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PLANNING AND ARCHITECTURE

Aurora Place Commercial Office Tower – 88 Phillip Street, Sydney

Rocco Bressi

1.1   INTRODUCTION

Aurora Place is a landmark mixed use development situated in the core of Sydney's Central Business District on the site of the former State Office Block.

The project, completed in December 2000, comprises a 44 level office tower, 18 level residential building and supporting retail facilities. The 4,262 square metre site is bounded by Phillip Street, Bent Street and Macquarie Street.

Aurora Place has been developed by Lend Lease and East Asia Property Group, the extraordinary design is the work of world renowned architect Renzo Piano. Bovis Lend Lease is responsible for the project management of the design and construction.

The office tower at Aurora Place has a net lettable area of 49,500 m^2 and has attracted several pre-eminent companies as tenants such as ABN-Amro, Minter Ellison, Challenger International and the Executive Centre. The office tower not only boasts a prime location, landmark design and quality, it also rates as one of Sydney's most efficient and effective office towers.

Figure 1   East view of Aurora Place Development.

The residential building at Aurora Place is known as Macquarie Apartments and comprises 62 luxury residences, all of which enjoy unobstructed views across the Royal Botanic Gardens to Sydney Harbour and the Opera House.

This paper deals primarily with the structural design aspects of the commercial office tower.

1.2   BASIC BUILDING STATISTICS

The following statistics summarise some of the key features of the commercial tower:

- Number of commercial levels – 38
- Dedicated plant levels – Levels 3, 21, 42 and 44
- Number of basement levels – 4
- Floor to floor height of commercial floors – 3,720 mm
- Ceiling height to office levels – 2,700 mm
- Approximate NLA commercial floor areas including wintergardens (square metres) – 1,285 LR, 1,358 MR, 1,435 HR
- Core areas (square metres) – 354 LR, 324 MR, 260 HR
- Overall building height (to top of sail) – 200 metres
- Core slenderness ratio (height/depth) – 21
- Maximum lateral east-west sway movements – 300 mm
- Tower building frame lowest natural frequency – 0.18 hertz
- Stability frame – No outriggers used, building stability relies on combined frame action on core, floors and columns
- Average overall axial shortening of tower frame due to creep and shrinkage – 75 mm
- Volume of concrete used – 35,000 cubic metres
- Tonnes of reinforcement used – 6,200 tonnes
- Tonnes of post-tensioning used – 350 tonnes
- Tonnes of structural steel used – 650 tonnes
- Overall construction time including site demolition – 4 years

1.3   DEMOLITION OF THE STATE OFFICE BLOCK

As sites become scarce for tall buildings, these types of projects usually involve consolidation of land which often require demolition work.

This preliminary work is almost synonymous with tall building projects and are often complex.

The Aurora Place project was no exception. The site to be cleared involved the "de-building" of a 31 storey building, a 12 storey building and a 9 storey building. Known as the NSW State Office Block, the buildings were completed in the mid 1960's.

Obtaining information on these buildings is often difficult and often requires a significant level of exploratory work to analyse and understand the structures to be demolished.

Preparation and detailed planning with an integrated team of structural engineers and the builder was the key to the success of this "de-building" project.

Figure 2   State Office Block tower demolition.

The main building included a large central core of 6 lift banks, including in total 18 lift shafts.

The floor plates included steel beams and composite columns with a slab on ribbed sheet metal which had no shear studs attachment to the beams.

The integrated team evolved a method of "de building" that produced a 2.5 day cycle per floor.

This was an outstanding achievement and saved 4 months on the overall project program.

In the process, 98% of the base building material was recycled and the de-building work was completed with an outstanding safety record.

1.4   SITE GEOLOGY AND FOUNDATIONS

As the site was formerly occupied by the State Office Block between two and three levels of basements had been previously constructed over the entire site. The proposed development required that the excavation be extended into the Hawkesbury Sandstone by a further 10 metres to allow for the construction of two additional basement levels.

Maximum tower column working loads are in the order of 40,000 kN and are supported on reinforced pad footings. The central core having a total working load in the order of 730,000 kN is supported on a 1.5 metre thick

continuous core raft projecting 1.5 metres beyond the external perimeter wall lines.

