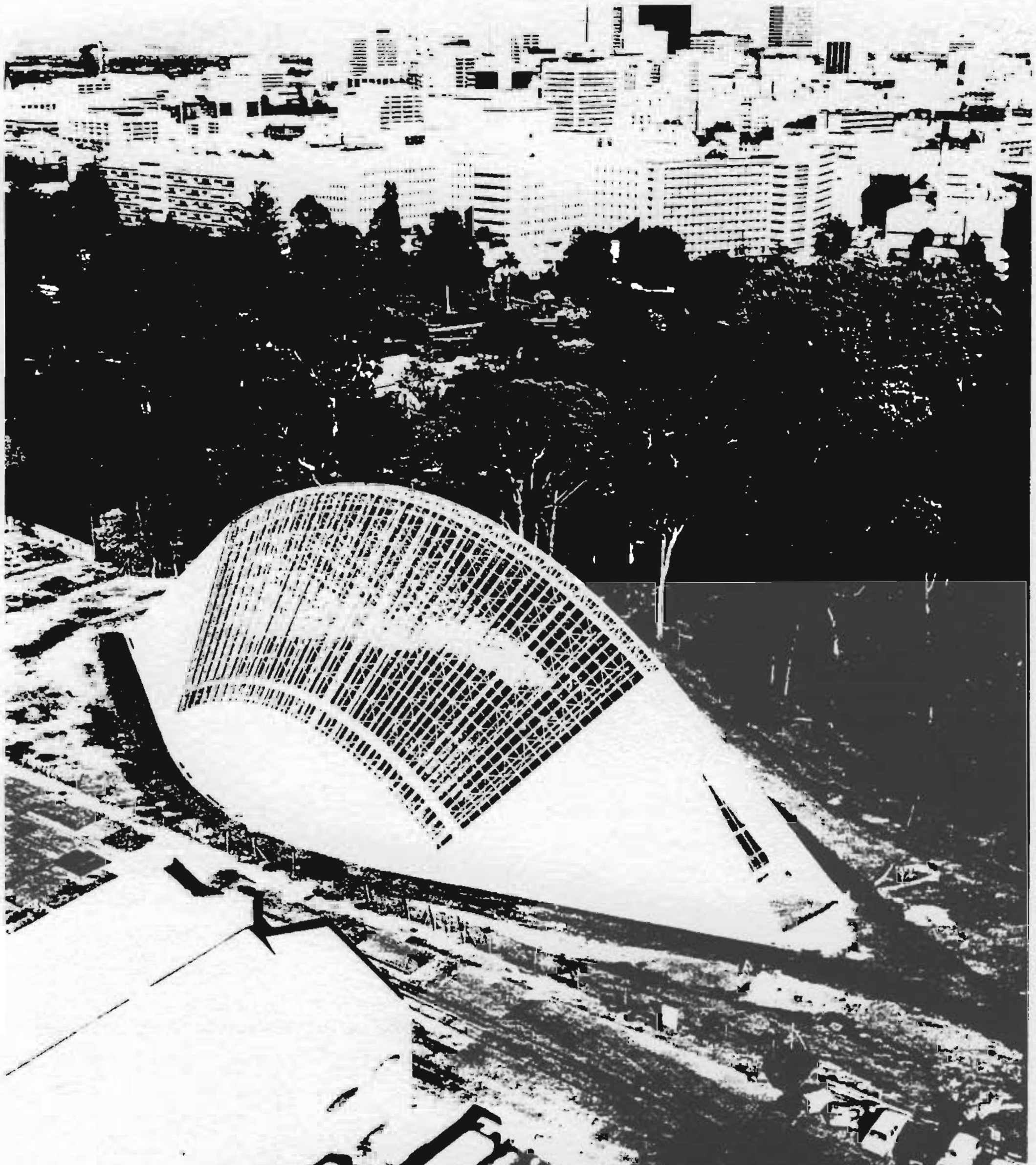
THE BICENTENNIAL CONSERVATORY



Architects: Raffen Maron Architects Pty Ltd

Structural & Civil Engineers: Connell Wagner (SA) Pty Ltd

THE BICENTENNIAL CONSERVATORY

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Introduction

The Bicentennial Conservatory in the Botanic Gardens of Adelaide was built as part of the Federal and State Bicentennial Commemorative Programme, and replaces the existing 100 year old Herbarium with a modern tropical conservatory of imposing proportions.

