RECENT EXAMPLES OF LIGHTWEIGHT STRUCTURES IN AUSTRALIA

Owen Martin,Connell Wagner, Sydney
John Webb, Connell Wagner, Sydney
Brian Dean, Connell Wagner, Melbourne
Craig Stevenson, Connell Wagner, Auckland

Introduction

Connell Wagner has been responsible for the design of a number of lightweight steel structures in recent times. Prominent examples of such structures used in projects in Australia are described in this paper. These projects include:

- Sydney State Hockey Centre
- Melbourne Sports and Aquatic Centre
- Melbourne's Dockland Stadium
- Sydney's Railway Square glazed canopies
- Adelaide's Rundle Mall Canopy
- Sydney's Capital Square Galleria
- 60 Castlereagh Street, Sydney

The presented structures range from lightweight cable stayed and lattice roofs to ferro-vitreous canopies and fully glazed wall systems.

Due to the highly exposed nature of the load-bearing systems, designing aesthetically pleasing structural components and details has been a paramount goal.

Through cooperation with the architects on each project, Connell Wagner strove to achieve this goal whilst maintaining the cost effectiveness and buildability aspects of the design.
The efficiencies of resisting loadings through axial action, rather than flexure are demonstrated in many of the projects.

Sydney State Hockey Centre Roof

General

The new state hockey centre is located on the site of the existing state hockey centre at Homebush Bay, Sydney, and will function as the main competition venue for the 2000 Olympic games. It includes a grandstand, seating over 1,500 spectators and officials under a spectacular pavilion-style roof. Connell Wagner has developed a dramatic lightweight roof structure, which responds to the client's request for a state-of-the-art facility portraying a distinctive image and identity for the sport of hockey in Sydney for the 2000 Olympics and beyond.

The Roof Structure

Several conventional and semi-conventional solutions for the roof in line with the master plan, and to complement the adjacent State Sports Centre were investigated. However, the one which seemed to best address the architect's vision of a pavilion was the concept of a cable-stayed roof suspended from a single mast at the rear of the structure (Fig.1). An important benefit of this structural form is that it soars above the grandstand below, making it visually spectacular. It also avoids imposing any roof loads on the grandstand.

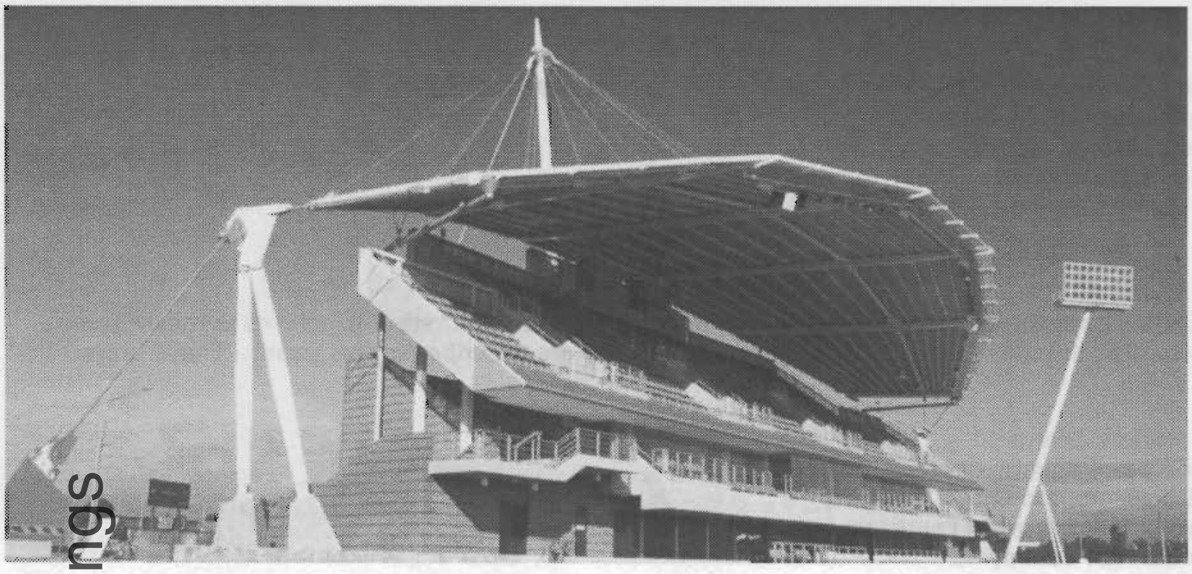

Figure 1. Grandstand Roof of Hockey Centre

The roof is lens shaped in plan and consists of five facets from the surface of a cylinder. It spans 70m from end to end with a maximum height of 9m at the centre. Steel was selected for the supporting members with steel roof sheeting. Six rafters are positioned at approximately 12m centres defining the borders of the five facets of the roof (Fig. 1). The rafters on the side edges of the roof are 406mm CHS members, while the four intermediate ones are 400mm deep, 300mm wide box sections tapering to a 200mm depth over a 4m length at either end.

A total of 12 forestay cables, 35mm and 45mm in diameter, are suspended from the rear 40m tall mast. They are connected to the rafters at the rear edge and at the two-third position from the rear, hence supporting the self-weight and downward imposed loading of the roof (Fig. 2). The mast is a fabricated steel tube with a maximum diameter of 800mm, tapering to 400mm over a 5m length at either end (Fig. 3). Two backstays, each measuring 100mm in diameter, stabilise the mast. Figure 4 depicts the downward load-resisting system for the roof.

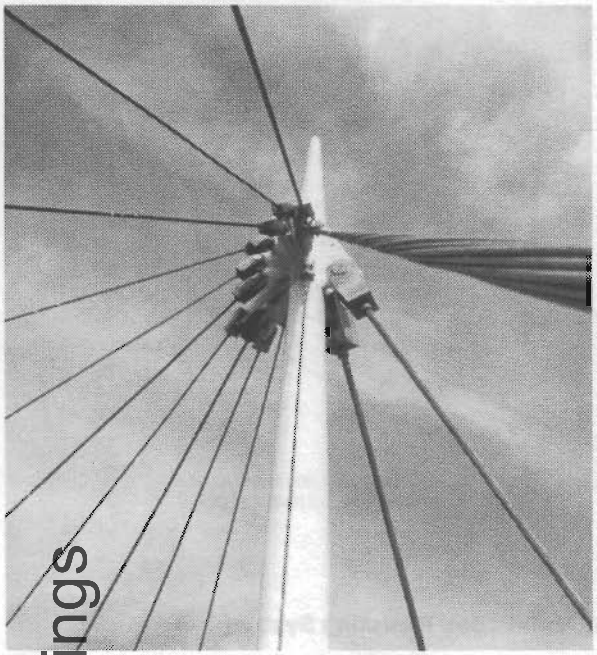

Figure 2. Stays Merging to Mast Head

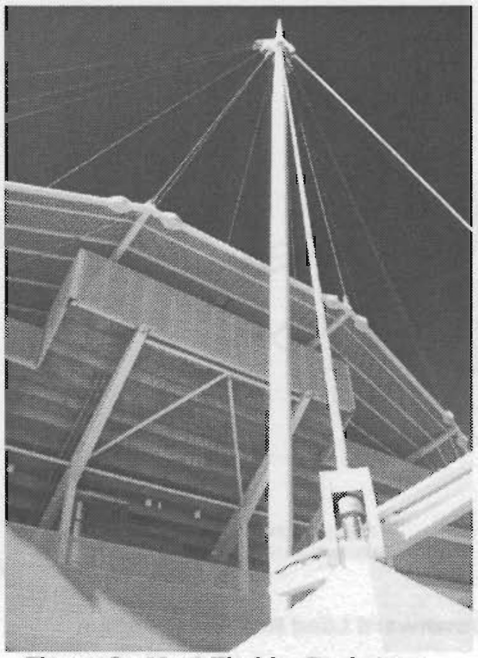

Figure 3. Mast Tied by Backstays

Two primary catenaries run along the rear edge and the two-third position of the roof, linking the forestay-rafter connection nodes (Fig. 5). They are connected to and stressed against the forestay cables, providing the main strength and stiffness against uplift wind loads. A third, essentially non-prestressed catenary is also positioned along the front edge of the roof providing additional strength and stiffness to the roof under wind uplift. The front two (prestressed and non-prestressed) catenaries also resist loads tending to push the roof towards the hockey pitch, while the rear catenary resists the loads pushing the roof away from the hockey pitch. The uplift and lateral load resisting components are highlighted in Figure 5.

