The Roof to the Busby Hall National Wine Centre

G.V. Wallbridge
Wallbridge & Gilbert

A.D. Woods
Wallbridge & Gilbert

INTRODUCTION

The National Wine Centre

As everybody knows, the best wine in the world comes from South Australia!

So it came as no surprise to all South Australians when in 1997 it was announced by the National Wine Industry Steering Committee that the proposed National Wine Centre was to be built in Adelaide.

But seriously, to briefly explain the philosophical idea behind the construction of a National Wine Centre. It was to showcase to the world the very best wines from all of the many wine producing districts around the whole of Australia and to provide a centre for education in wine making and grape production. The centre also features a wine museum and has a fully interactive exhibit which not only educates, but even encourages visitors to make their own virtual wine by selection of 10 different parameters (such as when to pick your grapes, how long to keep them under skins etc). My personal efforts resulted in the computer announcing that I had single handedly ruined the reputation of my wine label!

The Building

After a fiercely fought competition the design team was appointed to design what was required to be an Iconic building of outstanding Architectural merit. The successful Architect was Cox Grieve Gillett, a pre-existing joint venture between the internationally famous Sydney Architect, Philip Cox and local Adelaide firm Grieve Gillett. Wallbridge & Gilbert were appointed structural and civil consulting engineers.

The brief as mentioned called for an icon for the wine industry and a building of outstanding Architectural significance. Philip Cox in his own words chose “a celebration of simple materials – timber, earth and glass to reflect the good things in life, wine and food”. A building complex in a radial plan evolved from the constraints of the selected site in the Adelaide Parklands. These constraints included a five metre high heritage listed wall which formed part of a Colonial lunatic asylum, and a line of beautiful jacarandas bordering the adjacent Botanic Gardens. In addition to these guidelines Philip chose to draw further concepts from the form of the wine barrels.

The building with a total floor area of 6,000m^2 was designed in concept as a series of pods in radial plan layout. The various pods housed the exhibition centre, the massive wine cellar, a five star restaurant, a café, wine industry offices and the centre piece, the Great Hall, which is now known as the Busby Hall, as a major function centre to hold up to 800 people.

Philip’s concept for the roof of this Great Hall was for an exposed timber structure with double curvature rising to a height of 12 metres at the rear.

DESIGN PHILOSOPHY

This structure was very much an answer to an Architectural concept. Philip Cox is a designer of strong ideas and firm concept. From the outset Philip used the term "Diagrid Roof" to describe his vision for the roof/ceiling to the Great Hall as we called it during the design and documentation. The geometry of the roof is technically a part toroid with minor (vertical) radius of 28.6 metres and major (horizontal) radius of 94.4 metres. The part of the toroid utilized was an outer lower section with an included minor angle of 31º and major angle of 22º.

Philip had in his mind an all timber structure which he considered would be "easy" to design. Indeed, in its final form at only 13.0 metres clear span, 15.85 metres chord length and length of 35 metre it is not a huge roof by any measure. However several issues needed careful consideration.

- The use of timber members in an inverted arch and hence, in tension under long term loads.
- The multiple timber joints necessary to produce the smooth curvature both inside and out. (There are 430 nodes in the roof each with six timber beams attached).
- The desire to construct a thin "shell" structure with a maximum total depth of less than 250mm or an effective span to structure depth ratio of 52.
- The exposed structural connections needed to be neat and of simple appearance.

Our design team then undertook a series of preliminary design studies to work up the design concept in conjunction with the Architectural team. This process included some "healthy" debate as various options were considered for architectural and structural merit. After much debate the decision was taken to include light stainless steel cables beneath the diagrid to pre-stress the arch into a compression state under long term dead load. These cables were 8mm diameter designed in pairs at every second row of nodes along the diagrid.

Having effectively removed the long term tension problem we were able to greatly reduce the timber beam sizes and the end distances in the timber beams at the bolted node connections. This produced a much smaller node of neater appearance. The nodes were to be fabricated stainless steel but later budget constraints saw these fabricated in mild steel and then galvanized. The pre-stressing also overcame possible problems of dimensional control by ensuring that there was no slack in the bolted timber joints which may have allowed a sagging effect over the roof due to small movements at multiple joints. The nett dead load induced cable tension was designed to be resisted at the lower edge by robust cantilever columns and at the upper edge by the large triangular tube truss which spans 38 metres horizontally over the rear of the hall and rises a further 5 meres in height.