With a length of 96 metres, width of 47 metres and maximum height of 27 metres, the building is the largest single span conservatory in the Southern Hemisphere and one of the largest such buildings in the world. It provides the ideal environment for a tropical rainforest, with species ranging from the smallest sub-canopy plants to trees in excess of 20 metres tall.

The design and construction of the building presented many challenges to the Architect, Raffen Maron, and Consulting Structural Engineers, Connell Wagner, requiring close collaboration from the earliest stages of the design process.

This paper discusses the more interesting aspects of the design of the foundation and the superstructure. In particular, it describes the unique structural solution which was developed to permit the entire building to be glazed prior to erection.

The Bicentennial Conservatory opened its doors to the public in November 1989 and has already received wide acknowledgment as one of the finest buildings of its type in the world.

Foundations

It was a functional requirement of the Conservatory that public entrances and exits be provided at ground level, and also that adequate soil depth be provided within the building to support the roof structure of large tropical species. This led to the selection of a dished raft foundation, founded approximately 2.0 metres below the surrounding natural ground level. While not required to be absolutely waterproof, the raft formed a barrier between the specially treated soil within the building and external soil and ground water, both of which were known to contain salts detrimental to tropical species.

It was known from previous experience that the permanent ground water table in the Hackney area is relatively high. As part of the geotechnical investigation, stand pipes and piezometers were installed at the site and monitored over an 18 month period in order to determine seasonal variations in the water level. The exercise confirmed the presence of abundant ground water approximately 2.2 metres beneath the existing ground surface. Only minor seasonal variations in this level were detected.

To avoid possible flotation effects on the huge dished raft foundation prior to the placement of 5,000 tonnes of internal soil, a sump system was designed for the base of the raft enabling continuous pumping if necessary, thus relieving ground water pressure on the underside of the raft.

This system was so designed that after placement of the internal soil, negating any possible flotation effect, the sumps were sealed against external ground water and were then incorporated into the internal drainage system.

Superstructure

The design and detailing of the three dimensional structural frame presented a significant challenge. The Architects' requirement that the superstructure allow maximum light into the Conservatory led to the selection of relatively lightweight three dimensional secondary trusses of triangular cross section, supported on a curved central spine truss.

Connell Wagner also worked with the Architect and other Consultants on the structural design and development of the glazing system, utilising available proprietary components where possible to limit cost, but ensuring suitability for the special requirements of the Conservatory.

The form of the three dimensional trussed frame was developed not only for the obvious requirement of structural integrity; being visible both from inside and outside the building, it was of the utmost importance that the structural frame be complimentary to the Architectural form of the building. Further, the form of the structural members in the vicinity of the glazing had to meet stringent requirements in terms of adequate light transmission, air circulation and ease of building maintenance.

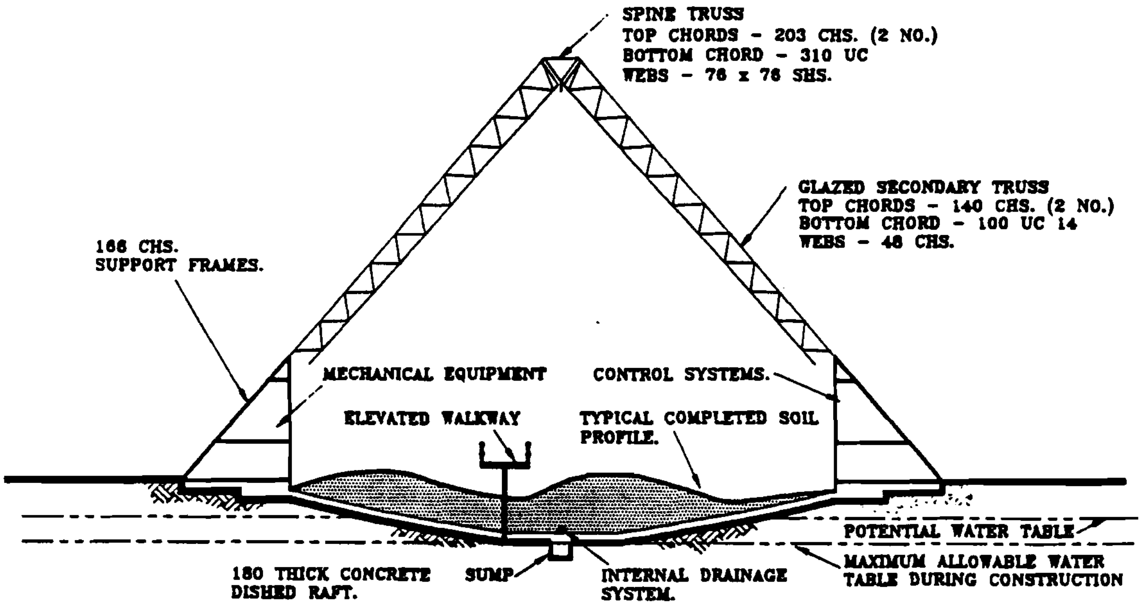

FIGURE 1 – TYPICAL CROSS SECTION

The complex three dimensional shape of the building demanded a sophisticated design approach to ensure that account was taken of the appropriate combinations of all imposed loads, including the structure's self weight, temporary loads during construction, service live loads, forces due to wind and earthquakes, thermal loads.

