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Wind Tunnel Model Tests on Wind Sensitive Structures

Essais sur modèle de structures sensibles au vent

Einsatz von Windtunnel für windempfindliche Tragwerke

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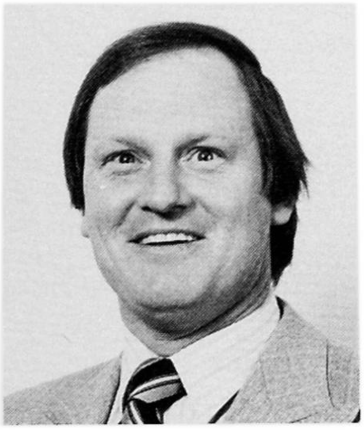
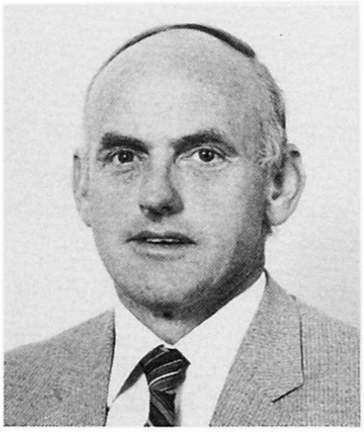
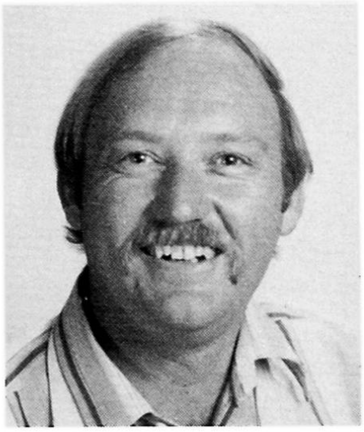






SUMMARY
This paper discusses the use of dynamic modelling applied in wind tunnel testing of a slender large span steel cantilevered grandstand roof, two 50 storey office towers in close configuration, and a 120 metre tall steel lattice cantilever tower. In each case a well documented knowledge of the directional nature of wind was used to permit cost savings to be achieved.

RESUME
Cet article décrit l'utilisation de modèles dynamiques dans une soufflerie pour tester un grand porte-à-faux élancé en acier, une toiture de tribune, deux immeubles-tour de 50 étages et une tour en treillis métallique de 120 m de haut. Dans chaque cas, une étude poussée de la direction et de la nature du vent a été faite pour permettre de substantielles économies.

ZUSAMMENFASSUNG
Dieser Beitrag behandelt den Einsatz dynamischer Modelle im Zusammenhang mit Windkanaluntersuchungen von einem schlanken, weitgespannten, auskragenden, stählernen Tribünendach, von zwei nebeneinander stehenden 50-stöckigen Bürohochhäusern und von einem 120 m hohen stählernen, netzförmigen Turm. In allen Fällen wurde von den genau dokumentierten, örtlich herrschenden Windverhältnissen Gebrauch gemacht und dadurch eine Kosteneinsparung ermöglicht.

1. INTRODUCTION

The Australian Loading Code AS1170, Part 2 - Wind Forces, like other international wind codes, determines wind forces based on wind velocity profiles, terrain category velocity modifiers and drag coefficients which vary with the shape and orientation of members and buildings. Basic design wind speeds for 5, 25, 50 and 100 year return periods are provided for all major centres in Australia. Design wind pressures may then be calculated as follows:

$$P_z = C_p \times 0.6\, V_z^2 \times 10^{-3}\ \mathrm{kN/m^2}$$

where P_z = wind pressure at height z

V_z = design wind velocity at height z, m/sec

C_p = pressure coefficient on a surface

It further permits, as an alternative to the above "quasi-static" approach, the carrying out of Wind Tunnel Tests for Dynamic Response, to establish the wind forces. This is permitted providing that:

- the natural wind has been modelled to take account of variation of wind speed with height,
- tests on curved shapes are conducted with due regard to effects of Reynolds numbers,
- the natural wind has been modelled to account for the scale and intensity of the longitudinal component of turbulence,
- the model is scaled with due regard to mass, length, stiffness and damping.

