Lattice Shell Roof of the Sydney International Aquatic Centre

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1. INTRODUCTION

The Sydney International Aquatic Centre (Fig. 1) has been constructed at Homebush Bay, Sydney, to function as the main venue for the whole range of aquatic competitions (swimming, diving, water polo and synchronised swimming) in the 2000 Olympic Games. It includes four swimming pools: a main competition pool, a utility pool used for diving and water polo, a training pool, and a large free form pool for public use. In addition to the pools, the building incorporates accommodation and facilities for competitors, officials, vendors, VIP's and sponsors.

The facility has been planed to have two modes of operation: (a) permanent mode; (b) Olympic mode. The base building has been designed and constructed for the permanent mode with 4,400 seating capacity. The design however allows for the expansion of the building to the Olympic mode, which seats a total of about 17,500 spectators with uninterrupted views of the pools, and scoreboards during the 2000 Games.

Connell Wagner was the structural engineer for the base building, and is currently involved in the design of the Olympic mode expansion.

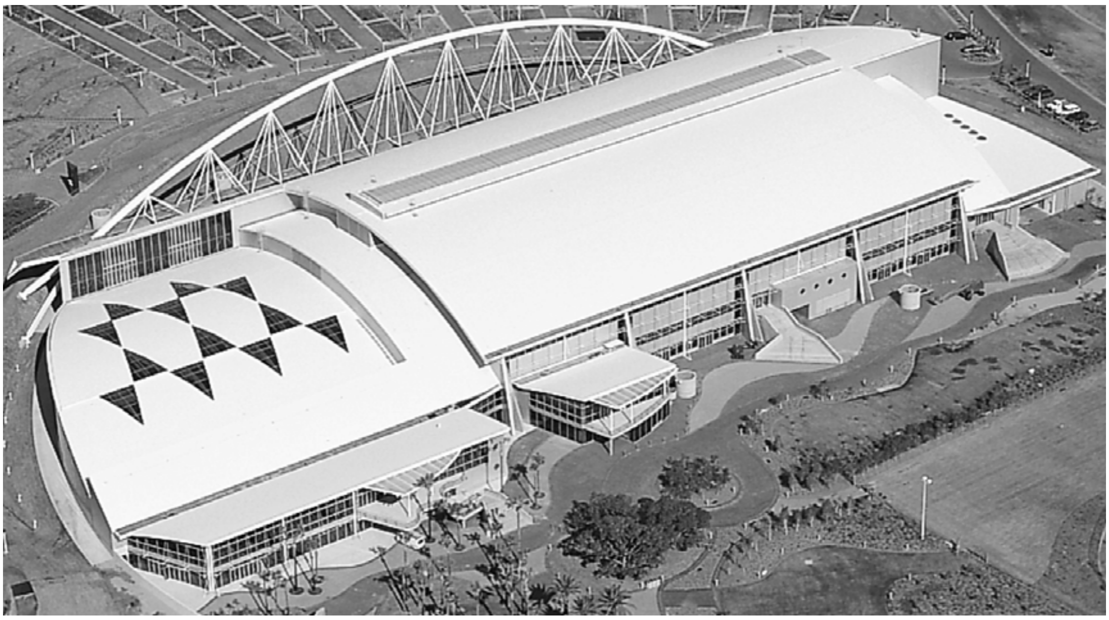

Figure 1. The Sydney International Aquatic Centre

The centrepiece structure for the whole facility is an innovative dramatic lattice shell roof covering an internal column free space of 100m×67m (Figs. 2 and 3). This paper describes the structural engineering of this roof.

Figure 2. Lattice Shell Roof of the Sydney International Aquatic Centre (view towards North)

2. STUDY OF ALTERNATIVES FOR THE ROOF STRUCTURE

The basic planning of the facility was determined by the need to limit the height of external facades for economic reasons, provide an acceptable height at the centre of the hall, and allow uninterrupted views to the pools and scoreboards during competition. These constraints resulted in the selection of a curved roof profile with a clear span of 67m (Figs. 1, 2 and 3).

Prior to, and during the development of the various possible structural solutions for spanning the main hall, an extensive literature survey was carried out to examine solutions for this type of roof, which have been recently used elsewhere in the world. There have been numerous halls of similar dimension to the Sydney International Aquatic Centre and equally numerous structural solutions for their roofs. Most of the solutions are integral with the architecture, and are governed by the required volume internally. They are also influenced by local practice, available technologies, materials, components etc, so that many of the solutions which have been adopted overseas could only be pursued in Sydney if substantial importation of materials and technology were to occur. A good example is the proprietary system space structures in Europe.

