Victorian Arts Centre spire, Melbourne

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Flèche du Victorian Arts Center, Melbourne

Der Turm des Victorian Arts Center in Melbourne

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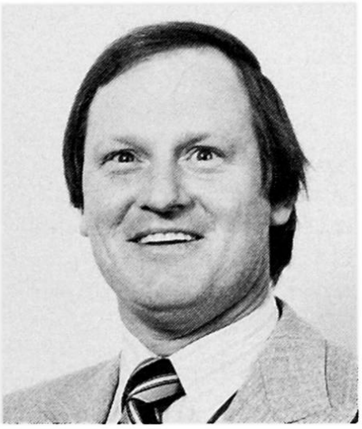

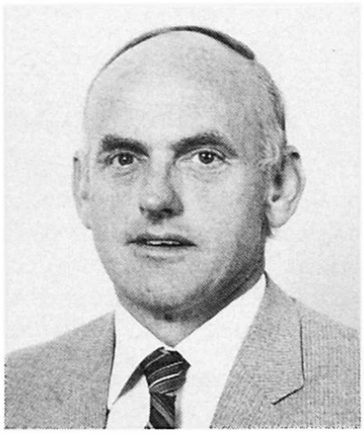

SUMMARY
This paper describes the concept development, structural analysis and design of the Spire for the new Victorian Arts Centre in Melbourne, Australia. The structure comprises both the Mero and Triodetic space frame systems and involved some of the largest single layers hypars constructed in the world to date.

RESUME
L'article décrit le concept de base, l'analyse structurelle et le calcul de la Flèche du nouveau Victorian Arts Center à Melbourne, en Australie. Cette structure comprend à la fois un treillis tridimensionnel Mero et Triodetic et constitue un des plus grand paraboloïde hyperbolique en une seule couche construit jusqu'à maintenant dans le monde.

ZUSAMMENFASSUNG
Der Beitrag beschreibt die Entwicklung des Konzepts, die Bemessung und das Projekt des Turmes für das neue Victorian Arts Center in Melbourne, Australien. Das Bauwerk besteht aus räumlichen Mero- und Triodetic-Rahmensystemen und weist auch einige der grössten Schalen vom Typ des hyperbolischen Paraboloids auf, die bis heute weltweit gebaut wurden.

1. INTRODUCTION

The Victorian Arts Centre (VAC) comprises three separate buildings, an Art Gallery, a 2700 seat Concert Hall commissioned in 1982, and a Theatres Building incorporating 2,000, 850 and 450 seat theatres due for completion in mid-1984. The Architect, Suendermann Douglas McFall Pty. Ltd. (formerly Roy Grounds & Co. Pty. Ltd.), decided that a spire was required to act as a focal point for the overall complex. Several concepts and geometrical forms were investigated prior to the adoption of an open latticework tapering spire section flowing upwards from a base "skirt" formed by twelve hyperbolic paraboloids (hypars), Figs. 1 & 2. The "skirt" performs the secondary function of concealing, to a certain extent, the somewhat amorphous shapes of roof plantrooms and stage towers which are associated with a theatres complex.

The structure was required to adhere to the following major design criteria:-

- Wind loading - a 100 year return period was specified.
- Durability - minimal maintenance for a 100 year design life.
- Constructability - erection above a completed building.

2. STRUCTURAL SELECTION

Tower structures with bolted, welded and proprietary connection systems were evaluated. Member shapes consisting of single angles, rolled hollow sections, as well as tubular members, were examined. The use of a cable stayed tensioned tower sweeping down to the Lower Spire was also entertained and dismissed. Materials considered included mild steel, stainless steel, weathering steel and aluminium. Various corrosion protection systems were investigated.

From an aesthetic viewpoint, a space frame structural form was preferred, particularly the use of "ball type" connections for the Upper Spire section. Space frames were also favoured for ease of erection as relatively small components could be delivered and erected on site, these members also being of human scale as required by the Architect.

A series of proprietary space frame systems were therefore investigated prior to the Mero joint system being adopted for the Upper Spire, Fig. 3. The Mero joint had not been used previously in such a major cantilevering tower structure, and hence extensive design and prototype testing was required to prove the application of the system.