Wind forces are not static loads but a complex dynamic interaction between the wind and the structure that obstructs its path. As the stiffness of a structure decreases the wind excitation increases and the displacement of the structure is magnified beyond that predicted by an "equivalent" static load application.

The real response is very complex to predict as it depends on the stiffness, mass, dimension, shape and orientation of the structure, its location relative to other buildings and topography, as well as the basic wind speed which varies with height, time and direction. The only reliable means to predict the dynamic response of wind sensitive structures is the testing in wind tunnels using dynamic modelling.

This technique has been applied to three different structures for which the application of "quasi-static" loads were considered to be inappropriate. Use was further made of the directional nature of wind which has been documented for Melbourne (Fig. 1) drawing on anemometer data.

All of the dynamic model testing reported in this paper was carried out by Professor W.H. Melbourne at Monash University, Melbourne. [1]

2. WIND TUNNEL TESTING

The 450 KW closed circuit wind tunnel developed and built in the Department of Mechanical Engineering has an overall loop dimension of 10m x 28m and two working sections. One has a cross section of 4m width and 3m height, while the second measures 2m x 2m. This tunnel has been in operation since 1970 and has been used to add enormously to the application of advanced wind engineering techniques to major structures in Australia and the Asian region. Particular use has been environmental wind effects in central city locations, wind forces on cladding elements, dispersion of wind effluents and in particular the dynamic response of wind sensitive structures.