Founded onto Class II and III sandstone, the design bearing pressures vary between 3.0 MPa to 6.0 MPa as recommended by the geotechnical investigation work carried out by Coffey Partners International Pty Ltd.

The existing reinforced concrete basement walls of the State Office Block building were retained and underpinned along Bent Street to maintain support for the roadway and high voltage electrical cables. Temporary support of the other boundaries was achieved using conventional methods such as anchored soldier piles and shotcrete walls.

Figure 3   Core raft construction.

Figure 4   Axial core stresses at raft interface.

1.5   BUILDING CODES AND REGULATIONS

The structural design of the building has been carried out in accordance with the relevant SAA Codes and the Building Code of Australia. Fire engineering principles were adopted where appropriate dispensations could be sought by Sydney City Council and other relevant authorities

From a structural engineering viewpoint, fire engineering design principles were applied to the composite structural steel floors, roof and sail elements above Level 41 to assist with the deletion of traditional fire rated steel construction. Active sprinkler systems were used throughout the building.

Figure 5   Level 42 plantroom needle columns and steel framing.

1.6   STRUCTURAL FRAMING DETAILS

The structural elements of the commercial office tower can be considered as two basic components, the primary building frame and the secondary structural façade support elements attached to the building frame.

The primary building frame is mostly constructed from a combination of reinforced and post-tensioned concrete. Composite structural steel has also been used for the accelerated building frame structures erected to Level 3 and the plantroom roof elements.

The secondary structural elements are constructed from structural steel and interact directly with the projecting external façade components. These elements are termed fins, tusks, sails and mast. Dog-bone mullions manufactured from aluminium are used and span between the cantilevered secondary steel support members. The externalised glass panels are supported directly from the dog-bones with silicon only.

1.7   BUILDING FRAME AND LATERAL STABILITY

The reinforced concrete core works integrally with the floor plates and columns to form a combined moment resisting frame. The distribution of loads to the core, floors and columns has been determined by finite element analyses. The wind loads applied to the building have been verified by aeroelastic wind tunnel model tests carried out by MEL Consultants at the Department of Mechanical Engineering, Monash University.

The frame action of the core, slabs and tower columns contribute to the lateral stability of the building. This requires that the floor-to-core connection be designed to resist the applied frame moments. Each floor acts as a mini-outrigger eliminating the need to adopt a centralised floor to floor outrigger system that would normally occupy valuable space within the building.
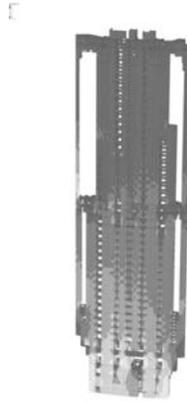
Figure 6   Core stresses.

A 320 mm thickened slab around the core enhances the floor to core moment connection. Two layers of 20 mm diameter screwed couplers are placed continuously at 150 mm centres and splicing with top and bottom reinforcement to provide a continuous connection for the thickened slab to the core.

Figure 7   Core jumpform construction.

The core provides 70% of the lateral load stiffness and the remaining 30% is taken by the combined frame action of the slabs and tower columns.

The lozenge shaped reinforced concrete core has a maximum width of 9.5 m. Permanently anchoring the 1.5 metre thick core raft into the bedrock provides base fixity of the core structure. Along the perimeter of the outer core walls, ten permanent rock anchors each of 8000 kN working load capacity were drilled through the core raft and anchored approximately 16 metres into the sandstone bedrock. The ground anchors ensure that no net tension results under the raft when the most adverse lateral and eccentric load conditions are applied to the building frame.
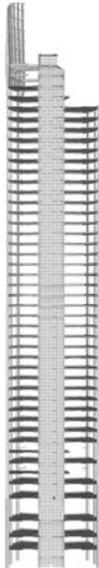
Figure 8   Finite element model cross section.