The relatively thin nature of the shell was considered by analysis in a design case which became known in our office rather quaintly as "panting" or the "pop through failure". The chord length of 15.85m is generated by an arc length of 15.10. With 26 bolted joints between top and bottom we had to be sure that, long term, slack (due to timber shrinkage or bolt hole slack) at each joint did not add to become a problem. This would have allowed the shell to "pant" up and down under turbulent wind suction and possibly, under arch compression, even "pop through" into an upward curvature state.

During the preliminary phase it was decided to analyse the shell structure with 200 x 75 segmented beams of seasoned Queensland Hoop Pine because of its good dimensional stability as well as architectural quality, low spread of flame indices and cost. Joint strength was considered as well as axial and bending strength characteristics. An upper skin of two layers of 12mm marine grade plywood fixed in a checkerboard pattern provided the smooth curve in two directions. By both gluing and screwing the plywood to the top edge of the segmented beams in this manner we were able to produce a homogeneous diaphragm of known structural performance.

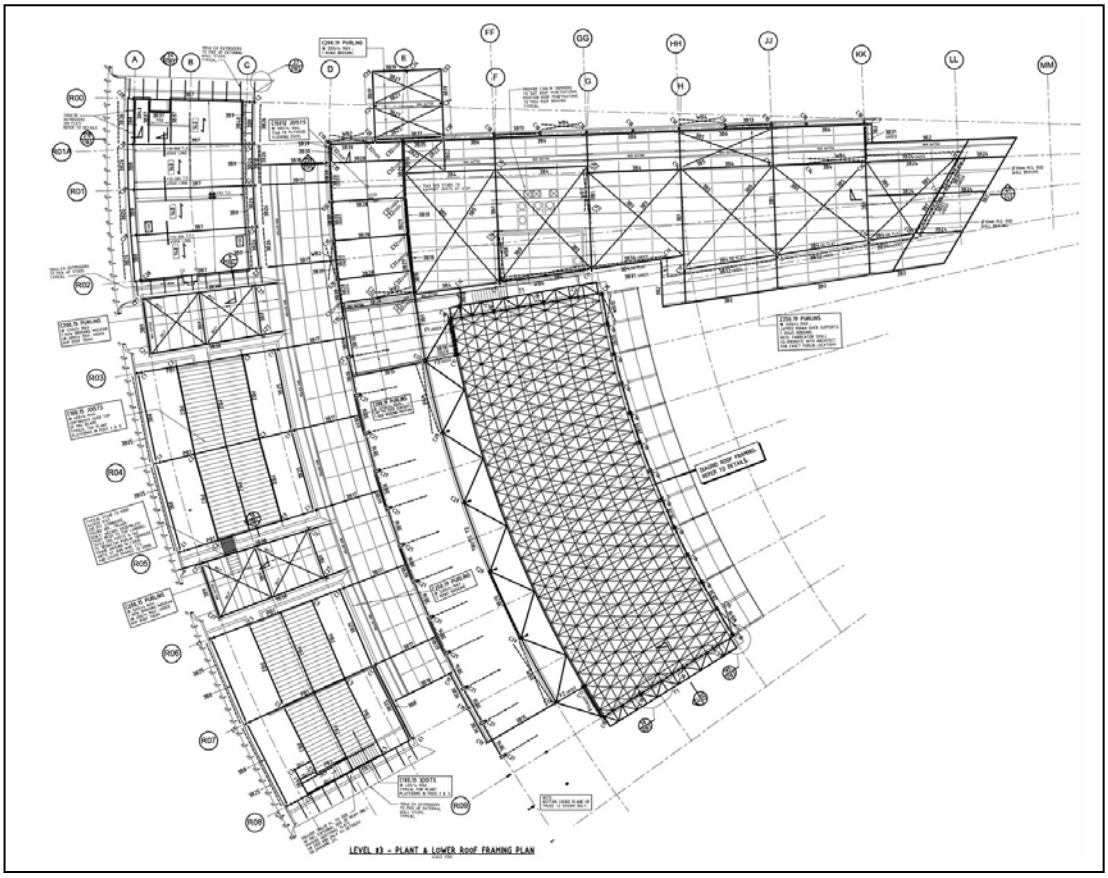


Figure 1

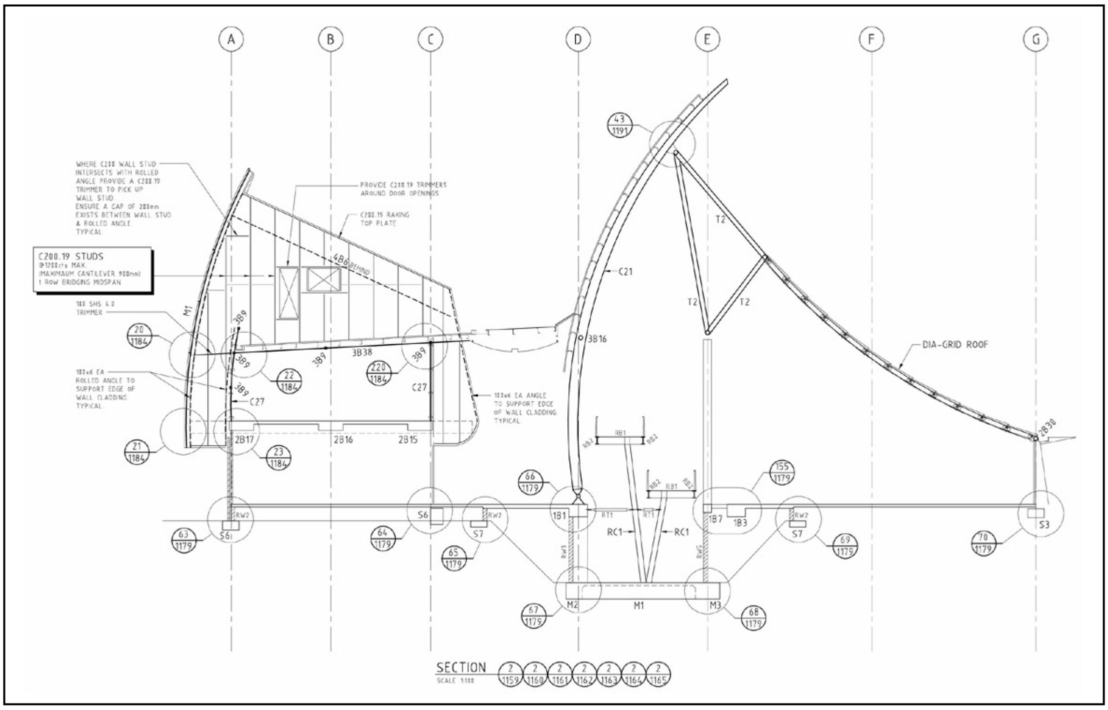


Figure 2

DESIGN SUMMARY

Wind Loads

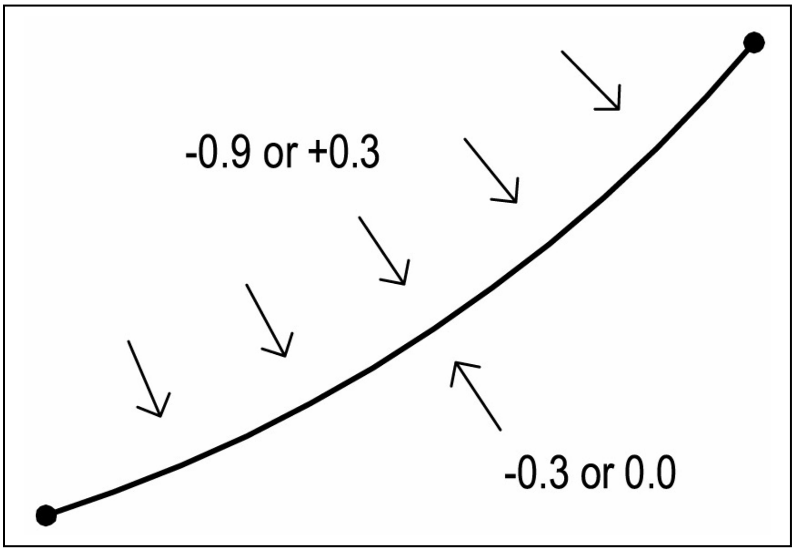
Wind loads were calculated in accordance with AS1170.2. Due to both time and cost constraints it was decided by the team that no specific modeling or wind tunnel testing was to be undertaken.