To accurately predict wind loads for structure of so unusual a shape as that of the Conservatory, Connell Wagner commissioned the building and wind tunnel testing of a 1:200 scale model of the building.

The testing programme, which took account of the existing topography, and in particular the large trees in the Botanic Gardens to the west, indicated that the most critical wind direction was from the North East; design wind pressures were obtained from the model for all parts of the building shell varying from a minimum of 0.68 kPa at the base of the structure to a maximum local pressure of 1.53 kPa at the extreme ends of the spine beam.

Seismic forces were determined by applying the provisions of the SAA Earthquake code AS2121.

During the preliminary design phase of the project, it became apparent that a significant proportion of the members in the superstructure would undergo their most severe stress condition during the construction of the building.

For this reason, it was decided to depart from normal industry practice (in which Contractors are generally responsible for the design of temporary works) and the Consultants undertook full documentation of all temporary works associated with the structure, including the 30 metre high scaffold towers and guying systems supporting the spine truss. The contract documents also fully specified the construction sequences to be adopted for all primary structural elements of the building.

Load Analysis

After a careful assessment of the proposed construction sequence, and the various combinations of superimposed load which the building undergoes after completion, it was decided that 14 separate load cases should be considered.

Of these, eight were required to determine the maximum stresses in the various structural elements during construction of the building. In broad terms these load cases covered the following:

- Spine truss erected and supported on temporary towers.
- Progressive erection of the tubular frames at the planar sections at the northern and southern ends of the structure.
- Progressive installation of secondary trusses, commencing at the centre of the building and moving radially in both directions towards the most severely inclined trusses at the northern and southern ends.

In conjunction with these construction stages, the following loads were considered:

- Deadload
- Windload (5 year return wind)
- Thermal Load (30°C differential)

The remaining six load combinations were analysed for the mathematical model of the completed structure and covered the following:

- Deadload
- Liveload (0.25 Kpa generally)
- Windload - 50 year return wind
- Thermal Load - 40°C maximum temperature differential
- Seismic Load - generally non-critical
- Deadload plus partial wind (5 year return) plus thermal (30°C differential).

Three Dimensional Frame Analysis

In order to carry out such a rigorous analysis of a three dimensional structure containing some 3,000 individual elements, a rationalised mathematical model in two stages was developed.