Figure 3. Lattice Shell Roof of Sydney International Aquatic Centre (view towards South-East)

Three families of structural forms, capable of spanning the hall of the Aquatic Centre, were investigated. They included: (a) arch structures with internal ties or external buttresses; (b) flexural systems; (c) cable stayed structures. More than ten alternative structural forms were initially analysed (Figs. 4 and 5). This was then narrowed down to a more detailed analysis of three structural systems, namely, a tied arch, a cable suspended structure, and a beam and truss system. The economy of these proposals was analysed by extensive parametric studies and then confirmed through consultation with the fabrication and erection industry who were asked to provide detailed pricing input.

These studies demonstrated that a tied arch was a more efficient means of spanning the 67m distance than a flexural truss system. A more economical structure was possible using a tied arch, as long as out-of-balance wind or live loads, which caused bending in the arch and contributed to premature buckling, could be contained. Also, the shallow depth of the arch structure had minimum impact on sightlines to scoreboards and diving tower, and permitted a slightly lower roof than other schemes.

The cable stayed roof was also very economical, particularly as the main roof girder could be used to resist the wind load in catenary action against uplift, the Achilles heel of cable suspended roofs. However, the cable-stayed roof was more difficult to erect than either a tied arch or a truss roof, and this resulted in its elimination.