The Triodetic system was selected for the Lower Spire because its 'keyed' joint detail provides 'out-of-plane' stiffness which was required for the hypar shells, Fig. 4. The single layered hypar spans of up to 30 metres are amongst the largest constructed in the World.

3. ANALYSIS AND DESIGN OF THE UPPER SPIRE

The Upper Spire flows up from the Lower Spire through the form of four complete hypars being supported by eight of the twelve main support legs of the tower, Fig. 2. The upper section of the Spire consists of a straight taper. The lower section of the Upper Spire forms a transition region between the Lower Spire shells and the straight tapered section.

Both analysis and design was computer based [1] . The dominant effect of wind loading and the dynamic response to that loading were the governing design criteria for the selection of the Upper Spire member sizes. As the design development process was a total interaction between aesthetics, structure, and compatibility with the Theatres Complex below, a computer plotting package was developed to permit the visual aspects of the Spire and the Theatres Building to be studied from all possible vantage points.

As the applicable building regulations did not cover the use of stainless steel, the building authorities required prototype testing and independent proof checking of the design. As fatigue was a critical limit state, use was made of risk analysis related to social criteria, human safety and the probability of failure and its impact on the community [2] .

3.1 Wind Loading

A 1:100 scale aero-elastic model of the Upper Spire was tested in a wind tunnel after first having tested a six level, full scale prototype of the Upper Spire to establish the damping characteristics of the Mero construction [3] . The wind tunnel test result permitted the adoption of a gust load factor of 1.64 compared with the Wind Loading Code, AS1170, Part II, which uses a factor in excess of 2.0, thereby resulting in an overall overturning moment approximately 20% lower than that predicted by direct application of the Wind Loading Code.

3.2 Fatigue Analysis

As the dead load of the Spire is relatively low, compared with the alternating compression and tension forces generated by the wind, the fluctuating forces result in fatigue of the bolts and welds being critical. A detailed load cycle-load level-wind direction study was carried out on the Spire. This made use of the well documented knowledge of the directional nature of wind in Melbourne. Wind velocities (and stress levels) were divided into eight levels of intensity for twelve segments of wind direction of 22.5^{o} each. The along-wind and cross-wind responses of the Spire were combined having due regard for the varying ampitude of vibration and the fact that the cross-wind and along-wind do not act in constant phase with each other.

3.3 Fatigue Design

3.3.1 Welds

The selection of stainless steel components for the Mero connections results in a weld between stainless steel forged cones and mild steel tubular members, Fig. 1. The weld geometry is such that the influence of stress-raising and metallurgical imperfections due to the welding of dissimilar metals could only be evaluated by fatigue testing of prototype and production welded components. For correlation purposes, mild steel to mild steel welded components were also subjected to fatigue testing in series with the mild steel to stainless steel welds. In all cases, the fatigue strength of the stainless steel to mild steel welds were at least twice that of the mild steel welds.

3.3.2 Bolts

The bolts vary in diameter from 20mm to 39mm and fatigue tests were carried out on several bolts of each diameter. From these tests S-N curves were prepared. These were then used as design parameters for establishing the probability of failure of the bolts throughout the Spire.

When a bolt fails a new structure results and a change in load distribution follows. A true failure analysis of the Spire therefore, required the failed member to be removed from the analysis model to permit the next failure location to be found. This analysis showed that the true design life of the structure could be expressed in terms of a 1 in 10^{6} probability of two bolts failing in a period of approximately 115 years (compared with a required design life of 100 years). Even with two bolts failed, the reduced structure showed reserve capacity. For a 1 in 100 year return period wind storm the maximum overstressing of the modified structure was 26%.

3.4 Material Selection

The material selected was dictated primarily by the durability considerations for a 100 year life. Stainless steel was selected for the connections as the Mero joints could never be totally sealed against moisture and environmental pollution. The tubular members were left as mild steel because of economics. The cones were selected as an austenitic stainless steel so that they could be welded to the mild steel tubes. The bolts use a high strength stainless steel with an ultimate tensile strength of 1000MPa. The nodes and sleeves are also stainless steel for durability and appearance. The corrosion protection of the mild steel members (with the welded cone assembly) was achieved by hot dip galvanising after welding. The cone/tube assembly was then lightly sandblasted, acid etched, then primed and coated with a two pack epoxy white paint.