A view of the broad elevation of the core shows how corbelling out of the main walls above the lobby level and carpark entry transfers the northern and southern ends of the building. The corbelling extends the core by 10 metres approximately in the north and south directions along its longitudinal axis.
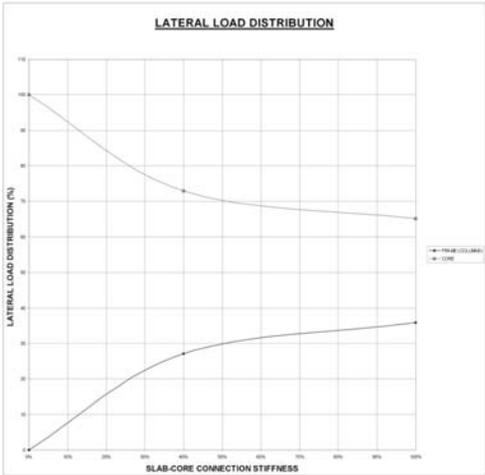

Figure 9   Graph of lateral load distribution.

To form a concierge through link at the lobby level an opening was made through the main walls adjacent to the high rise lift lobby. The structural opening through the eastern and western core walls is 5.0 metres high by 5.9 metres long. To transfer the truncated perimeter wall loads 3.0 metres deep by 1.0 metres wide reinforced concrete lintel beams are flinched to the core walls above the openings. A 1,200 mm thick reinforced concrete plate spans between the flinched beams to support the secondary walls located above the concierge.

Verticality of the core under eccentric dead and live loads was maintained by ensuring that the stress levels in the outer walls were similar on both sides. This was achieved by tuning the main perimeter wall thicknesses, resulting in a 500 mm base wall thickness on the eastern side and a 400 mm base wall thickness on the western side. Concrete strength for the core walls varied between 60 MPa and 32 MPa.

Figure 10   Aurora tower construction.

Splitting the jump form system into two segments enabled construction of the core to be carried out more efficiently. Screwed coupler bars are used to join the construction joint match lines of the core segments.

1.8   BASEMENT AND LOBBY FLOORS

For the commercial tower up to four basement levels extend out to the perimeter of the site. A combination of reinforced and post-tensioned band beams supporting reinforced slabs have been used to frame out the basement carparks, ramps, loading docks and plant areas. Earth pressures below the ground level are resisted by concrete retaining walls braced by the diaphragm action of the basement floors.

1.9   ACCELERATED TOWER CONSTRUCTION

Utilising composite structural steel framing, Level 3 floor plate of the tower and the twelve perimeter columns were constructed under accelerated conditions to facilitate a work front above and ahead of the final site excavation and prior to the construction of the basement and lobby floors. Prefabricated reinforced cages were fixed inside 12 mm thick tubular steel column form liners measuring 1,250 mm in diameter. The tubular steel forms were then filled with high strength concrete pumped from their base up to a height of 25 metres.

Basement slabs were connected to the tubular steel liners with internal and external stud attachments and lintel flange plates coinciding with the relevant floor levels.

Figure 11   Corbelled south core above lobby level.

The typical floor table formwork system was then introduced. The enhanced structural capacity provided by the composite beam action alleviated the need for back propping of the formwork system below Level 3.

Figure 12   Accelerated Level 3 structural steel framing.

1.10   TYPICAL OFFICE FLOORS

The typical office floor slabs span from the core a distance of 10.8 metres to the west and 12.0 metres to the east to the perimeter beams. Six tower columns, spaced at 10.8 metre centres, support the floors each side of the core and are located along the curved east and west building perimeter. The floor plates cantilever at the four corners of the structure. The south-east corner of the tower structure gradually extends and leans eastward at every level. Summation of this floor to floor incremental offsets results in a 5.0 metre change in span of the edge beam between Level 3 and Level 41.
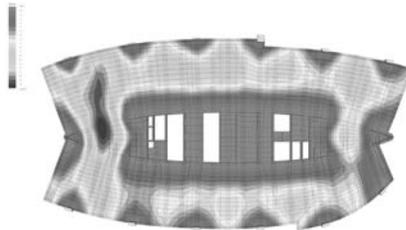

Figure 13   High rise floor deflections contours.