Within the roof plane, the three catenaries are made of segments of 324mm CHS members. Beyond the side edges of the roof, however, the catenary actions are extended by three cables linking the ends of the steel tube catenaries to a focal point at the tip of a tied-down steel strut system. The strut system is fabricated in a shape similar to a wishbone, whose 10m long legs are tapered cruciform sections with maximum dimension of 600mm (Fig. 6). A single 95mm tie-down cable anchors the strut system back to the ground, hence completing the catenary actions (Fig. 7).

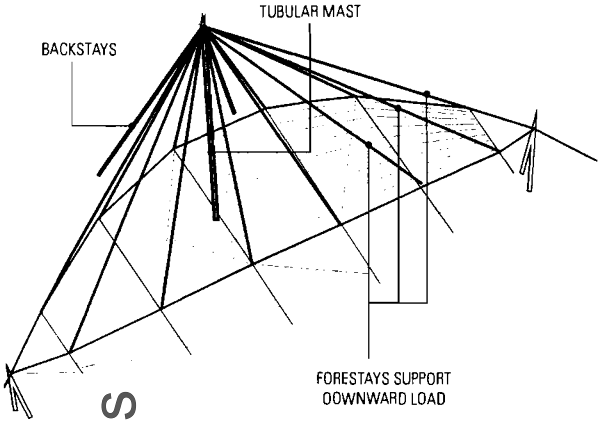

Figure 4. Downward Load Resisting System

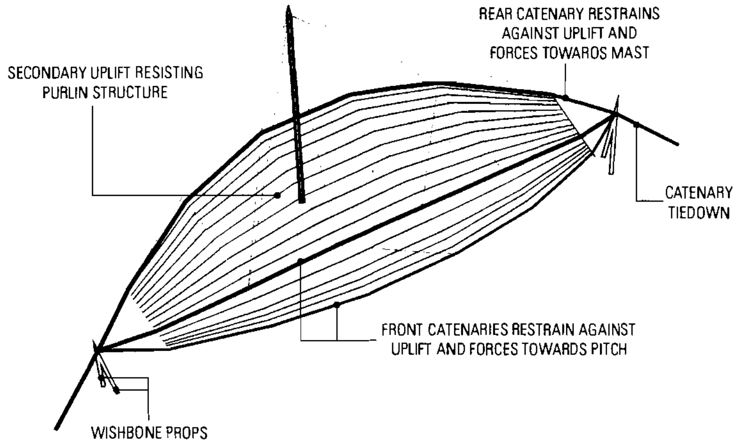

Figure 5. Uplift Load Resisting System

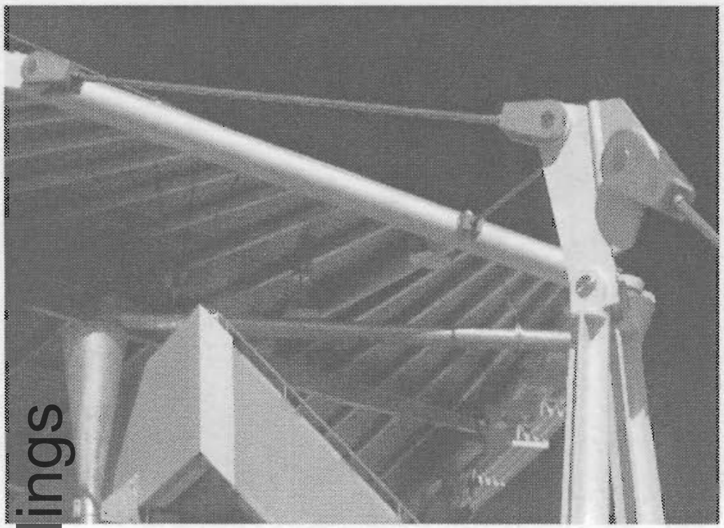

Figure 9. Wishbone Top and Link Cables

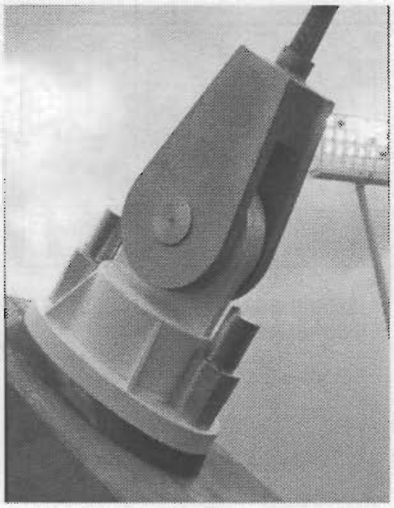

Figure 10. Tie-down Base

To further refine the concept of converging catenaries, axial capacities of the purlins (RHS sections) were also used as supplementary catenaries for additional strength and stiffness (Fig 8). This innovative solution substantially reduced the required pretension without penalising the design of the purlins, and hence, led to a significant economical benefit associated with reduced sizes for cables and foundations.

The innovative engineering approach employed on this project has created a remarkably lightweight roof structure of only 57 kg/m^2.

Melbourne Sports and Aquatic Centre Cable Stayed Roof

Introduction

The Melbourne Sports Centre complex is the largest integrated sports and leisure complex of its type in Australia, with a total floor area of over 35,000m^2. The complex houses the new Victorian Centres for basket ball, badminton, table tennis and squash as well as a wet sports area which includes a 79.5m Olympic standard pool, a 25m lap pool, a multi purpose pool and a wave pool. (Fig: 8).

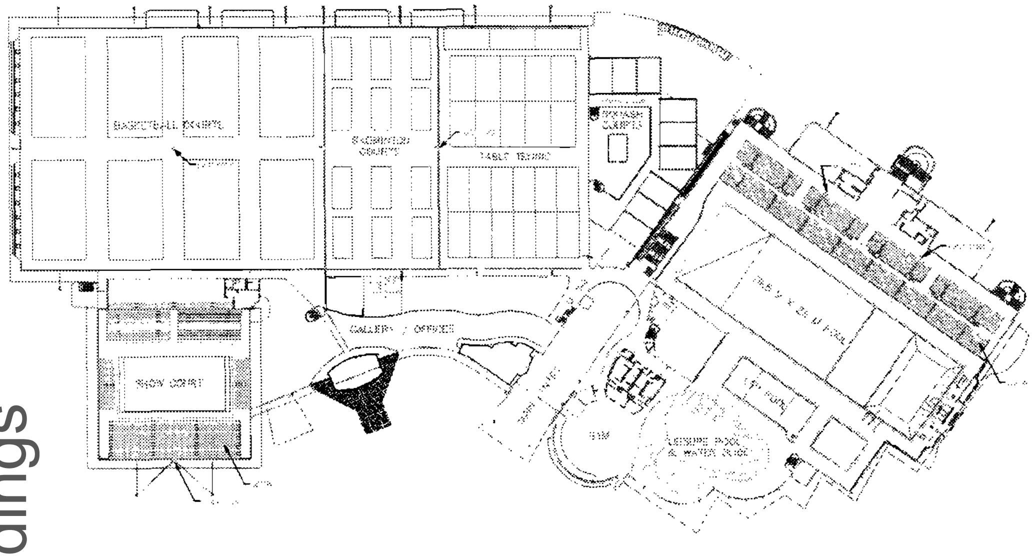

Figure 8: Floor Plan

The design of the Centre had to cater for the long span needs of the halls housing the facilities and provide the varying heights required to accommodate the diving board and sports such as badminton, basketball and volleyball. The dimensions of the three main halls are:

