniques presented in this paper.

2. Outline of the various techniques and their application

2.1 Structural loads on tall buildings with coupled modes

Many modern tall buildings have dynamic modes that involve simultaneous sway and twist motions. This often results from differences between the centre of mass and centre of stiffness (shear centre) of the cross-section. Techniques are now available to process the outputs of the High Frequency Force Balance (HFFB) to make reasonable predictions of the resonant contributions from the coupled modes of tall buildings.
The best way is by directly forming the time histories of the generalized forces for each mode, by weighting the time histories of the measured base moments, and then calculating the spectral densities from the new time series (Holmes, Rofail and Aurelius, 2003).
In cases where two or more buildings have a connection at the lower levels, it is common to see that more than 3 modes need to be considered in the calculation. This can be carried out by extending the technique presented in Holmes, Rofail and Aurelius (2003) to more than 3 modes.

2.2 Structural loads on linked tall building structures

The current trend to produce striking and innovative building forms increasingly leads to the design of tall buildings with linkages between what are essentially separate towers but linked structurally via a common floor at one or more points along the height of the linked structures. The question is often raised as to how to model the effect of a rigid linkage as opposed to a simply supported bridge on rollers. Rofail and Holmes (2007) have developed a technique as an extension of the technique published by Xie and Irwin (1998) and in the form of a direct extension of the methodology for coupled modes Holmes, Rofail and Aurelius (2003).

This methodology has the advantage in that it can account for the following:
- The effect of the linkage on the mean and background base moments
- The critical load cases, considering the combined effect of the mean, background and resonant base moments.
- The distribution of the resonant response across the substructures
- The effect of the variation in the mass distribution and mode shape between the individual substructures
- The methodology can be expanded further to account for other effects such as shear forces acting on the linkages

The effect of the linkage on the mean and quasi-static (background) base moments is derived from the measured (unlinked) base moments using an influence coefficient matrix. The influence coefficient matrix, where $\alpha_{lx,kx}$ is the influence coefficient for moments about the x-axis at the base of substructure k in the prototype due to a unit force applied at an assumed height of the line of action of the overall wind force of tower *l*. These values are to be obtained from the structural engineer's finite element model of the structure, in the form of a 3k x 3k matrix. Thus, the time histories of the recorded base moments from the isolated models in the wind tunnel, denoted by M^*, can be used to estimate mean and quasi-static base moments for the prototype (linked) buildings as follows, for a group of three buildings :

$$M_{11}(t) + M_{12}(t) + M_{13}(t) = \alpha_{11} \,.\, M_{11}^*(t) + \alpha_{12} \,.\, M_{22}^*(t) + \alpha_{13} \,.\, M_{33}^*(t)$$
$$M_{21}(t) + M_{22}(t) + M_{23}(t) = \alpha_{21} \,.\, M_{11}^*(t) + \alpha_{22} \,.\, M_{22}^*(t) + \alpha_{23} \,.\, M_{33}^*(t)$$
$$M_{31}(t) + M_{32}(t) + M_{33}(t) = \alpha_{31} \,.\, M_{11}^*(t) + \alpha_{32} \,.\, M_{22}^*(t) + \alpha_{33} \,.\, M_{33}^*(t)$$

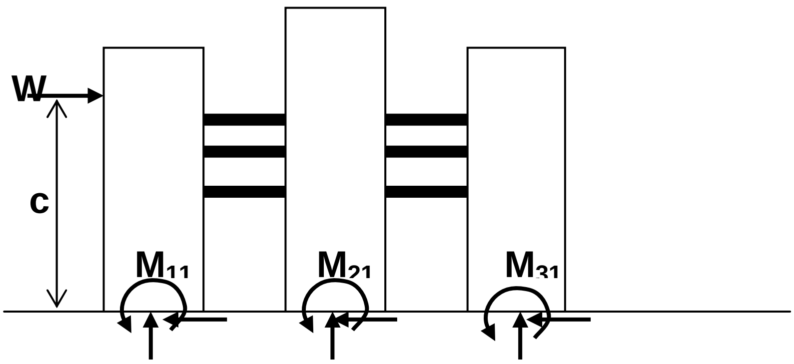

Figure 1. Hypothetical group of linked tower buildings

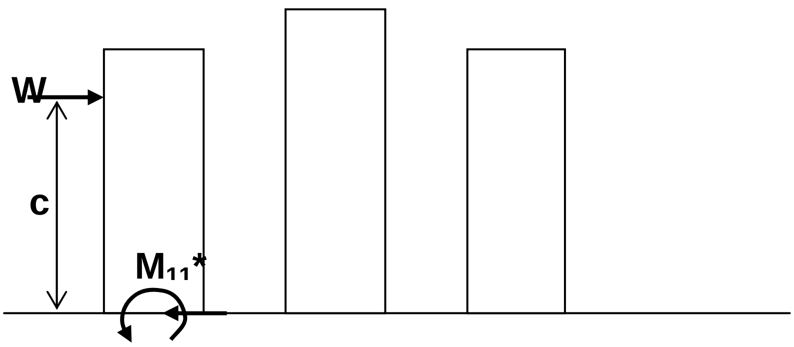

Figure 2. Set-up of the linked tower buildings in the wind tunnel

2.3 Wind loads on complex large and long-span roof structures

In the case of some large roof structures, it becomes virtually impossible to model their dynamic properties physically. This calls for pressure integration techniques such as the common direct integration method or in the case of the stiffer types of roof structures, the pressure correlation method.

It turns out that these techniques have an added benefit in that the resulting loads for some structures tend to be substantially lower than was previously thought due to the effect of low correlation of the peak pressures across the structure. These techniques also provide far more information to the structural engineer than was previously possible. This additional information includes the ability to monitor the load effects at any desired number of elements as well as specific information regarding the most critical load distribution for each maximum or minimum critical load effect.

The pressure correlation technique was developed by Dr Michael Kasperski (Germany) and Dr John Holmes (Australia) about 15years ago (Kaspeski and Niemann, 1992 and Holmes, 1992). This technique is applicable to structures where the dynamic component of the structural response predominantly consists of the background component of the response and there is little chance of aerodynamic damping effects. Structures that fit into this description are stadium and long-span roofs as well as any canopy or large roof-fin structure. This technique is also applicable to quasi-static structures, where the form is relatively complex such that the use of wind loading standards is not possible.

This technique requires the use of the pressure integration technique. The entire surface of the structure is pressure-tapped and the pressure signals measured simultaneously (in real-time). The importance of having the pressures being read in real-time (minimal phase lag between the various pressure taps) stems from the need to accurately determine the following:

- a pressure correlation matrix, representing the relationship between the pressure signal at different areas of the building surface
- the various load effects, which are determined by area-weighting the pressures from the different areas (pressure integration)

It is important that the structural engineer carefully selects the various load effects that are to be monitored as these will determine the load cases that would be used to design the rest of the structural elements. Hence the load effects need to be for key structural members from different parts of the structure that are sensitive to pressures from the different areas of the building surface. In some cases it would also be helpful to determine different types of load effects such as displacement, shear force, axial force, bending moments (about different axes) for the same member.