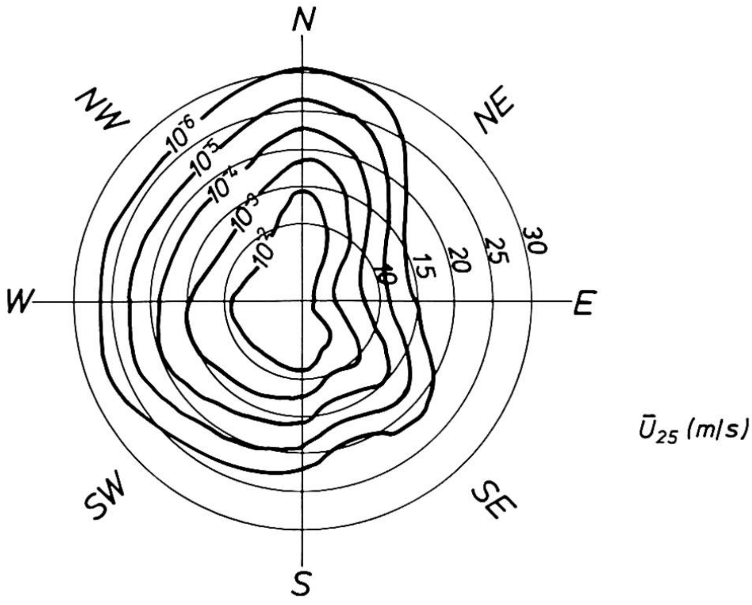


Fig. 1 Directional Wind Speeds, MELBOURNE

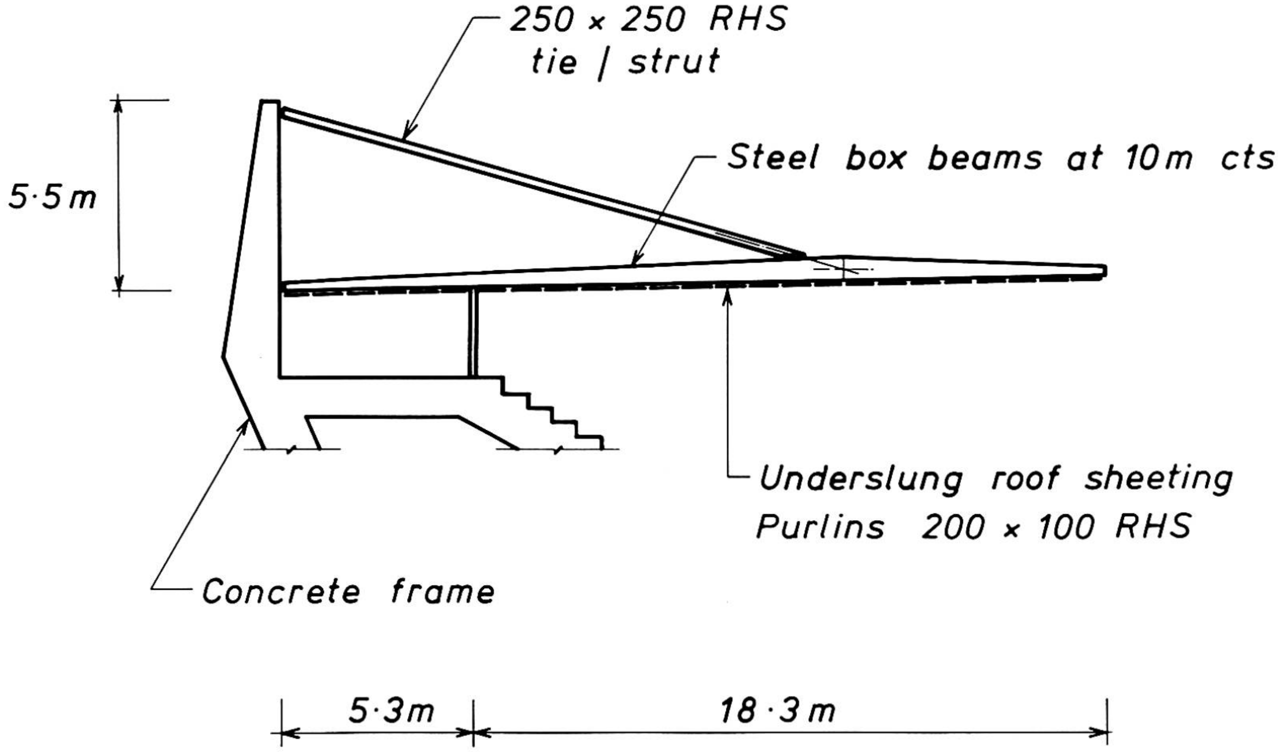


Fig. 2 VFL Park, Waverley, Roof Cross Section

3. VFL PARK, WAVERLEY

This is the home of Australian Rules Football and has been developed to provide seating for over 100,000 spectators. The cantilever roof dimensions are shown on Fig. 2. It forms part of a 150 metre long roof which soars some 25m above the playing field.

A 1:100 scale, aero-elastic model of 8 bays (half of total) was tested for various wind orientations. The members of the roof beams were made from several types of timber, selected to obtain appropriate elastic properties. The roof was made of balsa to which small amounts of additional mass were added to obtain the correctly scaled values. The action of the rear column (reinforced concrete) and main beam (structural steel) was taken to be pure bending while all other members were considered loaded in pure tension or compression for the purposes of scaling. The velocity scale was selected as 0.50.

During wind tunnel testing at low wind speeds, the roof displacement closely followed the deflection pattern of a cantilever. As the wind speed increased a low frequency wave, travelling along the leading edge, started to form (Fig. 3) and was superimposed on the cantilever bending mode. Fig. 4 shows the mean and maximum displacements at the leading edge in the centre of the 8 bay model. The increased contribution from the "cross-wind" action is seen to dominate as the wind speed increases and shows rapid divergence above 25m/sec. This means that at high wind speeds, the real displacement will be up to twice that predicted by the "quasi-static" load obtained by applying the wind loading code.

The significance of the wind tunnel model results must be related to the orientation of the roof and the wind speed acting on the actual structure. Fig. 1 shows the distribution of mean wind speed for Melbourne. Each circle represents the 10 minute mean wind speed measured by anemometer at Essendon Airport, Melbourne, adjusted to a height of 25 metres in open country. The 10^{-6} contour is roughly equivalent to a 100 year return maximum wind speed. The distribution clearly shows considerable variation with orientation. The grandstand faces south and therefore the probability of the design wind load being exceeded is less than 1% per annum which corresponds to designing for a 100 year return wind. If the grandstand faced north, then clearly the stiffness would need to be increased since the mean wind speed from this direction for the same 100 year return period is 36 per cent higher.