- Dry Sports Hall — 160m x 70m x 15m high
- Show Court — 42m x 42m x 13m high
- Competition Pool Hall — 100m x 48m x 18m high

Alternatives for the Roof Structure

Numerous approaches to the roof structure were investigated for cost comparison and benchmarking against the tight budget. Various internal column configurations were also examined to assess the additional costs associated with more open span solutions.
Structures investigated included:

- Portal frame
- Portal truss
- Cable stayed trusses
- Cable stayed curved beams

From an aesthetic viewpoint it was preferable to adopt a uniform structural form throughout the Centre. The main evaluation criteria were:

- economy
- flexibility for planning and extendability
- durability
- aesthetics

The Adopted Design

The selected structure consists of a cable stayed curved rafter solution, where five guyed masts with radiating rods act as the primary support for downward loadings acting on the roof. To resist the wind uplift, the upward curving rafters are restrained at their ends enabling them to resist net uplift pressures by tension action. This is a far more structurally efficient method than conventional flexural (bending) action. As a consequence the main rafters in the Aquatic Centre which span 48m are only 460 mm deep. If flexural action was the only method used, trusses in the order of two metres deep would have been required at 10.5 metre centres.

The area housing the main pool is 100m long and 50m wide and is supported by two masts which are located on one edge of the building with backstays.

Figure 9 shows a section through the competition pool enclosure with its mast support system on the west side of the pool. On the east side of the pool, the rafter tension forces under uplift are transferred to vertical bracing by way of a horizontal truss within the lap pool roof structure.

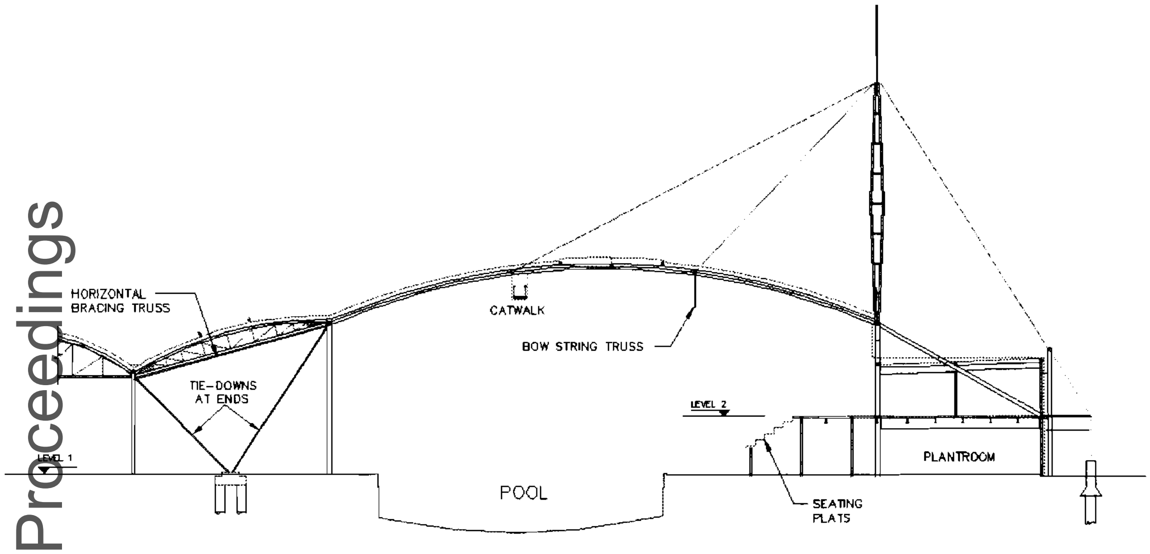

Figure 9: Competiton Pool Hall

A similar scheme was adopted for the dry sports hall. The hall is approximately 160m long and 70m wide, but has only three internal columns. Two of these columns extend into masts which protrude 20m above the roofline and support the roof via suspension rods. However, in this case the masts are centrally located within the hall, resulting in a balanced cable stayed scheme. (Fig.10).

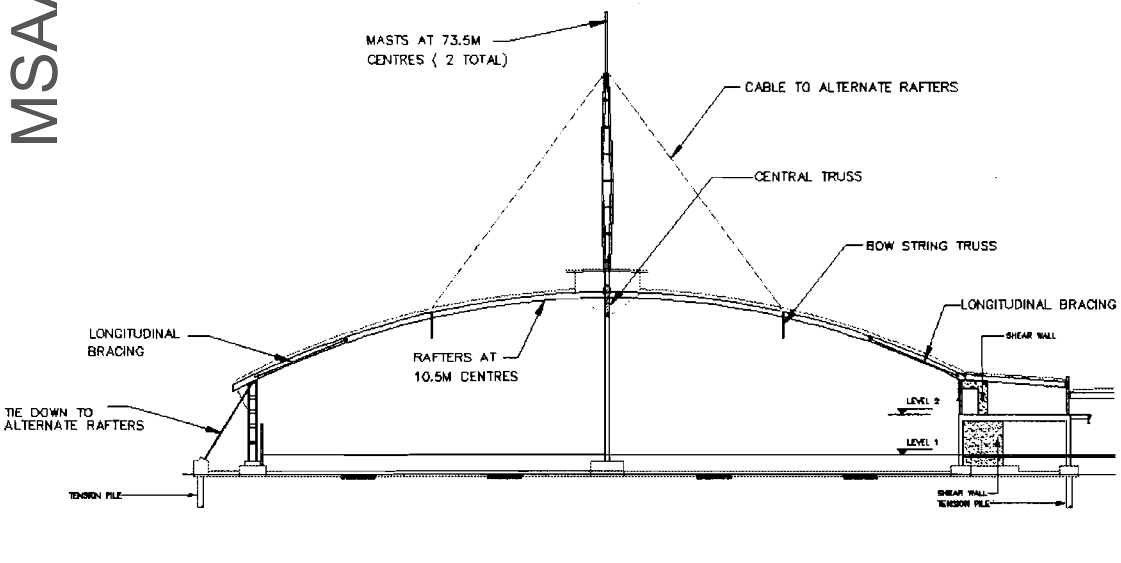

Figure 10: Drysport Hall

The roof design is both light and elegant, with the dry sports roof structural steelwork weighing only 20kg/m^2, and the competition pool roof (with its heavier ceiling and catwalk requirements) only 24kg/m^2. This is very light for structures with roof spans up to 70 metres.

Docklands Stadium Roof, Melbourne

Introduction

The Docklands Stadium, currently nearing completion alongside the Melbourne CBD, features one of the largest retractable roofs in the world.

Essentially, it has been designed as an open stadium which can then be closed to operate as a multi-purpose venue. Although the main sport will be AFL Football, the arena will also host both soccer and rugby. To facilitate this, the lowest tiers include retractable seating stands, which can be moved up to 18m forward taking spectators close to the rugby/soccer pitch. Covered seating for in excess of 52,000 spectators is provided.

The roof, with a clear span of 165m, is the main structural element which drives the rest of the stadium design.