For quasi-steady structures, the only input required by the structural engineer is a pressure correlation matrix. A stadium or long-span roof structure is considered quasi-steady if the first natural frequency is greater than 0.8Hz, whereas a vertical structure such as roof fin is considered quasi-steady is the first natural frequency is greater than 1.2Hz. A pressure correlation matrix determined by first dividing the envelope of the structure into a number of patches. There are typically between 30 and 60 patch areas for a stadium roof structure. The patch roof is divided into patches with consideration of the following:

- Areas that would be expected to have different range of pressures for the same wind direction (such as due to surface discontinuities, layout of or for aerodynamic reasons, such as wall or roof edges).

- Areas divided by beams or columns and therefore are likely to result in different reaction either side of the beam or column for the same pressure range.

Each patch consists of a number of pressure taps that are area averaged.

A pressure correlation matrix is determined by applying a unit pressure (such as 1kPa) normal to the surface of one patch and then read out the reactions for each of the key load effects that are being monitored. This would provide the data for one row of the matrix. The process is then repeated for the next patch area and so on..

For structures that are likely to have a significant resonant component, the following additional inputs are required from the structural engineer for the various modes that have natural frequencies less than 0.8Hz in the case of a stadium or long-span roof or 1.2Hz in the case of vertical structures such as roof fins:

- the natural frequency and mode-shape for each of the applicable modes of vibration. The mode-shape is normally expressed in the form of the displacement at the middle of each patch, normal to the patch surface.
- The estimated mass and area of each patch

This additional information is required to determine a generalised force for each of the load cases being monitored. This generalised force is then analysed spectrally to determine the prototype response of the structure. Usually the resonant response will comprise no more than 10 to 20% of the peak values of critical load effects and this contribution can be calculated separately and added to the fluctuating background response using a 'root-sum-of squares approach'. The effective static load distribution corresponding to each peak load effect can then be scaled up to match the recalculated peak load effect. When the resonant response is more significant, the inertial loading from the resonant component is correctly combined with that from the direct wind pressure, by a weighted summation (Holmes, 2002).

The advantages of this technique can be summed up as follows:

- no limitation to the complexity of the structural behaviour as there is no need to model such structures
- More accurate information regarding the critical load combinations between the various patches taking into account the effect of correlation of pressures across the structure
- an ability to accurately determine any number of load effects directly, without having to make assumptions regarding pressure distributions
- more accurate than traditional aeroelastic model techniques, which are limited in their ability to accurately model the dynamic behaviour of the structure and provide little guidance in relation to critical load cases
- the technique is more cost effective and requires less time that the traditional technique, provided that the computers can be effectively mobilized to perform the extensive data analysis required

2.4 Wind loads on special structures

Each special structure often calls for a special approach to suit its geometry and the load effects that need to be determined. Two cases of special structures are presented in this paper.

3. Case studies

3.1 Aurora Tower, 420 Queen St, Brisbane

This example illustrates the differences in results obtained from the results of processing HFFB data with and without the technique for tall buildings having coupled modes. The example of tall building presented here has significant coupling in the first and third modes of vibration but no coupling in the second mode. The proposed building was approximately 200metres in height with the cross section shown in Figure 3. Due to an eccentricity in the lift core, the building has significant coupling between sway in the x-direction and twist in both Modes 1 and 3. Wind-tunnel tests were performed on a tall building model (scale 1:250) placed in Windtech's boundary-layer wind tunnel with a blockage-tolerant test section. The tower section of the model was attached to a high-frequency base balance. The axis convention adopted for the tests is shown in Figure 3. Time histories and spectral densities of generalized forces for three modes were calculated, using the mode shape corrections for sway and twist described in Holmes, Rofail and Aurelius (2003). The resonant response contributions to the total base moments, and the accelerations at the top of the building, were subsequently calculated using standard procedures.

Figures 4a to 4d show the variation in the results between the HFFB technique, as presented in Holmes, Rofail and Aurelius (2003) versus the HFFB technique where coupling effects are ignored. The results are presented in form of base moment coefficient as a function of wind direction for the maximum peak, mean and minimum peak base moments. It can be seen that values ignoring the effect of coupling are generally greater than those derived from the method presented by Holmes, Rofail and Aurelius (2003). This can be attributed to negative correlations between the moments M_y and M_z.

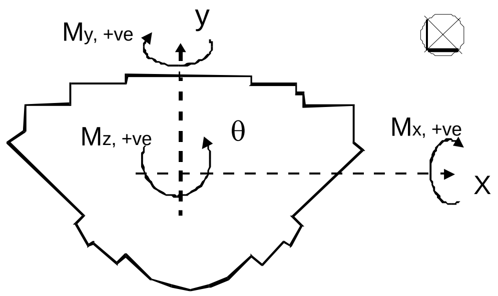

Figure 3. Building cross section and axis convention

Table 1 gives the comparative results of some final peak response predictions for the building. Coefficients of the base moments are defined in Equation (1). h is the building height, b is the maximum breadth, and $\bar{U}_h$ is the mean wind speed at the top of the building.

$$C_{Mx} = \frac{M_x}{\frac{1}{2}\rho \bar{U}_h^{\,2} b h^2} \qquad C_{My} = \frac{M_y}{\frac{1}{2}\rho \bar{U}_h^{\,2} b h^2} \qquad C_{Mz} = \frac{M_z}{\frac{1}{2}\rho \bar{U}_h^{\,2} b^2 h} \tag{1}$$

The maximum coefficient of the base bending moment, M_x, which is not affected by the coupling, is similar whether or not coupling effects are included in the analysis. However, ignoring the effect of coupling gives significant overestimates for M_y and M_z due to neglect of the cross-coupling terms in calculating the resonant contributions, as discussed previously. In this case, the resultant acceleration shows close agreement between the two approaches. Note that it should not be implied from this example that loads will always be lower when the effect of coupling is accounted for.

Table 1. Comparison of response predictions for a building with two coupled modes

	5yr. Std. Devn. Acceleration (mg)	Coefft. of peak Base bending moment M_x	Coefft. of peak Base bending moment M_y	Coefft. of peak Base torque M_z
Including effect of coupling	2.7	1.33	0.58	0.31
Uncoupled	2.8	1.33	0.70	0.41