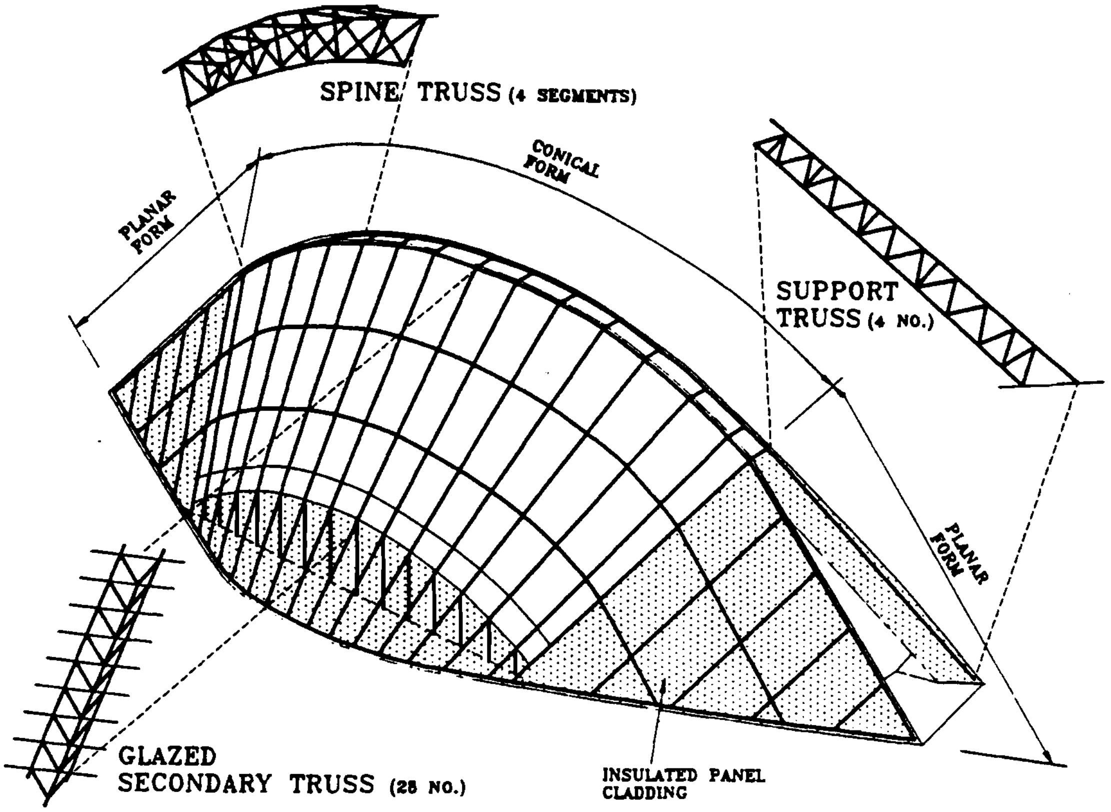

FIGURE 2 – SHOWING GLOBAL MATHEMATICAL MODEL AND MODELS USED FOR SECOND STAGE ANALYSIS.

1 Global Model

This mathematical model was developed in order to represent:

- The segmented curved spine truss
- The trussed frames at the northern and southern ends of the building supporting the spine truss
- Tubular framing at the planar sections of the building
- The 28 secondary trusses.

Each of these elements was represented by a series of "single line" elements, the properties of which were calculated to accurately represent the stiffness and flexibility of the members concerned. In this global model, the structure was represented mathematically by a total 228 single line elements.

Following computer analysis of this rationalised model for the 14 load cases outlined above, the critical "single line" elements were identified for further detailed analysis.

2 Second Stage Analysis

For the Second Stage Analysis, precise mathematical models were established to represent the major elements of which the Global Model was comprised. Account was taken of the individual chord and web members in the trusses, and of secondary elements such as purlins.

Nodal forces which had been established by the Global Analysis were then imposed on these detailed models, in order accurately to determine critical stresses and deflections in all members and joints.

From the analytical procedures described above, final member sizes for each individual component for the structural frame were confirmed, stresses calculated and deflections determined for each of the major members.

Construction Tolerances

During the design and documentation of the Bicentennial Conservatory, particular attention was paid to building tolerances.

After a close examination of the many and varied jointing details which were required, it was decided that the general tolerances prescribed in AS1250, the SAA Steel Structures Code, would be adopted.

By careful attention to detail, bolted joints were achieved in all cases by the use of standard (non slotted) holes with 2mm clearance on bolt diameter.

It should be noted that the purlin connections, which were provided with nominal 10mm stainless steel shims, were further complicated by the tendency of the secondary trusses to deflect laterally during the erection process. This matter is discussed further under the next section, "Preglazing of the Structural Frame".

Preglazing of the Structure

The primary glazed walls of the conservatory lie on the east and west facades and each contain in excess of 1,400m^2 of glass.

Preliminary cost analyses during the design development stage of the project indicated a clear advantage in preglazing the secondary trusses prior to erection, due to the obvious difficulties and dangers of access on the building itself.

Having accepted that the trusses should be preglazed on flat at ground level, a major challenge lay before the structural engineers - to devise a structural system and a method of erection which would maintain lightness of structure, while limiting distortion of the trusses to prevent breakage of the glass panes during the lifting process.

The problem of distortion was exacerbated by the aesthetic requirement that the trusses should not be triangulated in the plane of the glass - this would have produced an unacceptable "cluttered" appearance.

The solution adopted was to attach temporary prestressed cables to the trusses in the manner indicated in Figure 3. These cables were 12.7mm diameter high tensile strands of the type normally used in prestressed concrete construction; each cable was secured to the truss by a simple anchor at each end and was threaded through eyelets provided on the chord members. Tensioning was achieved using a turn-buckle system, monitored by a load-cell device attached to the cable. Detailing of the entire system was developed in such a way that the cables could be removed after erection of the truss, leaving little or no trace on the finished structure.