A fundamental requirement of the roof design is the need for a natural turf pitch which requires maximum natural light on the playing surface as well as an engineered pitch sub-grade system. This requirement dictated that the roof opening should be as large as possible and that the roof profile should be kept low to minimise shadows on the pitch.

Figure 11 shows the plan of the stadium which is formed by a combination of two main radii with a major axis dimension of 245m and a minor axis dimension of 215m.

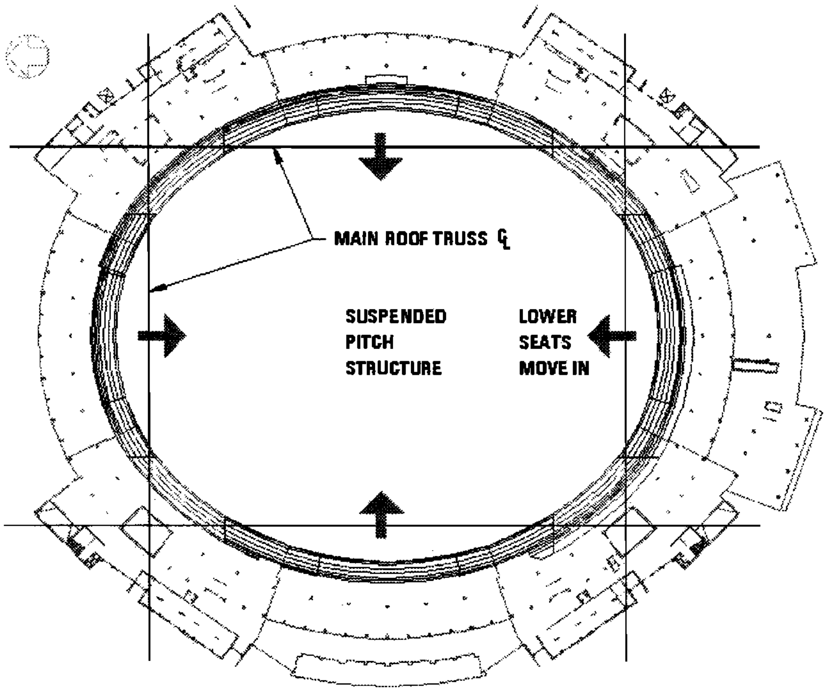

Figure 11: Stadium Plan

Roof Options

With the decision that the roof should be retractable, options were conceived and developed to find a solution which minimised these costs. Not just economy but lightness of the roof structure was a primary engineering objective particularly as the Stadium needed to be extensively piled due to exceptionally poor ground conditions consisting of 30-40m thickness of Coode Island silt. This, together with the opportunity of creating a complete ring structure in which lateral component forces generated in transferring vertical shears balance, suggested a shell or tensegrity structure. In order to operate in the closed concert mode, passive acoustic characteristics were required of the roof which essentially precluded the use of fabric structures, even for the moving section. The need to maximise the incidence of natural light on the pitch was considered vital to the facility's future operation as well as the avoidance of distinct shadows which might interfere with play, or most importantly, television coverage. In addition to requiring a very large opening there was also preference therefore that the fixed roof and parked moving sections be as low as possible with any supporting structure above the roof angled back to reflect sun paths and no substantial fixed structure across the infield. Roof drainage considerations favoured a domed, as opposed to a dished geometry, as the latter, if symmetrical, would rely on a mechanical system requiring ongoing maintenance.

From these considerations the following options were born.

Option 1: Toroidal Shell
This scheme used a segment of the surface of a torus to give a geometric logic, which respected the elliptical plan of the stadium, to a shell or compression (under gravity) surface. Relatively efficient shell action could be achieved for the fixed portion of the roof with a compression ring at the inner boundary whilst the moving roof panels necessarily involved conventional truss design.

The geometry born naturally out of that of the bowl helped with overall aesthetics. However, to improve efficiency, which was marred by the scale of the opening, as much curvature as possible was required reducing the amount of light striking the pitch. Also the curved path of the moving roof segments necessitated a fairly complex motive system, here involving rack and pinion drives, which could increase operational risk and ongoing maintenance requirements.

Option 2: Shell Roof with Fabric Iris
This scheme investigated an innovative way of closing the roof over the infield using folding trusses and fabric rather like the webbed wings of a bat.

The fixed roof is again a shell with an inner compression ring and from this ring spring a series of steel trusses hinged at their ends about a vertical axis. In the closed position trusses cantilever out up to 50m with the end moments taken back through the structure of the shell. Between the trusses a structural fabric is kept taut using a valley cable stressed with hydraulics at the truss tips. By rotating the trusses in plan until they fold against the inner compression ring the roof opens and throughout this operation tension is maintained in the fabric by hydraulically controlling the tension in and position of the valley cable.

More active acoustic control was required with this scheme to reduce noise break out and this coupled with the use of an approach untried on this scale were contributing factors in the decision not to pursue this scheme.

Option 3: Space Truss Scheme
This third scheme is a space frame truss design. The roof scheme comprises four main trusses which frame the 160m x 100m opening above the pitch and which support the fixed roof secondary trusses spanning from columns at the rear of the upper tier.

In order to minimise the roof self weight, and provide the necessary stiffness, a combination of deep bow string and tied arch trusses have been adopted. (Fig: 12).

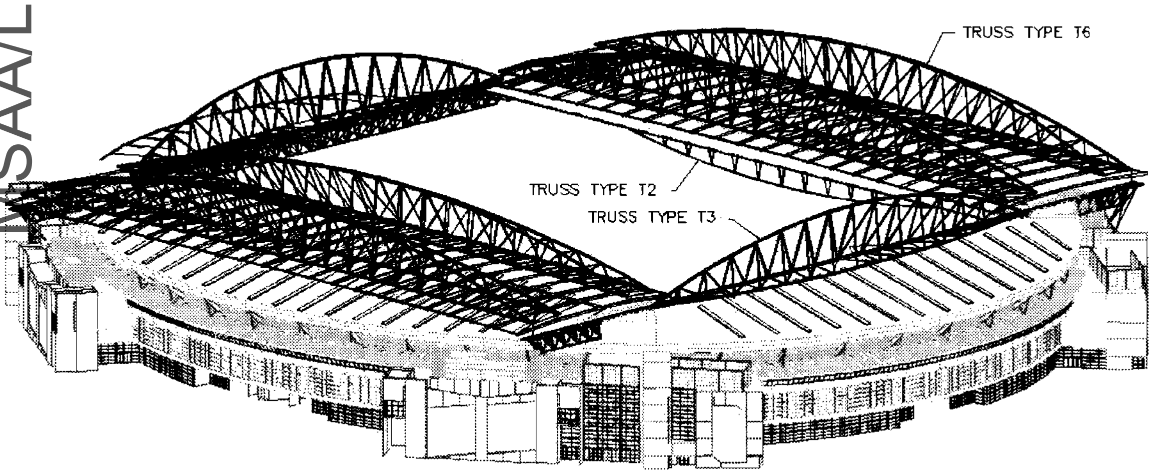

Figure 12: Space Truss Scheme

The moving roof sections each measure 50m wide and span 165m between the fixed roof trusses type T3. The moving roof panels are underslung from twin 3-D bow string trusses. (Fig: 12).

The truss T3 which also supports the moving roof and the north and south fixed roof sections, is in the form of a tied arch which is 14m deep spanning 120m between the four corner core structures. These cores provide the principal stability to the building, house the main service risers and support the video display scoreboards. This is an efficient system as the roof loads are primarily carried by compression in the top chord member acting as an arch.

Buckling of the top chord member is a major consideration. Bucking is restrained by the fixed roof secondary trusses T1 which span back to bearings at the rear of the stand.

The east and west side of the roof opening is framed by two bow string trusses type T2, which have an underslung bottom chord tension member in the form of a catenary below the soffit of the secondary trusses.