Notched post-tensioned band beams spanning between the core and the perimeter beams have been designed to a minimum depth to optimise services clearances and are typically 470 mm deep and 600 mm wide. The beam-ends are notched 150 mm for a length of 1.3 metres. This allows the underside attachment of the live end anchorages and stressing of the post-tensioning cables. The floor beams are spaced radially at 2.7 m centres and support a 120 mm thick mesh reinforced concrete floor plate. The floor to floor height is typically 3.72 metres. Perimeter edge beams are 880 mm deep × 400 mm wide and cantilever northwards and southwards to support the wintergardens and projecting façade elements. The edge beams and attached facade elements (fins) cantilever in some instances in excess of 10 metres beyond the first internal column.
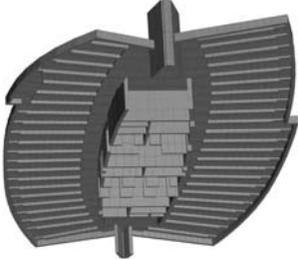
Figure 14   Finite element model of typical floor soffit framing.

Located continuously for a 2.0 metre width around the perimeter of the core, a 320 mm thickened slab section provides connection of the band beams to the core. The thickened slab section transfers the plate bending moments and in-plane diaphragm actions generated in the floors to the core walls. Circumferential post-tensioned cables are located within the thickened slab adjacent to the core. The curved plan profile of these cables assists in applying extraneous in-plane forces from the floors to the core similar to the concept of placing an elastic rubber band around the perimeter of the core. These cables also restrain the forces generated by the cantilevered and corbelled core walls located on the northern and southern ends of the building.
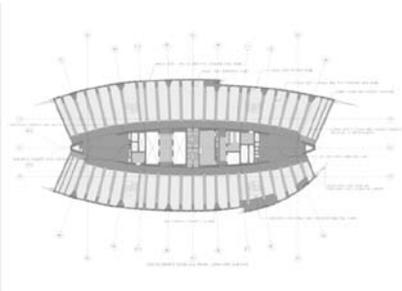

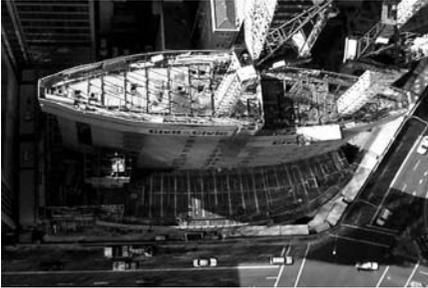**Figure 15**   High rise floor framing plan.

Figure 16   Aerial view of the core and floor construction.

The typical floors are generally designed for the following loads:

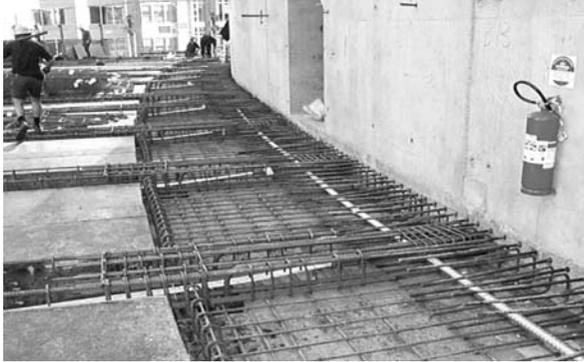|	
Live loads	3.0 kPa
Partitions	1.0 kPa
Services and ceilings	0.5 kPa
Raised computer floors	0.6 kPa

Figure 17   Typical floor band connection to the outer core wall.

Structural provision at four locations between slab bands has been made for additional penetrations through the floors to take the loads imposed by tenant specified interconnecting stairs.

1.11   PLANTROOM FLOORS AND ROOF

Double height plantrooms are located at Level 21 and Level 42 and the floors have a similar structural configuration as a typical floor. The required enhanced live load capacity of 7.5 kPa for equipment is obtained by increasing the overall structural depth to 500 mm and providing additional post-tensioning cables and reinforcement in the bands and slabs. Level 3 and the roof slab at Level 44 are supported from composite steel beams and profiled steel sheeting. The surfaces of external areas and wet areas are cast integrally with falls to assist with drainage and waterproofing.

1.12   STRUCTURAL ANALYSIS

Bovis Lend Lease structural engineers used Strand7 finite element program to perform numerous structural analyses on the building frame of the commercial tower.

A rigorous structural analysis of the tower building frame was carried out to comply with Clause 7.8 of AS3600, Concrete Structures Code. The analysis of the building frame takes into account the relevant material properties, geometric effects, three-dimensional effects and interaction with the foundations.