The basic design wind speed for Melbourne is 39m/sec for a 50 year return period and to limit excessive dynamic response for wind sensitive cantilevers a stiffness criteria has been adopted which limits the deflection of the cantilever tip to span/200 for the particular wind speed from the direction in which the roof is facing. This criteria has been adopted with success on other wind sensitive roofs including the 28m steel truss cantilever roof at the Victorian Racing Club Grandstand for horse racing in Melbourne. This cantilever roof has suspended at its very tip a 2 storey judges' box and photo-finish camera.

4. COLLINS PLACE PROJECT

This is the largest commercial development project completed in Australia to date. It incorporates two (2) fifty (50) storey towers (ANZ and Collins Towers) linked at its base by a 1 hectare retail/commercial and hotel-lobby area under a vast glassed space frame roof suspended from the towers. The top sixteen (16) storeys of one of the towers is an international hotel containing 375 rooms. The location and orientation of the towers on the site are shown on Fig. 5.

Earlier static model testing in the wind tunnel on a 1/600 scale model indicated that there could be significant aerodynamic interaction between the two towers. This led to a dynamic test being carried out on a 1/384 model and a velocity ratio of 0.20. The towers are of "tube-in-tube" construction with an extremely

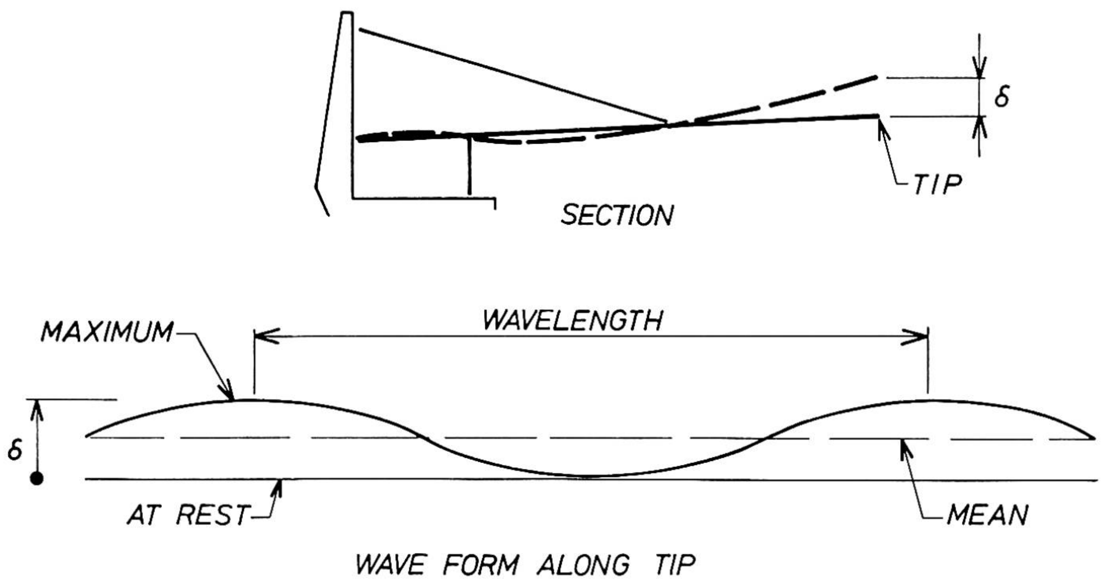


Fig. 3 VFL Park, Waverley, Tip Displacement

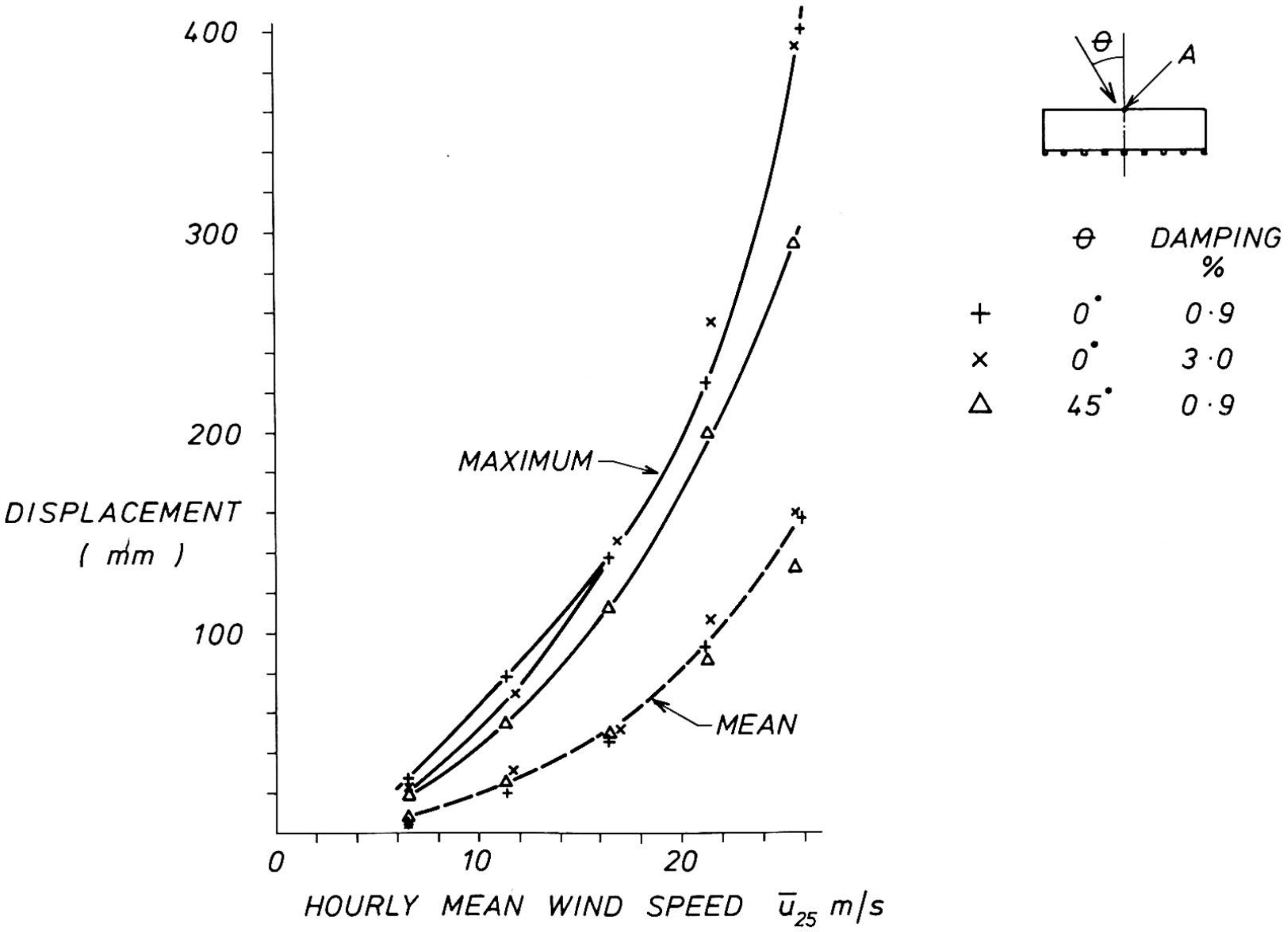


Fig. 4 VFL Park, Waverley, Displacement vs. Wind Speed

stiff facade which permitted the towers to be simulated as rigid blocks placed on spring mounted bases. The tower models were made of Daycel rigid foam with a 1mm plywood skin mounted on steel cantilevers. The positions of the towers on the cantilevers were such as to permit rotation about a pivot point at the footing level of the towers. The cantilevers were strain gauged to allow the overturning bending moments to be measured about two perpendicular horizontal axes at the footing level. Variation in damping was obtained by using oil dashpots at the footing level. The damping used was 1.1% of critical damping and the natural frequency used was 0.3H_z. A full scale mean hourly maximum wind speed of 30m/sec at a height of 184m above ground level (200m above footing level) was applied.

