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Front cover: INTELSAT Headquarters, Washington D.C. (Photo: courtesy of INTELSAT)
Back cover: R&I Tower, Perth, Western Australia (Photo: Whitfield King)

Foreword

John Nutt

The Australian practice again presents a sample of work since an issue of *The Arup Journal* on Australia was last compiled. Those five years have been a busy and exciting time, and the substance of the projects reflects the character of this practice. We are delighted to have built a major project in Washington DC with an architect of the calibre of John Andrews — an urban building of quality in the capital of the world's largest democracy. Contrast that with the sensitivity of the Yulura project by Philip Cox. Set in the desert in the heart of Australia, an isolated location of harsh environmental extremes, the results are simple buildings and structures of great beauty.

Then there is the R&I Tower, a project put together by Alan Bond, winner of the America's Cup yachting event — an exciting high rise project in which the technical assessment of wind-induced problems was demanding. The architects, Cameron Chisholm & Nicol have a fine reputation for such buildings. There is a thrill about contributing to good architecture which influences us all.

Finally, there are the civil engineering projects of which we are masters.

We have grown substantially in those five years. In itself, size is unimportant. Nevertheless, engineering works of magnitude frequently can only be undertaken by firms of substance, diversity of resources and financial strength, and we wish to practice at that level. However, our real reason for growth is to extend people professionally, to attract staff of talent so that the future is assured, to find suitable roles for people as they age, and to have the appropriate resources to work on the type of projects that interest us. We have rejected the philosophy of remaining static in size. We do not believe it is possible in the long term, and we do not wish to shrink because of the connotations associated with a decreasing organization.

A national practice in Australia has certain characteristics. Each of the States has sovereign rights, many ceded to the Commonwealth Government at the time of Federation. Those states are an important source of work, and if we wish to do that work, we must have a strong presence in each state capital led by men of stature resident there, highly respected by the communities they serve. The Australian practice is decentralized as a result, knitted together through bonds of common interest and friendship. A parallel is a university of strong departments where the reputation and skill is vested in the staff and the faculties.

That decentralization has good and bad influences. So it breeds an independence of mind and a strength of character. But it also fragments our resources into comparatively small pockets. On balance however this diversity of background and the direct continuing relationship between the individual, his clients and the community has served us well and given us great strength.

Future plans

We plan to undertake a new initiative every two years — a procedure aimed at taking us where we wish to go rather than responding freely to market forces. In the last five year period, Arup Geotechnics has been placed in a sound technical and commercial position, the Canberra office has been started and is now operating successfully, and we have opened in Auckland, New Zealand. We have not yet completed the network of Australian state offices and plans are afoot to open in Adelaide.

Professions are changing, and the consulting engineering business is changing too. At one time, the professional endeavoured to undertake the work personally. That can no longer be the case, nor does the community expect it of him. Nevertheless, a soundly based consulting practice requires a personal commitment on the part of individuals to a client.

Practices are becoming more capital intensive, and as they do, outside investors are being invited to share in the ownership of firms. This worldwide trend in our industry will bring about a commercialism which could readily overshadow the ideals of a firm such as ours where quality of work, breadth of vision, social usefulness and a humane organization are important principles. The business world demands a return on capital and efficiency in a competitive environment but those goals and ours are not mutually exclusive.

New departures

The Australian practice moves in responses to these technological and community changes. We now advertise on occasions and actively promote our skills. We have appointed as Director of Business Development one of our number, a senior man of calibre whose role is to follow those projects of long gestation time, or clients in fields where our contacts are weakest. It is a departure from our past. The challenge for us will be to keep an appropriate balance. Being a staff-owned firm, we enjoy the privilege of setting our own targets.

The shame of a selection such as we have made in this issue is that many good projects are omitted and that only some of the people who have made significant contributions to the jobs and the well-being of our practice are acknowledged. Hopefully, another issue will be forthcoming when the richness and diversity of our work can again be recorded. Perhaps we can write on our work in developing countries, or on embassies overseas or our bicentennial structures, or on more of the disciplines which go to make our practice.

Maybe — if we have the strength!

Yulara tourist resort

Peter Thompson

Architect: Philip Cox & Partners

Yulara tourist resort: site plan

Introduction

The mainstream of our work is located almost entirely in Urbia and Suburbia. We rarely venture out to the surrounding countryside, let alone the outback. To the average Australian the outback is a mysterious and unknown somewhere, sparsely populated by unconventional Caucasians and Aborigines outnumbered by hordes of kangaroos and rabbits.

Quite by chance, as a result of being in the right place at the right time, we were given the opportunity to participate in the development from scratch of a township in the red centre of Australia — the real outback. The town was Yulara.

Yulara is the gateway to the Uluru National Park in the heart of Central Australia. The Park covers some 1,325km^2 and is situated 460km by road south west of Alice Springs.

Contained in the Park are a number of unique geological formations including Ayers Rock and the Olgas. The ecological system comprises plants and animals unique to the desert environment.

The area has been inhabited by Aborigines for many thousands of years. They have a deep and mystical attachment to the land and its features.

Ayers Rock is the Park's most popular attraction, second only in national tourist appeal to the Sydney Opera House. Tourist demand to the area increases by more than 100% each five years with 150,000 visitors to the Park in 1985.

In the past visitors were accommodated in motels and a camping ground in close proximity to the Rock. These facilities were no longer able to cater for the growing demand and were generally unsatisfactory, given the standard of amenity demanded by the average present-day traveller. In addition development had not been properly controlled or co-ordinated, leading to a general deterioration of the natural environment around the Rock.

It was decided in 1973 to relocate the accommodation and airstrip outside the Park boundary and restore the environmentally damaged areas. The new village thus formed was to be named Yulara.

Various attempts were made throughout the late 1970s to develop Yulara on a piecemeal or individual component basis and some infrastructure, mainly roadworks, had been installed to a notional town plan. These attempts failed basically through lack of developer interest. However, following an invitation from the Northern Territory Government, White Industries Ltd., a large developer/constructor, put forward a formula based on a town plan prepared by the architect Philip Cox for the construction of a complete tourist resort which was accepted. A joint venture company known as Yulara Development Co. was set up by the N.T. Government and White Industries Ltd. and became the client.

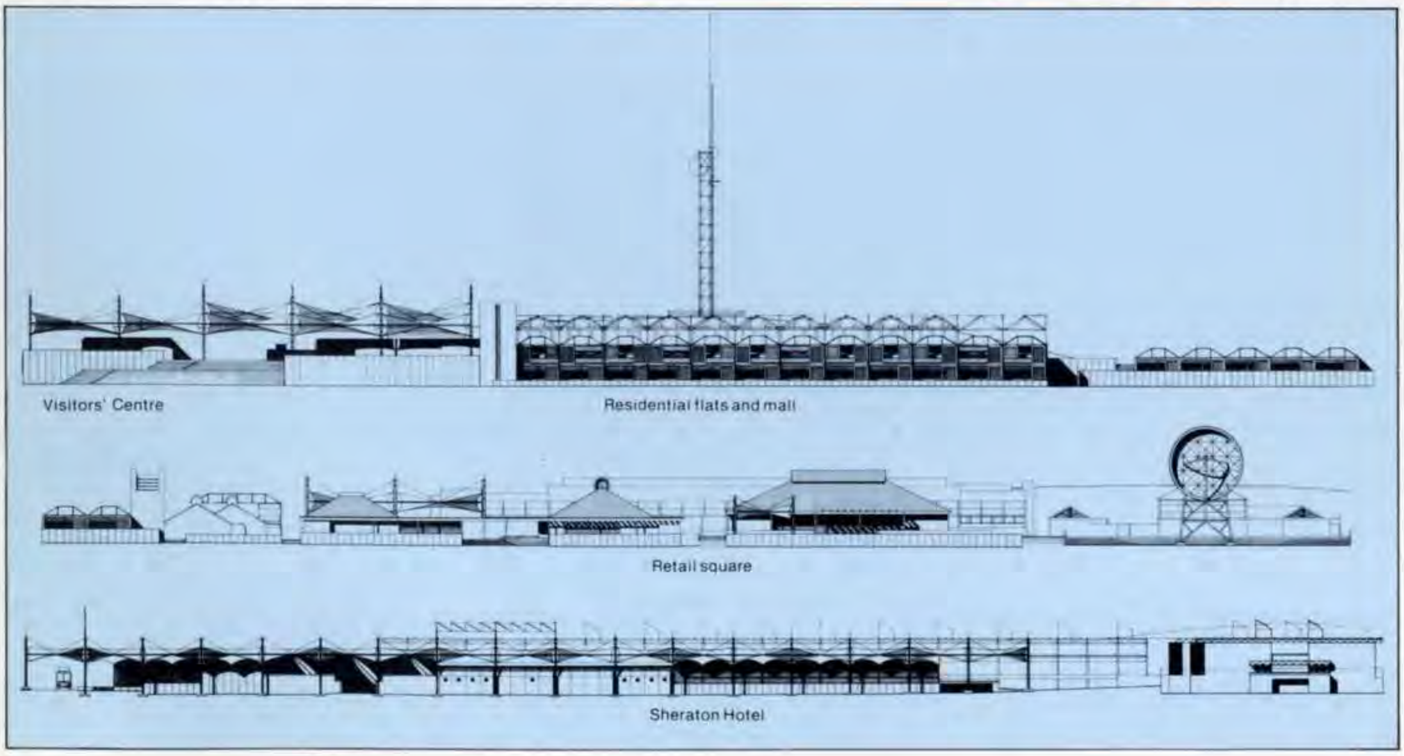
White Industries Ltd. were appointed project and construction managers and as we had assisted them in their presentation to the N.T. Government we became consulting civil and structural engineers to the project. Arup Geotechnics were in turn appointed to provide soils investigation and material evaluation services.

The resort was to cater for a mixed tourist population of 5,000 per day and 500 permanent residents. The project was estimated to cost some A$120 M and was to be designed and built in a 120-week period.

The site

The site is an area of about 2km x 2km located about 16km to the north west of Ayers Rock.

The climate of the region is arid with less than 250mm of rainfall per annum and a total evaporation of about 2800mm per annum.



The altitude is around the 500m mark contributing to the extreme diurnal temperature changes. Temperatures of 50°C are not unusual during summer, whilst winter temperatures of −5°C have been recorded.

Much of the area's topography consists of sand ridges or dunes averaging about 13m in height above an otherwise flat plain. The vegetation consists of plants expert at resisting long periods of drought either by remaining dormant, or by adjusting their life cycle such that they exist only while moisture is present. About 390 species of plant have been identified and these provide habitat for 22 species of mammals and 151 species of birds. During the past century a number of medium-sized mammals have become extinct, mainly due to the ravages of imported species such as cats.

Arriving at what was to be Yulara for the first time, it was difficult not to be overawed with the task in front of us all. Endless sand plains stretched in all directions, save for the overwhelming presence of Ayers Rock about 16km away. It was hard to believe that such a large development was to happen in such a desolate spot.

On taking stock however, things got better. An airport capable of landing Fokker F28 type aircraft had just opened about 5km from the site, a 2.8 Mva oil-fired power station was about 2/3 complete, and what was eventually to become the village service area was in embryo form as a depot for Park rangers.

The main road from Alice Springs was almost completely bitumenized and on the site proper there was a system of bitumen roads going seemingly to nowhere but complete with stop and give way signs. Given the topography these existing roads became a major determinant in the evolution of the central town area as building was only to be possible in the flat area between the roads and the dunes.

The resort

Yulara can cater for about 5,000 visitors per day and has a permanent population of about 500. The permanent population is largely concerned with the resort function but includes National Park staff plus their families.

The resort has three main components:

(1) The central spine

(2) Peripheral development around the main ring road

(3) Service area.

(1) The central spine is moulded to the dune formations in an S shape and contains most of the major resort components including the Visitors' Centre, two hotels, one of 100 rooms, one of 250 rooms, retail area, staff accommodation and community facilities. The buildings are arranged in clusters around civic squares and spaces. The hotels are placed at each end of the spine and linked to each other by a series of pedestrian streets defined by one and two-storey buildings. The whole of the spine is built on an elevated podium of varying levels defined by a perimeter wall. A central energy plantroom is located beneath the Visitors' Centre and services the total facility.

(2) The peripheral development includes components such as detached housing, school, police station, camping grounds, Aboriginal housing and service station. The camping grounds cater for up to 3,600 at four sites. The sites are fully serviced and have swimming pools. Caravans and bunk house accommodation are available for campers without tents.

(3) The service area east of the resort contains the water treatment and sewage treatment plant as well as the major storage and maintenance facilities.

Sheraton Hotel:
Top: View of central facilities
Centre: General view of main entrance
Bottom: Main entrance showing fabric shades and roof elements

Facing page: View of main resort facilities and Ayers Rock (Photo: John Gollings)

The main water supply is from the Dune Plains Aquifer located about 10km from the resort and which, it is estimated, can supply about 400,000m^3 per annum. The raw water has a total dissolved salt content of around 2,500mg/l and a fully automatic electrodialysis desalination plant has been installed which reduces the salt content to 500mg/l. Storage is provided for 200,000l of treated water, about two days supply at peak periods.

Four blends of saline, treated and reclaimed water are reticulated through the resort for differing purposes: fully treated for drinking and bathing, untreated for cisterns and fire hydrants, and blended for trickle irrigation to every tree and shrub planted within the resort.

Sewage is carried by a system of gravity and rising mains to a high speed extended aeration activated sludge process plant. Effluent is treated in a single process with the resultant sludge pumped into drying beds from which it is distributed to landfill areas.

The treated water is held in an artificial lagoon from where it is recycled to spray irrigate the main sports oval and to water a 5ha red gum plantation which is to be the source of firewood for fires in the camping grounds.

Fabric shades over Visitors' Centre

Sheraton Hotel rooms and central facilities

Foundations and ground works

Yulara is built in the swales between a linear system of sand dunes. The dunes are formed by wind blown sands and in the immediate area of the site are relatively stable, supporting quite dense spinifex growth, bushes and desert oaks. However, disturbance of wind patterns and removal of vegetable cover could initiate a new erosional phase and construction was therefore kept away from the dunes proper.

The built site covers an area of 1500m × 1000m with the central spine occupying 1000m × 400m. 20 trial pits were dug at what were considered to be significant locations. Penetrometer probes were used to obtain density profiles particularly through the higher levels of dune sands where backhoe pits were environmentally inappropriate.

The investigation revealed a consistent geology across the site with a 3m layer of sand overlying a layer of lateritic gravel which became cemented within a few hundred millimetres of its upper surface into a low strength, coarse-grained sandstone.

The sand material was very uniform across the site and fell into two categories, a well graded swale sand and a finer dune sand, blending into each other with elevation.

Tests were carried out on the sands to determine in situ density, compacted densities at natural and optimum moisture contents, CBR values and the internal friction angle.

Design values resulting from these tests were as follows:

compacted density	1.7 T/m^3
CBR value	20%
angle ϕ	33°

Allowable bearing pressure derived from these values varied from 150kPa for a 600mm wide footing to 300kPa for a 2m wide footing.

The penetrometer probe values were used in conjunction with the density results to correlate footing capacity with blow count for future site supervision.

The central spine complex is built on a series of raised soil platforms. It was considered economical to undertake an early earthworks contract for the whole complex; one of the reasons being the long distance for mobilization of heavy earthmoving plant. This meant however that the surface (covering some 200,000m^2) and sides of platforms had to be protected from wind and rain erosion over a one year period. It was clear that this could only be done by utilizing some form of surface treatment and a variety of alternatives were investigated. The cheapest and in fact the most appropriate was found to be a 150mm layer of compacted lateritic gravel spread over the total area.

Given the site contours, a cut to fill solution for the platforms was not possible and it was important that there was minimum disruption to the sand dune regime. Location of borrow pits was critical yet some 50,000m^3 of fill was required. A source of sand fill was finally established quite close to the site service area and the resulting excavations were eventually used for sanitary landfill.

Local gravel was available from borrow pits previously used for the roads. These pits were therefore extended to produce some 80,000m^3 of material required for the protective gravel layer and for basecourse to roads and parking areas.

Before earthworks commenced the spinifex and desert vegetation was harvested and stockpiled for future use as mulch. The top 200mm of soil which contains dormant seeds was similarly removed and stockpiled. Both mulch and soil were used as important ingredients of the desert zone restoration work carried out where damage from construction was unavoidable.

The platform filling and compaction was particularly effective, allowing trenches with 2m high vertical sides to stand open for quite lengthy periods. Control of compaction was achieved by constant field density testing plus penetrometer and nuclear density testing.

Site survey data was given in 1:500 map form. This was digitized and collated to form a digital terrain model suitable for computer design and volume calculation of roads, hard stands and earthworks generally.