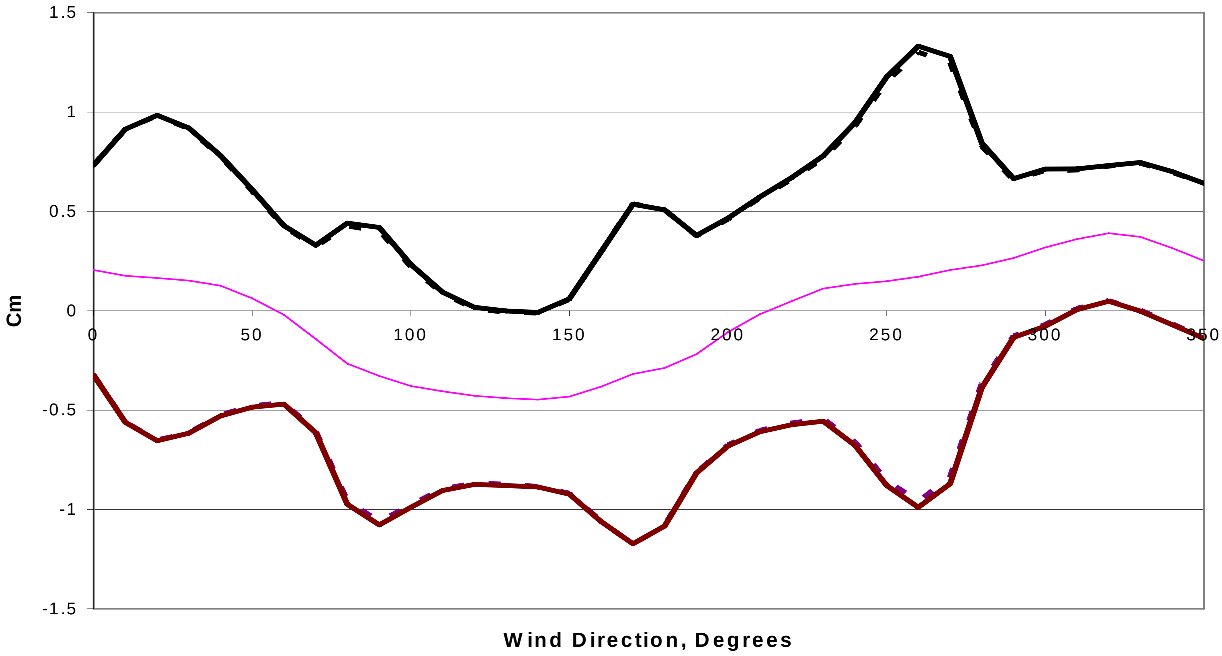

Figure 4a. Comparison of the About X base moment coefficients for the cases where coupling effects are ignored (dashed lines) and with the effect of coupling (solid lines).

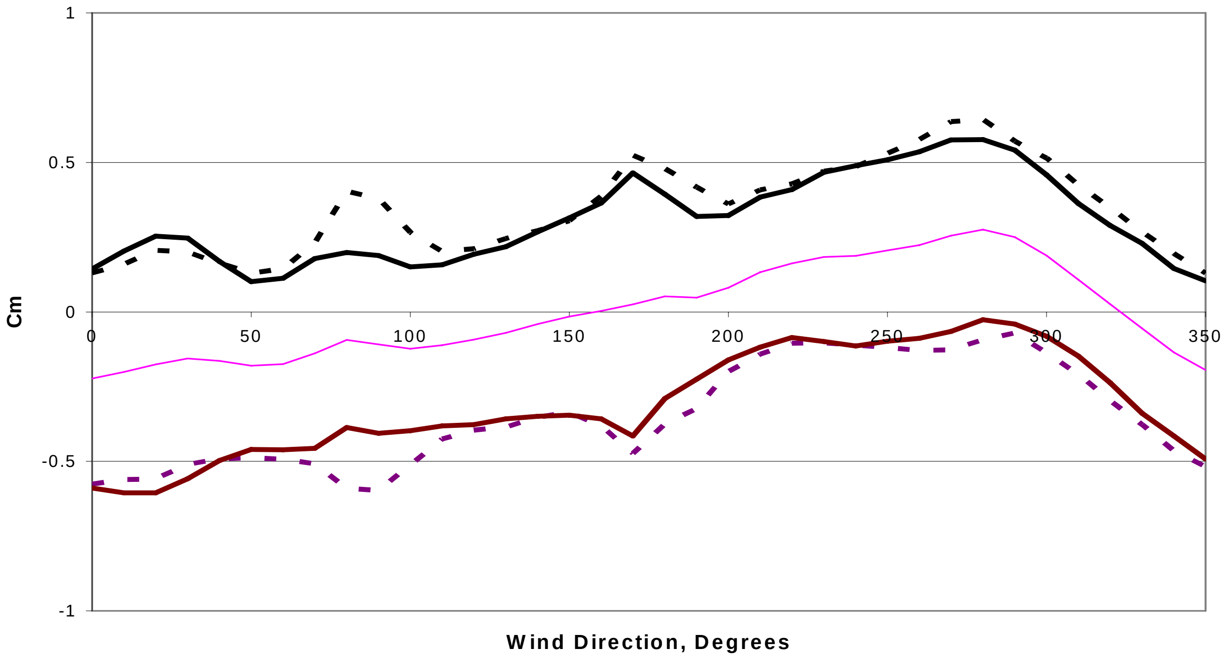

Figure 4b. Comparison of the About Y base moment coefficients for the cases where coupling effects are ignored (dashed lines) and with the effect of coupling (solid lines).

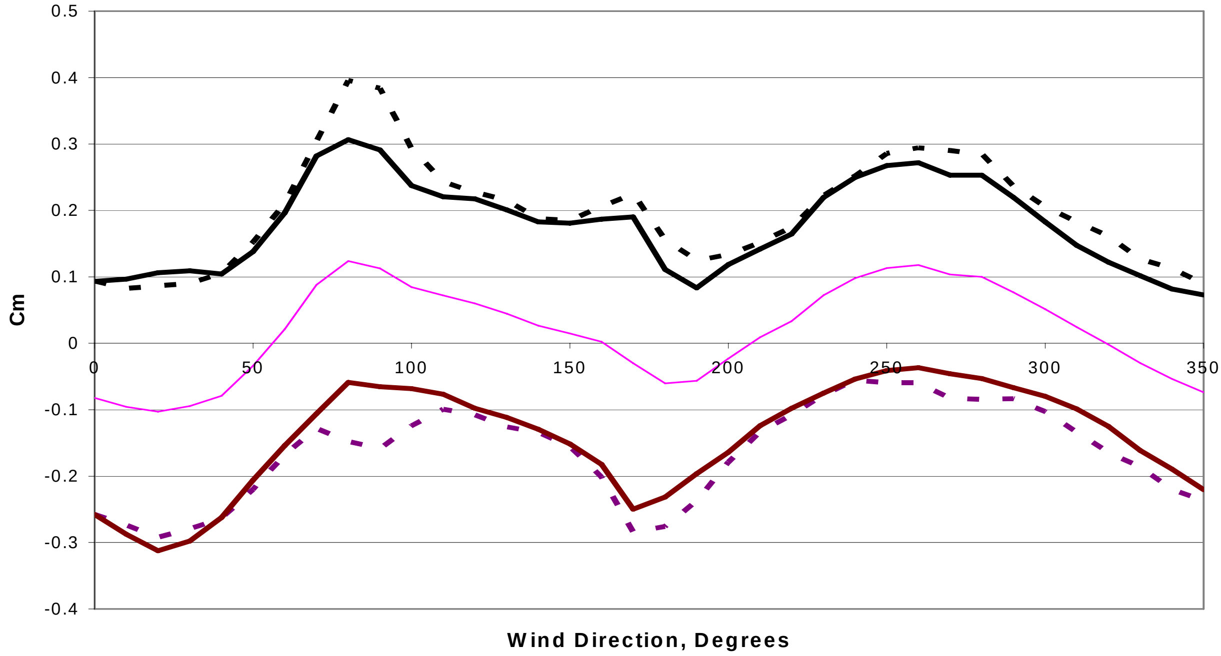

Figure 4c. Comparison of the About Z base moment coefficients (torsion) for the cases where coupling effects are ignored (dashed lines) and with the effect of coupling (solid lines).