The design wind loading obtained from the model tests indicated that the interaction between the two towers results in overturning moments about the one axis of approximately 75% of that occurring simultaneously about the other axis. Maximum overturning moments about the base were:

$$M_1 = 1.442 \times 10^6 \text{ kN.m}$$

acting simultaneously with

$$M_2 = 1.055 \times 10^6 \text{ kN.m}$$

The above maximums occur for the ANZ Tower with the wind direction from NW to N from which approximately 50% of the yearly maximum winds occur in Melbourne. These moments are equal to mean plus 3.5 times the standard deviation ($\overline{M} + 3.5\,\sigma_m$) and this corresponds to a 10% probability over a life of 100 years. Higher moments by approximately 3% do occur for the Collins Tower but this is for a wind direction from S to SSE which as discussed earlier, is a considerably lower probability in Melbourne.

The lateral displacement and accelerations at the top of each tower were also calculated and the following values obtained for a 100 year return period.

	displacement	acceleration
ANZ Tower	132mm	3.5% of g
Collins Tower (Hotel)	91mm	1.7% of g

Reliable data on levels of perceptability of motion and acceleration was scarce at that time. However, they were considered less than those predicted on tall buildings then under construction in the USA and were considered acceptable even allowing for the close proximity of the two towers. After several years of occupancy, no complaints have been registered.

5. VICTORIAN ARTS CENTRE UPPER SPIRE

The spire over the Theatres complex rises to a height of 120m above street level and comprises a tapered open latticed spire which rises 84m from its supports (Fig. 6). The Upper Spire has been built using the Mero Spaceframe "ball-joint" connection as shown in Fig. 7. The connection is effectively a "pin-joint" with axial compression being transmitted from the mild steel tube through the cone and sleeve to the node. Axial tension is transmitted from the mild steel tube through the cone and bolt to the node. The bolt, therefore, is never in compression but under wind, experiences a variable tension.

When designing the tower it was considered that the behaviour of the tower could not be predicted by using the "quasi-static" loads from the Wind Code. In fact, interpretation of the resistance to wind offered by the joints and members and the varying degree of shielding offered, was considered to be too great an uncertainty. As the bolts in the connection would be subjected to fluctuating loads under wind, the fatigue life of the bolts needed investigat-

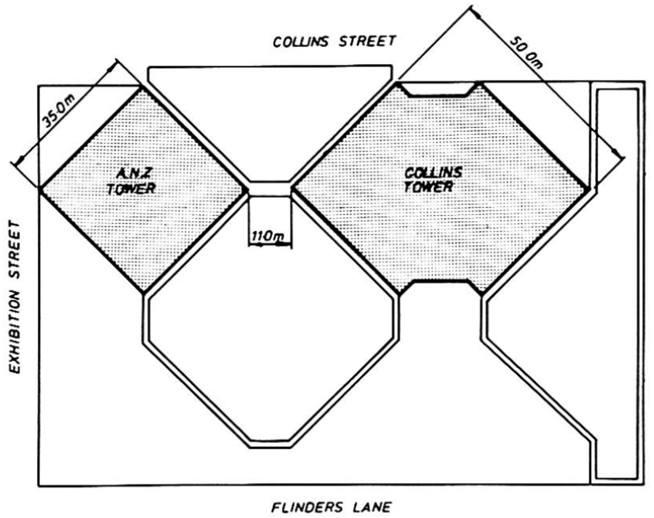
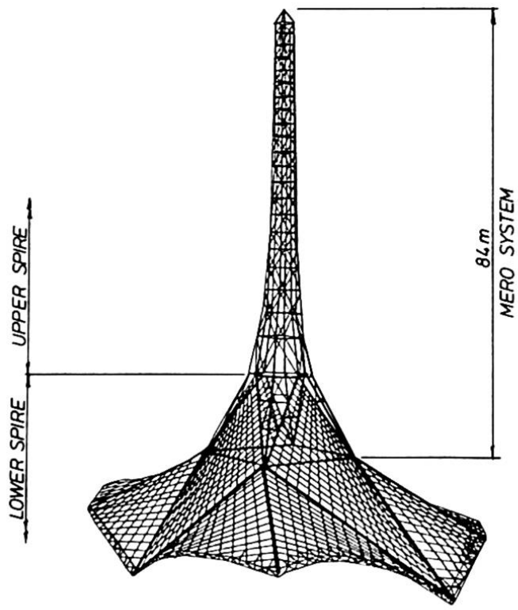