The buildings

In general the buildings are of simple domestic scale and are masonry structures with pitched metal roofs. Minor elements such as verandahs, stairs and pergolas are of light metal construction.

The structural philosophy was developed around a minimum of basic elements which would afford some measure of standardization whilst allowing flexibility in building design. As it was necessary to house all labour in an on-site construction village, systems which minimized labour were more than ever important.

For this reason masonry construction was at first discounted and a precast walling system investigated. This was quickly ruled out due to the high basic cost of concrete and the cost and time involved in setting up an on-site factory. Other systems, incorporating metal or compressed cement

Retail square with shades over the plaza

Residential flats and shades over Visitors' Centre

panels, were adjudged to be too temporary in appearance and thus architecturally unsatisfactory.

The principal wall element thus became the hollow concrete block, as its larger dimension, compared to brickwork, ensured speedier erection plus superior thermal qualities.

Although some timber was used for roof trusses to houses, light steel frames and trusses were generally used to minimize transportation damage and reduce long-term maintenance costs. Most steelwork is exposed and tubular sections, bent to curves rather than cut and welded, were used to create acceptable forms.

Foundations and ground floor slabs were generally cast integrally on the compacted sand base. Where suspended slabs were required these were built, whenever appropriate, without falsework using profiled metal decking as formwork and reinforcement.

The scope for structural engineering ingenuity was strictly limited and most of our endeavour went into 'getting the details right' both visually and constructionally.

Concrete

A pre-mix concrete supplier was appointed to establish a central weighbatch facility in the service area and from whom contractors bought concrete at predetermined prices.

The batching plant capacity was $60m^3$/hr and averaged about $45m^3$/hr. The concrete was delivered from the service area to individual sites by a fleet of six $5m^3$ agitator trucks.

Aggregates were won and transported from a source 80km away and cement supplied from Adelaide via Alice Springs. Untreated bore water was chemically satisfactory. As could be expected the cost of concrete was prohibitive, being three times the average rate for major centres and consequently was used sparingly.

An independent testing laboratory was established on site. This carried out whatever tests were required, mainly for concrete and soils density and compaction.

One major problem with concrete was shrinkage due to a variety of causes. The aggregates supplied were invariably at the 'fine' end of the allowable range and the very fine material tended to bulk up in the stockpiles. Consequently the amount of water in the mix varied daily and was often excessive, which did not worry us too much as far as strength was concerned but aggravated the shrinkage problem.

Although concrete was placed very early in the morning during the hot months, the temperature nevertheless rose very quickly and unless curing was initiated promptly, plastic shrinkage resulted. This was particularly bad over the ribs of the profile deck floor where concrete thickness was only 30mm and no reasonable amount of reinforcement was effective. In these conditions the only positive method of curing is water inundation or spraying. This of course is not a particularly attractive proposition for a builder in a hurry and curing or the lack of it became our major supervisory problem throughout the job.

Fabric membrane structures

Shade is a very important commodity in an Australian summer and hard to come by in the central desert. It has been created at Yulara by extensive use of tensioned fabric membranes over public areas and buildings in the central spine.

The fabric membranes or 'sails' not only provide shade, but generate a significant visual impact and sense of three-dimensional enclosure; they have become the resort's most dominant architectural feature and allowed us to exhibit some engineering ingenuity.

Tensioned fabric is also used as a roof to the restaurants and public spaces within the major hotels central facilities building.

Of the total fabric area of $11,000m^2$, the major part, $8,000m^2$, is in the form of hypars. These are based on a 10.8m square plan module with each shade having a rise of 2.6m between high and low points.

The second basic form used in an inverted cone on a 7.2m module. Both the hypar and cone have boundary cables.

The third basic form is a saddle or barrel vault which was used to form the roof to the Sheraton Hotel. Preformed arch ribs and box gutters provided support for the fabric and boundary cables were not necessary. The barrel vaults have a system of hypars mounted above them to create a double fabric roof system.

The largest single shade is 20m × 10m and covers the amphitheatre forming part of the community facilities. The shade is in saddle form and is supported by four masts stabilized by cables. Unlike the other fabric elements on the project in which the supporting structure is independent of fabric, in this case fabric and supports are mutually dependent.

At an early stage consideration was given to the way in which the hypar elements were to be erected and if necessary replaced during the project life. The fabric panels are a specialist supplied item from outside Australia and it was recognized that for a number of reasons the support structures should be independent and not rely on the fabric for stability in any way.

The architect required that the support masts be as light as possible and it was initially envisaged that the shade force of about 80kN at each hypar support point be resisted by bending in masts which were freestanding. The magnitude of bending forces was such that a free-standing structure was not possible and horizontal stabilizing cables were provided between masts located at upper and lower shade support points and crossed to form an X pattern.

At the boundaries, support cables are taken to the ground at a 45° plan orientation to the main grid. In this way, out-of-balance forces were accommodated close to the point of application rather than being allowed to accumulate over a number of modules. This also allowed the minimum mast diameter (168mm) to be used throughout. The support and stabilizing cables are pretensioned using turnbuckles to avoid slackening under frequently occurring loads and to reduce mast displacement under wind loads.

Uplift forces were catered for by simply burying a concrete pad and mobilizing soil mass as resistance. Tension piles or ground

Four Seasons Hotel central facilities

anchors were considered but rejected due to the limited number required and possible geometrical problems.

Loadings considered in design were those associated with pretensioning and wind effects. Self-weight and maintenance loads were neglected with the exception of the roof barrel vaults where access loadings were considered. Surface pressures on the shades were determined from wind tunnel tests on similarly shaped rigid bodies. This was considered to be a conservative but acceptable approach.

At the outset a detailed investigation was instigated to identify suitable fabric materials. For the roof elements a coated fabric was essential but uncoated or open weave fabrics were initially considered a possibility for the shades. However we very quickly came to the conclusion that these were not suitable on strength and durability grounds and we concentrated our attention on coated fabrics.

We canvassed (no pun intended!) known fabric suppliers on a worldwide basis and a number of suitable fabrics were identified. These included pvc-coated polyester, silicone-coated fibreglass and teflon-coated fibreglass. A specification was developed on the basis of these materials and we called tenders. As could be expected the fibreglass alternatives, whilst offering the most durable solution, were significantly more expensive than the pvc-coated polyester which was chosen. The material was supplied by Sarna Kunstoff from Switzerland.

The required shapes of shade were determined which satisfied minimum surface with uniform tension criteria. This process, known as 'form finding', utilizes finite element and the dynamic relaxation method of analysis developed by Arups in London. The method is based on the fact that to go from one state of equilibrium to another a structure must move. By writing equations of motion for the structure and applying damping to make the structure come to rest, the computer methods follow the procedure that happens in nature (I think!).

Having established the 'form found' shape, loadings defined as pressures and fabric and cable pretensions were applied and the final deflected shape, fabric stresses and border forces determined.

The support structures were analyzed in two stages. Firstly the cables were assumed not to yield and a serviceability loading applied.

Compressive cable forces were thus identified and these values used as a basis for cable pretension. All cables were assumed to be stressed to the maximum value obtained.

Secondly cable pretensions and external applied loads were analyzed to establish cable and mast forces and associated reactions. This was an iterative process, with the number of iterations sufficient to limit out-of-balance forces at any node to acceptable limits.

Fairly early on, a scaled down prototype (7.2m x 7.2m) was erected over the camping grounds' swimming pool to check the system and develop an erection procedure.

This shade stood happily for some months but after a night of very strong winds one corner became detached, much to the consternation of all concerned. The problem was due to failure of a U-bolt connecting the fabric to the support structure. This was able to rotate in a vertical plane but not horizontal plane, and the very small in-plane forces from the fabric had eventually fatigued the bolt, which failed in the thread area. Fortunately the problem was fairly quickly solved by increasing the bolt diameter and changing the detail so that bearing did not occur in the thread plane.

The use of fabric elements for shade has proved very successful and although individually small in scale, the overall use has helped give the resort its unique architectural character.

Construction

The 120-week design and construction programme allowed for the project would have been ambitious for a project in or near a major centre, let alone one of such scale and remote location.

The project was divided into 24 major elements or packages, each with its own design and construct programme within an overall programme for the work. Tenders were called by the construction manager from companies which had registered interest and had attended briefing meetings in Alice Springs and Darwin where the scope, locations, logistics and site conditions and constraints had been fully explained.

The construction programme got under way in June 1982, 14 weeks after design commencement, with the construction of a 500 man construction camp. Once under way construction was programmed to keep camp population at a constant level.

All materials were brought in by double trailer road trains having a capacity of 50 T at an average of six arrivals per day and 12 at peak periods.

In order to give the project a cash flow, facilities were opened to the public when completed, commencing with the camp grounds after 14 months and the first of the hotels in 17 months. To achieve this the water sewage and central energy plant were also completed within this time. The final component, the Sheraton Hotel, was adjudged practically complete in September 1984.

Quite remarkably the total project was completed on time, on budget and with only the loss of 2,000 man hours out of a total of 1.6m through industrial disputation.

During the construction period we had two personnel stationed on site, one engineer and one supervisor. Direct supervision of the work was carried out by the construction manager with our people advising, this system creating a few differences of opinion until a working relationship evolved.

Communications by telephone or telex from the site were not particularly good, comprising one only radio-telephone circuit which 'dropped out' continuously during the hot months. The site staff were therefore fairly autonomous and dealt with the checking of shop details and minor amendments in addition to regular supervision.

Conclusion

Yulara was a demanding project even though not technically complex. The short time-scale for design and construction, coupled with site location and poor communication, made it necessary for documentation to be absolutely spot on, as small errors easily dealt with in the city took on the appearance of major disasters in the desert!

It would be true to say however that most of us associated with the project and particularly those lucky enough to go to the site, experienced a real sense of achievement for a job well done.

The persons responsible in our office for various sections of the work must not go unmentioned:

Bill Thomas: Structures

Colin Mathison: Civils

Alan Saxon: Fabric structures

Tony Phillips: Geotechnics

Walter McIvor: Site engineer

Harry Calverley: Site supervisor

As one would expect Yulara has had much publicity and has gained the country's highest awards for architecture, landscape and construction. Alas no engineering awards to date, but we remain in hope.

Credits

Client:
Northern Territory Government
Architect:
Philip Cox & Partners
Consulting engineers:
Ove Arup & Partners
Project and construction managers:
White Industries Ltd.

Photographs: Courtesy of Philip Cox & Partners

The Sydney Cove Bicentennial Project marine hydraulic studies

John Nutt

Architects:
New South Wales Government Architect in association with Hall, Bowe & Webber

Sydney Cove, in the centre of Sydney, is the cradle of white settlement in Australia. Almost 200 years ago, close to a fresh water stream, Sydney was founded by a party of soldiers and convicts, sent from England in the First Fleet. The Tank Stream can no longer be seen but a plaque on the shores of Sydney Cove will soon mark its locations in one of the many actions that, in 1988, will celebrate the bicentenary of Australia's founding (Fig. 2).

History abounds around this small inlet which has changed its character so greatly over the years. Now a bustling metropolis flanks its shore. Tall buildings stand proudly. The Sydney Opera House glistens in the bright sun on its eastern promontory, the Sydney Harbour Bridge springs majestically from the western. Ferries and hydrofoils hurry to the Circular Quay terminal, so-called because in sailing ship days the cove was shaped in a semicircle to berth the wool and grain square riggers (Fig. 1). Many of the old colonial stores and warehouses are still to be seen in The Rocks area which fringes the western shore, and which has become a flourishing tourist precinct.

That now is the setting for an exciting and technically challenging project for the Bicentennial Year in which we take part.

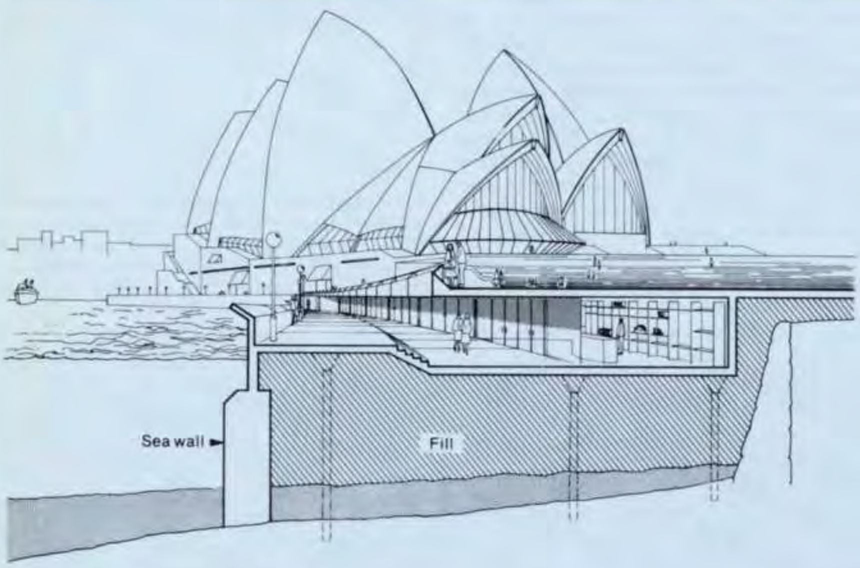
First, there is the completion of the Opera House forecourt and surrounds. When opened in 1973 this area was substantially unfinished, with acres of bitumen totally out of character with the elegance of the building. That will be repaved with more appropriate materials. The temporary covered walkway connecting the Opera House to the city will be demolished and another built below ground, so that the forecourt has visual linkage with the water. A wonderful waterfront promenade will be constructed around the whole length of the cove. There will be parks and plazas, pontoons and jetties. Dilapidated buildings will be demolished and those that are kept will be refurbished.

1. Sydney Cove *circa* 1880

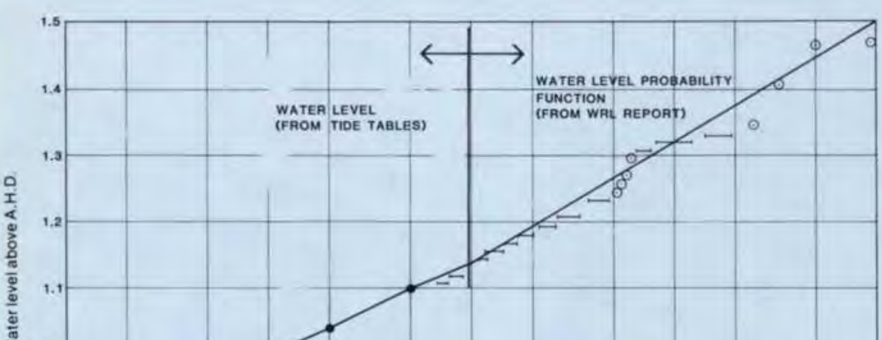
(Circular Quay viewed from Darling Harbour, from a hand coloured wood engraving by S. Calvert in the Nat West Australia Bank collection).

2. Sydney Cove. The Sydney Opera House is on the left, the Overseas Passenger Terminal (right) is partially reconstructed, The Rocks area with the southern approaches to the Harbour Bridge is on the right, with the centre of Sydney at the head of the cove (Photo: Horizon/Neil Duncan)

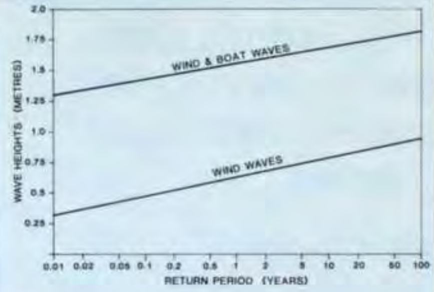
3. Sydney Opera House approaches



4. Upper and lower concourse levels



5. Return period of recorded water levels at Fort Denison (AHD)



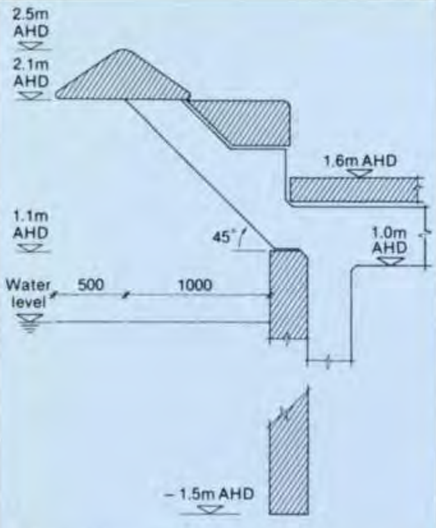
6. Wave height probability

7. Hydrofoil ferry (Photo: John Nutt)

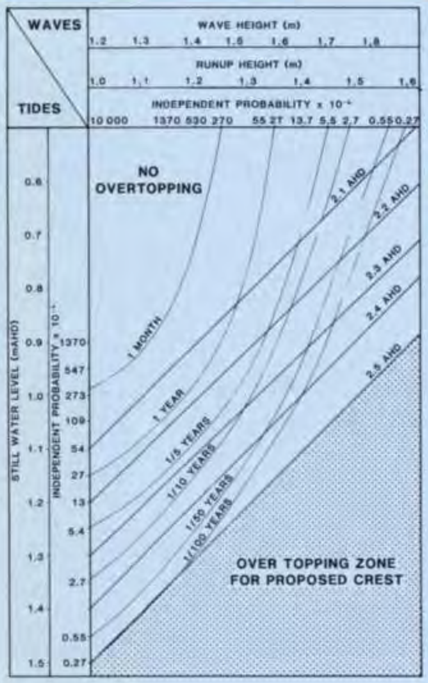
8. Water Research Laboratory wave flume test

The railway station and ferry terminal are replanned. Part of the overseas shipping passenger terminal, built in the days of mass migration to Australia and now too large for its cruise-related activities, will be demolished to make way for a park; and expanded to accommodate restaurants. The Maritime Services Building, headquarters for Sydney's port authority, will become the home for the Power Collection of Art. Campbell's Cove, a small indentation close to the tourist area of The Rocks, will be stripped of its ageing timber wharves, buildings and car parks and rejuvenated by landscaping and paving. First Fleet Park will be reshaped and replanted. The results of haphazard development will be put right.