4. ANALYSIS AND DESIGN OF THE LOWER SPIRE

With reference to Fig. 2, the Lower Spire hypars comprise a latticework of aluminium tubular members, typically 100mm diameter, supported by mild steel tubular 'edge beams'. Eight of the hypars were truncated with a double layered stiffening space truss across the outside edge.

The basic concept was to ensure that the single layered latticework acted as a shell structure with external loads being primarily carried by 'in plane' forces rather than flexural moments. As with all shell structures, the overall buckling resistance was a major design consideration with respect to the 100 year wind loading. Limited literature regarding hypar buckling data was found to be available. A full scale prototype of a portion of one of the truncated hypars was therefore constructed and load tested at the University of New South Wales, Sydney [4] .

Initially a single layered framework was load tested and the relationship between the edge beam stiffness and the hypar buckling load evaluated, Fig. 5. These results were then used to set deflection criteria for the steel support beams in the final structure. The main conclusion from the first phase of the test was the need for a stiffening space truss along the truncated edge to ensure that the truncated hypar acted as a shell rather than a series of parabolic arches. The prototype was stiffened accordingly and load tests verified that the single layered section acted as a 2-way catenary tension zone between the stiff bending elements. The shell buckling load was compared with theoretically computed values and the results extrapolated to check that the constructed hypars had an adequate factor of safety against buckling.

Computer Aided Design techniques were extensively used for load generation, hypar member and joint design, and the provision of fabrication data. The analyses comprised a linear elastic 3-D frame programme for which special pre and post processor programmes were written [1] .

During detailed design it was found that certain hypar members were overstressed. The originally proposed 6063-T6 aluminium alloy had been selected mainly to give good "workability" characteristics during the fabrication stage prior to the anodising and heat treatment process. For visual and geometrical constraint reasons, it was not possible to increase the section sizes and hence a "special practice" 6063 alloy was developed for the project. This alloy was satisfactory in its T4 temper for the extrusion and fabrication processes but had the required increased strength when heat treated to the T6 temper.

To meet the stringent durability requirements, the edge beam steelwork was hot dip galvanised and finished with the same paint system as the Upper Spire tubular members. The hypar members have a high resistance to corrosion due to their aluminium base material which is further protected by the gold anodised finish.