However, the shape of the hall roof and roofs of the surrounding pods combined to create a complex form which was not easily matched to the cases provided in AS1170.2. Our team was fortunate to have at their disposal the early results of wind tunnel testing being undertaken for our design team the proposed new Adelaide Airport terminal building. This building will be considerably larger but had many similarities in geometry such as leading edge angle, building height to width etc. However the match was not perfect. The team, studied these results in combination with all the available literature and decided to carry out analysis for an overall maximum load case and several non-uniform load cases to achieve a conservative upper bound solution.

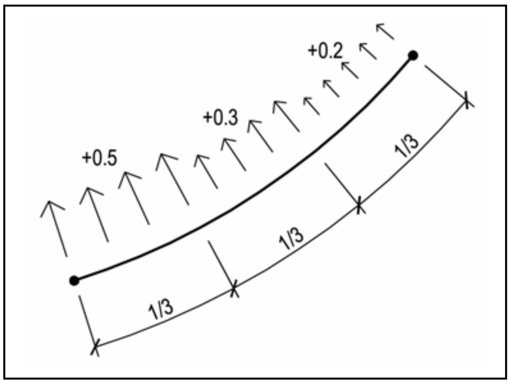
The following parameters were used:

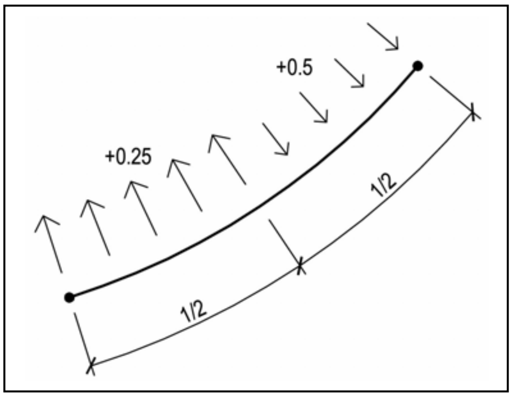
Adelaide region permissible wind speed	V_P = 41 metres/second generally
Terrain Category 3	M_{Zcat} = 0.89 at top of roof (adopted throughout)
Shielding Multiplier	M_S = 0.95 (structure is well screened on south & west)
Importance Multiplier	M_i = 1.0
Topographic Multiplier	M_t = 1.0
Design Wind Speed	V_{2p} = 34.7 metres/second
Design Wind Pressure	Q_z = 0.72kPa (permissible)

The following uniform pressure coefficients were adopted:



Due to the slope of the roof being near horizontal at the base, and near vertical at the ridge, and the proximity of non-uniform coefficients, we were concerned about the potential for variable suction and turbulence to be created by the curved shaped of the concourse roof above and immediately to the south. The following non-uniform coefficients were also adopted.





The cross sectional curve naturally acted as either a tension or compression arch under the uniform wind loads but the non-uniform wind loads but the non-uniform loading had the potential to induce significant local bending moments within the relatively thin shell structure.