The fixed roof secondary trusses are conventional designs spanning up to 40m with a 15m cantilever.

The main attributes of this scheme included the following:

- Roof opening matches the fixed seating configuration
- Flat roof profile maximising natural light incidence on the pitch;
- Horizontal crane rail beam enables wheel driven bogies and avoids the need for cable driven or rack and pinion driven system.

The Space Truss Scheme was selected as the most cost effective option and developed by the Consultant Team as the Exhibited Design for BOOT tendering, and subsequently adopted in principle by the successful tenderer.

Conclusion
The Victoria Stadium roof will be one of the largest of its type in the world when it is completed in early 2000 and will provide a world class facility close to the Melbourne CBD. Even with substantial 200m long trusses necessary to support the massive moving panels, the structural steel weight of the roof still only averages around 80kg/m^2.

Sydney's Railway Square Glazed Canopies

Introduction
The glazed canopies at Railway Square realise the architect's concept of a structure that communicates the idea of shelter, arrival and gateway. The main canopy is a covering of glass with a finely engineered steel structure. The canopy comprises two roofs of regular plates portraying the dynamics of arrival.

Structural Systems
The architect's ideas for the shelters were broken down into two principal forms – the main bus canopy and the bus shelters. The structural form for both canopies is similar. (Fig: 13).

Figure 13: Completed Canopy Structure

The canopies are totally glazed and the supporting structure has been engineered to have minimal impact on the glazing. The design is also characterised by the obvious lack of bracing. (Fig. 14).

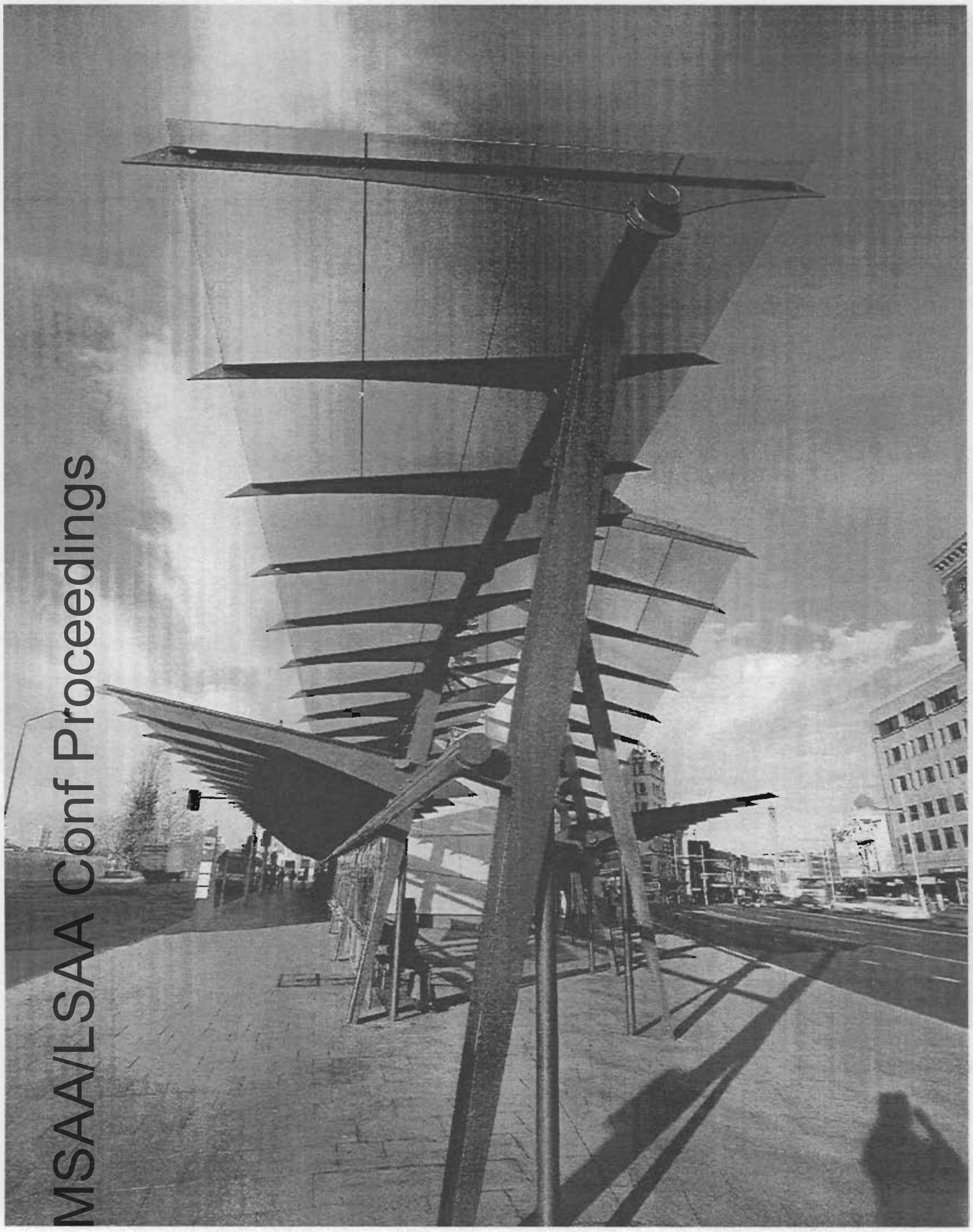

Figure 14: Canopy with Complete Lack of Bracing

The solution developed by the architect and structural engineer consists of a series of cantilever blades spanning up to 4m, supporting glazed panels. The cantilever glades are supported by torsional tubular girders, which in turn are supported by a series of inclined 250 UC columns which are tapered towards their bases. The columns are stabilised in their sloping plane by vertical 102
CHS props. The overall overturning forces are resolved into bending moments and axial forces (both tension and compression). This approach forces leads to a minimisation of member sizes compared to other approaches. (Fig. 15). Stability in the longitudinal plane is provided through portal action, with moment connections between the columns and tubular girders.

Figure 15: Details of Slender Cantilever Blades

The dominant design load case for the blades is downward load (dead and wind) for both strength and serviceability, and overall movement prediction and limitation was critical to the success of the design. Extensive computer studies were carried out to determine overall frame displacements and forces by both non-linear and finite element analysis techniques. The blades taper parabolically from the tip to the support, reflecting strongly the non-linear increase in stress. The slenderness of the cantilever blades is less than half that of conventional cantilever design.

In recent times the appearance of a number of glazed structures has been devalued by the presence of bracing and large numbers of glazing bars. On this project a very deliberate approach was taken to avoid this. The structure, while achieving in-plane stability through strut-tie action, adopted a completely different approach for out-of-plane stability. In the end bays of each canopy, glazed panels are rigidly bolted to the blade sections so that horizontal forces are transferred to the steel structure by diaphragm action. The result is the total elimination of cross bracing, enhancing greatly the overall appearance of the canopies. (Fig. 16).