FIGURE 1 - VAC SPIRE AT NIGHT

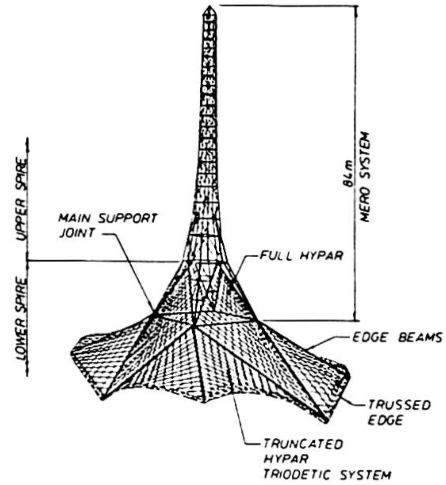

FIGURE 2
VICTORIAN ARTS CENTRE SPIRE
ELEVATION

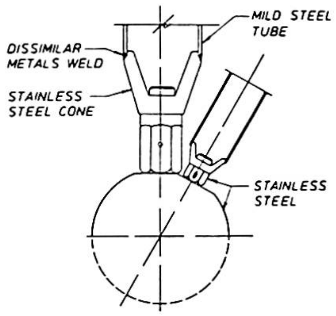

FIGURE 3 - MERO JOINT

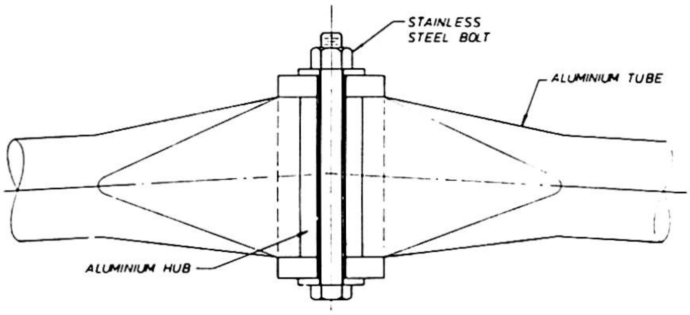

FIGURE 4 - TRIODETIC JOINT

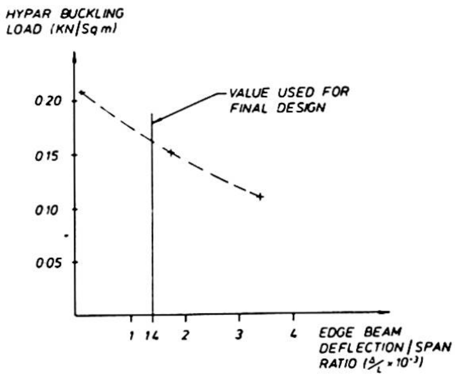

FIGURE 5 - HYPAR BUCKLING LOAD VERSUS EDGE BEAM STIFFNESS

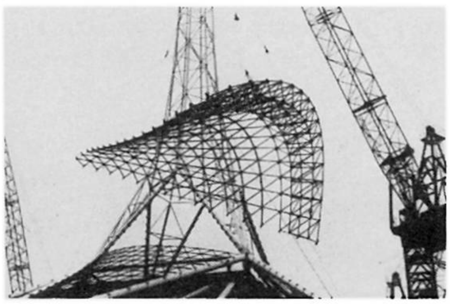

FIGURE 6 - HYPAR ERECTION

5. ERECTION OF UPPER AND LOWER SPIRES

The erection method adopted followed directly the type of structural system selected. It recognised the limited site access and the need to construct above a near completed Theatres Building.

With reference to Fig. 2, three erection phases were involved.

5.1 4 Main Support Connections and Hypar Edge Beams

These were erected in sections by crane and set out to within ± 5mm of their computed locations prior to being fully butt welded together. The tight tolerance specification was essential to avoid 'lack of fit' problems with the subsequent space frame erection.

5.2 The Upper Spire Section

This was erected in-situ using a central Alamak hoist system which necessitated a temporary platform below the base of the Spire to act as a working area and protection platform. Individual tubular members and nodes were lifted into position and assembled on the part completed Upper Spire.

5.3 Lower Spire Hypars

Due to the large plan dimensions of the hypars and the need to avoid loading the roof below, it was decided to completely pre-assemble the shells at ground level prior to craning them into position using a 3 point lift, Fig. 6. A full computer stress check of the 'space frames' during assembly and lifting was carried out. This was to ensure that no overstressing or permanent distortion occurred, although only self weight forces were involved, the 'shell' action was greatly reduced due to the absence of the edge stiffening members. The hypars were connected to the edge beams by means of a torqued bolt connection, thus maximising the available erection tolerances.

6. SUMMARY

It is rare to be engaged on an engineered "architectural sculpture" whose primary function is to indicate to the people of Melbourne the location of their cultural centre. The Spire, by its unique shape, materials, location and loading response required innovative application of technology and computer methods. The finished structure serves the design brief with flair.

REFERENCES

1. CAD of the Victorian Arts Centre Spire, P.W. Kneen, B.K. Dean and S.M. Jeffery. Symposium - CAD/CAM in the 80's and Integration through Data Bases. Melbourne, 20-22 October, 1980.

2. Structural Codes - The Rationalisation of Safety and Serviceability Factors, Chapter 5, Proceedings of Seminar in London, 14th October, 1976. Published by CIRIA.

3. Application of Wind Tunnel Model Tests on Wind Sensitive Structures, Melbourne, Australia. J.H. Wynhoven, J.J. Peyton and J.A.F. Williams. IABSE Congress, Vancouver, 1984.

4. Prototype Testing for Victorian Arts Centre Lower Spire Shells, P.W. Kneen, Lecturer, School of Civil Engineering, University of New South Wales, June 1979.