Our role encompasses coordination, management and engineering. There are seven distinct precincts in the project involving four architectural firms acting under the direction of Andrew Andersons, the Assistant Government Architect. In addition to being lead consultant on the Opera House forecourt precinct, we are providing the civil engineering on three others, structural



9. Selected wave deflector profile



10. Probability of overtopping
for wave and tide combination

engineering for the reconstruction of the Overseas Passenger Terminal, and we act as coordinator between the lead consultants for three precincts in Circular Quay West. Perhaps the most technically challenging has been the marine work associated with the Opera House forecourt.

Jørn Utzon's concept of the Opera House shells standing on a granite plinth, to be approached by the monumental stairs from the forecourt, has been the starting point. A covered walkway, above ground, would separate the forecourt from the harbour. In an enlightened solution, Andersons instead has positioned it below existing ground level along the water's edge. But those ground levels are very low, only 1.5m above the highest tides, and to avoid the passage becoming an enclosed tunnel, it has been left open on the seaward side. The result is a new concourse on two levels, one covered and the other open, along which pedestrians can walk (Fig. 3). The lower level will provide shelter and accommodation for cafes and shops, the upper concourse will be terraced to permit outdoor seating and eating.

The demanding technical challenge came from two areas. Firstly, having to place the parapet, which includes a wave deflector, at an appropriate level — not too high so as to block light and air from the lower concourse walkway, not too low so that on occasions 'green water' would overtop the parapet.

Secondly, devising a construction technique for the lower slab which was to be constructed beyond the existing sea wall at a level of 2m below the highest of tides.

The general arrangement, now somewhat changed in detail but not in principle, is shown in Fig. 4.

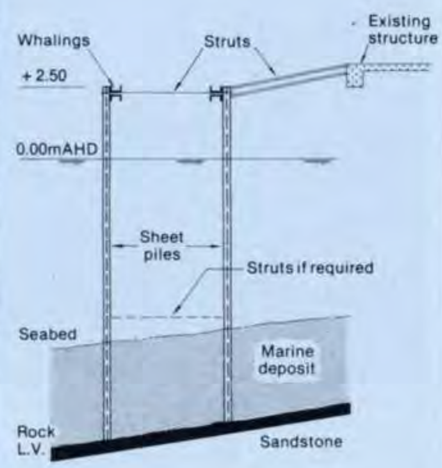
Water levels and waves

The investigations which resulted in the determination of the height of the wave deflector developed over a six-month period with major parameters changing from time to time as the design developed.

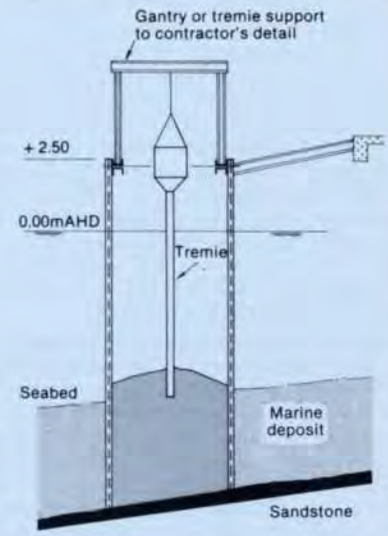
Water levels along a sea wall are determined by the height of the tides and the interaction of waves impinging on the wall. In the extreme case of a vertical wall, the reflected wave interferes with the approaching wave so that a standing wave of double the height results. A wave deflector would turn this back on itself and permit a lowering of the wall crest.

11. Probability of selected wave generating events overtopping a designated crest level

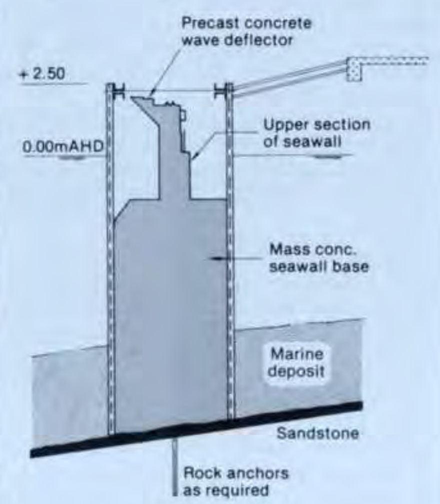
Crest level (m AHD)	Combined frequency	High tide level AHD frequency	Hydrofoil wave (m) frequency	Ferry wave (m) frequency	Wind wave (m) frequency
2.5 AHD	Once/100yrs	1.50 Once/100yrs	0.90 Highest/3hrs	0.30 Average	—
2.4 AHD	Once/20yrs	1.25 Once/5yrs	0.90 Highest/3hrs	0.30 Average	0.15 Once/week
2.3 AHD	Once/5yrs	1.15 Once/year	0.90 Highest/3hrs	0.30 Average	0.15 Once/week
2.2 AHD	Once/year	1.10 Twice/year	0.90 Highest/3hrs	0.30 Average	0.15 Once/week
2.1 AHD	Once/month	1.0 Once/f'night	0.90 Highest/3hrs	0.30 Average	0.15 Once/week



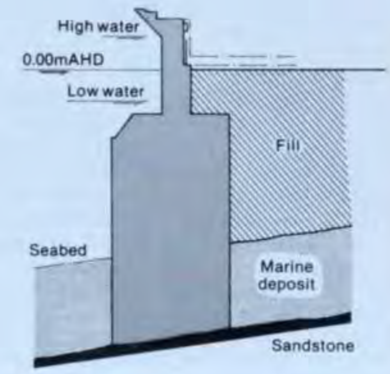
12(a). Drive two parallel lines of sheet piling and dredge to rock between



12(b). Tremie mass concrete wall



12(c) Dewater to top of mass concrete and form and pour upper wall section



12(d) Remove sheet piling, place fill behind wall, dewater to below slab level and pour floor slab

Tide levels in Sydney Harbour have been observed for nearly a century at nearby Fort Denison. The tidal range is not great — about 2m annually, and the highest observed level of storm surge and tide, at 1.48m Australian Height Datum (AHD), is approximately 0.3m above highest astronomical tide. This recorded data gave a solid base from which predictions can be made (Fig. 5).

Wave heights are not so readily determinable and were the subject of both theoretical study and site observation.

Waves are caused by both wind and boat traffic.

Wind waves were determined by the method of Bretschneider from wind data provided by Professor W. Melbourne of Monash University who had results of a comprehensive assessment of the wind climate of Sydney based upon the anemometer records of meteorological stations and terrain modelling in a boundary layer wind tunnel. The site is sheltered, the fetches were not great and the peak waves for the relevant directions are of the order of 1m for a 50-year return period (Fig. 6).

Several investigations were undertaken to determine the height of waves generated by different classes of boat traffic. The Water Research Laboratory of the University of New South Wales, together with the NSW Department of Public Works, were commissioned to measure waves generated by hydrofoil, ferries and other traffic near the site and elsewhere. Other movements were taken on a wave stick attached to the Sydney Opera House skirting panels, and back calculations carried out to give incident wave amplitude. Briefly, hydrofoils generate the highest waves (Fig. 7), particularly the outward bound hydrofoil as it climbs onto the plane. Catamaran ferries and heavy displacement ferries, although more frequent than the hydrofoil (one passage every two minutes at rush hour), generate markedly lower waves.

A basic design interval of 200 minutes was adopted for examination. This was selected because, within the tidal cycle, the water level remains within 0.2m of high water for approximately three hours. In this interval, there will be on average 30 hydrofoil passages, 100 ferry passages and 6,000 wind waves. The combination technique adopted is known as Turkestra's Rule and takes an appropriate peak wave from one source and combines it with the mean waves from other sources. The peak boat wave was taken as 2.0 standard deviations above the mean for the hydrofoil, and 2.3 standard deviations for the ferries. The peak wind wave was taken as 1.7 times the significant wave predicted (the average of the highest one third of all waves). The various combinations were determined and a design probability established (Fig. 6).

Wave deflector

Various configurations of wave deflector were tested in the flume at the University of New South Wales. The geometric arrangement of a wave deflector is for an overhang to turn the wave back on itself during reflection, which is illustrated by the photograph of one of the tests in Fig. 8. Both the angle and dimensions of the overhang can be changed, and experiments were carried out on varying combinations to find the most suitable within an acceptable architectural profile. The final selected shape is shown in Fig. 9.

The performance of this wave deflector under the influence of predicted waves for various probabilities could then be determined. It was established that at overtopping there was a ratio between the height of the incident wave to the height of the crest above still water. This ratio permitted the independent probabilities of incident waves and still water levels to be combined to develop the inter-related probability of overtopping, which is represented in Fig. 10.

13. Construction in July 1986 (photo: John Nutt)

There are a number of ways of expressing the risk of overtopping, one method being to calculate the frequency of overtopping for various crest levels and related wave generating events. A selection of these is shown in Fig. 11.

An assessment was made of the volumes of 'green' water which would surge over the crest if extraordinary wave events occurred, so as to design an appropriate drainage system.

Construction techniques

Being over the water, the concourse is in many ways like a wharf, except for one critical factor — it cannot be constructed using the same techniques since the deck level is within the intertidal range. Under both temporary and permanent conditions, uplift forces act on the underside. Those hydrostatic forces are both static and dynamic being due to tides and waves.

A number of construction options were developed and costed. One was to build the slab high above water level and lower large finished deck areas down onto the piles and lock into place — a reverse lift slab technique. Another was to build the slab within a dewatered sheet pile caisson. The adopted procedure is shown in Figs. 12(a) to (d).

The slab is built on reclaimed ground behind a mass concrete sea wall. Initially, two parallel lines of steel sheet piling, 4m apart, are driven along the line of the concourse edge. The marine sediments are dredged out to rock level, a depth of up to 5m in places, but averaging 3m. A mass concrete gravity wall is constructed by tremie methods to just below low water level, after which the top is dewatered so that the upper wall sections and the wave deflector can be constructed in the dry. Between this seawall and the land the area is filled to the underside of the slab so that, when dewatered, the slab can be cast on the fill surface in the dry. Piles at approximately 8m centres carry the loads to the underlying rock.

The work is currently well advanced (Fig. 13) and it is anticipated that construction will be completed by July 1987.

Credits:

Client:
NSW Department of Public Works

Architect:
NSW Government Architect,
Special Projects Branch, in association with Hall, Bowe & Webber

Construction manager:
John Holland Constructions Pty Ltd.

Seawall contractor:
Costain-Australia Ltd.

New Brisbane International Airport terminal access roads and car parks

Clive Humphries

Approximately 13km north east of the city centre, work is currently under way to construct a new international airport for Brisbane. Serving one of the most rapidly developing regions on the continent, the new facilities are scheduled to open in 1987, in time for the World Expo to be held in Brisbane in 1988 — the year of Australia's Bicentennial.

The decision to construct a new airport arose as a result of increasing passenger traffic and the trend towards larger aircraft in the 1970s. The outmoded terminals, limited runway length and noise factors served to restrict the extent of operations at the existing airport.

The location selected for the new airport lies immediately east of the existing main runway and extends to the fringe of Moreton Bay. Much of the site was an estuarine area of mangrove swamp with some low lying farmland. Work began on site in 1980 with the construction of a floodway to replace natural drainage channels. These would be lost under later stages of airport construction. A massive reclamation programme followed, involving the placement of 15,000,000m^3 of white sand dredged from the 'Middle Banks' area of the bay.

The entire project, currently estimated at A$380 M, is the responsibility of the Federal Department of Housing and Construction. The work is being undertaken on behalf of the Department of Aviation who will own and operate the completed facilities.

Arups' involvement

Ove Arup and Partners has been associated with several aspects of the new international airport. The most significant of these are the roads, car parks, services and other landside facilities associated with the main terminal area. This is the single largest and most complex civil engineering consultancy associated with the project.

The site of the car park complex lies at the north eastern end of Airport Drive, a new dual carriageway linking with the Gateway Arterial — part of the National Highway network presently being built through Brisbane's eastern suburbs.

Our transport planning involvement at Brisbane Airport commenced in 1981, some two years prior to the first of our commissions from DHC. At that time, the Department of Aviation (then the Department of Transport Australia) commissioned the Brisbane Airport Transport Study, in which airside and landside transport movements at the existing airport were studied. Surveys were conducted of traffic movements, parking demand, airport terminal occupancy, apron occupancy, general aviation movements and air freight activity. Analysis of this data yielded important planning parameters subsequently used by the Department in the planning of the new airport.

In late 1982, DHC invited us to undertake a review of car parking and associated access arrangements proposed for the New Brisbane International Airport. These plans had been prepared by DHC in response to the functional brief prepared by DoA and dealt basically with the roads, car parks and terminal frontage areas immediately adjacent to the domestic terminal.

Our brief required us to review the then 'current' scheme and two alternatives, and to develop one of these schemes (or an additional, preferred arrangement) to sketch plan stage. Assessment criteria included traffic circulation, intersection performance, car park access/egress, terminal kerbside operations, pedestrian-vehicular interaction and implementation feasibility, and cost.

A Schedule Impact Model (SCIM) was developed to determine the landside transport demands generated by alternative air passenger arrival/departure and aircraft movement schedules. SCIM used information from the Functional Brief to generate a distribution of passenger movements by time of day for the terminal 'busy day' in 1995, 'the design year' (7.091 M annual passenger movements through an assumed Joint User Facility). The model was verified by application to a 1981 aircraft and passenger movement schedule and comparison with data collected in the 1981 Brisbane Airport Transport Study.

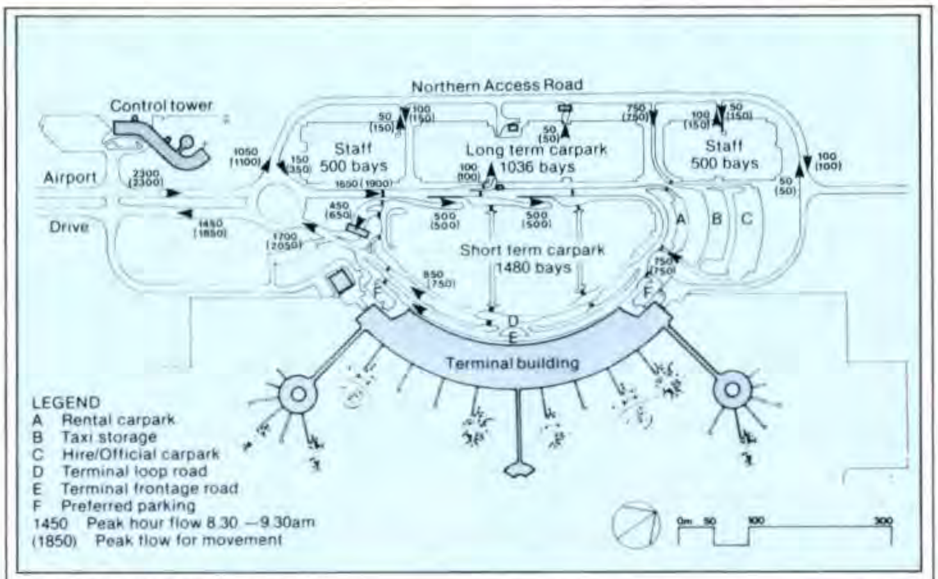
SCIM permits the user to specify a range of landside travel parameters: access modal split, proportional use of terminal kerbside, proportion of self-drive air passengers, air passenger vehicle occupancy and times of arrival before and departure after the flight. As the Functional Brief did not provide information on some of these criteria, a feasible range of values was tested for each factor, in order to determine the sensitivity of landside traffic movements to variation in these key influences.