Self Weight

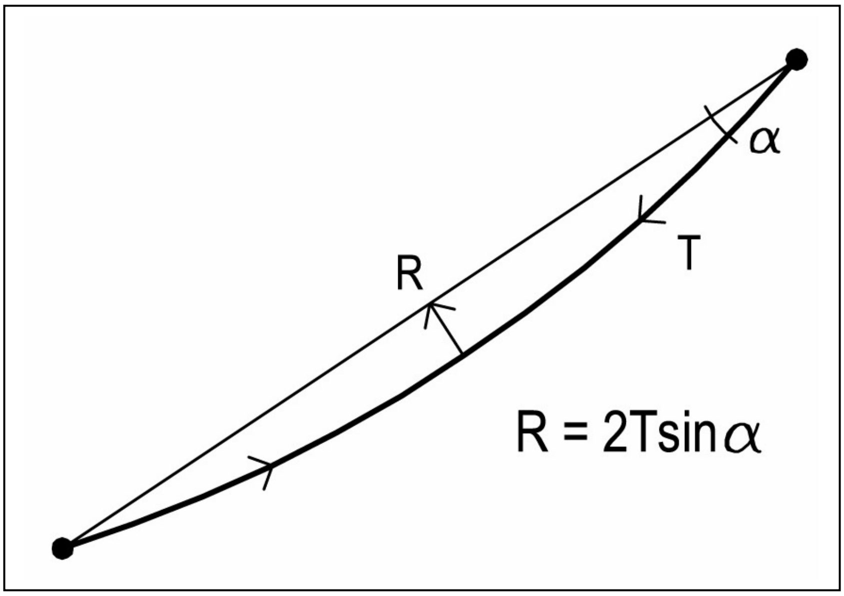
6mm thick zinc sheeting	0.42kN/m^2
Waterproof membrane	0.03kN/m^2
Plywood (2 x 12mm sheets)	0.20kN/m^2
Timber frames	0.10kN/m^2
Steel Nodes + Services & Cables	0.20kN/m^2
Total	0.75kN/m^2

Live Loads

Maintenance	0.25kN/m^2

Cable Stress

Loads imposed on the timber diagrid due to the tensioning of the steel cables was modelled as below.



The following critical load cases were analysed:

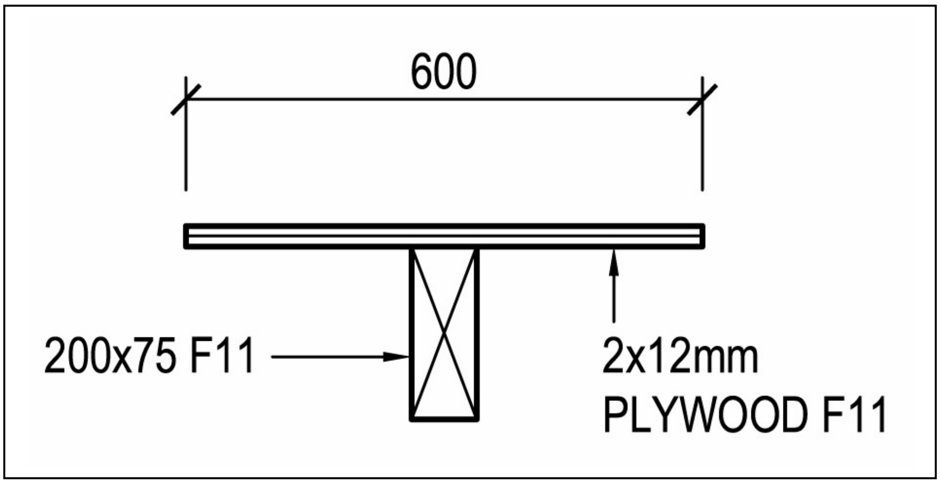
1. Self weight
2. Cable stress
3. Wind load uniform upward
4. Self weight plus wind load upward and cable stress
5. Wind load uniform downward
6. Self weight plus wind load downward and cable stress
7. Wind load non-uniform (up and down)
8. Wind load non-uniform plus self weight and cable stress
9. Live loads
10. Self weight and live load and cable stress

Due to the previously stated aim to eliminate long term tension within the timber frame and connections, the balance load in the cables to support the structure self weight was determined as being approximately 13kN per 600mm length of roof.

Initial analysis of a catenary was performed using SPACEGASS software within the following design actions being generated by the critical load condition (Case 4):

Maximum Bending Moment	2.6 kNm
Maximum Axial Load	9.0 kN compression (working loads)

The roof section was analysed as an effective ‘T’ beam 600mm wide as this was the average spacing of ‘radial’ timber struts and cable spacing (double cables at 1200mm).