The principal aim of the analysis was to effectively predict the structural behaviour of the unique shape of the tower frame subjected to various static and dynamic loading conditions. These loading conditions were generated from combinations of superimposed dead loads, live loads, wind loads and seismic loads.

Static load combinations complying with the relevant Australian Standards provided realistic predictions of the actions of the building frame, particularly due to applied lateral loads and eccentric gravity loads.

The dynamic response of the structure also needed to be assessed to evaluate the natural frequencies of the tower frame and consequently the response of the structure in terms of perceptible building accelerations.
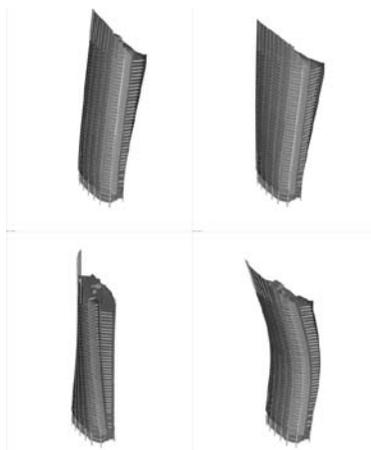

Figure 18   Natural frequency modal shapes.

A finite element linear buckling analysis enabled the determination of the tower column effective lengths, which then assisted with their final detailed design.

The columns measure 1,250 mm diameter up to Level 3, then change to a reducing rectangular shape through the tower. The low-rise column measurements are 1,200 mm × 800 mm, the mid-rise columns are 1,200 mm × 650 mm and the high-rise columns are 1,200 mm × 500 mm.

A suitable mesh grading to model the floor in plate elements was developed. Five plate element properties were used per level to account for the end notching of the beams and slab thickening around the core. Strand7 allowed the use of different membrane and bending thicknesses for each plate property. Beam elements were used to model the perimeter beams. The self-weight of the structure could be determined by assigning plate densities. Superimposed floor loads and live loads were applied as face pressures to plate elements.

Plate elements were used to simulate the façade of the building. The façade plate elements were a Quad4 Plate/Shell type, each node connected to the structural beam elements. The lateral wind pressures were then applied to the face of these plate elements.

The linear static, non-linear static, natural frequency and linear buckling solvers were used to evaluate the structural behaviour of the tower frame. Linear static analyses were used to predict the structural behaviour of the structural elements for a combination of lateral loads and eccentric vertical loads.
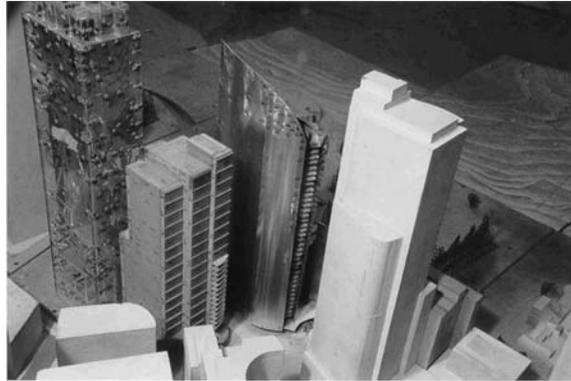

Figure 19   Aero-elastic model of Aurora tower.

The moment distribution of the lateral load between the core and the frame was also evaluated. It was important that a sensitivity analysis be carried out to account for the varying stiffness of the slab-core and slab-column connection.
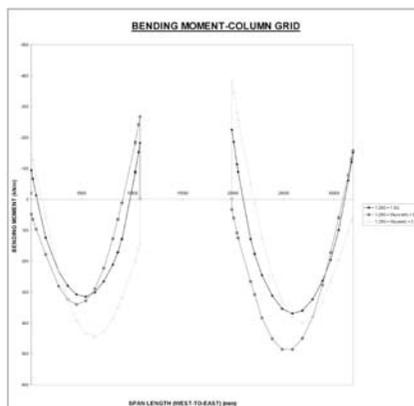

Figure 20   Typical floor slab bending moments.

The sub-modelling feature in Strand7 was used to study the detailed actions of the structural components in critical areas. This was achieved by using a much finer mesh in these areas compared to the coarser mesh of the global model. The sub-modelling feature allowed the bending moments, shear forces and axial forces to be found for the subsequent design of each structural component using the results of the global model.