A second computer model was developed to simulate the parking demands occurring at the terminal kerbside. This model uses the 'busy day' air passenger arrival/departure distribution and landside travel parameters to simulate peak period kerbside parking demands. Additional data inputs are proportions of air passengers with baggage and distribution of car and taxi drop-off and pick-up times at kerbside.

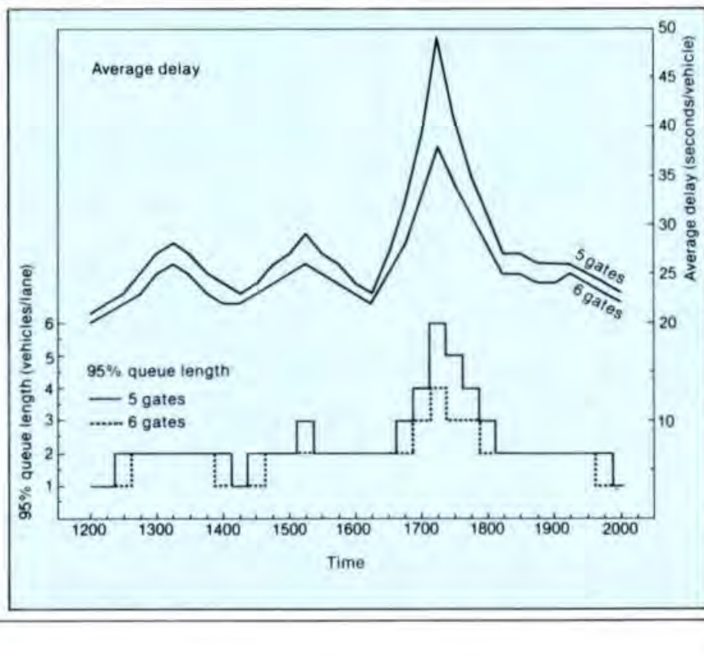
The review of the various schemes led to the recommendation of an alternative layout which displayed improved performance with respect to the nominated assessment criteria. In particular, the analysis of terminal kerbside parking demands had indicated an inadequacy in kerbside capacity; a twin carriageway terminal frontage road was therefore proposed, effectively doubling kerbside capacity.

By the time the design development stage commenced in April 1984, sand filling, varying from 2 to 3m in depth had been placed over the majority of the site. Analysis, undertaken by Arup Geotechnics, involved a review of the numerous investigations undertaken by DHC since the start of the project. These included subsoil data, consolidation predictions, and construction and settlement monitoring. From the parameters identified, assessments were made of short-term and lifetime settlement and other design criteria.

In addition to the design of terminal access, planning considerations included provision for: a through road to serve the general aviation area and airline freight and catering



1. Terminal area plan



2. Exit: Main short term car park.
Predicted exit queue length and average delay per exiting vehicle

facilities; parking for approximately 1,700 short-term, 1,200 long-term and 1,000 staff cars; storage and queueing areas for taxis, rental and valet cars and buses; and a series of pedestrian routes. Terminal design considerations dictated that both arrivals and departures be catered for at-grade. The sweeping arc of the building frontage, extending over 400m, provides undercover pick-up and drop-off areas, taxi ranks and bus stops.

At the commencement of the design development stage improvements were made to the SCIM model, in particular to accommodate an enhanced version of the Taxi 'call up' system so as to simulate the delays which might occur if insufficient taxis were queued on the airport. In addition, the model was refined so as to incorporate differences in peak and off-peak travel characteristics, reflecting differences between business and non-business travellers.

Traffic control system

Further analysis occurred immediately prior to the commencement of detailed design. Assessment of the operations of various key system components was undertaken in some detail, with the requirements of the design phase in mind. Peak period traffic movements for all links of the road system were generated, as an aid to the highway detailing. The operations of car park entry and exit facilities were analyzed in particular depth, in order to determine the adequacy of control system proposals and of queueing areas and to determine the appropriateness of the level of service to be offered to the consumer. The use of graphical output from the car park entry/exit submodel of SCIM proved invaluable in justifying the need for certain numbers of entry and exit gates if consumer levels of service were to be adequate in peak periods.

The design was generally based on the draft Australian Standard for car parking and where appropriate the Queensland Main Roads Department's highway design criteria. Special measures were necessary in a number of instances, e.g. to cope with long flat grades in the terminal frontage area and the irregular shape of the major short-term car park.

An aim of the design has been to separate the public, bound for the terminal environs, from service and delivery vehicles, staff users, etc. Access to all public car parks and the set down/pick up area is from the terminal loop road. This also provides for recirculation to car parks and the terminal frontage. Separate access roads have been incorporated for goods and commercial traffic servicing the terminal itself.

Use of computers

The detail layout was calculated using programs developed in house for the firm's HP 9845 and HP 200 series computers. Plans were subsequently drawn using an HP 9850 AO plotter. The ability to provide accurate base plans to a variety of scales for use by ourselves and the various subconsultants was of considerable advantage in meeting the tight documentation timetable.

The master planning of the facilities had nominated reserves for the main access road and trunk utilities. However, numerous underground services had to be accommodated within the site including high and low voltage electrical reticulation (involving separate essential and non essential circuits), communication cabling, DoA cabling, sewers, stormwater, water mains, landscape irrigation and telephone cabling. Considerable co-ordination of these services and the temporary supplies which crossed the site was involved.

Because of the low lying nature of the area and tidal influence it was necessary to design stormwater on the basis of near minimum grades. Pipe sizes were limited by the depth of cover available and a desire to retain the excavations in sand wherever possible rather than the soft underlying materials. As a result of these constraints the site was divided into a series of parallel catchments. Surface levels were designed to permit overland flow to occur in an extreme storm event.

The main pressure sewerage line passes through the car park complex. Construction involved sinking a 7m diameter caisson 11m deep to form the major pumping station for this part of the airport complex.

Several pavement construction alternatives were considered prior to the selection of granular base/sub-base with asphaltic concrete surfacing as the preferred option. Designs were produced for in service loading conditions ranging incrementally from car parks to the main access road. The thicknesses nominated were checked against various design methods in current use. For pavements built initially to serve construction traffic a bituminous spray seal has been used as a temporary surfacing, later to be overlaid by an asphaltic wearing course.

Approximately 50,000m^3 of high quality crushed rock was prepurchased by DHC. A large stockpile was formed adjacent to the main access road some 1 to 2km from the site. The aggregates were brought to a controlled moisture content in a pugmill prior to use. Wherever practical, placing has been by paving machine.

For the surfacing of the terminal frontage road, interlocking concrete blocks have been chosen both to harmonize in colour with the treatment of the building and to enhance driver perception of the pedestrian environment of this area. A consistent series of footpath finishes developed by the architects for the terminal have been followed through into the car park area.

Roadway and car park lighting has been an integral part of the design. Alternatives have been analyzed both on the basis of daytime appearance and night-time illumination characteristics. Lighting columns throughout the car parks have been kept relatively low and given the appearance of being randomly placed to blend in with the overall streetscape. Fully cut-off fittings are being utilized throughout to minimize glare effects.

Landscaping

Extensive landscaping using predominantly native species has been an inherent part of the design. Trees have been selected on the basis of their suitability for the site exposure and drainage conditions and to provide shade within the car parks. A particular requirement has been that they should not attract birds for feeding or nesting. To break up the flat appearance of the car parks, a series of landscaped mounds has been incorporated in the design.


3. Car park and internal access roads under construction


5. Roundabout under construction


4. Terminal and car park area: after reclamation, before construction


6. Subgrade trim and sub-base laying

7. Nursery for plants

8. Model of domestic terminal car parks and access roads

One of the initial contracts was for the establishment of an on-site nursery, including plantings. This has been done with the aim of acclimatizing the various plants during their early growth in the field.

Because of the arid and exposed nature of the location and the potential for saltwater ingress, an extensive system of drip feed irrigation and pop-up surface sprinklers has been designed to provide watering to most planted areas.

To enable parts of the site to be used for temporary facilities associated with the terminal building and control tower, and to allow access for other ongoing work, the construction stage was subdivided into seven separate and staged contract packages. The first of these, the General Aviation access road, went to tender in early 1984 whilst design of the remaining works was underway.

The sequential construction of the major underground services and stormwater lines, together with the need to maintain access routes across the site for concurrent contracts, were the major factors in determining the make-up of the various stages. Detail design and draft documentation of the six subsequent contract packages were undertaken on a four-week cycle. The first stage, consisting of road and car park works at the southern end of the site, was completed in late 1984. Package two, which included the majority of the underground services and main stormwater lines, was completed during early 1985. Stages 3, 4 and 6, which make up the majority of the car park and roadworks components, are being constructed.

Works associated with the terminal frontage area have been timed to coincide with the release of working space adjoining the main buildings and are about to go to tender (May 1986). This area contains numerous permanent and temporary services. Co-ordination of these, together with the sequence of foundation and pavement construction has been a significant consideration in the documentation process.

During 1985, we were asked to examine the possibility of incorporating an automated 'Pay on Foot' car park payment system into the development. Following an initial evaluation of the concept, a more detailed study was undertaken to assess the suitability for adapting such a system to the facilities as then designed (and under construction). The study involved an assessment of various systems available in Australia and included visits to operational sites in Australia and the UK. The introduction of a Pay on Foot system was seen as having operational and user advantages over a conventional 'pay cashier on exit' arrangement.

A decision was made earlier this year to proceed to detail design and documentation of a Pay on Foot system. The opportunity provided by a completely new facility is being used to establish a fully automated operation which will be the first on this scale in Australia. Seven automatic pay stations will be provided on the pedestrian routes into the car parks. The equipment will accept notes and coins as well as providing change. Intercom and TV monitoring will be linked to a central control.

Although the 'Pay on Foot' concept is not new to Brisbane, attention to aspects such as signage, equipment labelling and pre-publicity are considered essential if the system is to gain early public acceptance.

Credits

Client:
Department of Housing & Construction (DHC)
Principal consultant:
Ove Arup & Partners
Civil, traffic, geotechnical and structural
Associated consultants:
Bligh Jessup Bretnall:
Architecture
Lincolne Scott Australia:
Lighting and Electrical
Ledingham Hensby & Oxley:
Irrigation & Water
Reinhold Engineering Consultants:
Sewerage
Bernard Ryan & Associates:
Landscape Architecture

9. Aerial view (Photos: 3 and 9 courtesy Department of Housing & Construction)

Newlands coal wash plant, Queensland

Ron Bergin

INTRODUCTION

Coal mining in Australia is by far the largest export earner with receipts of A$5 billion in 1985, accounting for more than 15% of the country's total export revenue. The record 88 million tonnes of coal exported in 1985 placed it more than 10 million annual tonnes ahead of second-placed United States.

In 1981 MIM Holdings Ltd. became a major force in the industry when it announced plans that its Newlands export coal project was to proceed with planned operation by mid-1984.

A total of 5 million tonnes of coal per year was to be exported through a new coal port at Abbott Point (Fig. 1). This would comprise 4 million tonnes of steaming coal by 1985 from the Newlands open cut mine, and 1 million tonnes of coking coal a year from the Collinsville coking coal project. The railway would be extended from Collinsville to Newlands and the new town of Glenden constructed to house 1,000 people.

Abbott Point would have a stockpile and stacker-reclaimer facilities for 6.5 million tonnes per year with provision to extend. The port would be capable of accommodating vessels up to 160,000 dwt and, under certain conditions, 190,000 dwt.

THE COAL WASH PLANT

In August 1981, Mimets Developments Pty Ltd., the project arm of MIM, engaged Mitchell Cotts Projects Pty Ltd. to design the wash plant, a major component of the mine infrastructure.

Quoting from the brief to Mitchell Cotts:

'In essence the work will involve the preparation of all necessary design drawings, calculations, specifications and bills of quantities to enable the Company (Mimets), to contract others for the supply and construction of the total wash plant facility.

While the objective of the work defined by this brief is to design a wash plant to produce 4×10^6 t/a of washed coal (measured on an air dried basis) the design must take account of possible future production expansion. In particular space provision, and other provision as specified from time to time by the Company, must be made for the addition of a similar 4×10^6 t/a wash plant to give a total capacity of 8×10^6 t/a.[1]

Ove Arup & Partners had built up a strong relationship with Mitchell Cotts over the years on iron ore and coal projects in Western Australia and New South Wales. We were therefore invited to put in a proposal for the structural engineering of the wash plant and were successful.

We were not responsible for civil engineering or foundations. These were to be done by the consultant responsible for the overall civil engineering of the mine infrastructure. Mitchell Cotts were responsible for all process and services engineering, with Arups responsible for all supporting structure in concrete and structural steel from the underside of base plates upwards.

In addition to the wash plant buildings, our brief included all Conveyors and transfer towers plus the structural design of bins, settling cones and thickener tanks which formed an integral part of the process itself.

PROCESS DESCRIPTION

The raw coal is treated in two separate sections, a dry treatment section and a wet treatment section (Fig. 2).

The run-of-mine (ROM) coal, after primary crushing to −75mm, is stored in a raw coal bin and is dry-treated by further crushing, screening and dedusting to produce a suitable feed for the wet treatment section as well as remove a maximum of −0.5mm fines which are sent direct to product. A controlled amount of raw small coal, 20mm to zero, is bypassed around the wet treatment section to be blended with the wet treatment product and the dry fines, to produce a consistent final product.

Raw coal for the wet treatment is stored in a surge bin, where it is withdrawn at a controlled rate by vibrating feeders to undergo desliming, jigging, screening and centrifuging to produce clean coal while slimes are treated in a thickener before being discarded to a slimes dam. The reject material from the wet treatment is stored in a reject bin for disposal by a 77 tonne rear tip truck.

DESIGN

Design brief

The design brief issued by Mitchell Cotts called upon Arups to design all structure and cladding of the wash plant for prescribed dead, live and wind loads, taking into account 'dynamic loading from plant and equipment without undue vibration and within reasonable deflection limitations'.

SAA and MIM standards were to apply unless specifically altered by the brief.

Mitchell Cotts would provide loading schedules giving dead, live and dynamic characteristics of all equipment to be installed. Machine volumes and bulk densities would be provided along with blockage and spillage allowances for certain items of equipment.

The wash plant would be represented by a set of general arrangement (GA) drawings showing process, access and maintenance space requirements in plan and elevation.

These would form the basis for the structural engineer to put an orthogonal structure of beams, columns and bracing about the process in the first of many design iterations, leading to a final solution as the total design evolved.

The brief included design criteria for beams and columns supporting vibrating loads. To avoid resonance during start up and operation, the natural frequency of beams was to be not less than 1.5 times the imposed frequency. Due to the nature of the structure, zero damping was assumed and the maximum permissible bending stress limited to the range 48-88 MPa, the upper values applying to low ratios of imposed frequency: natural frequency.

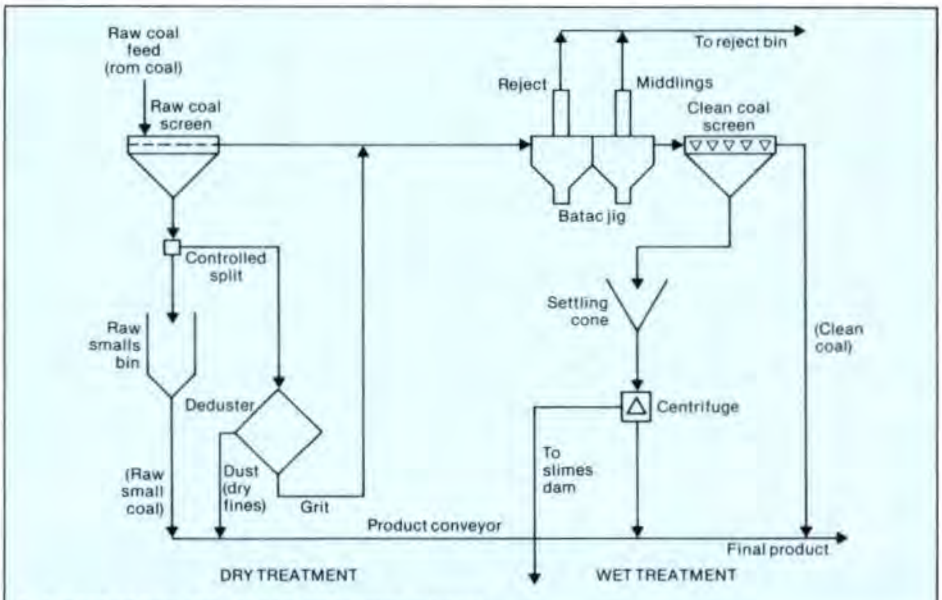
In addition to vibrating load calculations, all beams were, of course, to be designed for dead and live load with mandatory deflection checks prescribed for span: depth ratios in excess of 16.5.