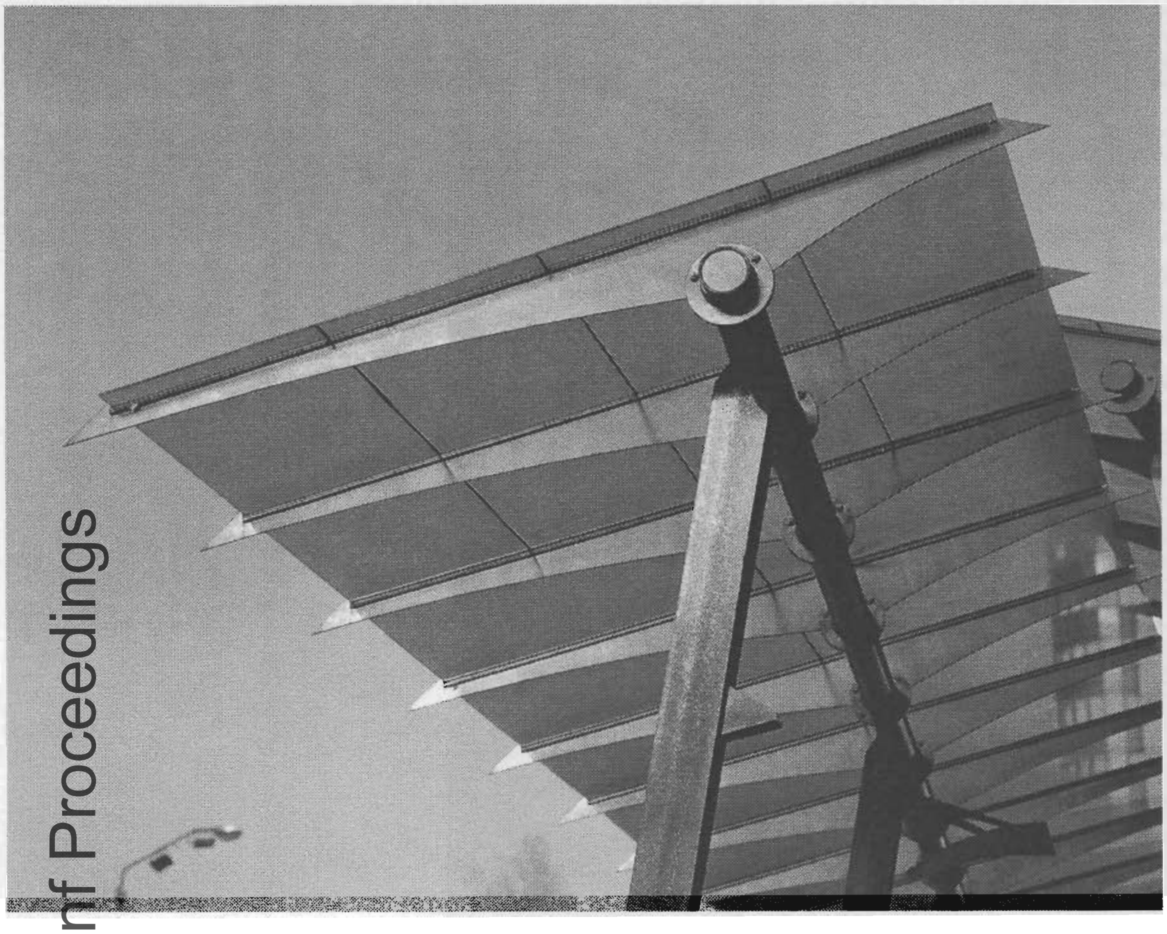

Figure 16: Details of Cantilever Blades, Torsional Element and Support Structure

Construction

Fabrication of the overall structure, tolerances and their relationship to the final geometrical layout was absolutely crucial. Computer studies indicated that the slightest lack of fit or fabrication errors would lead to considerable misalignment of the cantilever blades. Expected movement diagrams were included on the drawings for the builder's information. The documentation also reinforced the precision of tolerance and fit between various components.

Once various elements had been fabricated, trial assemblies were carried out by the builder to verify the adequacy of the bolted connections. Any slip in joints connecting the blades to their torsional elements would lead to significant unplanned movements. As a result of trial assemblies, changes were made to a number of bolted connections, both bolt size and configuration to minimise slip in the final structure.

Adelaide's Rundle Mall Canopy

Introduction

The prominent feature of the current upgrade of Adelaide's Rundle Mall is the sculpture-like, glass canopy at the intersection of Rundle Mall with Gawler Place (Fig: 17). Th0is highly three-dimesional structure is set on four tree-like columns (Figs. 17 and 18) branching out to support four circular roof quadrants, which are sloped towards a common focal point. The "spider web" appearance of the canopy is maintained by using very slender steel fins supported by circular blade-type girders. A tensile fabric shade structure is also installed to the underside of the glazed canopy.

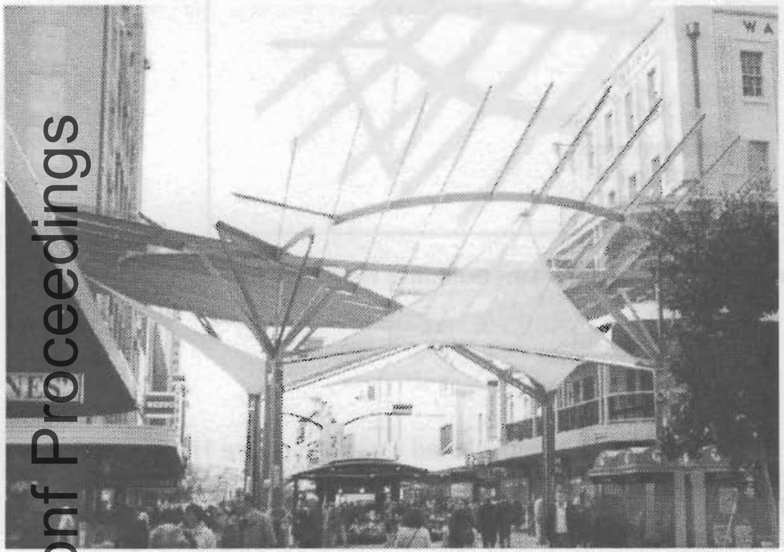

Figure 17: Rundle Mall Canopy

Structural System

The many loading combinations, including dead, live and wind loads as well as tensile forces imposed by the shade structure, necessitated extensive computer analyses to resolve the final geometry and to determine minimum member sizes. The selection of member sizes was a balance between producing elements with least visual impact on one hand, and controlling stresses, deflections and movements on the other.

Accurate assessment of the structural response to the imposed loads was obtained using advanced finite element analysis of a typical quarter of the canopy. This was required since the conventional code-specified design rules could not justify the adequacy of the members having the (architecturally) desired level of slenderness. Of particular importance was to determine accurate flexural and lateral-torsional buckling capacities of the fins, circular girders and tree branches. Figure 19 depicts the buckling modes of the critical tree branch and fin members, which together with the corresponding numerical values were used in member design.