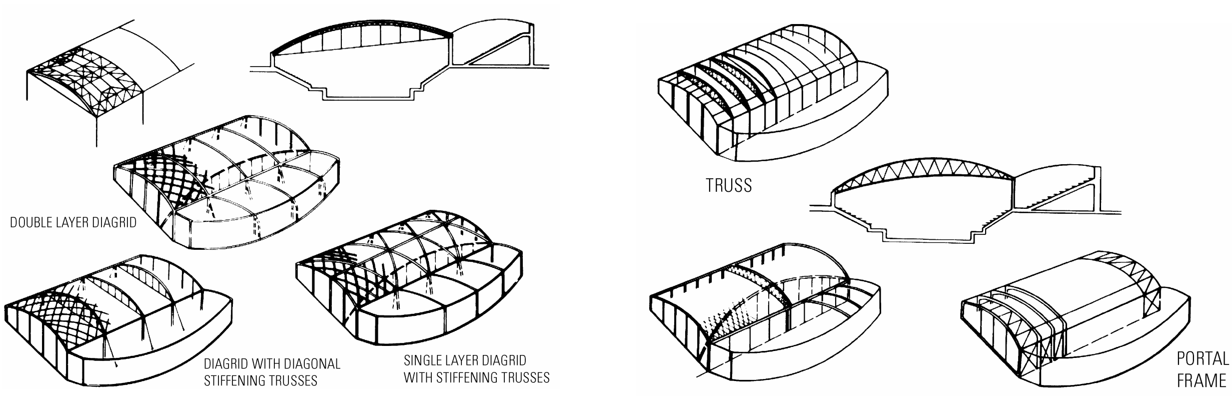

Figure 4. Alternative Arch Structures and Flexural Systems for the Roof

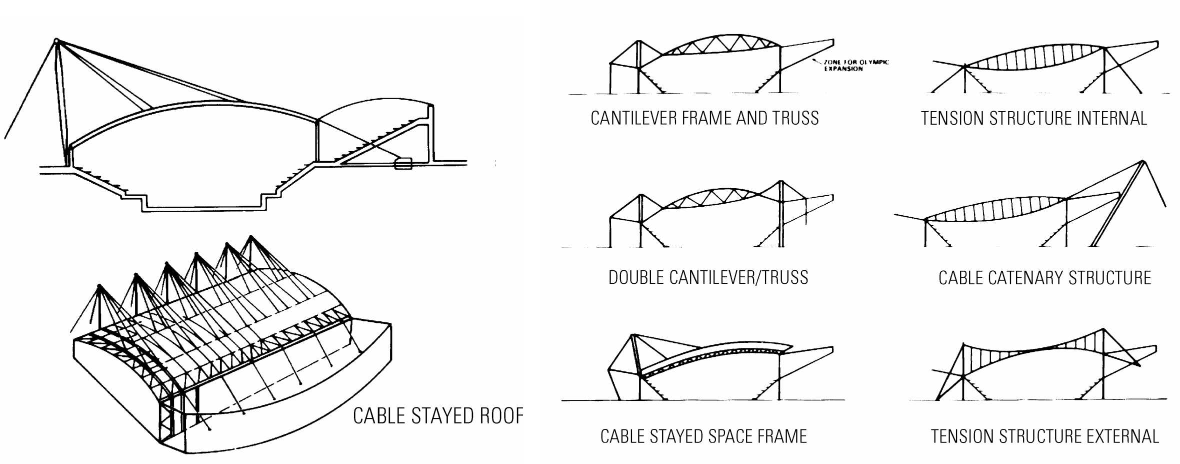

Figure 5. Cable Stayed Alternatives for the Roof

3. THE DIAGRID SHELL

A very interesting type of cylindrical shell has been developed which falls into the "lattice" family of structures, and has been called "diagrid". The diagrid shell can be likened to a series of arches with members running on uniform diagonals, 6.3m apart (Fig. 6). While the diagrid resists uniform vertical loads through its arch action, stiffening ribs are provided to prevent the global and "snap-through" buckling of the arch. These stiffening ribs, which are constructed as double-layer shell (1m deep) and are positioned at 25m centres, also provide the required flexural stiffness and strength to resist non-symmetrical loading conditions. The diagonal arrangement of the shell members automatically provides all the bracing to the roof and the membrane action necessary to affect the stiffening ribs actions. This enables the roof to behave as a stiffened shell, and significant economy of material and lightness of appearance is gained. The members of the lattice shell structure are only 270mm deep, which is extraordinarily light for a span of 67m.

The opportunity to use the natural curve of the roof to span the hall using shell action has been fully utilised. With a rise of around 9m, a very economical shell structure has been designed which uses significantly less material than a system using trusses. Furthermore, the adoption of diagonally aligned members has led to a uniform and repetitive structure, where members are straight, of identical length and generally identical size. All members of the diagrid are 273mm CHS sections with wall thicknesses ranging from 6.4mm through to 28.6mm. The latter thicker wall tubes, which had to be imported, were used at limited highly stressed locations.

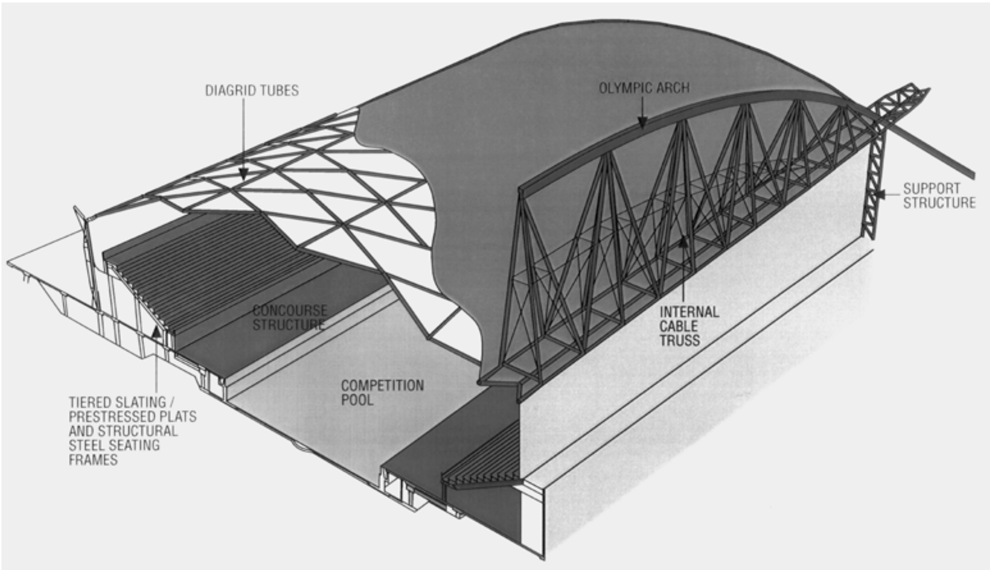

Figure 6. Sydney International Aquatic Centre Roof Cutaway

Intersecting ties have been provided at 25m centres across the roof span to resist the lateral thrust of the shell structure under downward (dead, live and wind) loadings (Fig. 7). These ties are 50mm diameter bars, stressed to approximately 600kN to control movements, and fully grouted inside a special sleeve for corrosion protection. To resist uplift, the shell structure goes into tension as a catenary with an external tie provided to complete the catenary action.