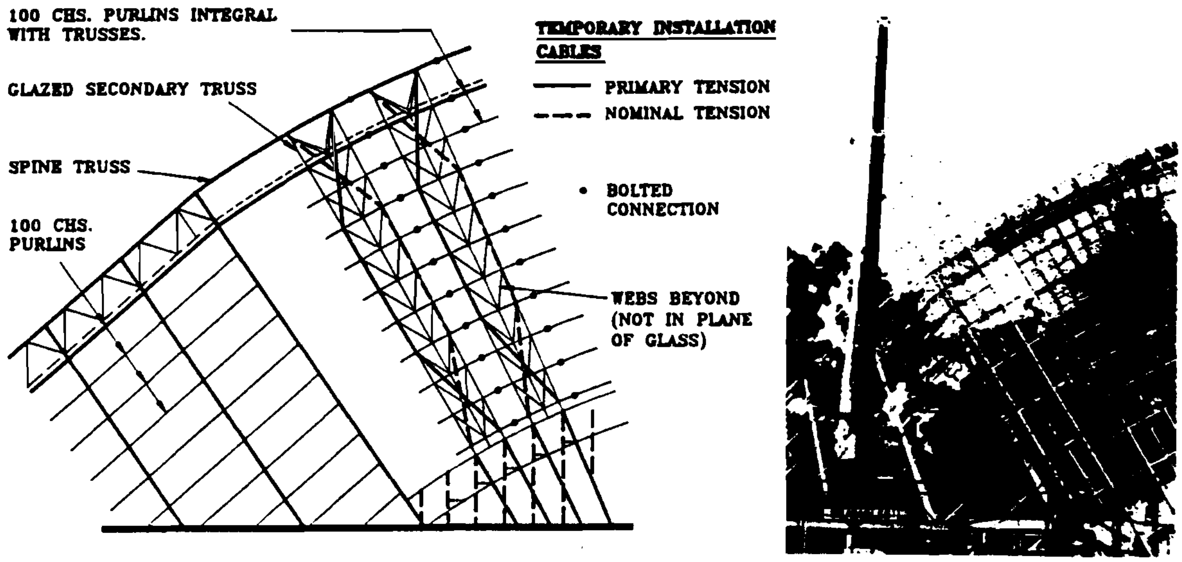

FIGURE 3 – PRESTRESSING SYSTEM – SECONDARY TRUSSES.

The purpose of the temporary cable system was twofold:

- By providing a nominal tension of 5KN in both cables attached to each truss, the stiffness of the truss in the plane of the glazing was significantly increased. In this way, calculated lateral deflections of the most severely inclined trusses (at the extreme ends of the building) were reduced from 65mm to 25mm. The latter figure, equivalent to a span/deflection ratio of approximately 1000, was considered acceptable; an examination of the glazing details confirmed that there was sufficient "take-up" in the glazing gaskets to accommodate distortions of that magnitude.

- By varying the tension in the lower cable (denoted as primary tension in Figure 3), the system provided for effective lateral precambering of the secondary trusses.

With potential lateral distortions varying from zero at the centre of the building to 25mm at the northern and southern ends, variable precambering was necessary in order to permit erection of the trusses within the relatively tight tolerance (10mm) permitted by the purlin-to-purlin connections, described above.

To confirm the parameters adopted in the design and also to establish the additional lateral stiffness afforded to the secondary trusses by the presence of friction in the glazing system, an initial on-site test program was conducted. After glazing of the first truss on the ground, the prestressing cables were attached, the nominal 5KN tension induced in the secondary cable, and lateral truss deflections were monitored as the primary tension was varied from 5KN up to 30KN.

From the results of this test program, the primary cable tensions required to precamber the secondary trusses for various angles of inclination were confirmed.

The erection of all 28 secondary trusses, carrying a total of 2016 individual panes of glass was achieved without a single breakage, and dimensional tolerances were maintained, using the precambering system described above, with sufficient accuracy to enable all connections to be satisfactorily and promptly made.

Summary

The Bicentennial Conservatory is a unique building in its form, dimensions and function. The complex three dimensional shape demanded an exacting and innovative approach to both the structural design and construction.

The preglazing of the structure, in particular, presented a unique challenge; the trusses, each carrying 100 square metres of glass, which were erected on the eastern and western facades of the building, are believed to be the largest prefabricated steel and glass components ever lifted.

Since the completion of the glazed structure in late 1988, the building has undergone almost three years of expsure to seasonal temperature variations, and has performed well.

The steel three-dimensional space frame and the glazing system which it supports have proven to be compatible; jointing, drainage and other details have behaved as predicted in the design.

This is particularly significant for a structure in which almost fifty percent of the members experience maximum stress conditions as a result of thermal loads.