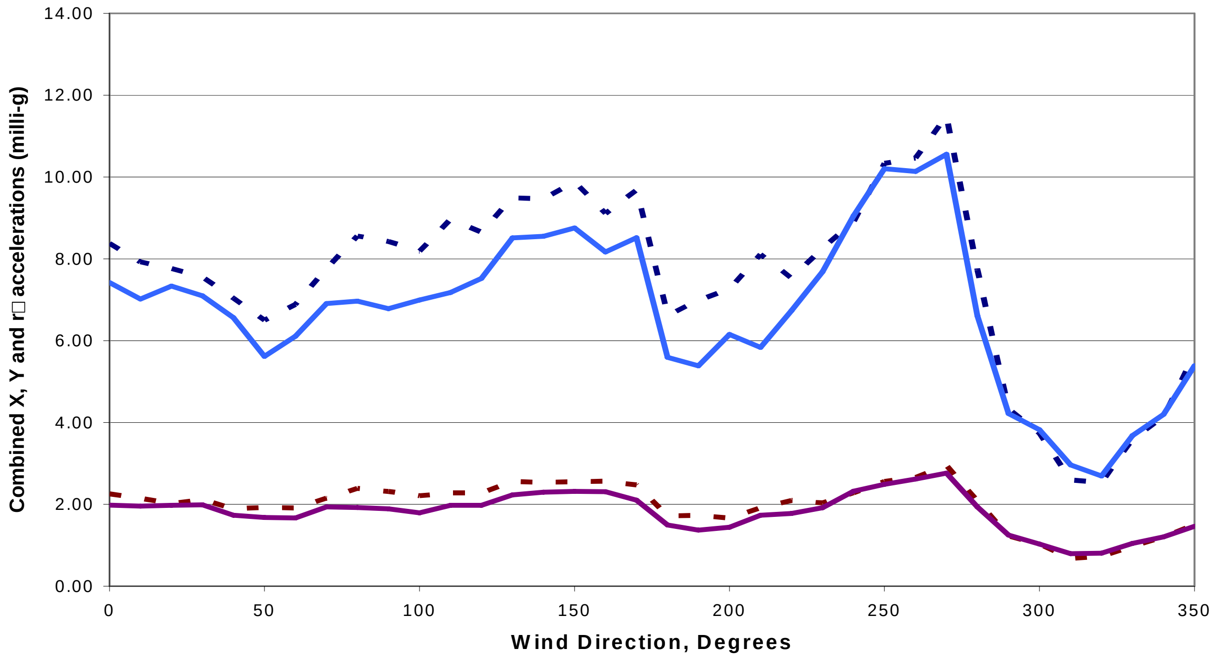

Figure 4d. Comparison of the resultant 5yr standard deviation and 10yr peak accelerations for he cases where coupling effects are ignored (dashed lines) and with the effect of coupling (solid lines).

3.2 One Shenton Way, Singapore

This two tower development in Singapore is one of several projects since 2004, where Windtech Consultants have applied the technique for linked tower structures by Rofail and Holmes (2007). These two towers incorporate rigid links at 3 levels up the towers, in addition to a common podium. This methodology described in this paper was applied to this project. The taller of the two towers, Tower A, is approximately 230m high. Figure 5 shows a model of these linked towers in Windtech's wind tunnel facility.

An influence matrix was prepared based on outputs of reactions at the base of the two tower buildings provided by the structural engineers based on the line of action of the wind force acting at 2/3 of the building height for each of the two towers. This height was later confirmed as closely representing the line of action of the wind force.

The effect of the linkage tends to result in the critical wind directions (and the corresponding load cases) becoming governed by the same wind directions. This can be seen by comparing the results in Figure 6 with those of Figure 7.

It can also be seen that the linkage has resulted in a reduction in the dynamic component of the response of larger tower (Tower A), which has the largest dynamic response. At the same time the linkage has resulted in an increase in the dynamic response of smaller tower, Tower B.

This case study highlights the benefits of this technique in not only determining the effect of the linkage but in assisting with the derivation of the critical load cases as well as providing an accurate equivalent static load distribution.

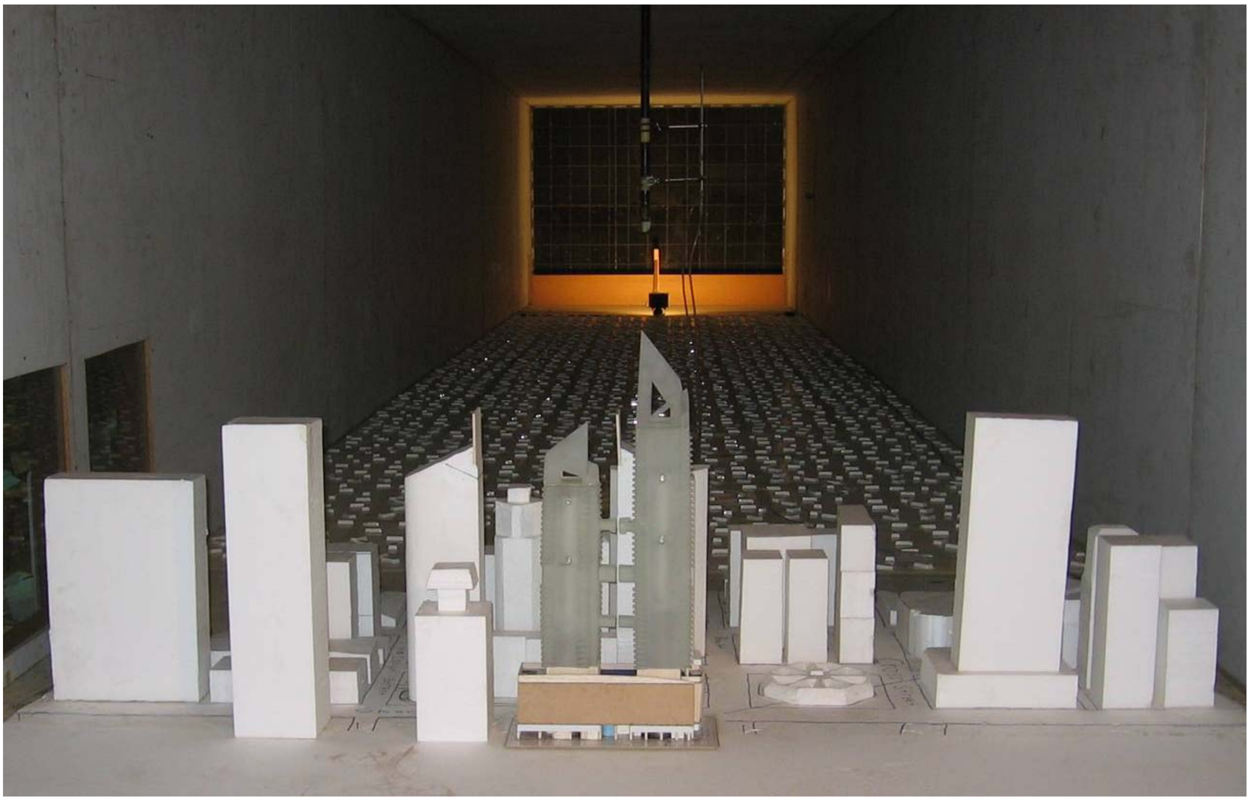

Figure 5. Model Set-up in Windtech's wind tunnel facility for One Shenton Way, Singapore