Columns supporting vibrating loads were to have a maximum slenderness ratio of 80 commencing from the level immediately above excitation and terminating at column base. All structural steel was to be grade 250 in accordance with *AS1204* Structural Steel Ordinary Weldable Grades.

Wind loading was to be in accordance with *AS1170: Part 2* 'Wind Loads', assuming a return period of 50 years, Terrain Category 2 and basic wind speed of 45 m/sec. In general, an impact factor of twice the dead load and/or torque reaction was to apply to selected equipment and conveyor head and drive pulleys, and the full effects of temperature movement taken into account. Live load on general floor areas was to be 7.5 kPa unless specifically altered for installation or maintenance.

Our work was to culminate with a full set of dimensioned working drawings suitable for steel shop drawings to be prepared, reinforced concrete drawings for solid floors complete with kerbs and drainage, and open

1. Location map



2. Simplified flow chart of coal preparation process

3. Raw coal building

mesh floor layouts for minor floors and platforms.

Interim milestones included a preliminary steel order of major sections and plates to offset long lead times with steel delivery, and load diagrams for the civil engineers to design foundations for our structures.

Structural planning

The structural components of the wash plant could be broken down into a number of discrete design packages. There was the raw coal building and the washery linked by a series of conveyors transporting coal in various stages of preparation (Fig. 4). Within the raw coal building and washery were raw smalls bins and surge bins respectively, each a separate design package as were the settling cones and thickener tanks beyond the washery on the 'product' side. There was also the stand alone reject bin.

We therefore set up two design teams, one under John Ryder responsible for both the raw coal and washery buildings, the other under Alan Saxon for conveyors, bins, settling cones and thickener tanks.

Ian Mackenzie was consulted on the most appropriate way to plan design and documentation, for he had led the design team on two previous heavy medium coal washerys at Saxonvale and Stockton Borehole in the Hunter Valley of New South Wales. We resolved that plans and elevations for each level and each grid line longitudinally and laterally would be drawn for both buildings in order to give us flexibility to document changes to any detail, which invariably arose given the iterative nature of this type of design.

We were not starting from scratch. Mitchell Cotts GA drawings were set out around an orthogonal spacing for columns and beams that, by experience, could be quickly worked into a supporting structure. We were thus able to commence drafting plans and elevations concurrent with frame analysis for overall load support and stability.

It was necessary at this time, to agree upon a bracing principle so that the distribution of horizontal force through the structure could be assessed. Despite the disadvantage of low dead load to counteract uplift, we agreed all bracing be placed in outside bays clear of the main process itself.

Storage structures

The raw smalls bins, surge bins, settling cones and reject bin comprised the storage structures of our commission. In all cases these were of steel plate construction although the form and structural system varied.

The principal loadings on these structures resulted from the stored material, this being coal at various stages of the process through the plant. Current design methods for this type of structure recognize two basic loading conditions during charging and discharging, these generally being referred to as static and dynamic loadings. Generally the dynamic loadings are more onerous than the static case.

A number of methods for the determination of pressures within the bin are available, which, to a lesser or greater extent, recognize the effects of static and dynamic loadings. In Australia, at the Universities of Newcastle and Wollongong, considerable research effort has been directed to attempting to rationalize the various design approaches and identify suitable methods in an area of engineering analysis which is still somewhat empirical.

After a review of the available methods we followed the approach identified by the above-mentioned universities and this resulted in pressure determination using the methods proposed by Janssen, Jenike, Walker and Walters. The basic input to the analysis was the bin geometry, which was defined by Mitchell Cotts, and coal properties. Tests were carried out on what was considered to be representative samples to establish the necessary properties. From this data, using the methods mentioned, pressures were established. These varied with location and structure up to a maximum of around 300 kpa as a local pressure concentration.

The structural analysis methods adopted varied with the structures because of their geometry and form. The raw smalls and surge bins, being plane-sided, consisted of a series of two-way plates supported by beams, and the approach adopted was to use standard formulae for plates together with skeletal frame analysis where necessary. The reject bin and settling cone were cylindrical and conical in shape and the finite element method was used with 2D membrane or plate bending elements. The analysis was carried out using the finite element program PAFEC.

In the case of the settling cone, an axisymmetric model was used and for the reject bin a ¼ sector was analyzed.

Design sequence

Structural design commenced by studying the GAs and locating loads from the equipment load schedules. Preliminary beam sizes were determined for the purpose of checking headroom and clearances for equipment, access, chutes and pipework. Attention was focussed on areas carrying vibrating machinery and although much of the load information was preliminary at this stage, with some knowledge of the imposed frequency, a reasonably accurate estimate of beam size could be determined and future problems avoided in these critical areas.

A system of simple cross bracing was developed for each grid line in accordance with the principle to avoid the main plant area, and checks made that adequate clearance existed for access, pipework and chutes. Where clashes occurred, the bracing was converted to K or A bracing to avoid the conflict.

Design then proceeded on an iterative basis between structural and process/services engineer until it was complete for the final choice of equipment type and load. Marking plans and elevations were completed for each level and grid and cross-referenced to connection details for each different node type.

Concurrent with building design, separate designs were prepared for the conveyor system, raw smalls and surge bins, settling cones, thickener tanks and reject bin.

The design programme called for a preliminary steel order (PSO) to be placed at the end of week 15 so that the majority of steel would be available to the fabricator at the start of his contract. The PSO was to contain all section and plate that had been designed to that date, especially those sizes not normally held by stockists. Contract documents were written that the successful fabricator would take possession of the order from the client and be responsible for a second order enabling complete fabrication in accordance with the working drawings.

COMPONENT PARTS

Buildings

The raw coal building is a seven level structure some 28.5m in height over a plan area of 28.7m × 23.8m (Fig. 3). It is set out on a regular grid varying in spacing from 3.7m – 8.3m to suit plant layout. Generally 310 UC columns were used with beams ranging from 200 UB – 760 UB depending on load and span. The major items of plant contained in the building and their operating loads are tertiary crushers (92 tonnes), raw coal bins (300 tonnes) and dedusters (196 tonnes). Raw coal screens and raw smalls feeders imposed loads at $14H_z$ and $33H_z$ respectively.

The washery is a larger structure measuring 39m × 40.4m in plan but rising to approximately the same height (Fig. 5). As with the raw coal building it is completely open at the sides but does have roof cladding, a detail most suited to the high temperature, low rainfall climate. Grid spacing varied from 3.8m – 6.0m with 310 UC and the complete range of UBs being employed for columns and beams. Major loads carried in the building were from surge bins (170 tonnes) and batac jigs (240 tonnes). Screens and feeders imposed loads of similar frequency to the raw coal building.

Beams were generally designed as simply supported with continuous two span beams only used to meet headroom requirements or avoid plate girders. In all cases these were beams subject to dynamic or exceptionally heavy loading. Where effective lengths were excessive, horizontal floor bracing was provided to reduce beam sizes.

Beam end connections were rationalized by adopting a simple fin plate detail as standard for the static load case (Fig. 6). Where the shear exceeded the fin plate capacity, this was replaced by an end plate detail bolting to the beam or column as the case may be. For dynamically or torsionally loaded beams, full depth end plates with lowered flange plates on coped beams were adopted as standard detail.

M20 bolts were adopted as standard with some M24 used in larger connections. All bolts were torqued to avoid nuts vibrating loose. TB (bearing) was specified as the bolting system with TF (friction) used only where non-slip joints were required.

31OUC97 worked well as the standard column size with heavier 31OUCs used as required. End-bearing column splices were specified, with M24 and M30 HD bolts used in a series of standard base plate details.

Angle X-bracing was used as a first preference, with SHS A-brace or K-brace configurations adopted where X-bracing could not be coordinated with process or access requirements. Centrelines of bracing members were made coincident with the face of the column web or flange. In this way bracing cleats were always welded to the beam and hence the beam end connection was required to transfer shear (and triaxial where these occurred) forces only. Resulting eccentric load bending moments about column major axes were quite low and easily accommodated.

Horizontal 'in floor' bracing was used where concrete floors were not present to transfer horizontal forces to vertical bracing. These were detailed as pre-welded horizontal trusses bolted into position to avoid the need to fit numerous small members and so slow down the erection process.

Conveyors

The conveyors within the wash plant are all elevated and supported from a series of trestle frames (Fig. 7). Each conveyor either terminates at a building or transfer tower at which the conveyor is supported both vertically and laterally.

4. Coal preparation plant

5. Washery

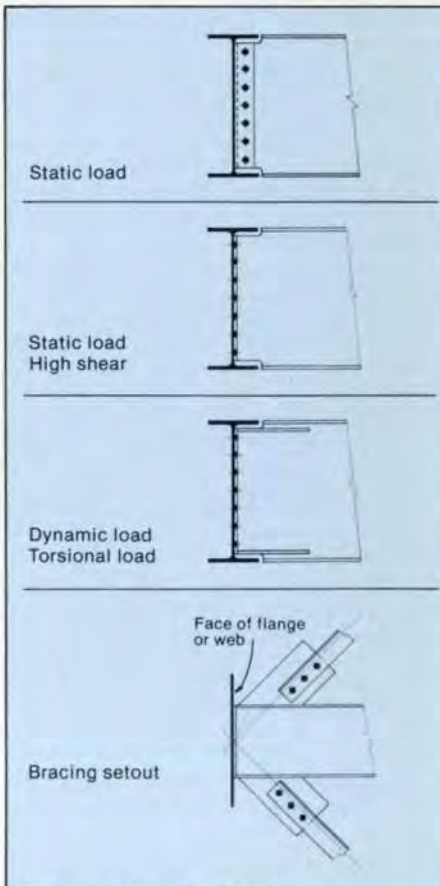
During the early stages of design development, efforts were made to rationalize the conveyor supporting structure to simplify construction and reduce costs. Typically each conveyor is supported by a box truss which generally spans 12 to 18m. The trusses are 1.34m wide and 1m deep and fabricated from angle members with RHS end frames, all being fully welded construction.

The support trestles are basically plane frame structures ranging in height from 3m to 21m. The trestles were rationalized into three families and were fabricated from universal beam section legs between 410 and 250mm in depth with angle K or X-bracing and bolted connections.

Raw smalls bin

The raw smalls bin is located in the raw coal building and includes a series of screens within the bin itself. The bin is in fact one of a pair consistent with the two parallel production lines which run through the wash plant complex. Each bin is 7.1m wide × 6m deep with an overall height of 13m. Charging is through a single hopper over the full width with two discharge hoppers at the base of each bin. The nominal washing capacity of each bin is 175 tonnes and for structural calculation the bins were rated at 300 tonnes each.

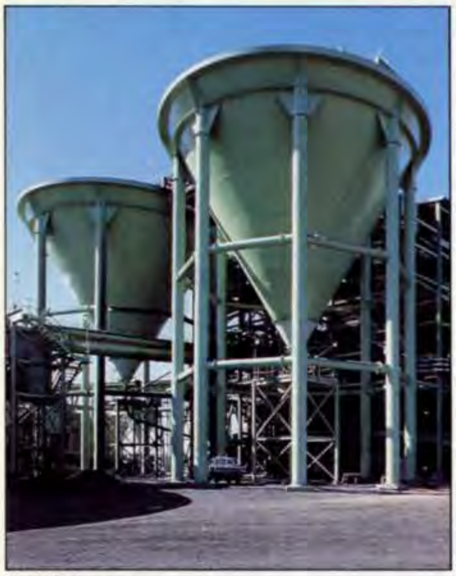
The bin sides are planar and constructed from flat plates with thickness varying from 10mm to 20mm. The plates are supported on a series of secondary supports from 127 × 64 channel, and primary supports acting as ring beams ranging in size from 200 UB to 760 UB.



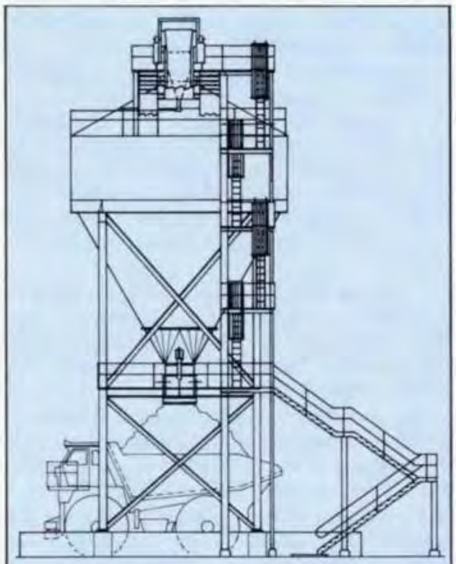
6. Beam end standard details

7. Conveyors

10. General view of Newlands coal wash plant



8. Settling cones



9. Reject bins

Surge bin

The surge bin is located in the main washery building at the start of the coal washing process. As with the raw smalls bin, the surge bin is in fact a pair of bins to satisfy each process line. The bins are 7.2m wide, 4m deep and 7.5m high. As with the raw smalls bin, each bin has a single charging hopper and dual discharge chutes.

The nominal working capacity of each bin is 100 tonnes with a capacity for structural purposes of 170 tonnes.

The bin sides are planar and made up from plates ranging in thickness from 8mm to 16mm supported on 230 × 76 channel secondary beams. Again primary beams act as ring beams and are fabricated from 310 UC sections.

The bins are supported from the main frame members in the washery building and are totally integrated into its structure.

Settling cone

The settling cone (or pair of cones) are located outside the washery building at the end of the washing process (Fig. 8). As the name implies they are used to settle solids from what is basically a liquid and hence the loading is different from the bin structures described.

Each inverted cone is 16m in diameter, 14m high with the top of the cone located 24m above ground level. The operating capacity of each cone is 160 tonnes, the self weight approximately 60 tonnes and the support frame 40 tonnes.

To give a pleasing form to such large structures, cross-bracing between the supporting columns was eliminated and vierendeel action relied upon between tubular beams and columns to provide stability.

Plate thickness within the cone varied from 12mm to 20mm at the ring beam, with the support frame fabricated from 813mm diameter tubes with 508mm diameter horizontal bracing members.

Reject bin

The reject bin is a self-contained cylindrical structure with cone-shaped roof and hopper elements supported at four points on a braced frame (Fig. 9).

The bin is 9m in diameter with an overall height of 15m. The nominal capacity of the bin was 300 tonnes with a capacity for structural purposes of 450 tonnes. It was constructed from 10 to 12mm plate and the support frame was fabricated from 310 UC and 630 UB sections.

PROJECT CONTROL

As with all projects in the natural resource development industry, project control procedures on time and cost were comprehensively defined in the brief.

The proposal on which we were commissioned contained a complete schedule of the drawings and documents to be produced together with an outline programme detailing engineering and drafting assignments over the duration of the project. These were expressed as cumulative man weeks S-curves and formed the basis for monitoring our progress at four-weekly intervals, coinciding with the client's overall project monitoring.

At the end of each so-called cost period, the brief called for a structural engineer's report detailing the achievements in the period, the current approved budgeted manhours for the entire project, any claims for increased manhours due to change of scope, and a progress report comparing actual progress against the S-curve. Corrective proposals were required for any shortfall.

Time basis fee claims were submitted at the end of each cost period supported by copies of project time sheets drawn up to coincide with the period dates.

CONCLUSION

Mobilization for the Newlands Mine Project — Coal Wash Plant had commenced in November 1981. In June 1982 the commission was formally concluded. In that period we had designed and detailed some 1,400 tonnes of structural steelwork, extensive concrete works and accepted extensions of our commission to lead the building design team on the four-storey control building and other peripherals.

We had worked with process engineers as our principal consultant and contributed to a design where structure is totally subordinate to the process itself. We had made judgements to ensure reliable operating performance, for any malfunction causing partial or total shut-down involved losses far in excess of any savings accrued by refined design. Newlands had required a sound first principles approach to its unique set of design problems.

On 3 December 1983, the Governor of Queensland, Sir James Ramsay, officially opened the Newlands Mine Project. In mid-1984 Newlands produced its first coal, on schedule. It is now operating at full capacity achieving outputs in excess of targets, the hallmark of a successful enterprise.

Credits

Client:
Mimets Developments Pty Ltd.

Principal consultant:
Mitchell Cotts Projects Pty Ltd.