4. THE OLYMPIC ARCH

The design of the Aquatic Centre roof structure had to accommodate for the future expansion providing extra seating for the Olympic mode. This led to the adoption of stabilising buttresses on one side of the roof only, freeing up the other side for future expansion.

The Olympic mode requires a clear span structure of 138.6m along the east side of the building because columns are not permitted in the viewing area. This clear span has been provided using an arch structure (Fig. 8). The arch enables the external wall of the building to be completely removed in the Olympic expansion mode and additional seating to be constructed outside the building envelope without major strengthening of the structure, difficult removal of supporting columns, etc. The wall will be reinserted again after the 2000 Olympics, reverting the building back to its desired regular format.

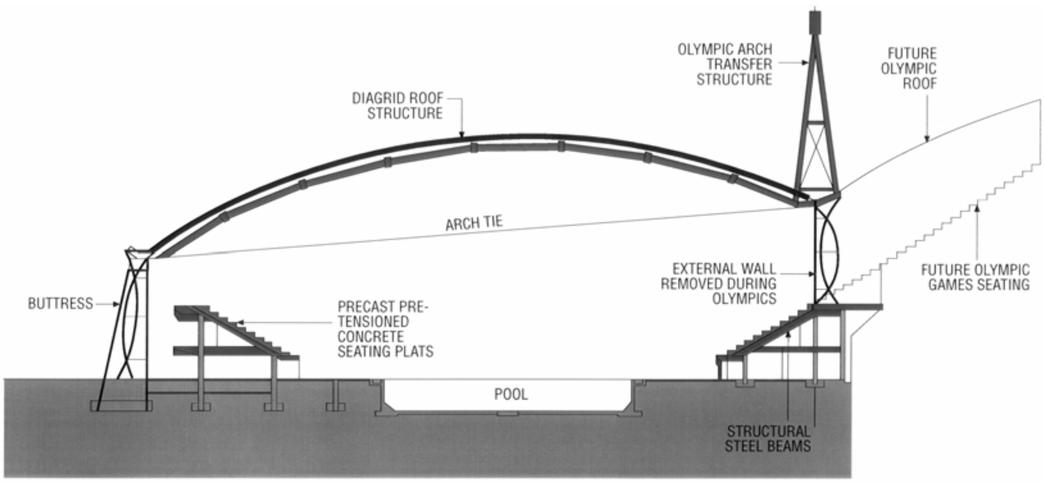

Figure 7. Sydney International Aquatic Centre Cross-Section

Figure 8. The Olympic Arch of the Sydney International Aquatic Centre

The arch is a fabricated 1000mm deep, 500mm wide steel box, which is stabilised against buckling and stiffened for non-uniform loads by a network of small diameter tubes (Fig. 8). These tubes, which are up to 19m long, also transmit the up and down loading from the roof to the arch. Thus, they are in axial compression under wind uplift, and hence prone to bucking. A light cable system has been positioned at approximately mid-height to reduce the buckling length of the tubes. This resulted in the size of members being reduced from 356CHS (55kg/m) to 219 CHS (34kg/m) and an overall reduction in weight of some 50 tonnes compared with early tied arch schemes.

Specific measures were taken to ensure that the arch was as flat as possible following construction so that sightlines were not compromised. This was achieved using a cambered structure where the profile was determined from not only the final structural configuration but also computer simulation of the construction sequence. This methodology was extremely successful with Olympic arch finishing virtually horizontal.

5. STRUCTURAL MODELLING, ANALYSIS AND DESIGN

Due to the extreme lightness of the roof, it was found necessary to conduct wind tunnel testing using an aerolastic model of the roof, in order to evaluate the dynamic sensitivity of the structural response, and generate appropriate design wind load cases. The wind tunnel test results showed that maximum responses of the roof were not dynamically induced. Nevertheless, significantly different global design wind pressures were obtained when compared with the pressure tap model, which was used to measure wind pressures for design of the cladding.

Preliminary analyses were based on consideration of the roof as an equivalent steel shell continuum. Stiffening ribs across the shell were also included. Estimated sizes from these analyses were used for initial costing purposes and as the basis for a preliminary space frame analysis using the finite element programs MSC NASTRAN and STRAND.