A sensitivity analysis was also performed on the model by varying the dynamic and static moduli of the structural sections. This enabled the range of possible natural frequencies for the tower to be determined.

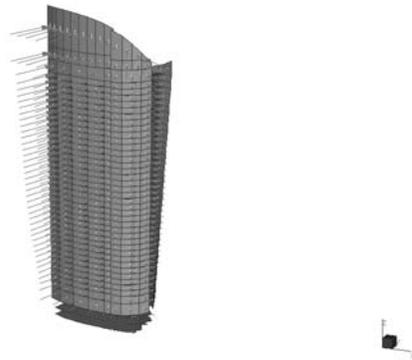**Figure 21**   Westerly wind pressures apply to global FEM model.

The occupancy comfort was gauged by a study on the accelerations of the building under dynamic loading conditions. For the lowest natural frequency, the calculated building frame accelerations were compared to maximum peak recommended values for a mean wind return period of 0.5, 1, 5 and 10 years as set out by AS1170 Part 2, Wind Loading code.

Horizontal acceleration criterion for occupancy
comfort in buildings.
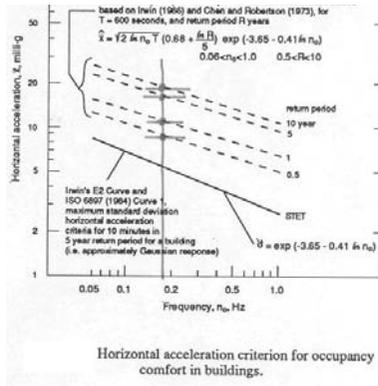
Figure 22   Building accelerations for occupancy comfort.

The base finite element model has the following statistics:

Number of nodes	36,097
Number of beam elements	4,840
Number of plate elements	38,566
Number of equations	215,856
Run time	7 hours approximately

The model was run in excess of 160 times to evaluate the sensitivity and effect
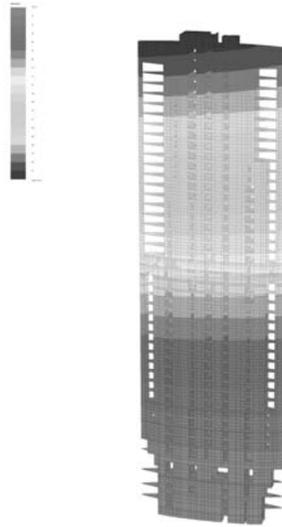of different structural parameters relating to the building frame.

Figure 23 Lateral deflection of core elements.

1.13 CONCRETE QUALITY

For the project, the specified concrete 28-day strengths varied between 32 MPa and 80 MPa. High performance concrete was used to control long-term differential elastic axial shortening, shrinkage and creep between the columns and core walls. All concrete mixes were super-plasticised and the maximum 56-day shrinkage was limited to 600 micro-strain.

By way of trying to equate the overall axial elastic shortening, shrinkage and creep between the core and tower columns, the concrete shrinkage specified for the concrete used in the columns was limited to 450 micro-strain compared to 600 micro-strain used for the core.

1.14 FAÇADE SYSTEM

The aluminium and glass curtain wall system spans from floor to floor and is supported by the edge beams with cast-in anchorage brackets. The double glazed façade system is designed to cope with the anticipated building movements determined by structural modelling.

The external layer of glass is chemically treated with a ceramic silk screen frit that helps to reduce the effects of direct solar heat gains and re-radiated heat gains. The ghost-like appearance is created by grading the frit around the perimeter of the vision panels from 80% at the body of the building to 40% at the edges of the fins and sail. The frit also modulates the transparency of the vision panels and hides the column and spandrel structures. High-energy

efficiency, maximum thermal performance and optimum visual comfort are achieved by using a moderately reflective glass and a low-E coating.

The wintergardens located on the northwest and southeast ends of the tower have clear glass and framed operable windows protected by cantilevered aluminium louvred sunshades.

The projecting façade glass that extends beyond and above the building footprint is supported by cantilevered structural steel framing attached to the edge beams.

1.14   FINS, TUSKS, SAILS AND MAST

Figure 24   Mast and west sail.