The ‘T’ section was initially analysed utilizing a 75 wide timber section with properties of as follows:

Minimum Timber Grade	F11
Area	$A_g = 29400mm^2$
Stiffness	$I_X = 143 \times 10^6\ mm^4$

The section was checked for the timber struts to act as compression members ignoring the plywood except as providing lateral restraint. The plywood was also checked as a diaphragm in accordance with Appendix E of AS1720.

The combined section was checked to resist bending moments.

Axial force capacity of strut	F_c = 15MPa compared with applied f_c = 0.9 MPa
Diaphragm capacity of section	F_o = 4.4MPa compared with applied f_c = 0.9 MPa
Minimum section bending capacity	ØM = 31.3 kNm compared with applied M^*= 3.9kNm

The timber section was deemed sufficiently robust to accommodate the applied loads and was reduced in section by changing to a 200 x 60 strut in lieu of the 200 x 75 strut. No further reduction was considered warranted due to the necessary capacity required at timber connectors. The stiffness of the section yielded deflections of less than 10mm.

The cable capacities using 8mm diameter (1 x 19 Grade 31b stainless steel) are 53kN, thereby providing greater than 3 factor of safety.

The node connections formed a critical component to the roof and were designed to enable the maximum bending moments.

The connection was designed to resist a minimum applied moment of M^k = 3.9kNm by utilizing a bolt in a shear plate connector at the base of the timber and the plywood over the node to maximize the available lever arm between fixing points.

The relevant bolt and screw fixings were determined as follows with the minimum numbers of screws maintained across both sides of the node between plywood and strut.

In addition to the above, low creep resorcinol adhesive was used between layers of plywood and at the plywood/strut interface to supplement the fixings provided by screws and ensure that the diaphragm remained rigid under the influence of long term loads and wind load fluctuations.

The surrounding steel frame structure was designed to support the loads imposed by the tension in the cables generated by the dead load and the arch thrusts from externally applied loads.

CONSTRUCTION

Throughout the design our team spent a lot of time on issues of how this multi-directional structure could be economically built. Obviously given the problems of geometric control of the double curvature together with the size and height of the roof it would not have been economical to build it stick by stick in its final location. We worked closely with the builder, Hansen Yuncken who proposed building a template of one quarter of the roof set up at ground level in the forecourt of the Wine Centre.

This template was constructed as a pinus timber framework supported on temporary concrete pad footings. Dimensional control was maintained by electronic survey such that the framework held each steel node in its exact location and orientation remembering of course that the nodes are further apart as the top of the roof is approached and the axis of each node is tilted in two directions relative to the adjacent nodes. The timber members were bolted to the nodes and the two layers of plywood added. The entire section was then lifted by mobile crane utilizing a steel lifting frame and multiple lifting rods. The crane slings were sized so as to rotate the section from its position “on its back” on the framework to the correct orientation for bolting to the steel structure. The stainless cables were connected to the steelwork at top and bottom and with slotted holed bolts only finger tight the cables were tensioned until the crane registered no load. The bolts were then fully tightened and the crane disconnected.

After all four sections were placed the missing panels of timber and ply were inserted between the edges. The pre-stressing of the cables to pre-load the arch into compression was carried out in incremental 2kN stages to a total of 12kN with the aid of cable tension meters.

Finally, the waterproof sheeting, which can accommodate the taper and curvature, was fastened by workers abseiling from the tube truss.

SUMMARY

The Diagrid Roof to the Great Hall of the National Wine Centre, as conceived by Philip Cox, has been delivered in an economic and structurally sound manner and has to date performed exceptionally well demonstrating good stiffness under wind loading and most importantly no evidence of movement or leakage.

The National Wine Centre and in particular the diagrid roof has to date received five Architectural and Engineering awards including the top prize at the recent International Timber Engineering Conference in Kuala Lumpur. While not of enormous span or constructed of revolutionary new materials it exists as a result of Architect and Engineer working together with a harmonious use of timber and steel to produce a sound structure of great beauty.