Structural engineers:
Ove Arup & Partners

Acknowledgement

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The R&I Bank tower

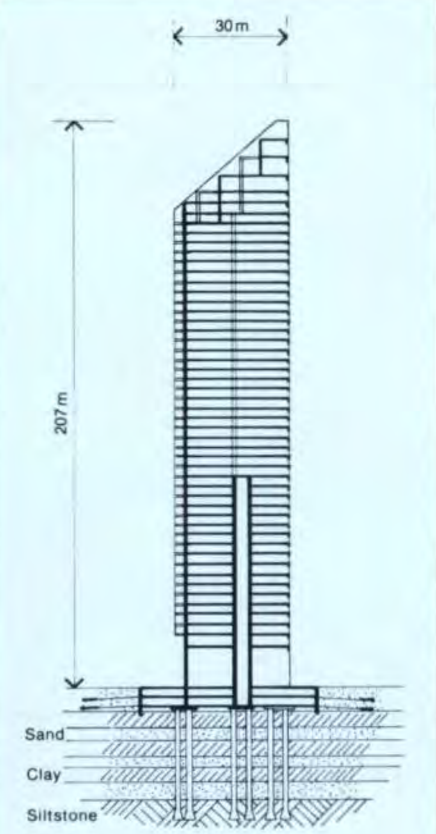
Dan Ryan

Architect: Cameron, Chisholm & Nicol

The brief

In August 1980 the Bond Corporation issued terms of reference to study and evaluate the design of an office tower on the Palace Hotel Site, St. George's Terrace, Perth, while retaining the original three storey Palace Hotel Building which carries a National Trust Classification.

The architects, Cameron Chisholm & Nicol, and the consultant team, developed a dramatic 48 storey building which satisfied the major parameter that all offices should face the excellent views of the Swan River, down its length to Fremantle Harbour and the Indian Ocean. At the same time it responds to the Old Palace Hotel and the two major roads which intersect at the corner of the site.



1. Section through the building

The site

The site is on the north east corner of the intersection of William Street and St. George's Terrace in central Perth and is 67.1 × 58m. The original Palace Hotel occupies the front corner of the site. Constructed during the hectic goldrush days in 1894 for John de Baun, it has been a landmark for Western Australians for many years.

The design

The solution adopted was to provide a building with a triangular floor plan, that faced south west towards the predominant views and the Old Palace Hotel. A series of bay windows on a 6m module were formed on the main facade to provide six 'corner window' offices in the prime position on each floor. All service areas, lifts, stairs, toilets and air handling rooms are located on the northern and eastern facades where the outlook is much less interesting.

The building at 207m high above ground floor level will be the tallest in Perth and the third tallest in Australia. The triangular floors have a side dimension of 42m, giving net lettable floor areas of 866m^2 for medium and high rise and 782m^2 for low rise floors. There are two basements for car-parking and an open pedestrian plaza area at ground floor.

The ground floor foyer is 20m high so that the first floor is above the roof level of the Palace Hotel. An acrylic canopy will link the tower ground floor and plaza area with the Old Palace Building.

Above ground floor, there are 48 storeys of office and executive accommodation. The three topmost floors in the triangular spire will house the principal executive suites.

The cladding to the building consists of double-pane glass and fluoropolymer-coated aluminium panel curtain walling.

Programme

Construction of the diaphragm walls commenced in August 1981. Excavation and installation of ground anchors followed, then piles were installed from basement level. Construction continued until the ground floor was complete in April 1983 then ceased while a principal tenant was sought for the building.

The Rural and Industry Bank of Western Australia joined with the Bond Corporation as a joint venturer and principal tenant in March 1985 and construction recommenced in May 1985. The building is due for completion in December 1987.

General description of the structure

The building is a reinforced concrete structure. It is triangular in plan with concrete shear walls on the two shorter sides and columns and framing beam along the hypotenuse. As the centre of rigidity is eccentric from the centre of mass and it is a reasonably slender tower, there was concern that side wind forces could result in unacceptable torsional vibrations.

A wind tunnel test was undertaken to establish and confirm wind pressures to be used for the static and dynamic analysis, with particular attention being given to the torsional characteristics of the building.

The typical floors are of conventional in situ reinforced concrete beam and slab construction. Various floor systems including steel beams were considered but the in situ beam and slab proved the most practical and economical for this building.

The building is founded on forty-three 1.5m diameter piles belled out at the base to found on consolidated siltstone approximately 30m below basement level.

The basement walls are in situ concrete diaphragm walls constructed from the original basement level. These were retained by prestressed ground anchors installed as excavation proceeded. Diaphragm walls were selected to minimize possible damage to the adjoining perched buildings.

Structural analysis for lateral loads

The final structural analysis for lateral loads was carried out in the following sequence:

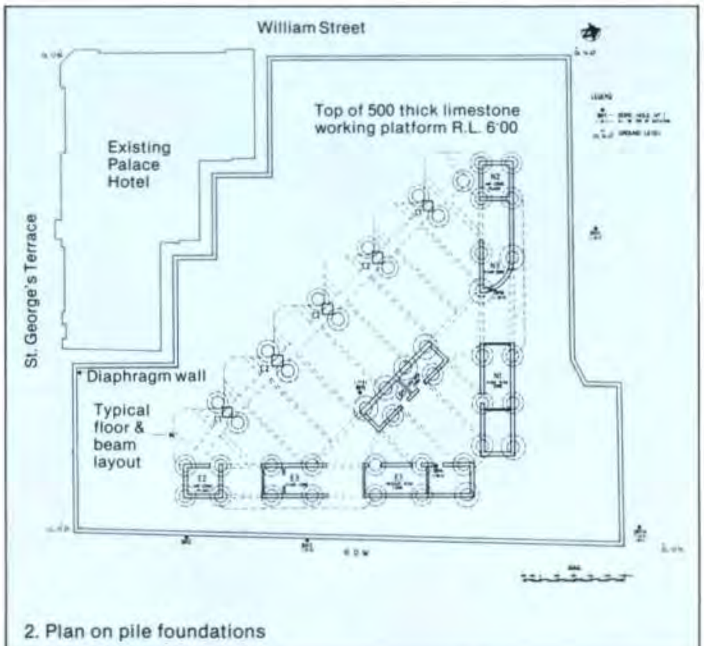
(1) Static analysis with unit loads of a 54 storey half model fully fixed at pile cap level

(2) The deflections due to the unit loads were used to construct a flexibility matrix for a 6 node model on which one model analysis was carried out to determine periods of vibration

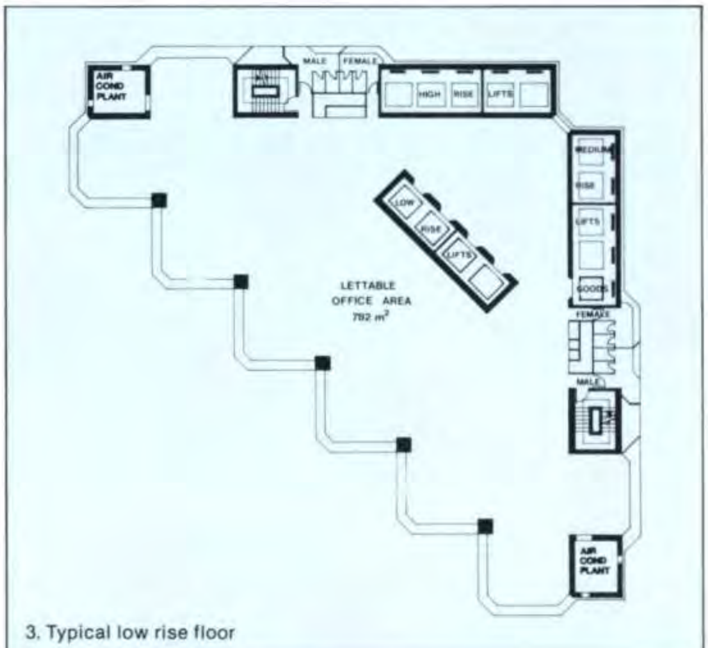
(3) The wind loads from the wind tunnel test were applied to the static analysis model to give displacements and member forces.

Wind tunnel testing

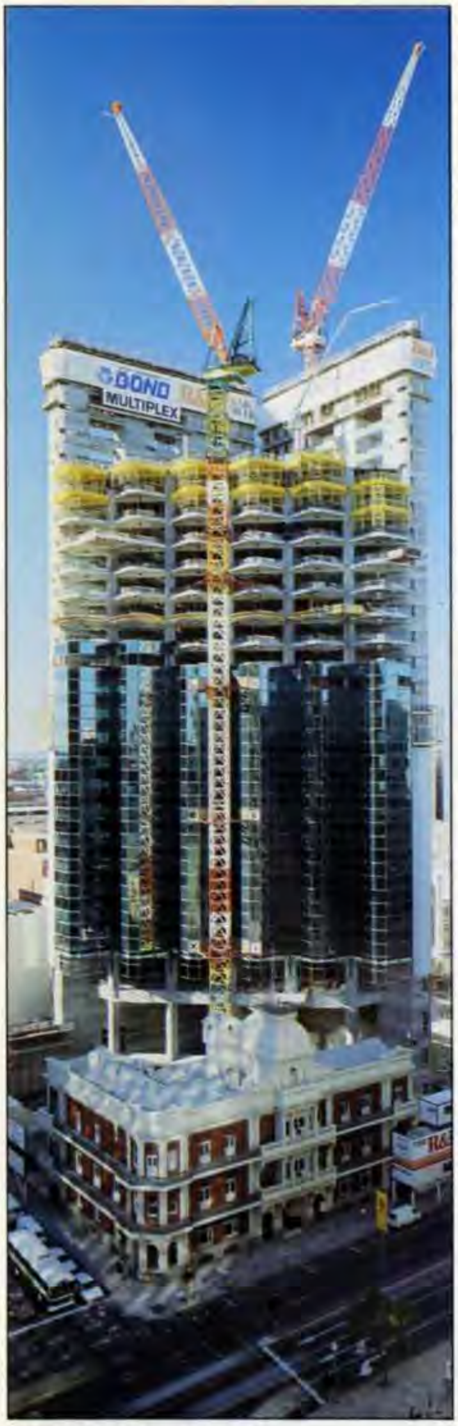
The shear centre of the structural arrangement is very close to the right angled corner and the centre of mass a considerable distance away at about the centre of the triangle. The potential for significant torsional coupling with the sway modes with the centre of mass significantly offset from the shear centre is considerable.



2. Plan on pile foundations



3. Typical low rise floor



4. Tower under construction


5. Photo montage: the tower as it will appear

The modelling of the torsional behaviour becomes particularly important for obtaining representative estimates of the wind-induced accelerations near the perimeter of the structure. In addition the wind design pressures determined from the wind code require some verification as there is little published information on drag coefficients for triangular buildings.

A programme of aeroelastic model testing on a 1:400 model was carried out in the 450kW Boundary Layer Wind Tunnel at the Department of Mechanical Engineering, Monash University, during February and March 1982.

So that the model would reproduce the structural action as closely as possible to that of the full-scale structure, two aluminium sheets were used for the north and east core walls, and, the south west facade was modelled using moment-resisting columns and beams, calculated to give a correctly scaled facade sway stiffness. For the test series this facade stiffness was varied over an order of magnitude to determine the building response characteristics as a function of the overall building torsional stiffness. The model was mounted on a strain gauge balance which could measure base overturning moments about the x and y axes, and a torsion balance was fitted to measure the torque produced by a rotational displacement of Level 43 about the base about the z axis which was fixed at the estimated mean position of the shear centre.

The mean wind velocity and turbulence intensity profiles were designed to approximate Terrain Category 3. The effect of the city buildings (Terrain Category 4) was taken directly into account by including the surrounding city buildings on the model.

The structural damping was varied between 0.5% and 2% of critical dampings.

Comparison of code and tunnel derived design forces

For triangular buildings *AS 1170* gives the following pressure coefficients:

(1) For a wind direction of $\theta = 180°$
   Cp = +0.85 on the windward wall surface
   Cp = −0.50 on the leeward wall surface

(2) For a wind direction of $\theta = 45°$
   Cp = +0.85 on the windward wall surface
   Cp = −0.70 on the side wall surface
   Cp = −0.50 on the leeward wall surface.

These values of Cp for a wind direction of 180° are consistent with the drag coefficient of 1.30 recommended for a tower-like structure of triangular cross-section.

For a basic regional design speed of 40m/s in Terrain Category 3, the base moments determined in accordance with the Peak Gust Method, Gust Factor Method and the Wind Tunnel Tests are compared in Table 1 for a wind direction of 180° and 45°.

Contrary to code data, the mean response of the structure is similar for wind directions of 0°C and 180°. The corresponding value of the drag coefficient for the tests is approximately 1.25, whereas *CP3*, SABR, ESDU and *AS 1170* recommend values of 1.3 for wind on the flat face and 1.1 for wind on the apex. It has been suggested that in turbulent flow, reattachment does not occur and consequently the value of Cp for a triangle in either orientation approaches that of a flat plate.

The aeroelastic model tests indicate that the Peak Gust and Gust Factor methods overestimate the along-wind response. Where the wind flow is not parallel to the plane of symmetry, however, the structure is susceptible to cross-wind effects, thereby significantly increasing the overall forces on the structure. Consequently the wind directions producing the highest moments are $\theta = 45½$ and 135°.

The overall effect of the surrounding city buildings is relatively small. Although there is some change in the wind direction for which the maxima occur excluding the surrounding structures, there is no significant change in the magnitude of the base moments.

Aerodynamic coupling

Cross-coupling effects of the sway into the torsional mode were shown to be negligible even when the frequencies of the torsional Mode 1 and sway Mode 2 were coincident. This is because the out of balance mass is in the plane of the sway oscillation.

In the orthogonal plane the centre of mass is offset from the shear centre, resulting in significant superposition of the torsional Mode 1 into the base moment for Mode 4. If the frequencies of Modes 1 and 4 became coincident, significant cross-coupling of the sway mode into the torsional mode would be expected.

Displacements and accelerations

Based on a 50-year return period, the maximum horizontal displacements and twist for the uppermost floor level are as follows:

Table 2: Maximum displacements for a 50-year return period

Wind direction	Y-Displacement (mm)	X-Displacement (mm)	Rotation (radians)
$\theta = 45°$	115	23	−0.0064
$\theta = 135°$	−176	45	−0.0043

Analysis of the peak acceleration levels indicates an annual maximum torsional acceleration of 0.7%g and an annual maximum sway acceleration of 0.9%g. Based on an integration of the response data with wind speed data, it can be shown that the horizontal accelerations near the human perception limit of 1%g will occur only once in a return period of between two and five years which is well within acceptable limits.

Sub-soil conditions

Golder Associates carried out a site investigation of six boreholes. The results, together with those from an earlier borehole,

Table 1: Comparison of Code and wind tunnel-derived design moments

Wind direction	Design method	Base moments (MN – m) Mx	My	Mz
$\theta = 180°$	1 Peak gust method	1870	—	—
	2 Gust factor method	1674	—	—
	3 Wind tunnel tests	1397	500	118
		(1614)	(700)	(206)
$\theta = 45°$	1 Peak gust method	−710	1028	−212
	2 Gust factor method	−691	1016	−214
	3 Wind tunnel tests	−1088	1122	−226

Notes:

1 The base moments determined by the Peak gust method have been reduced in accordance with Table 5.2 – *AS 1170*.

2 The values in brackets are the maximum base moments for wind direction of $\theta = -135½$.

allowed the stratigraphy below the site to be generalized as follows:

- from existing ground level at about RL 14 to about RL 6: Sand varying from loose to very dense with increasing depth
- from RL 6 to about RL −13: interbedded layers of silty clay, clayey sand and sand; clayey materials stiff to very stiff, sandy materials dense to very dense
- from RL −13 to between RL −17 and −21: sand, dense to very dense
- between the sand and the siltstone a layer of silty clay about 1m thick
- from between RL −18 and −22 to the depth investigated (RL −43): siltstone, thinly bedded varying from extremely weathered to slightly weathered.

The bedrock beneath the city area is a calcareous sandstone, siltstone or shale known as the King's Park shale. Fossil analyses of this material confirm that it was formed during the early Tertiary Age under marine conditions. Subsequent changes in relative levels between land and ocean exposed it as a land surface during the late Tertiary period. There was then a break in further sedimentary action until the start of the deposition of the younger unconsolidated alluvium during the Quaternary period. The King's Park shale occurs fairly consistently throughout the metropolitan area and has been proved to a maximum depth of about 300m.

In colour the material ranges from dark grey to black; in composition a clayey sandstone, siltstone or shale — all calcareous; it contains fossils, lime nodules and is laminated or thinly bedded.

The rock tends to break readily along the weak bedding planes, which makes handling and laboratory testing extremely difficult.