These elements are fabricated from structural steel and attached to the concrete building frame using cast-in embedments and bolts. The wind forces to be carried by these projecting elements have been determined by wind model testing as part of the aeroelastic studies carried out by MEL Consultants.

Figure 25   Façade fin cantilevered steel framing.

The cantilevered sail needles that project above Level 44 are laterally restrained by composite structural steel floor systems incorporated in the two top levels of the tower. The cantilevered needles back span through the high rise plantroom between Levels 42 and 44. Roll formed and flat steel plate sections are profiled and continuously butt welded to form the tubular and tapering needle sections. Being of grade BHP-300PLUS, the plate thicknesses used varied between 12 mm to 40 mm. Cantilevering in excess of 30 metres, the tallest needles located on the northwestern end of the sail weigh approximately 28 tonnes. The needles reduce in height and plan area as the sail extends southward.

Figure 26   West sail finite element framing model.

About their major axis the needles are required to resist ultimate wind pressures up to 3.6 kPa normal to the sail. Acting simultaneously, significant lateral load on the needles will develop produced by wind shear flow on the sail

plus wind drag on their projected faces thereby causing potential in plane sway movements of the sail. These movements are significant and will cause racking of the glass façade panels if not controlled either by increasing the bending stiffness of the needles or by providing in plane bracing of the sail (or both). To minimise the width of the needles, three sets of post-tensioned stainless steel wire bracing was installed. Offset and curved 168 mm diameter pipe transoms span between the needles and are used to fix welded outriggers that support the 1,350 mm wide by 3,700 mm high facade panels.

To eliminate in plane shear forces developing in the glass façade panels due to the potential racking of the sail, the connections of the transom outriggers to the dog-bone mullions were designed so no restraint against in-plane movement would develop.

Figure 27   Aerial view of south fin projections.

A continuous 323 mm-diameter pipe that also supports a continuous access way for maintenance and cleaning of the west sail links the tops of the needles.

Figure 28   Needle fabrication.

The tapered tubular steel sections used to fabricate the mast have a wall thickness of 12 mm. The mast diameter varies from 1,450 mm to 150 mm and the mast extends 99 metres above the crow's nest located at Level 34. Outriggers connected to cast in plates set flush in the core provide support for the mast.

Figure 29   West sail needle erection.

1.15   PIAZZA CANOPY

Ove Arup and Partners were appointed as specialist-engineering consultants to develop the concept for the canopy as a thin layer of glass with no secondary structural members supported by a "spiders web" cable net, slung between the two buildings.

Figure 30   Model of Piazza and canopy.

Having a maximum span of approximately 30 metres, the glass canopy has a plan area of about 650 square metres and is slung between the residential tower and office tower directly above the linking piazza. The cable net is formed into an anticlastic surface to ensure structural resistance to both downward and upward loadings and provides support for the frameless suspended glass.

The cable net is typically formed of 18 mm diameter, high tensile, and stainless steel rods connected at each intersection via stainless steel cast nodes. The bars specified were cold worked grade 316 stainless steel with a yield at 0.2% strain of 530 MPa. The glass is mostly 16 mm thick laminated, toughened and patch supported at each corner with stainless steel cast spiders. Varying diameter and length, stainless steel hangers, makes the link between the glass plane and the cable net.

To determine more realistic wind forces on the canopy surfaces, a 1 to 400 scale model of the canopy was pressure tapped and tested in the Monash University boundary layer wind tunnel. Surrounding buildings were modelled to at least 500 metres from the canopy.

The cable net is a structure that derives its strength and stiffness purely from its form; therefore, considerable design effort was spent to ensure the anti-clastic shape was stiff for both upward and downward loads.

Figure 31   View of canopy and lobby entrance.

To determine the form and shape of the canopy the following loads were considered:

- Self weight of the rods, hangers, castings and glass
- Live loads though not critical for cable net design
- Wind forces as derived from the wind tunnel testing
- Prestress applied to the rods forming the supporting net
- Temperature differentials for a range of plus or minus 25 degrees
- Seismic forces though found to be negligible for such a light structure

For the main backstay rods between the "puntone" or bowsprit struts and the building anchorages, a higher grade of stainless steel bar was required to resist the large forces. In this instance the bar grade has an UTS of 1000 MPa.