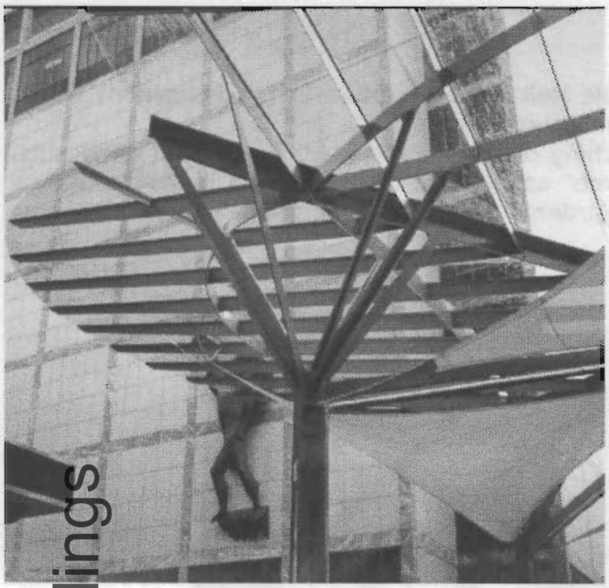

Figure 18: Rundle Mall Canopy and Column Trees

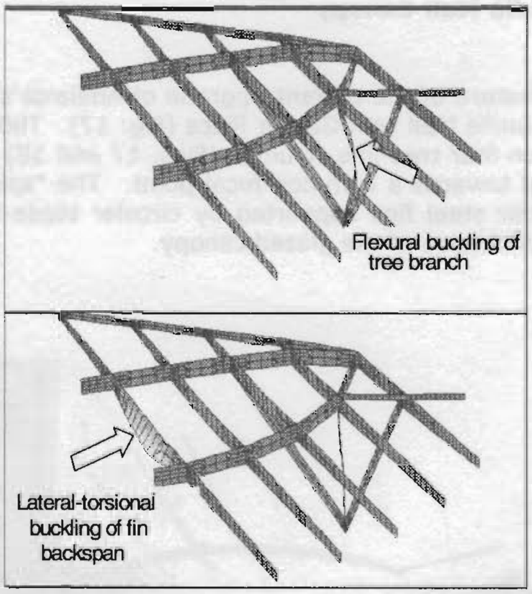

Figure 19: Buckling Modes of Canopy

Sydney's Capital Square Galleria

Introduction

The Capitol Square Galleria is a 20m high, 880m^2 atrium linking the refurbished Capitol Square Hotel building to the Capitol Theatre in Sydney, and housing a retail centre solely dedicated to digital shopping. The structural system is a fully glazed, steel framed roof and wall system (Fig. 20), which has been developed to reflect the high tech, futuristic functional aspect of the Galleria. A frame of RHS mullions and transoms, which hang from the roof, support the glazed walls.

Atrium Structure

The roof structure consists of diagonally arranged bowstring girders spanning 20m, and RHS purlins holding the glazing frames. The girders are supported at one end by the concrete structure of the Capitol Square building and at the other by the raking, diagonally framed tapered columns adjacent to the Capitol Theatre wall. The column frames provide lateral stability for the structure in addition to a unique appearance. Their design was partially driven by the presence of the Capitol Theatre wall and its significant heritage value. This prevented any form of connection to the wall and demanded minimum visual obstruction by the columns. The diagonal arrangement of the girders however stabilises the roof in its plane, and avoids the need for visually undesirable bracing elements.

The process of structural design of the steelwork has incorporated a high level of attention to aesthetic proportioning and detailing of members and connections (Fig: 21).

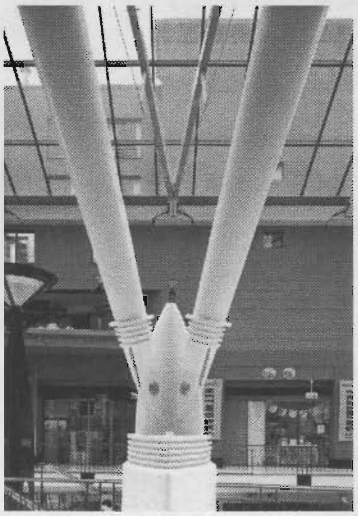

Figure 20: Capitol Square Galleria – Atrium Structure

Figure 21: Raking Columns

60 Castlereagh Street, Sydney - Glass Feature Walls

Introduction
The two Glass Feature Walls in 60 Castlereagh Street, Sydney, form part of the transparent entries to the building at Castlereagh and Elizabeth Streets.

Each Glass Feature Wall is approximately 9m wide and 6m high. The glass walls enclose the entrance atria above revolving doors and are designed to provide maximum transparency. A key design requirements was to maximise effect at minimal cost. For the client and tenants, the result is a visually striking gateway through an "invisible" shield into a calm interior. (Fig: 22).

Maximum transparency is achieved by the use of minimal support structure. The glass walls are supported against wind loads by a series of single horizontal stressed cables. Vertical rods resist the dead load of the glass and the door head beam. Horizontal and vertical elements meet at the glazing nodes to support the glass. The overall structural depth is less than 200mm and the cables and rods are 32mm in diameter. This planar stressed glass support structure is the first of its kind in Australasia.

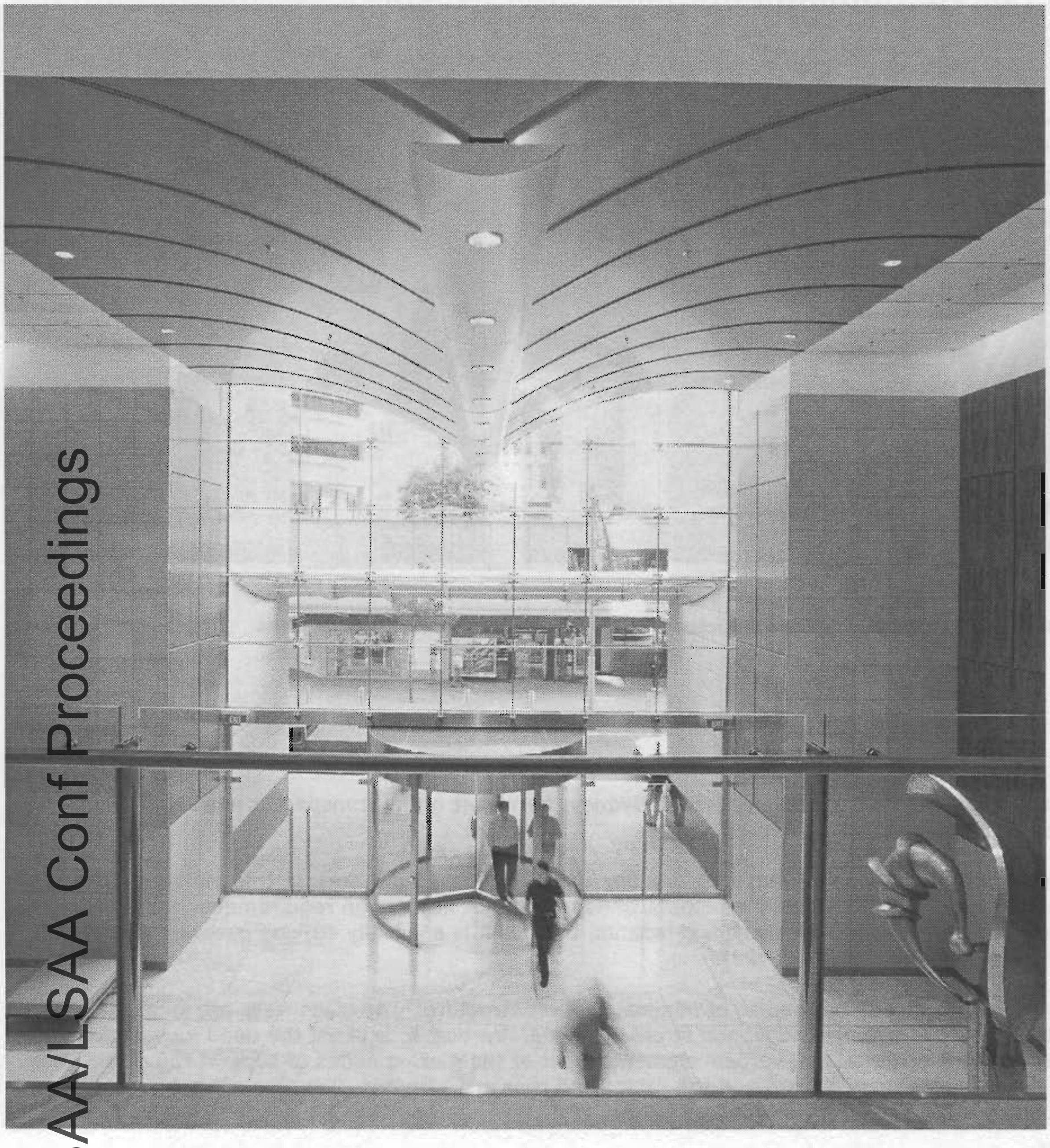

Figure: 22 Castlereagh Street Entrance

Structural Systems
The main load case on the large area of glass is from wind pressure. The out-of-plane wind loads are resisted by a series of tensioned horizontal cables.