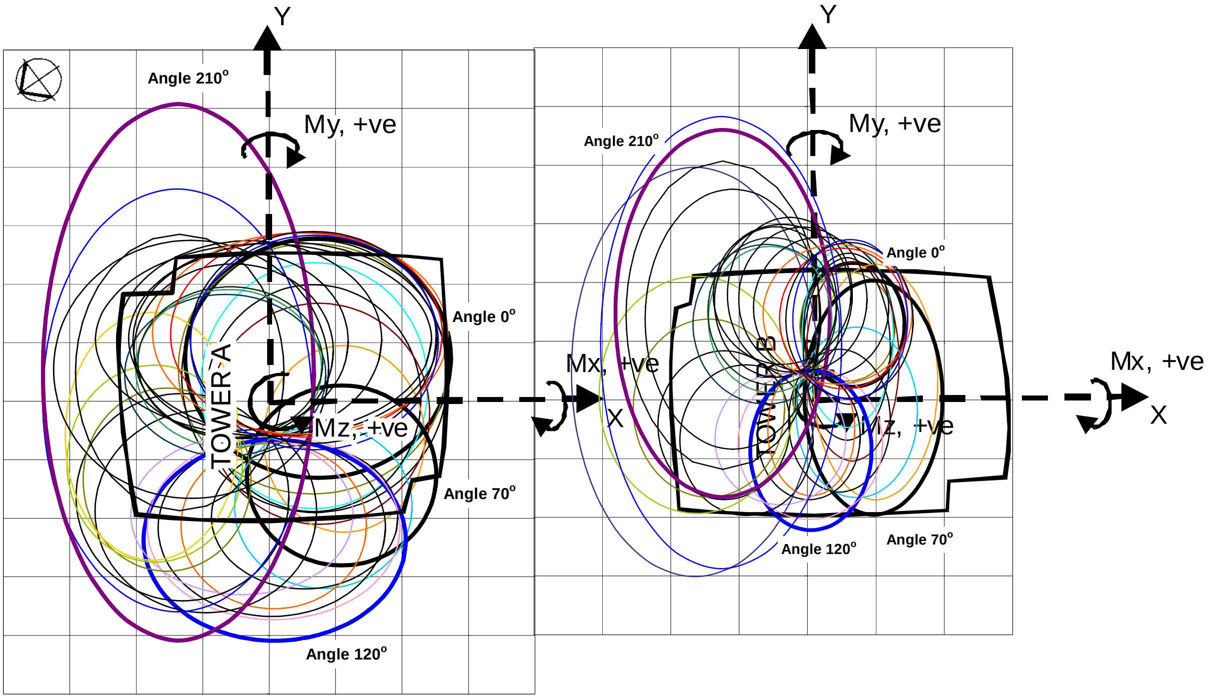

Figure 6. Relative Translational Displacements at the aerodynamic centre near the top of the two buildings for the various wind directions without the effect of the linkage.

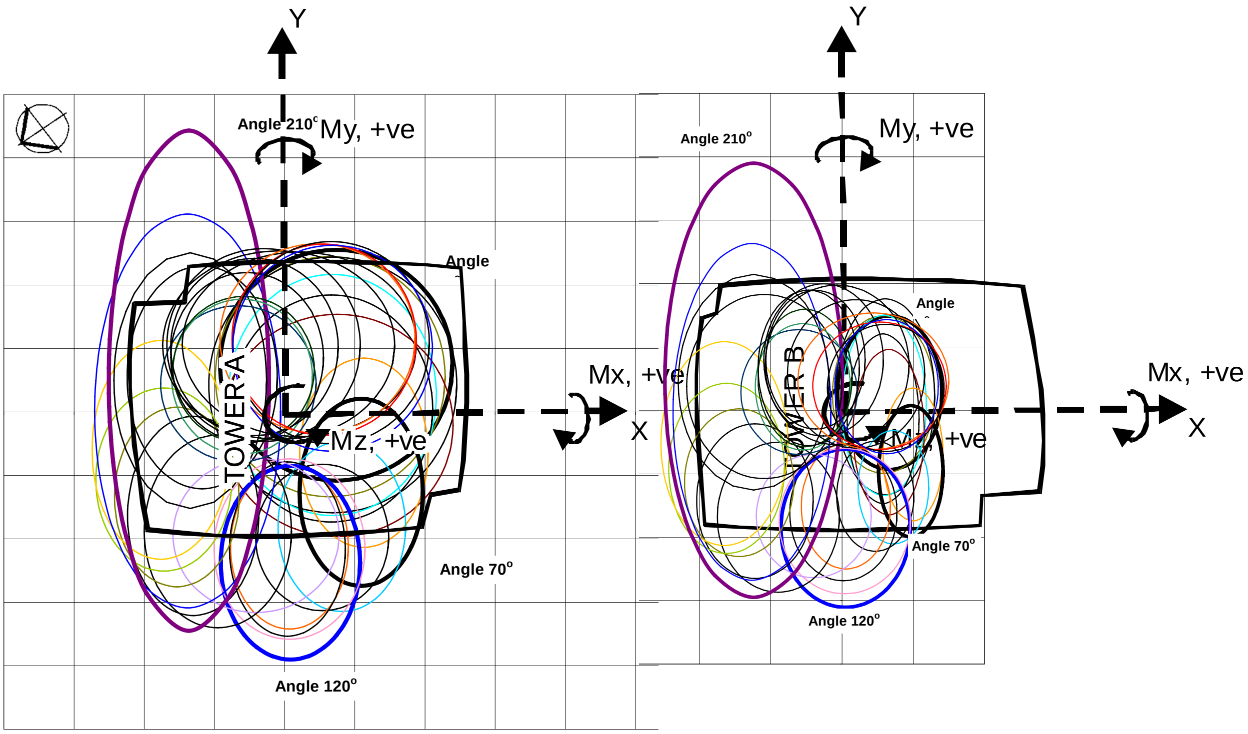

Figure 7. Relative translational displacements at the aerodynamic centre near the top of the two buildings for the various wind directions with the effect of the linkage.

3.3 Gold Coast Stadium

A study of the wind loads on the structure and cladding for this stadium was commissioned by SKM and Tensys as part of a value engineering exercise to meet tight cost constraints. A wind tunnel study was carried out on a 1:150 scale model of the stadium. A model scale as large as 1:150 was required to ensure correct simulation of the flow regime around the curved surface of the stadium roof, due to Reynolds Number effects. A photograph of the model in the wind tunnel is shown in Figure 8.

The pressure correlation technique was used in analysing the wind loads on the structure. In addition, a number of options were investigated to alter the roof edge configuration with a view to further reduce the wind loads. It was later decided not to modify the roof edge configuration. The entire envelope of the roof structure was divided into 64 panels. Each panel was pressure tapped on both sides using between 4 and 8 pressure taps per panel. Pressures on the entire roof structure were measured simultaneously to derive a correlation matrix as well as to determine time series of the generalised forces to determine the peak responses. A total of 15 load effects were monitored for maximum and minimum responses. These include 6 bending reactions, 6 axial loads, 2 tip deflections and 1 pile reaction. The corresponding 30 critical load combinations were rationalised to 24 load cases. The maximum 100year return patch pressure was approximately 2kPa represents a substantial reduction over the estimate by AS/NZS1170.2:2002. Furthermore, the effect of the correlation study is such that it significantly reduces the incidence of large pressures on more than a couple of patches at a time in a given load combination.

In the study of the wind loads on the cladding, area averaging was used as the cladding consisted of a tensile fabric. This approach resulted in reductions in the design loads for the cladding of the order of 20% over the measured point pressures. The maximum 10year return panel pressure was around 2kPa, which represents more than 40% reduction over the estimates in AS/NZS1170.2:2002, even with the effect of the area reduction factor of 0.80. This is significant reduction in the loads is largely attributable to the curved, aerodynamic form of the roof.