Perched groundwater levels were measured at about RL 9 in the upper sand, with artesian piezometric heads in the lower sand layers at about RL 2.

Substructure

The existing ground level on the two street frontages varied from RL 13.40 to RL 14.50, and the underside of the lower basement level is RL 4.00. The existing Palace Hotel, which is founded at RL 10.20, and a new six storey building on the North boundary founded on a raft at RL 10.26 had to be safely supported so an anchored in situ diaphragm wall was considered the most economical and reliable means of supporting the adjoining buildings and the excavation. The diaphragm wall, a nominal 500mm thick, was cast in panels varying in length from 4.5m adjoining the rather delicate Palace Building, to 7.0m where sensitivity to movement was not as critical. The panels extended to a maximum depth of RL 1.75. The single depth basement walls on the street boundaries to the original building on the site were left in place and stabilized by ground anchors. The diaphragm walls were constructed inside of these and from the existing basement level RL 11.25. On completion of the diaphragm walls a 250mm thick retaining wall was constructed up to Ground Floor Plaza level.

Prestressed ground anchors to stabilize the walls were installed as excavation proceeded.

Precise survey monitoring points were established on the top of the walls to check for possible movement during excavation and stressing of ground anchors. The maximum movements recorded were 2mm horizontally and an upward heave of 5mm.

The basement slab is 300mm thick on blinding over a 400mm thick blue metal drained subgrade.

Piled foundations

Initially a raft foundation as has been used on other high rise buildings in Perth was considered, but the average bearing pressure of about 500kPa would have resulted in unacceptable settlements. Therefore a number of deep foundation options were considered:

(1) Piles or barrettes founded in the siltstone

(2) Piles founded in the lower sands

(3) Raft sharing the load with piles founded in the lower sand.

It was considered that the design of piles founded in the lower sands should ignore the end-bearing capacity due to the difficulty of ensuring that the pile base rests on undisturbed material. The skin friction available for a pile founding just above the siltstone was not sufficient to allow a practical configuration of piles of the required capacity. Even sharing the load with a raft they would have had to extend nearly to the siltstone, so this would not have been viable when the cost of the raft was included.

It was therefore decided that piles or barrettes founded in the siltstone and using a combination of side friction and end-bearing would be the most suitable type of foundation.

The minimum value of shear strength of the clayey layers obtained from the direct shear tests and unconfined compression tests was 219kPa, but shear strengths as low as 140kPa were estimated using the pocket penetrometer. Therefore, a conservative value of 150kPa was assumed for design purposes.

In all of the in situ pressuremeter tests on the siltstone, the pressure required to initiate failure in the rock was greater than the capacity of the pressuremeter (7MPa). From this it was deduced that the shear strength of the rock is greater than about 2MPa.

However, the unconfined compression tests showed much lower values of strength and considerable variability, therefore a design shear strength for the siltstone of 1MPa was chosen.

Though this value was believed to be reasonably conservative when the pressuremeter test results are taken into account, it was decided that piles or barrettes should be founded at least 2m below the surface of the siltstone to avoid the possibility of founding on local areas of softer material which might be present close to the rock surface.

A letter of invitation was issued to five selected contractors giving foundation design loads and design criteria for piles/barrettes and explaining that, initially, tender documents based on a system of barrettes would be prepared, but that alternative barrette shapes or cast in situ piles would be considered.

Frankipile Australia Pty Ltd. were the successful tenderers with a proposal for forty-three 1.5m diameter cast in situ piles with various diameter belled bases up to 2.9m founded a minimum of 4m below the surface of the siltstone. The piles were to be constructed inside a temporary steel casing and drained to allow inspection of the bases.

The structural design of the piles was based on an ultimate (factored) load method using the SAA Concrete Code *(AS1480 – 1982)*.

Ultimate compressive loads varied from 18MN to 49MN.

Only 12 piles had tension loads, the greatest of which required 34 C36 bars, bundled in pairs.

A typical compression pile had 0.5% longitudinal reinforcement (9 C36 bars) and the concrete strength was either 30, 35 or 40MPa depending on the load. All concrete used 70/30 blend of OPC/blast furnace slag with a minimum cement content of 400kg/m^3, a water cement ratio of 0.6, slump of 180mm±30mm and 20mm maximum aggregate size. Retarders were used to ensure a minimum of 4 hours to initial set.

In half of the piles integrity testing was carried out using gamma ray back scattering equipment.

Four 50mm internal diameter steel pipes were tied to the inside of the reinforcement cage, equally spaced, and cast into the pile.

The test instrument consisted of a radioactive source and a detector of gamma radiation, assembled in a waterproof container, 600mm long and 48mm in diameter, connected to the surface electronics by a cable.

One pile on which the reinforcement cage had lifted during pouring was found to have suspect concrete. This was confirmed by coring and an additional pile was constructed.

Settlement of the piles is being monitored by precise levelling onto reference points on the structure immediately above each pile group using a bench mark remote from the site.

Three of the piles have been instrumented in an attempt to compare their performance against the design.

In each of the piles to be monitored, three strain gauges have been installed about 1.0m above the pile base and three strain gauges immediately under the pile cap.

The strain gauges are the acoustic type manufactured by Soil Instrument Ltd. and are connected to a vibrating wire readout unit.

It is hoped that it will be possible to compare the load in each pile with the design load and also the proportions carried by skin friction and end bearing.

Core walls: frame structure

Lateral forces and torsions are resisted by a combination of coupled core walls on the north and east facades acting with the column-beam frame across the hypotenuse and the beam-coupled internal low-rise lift shaft (which becomes blade columns above Level 17).

The core walls to the north and east facades comprise the medium and high rise lift shafts coupled to the escape stair walls and air-handling plantrooms which are situated on each floor. The inner and outer walls are 450mm thick from the Lower Basement to Level 15, reducing to 350mm to Level 31 then to 250mm to the top level. The coupling beams are 1150mm deep and the same width as the walls.

The central low rise lift shafts extend from Lower Basement to Level 17 with side walls 350mm thick. These are coupled to the stair and main lift shaft walls with 740mm deep beams by 350mm wide. Above this level, five intermediate-shaped columns support the floor beams and are coupled to the side cores by 740mm deep x 700mm wide beams.

The columns on the diagonal facade are spaced at 8.5m centres and are coupled with a beam 740mm deep by 1300mm wide. The columns are 1300mm square from Lower Basement to Ground, 1200mm square from Ground to Level 1, then reduce to 1140mm square from there to the top of the building. The column size is to retain both architectural requirements and to maintain stiffness, although a central 600mm diameter void is introduced at Level 33 to reduce the concrete volume.

The columns are heavily reinforced at the lower levels with up to 7.5% Grade 410 reinforcement. Bars up to 36mm diameter are used in bundles of four with staggered *G-loc*

splices used in lieu of laps up to Level 4 when reduction in the numbers of bars enabled conventional laps to be used.

65MPa (F'c cylinder strength) concrete was used in the columns up to Level 18 when the strength is progressively reduced to 50, 40, 30 then 25MPa above Level 36.

The core walls are 40MPa concrete at lower levels reducing to 30 then 25MPa at Level 10. The core walls are being constructed by slip form and can proceed up to five levels above floor construction, although this can be exceeded if the lobby portion of the floor slab is advanced not less than five floors below the top of the slip. To date, the builder has elected to pour the floors complete and maintain the slip for five floors.

Typical floors

The floor to floor height of typical floors is 3.6m with a floor to ceiling height of 2.7m.

Air-conditioning is provided by a variable volume air system with two fresh air handling plants on each floor providing flexibility for tenants. Cooling is provided by chilled water distributed to each floor from central chilling units located in a ground floor plantroom. Heating requirements are not significant and a small base load gas boiler is used and hot water circulated in the same piping system used for chilled water.

Electrical, telecom and computer cabling runs in three channel skirting ducts around the perimeter of floors, supplemented by 40mm deep floor ducts in limited selected locations.

Floors are finished monolithically.

Floor construction

The structural system is reinforced concrete beam and slab. There is generally a 130mm thick slab. Over the main floor section there are beams 600mm deep by 350mm at 4.243m centres (i.e. $3 \times \sqrt{2}$) spanning 10.5m. The beams reduce to 400mm depth by 600mm at the south west end to provide adequate space for air-conditioning ducts. The 400mm × 600mm section cantilevers out from the main 740mm × 1300mm framing beam to support the 'bay window'. The lobby slab is also 130mm thick with beams 550mm × 350mm and 400mm × 400mm.

The floors are designed for a general office loading of 3kPa with a 1kPa provision for lightweight partitions, ceiling loads and finishes. A 2.4m wide thickened slab section is provided on the south west side of the low rise lift core (or upper level columns) for a uniform heavy load of 7.5kPa or 12.5kPa concentrated over a 1.6m width.

The nominated concrete strength is 25MPa although the specified minimum OPC cement content of 280kg/m^3 generally achieves a minimum of 23MPa at seven days.

The builder is using a table form system for the floors and achieves a complete floor cycle of nine working days with two pours.

Levels 43 to 48 within the triangular pyramid section at the top of the building are executive suites and plantrooms, and are also beam and slab construction, but with the layout varying from the typical floors to accommodate the stepped facade and various voids and setbacks.

Credits:

Client:
Rural & Industries Bank of Western Australia and Austmark International Ltd.

Architect:
Cameron Chisholm & Nicol

Project managers:
Austmark International Ltd.

Project co-ordinator:
Bond Corporation

Main contractors:
Multiplex Construction Pty Ltd.

Arup engineers:
Dan Ryan, Adrian Roberts, Bill Haythornthwaite

Photos: Whitfield King

INTELSAT Headquarters Building, Washington DC

Peter Thompson

Architects:
John Andrews International
Anderson, Notter, Finegold

Introduction*

INTELSAT is an international organization owned by 110 member nations which provides international satellite communication facilities to most of the western world. The organization is based in Washington DC and in 1979 it was decided to hold a competition for the design of a new headquarters complex in that city.

Architectural institutions of member countries were invited to register the credentials of organizations who in their opinion were qualified to carry out the work. About 400 registrations were received which were reduced in three stages by selection panels to six, one each from West Germany, Norway, Canada, Australia and two from the United States. Each finalist was to be paid a fee on presentation of an entry fulfilling the competition requirements. A briefing/site visit for two persons was also included.

The competition, judged by an international jury, was won by the firm of John Andrews International of Sydney, who subsequently were appointed to further their design, prepare construction documents and supervise construction. We assisted John Andrews with his entry and were subsequently appointed as consulting structural engineers.

This article briefly describes the architectural planning and structural form of the complex, together with some of our experiences of working in the United States. Imperial units are used throughout.

Architectural planning

The site of 12 acres is on Connecticut Avenue between Tilden Street and Van Ness Street and forms the eastern boundary of a new international centre being developed to augment the diplomatic facilities of Washington. Immediately to the north lies the University of the District of Columbia. The site is heavily wooded on a sloping terrain and rises 40 ft above Connecticut Avenue at its highest point (Fig. 1).

The INTELSAT building programme was staged with about 600,000 ft^2 gross area being built in Stage 1 and a further 150,000 ft^2 in Stage 2.

Space requirements included accommodation for satellite control, operation, simulation and maintenance, convention facilities and office space all with their respective ancillary facilities contained within a mixture of separate offices and large flexible areas. Factors which were to be maximized included exposure of staff to sunlight, building efficiency and energy efficiency.

Planning restraints dictated that the buildings be accommodated on 8 of the 12 acres with the remainder designated as public parkland and the Washington building height limit of 392ft above sea level was to be observed.

Analysis of the client's brief indicated that areas requiring perimeter location accounted for 70% of the total. To maximize the exterior zone and minimize corridor space, modules of 90 ft square plan form were developed incorporating a 20 ft exterior zone. Columns were placed at the periphery and on the perimeter of the inner and outer zones to give maximum flexibility of partitioning.

The module was modified to an octagon to create rectangular connecting links containing duct risers and toilets and for connection to stair towers (Fig. 2).

Nine four-level modules satisfied the space requirements for Stage 1 and four six-level modules for Stage 2.

The module/link complexes are arranged in a chequerboard pattern and, when interlinked with a roof structure, form interior courtyards or atria. Pedestrian movement in the vicinity is concentrated on Connecticut Avenue, particularly at the Van Ness intersection where there are both subway and bus stops. Modules and courtyards are therefore arranged to create a pedestrian access spine stepping up the slope connecting the Connecticut/Van Ness intersection with the International Centre.

The spine of atria, apart from acting as the building's axial street, provide a source of interior daylighting and climate control. They also contain vertical and lateral circulation. Elevator towers encircled by a stair are located within the courtyards with access to modules via bridge links.

The courtyards are landscaped with water features and planting.

Below-grade parking for 500 cars and plantrooms are provided at three levels stepping up the hillside, linked by a feeder road off Van Ness Street.

Structure

Architectural requirements for column-free spaces within the modules, economy and speed of construction resulted in the choice of structural steel/concrete composite construction for office modules, service links and connecting bridges above grade. Reinforced concrete was used for construction below grade plus stair and elevator towers. The atria roofs are of tubular steel spaceframe construction. External concrete is fair-faced.

Building construction in Washington is carried out to the requirement of the Building Code of the District of Columbia. Reinforced concrete and structural steelwork design are carried out to the recommendations of the American Concrete Institute and American Institute of Steel Construction.

In addition all construction must conform to the requirements of the Fire Insurance Industry. This in effect means that unless a very expensive test is carried out, only a combination of structural elements which have previously been tested and given a fire resistance rating may be used. Results of tests are given in a document known as the Underwriters Laboratories Inc. Fire Resistance Directory.

Statutory loadings and permissible reductions are similar to Australian and British Codes. Wind loading is however extremely basic in comparison, the requirement being that buildings where the height exceeds 2½ times the width shall be designed to resist a uniform load of 20lb/ft^2. Roofs are generally designed for a snow loading of 30lb/ft^2. However, in areas where snow drift was considered to be a possibility a localized loading of 60lb/ft^2 was adopted.

Typical module structure

The typical module or pod floor is repeated a great number of times and much effort was put into creating an efficient and economic layout (Figs. 3 and 4).

A typical floor has 5¼ in slab spanning 8 ft 4 in onto a secondary beam system. The slab is a Robertsons *Q-Lock* metal pan with 2 in deep ribs acting compositely with 3¼ in of lightweight concrete. The lightweight concrete is used not only for load benefit but for its superior fire-resisting qualities.

The slab is supported by and acts compositely via shear studs with 12 in deep steel beams spanning 20 ft and 14 in deep beams spanning 25 ft. The secondary beams are arranged to ensure equal loading on the primary beams and thus equal loading on columns. The large range of steel sections available in the US lends itself to economic design.

The primary beams of 30 in depth and maximum span 50 ft are designed as partial composite sections and are continuous with the internal columns. These beams have either web penetrations or are notched at the ends connected to the perimeter columns, to allow passage of services. All beams are grade 36 ksi steel.

The four internal major columns are 19 in square sections of welded plate, grade 50 ksi, varying in thickness from 1⅛ in to ⅝ in depending on level. Fire rating is achieved by concrete encasement resulting in a circular column 30 in in diameter.

The eight external columns are 16 in diameter made up of a 12 in diameter tube, grade 46 ksi, of wall thickness ½ in, surrounded by 2 in of fire proofing contained by a thin steel tube. This whole assembly is a proprietary section in the US. The column is filled with concrete of 5,000lb/in^2 acting compositely with the structural tube.

Continuity between beams and internal columns is achieved by site butt welding. This procedure, which at first appeared to us to be a risky one, was the unanimous choice of the local steel fabricators whose opinion was sought. A full-scale mock up was constructed by the successful fabricator with all welds ultrasonically tested. Sample sections were taken from the mock-up and etch-tested for correlation with the ultrasonic test results.

The pod roofs are landscaped with paving and low planting and used for recreational purposes. Due to this the loading is higher than for a typical floor with a consequent increase in structural sizes. Roof members are arranged to provide a fall in all directions to the centre columns.