Fig. 5 Collins Place, Melbourne, Site Plan



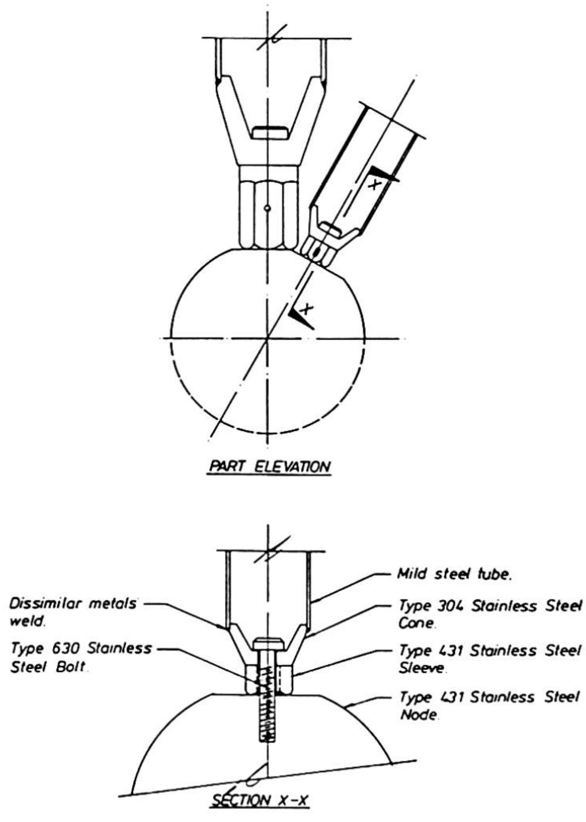


Fig. 6 VAC Upper Spire

Fig. 7 Mero Connection

ion. An underestimate of bolt forces would considerably affect the life of the structure. A design life of 100 years being the criteria.

A full scale model of the upper six levels of the spire was tested to establish the damping characteristics of the joint system, as this directly affects the dynamic response and therefore the loads. The damping was measured by inducing a natural oscillation and then measuring the degradation using an accelerometer and chart recorder. This was carried out for the condition of torqued bolts and snug-tight bolts. From this, a conservative value for critical damping of 0.004 was adopted for design purposes.

The aeroelastic model for the spire was built to a linear scale of 1:100 using sugar pine with all members loaded purely in tension or compression, the velocity ratio was 0.3.

The results from the model tests provided total overturning moments at the base of the spire approximately 30% less than those obtained by direct application of the Wind Code. This reduction was of great significance in the design and more than offset the cost of wind tunnel testing.

The design loads for fatigue design of the bolts was established by using the natural frequency of vibration, directional wind data (Fig. 1) available for Melbourne, computing the response of the spire in 16 directional areas of wind and for 8 intensities of wind speed. This then permitted the number of cycles of the 8 stress levels that would be experienced over the 100 year design life of the spire to be summated. This data was then used as the stress-cycle history for the fatigue analysis of bolts, members and welds. A limit state design approach being adopted. Without this data a much more conservative approach would need to have been adopted. This being of extra significance as the fatigue life of the bolts decreases with increased diameters.

6. SUMMARY

The use of wind tunnel model testing in predicting the dynamic response of actual structures has permitted a real understanding of the behaviour of structures for those cases where wind loads are a dominant design criteria. Direct application of the "quasi-static" approach of the Wind Code would in the case of wind sensitive structures have resulted in excessive deflection and overstressing. In other cases, a reduced wind loading was able to be applied resulting in considerable cost saving. In all cases, the use of available directional wind data has permitted this to be applied to give direct savings in design loading.

All designs of major structures which are suspected of being sensitive to wind should draw on the advice of experienced wind engineers and use the available wind tunnel facilities for dynamic testing.

REFERENCES

1. Dr. W.H. Melbourne. "Development of Natural Wind Models at Monash University", Fluid Mechanics Conference, Adelaide, Australia. 5th - 9th December, 1977.