Figure 8. The 1:150 scale model of the Gold Coast Stadium in Windtech's Boundary Layer Wind Tunnel.

3.4 Nanjing Stadium

This was the main stadium used for the 2006 China Games. This stadium roof structure consists of a perimeter truss support as well as cable ties supporting the end of the cantilever by means of overhanging arches (see Figure 9). This relatively complex structure would be virtually impossible to model using the traditional aeroelastic modeling technique. Windtech Consultants were engaged by SKM to perform the wind tunnel study for this structure.

An accurate 1:300 scale perspex model of the structure was prepared using the computer-aided rapid prototyping process, as shown in Figure 10. This scale is more than sufficient for modeling of Reynold number effects for this structure. The entire envelope of the structure, including the overhanging arches was divided into 64 panels as shown in Figure 10. A total of 512 pressure taps were distributed over the entire surface of the roof structure and simultaneous pressure measurements were performed over the entire structure. A total of 15 load effects were monitored for one quarter of the structure to take advantage of the double symmetry. The load effects consist of a mid-span displacement, 5 moment reactions and 9 axial loads.

The maximum and minimum load effects resulted in 30 critical load cases, which were rationalised down to 8 load cases as shown in Figure 11. Note the difference between the instantaneous peak patch pressures required based on the pressure correlation technique in comparison to the maximum and minimum pressures derived from a simple discretised area-averaged patch pressures indicated as solid lines in Figure 11.

Figure 9. A perspective image of the Nanjing Stadium

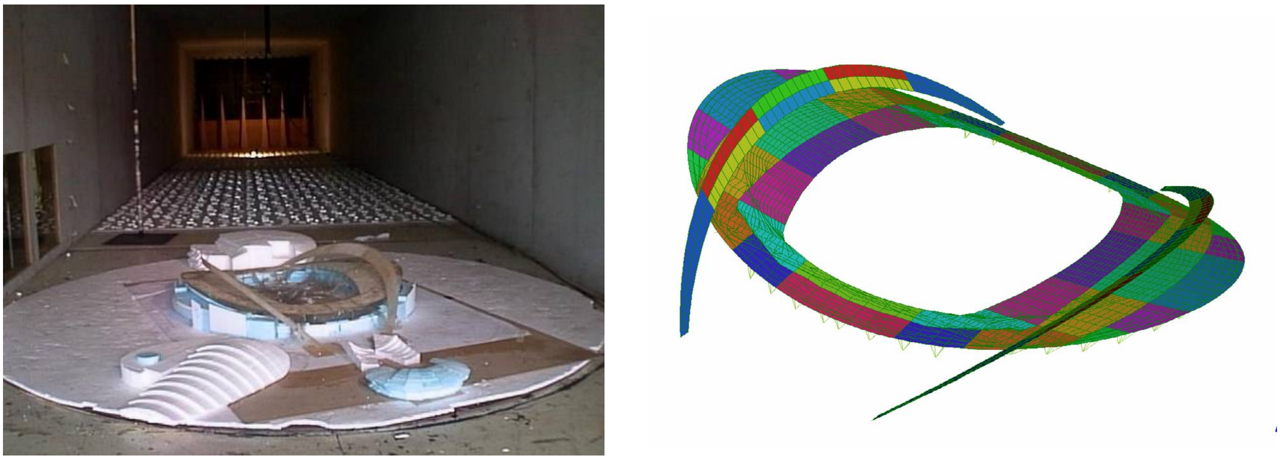

Figure 10. The 1:300 scale model (left) and a diagram showing the different patch areas (right)

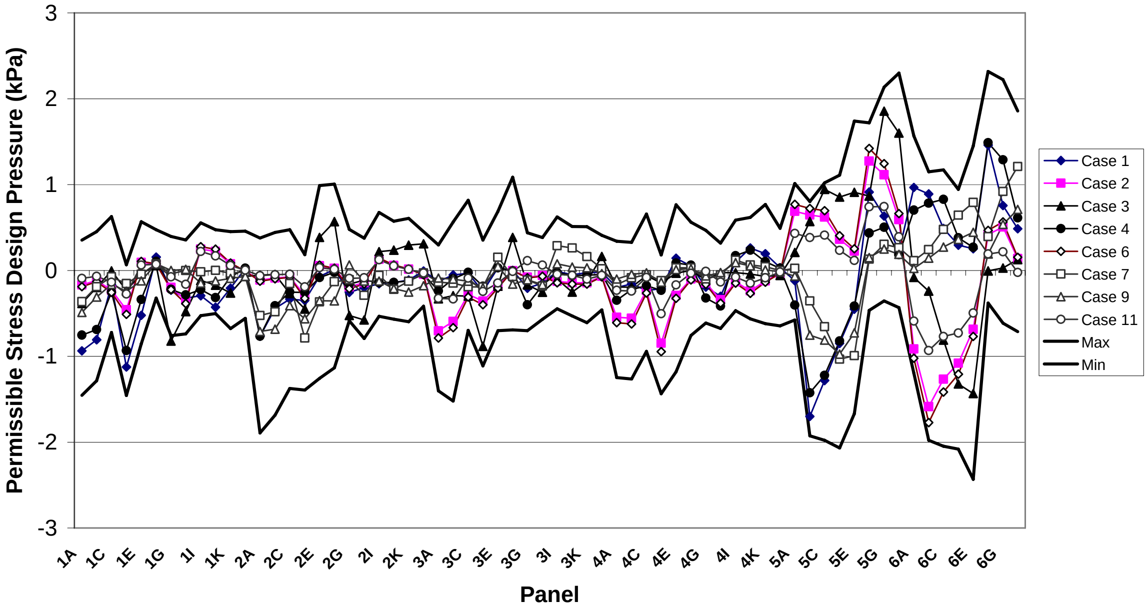

Figure 11. Discretised panel pressures (Max and Min) versus the 8 load cases for Nanjing Stadium

The natural frequencies for the first 5 modes were 0.75Hz, 0.77Hz, 0.79Hz, 0.81Hz, 0.95Hz. Hence in addition to the pressure correlation technique, a pressure integration method was applied to analyse the resonant response from the first four modes of vibration. For this structure, modes higher than the fourth mode would have a negligible contribution to the resonant response. The results of this analysis indicate that the resonant component of the response has a maximum additional contribution of 10% to the total dynamic response.

3.5 Cauldron for the Asian Games 2006, Doha

This complex structure consists of 2 slim rotating rings that revolve around a main ring located on a 25m high shaft. This is a temporary structure and is intended to be in place for about 4 months from December 2006. A 1:50 scale model of the structure was prepared using the rapid-prototyping technique, as shown in Figure 12. This model was placed within a 1:50 scale model of the Khalifa Stadium.

A model was also configured with pressure taps within each ring and over the supporting shaft to enable the use of the pressure correlation technique. The model was designed such that the rings can be rotated to simulate the effect of different stages of the rings' revolution relative to each other and for the erection mode configuration. A total of five ring configurations were tested.