All connections between rods within the net and at its boundaries were made using stainless steel investment castings. The casting material is grade SAF2205 that yields at 0.2% strain of between 450 MPa and 550 MPa.

Spiders with one, two, three and four legs were developed to connect the glass to the slender circular pipe hangers. Architecturally shaped, the spiders had a requirement to rotate at the neck to accommodate the varying angle between

the glass plane and the vertical hangers. Once in place the neck is locked to resist bending moments from out-of-balance loads on the glass panels.

Figure 32   Piazza canopy spider and hanger connections.

The primary design action for the hangers was mainly for the downward tension loads developed by the dead, live and wind loads. However, upward wind loads are sufficient to induce compression in the hangers.

In order to verify design and as a check on the manufacturing process, three of each type of castings was load tested. The load testing included proof load testing to 1.25 times the calculated working load and then testing to destruction.

Due to the irregular shape of the glass and the point supports, non-linear finite element analyses were used to size the glass. Typically the glass comprises of one sheet of 8 mm thick toughened glass and one sheet of heat strengthened glass that is laminated with a pvb inter-layer. Glass panels with dimensions greater than about 2,000 mm had the thickness of each layer increased to 10 mm thick.

Erection of the cable net and the irregular and warping glass plane in the correct shape was essential to the structural performance of the canopy.

To erect the canopy, six stages were required as follows:

- Erection of the cast-in plates on the commercial and residential towers
- Erection of the cable net to a snug tight condition
- Stressing the net
- Erection of the droppers
- Erection of the glass panels
- Sealing and setting the joints between the glass panels

1.16   BUILDING MAINTENANCE UNIT

The Building Maintenance Unit (BMU) located on top of the building is one of the largest in the world with a total weight of one hundred tonnes. The BMU services almost all of the façade area of the building. A gantry cradle and a davit cradle system service the areas inaccessible to the BMU.

When the BMU is parked it is hidden within the building structure, and cannot be seen from ground level. When operating, the entire BMU rises five metres from its parked position and then the five stage telescopic jib can extend out forty-seven metres to reach all around the perimeter of the building. The jib also luffs up forty-five degrees from the horizontal and luffs down twenty-six degrees from the horizontal to bring the cradle as near as possible to the building façade.

Figure 33   Aerial view of BMU and sloping roof.

Operation of the BMU is simplified by the installation of a programmable logic controller (PLC). The PLC, through various counters, limit switches and sensors, determines the speed and movement of the BMU functions to give the operators the safest and most efficient ride. The acceleration of the BMU functions is also controlled to prevent any "whipping" action in the jib and thus provide the operators with a much more comfortable ride.

Other functions are also included on the BMU such as; a touch screen monitor to provide detailed information about the BMU condition, glass handling facilities on the cradle, and telephone communication between the operators in the cradle and the Building Superintendent.

Figure 34   BMU in operation.

All static and dynamic loads generated by the BMU are transferred to the tower columns and core walls by one storey deep steel trusses located within the roof plant-room space. The same trusses also assist with the strutting of the western façade cantilevered sail needles.

Figure 35   Completed Aurora Place project – December 2000

1.17   PROJECT CONSULTANT TEAM

1.17.1   Joint Venture Partners

Lend Lease Developments
East Asia Property Group
Mirvac

1.17.2   Project Management and Construction

Bovis Lend Lease

1.17.3   Architecture

Renzo Piano Building Workshop – Overall concept and façade documentation
Bovis Lend Lease – Building co-ordination
Gazzard Sheldon – Detailed documentation

1.17.4   Structural Engineering

Bovis Lend Lease – Overall structural concept and detailed design of the commercial tower
Taylor Thomson Whitting – Detailed design of the apartment building
Ove Arup and Partners – Secondary attached structures and glass canopy

1.17.5   Façade Engineering

Arup Façades
Permasteelisa

1.17.6   Wind Engineering

MEL Consultants – Professor Bill Melbourne

1.17.7   Geotechnical Engineering

Coffey Geosciences

1.17.8   Mechanical Engineering

Ove Arup and Partners
Environ

1.17.9   Other Building Services

Bovis Lend Lease