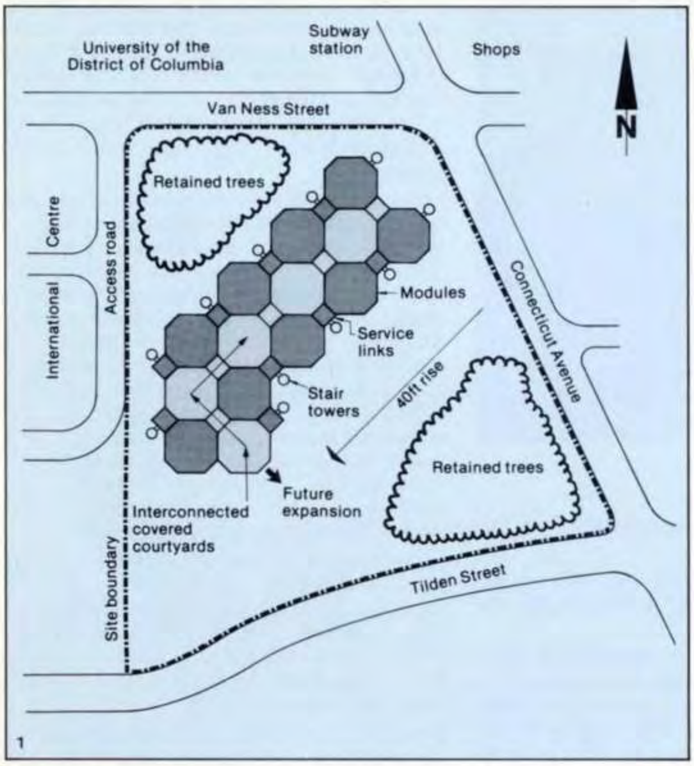
Each pod is separated from its neighbour not only by a link block but by movement joints. Temperature range is particularly severe on the eastern coast of the US and it is standard practice to provide joints much more often than we would in Australia. Consequently, each pod is designed as a two-way multi-storey portal frame providing its own stability against wind and out-of-balance loadings.

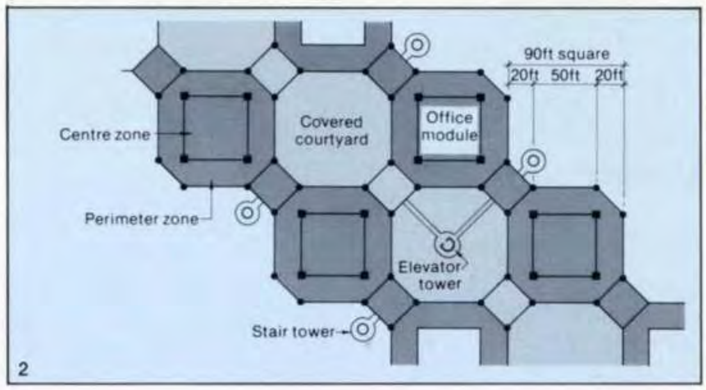
Bridge links between elevator towers and the pod floor links span a maximum of 50 ft and are designed as composite steel/concrete members with vibration criteria controlling member sizes. Expansion joints are provided at the end supported on the elevator tower.

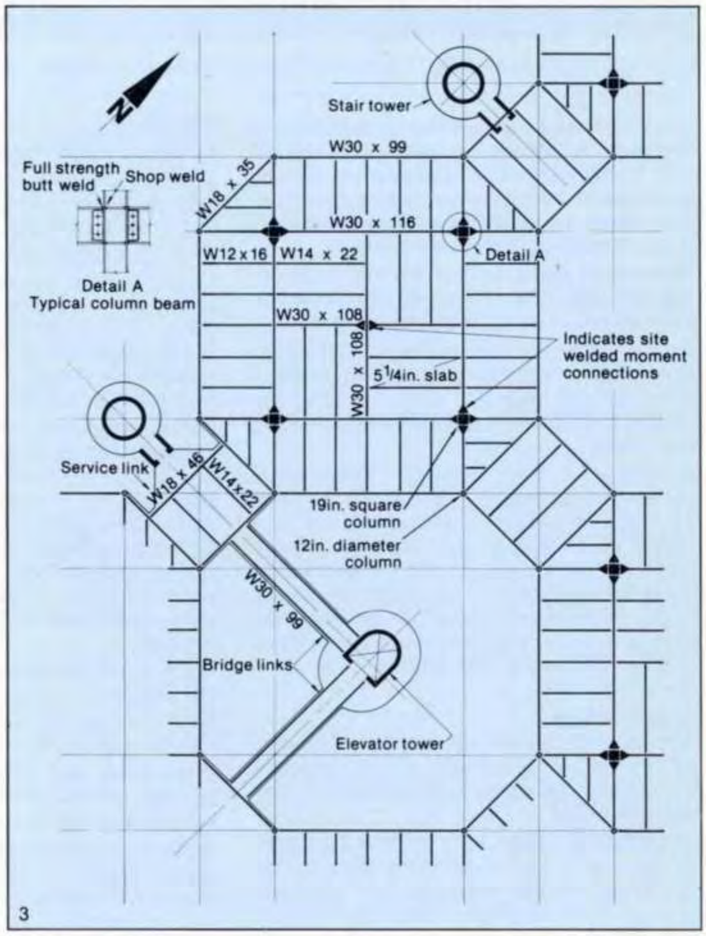
The towers, as with all exposed vertical surfaces are smooth fairfaced concrete.

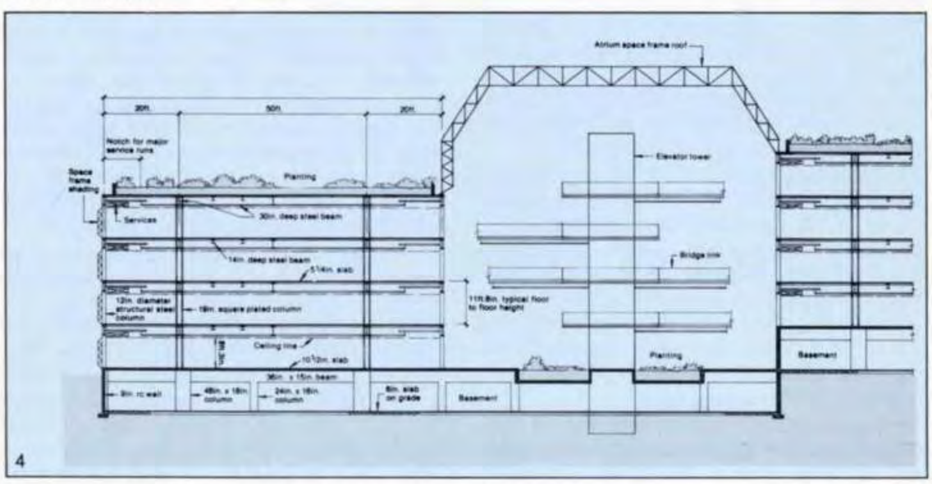
At the end of design development the client, without our knowledge, obtained a technical audit of the pod structure from Bethlehem Steel. This is offered as a free service in the US. Whilst not disagreeing directly with our design approach, Bethlehem were not too happy with the continuity in the floor system and put forward an alternative using simply-supported beams throughout, plus diagonal bracing for stability. We, in turn, were asked to comment on their proposals and after submitting a carefully worded adverse report, the matter was dropped.

*Editor's Note:
As this project was designed in imperial units we have not converted them into metric.









1. Site plan

2. Office module

3. Typical floor beam layout (W'30 × 99, and similar expressions mean: 'depth in inches × lbs per footrun')

4. Section through office module and covered courtyard

Atria roofs

Roofs to the covered courts are dome-like with a horizontal 50 ft × 50 ft central area and 50 ft × 20 ft sides sloping at 57°. Due to the difference in pod roof levels, each roof has a vertical component of a floor height on three sides.

A space frame based on a 5 ft module is used to support double glazing which forms the outer skin of the roof.

Apart from dead, wind and snow loads the space frame is required to support loads such as hanging planter boxes, window washing equipment plus a load of 150lb at the centre of each member for miscellaneous purposes. A temperature variation of 54°F is also taken into consideration.

Each roof connecting four pods acts independently and is supported at eight perimeter columns. Support releases have been provided such that only vertical load and wind forces are transmitted to the pod structures. A total of 25 separate loading combinations were considered in the design.

After preliminary design to prove the system viability, and to establish a budget cost, a performance specification was written to enable tenders to be called from manufacturers of proprietary space frame systems. Many systems are available in the US and the scope and size of the roofs excited much interest from suppliers. The system eventually adopted is known as *Spherobat,* a West German system manufactured under licence in the US. This is similar in appearance to the better known Mero system but utilizes split steel hemispheres bolted together to form coupling at nodes and steel tubes between node points.

Detail design calculations were prepared by the supplier and checked by us. Load testing of the node assembly was required prior to erection which proved to be a wise precaution as a significant amount of failures were recorded in nodes first supplied.

Substructure

The car park and plantroom structures stepping up the site act as podia to support the pods. The reinforced concrete beam and slab construction provides a transition between the pod steel columns and the parking requirements at basement level. Reinforced concrete columns are located to suit both the requirements of the pods and that of parking. 10½ in slabs spanning a maximum of 28 ft onto beams 36 in deep running normal to parking bays is the general form of construction. 48 in × 18 in reinforced concrete columns support the major steel columns above.

Foundations

Bedrock at the site is a gneiss belonging to the Wissahickson formation. This is a metamorphosed sedimentary rock generally moderately folded with bedrock elevation varying significantly over short horizontal distances.

Test borings carried out pre- and post-competition generally indicated disintegrated rock or rock about 12 ft below grade, these being weathered residual materials derived from the underlying parent bedrock. Both materials were suitable for founding of pad footings with an allowable bearing capacity of 5 tons/ft^2 for the disintegrated rock and 20 tons/ft^2 for the slightly weathered rock.

At the southern part of the site towards Tilden Street the depth to rock increased to

5. Connecticut Avenue elevation during construction (Photo: Bill Thomas)

some 35 ft and caissons were necessary in this area. A bearing capacity of 30 tons/ft^2 was recommended for end bearing with no allowance for skin friction, and caissons of 36 in diameter with belled ends were selected to support major pod columns.

Differential settlement of footings supporting the major pod columns founded in differing rock strata was allowed for in the design.

Groundwater was encountered at heights varying up to 5 ft above basement levels. A sub-drainage system was installed to reduce hydrostatic head on basement walls and floors.

In Stage 2 the building configuration developed somewhat differently to that originally anticipated and we found ourselves excavating a reasonably deep basement immediately adjacent to and below some of the Stage 1 caissons. Given the highly fractured nature of the rock we were concerned, but with prodigious amounts of ground anchoring the problem was overcome.

Energy design

Although not within our terms of reference as structural engineers, some mention should be made of the building's energy saving design.

The building has a utility bill less than 40% of the norm for comparable Washington buildings. These savings are achieved through a combination of passive and active systems.

The key to the passive system is the spine of linked atria. Air entering at the base of the service towers is drawn over planted terraces and cooling ponds then spray-washed and mixed with conditioned air before entering the atrium by way of interior pools.

At the building face the full height glazing is shielded by sunscreens mounted on a space frame of 1 in diameter stainless steel.

The active mechanical systems emphasize the recovery and reuse of waste heat. Surplus energy is stored in ice and warm water tanks. During hours of peak demand the building's standby plant augments utility supplied power.

Joint venture arrangements

The conditions of appointment required that each discipline be represented in Washington by local firms. After talking to various organizations we entered into an arrangement with MMP International who had started life as an offshoot of RMJM — well known in the UK. The work was shared. We provided all design functions throughout the project plus drawings to design development phase and general liaison with other consultants.

MMP advised during design on local conditions and mores, provided construction documentation, checked shop drawings and carried out site inspections. As designers we assumed overall responsibility for the structural sufficiency.

MMP also provide cost planning and evaluation services and were employed by the architect in this role.

Each individual State in the USA requires that structural documents be attested by a professional engineer registered in that State. It is possible in Washington DC to obtain dispensation for a specific project, particularly one of a diplomatic nature but, in our case MMP provided this service.

At the completion of design development for Stage 1, Bill Thomas took himself and all documents to Washington for six months to complete the design and supervise documentation in MMP's office. To make him feel at home and lighten his workload we installed in Washington similar desktop computer facilities to our own in Sydney. These were repatriated to Australia at the end of his stay.

For Stage 2 the process was repeated with Ian Ainsworth stopping over in Washington for three months, whilst en route for a two year stay in London office.

Throughout the design and construction phases additional visits were made from Australia at about six monthly intervals.

To date our relationship with MMP has been mutually beneficial and a fair share of credit for the successful outcome of the project is due to them for their efforts.

Construction

After a lengthy selection process one of North America's largest contracting organizations, Gilbane Building Co., was appointed to provide construction advice during design development and documentation. This company was eventually awarded a contract as construction manager for Stage 1 for a fixed lump sum price of $54 M with a construction period of 27 months.

The Stage 1 contract was completed on time and budget without any major problems and a grand opening attended by the Diplomatic Corps and delegates from INTELSAT member countries took place in April 1985.

Stage 2 commenced in the autumn of 1985 but for reasons known only to the client, a contract was negotiated with a much smaller contractor, W.P. Lipscomb for a fixed lump sum price of $26 M over a 24 month period. This contract is progressing satisfactorily.

The American system

We learned something from our brief encounter with the American system, particularly in the area of documentation. Although direct comparisons are difficult, it is clear that we tend to over-document our work and not take advantage of industry capabilities. In the US, bills of quantity are rarely necessary with a consequent reduction in documentation. In most instances builders are prepared to give a GMP — Guaranteed Maximum Price — for a contract based upon a preliminary set of layout drawings, schedule for finishes and a performance specification for services theoretically reducing the overall documentation/construction time and cost to the owner.

Shop drawings for structural steel and concrete reinforcement are the responsibility of the contractor with the consultant responsible for checking that his requirements have been incorporated within these documents — similar to the procedure used in Australia for steel shop drawings. The resulting reduction in drafting effort means that the average consulting engineer's office in the US has a higher ratio of engineers to draftsmen than is normal here, often as high as five to one. In addition a considerable amount of drawings, particularly details, is done on small calculation-size sheets by engineers.

Supervision of construction is handled differently in the US. During this phase of the work the consulting engineer performs a service termed 'occasional observation', checks shop drawings, assesses reports from inspection companies and remains available for consultation.

Detailed supervision of foundation works, concrete and reinforcement placement, and structural steelwork is carried out by specialist companies who combine inspection with laboratory testing and who work to a specification prepared by the consulting engineer.

All in all we feel the American consulting engineer, being more design-orientated, is more efficient. Given the prevailing fee-scales he has to be. The average fee payable to a structural engineer would be of the order of 0.5% – 0.75% of total cost with the total fee for architect and structural and services engineers in the range 5.5% – 6%. In a highly competitive situation this fee percentage can be forced down to the 4% – 5% level.

Epilogue

Our brief foray into the North American market came about in exceptional and unplanned circumstances. After our initial surge of euphoria following the competition success we had some apprehension as to how we would manage a project so far away but we were helped by the architect, John Andrews, who had practiced in North America for some years previously, and was familiar with the ground rules.

After a couple of visits we had rediscovered that engineering is a fairly universal language and the project has become one of our most successful in all respects.

The building has been well-received by the client and the architectural fraternity. It has won some awards and should serve as a useful marketing tool in the present Arup conquest of North America.

Credits

Client:
INTELSAT

Architects:
John Andrews International
Anderson, Notter, Finegold

Services engineers:
Don Thomas & Associates
Benham Blair Inc.

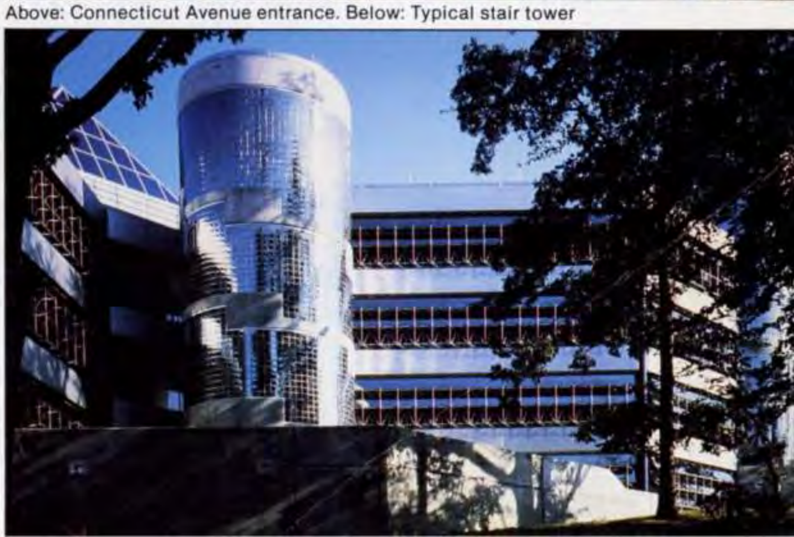
Structural engineers:
Ove Arup & Partners
MMP International

Cost planning:
D.R. Lawson & Associates
MMP International

Photos: courtesy INTELSAT

Above: Side elevation. Below: Elevator and walkways in atrium

Above: Connecticut Avenue entrance. Below: Typical stair tower



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