Before commencement of testing, a study was carried out using a 1:300 scale model of the sports precinct to determine the effect of the surrounding structures on the upstream velocity and turbulence intensity profile for wind incident from different wind directions. These wind profiles were replicated at 1:50 scale to be used for this study. Also an extensive analysis was carried out of the wind climate for Doha. This included a seasonal extreme wind speed analysis to correspond with the time of year when this structure is to be in place. Fortunately, the December to April season co-incided with the most benign season for extreme winds in Doha and therefore a lower design wind speed was adopted.

For each of the five ring configurations, two wind tunnel testing techniques were performed:

- high-frequency force balance
- pressure correlation technique

The high-frequency force balance technique was important as a check on the results of the pressure correlation technique. The need for the pressure correlation technique stems from the need to determine the relative displacements of the ring elements, as they need to operate within very strict limits to avoid the rings colliding into each other (there is only a few centimeters gap between them). The pressure correlation technique is also useful in providing a more accurate set of equivalent static loads for such an unusual structure.

The results showed that the value of drag from the pressure correlation technique was about 20 percent higher than those obtained using the high frequency force balance. This is due to complexity of the form of the structure and the curved form. Nevertheless it provided an acceptable level of confidence in the predictions.

Figure 12. The 1:50 scale model of the Cauldron for the 2006 Asian Games, Doha. Left: On the force balance before inclusion of the Khalifa Stadium section model. Top Right: Within a section of the Khalifa Stadium and using the pressure correlation technique.

3.6 LED Screen for the Asian Games 2006, Doha

Windtech was engaged to investigate the wind drag forces on this 150m long and 58m high porous screen. This screen consists of a matrix of LED screens connected via 25mm diameter vertical cables. The screen was used for the opening and closing ceremonies of the 2006 Asian Games in Doha. The aim is to determine the amount of wind drag for which to design the supporting structural frame, which consists of a series of 8 vertical space trusses.

The modeling of porous screens requires special care. Where a screen consists of elements of circular section Reynolds's Number effects can be significant. The Reynolds Number is a non-dimensional number that is proportional to wind speed and scale. The flow regime around bluff bodies varies with different Reynolds Numbers. Flow around Circular Cylinders is particularly sensitive to Reynolds Number. In some cases the only way to accurately model the drag of such screens is to artificially enlarge the diameter of the cylindrical elements such that the flow around them in the wind tunnel is operating under the same Reynolds Number Regime.

For net-type screens with a relatively even porosity, our research shows that the porosity of the wind tunnel model of the screen will need to be increased by a certain factor to provide equivalence in the value of the drag.

To be able to model the effect of the Khalifa stadium on this structure, a scale of 1:300 was required. However, to be able to model the flow regime around the cables the scale of the cables was exaggerated by 60 times relative to the model scale (1:5 scale). This was to ensure similarity of flow regime between the model and full-scale.

The model was attached to a force balance and was tested with two configurations: a porous screen configuration as shown in Figure 13 and an impermeable screen configuration as shown in Figure 14. In addition the drag on the force balance shaft alone was also measured and he drag subtracted from the two measurements. This enabled the determination of the porosity factor for wind from different wind directions. An example of the results (for the main axis) is as shown in Figure 15.

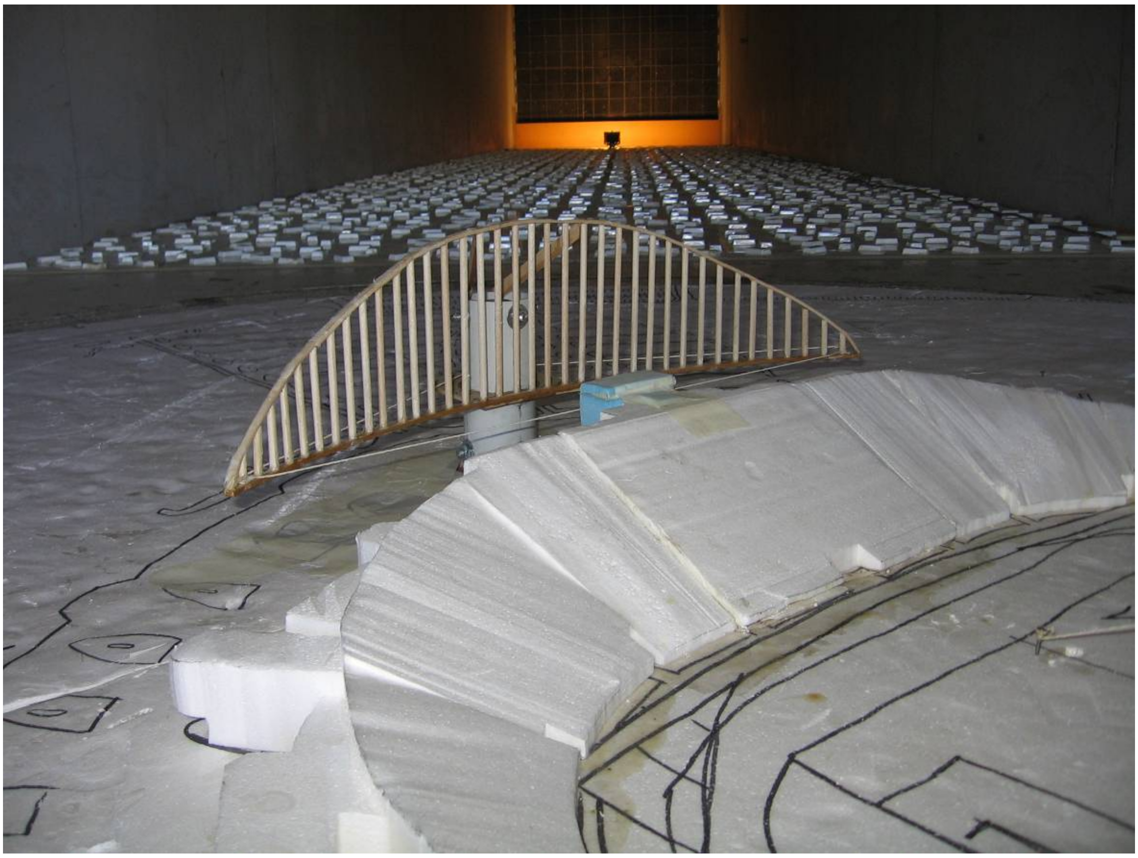

Figure 13. The 1:300 scale porous model of the LED screen for the 2006 Asian Games, Doha connected to the force balance.