In a number of recent projects, transparency has been 'achieved' using twin tensioned bowstring trusses. While each bowstring truss is designed to resist either positive or negative wind loads, a straight cable resists wind loads from both directions.

A planar cable system is more flexible than a bowstring system as it does not have the same depth or catenary shape. Accordingly, the required pre-tension in a horizontal cable system is higher than for a bowstring system. Deflections are also higher, a major design consideration for the glass and glazing connections. Given the location of the glass walls above the entry doors, large deflections can be tolerated.

With the required high pre-tensions in the horizontal cables, the support structure at each end of the cables needs to be significant. The existing structural layout of the building included concrete columns adjacent to the glass walls. These provided the support. The selection of the tensioned horizontal cable support system was appropriate, given the preexisting columns and the moderate opening of 9m. No additional cost for these dedicated supports was incurred.

The structural design justification of a 'simple' horizontal cable is not straightforward. A geometric, non-linear analysis of the cables was carried out, and multiple load cases were considered. These included wind load, thermal load, support deflections (elastic and creep), cable pretension and cable relaxation. The strength of the system was determined by the load case of a cold, windy day just after construction (maximum wind load, negative thermal load, no support creep and no loss of cable pretension). The serviceability of the system was determined by the load case of a hot, windy day 30 years after construction (maximum wind load, positive thermal load, support creep and cable pretension relaxation).

Vertical hanger rods support the dead load of the glazing and the door head beam. They are located close to the plane of the glass and are aligned with the glass joints to minimise visual impact. The tensioned horizontal cable counterbalances the eccentricity of the glass dead load about the centreline of the hanger rod. The overall structural depth is less than 200mm and the cables are 32mm in diameter. The comparable depth of a bowstring truss would be approximately 500mm.

Innovative material selection of the main horizontal cable was made by the specialist glazing subcontractor. To minimise the cable size, a high strength material is required. High strength stainless steel rods are expensive and delivery lead times can be a problem with certain sizes. A standard 25mm diameter, high strength, stressing bar was used. This was clad in a light gauge 32mm diameter stainless steel welded tube. The strength of the inexpensive standard rod and the aesthetics of the inexpensive stainless steel tube were combined to achieve the desired result in a cost-effective manner.

The same 32mm diameter stainless steel welded tube was used for the dead load hanger rod. While the diameter used was larger than required for strength, it was acceptable aesthetically and allowed for easy erection. Vertical tolerance adjustment was easily accommodated as the thin-walled, stainless steel could be site drilled on the node connections.

The nodes used in typical bowstring truss designs are stainless steel castings. While these allow flexibility in design and aesthetics, they are expensive and have a long delivery lead time. Nodes of fabricated stainless steel plate were used on the project. Finishing and careful finishing of welds achieved the required architectural appearance. Because of the catenary shape of bowstring trusses, they generally have several different geometries within any installation. In the planar system, all node geometries are identical achieving further savings.

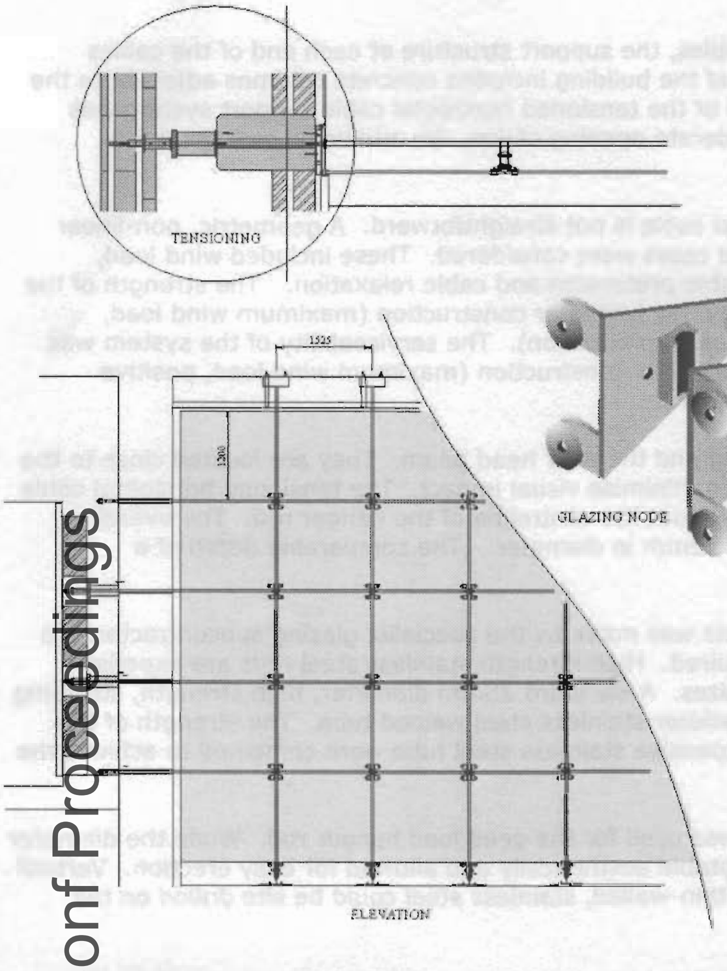

Figure 23: Glass Wall Details

Glazing
The glass used on the feature walls is 12mm toughened. As stress concentrations can occur in glass at patch fitting connections toughened glass was selected for strength. A glass module width of 1280mm was selected to suit existing architectural modules and to enable the use of locally available 12mm glass. A wider module would have meant that disproportionately more expensive, imported, 15mm toughened glass would be required.

A critical element of the glass design is the patch fitting. For large displacement support structures, a 'rigid' patch fitting connection results in excessive glass stresses. Articulated ball joint patch fittings are traditionally used for engineered glass systems. However, these fittings can be expensive. A less expensive, 'semi-rigid' patch fitting connection, based upon relatively thick, soft silicone pads, was used on the project. This was carefully analysed using a brick element FEA model. The local stresses in the glass at the patch fitting were found to be only marginally higher than for an articulated ball joint patch fitting. Overall glass deflections were also similar to a 'pinned' connection analysis. As a result, a full plate element FEA model assuming 'pinned' connections was then used to check the effects of differential support-imposed corner displacements on the glass. The simplified patch fitting saved considerable money on the componentry for the project.

Construction
The method used for tensioning the cables on the project was simpler, and less expensive, than conventional jacking. Thin-walled tubes were installed at the end of each rod and the rods tensioned, using a standard jack. Compression of the tubes was accurately measured using a dial gauge. Each tube was tested and calibrated prior to installation so that a measured displacement can be related to a known compression force. Each rod can be incrementally tensioned to minimise the effect of differential support deflection. The tubes are left in place so that rod tensions can be checked throughout the life of the structure.

Conclusions

Connell Wagner's recent experience in designing various lightweight structures was described in this paper. The selected projects demonstrate how structural and architectural engineering principles can be employed to achieve buildings of sophisticated design as well as great elegance and economy).

Acknowledgements

The authors wish to acknowledge the contribution of the following architects to the success of the projects described in this paper:

- Ancher Mortlock and Woolley for Sydney State Hockey Centre
- Peddle Thorp Architects for Melbourne Sports and Aquatic Centre
- Daryl Jackson Architects Pty Ltd/Bligh Lobb Sports Architecture for Melbourne's Docklands Stadium
- Noel Bell Ridley Smith & Partners for Sydney's Capitol Square Galleria and Railway Square CanopiesSteve Grieve Architects Pty Ltd for Adelaide's Rundle Mall Canopy
- Scott Carver for 60 Castlereagh Street, Sydney