Design for buckling of the lattice shell structure was given particular consideration. Several modes of buckling were studied, including individual members buckling, snap-through buckling of a local area of the shell, and overall buckling of the roof. Initially, an eigen-value buckling analysis was undertaken which was followed by a full nonlinear large-displacement analysis of the roof under progressive application of load using the finite element program ANSYS. These analyses were carried out for both symmetrical and non-symmetrical load cases.

A non-linear analysis was also used to investigate the influences of local variations from the theoretical geometry of the structure (geometric imperfections). The results of these analyses demonstrated that a factor of safety of close to 3 existed with respect to buckling.

The second order non-linear analysis results were used to modify the multi-load case linear elastic analyses to provide design actions. Post processors were developed to automatically combine the various load combinations and compare member design forces with member capacities; all design being carried out in accordance with AS4100. Non-uniform wind and live load cases were significant in determining required member sizes.

6. CONNECTIONS AND CASTINGS

An extremely important design consideration was the detailing of all connections, which are exposed to view. In many cases models of critical connection were made to examine their ease of fabrication, quality of workmanship and appearance. Extensive use has been made of single pin connection with profiled ends to the circular structural members. Close co-operation and understanding between Connell Wagner and the architect was crucial in achieving efficient yet elegant connection details.

An interesting feature of the design of the roof has been the use of structural steel castings for the connection nodes of the diagrid members. These are in the shape of a cruciform and provide a simple, repetitive and aesthetically pleasing connection. Castings have potentially great economy where large numbers of difficult shapes are required, as in the case to the Aquatic Centre. Finite element studies using the computer program MSC PAL were carried out in conjunction with full scale static, and dynamic testing to confirm stress distributions, ductility and strength of the castings under fluctuating loads. This was particularly necessary at the location of the holes, which are used for inserting and tightening of the bolts, as significant stress concentrations were of concern. The castings were constructed from grade 350 steel and were then subject to extensive quality control to confirm the metallurgical composition of each casting prior to acceptance.

Initially, a range of wall thickness varying from 12mm to 20mm was proposed for the castings. However, following tests indicating the possibility of tearing during cooling and subsequent confirmation of thermal stresses by finite element studies, wall thicknesses were rationalised to 20mm for all castings.

7. DURABILITY

Stadia are particularly vulnerable to corrosion as the structure is invariably substantially exposed to the weather. In the case of the Aquatic Centre the internal environment is also corrosive due to the atmosphere in the pool areas.

On this project (and most of our recent projects) the steel has been protected against corrosion by a three coat system of inorganic zinc silicates, high-built epoxy and catalysed acrylic gloss, applied in the fabricating shop to a total dry film thickness of 340 microns. Hot-dip galvanising throughout was not practical due to large member sizes and hence the cost. However, the connection plates, castings, and assembly bolts and pins were hot-dip galvanised and then painted for additional protection. This accounted for the fact that the connection zones are most prone to damage during erection and are most difficult to repaint.

A high level of quality control and review was achieved by regular inspection by independent third parties during fabrication of steelwork. Specific procedures were also developed by the fabricator and a supplier to enable touch up work to be carried out on site without comprising the longevity of the paint system.

8. CONSTRUCTION

In keeping with the innovative structural engineering solutions developed for the roof structure, a unique methodology was developed and successfully implemented for the construction of the diagrid and Olympic arch. The construction method adopted was the incremental launching technique, and while popular for bridge construction, it was first for a roof of this size in Australia. This method involved the construction of an erection bay outside the building and progressive construction, stressing of the internal ties, and monitoring of movements during launching. The roof was launched in 25m segments using specially fabricated roller skates. The roof segments were simply pulled into position by winches.

The significant advantages of this scheme were that no scaffolding was needed in the pool hall itself, and work was able to continue on the pools, and all substructures without interruption during the entire period of roof erection. This was a huge advantage from time/cost and safety points of view. Construction proceeded extremely well with the building completed in May 1994, 3 months ahead of schedule.

9. CONCLUSIONS

Different aspects of analysis, design and construction of the roof structure of the Sydney International Aquatic Centre was described in this paper. The fact that the facility had to be designed for the permanent mode and yet be capable of expanding for the Olympic mode imposed many design challenges. Connell Wagner engineers in a close alliance with other members of the design team responded to these challenges with their innovative solutions. The result was an efficient, buildable and elegant lightweight roof structure, which is worthy of hosting the Sydney 2000 Olympic Games.

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