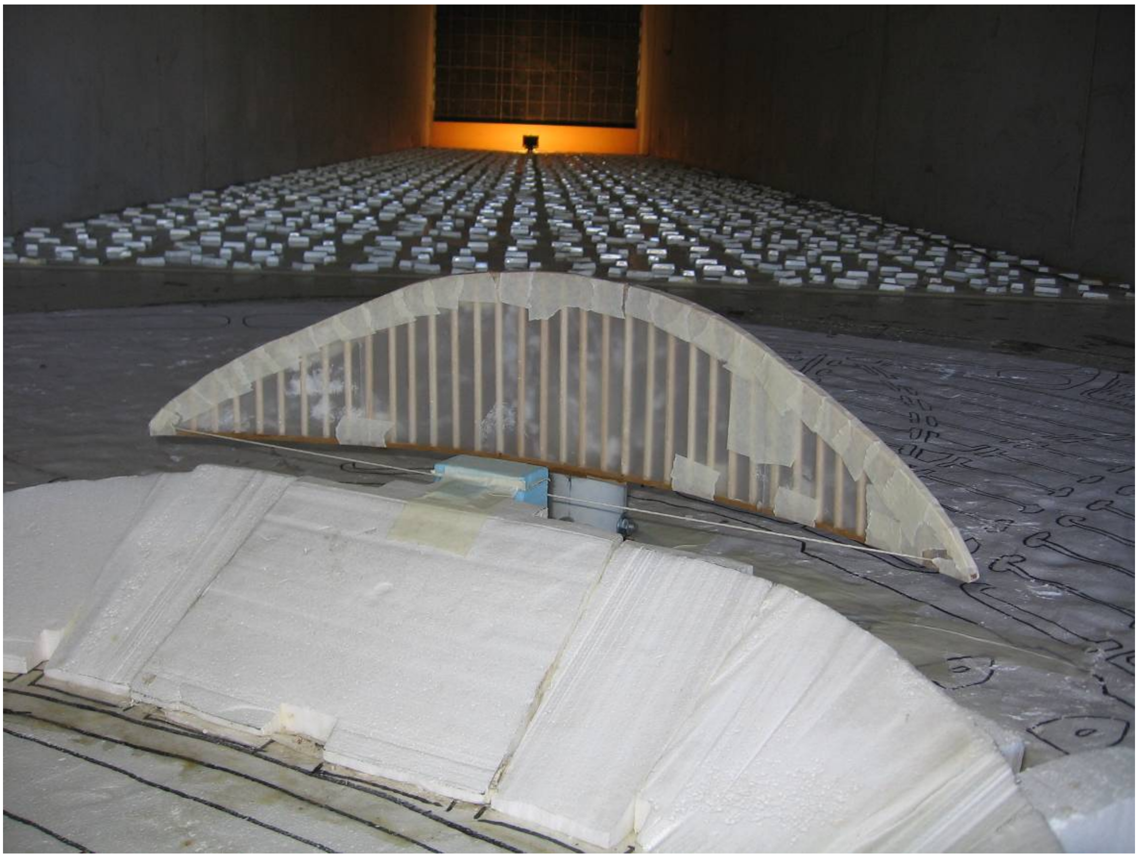

Figure 14. The 1:300 scale impermeable model of the LED screen for the 2006 Asian Games, Doha connected to the force balance.

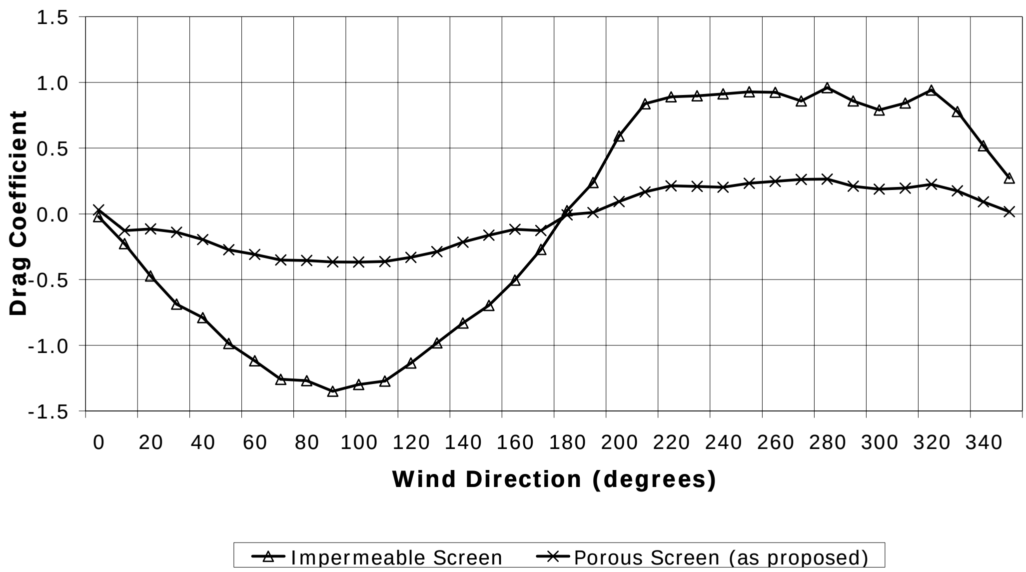

Figure 15. Drag Coefficient Comparison between the Impermeable and Porous Screen (along the East-West axis)

Note that the results for wind from the critical west direction are somewhat affected by the presence of the Khalifa stadium structure.

The pressure tapped model was then used to provide an accurate distribution of the loads between the 8 panels areas that correspond to the 8 supporting space trusses (refer to Figure 16, below). A photograph of the pressure tapped model before inclusion of the Khalifa Stadium model is shown in Figure 17. The test using this model was also used as a check on the results of the force balance tests using the impermeable screen option.

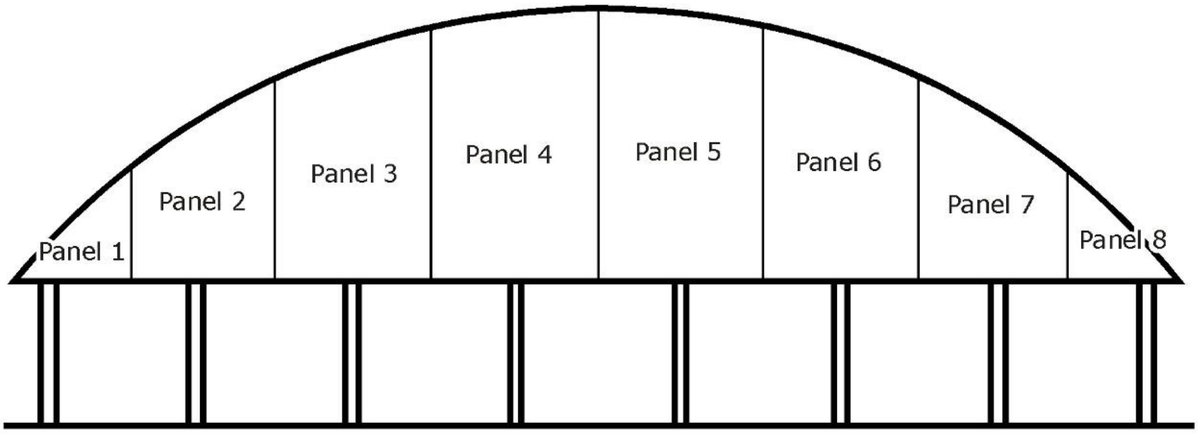

Figure 16. The tributary areas for each supporting vertical space frame structure.

Figure 17. The 1:300 scale pressure-tapped impermeable model of the LED screen for the 2006 Asian Games, Doha.

4. Conclusions

Advanced wind tunnel testing techniques have been developed, particularly over the past 10years to tackle the increasingly challenging task of predicting the dynamic wind induced response of unusual structures. This includes structures with complex dynamic behaviour and/or complex form. In the past these structures were simplified to some degree for the wind engineering consultant to be able to model their effects. Today, techniques are now developed to the extent that complex behaviour can he accurately and reliably modelled to the extent that the limitation is in the ability to reliably predict the dynamic response of the structure rather than how to model the interaction of the complex response to wind excitation.

Techniques have also been developed to more reliably determine the critical load cases for wind loads on large structures such as stadium roofs or linked tall buildings.

For some unusual structures it is important to recognise that the wind tunnel model will need to be somewhat distorted to enable the accurate modelling of the wind flow regime through and around the structure to be able to arrive at the correct prediction.

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